RIVER REINTRODUCTION INTO MAUREPAS SWAMP
ASSESSMENT OF AN ALTERNATIVE SIPHON SYSTEM INTAKE

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TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

APPENDICES

MISSISSIPPI RIVER REINTRODUCTION INTO MAUREPAS SWAMP
ASSESSMENT OF AN ALTERNATIVE SIPHON SYSTEM INTAKE

EXECUTIVE SUMMARY

ES.1 - INTRODUCTION

In July 2008, URS Corporation (URS) provided the Louisiana Office of Coastal Protection and Restoration (formerly DNR) with the 30% design plans for the "Mississippi River Reintroduction into Maurepas Swamp" (PO-29). Those plans included a gated culvert system consisting of a river intake structure with box culverts through the levee. Subsequently, OCPR requested that URS develop a conceptual cost estimate for an alternative approach using a siphon system with pipes going over the levee. The subject report is a composite of three separate Technical Memoranda (TM) that were compiled as the analyses of the siphoning options and the projected project construction and O&M costs were developed:

- TM-1: "River Reintroduction to Maurepas Swamp – Preliminary Hydraulic Assessment of an Alternative Vacuum Siphon System Intake", issued by URS in June 2010
- TM-2: "River Reintroduction to Maurepas Swamp – Preliminary Assessment of an Alternative Pumped Siphon System Intake", issued by URS in July, 2010
- TM-3: "O&M Cost Opinion for a Pumped Siphon System", issued by URS in September, 2010

Three types of river diversion systems are described in this report:

1) Gated Culvert – a purely gravity flow system consisting of an intake with sluice gates and reinforced concrete box culverts through the levee.

2) Vacuum Siphon – a partially gravity flow system consisting of piping over the levee and a series of vacuum pumps required to prime the siphoning operation. This system is often simply referred to as a Siphon.

3) Pumped Siphon – a system that can operate solely by pumping, or by gravity flow. It consists of piping over the levee and a series of pumps. The pumps alone can be used to pump the flow during low river stages or to prime the siphon operation during higher stages. Such a system may also have vacuum pumps to assist in the establishment and maintenance of the siphoning operation. This system is also referred to as a Pumped Diversion.

ES.2 - TM-1: VACUUM SIPHON ASSESSMENT

On February 15, 2010 URS provided OCPR a proposed scope of work, schedule and budget to conduct the initial siphon system analysis. OCPR subsequently authorized URS to perform the work and provided the design plans for the West Pointe a la Hache Vacuum Siphon system as a "Go-By" to assist in conducting the study. On June 3, 2010 URS presented TM-1 cited above to OCPR, the findings of which are summarized below.

URS performed a statistical analysis on historical river stages for the Mississippi River at the Reserve Gage (5 miles downstream from the Maurepas site), defining the river levels throughout the "average river year". The overall average river stage for the entire 57-year period analyzed is 9.0-ft. At that river stage, the gated intake structure, as presented in the 30% Design plans, can deliver 1,980 cfs discharge. At the same 9.0-ft river stage, an 8-pipe (72-in) Vacuum Siphon system can only deliver 1,275 cfs. However, the Vacuum Siphons cannot be primed at this river stage - they can only sustain flow once priming has been established.

Priming a siphon entails vacuuming enough air from a pipe to decrease the internal pressure below that of the atmosphere. The greatest water head that a practical vacuum system can provide is about 28-ft. Thus, a vacuum unit cannot establish a siphon over the 31.8-ft levee plus fill the 6-ft high pipe, which sits on a 1-ft thick foundation, until the river is at elevation 10.5-ft. Rating curves were developed and applied to the historical river stage data to project the Vacuum Siphon system operating schedule. An operating logic was also developed to determine when a siphon pipe should be brought on- or taken off-line, thus defining the required number of pipes for each day. To achieve a flow of 2,000 cfs through the Gated Culvert structure, the Mississippi River needs to be at river stage 9.12-ft. For delivery of 2,000 cfs through an 8-pipe Vacuum Siphon system, the TM-1 findings indicated that the river would have to be at stage 17.14-ft, a water surface elevation never reached during the average river year.

Thus, an 8-pipe (72-in diameter) Vacuum Siphon system cannot deliver the required 2,000 cfs at the Maurepas project site at any time during the average river year, since the highest stage is only 15-ft. The analysis showed that the Vacuum Siphon system would only be operable from mid-February through mid-July, or approximately 168 days during the year. Even during that period, the mean flow would be less than 1,450 cfs - significantly below that required, and its maximum discharge would only reach 1,900 cfs, for which it would operate just 8 days per year. Most importantly, the Vacuum Siphon system would deliver no flow at all during the fall months, which is the most critical period for the Maurepas Swamp, since it is the driest time of the year, when salinity concerns are the greatest.

TM-1 thus demonstrated that a Vacuum Siphon system will not meet the goals of the Maurepas project. Based on those findings, URS recommended discontinuing evaluation of the Vacuum Siphon system; instead, it recommended that a Pumped Siphon system be evaluated for comparison to the Gated Intake structure. OCPR concurred and instructed URS to analyze that system in sufficient detail to develop a conceptual construction cost estimate.

ES.3 - TM-2: PUMPED SIPHON ASSESSMENT

To assist in the assessment of a Pumped Siphon, OCPR provided URS a copy of the Pre-Decisional Draft - Volume IV of the "Integrated Feasibility Study and Supplemental Environmental Impact Statement" prepared jointly by the U.S Army Corps of Engineers, New Orleans District and the Louisiana Coastal Protection and Restoration Authority. This study, conducted for the LCA Small Diversion at Convent/Blind River in May 2010, compared a number of alternative approaches to construct a freshwater diversion project in the vicinity of Romeville, Louisiana. (Note that the Romeville location is at River Mile (RM) 162, which is only 18 miles upriver from the Maurepas site at RM 144.) The alternatives reviewed in that study included a Gated Culvert, a Vacuum Siphon, and a Pumped Siphon. The report presents the following conclusions on page 3-31:

> "**Alternative P-6 – Diversion at Romeville by a Gated Culvert System** (plus Inline Treatment.) This alternative … [has] … a gated culvert system through the levee instead of the siphon. The Gated Culvert system is more cost effective for larger flows and for operation during longer periods of low Mississippi River stage.
>
> The culvert system has several advantages over the [Vacuum] Siphon system. It is less [of an] operational concern since the adjustment of the flow is simplified by closing or opening a gate. The siphon system involves the use of electrical and mechanical vacuum pumps which must be used each time the siphon is restarted. The culverts would be able to operate over a wider operating river stage range. This would allow for longer periods of diversion when the River is low compared to the siphon, which must have a minimum river stage to operate."

On page 3-40, the report also presents **Figure 3-3: Comparison of Romeville structure options based on price and desired flow rate (cfs).** The figure compares the construction cost of the Gated Culvert and Vacuum Siphon systems versus design flow rates using October 2009 prices as the base cost year. The comparison is specifically for the Romeville location at Mississippi River Stage 11-ft, however, the general concept is applicable to the Maurepas project site as well. The graph shows that the construction costs for a 1,000 cfs diversion are approximately the same for the Gated Culvert and Vacuum Siphon systems. However, at 2,000 cfs the construction cost for the Vacuum Siphon system is about 34% more than that of a Gated Culvert system capable of delivering the same flow.

Since the target flowrate for the Maurepas diversion is 2,000 cfs, the findings of the LCA Small Diversion at Convent/Blind River study indicate that the Vacuum Siphon system would not be cost-effective. This data provides additional support to URS's recommendation to eliminate the Vacuum Siphon system from further consideration. Thus, the analysis was shifted to the comparison of a Pumped Siphon system to that of the Gated Culvert system presented in the 30% Design plans.

The second memorandum cited above, TM-2, therefore presented a conceptual design of a Pumped Siphon system capable of conveying 2,000 cfs over the levee into the Maurepas Swamp. It also described the operation of such a system based on historical river stages near the project site. An estimate of the probable construction cost of such a system, along with a comparison to that of the Gated Culvert system presented in the 30% Design drawings, was also presented.

The main elements of the Pumped Siphon configuration studied included: 1) a riverside forebay in the batture, 2) a sedimentation basin within the forebay, 3) a river-side pump station with eight reinforced concrete intake bays, 4) eight 72-in vertical pumps, 5) eight 72-in throttling valves, 6) eight 72-in steel pipes over the levee supported on a concrete slab, 7) a light-duty vehicular access bridge to reach the station from the levee crown, 8) a levee crown road over the pipes, 9) a pipe crossing underneath River Road, 10) a raised section of River Road, and 11) an outfall structure discharging into the conveyance channel.

A forebay style intake from the river was selected over a piped intake due to the reduction of localized velocities and its ability to control the entrance of sediment. The design basis for the forebay is to achieve a velocity less than 1 foot per second (ft/s) so that sands > 0.2 mm settle out. This would prevent the heavier suspended solids from reaching the pump intakes and their subsequent transport into the conveyance system.

The pump station will consist of eight pumps with a rectangular intake located on the batture. The pump bays would be oriented perpendicular to the river to channelize water flow toward the pumps. The station would be a concrete structure with eight wet wells design based on the Hydraulic Institute (HI) standards to minimize turbulence and vortexing. To achieve the requisite 2,000 cfs, eight vertical propeller pumps supplying 250 cfs each were selected for discharge through individual 72-in discharge pipes, yielding a flow velocity of 8.84 ft/s. The pump motors would be located on top of the main deck while the discharge piping would run below the deck. A control building would also be located on the main deck along with the electrical components, vacuum priming pumps, and actuated control valves. A utility substation would be located on the land side of the levee to transform the power from the utility voltage down to the pump station requirement of 4,160 Volts. The low flow condition at river stage 0.0-ft governs the submergence requirements, dictating a sump floor elevation of 16.5-ft. The intake channel to the pump station will have an invert of -7.0-ft. The total dimensions of the pump station would be 48.5-ft high, 142.5-ft wide, and 99.5-ft long.

Due to the significant variation in river stage over time, a single system curve does not represent the range of heads against which the pumps will operate. Therefore a series of curves was developed for various river stages. At the priming point of 1,585 cfs, the required TDH would be 12.0-ft; while at the 2,000 cfs operating point, the TDH would be 5.6-ft. Because the tail-water elevation in the channel is dependent upon the flowrate, fewer than three pumps cannot generate sufficient head loss in the channel to create any significant WSE at the upstream end. As the river stage rises and the differential head between the river and channel increases, the pumps reach excessive rotational speed before their curves intersect the system curve. Thus, as the river stage rises more pumps must be brought on-line to prevent damage to the pumps and motors, so the station controls must be automated to adjust the number of pumps operating based on river stage.

With all eight pumps operating, if the river rises above stage 9.12-ft, the total flow exceeds the capacity of the conveyance channel. To prevent the channel from overflowing, one or more pumps need to be shutoff. But, once a pump is turned off, siphon flow can not be re-established until the river stage rises to 11.50-ft; and at this stage, only approximately 1,550 cfs would be diverted via siphoning. The eight siphon system would not be able to divert the design 2,000 cfs by siphoning until the river reaches stage 16.11-ft. To address this gap in flow delivery, either the pump curves or the system curves would have to be adjusted.

To modify the discharge curve for a given pump, the impeller diameter, the speed of its rotation, or both can be changed, which would require a higher horsepower adjustable speed motor. An alternate way to address the gap in flow coverage is to adjust the system curves by the use of throttling valves. The valves could be partially closed for river stages above 9.12-ft to increase the head required for a given flow. This would maintain the system within the pump operating range until siphoning could be established at river stages above 16.11-ft.

The power required by a pump is a function of the flowrate and head as well as its efficiency. The efficiency of typical axial flow vertical pumps within this flow/head region is high, ranging from 86% to 90%. At the 2,000 cfs design point at the average river stage elevation of 9.12-ft, the required input power is 188 hp. For a river stage of 0.0-ft the power required to deliver the same flowrate over the levee and into the conveyance channel rises to 291 hp. To provide a measure of reserve capacity the selected pumps would be specified with 350 hp motors. The initial step during pump start-up would be to engage the vacuum assist units to lower the internal pressure within the lines. Once flow has been established, there are several potential system operating regimes:

1) Pump Flow: River stage <9.12-ft, - The vacuum units assist to prime the pipes, the pumps run, the throttling valves are fully opened, and the flow is a function of the river stage.

2) Siphon Flow: River stage >16.11-ft - The vacuum units both prime the pipes and maintain siphoning. Individual pipes may be taken off-line to limit flows in the conveyance channel.

3) Throttled Flow: River stages 9.12-ft - 11.50-ft - The vacuum units help prime the pipes, the pumps run to sustain flow, and the throttling valves are used to limit the discharge flowrate to 2,000 cfs.

4) Mixed Flow: River stages 11.50-ft - 16.11-ft - The vacuum units prime the system, which can deliver 1500 to 1750 cfs by siphoning alone; the pumps are required to deliver the full 2,000 cfs.

To support the pumping operations, a sophisticated control system is required to operate the vacuum assist units, turn the pumps on and off, add or remove pipes from service, and control the throttling valves.

Two options were considered for the discharge piping to cross River Road (LA Highway 144): 1) an aerial crossing, and 2) a subsurface crossing. An aerial crossing would require a piled supported foundation, which is prohibited because the piles would have to be driven through the Stability Control Line (SCL) of the extended levee section. For the subsurface option, there is not enough clearance to beneath the existing roadway and above the SCL. Since shifting the roadway is not an option due to adjacent residence, the road must be elevated. To obtain the needed clearance and meet the LDOTD standards, the total length of roadway to be raised would be 1150-ft.

The outlet structure would be an open-top, concrete box, located at the head of the conveyance channel. The back wall would be a weir outfall into the conveyance channel. The top of the weir would be below the design water surface elevation in the channel, and the exiting pipes would have down-turned elbows at an elevation lower than the weirs to maintain a submerged condition.

The construction costs were separated into the following major elements: 1) Raising River Road, 2) Raising the levee to the Design Section elevation, 3) Installing the discharge piping over the levee, 4) Constructing the forebay and sedimentation basin, 5) Construction of a light-duty vehicular bridge, 6) Construction of the pump station, 7) Construction of the discharge structure. The most expensive cost items are constructing the Pump Station at $26,860,000 and raising of River Road at $3,980,000. The total probable construction cost of the Pumped Siphon was estimated as $35,346,982.

To compare the cost of the Gated Culvert to that of a Pumped Siphon, the common elements in the overall project, such as the conveyance channel, US 61 crossing, two railroad crossings, Hope Canal pump station, etc. are presumed to cost the same to construct under either alternative. Excluding the common elements the estimated construction cost of the Gated Culvert installation is $27,208,228. Thus, the differential cost of installing the Pumped Siphon system over the Gated Culvert is $8,138,754 (or 30%) more, which is roughly the same percentage differential as that presented in the Romeville analysis.

ES.4 - TM-3: OPERATING & MAINTENANCE COSTS

Based on the findings in TM-1 and TM-2, showing the additional capital cost of the Pumped Siphon system, URS recommended that the Gated Culvert system as presented in the 30% Design plan be pursued as the selected option for the Maurepas diversion. Upon review of the second memorandum, OCPR accepted the URS findings, but requested that URS prepare an estimate of the probable operating and maintenance (O&M) costs over a twenty year span for the Pumped Siphon alternative to supplement the capital cost estimate prepared for the TM-2 report. On August 4, 2010 URS provided OCPR a fee proposal, scope of work, and schedule to conduct the additional work quantifying the O&M costs, which OCPR subsequently authorized. The findings of that analysis are briefly summarized below.

The O&M costs presented in TM-3 were divided into three categories: 1) utilities costs, 2) labor costs, and 3) equipment costs. The following sections describe the approach taken to quantify each of these costs along with the overall O&M Cost Opinion for operation of the conceptual Pumped Siphon system.

To estimate the annual utilities costs, several steps were taken:

- The average run-time of the pumps was estimated.
- The average operational time of the vacuum units was estimated.
- The approximate kW-hrs required to operate the pumps, vacuums, and ancillary components were computed.

- Local utility cost data was applied to the usage numbers to derive an annual electrical supply cost.
- The water usage requirements to run the vacuum system were approximated.
- The water supply cost was calculated based on the local water rates.
- The average annual electricity and water supply costs were added.

Derivation of the labor costs included estimating both the cost to operate and to maintain the station. The approach taken included the following five steps:

- The average yearly hours required to operate the subject facility was estimated.
- The labor rates of the operating and service personnel were obtained from other facilities.
- The annual preventive maintenance hours required for the station components was estimated.
- The routine housekeeping hours, such as trash rack cleaning, were estimated.
- The total labor costs were summed.

To determine the cost of replacing the major mechanical and electrical components with less than a 20-year service life the following steps were taken:

- The useful service life of the pumps, motors, vacuum units, controls, etc. was estimated.
- The removal and reinstallation cost for the major components was estimated.
- The average annual cost was computed over the 20-year period.

The total probable Operating and Maintenance cost over the 20-year analysis period, in 2010 dollars is approximately $24.5 million. The annual costs are predominantly comprised of the electrical power costs at approximately $11 million and the equipment replacement cost of roughly $12 million. The cost for labor accounts for about 5% of the total, while the water supply cost is inconsequential.

ES.5 - CONCLUSIONS

The three Technical Memoranda describe the operational issues associated with the various siphon systems and quantify the significant increase in the probable construction and O&M costs for the Pumped Siphon in particular. Based on those findings, URS recommends that the original design of the Gated Culvert system as presented in the 30% Design be pursued.

MISSISSIPPI RIVER REINTRODUCTION INTO MAUREPAS SWAMP
ASSESSMENT OF AN ALTERNATIVE SIPHON SYSTEM INTAKE

1. INTRODUCTION

The Louisiana Office of Coastal Protection and Restoration (OCPR) requested that URS Corporation (URS) perform an alternatives analysis to develop a conceptual cost estimate for a vacuum siphon system to compare to that of the proposed gated intake structure for the Maurepas Diversion.

In July 2008, URS provided OCPR (formerly DNR) with the 30% Design plans for the "Mississippi River Reintroduction into Maurepas Swamp" (PO-29). In the accompanying design report, under the same title, URS presented the results of a HEC-RAS model of the diversion channel with a gated intake structure. The model included: 1) a 250-ft long intake channel, 2) three 10-ft x 10-ft sluice gate structures at the toe of the levee, and 3) three 10-ft x 10-ft reinforced concrete box culverts, approximately 300-ft in length, through the levee. At the design flow of 2,000 cubic feet per second (cfs), the velocity through the gates and culverts was calculated to be 6.67 feet per second (fps). Figure 1 presents the hydraulic rating curve, or water surface elevation (WSE) versus the diversion discharge (Q), for the box culvert intake structure and the downstream diversion channel, as developed from the model. To achieve a Q of 2,000 cfs through the gated intake structure, the Mississippi River needs to be at river stage 9.12-ft, which corresponds with a WSE in the diversion channel of 7.53-ft.

On April 22, 2010 URS presented a draft Technical Memorandum to OCPR titled "Preliminary Hydraulic Assessment of an Alternative Siphon Intake Structure". The memo was based on information from the West Ponte a La Hache vacuum siphon system, provided to URS by OCPR as a "Go-By" for the subject analysis. URS developed preliminary hydraulic rating curves for a similar siphon structure at the Maurepas Project location and compared them to those of the proposed gated intake structure. The preliminary findings indicated that the river would have to be at stage 17.14-ft for delivery of 2,000 cfs through an 8 pipe siphon system. This is substantially higher than the 9.12-ft stage required for the gated structure to deliver the same flow-rate. URS also estimated that 20 siphon pipes would be required to achieve the requisite 2,000 cfs through the diversion channel at river stage 9.12-ft. This preliminary assessment did not consider vacuum operational requirements or the historical river stage data.

This current memorandum addresses the operability of a vacuum siphon system based on historical Mississippi River stages near the project site. The hydraulic rating curves for the siphon intake structures have also been adjusted to account for operational limitations. The analysis has demonstrated that the historical stages of the Mississippi River are not regularly high enough to operate a siphon system at the project location for more than half of an average year.

2. ANALYSIS OF MISSISSIPPI RIVER HISTORICAL STAGES

URS performed a statistical analysis on historical river stages for the Mississippi River at the Reserve Gage. The Reserve Gage is located on the Globalplex Terminal Pier owned and operated by the Port of South Louisiana. This gage location is just 5 miles downstream from the Maurepas site, and thus the stage data collected there are very representative of the subject project conditions.

Daily Mississippi River stages, comprising 20,611 data points and representing 98% of the period, were collected from January 1953 through May of 2010. The minimum and maximum river stages over the entire period were 0.2-ft and 24.5-ft, respectively. Figure 2 is a plot of the historical river stages versus time over the entire 57 year period. While the data varies significantly from year to year, the plot does show that the river exhibits seasonal behavior. The high water stages generally occur between January and June, whereas the river is consistently low from August through early November.

Figure 3 presents the results of a statistical analysis of each day of the year using all of the available data. As the figure shows, the seasonal nature of the river stages is evident in the statistical stage curves. The heavy line in the middle of the curves represents the average stage of the river throughout the year. Also presented are the 5th, 10th, 25th, 50th, 75th, 90th and 99th percentile stage curves, the minimum and maximum recorded stages, and the Standard Error curves, which represent three standard deviations (encompassing 99.7% of the data points) above and below the average river stage curve.

Several significant observations can be made from Figure 3. First, the average river stage is slightly above the 50th percentile, particularly during the low water season (July 1 to Nov 30). This means that there are more occurrences of river stages below average than there are above average (the average being skewed by significantly higher stages in a few years). Thus, during the low water season, there will be more years with river stages below average than with stages above average. Second, the various statistical stage curves form a tight formation during the low water season, indicating that the river is consistently low at this time of year. Therefore, during the dry season, even during flood events or years where the river stage is above average, the stages do not vary far above average in magnitude or remain there for a prolonged period of time. These are two key observations, because it is vitally important to divert the fresh river water into the swamp during the dry season, since that is when the vegetation is the most stressed.

Third, during the high water season (Jan 1 to June 30), the river stage can vary from 0.2-ft, an extremely low level, to 22.0-ft, which is above flood stage. This high degree of variation is due to two factors: 1) the magnitude of the high river stages vary greatly from year to year, and 2) the seasonal high river stages migrate over the January to June period. This indicates that the seasonally high river stage for any given year can be well below average and that the high river period can be relatively short.

Overall, the average Mississippi River stage for the entire period is 9.0-ft. At that river stage, the gated intake structure, as presented in the 30% Design plans, can deliver 1,980 cfs discharge. An 8 pipe siphon system, as presented in the April 22 Technical Memorandum can only deliver

1,275 cfs; however, as shown later in this memorandum, the siphon system cannot be primed at this river elevation, it can only sustain flow once priming has been established.

3. EVALUATION OF A VACUUM SIPHON SYSTEM

3.1 DEVELOPMENT OF THE SIPHON SYSTEM HYDRAULIC RATING CURVES

The April 2010 Technical Memorandum presented preliminary hydraulic rating curves for both an 8- and a 20-siphon system. Those rating curves were based on several assumptions that have been further refined for this study. The previous curves did not account for the priming requirements of a vacuum siphon system. Additionally, the previous rating curves did not account for being unable to maintain a primed siphon when flow velocities fall below 5 fps. Finally, the K coefficient in the preliminary assessment was adjusted downward based on refinement of the siphon pipe plan and profile information.

Figure 4 plots the hydraulic rating curves for a vacuum siphon system located at the Maurepas intake site with 4 through 14 pipes in operation.

The development of Figure 4 is described in Appendix A. The discharge flows provided in Figure 4 are calculated from an iterative solution of the following equation:

$$Q = n \times A_{Pipe} \times \sqrt{\frac{(Z_{River} - Z_{Channel}) \times 2g}{1+k}}$$

Equation 1: This equation is used to solve for the flow through one siphon pipe. Note that $Z_{Channel}$ is a function of Q and therefore the solution to the equation is derived by iteration.

where,

- Q = total discharge in cfs
- n = number of siphon pipes in use,
- A_{Pipe} = area of a single 6-ft diameter pipe,
- Z_{River} = WSE in the Mississippi River,
- $Z_{Channel}$ = WSE in the discharge channel, which is a function of Q (see Figure 1),
- g = gravitational acceleration constant, 32.2 ft/s^2
- k = loss coefficient for the siphon system, taken as 6.07, as developed in Appendix B.

This equation is a simplified version of the Navier-Stokes equations, which combine the Continuity equation with the modified Energy equation as presented in Appendix A. A tabular form of sample data is also provided in Appendix A.

The rating curves in Figure 4 are divided into three regions, velocities above 7 ft/s, velocities below 5 ft/s, and velocities between 5 and 7 ft/s. To prime a siphon pipe, a velocity of 7 ft/s or higher must be achieved to flush all air out of the pipe so that it can flow full. Once a pipe is primed and operating, the siphon can be maintained at velocities as low as 5 ft/s (with the use of a supplemental vacuum system to remove entrained air that out-gasses from the water column and collects at the top of the pipe). However, if the velocity falls below 5 ft/s, a siphon can no longer be maintained. When declining velocities approach the limit of 5 ft/s, one or more pipes must be taken off-line to lower the WSE in the diversion channel. This will increase the differential head between the river and the channel, allowing the velocities in the remaining pipes to increase. If sufficient pipes are not taken out of service, the consequence will be the loss of prime in all of the siphons.

3.2 SIPHON SYSTEM PRIMING REQUIREMENTS

Priming a siphon system entails using a vacuum pump to evacuate enough air from the pipe to decrease the internal pressure below that of the atmosphere. Once the pressure is reduced, the outside atmospheric pressure pushes water out of the river and into the pipe to fill the void left by the vacuum. With sufficient vacuum, the water rises to the high point in the pipe, and begins to flow down the other side into the diversion channel. Continued evacuation of air from the pipe enables the water to eventually fill the pipe. Once the pipe is flowing full (at approximately 7 ft/s), the water will flow by siphoning at the rates illustrated in Figure 4, without requiring additional energy input from the vacuum pump. However, as discussed above, the velocities must remain above 5 ft/s to ensure adequate purging of entrained air, which does require periodic use of a vacuum pump (which may be considerably smaller than the priming pump), otherwise the vacuum will be lost and the siphon will cease to flow.

Atmospheric pressure is approximately 14.65 lbs per square inch (psi) or about 2,110 lbs per square foot (psf). Since the density of water is around 62.4 lbs per cubic foot, atmospheric pressure can push a water column 33.8 feet high ($2110 \text{ lb/ft}^2 \div 62.4 \text{ lb/ft}^3 = 33.8$ ft). Thus, 1 atmosphere represents 33.8-ft of water head. Industrial vacuum pumps cannot achieve anywhere near a perfect vacuum under field conditions; in fact, they can only lower the internal pressure within a pipe to approximately 12.3 psi, or 1,768 psf. Thus, without considering system head-losses, the greatest water head that a practical vacuum system can provide is about 28-ft ($1768 \text{ lb/ft}^2 \div 62.4 \text{ lb/ft}^3 = 28$ ft). In other words, the Mississippi River stage must be within 28-ft of the top of the discharge pipe for a vacuum to fill the pipe. The top of a 6-ft diameter discharge pipe over the levee at the proposed Maurepas Diversion location is estimated to be at elevation 38.5-ft. Thus, a vacuum pump cannot establish a siphon until the river is at elevation 10.5-ft (38.5 ft – 10.5 ft = 28 ft). This elevation is highlighted in Figure 4 with a bold red horizontal bar. The figure also shows that if the siphon is initiated at river stage 10.5-ft, only 6 siphon pipes can be primed. This occurs because the flow from an additional pipe would raise the WSE in the discharge channel, which would reduce the differential head between the river and the channel, which in-turn would reduce the velocity in the pipes to less than the 5 ft/s criterion. In fact, as shown on Figure 4, a 7th siphon pipe could not be brought on-line until the water in the river rises to a stage of nearly 11.0-ft.

3.3 SIPHON SYSTEM HEAD LOSSES

In the April 2010 memorandum, the siphon system loss coefficient was estimated to be 6.97. Since issuing that memorandum, URS has refined the plan and profile of the alternate siphon piping arrangement. The head loss calculations have also been updated to reflect the revised configuration and are provided in Appendix B. The revised loss coefficient is 6.07. This refinement will result in an increased flow per siphon pipe as compared to the April 2010 report.

The reduction of the loss coefficient is primarily due to shorter pipe lengths than originally expected.

Another feature of the siphon pipes incorporated into the refined analysis is to use 10-ft diameter intake and discharge openings at each end of the pipe. These enlarged ends will significantly reduce the entrance and exit losses due to the lowering of the water velocity entering and exiting the pipe. (Note; however, that these losses do not comprise the major components of the total head-loss through the system.) As a practical matter, the reduced velocities would also result in less vortex formation at the siphon intake in the river and reduced turbulence and scour at the siphon discharge into the diversion channel.

3.4 OPERABILITY OF A VACUUM SIPHON SYSTEM

To project what the alternate siphon system operating schedule would look like, the rating curves from Figure 4 were applied to the historical average river stage data provided in Figure 3. The operating logic diagram shown in Figure 5 was developed to determine when a siphon pipe could be brought on-line and when one must be taken off-line. This logic algorithm was programmed into an Excel spreadsheet and applied at each daily average river stage. Using the program, the number of pipes for each day (N_{Pipes}) was determined, simulating bringing additional pipes on-line when the conditions of the logic system dictated and taking pipes off-line when the opposite trigger points were reached. As shown by Figure 4, during a typical river year with a stage of 10.5-ft, only 10 siphon pipes can operate. This occurs because achieving a velocity of 7 ft/s in the 11th pipe would require generating more flow than the 2,000 cfs capacity of the conveyance channel.

Figure 6 depicts the projected flow for the Vacuum Siphon system in comparison to that of a Gated Culvert system based on an average river year; the chart shows:

- Average river stage elevations from Figure 3
- Projected flows through the preliminary gated intake structure as per the rating curves presented in Figure 1
- Projected flows through the proposed alternate 8 siphon system intake
- Projected flows through an expanded alternate 10 siphon system intake
- Number of siphons in operation for each day

Several notable observations can be made from Figure 6:

1. The gated intake structure is operable 365 days of an average river year and provides 2,000 cfs from January 1 through July 1.
2. The alternate vacuum siphon system is only operable from February 7 through July 25 for an average river year (168 days).
3. The originally proposed 8 siphon system peaks at 1,914 cfs and it only exceeds 1,900 cfs for just 8 days.
4. The expanded 10 siphon system provides flows exceeding 1,900 cfs for 90 days.

5. The two additional siphon pipes provide, on average, an additional 100 cfs throughout the siphon system operating season. This includes an additional 155 cfs on average through March and an additional 185 cfs on average through May and June.
6. With sufficient river stage during the high water season, the siphon requires a minimum flow of 1,200 cfs for priming and requires 700 cfs in a falling river condition. The siphon will not operate at flowrates below these.
7. The gated intake structure does not have a minimum operating flow rate requirement and can sustain 2,000 cfs for over half the year. Also, it can still convey at least 460 cfs for even the lowest river stage during the average river year.

Figure 7 illustrates the percentage of the year that the gated intake structure and the 8 and 10 siphon systems operate at various flowrates (Note: 1% = 3.65 days). As shown, the gated intake structure flows at 2,000 cfs for 53% of the year. Neither of the siphon systems can flow at the target 2,000 cfs. The gated intake structure is operational above 250 cfs year-round (100%). Conversely, the siphon systems are not operational for over one-half (54%) of the year. Table 1 summarizes the operating durations in days for the various systems during the average river year.

Table 1 – Annual Operating Durations for the Average River Year

Discharge Rate	Gated Intake Structure	Siphon System 8 Pipes	Siphon System 10 Pipes
2,000 cfs or Greater	193 days	N/A	N/A
1,750 cfs or Greater	215 days	80 days	117 days
1,500 cfs or Greater	234 days	124 days	142 days
1,250 cfs or Greater	252 days	150 days	157 days
1,000 cfs or Greater	277 days	161 days	164 days
750 cfs or Greater	325 days	164 days	168 days
500 cfs or Greater	361 days	168 days	172 days
250 cfs or Greater	365 days	168 days	172 days
Not Operational	N/A	197 days	193 days

3.5 SIZING THE VACUUM SIPHON SYSTEM

OCPR provided the West Pointe a la Hache siphon intake structure as a "Go-By" for this preliminary analysis. That siphon structure consists of eight 6-ft diameter pipes. As illustrated by Figures 4, 6 and 8, an 8 siphon system cannot deliver the required 2,000 cfs at the Maurepas project site for the average river year. Further, as seen in Figures 4 and 6, the maximum number of siphon pipes that can be used at this location is 10. Figure 8 illustrates the number of days each siphon pipe can be used for an average river year. As shown, the siphon system can operate for 169 days; however, all eight pipes will only be operable for 149 days. A 9th siphon pipe can operate 96 days, while a 10th would only be operable 40 days of the year. The 9th and 10th pipes provide a 10% increase of the total flow, for their limited operation period. These two additional pipes can provide significant additional flow during non-average river years. Such conditions include a rising river stalled for a long time between elevations 12 – 14, and for a prolonged falling river between elevations 14 - 8, as shown in Figure 6. These special conditions have occurred 1,026 days in the recorded data set since 1953, representing only 5% of the period. Thus, the cost-effectiveness of the two additional pipes is questionable.

3.6 REVISED SIPHON SYSTEM RATING CURVE

The hydraulic rating curve provided in the conclusion of the April 2010 memorandum did not account for several factors discussed in this report. Subsequently, URS developed the revised rating curve presented in Figure 9. The saw-tooth curve uses the rating curves provided in Figure 4, emphasizes the goal of maximizing the flow (up to 2,000 cfs), and accounts for the vacuum siphon system's operational limitations. Four significant observations from the chart are:

1. The gated intake structure can provide 2,000 cfs at significantly lower river stages than the siphon systems. It can also readily and incrementally restrict the flow to a maximum of 2,000 cfs at high river stages, thus maintaining a steady flowrate.
2. The siphon systems cannot operate on a rising river until a river stage of 10.5-ft is reached due to vacuum priming limitations.
3. The siphons cannot maintain a consistent flow at high river stages because entire pipes (representing 1/10th to 1/8th of the system flow) have to be taken off-line or returned on-line to stay between the 5 to 7 ft/s velocity criterion.
4. The siphon systems can provide some extended flow service in a falling river since they can continue to operate at river stages below 10.5-ft. However, the siphon pipes cannot be re-primed if the siphon is lost until the river returns to the 10.5-ft stage.

3.7 SIPHON SYSTEM OPERATING CONTROLS

The operation of a vacuum siphon system requires a sophisticated control system, as discussed in the following reports:

- Equipment and Operation Recommendations, Freshwater Diversion Siphons, Plaquemines Parish, July 2003, Perrin & Carter
- Meeting Report, BA-4c West Pointe a la Hache Project Team Coordination Meeting, February 29, 2008, NRCS
- 2005 Operations, Maintenance and Monitoring Report, West Pointe a la Hache Siphon Construction, June 2005, LADNR

- Progress Report 3, West Pointe a la Hache Freshwater Diversion BA-04, May 2000, LADNR
- West Pointe a la Hache Freshwater Diversion (BA-04), State Funded, September 2002 (source not provided).
- CWPPRA Adaptive Management Review Final Report, July 2002, CWPPRA Planning and Evaluation Subcommittee, Technical Committee and Task Force.
- Three-Year Comprehensive Monitoring Report, West Pointe a la Hache Freshwater Diversion BA-04, April 1998, LADNR.

These reports repeatedly mention the loss of prime and therefore flow as a constant problem. An example of the control system logic required along with the various set points is provided in Appendix C. A master control unit will be required to determine whether a falling or rising river stage condition exists and then execute the logic algorithm based on the set points. Program interrupts must be provided to monitor WSE's in the diversion channel so that it does not overflow the guide levees. Programming routines would also have to be developed to handle the multiple valve operations required for priming, maintaining, opening, and closing each of the pipes. These routines must control the valve operations in the proper order and then verify that they have completed. The verification is critical so that the priming of the entire siphon system is not lost due to a stuck solenoid valve, for example. Further, stand-alone duplex controllers will be required to operate the priming vacuum and maintenance vacuum pump sets.

While this memorandum is focused on the hydraulics of a vacuum siphon system, the complexity of the controls package will contribute significantly to the overall cost of such a system. This may make a vacuum siphon system uncompetitive on the basis of cost when compared to either a pumped siphon or a gated intake structure.

3.8 OVERALL ASSESSMENT OF VACUUM SIPHON SYSTEM

This Technical Memorandum demonstrates that a vacuum siphon system will not meet the goals of the Maurepas freshwater reintroduction project. The alternative would provide only 50% of the water that a gated intake structure would. The siphon structure is not operable at river elevations below 10.5-ft and thus can not deliver freshwater to the Maurepas Swamp during the crucial dry period of the year. During those times when the siphon system is operable, it delivers less water than the gated intake structure, even at very high river stages because the only method for restricting flows to less than 2,000 cfs is by taking siphon pipes off-line.

From a construction standpoint, the vacuum siphon system eliminates the need to excavate the levee or build a large cofferdam, which would be required for installation of the culverts of the gated structure. On the other hand, the siphon system will require a substantial amount of piping for the discharge pipes, the vacuum delivery network, and connection to the potable water system for the vacuum pumps. From an operational standpoint, the vacuum system will require substantial amounts of potable water and electricity to operate. Further, the control system for the siphon alternative will be complex and difficult to calibrate to continuing changing field conditions.

Based on these findings, URS recommends not continuing the conceptual design on a vacuum siphon system to the level required to develop a construction cost estimate. Instead, it is recommended that a pumped siphon system be evaluated for comparison to the gated intake structure.

4. EVALUATION OF A PUMPED SIPHON SYSTEM

4.1 OVERALL DESIGN DESCRIPTION

The main elements of the pumped siphon configuration are illustrated in Figure 10 and are listed below:

- A riverside forebay in the batture between the river and the levee
- A sheet-pile walled sedimentation basin within the forebay
- A river-side pump station with eight individual reinforced concrete intake bays
- Eight 72-in vertical propeller pumps
- Eight 72-in diameter throttling valves (not shown in Figure 10)
- A light-duty vehicular access bridge to reach the station from the crown of the levee
- Eight 6-ft diameter steel discharge pipes over the levee on a concrete slab with supporting pipe saddles
- A levee crown road crossing over the pipes with T-wall retaining structures and guardrails
- A discharge pipe crossing underneath River Road
- A raised section of River Road with T-wall retaining structures
- An outfall for the piping with an overflow weir for discharge into the conveyance channel

Each of these components will be discussed in detail in the later sections.

4.2 RIVERSIDE SITE SELECTION

The decision to locate the pump station on the riverside for the alternative analysis was made to eliminate the need for penetrating the levee to bring the river water into the station. A landside site would have required the piping to run through the levee to allow for gravity inflow of water. On the other hand, the batture at the selected diversion corridor is relatively narrow, which minimizes the allowable forebay size and sediment removal capabilities. Access to the station on the riverside also requires the construction of a light-duty vehicular bridge from the levee to the station for routine servicing of the pumps. However, the integrity of the existing levee and riverbank is a significant factor, and therefore the advantage goes to a riverside location.

4.3 FOREBAY RIVER INTAKE

A forebay style intake from the river was selected over a piped intake due to two primary advantages: 1) the reduction of localized velocities, and 2) its ability to control the entrance of sediment into the pumped flow-stream. With a piped intake, the water velocities near the suction of each pipe would be very high, presenting a potential problem for some species of aquatic life in the river. A piped intake would also transport all sediment entrained in the river water column directly through the pumps and subsequently into the downstream conveyance channel. A forebay intake of sufficient cross-sectional area would significantly reduce the approach velocity into the system, thus enabling marine life to escape from the influent current. A forebay intake of sufficient surface area would enable a majority of the heavier sediment particles, particularly sand, to settle out prior to being pumped, thus reducing the buildup of unwanted solids in the downstream channel and swamp. Collecting the sediment on the river-side of the system would simplify its removal because it could be periodically dredged instead of having to be excavated, loaded, and transported by truck if it were collected in a sedimentation basin on the land-side. Selection of the forebay style also eliminates the construction of intake pipes and protective dolphins out in the main river channel, thus preventing the risk of damage from maritime traffic. A log boom at the entrance of a forebay may be an additional design feature required to prevent floating debris from accumulating in the slower moving water areas.

4.4 RIVERSIDE SEDIMENTATION BASIN

The use of a sedimentation basin has been an integral part of the design concept for the Maurepas diversion since the beginning of the project. During the previous design stages, a sediment basin, where the velocity of the diverted flow would be low enough to settle out the sand particles, was planned at the upper end of the conveyance channel, just downstream of the River Road crossing. As shown in Figure11, the subject pumped siphon design incorporates both a wide forebay area off of the Mississippi River through the batture as well as a sedimentation basin within the forebay. The forebay itself would have sloped walls for stability and be oriented such that it minimizes the impact of river currents on flow into the station. The settling basin within the forebay would have vertical walls constructed of sheet piling with concrete caps and sufficient bracing to support the depth of excavation provided for sediment storage. The vertical wall arrangement also reduces the width of the basin while maintaining sufficient cross-sectional area for low velocities and effective sediment capture. The top of the pump station intake sill would act as a weir to enable the sedimentation capture the settling particles.

The hydraulic basis of design for the forebay is to achieve an average velocity less than 1 foot per second (ft/s). With this velocity, approximately a large portion of the sands that are 0.2 mm or larger would settle out. This arrangement would retain the majority of the heavier suspended solids prior to them reaching the pump intakes, thus preventing damage to the pump components. It would also stop the transport of such particles into the conveyance system.

The forebay settling basin can be dredged from a river barge, from excavation equipment located on the batture, or a combination of both, which minimizes the maintenance expense burden. Such an arrangement would also reduce the annual accumulation of sediment in the downstream channel sedimentation basin. This would reduce the frequency of maintenance activities required for the more expensive landside removal of unwanted solids.

Periodic monitoring of sediment depths throughout the forebay, riverside settling basin, and channel sedimentation basin is recommended along with a maintenance dredging program. During flood years when the batture would be submerged, additional sedimentation in the forebay would likely occur. A log boom consisting of a series of floating barriers linked together and held in place by foundations consisting of cribs containing large rip-rap, may be considered for placement in the river at the forebay entrance. The booms would prevent floating debris from entering and accumulating in the forebay if the pumped siphon alternative is selected for design.

4.5 PUMP STATION

4.5.1 PUMP STATION DESCRIPTION

The general pump station configuration for this conceptual analysis is depicted in 11, while Figures 12 and 13 show the station sections. As the figures show, the pump station would be a concrete structure with eight wet wells, one for each main pump. The use of individual wet wells is based on the recommendations of the Hydraulic Institute (HI) to minimize turbulence and vortexing, both of which can lead to damaging cavitation of the pumps. Each pump would have its own discharge pipe routed from the pump station, over the levee, under River Road, and to an outlet structure at the head of the Maurepas conveyance channel.

The pump motors would be located on top of the main deck while the discharge piping would run below the deck as shown in the section views of Figures 12 and 13. The main deck, motors, control building, electrical equipment, and discharge piping would be elevated to match the top of the levee to provide flood protection in the case of an extreme event in which the river stage reaches the height of the levee crest. Access hatches in the main deck would provide entrance to a mid-level deck for assembly/disassembly of the pump discharge couplings.

The control building for the station would be located on the main deck and house the control room and office; electrical and equipment rooms; and a restroom. A bridge crane above the main deck would be provided for removal and replacement of the pumps as well as maintenance of the ancillary equipment components. Auxiliary equipment for the pump station consists of the vacuum priming pumps; actuated control valves; sewage ejector pumps; and heating, ventilating, and air conditioning; among others.

The electrical supply for the main pump motors would be 4,160 volt, three phase, 60 Hertz, and would be supplied by the local utility, Entergy Power Company. A utility substation would be located on the land side of the levee to transform the power from the utility voltage down to the pump station supply of 4,160 volts. A 4,160- to 480-volt transformer for auxiliary equipment and other uses at the pump station would be located on the main deck.

4.5.2 INTAKE STRUCTURE CONFIGURATION

The intake structure was designed in accordance with the Hydraulic Institute (HI) requirements for large pumping stations. The pump station will consist of eight pumps (the reasons for selecting this number is discussed below) and be located on the batture of the river, leading to the selection of a rectangular intake configuration. The pump bays would be installed adjacent to each other and oriented perpendicular to the river to channelize water flow toward the pumps. Since the river current is significantly greater than the flow directed into the intake, the farther back into the batture the station is located, the less turbulence created by cross-flow patterns.

This ideal installation is countered by the USACE requirements to maintain levee slope and channel bank stability; these issues are addressed elsewhere in this report. Despite the significant cross-flows, the pump station will be conceptually designed using HI standards for rectangular intake structures. Prior to final design, a hydraulic physical model will be required according to HI; a discussion on physical models follows this section.

The structural dimensions for the pump intake are proportioned by HI as a function of the pump inlet bell diameter (D). HI provides charts for determining D as a function of flow through the pump, which results in the selection of sizes that maintain flow velocities within an acceptable range. Using the HI chart with a target inlet bell velocity of 5.5 ft/s, yields a 90” suction bell for the design flow of 250 cfs. From the derived D value of 90”, the following dimensions are then established based on the HI criteria:

- Clearance from sump floor: 0.5 D = 45-in (3.75-ft)
- Clearance from back wall: 0.75 D = 67.5-in (5.625-ft)
- Pump bay width: 2 D = 180-in (15-ft)
- Clearance from trash rack: 4 D = 360-in (30-ft)
- Clearance from wing walls or floor slopes: 5 D = 450-in (37.5-ft)

A critical factor in pump intake design is determining how deep the pump must be submerged below the surface of the water. HI defines a set of equations and provides charts for determining the submergence requirements. The required submergence is a function of the Froude Number at the suction inlet and the diameter of the suction inlet. Based on these requirements, the required pump submergence is 166” above the top of the suction inlet bell at design flow. The total water column is the sump floor clearance and required submergence and is 211” at design flow. At a low river stage of 0 and a flow of only 208 cfs, the required submergence is 153” and the total water column is 198”. Since the design flow occurs at river stage 9.12 and low flow occurs at river stage 0.0, the low flow condition will govern. With these dimensions, the flow velocities approaching the pumps will be less than 1.5 ft/s for all flow cases.

The 0 stage or low flow condition will require a sump floor at elevation -16.5. The intake channel to the pump station will have an invert of -7.0-ft, therefore, the pump station floor will have to be sloped upward from -16.5-ft to -7.0-ft. HI requires this slope to not exceed 10 degrees and must be 450” away from the centerline of pipe. The horizontal length of this slope will be 54-ft.

Dividing walls would be constructed between each pump, creating a partitioned structure. HI requires partitioned structures for large pumps, where a large pump is anything greater than 5,000 gpm (11 cfs). Because the top slab of the structure will be at elevation +32.0 and the bottom slab will be constructed at an invert of -16.5, the total un-braced concrete wall height will be 48.5', neglecting top slab thickness. For these dimensions, 30" slab and wall thicknesses will be assumed.

The total dimension of the pump station based on hydraulic requirements will be 48.5' in height, 142.5' in width, and 99.5 from front to back.

4.5.3 Vertical Propeller Pumps

The pump station will be comprised of eight 250 cfs pumps. Eight pumps were chosen because the hydraulic assessment in the previous analysis indicated that eight to ten units would provide the design discharge at reasonable flow velocities and river stages. Additionally, 250 cfs is about the upper capacity limit of vertical drainage pumps that are commonly available; larger pumps can be made to specification, albeit at a substantially higher cost. For this application, 72" pumps will be required. The selection for this pump size was discussed beforehand and was generally drive by the large flows versus very small heads at higher river elevations. Again, 72" pumps are generally accepted as the largest standard vertical drainage pump diameter.

To achieve the requisite 2,000 cfs for the diversion, eight pumps supplying 250 cfs each were selected for the subject design. For a discharge pipe size of 6-ft diameter, 250 cfs yields a flow velocity of 8.84 ft/s, which is within the industry standard range of 2.5 to 10 ft/s. This is more than sufficient for preventing solids deposition in the lines, yet low enough so that erosion of the pipe material is not a concern. The 250 cfs flowrate per pump is sufficient to limit the units to a reasonable number and yet not too large to unduly limit the number of manufacturers capable of bidding on the pumps. The 250 cfs volumetric capacity and the static head required to pump over the levee dictates the use of vertical propeller axial flow pumps. The maximum size for another potentially viable option is submersible pumps; however, they typically have capacities of less than 200 cfs, which would require more pumps and intake bays to achieve the design flowrate. The vertical propeller pump is well suited for the installation requirements and is available from several well known manufacturers.

4.5.4 Pump Head Requirements

In the June 3, 2010 Technical Memorandum, URS demonstrated that a vacuum siphon system could only deliver 2,000 cfs at river stage 16.11-ft, enabling only 80 days of operation per year at the design flowrate. In comparison, the gated intake structure would be capable of delivering 2,000 cfs at river stage 9.12, or about 190 days per year. The use of vertical propeller pumps would enable the requisite design flow to be delivered year-round. The conceptual design is based on eight 72-in pumps connected to eight 6-ft diameter pipes routed over the levee, each conveying 250 cfs. For selection and sizing of the pumps, a series of system curves was developed for the discharge configuration, as depicted in Figure 14. Because the river stage varies significantly over time, a single system curve does not represent the range of heads against which the pumps will operate, therefore, a series of curves was developed for various river stages at 0.5 foot increments from 0.0-ft to 22.5-ft.

The system curves are comprised of three components: 1) the head losses through the discharge piping. With dedicated pipes for each pump, the head losses only depend on the flow in its assigned line; they are independent of the flows in the other pipes, 2) the static head imposed by the tail-water elevation in the conveyance channel. Because the discharge channel conveys the water from all eight pumps, its head loss and therefore tail-water elevation is a function of the total flow. For this assessment, it was assumed that all eight pumps would operate simultaneously. And, 3) the static head available from the Mississippi River stage.

Using the conveyance of 2,000 cfs through the gated intake structure as a reference operating condition yields the following calculation results for the total dynamic head (TDH) required:

- 6.82 feet of head loss through the discharge piping
- 7.53 feet of static head created in the diversion channel[1]
- 9.12 feet of available static head in the Mississippi River
- A net TDH of 5.23 feet (from 6.82-ft + 7.53-ft – 9.12-ft).

The pumps required for such an application would be vertical propeller axial flow drainage pumps, which are typically used for high flow and low head service conditions. Both 60” and 72” drainage pumps were considered for this application. The 60” pumps would run off their performance curves before reaching the design point. Patterson A&M-Flow 72” pumps would be able to reach the design point of 250 cfs (per pump) at 5.23 ft of head. The series of performance curves for an eight-bank, parallel pump configuration for these units, from a single pump operating to the full eight, is also provided in Figure 14.

4.5.5 VERTICAL PROPELLER PUMP LIMITATIONS

From the pump curves provided and system curves developed, several issues with the subject configuration are apparent. First, because the tail-water elevation in the channel is highly dependent upon the volume of flow, less than three pumps operating cannot create sufficient head loss in the channel to create any significant WSE at the upstream end of the channel. As the river stage rises and the differential head between the water levels in the river and channel increase, the pumps begin to “run-out” or reach excessive rotational speed before their pump curves intersect the system curve. Thus, counter intuitively, as the river stage rises more pumps must be brought on-line to prevent damage to the pumps and motors. The station controls must therefore include provisions to set a minimum number of pumps required for operation based on river stage. Table 2 provides some preliminary set points indicating the number of pumps that are required to be in operation versus the river stage to insure fail-safe operation of the pumping units.

[1] From TM-1: “Mississippi River Reintroduction into Maurepas Swamp Preliminary Hydraulic Assessment of an Alternative Siphon Intake Structure” issued by URS to OCPR on April 22, 2010.

Table 2 - Minimum Number of Pumps Required vs. River Stage

River Stage	0.0 ft	2.0 ft	4.0 ft	6.0 ft
Min. No. of Pumps Required	5 pumps	6 pumps	7 pumps	8 pumps

The second issue is also illustrated in Figure 14. When all eight pumps are in operation and the river rises above stage 9.12-ft, the total flow exceeds the design point of 2,000 cfs. Coincidently, for the high volume/low head axial flow pumps required for the subject application, the combined performance curve for eight pumps operating ends just beyond the design point. To prevent the conveyance channel from overflowing, one or more pumps would have to be shutoff.

This would move the operation to the seven pump curve, the end of which is significantly even further away from river stages above 9.12-ft. The other problem with shutting off one of the pumps is that once a pump is turned off, a siphon flow can not be re-established for all eight pipes until the river stage rises to 11.50-ft; and at this stage, only approximately 1,550 cfs would be diverted via siphoning. The eight siphon system would not be able to divert the 2,000 cfs design discharge by siphoning action until river reaches stage 16.11-ft. To address this gap in flow delivery, either the pump curves or the system curves would have to be adjusted.

4.5.6 FLOW GAP COVERAGE APPROACHES

To permanently modify the discharge curve for a given pump, the impeller diameter, the speed of its rotation, or both can be changed. Such modifications may require the use of a higher horsepower motor. Alternatively, the pump curve can be adjusted by the use of an adjustable speed motor. This can be accomplished by using motors that have discrete multiple operating speeds or by the application of variable frequency drives (VFD's). VFD's enable the power input to a pump to be adjusted, which in turn adjusts the rpm of the motor, the impeller speed, and therefore the water discharge flowrate. However, VFD's are not suitable for the subject application due to the characteristics of the large axial flow pumps required. Large decreases in the impeller speed significantly reduce both the head that the pump can produce and its rate of discharge. The combination of reducing these two factors for a given pump effectively moves its output down the group of system curves to the point at which it no longer contributes to the pumping capacity of the station. A third way to change the pump discharge curve is to select a different pump; however, for the type of pumps required in this application, there is not enough difference between various models to overcome the fundamental problem of the pump operating envelope ending prior to reaching the higher river stages.

Instead of adjusting the pump curves, an alternate way to address the gap in flow coverage is to adjust the system curves. One means of modifying the system curves would be to use smaller diameter pipes, which would increase the dynamic head losses through the discharge lines. This would raise the system curves so that they intersect the pump curves at higher points on the total head axis. Raising the operating points would enable the pumps to function effectively over a larger range of river stages. However, it would also restrict the flow at river stage 9.12-ft (the average river stage over a typical year), decreasing the number of days per year during which the pump station could provide the target 2,000 cfs. Further, the higher head losses would require higher river stages to establish siphoning operation and also reduce the capacity of the siphon system at all river stages.

A more viable solution would be to create a series of adjustable system curves. The curves could remain as shown in Figure 14 for low river stages, where static lift is significant, as well as for high river stages, where siphon flow can deliver the design discharge without requiring pumping. To address the gap in flow delivery versus river stage, the system curves could be increased for the in-between stages by the use of throttling valves. The valves could be partially closed for river stages above 9.12-ft to increase the head required for a given flow. This would maintain the system within the pump operating range until siphoning could be established at river stages above 16.11-ft.

4.5.7 PUMP POWER REQUIREMENTS

The power required by a pump at a particular operating point is a function of the flowrate and head as well as the efficiency of the pump. The target flowrate is fixed for the eight pump system at 2,000 cfs, or 250 cfs per pump. The required discharge heads depend upon the river stage, conveyance channel tail-water elevation, and the head loss through the system. In reviewing fifty-seven years of river stage data near the Maurepas intake location, the historical low and high water stages are 0.2-ft and 22.5-ft. For the design flowrate of 2,000 cfs, the WSE in the conveyance channel is 7.53-ft and the head loss through each discharge pipe is approximately 7-ft. The efficiency of the selected axial flow vertical pumps within this flow/head region ranges from 86% to 90%, which is very high.

Based on the above data, at the design point with a river stage elevation of 9.12-ft, the required input power for the subject pumps to deliver the desired 2,000 cfs is 188 hp. For a river stage of 0.0-ft the power required to deliver the same flowrate over the levee and into the conveyance channel rises to 291 hp. For the power requirements to go above 300 hp, there would have to be both a negative river stage and a very high WSE in the diversion channel, neither of which is a probable occurrence. As stated above, the river stage has not even fallen to 0.0-ft over the last six decades and the diversion channel guide levees will only be constructed to 9.5-ft, so the maximum foreseeable power requirements for each pump is 300 hp. To provide a measure of reserve capacity the selected pumps would be specified with 350 hp motors.

4.5.8 THROTTLING VALVES

Each of the eight main pumps would be constant speed units; however, their discharge lines would have throttling valves to allow for a much wider range of flows to be routed to the conveyance channel. Under the design configuration, the pumps could be operated in either an automatic or manual fashion; however, the normal state of operation would be in an automated control mode. In automatic mode, the operator would select a desired discharge flowrate and the pumps would turn on as needed to reach that level of flow. As the river stage and the water surface elevation in the conveyance channel change over time, the controls would bring additional pumps on-line, take pumps off-line, or actuate the throttling valves to partially restrict the flow as needed to meet the set condition.

The target flows would vary seasonally depending upon the needs of the Maurepas swamp and also with the desired pulsing flow regime selected to achieve distribution of water deep into the swamp. In manual mode, the operator could select the number of the pumps to be operated and the degree of throttling, which again would vary seasonally due to changes in the river stage and the desired water surface elevation in the conveyance channel. The design control system would have override capabilities to prevent the pumps from being operated outside of their operating envelope. Whether in manual or automatic mode, instrumentation would provide data to the operator, either on-site or remotely via a Supervisory Control and Data Acquisition (SCADA) system to confirm if the system is providing the desired flowrate output. The SCADA system is described in detail below.

4.5.9 SCADA SYSTEM

The SCADA system would provide for monitoring and control of the diversion facilities at the Mississippi River from a remote location via radio telemetry. The diversion facility would have eight constant speed pumps and eight actuated throttling valves, enabling the flow into the diversion channel to be controlled over a significant range. During either pumping or siphoning operations, flow control would be achieved in discrete increments by bringing pumps on-line or removing them from service. Finer control would be accomplished by engaging the throttling valves to reduce the flowrate through the individual the pipelines.

Under normal operation, the individual pumps and siphons would be monitored and controlled at the diversion facility by a local operator. However, the individual pump status, pump/siphon flowrates, total station flowrate, water surface elevations in the forebay and outlet structure, and the status of ancillary equipment would be communicated to the remote office via the SCADA system. Other pertinent data that could be made available via the SCADA system would be determined during the design process, if the pumped siphon system is selected as the alternative of choice.

4.5.10 PUMP STATION PHYSICAL MODELING

The configuration used for the subject conceptual design is based on established HI parameters. The HI guidelines use a pump’s flowrate and operational characteristics to establish dimensional and geometric requirements for the intake bays and pump suction configuration based on proven designs or research efforts conducted. This approach generally results in minimal adverse effects such as vortices, excessive differential velocities, and entrained air ingestion on pump hydraulic performance, all of which can significantly affect large propeller type pumps in particular.

However, for a facility of this size and nature the HI also recommends physical modeling of the system. The results of the model testing may indicate that modifications are required from the strict application of the HI standards to make the intake arrangement feasible or to simply refine and improve the functioning of the overall system. URS recommends that such tests be conducted during the design phase if the pumped siphon alternative is selected. The cost of such testing is on the order of $100,000 and is included as a line item in the pump station construction cost component of the overall estimate.

4.5.11 EQUIPMENT EVALUATIONS

Equipment evaluations are a routine part of the design process for any pumping facility and would be required for the pumps, motors, electrical, and control equipment. No effort has been made in the conceptual design to optimize the various equipment components for their cost effectiveness; instead generic equipment meeting the overall system requirements has been the basis of the cost estimate. URS recommends that the use of mechanically cleaned intake screens be evaluated during the design process. These units will increase the overall pump station cost but can create a long-term savings in maintenance labor for screen cleaning. The results of the physical modeling often help indicate whether mechanically cleaned intake screens are a cost-effective investment. However, the cost of such units has not been included in the subject overall estimate.

4.5.12 PUMPED SIPHON SYSTEM OPERATION

The pump station may be started at any time that the Mississippi River stage is at or above elevation 0.0-ft. The initial step would be to engage the vacuum assist pumps to remove as much air as possible from all eight discharge pipes and lower the internal pressure within the lines. The resulting vacuum would pull a column of water into the discharge pipes to within 12-ft or less of the centerline of pipe on top of levee, which is at elevation 24.0-ft. Theoretically, a complete vacuum is equivalent to a reduction of one atmosphere of pressure, which can raise a water column 33-ft. However, no vacuum pumps are 100% efficient and head losses in the piping system restrict the practical limit of water elevation by vacuum suctioning to approximately 28-ft. Even so, this is more than sufficient to raise the water level within the suction pipes to elevation 24.0-ft from a river stage of 0.0-ft. At that point, the pumps only need to supply the additional 12-ft of head to completely fill the pipe column and establish flow.

Once flow has been established, there are several potential system operating regimes:

1) <u>Pump Flow</u>: At river stages below 9.12-ft, the pumps should be engaged and the throttling valves left fully opened. The pump capacity would be a function of the river stage as illustrated on the pump and system curves in Figure 14. For this operating regime, the vacuum assist pumps would only be required to prime the pipes.

2) <u>Siphon Flow</u>: At river stages 16.11-ft and above, the vacuum system will be able to establish the design flow without the use of the pumps. Individual siphon lines may need to be taken off-line to limit flows in the conveyance channel to 2,000 cfs so that the channel does not overflow onto adjacent properties. This was discussed in detail in the June 2010 Technical Memorandum. For this operating regime, the vacuum pumps would be required to both prime the pipes and to maintain siphoning.

3) <u>Throttled Flow</u>: At river stages between 9.12-ft and 11.50-ft, the pumps would be required to provide flow through the discharge lines; however, without control measures, the flows would exceed 2,000 cfs and the pumps would operate off the curve. The throttling valves would thus have to be engaged to prevent exceeding the design flow. By maintaining a total system head of 5.25-ft against the pumps, the restrictions created by the partially closed valves would limit the discharge flowrate to 2,000 cfs. For this operating regime, the vacuum pumps would only be required to prime the pipes.

4) Mixed Flow: At river stages between 11.50-ft and 16.11-ft, siphon flow can be used to deliver freshwater at discharge rates lower than the target 2,000 cfs without the use of pumps. Siphon flow could provide between 1,500 cfs and 2,000 cfs, depending on river stage. To establish this range of flowrates, the operator would follow the procedure for regime 2. However, use of the pumps would be required to deliver the full 2,000 cfs. To operate under pumped flow conditions, the operator would follow the procedure for regime 3.

Counter intuitively, as discussed previously, pumped flow can not be limited by using fewer pumps or reducing the pump speeds, because the system would not produce sufficient head to operate the pumps in this manner.

To support the pumping operations, a vacuum assist system would be required. Such a system would operate under the following regimes:

1) Priming Flow: A duplex vacuum system would be provided to prime the discharge lines as discussed above. Valves and controls must be provided to manage the station start-up process. The control logic would determine whether all of the pipes are primed simultaneously, or whether each pipe is isolated so that a single line can be primed at a time. This operational control scheme would only be required for initiating the Siphon Flow operating regime.

2) Maintaining Flow: The same duplex vacuum system would also be used to maintain siphon in the discharge lines during the Siphon Flow operating regime. At lower river stages, there is the potential for air to accumulate in the discharge lines. Under these conditions, the vacuum system must be periodically engaged to remove the air from the system. During higher river stages and during pumped flow, there is sufficient velocity in the pipe to move this air out of the discharge end.

These two vacuum regimes can be operated from the same duplex vacuum system, however separate vacuum headers and vacuum tanks would be required for each system.

4.6 OVER-THE-LEVEE DISCHARGE PIPING

Regardless of the relative water surface elevations (WSE's) in the river and conveyance channel, to initially establish a flow (i.e., prime the system), the water must be pumped. The variation of the Mississippi River stages throughout the year lead to different operating costs for the over-the-levee versus the through-the-levee piping siphon configurations. There are additional pipe losses for the elevated and longer over-the-levee piping of the Pumped Siphon system which increase the electrical power consumption. However, the additional costs of routing the pipes over-the-levee are estimated to be less than 2% of the annual power operating expenses. Since running the discharge piping over-the-levee only nominally increases operating costs and it maintains the integrity of the levee, this alternative was selected.

Figure 15 shows two profile views of the levee and the pipe crossing. The top section illustrates the existing levee topography and the Levee Design Section. The bottom section shows the raised levee section with the discharge piping on top.

4.7 Elevation of River Road

As shown in Figure 16, to route the discharge piping from the protected side levee toe to its discharge point into the conveyance channel, it has to cross River Road (LA Highway 144). Two options were considered for doing this: 1) an aerial crossing, and 2) a subsurface crossing. An aerial crossing for eight large diameter water conveyance pipes would require a significant foundation that would have to be piled supported. This is prohibited because the piles would have to be driven through the SCL of the extended levee section. Therefore, only the subsurface option is a viable alternative.

There is currently not enough clearance to cross beneath the existing roadway while also staying above the SCL. Therefore, either the alignment of the roadway must be shifted, so that SCL is deeper at the crossing or the road must be elevated to achieve the needed clearance. Shifting the roadway to the north is not a viable option because there is an existing residential adjacent to the existing roadway and because the revised alignment would require the acquisition of additional R/W by the LDOTD. Elevating the roadway thus becomes the only way to achieve the desired objective.

According to the LDOTD, River Road at the subject crossing location is classified as an Urban Collector – 2, with a speed limit of 45 mph. It has 12-ft lanes, 2-ft shoulders, a normal crown with a 2.5% cross-slopes, and side slopes of 1V to 3H. The AASHTO requirements to achieve a smooth transition over the vertical curve proposed to raise the road at the crossing limit the maximum change in grade to 8%. Thus, approach and departure grades of 4% were selected for each side of the crossing. To obtain a reasonable K value of 62.5, with a ΔG of 8%, requires a transition length of 500-ft. As shown in Figure 17, the raised section was designed using two 250-ft vertical curves with a 50-ft tangent section in between them on each side of the crossing. As the figure indicates, the total length of roadway to be raised is 1150-ft. The pipe and roadway section at the crossing are also depicted on the figure.

4.8 Outlet Structure

Figure 18 shows the outlet structure at the downstream end of the discharge pipes in profile view. The outlet structure would be an open-top, semi-rectangular concrete box, located on the downstream side of River Road, at the head of the conveyance channel. It would have walls on three sides with a weir outfall on the fourth side (back) wall for discharge into the channel. The top of the roadside wall would be flush with the adjacent finished grade and the two side walls would transition to the channel section. The minimum width of the structure is dictated by the spacing of the eight pipes entering the box. The depth of the box is constrained so as to avoid penetration below the extended Stability Control Line (SCL) of the levee.

The top of the weir would be well below the design water surface elevation in the conveyance channel, insuring that the structure is always full of water up to the weir elevation. The 6-ft diameter pipe from the pump station enters through the roadside wall of the structure and goes into a 6-ft to 8-ft diameter expander, which connects to an 8-ft diameter 90° elbow turned down toward the bottom of the structure. This slows the water velocity from 8.84 ft/s to less than 5 ft/s, greatly reducing the erosion potential of the fluid flow.

The exit of the down-turned elbow is at an elevation lower than the weir wall to maintain a submerged condition on the end of the pipe. The submerged outlet allows water to enter the pipe when the vacuum priming system is engaged during pump startup operations, which enables a siphon to be established. The weir does create an additional head loss in the overall discharge system. However, the loss can be minimized by increasing the weir length and/or by lowering the weir elevation. This exit loss and the need to mitigate it would be more fully developed during the design process if the pumped siphon alternative was selected.

4.9 CONCEPTUAL CONSTRUCTION COST ESTIMATE

The conceptual cost estimate presented in this report can be defined as a Class 4 level estimate in accordance with the classification system of the Association for the Advancement of Cost Engineering International (AACEI). According to AACEI, a Class 4 estimate is typically used for project screening, determination of feasibility, and concept evaluation. Typically engineering is from 1% to 15% complete and the expected accuracy ranges are -15% to -30% on the low side and +20% to +50% on the high side, depending on the complexity of the project, the available reference information, and the inclusion of an appropriate contingency determination.

The costs presented do not include engineering, legal, administrative, or right-of-way acquisition costs. Costs are presented in 2010 dollars, and are not inflated to the midpoint of construction because the construction schedule is not known at this time. The cost estimate has been prepared for guidance in project evaluation from the information available at the time. The final costs of the project and resulting feasibility depend on actual labor and material costs, competitive market conditions, actual site conditions, final project scope, implementation schedule, and other variables. As a result, the actual project costs may vary from the estimate presented. Because of these factors, project feasibility and funding needs must be carefully reviewed prior to making specific decisions or establishing project budgets to ensure proper project evaluation and adequate funding.

Appendix B includes the detailed calculations upon which the estimated construction cost was based. The construction costs were separated into the following major elements:

- Raising River Road
- Raising the levee to the Design Section elevation
- Installing the discharge piping over the levee
- Constructing the forebay and sedimentation basin
- Construction of a light-duty vehicular bridge
- Construction of the pump station
- Construction of the discharge structure

Table 3 shows the cost estimates for each of the components described above along with the probable total construction cost of the pumped siphon alternative. Note that these cost elements only extend from the river to the sedimentation basin at the upstream end of the conveyance channel. As the table shows, the most expensive cost items are the raising of River Road, construction of the vehicular bridge, and construction of the pump station.

Table 3 - Pumped Siphon Construction Cost Estimate

Construction Component	Estimated Construction Cost
Raise River Road & Install Piping	$ 3,980,000
Raise Levee Section	$ 385,000
Install Discharge Piping	$ 2,755,000
Construct Vehicular Bridge	$ 2,500,000
Construct Pump Station	$ 26,860,000
Construct Discharge Structure	$ 1,400,000
Total Probable Construction Cost	**$37,875,000**

The overall objective of this alternatives analysis was to develop a comparative cost estimate between installing the gated intake structure with box culverts through the levee to that of constructing a pumped siphon system. Therefore, the common elements in the overall project, such as the landside sedimentation basin, conveyance channel, US 61 crossing, two railroad crossings, Hope Canal pump station, etc. are presumed to cost the same to construct under either alternative. The total estimated construction cost of the gated intake/box culvert installation, including the, common components, as presented in the 30% Design Report was $151,725,000 The costs of the major elements from that alternative that are applicable to the current analysis (i.e., those required to convey the flow from the river to the landside sedimentation basin) have been broken out and are tabulated in Table 4 below.

Table 4 – Gated Culvert Construction Cost Estimate

Construction Cost Component	Estimated Construction Cost
Equipment	$1,575,000
Structural Concrete	$16,510,000
Pilings & Structural Steel	$6,720,000
Miscellaneous Items	$2,060,000
River Road Crossing	$20,000
Coffer Dam	$385,000
Total Cost of Components	**$27,260,000**

Thus, the differential cost of installing the pumped siphon system over the gated intake structure/box culvert installation is $10,615,000.

Note that if the pumped siphon system as presented is deemed unacceptable to the USACE due to excavation and pile driving below the projected SCL, the costs of the alternative will escalate depending upon the distance that the pump station has to be moved away from the levee and toward the river. One option would be to locate the station such that the sill of the intake is at the existing grade of the batture immediately before the river bottom begins its steep descent. Under such a scenario, no excavation would be made below the SCL; however, support piles would have to be driven through it. The impact would be to increase the length and therefore the cost of the access bridge and the discharge pipes, while eliminating the cost of excavation for the forebay and riverside sedimentation basin. Of course, this would also mean losing the benefit of sediment capture upstream of the pump intakes. These changes would change the overall cost of the pumped siphon alternative.

If driving piling below the SCL is still unacceptable to the USACE, then the pump station will have to be constructed in the river, which will require a significantly different design and will escalate the costs substantially. Before conceptual design and costing out such an alternative, URS recommends that OCPR obtain the USACE's evaluation, input and commentary on the proposed configuration.

4.10 OPERATING & MAINTENANCE COST

4.10.1 O&M COST QUANTIFICATION APPROACH

The O&M costs presented in this supplement are divided into three categories: 1) utilities costs, 2) labor costs, and 3) equipment costs. The approach taken to quantify each of these components, the resulting cost estimates and the O&M Cost Opinion for a Pumped Siphon System are presented below.

4.10.1 UTILITIES

The largest expense of operating the proposed pumping system is the cost of supplying the requisite electrical power to operate the pumps. Other utility costs include electricity to run the vacuum units, automated valves, lights, etc. as well as the potable water requirements of the vacuuming system. The following steps were taken to estimate the cost of these items:

- Estimation of the pump operational time over the 20-year period. This includes estimating the number of days during the year the pumps will operate and the average run-time per day.
- Determination of the annual operational time that the vacuum system components will be required to function. This includes estimating the percentage of the year that the river stage will be at the point at which vacuum operation will be required to initiate siphoning operation.
- Computation of the approximate kW-hrs required to operate the pumps, vacuum units, and miscellaneous ancillary components per year. This includes calculating the horsepower requirements for the pumping system based on the target monthly flowrates and the variation of river stage throughout the year. It also involves computing the peak load demands for pump start-up.

- Estimation of the annualized electrical power supply costs. This will be based on the estimated equipment run times and the following cost data from the local utility: the typical cost per kW-hr, the fuel adjustment surcharge, the peak demand power factor, and any transmission and/or distribution system charges.
- Calculation of the approximate water usage requirements to operate the vacuum system. This involves applying the annual operational time of the units to an estimate of the volumetric water requirements of each unit.
- Estimation of the annualized water supply cost. This involves obtaining the local water rates and applying them to the supply demands of the vacuum units and the miscellaneous water requirements of the station.
- Summation of the average annual electricity and water supply costs to operate the pump station.

4.10.1.2 LABOR

The second major expense category includes the cost of labor to operate the station and to perform routine preventative maintenance on the subject facilities. The basic activities undertaken to estimate the cost of labor for operation and maintenance of the pump station are as follows:

- Determination of the average yearly labor hours by category required to operate a facility of the subject size and complexity.
- Estimation of labor rates for the required operating and service personnel.
- Estimation of the annual preventive maintenance hours required for the various equipment components. This includes both minor activities, such as pump bearing lubrication, valve stem re-packing, and replacement of electrical fuses, as well as major service overhauls, such as removal and servicing of the pumping units.
- Estimation of routine housekeeping operations, such as cleaning the trash screens and removal and disposal of the collected debris.
- Calculation of the total cost of labor to operate and maintain the pump station on an annualized basis.

Note that the cost of maintenance supplies, e.g., lubricants, fuses, etc. is neglected due to their minor impact on the overall O&M expense budget.

4.10.1.3 EQUIPMENT

The final significant cost element is that of replacing the major mechanical and electrical components comprising the system which have a service life less than the twenty-year analysis period. Estimating those expenses involved the following tasks:

- Determination of the replacement schedule for the various major system components. This includes estimating the useful service life of the pumps, motors, vacuum units, valves, control system components, etc.

- Estimation of the removal and reinstallation cost of the major components.
- Computation of the average annual cost of removal and replacement of the various pump station components.

The summation of the above three expense categories over the 20-year evaluation period then yields the overall O&M cost estimate. Note that the subject pump station will continue to operate beyond the analysis period and therefore no costs for decommissioning and demolition of the facility at the end of its service life are included in the overall cost estimate.

4.10.2 UTILITIES COSTS

The largest single O&M expense of operating the proposed pumping system is the cost of supplying the requisite electrical power to operate the pumps. The subject pump station is designed to operate 24-hrs/day, 7 days/week, and as close to year-round as possible. Therefore, it will see many more hours of continuous duty than a typical drainage pumping station, which makes the energy costs extremely important. Since the electrical power consumption is the highest operating cost component, the more efficient the pumping units, the lower the annual costs. To reduce the rate of power usage, the pumps should be designed such that the motor horsepower is as low as possible and the normal mode of operation is near the best efficiency point (BEP) of the pump.

The greatest demand for input power is during pump start-up, when the water must be lifted from the river to the top of the levee through the empty pump column to prime the system. The priming operation only lasts for a short period of time, so it would be very inefficient to optimize the pump and motor for these conditions because it would result in an oversized pumping unit that would operate far from its BEP during the majority of the time. Instead, a vacuum system can be utilized to assist a smaller pumping unit by lifting the water part way up the column. The vacuum units only operate for the short period of time it takes to assist with the priming of the system. Once prime is established, the vacuum units are turned off and the pumps are able to sustain the target flowrate while operating at their BEP's.

The vacuum units consume electricity as well, albeit at a much lower rate than the primary pumps. In addition to assisting in the short-term priming operation, the vacuum units also operate periodically to evacuate accumulated gases from the flow-stream and thus maintain siphoning conditions. Other minor electrical costs include electricity to run the automated valves, lights, and other ancillary components.

The second utilities cost is that of the water consumption required to operate the vacuum units. The water enables them to maintain their seal and establish a vacuum. This is discussed in detail in Section 4.10.2.5.

The following steps were taken to estimate the cost of the utilities:

4.10.2.1 PUMP OPERATIONAL TIME

To estimate the pump operational time over the 20-year analysis period, the number of days during the year that the pumps will operate and the average run-time per day must be established.

To achieve the target flow rate of 2,000 cubic feet per second (cfs), each of the eight pumps must convey 250 cfs from the river, over the levee, and into the conveyance channel. To move that volume of flow, against the corresponding system head based on an average annual river stage of 9.1-ft with 72-in discharge pipes, will require 250-hp to operate the pumps.

To determine a representative electrical power usage rate, the system operation was divided into three categories based on river stage and the corresponding pumping conditions: 1) At river stages below 9.1-ft, the maximum motor horsepower rating of 250 hp was used, with the system fully open and no siphoning occurring. 2) For river stages between 9.1-ft and 11.5-ft, a reduction in load to between 200 – 250 hp was used. Under these conditions the target flowrate could be achieved via a combination of head differential between the river and conveyance channel and a lower energy input from the pump. and, 3) During those times of the year when the river stage is above 11.5-ft, the pumps would only be used for a short duration to establish the siphoning operation. Once the siphon is established, the pumps would be turned off.

The following table summarizes the number of days of pump operation based on historic river stage data and the corresponding operating condition:

Table 5 – Days of Pump and Siphon Operation per Year

Time of Year	Typical River Stage (ft)	Operating Condition	No. of Days
January 1 to February 15	9.1 – 11.5	Pumps Throttled[1]	**45**
February 16 to June 15	> 11.5	Siphon Flow[2]	**120**
June 16 to June 30	9.1 – 11.5	Pumps Throttled[1]	**15**
July 1 to December 31	< 9.1	Pumps Fully Open[3]	**180**
Miscellaneous	N/A	Down-Time	**5**
		Total	**365**

1. <u>For Pumps Throttled</u>: All eight pumps operate continuously. Throttling valves will be used to adjust system losses. Pumps will operate at 250 cfs at 200 hp each. Two start-up sequences are assumed per month. The vacuum units will only operate during the 30-minute start-up periods for each pump.
2. <u>For Siphon Flow</u>: No pumps operate continuously. Vacuum units will cycle periodically, remaining on approximately 15 minutes per day to maintain the siphon. Two start-up sequences are assumed per month, during which both the pumps and the vacuum units will operate for a 30-minute period per pump.
3. <u>For Pumps Fully Open</u>: All eight pumps operate continuously. Flow and power usage will vary from 200 hp to 250 hp with river stage. Two start-up sequences are assumed per month. The vacuum units will only operate during the 30-minute start-up periods of each pump.

As Table 5 shows, for the latter half of the year, the low river stages require that the pumps operate fully open. The river stages, system flows, and power requirements for that six-month period are tabulated in Table 6 below.

Table 6 - Pumps Fully Open Operating Schedule

Month	Dates	River Stage (ft)	Flow (cfs)	Power (hp)
July	1-15	8.40	1980	200
	15-31	6.42	1925	220
August	1-15	5.15	1865	240
	15-31	4.35	1835	250
September	1-15	4.06	1825	250
	15-30	4.07	1825	250
October	1-15	4.52	1845	250
	15-31	4.67	1850	250
November	1-15	4.81	1860	240
	15-30	5.82	1880	240
December	1-15	7.43	1950	220
	15-31	8.84	1990	200

4.10.2.2 VACUUM SYSTEM OPERATIONAL TIME

The vacuum system will always be used to initiate flow operations, even during the spring of the year, when the river levels are the highest. The crest of the levee is at elevation 31-ft while the maximum river stage during the average river year is only 15-ft, leaving 16-ft of purely static lift just to reach the pipe invert at the levee top. The crown of the pipe will be an additional 6-ft higher, increasing the minimum head needed to fill the line to 22-ft. The frictional losses and the input energy required to accelerate the fluid add a couple more feet, resulting in a total dynamic head (TDH) of approximately 29-ft. When the river level drops to its lowest annual stage of around 4-ft in the fall of the year, the TDH increases to approximately 30-ft.

A pump capable of producing these heads would be inefficient to operate along the system curve applicable to the normal difference in elevation between the river and the conveyance channel, which ranges from -13-ft to +7-ft. The much lower horsepower vacuum unit can evacuate the air within the pump column, enabling the water level to rise about 24-ft. This allows a much smaller horsepower pump to be used to supply the additional 5-ft to 16-ft of head required to push the fluid up and over the levee and establish flow. Once the pipe is flowing full, the vacuum unit can be turned off, since the pump has sufficient power to maintain flow on its own once the system is primed. The lower head pump will have a reduced horsepower rating and will be able to operate nearer its BEP, thus requiring significantly less power to operate.

It is assumed that the vacuum system will operate discretely approximately twice per month year-round to initiate flow in the pipes and establish priming. The priming operation is assumed to take 30 minutes to establish flow for each of the eight pipes, yielding 96 hours of run-time per year (8 pipes x 24 uses/yr x 0.5 hr/use = 96 hr/yr).

The vacuum units will also operate intermittently to evacuate the gases from the flow-stream that would otherwise accumulate at the crest of the pipes, which would break the siphon. This intermittent operation will be required during the 120 days from late February through early June. It is hard to quantity the amount of time that the units will run to maintain the siphoning condition. Proper submergence of the pump intakes will prevent direct ingestion of air into the piping system, thus the only vapor accumulation will be from entrained air in the river water column and the evaporation of dissolved gases. The eight pipes will have air release valves connected to a header that leads to the vacuum units. So, the vacuums will remove the air from all eight pipes concurrently, which will reduce the required cycling frequency. A rough estimate is that the vacuums will operate, on average, once a day for approximately 15 minutes, resulting in an annual run-time of 90 hours (1 use/d x 365 d/yr x 0.25 hr/use ≈ 90 hr/yr).

Therefore, adding the priming plus intermittent run-times, yields a total operating time for the vacuum units of approximately 186 hours per year.

4.10.2.3 ELECTRICAL POWER USAGE

The total annual electrical power usage is the sum of the approximate kilowatt-hours (kWh) required to operate the pumps, vacuum units, and miscellaneous ancillary components per year. The pump operational times as discussed in Section 4.10.2.1 and tabulated in Table 5 must be multiplied by the horsepower requirements and converted to kilowatts to yield the annual electrical power consumption of the pumps. The kilowatt-hours (kWh) of energy consumption for all eight pumps during each 15-day period throughout the year are tabulated in Table 7. As shown, the pumps will consume a total of approximately 8 million kWh of electricity per year.

The horsepower requirement for a vacuum-assist unit is typically around 10 - 15% of the primary pump that it serves. For a pump rated at 400 hp, the corresponding vacuum power requirements will be 40 - 60 hp, so 50 hp motors will be assumed for the vacuum units.

Determining start-up loads: Entergy monitors the highest 30-minute electrical load for each 30-day period. The highest 30-minute load will occur during pump station start-up, assuming simultaneous start of all eight pumps after the system has been pre-primed. The start-up power needed to exert the torque necessary to initiate pump operation is calculated to be 700 hp. After the first few seconds, when the initial system inertia is overcome, the remainder of the priming operation requires approximately 350 hp. Once the system is primed, the required operating power is significantly less, varying from 200 hp to 250 hp.

Energy consumption will vary throughout the year based on the river stage, as discussed above. For a conservative estimate of energy usage, the pumps were assumed to operate continuously within the range of 200 – 250 hp, depending upon the differential head between the river and the conveyance channel. The energy consumption is tabulated as follows:

Table 7 – Monthly & Annual Energy Usage for Pumps

Month of the Year	Dates During the Month	Motive Power per Pump (hp)	Electrical Power for 8 Pumps (kW)	15-day Energy Usage (kWh)
January	1st – 15th	200	1200	432,000
	16th – 31st	200	1200	432,000
February	1st – 15th	200	1200	432,000
	15th – 28th	0	0	0
March	1st – 15th	0	0	0
	16th – 31st	0	0	0
April	1st – 15th	0	0	0
	15th – 30th	0	0	0
May	1st – 15th	0	0	0
	16th – 31st	0	0	0
June	1st – 15th	0	0	0
	15th – 30th	200	1200	432,000
July	1st – 15th	200	1200	432,000
	16th – 31st	220	1320	475,200
August	1st – 15th	240	1440	518,400
	16th – 31st	250	1500	540,000
September	1st – 15th	250	1500	540,000
	15th – 30th	250	1500	540,000
October	1st – 15th	250	1500	540,000
	16th – 31st	250	1500	540,000
November	1st – 15th	240	1440	518,400
	15th – 30th	240	1440	518,400
December	1st – 15th	200	1320	475,200
	16th – 31st	200	1200	432,000
Total Annual Pump Energy Usage (kWh) :				**7,797,600**

4.10.2.4 ELECTRICAL POWER COST

Estimation of the annualized electrical power supply costs is based on the estimated equipment run times and the following cost data from the local utility, as shown in Table 8:

Table 8 - Entergy Gulf States, Inc. Electric Service Billing Schedule[1]

Billing Category	May – October	November - April
Load Charge (All kW per month)	$6.49 per kW	$5.67 per kW
Energy Charge (All kWh Used)	$0.01034 per kWh	$0.01034 per kWh
Delivery Voltage Adjustment	Delivery Voltage	Charge (Credit) per kW of Billing Load
	< 69 kV	$0.13
	69 kV	$0.00
	138 kV	($0.17)
	230 kV	($0.35)
Fuel Adjustment Charge	Monthly kWh times variable factor from Rate Schedule	
Tax Adjustment Charge	No charge assessed – State funded operation	
Total Charges	Σ Load Charge + Energy Charge + Delivery Voltage Adjust. + Fuel Adjust. [Note: Higher charges would apply if Entergy had to install extensive new electrical facilities, but this will not be the case.]	

1. Note: The data in this report are based on the electric service charges listed above, which are the standard rates that apply to Large Power Service customers. However, the State will probably enter into a contract with Entergy for electrical service provision at a reduced price schedule.

Entergy determines the billing Load Charge based on the Customer's maximum 30-minute demand measured during any 30-minute period of the billing month or 2500 kW, whichever is greater.

At pump start-up, the 700 hp motors would only draw power at that load for a short time, rapidly ramping down to their operating load of 250 hp, yielding an average 30-minute start load of 312 hp. Even if all eight pumps were to start at once (an unlikely scenario), at their 312 hp average loading over 30-minutes, they would only create an 1862 kW demand [(8 x 312 hp) x 0.746 kW/hp = 1862 kW]. This is less than the 2500 kW lower limit for which Entergy assesses a service charge to industrial customers regardless of the actual 30-minute maximum load. Thus, the minimum 2500 kW demand will be used to determine the Load Charge.

The Energy Charge is based on a flat rate applied to the kilowatt-hours (kWh) of usage during the subject time period. As tabulated in Table 7, over the course of a year the pumps will consume approximately 8,000,000 kWh of electric power.

The electrical distribution system along River Road at the Maurepas project site is assumed to be 4160 V, which is considerably less than 69 kV and therefore a monthly Delivery Voltage Adjustment charge will be applied to the minimum 2500 kW demand.

The power plant supplying electricity to the pump station utilizes natural gas as its fuel source, thus the Fuel Adjustment Charge varies as the price of natural gas fluctuates. The US Energy Information Administration (EIA) states that the use of natural gas for power generation, which has been on a generally upward trend over the last several years, will continue to grow despite a year-over-year increase in natural gas prices. Over the last decade, natural gas prices have varied widely, rising from just below $2 per million British Thermal Units (MMBTUs) in 2000, to nearly $15/MMBTU in 2006, and back to the current price of approximately $4/MMBTU. The EIA recently projected natural gas prices would remain below $7 per million BTUs through 2025. This would represent an average escalation rate of 3.8% per annum, which is higher than the Office of Management & Budget's current forecast inflation rate of 2.1% per year through 2020.

Since the Maurepas diversion will be operated by the State of Louisiana, no Tax Adjustment Charges will be applied. The following table lists the above electrical power costs by category along with the total annual cost of electricity for the pump station.

Table 9 - Pump Station Annual Electrical Power Costs[1]

<table>
<tr><th colspan="2">Billing Category</th><th>Quantity (kW or kWh)</th><th>Rate</th><th>No. of Months</th><th>Charges</th></tr>
<tr><td rowspan="2">Load Charge</td><td>Winter</td><td>2500</td><td>$5.67</td><td>6</td><td>$85,050</td></tr>
<tr><td>Summer</td><td>2500</td><td>$6.49</td><td>6</td><td>$97,350</td></tr>
<tr><td colspan="2">Energy Charge</td><td>8,000,000</td><td>$0.01034</td><td>12</td><td>$82,720</td></tr>
<tr><td colspan="2">Voltage Adjustment</td><td>2500</td><td>$0.13</td><td>12</td><td>$3,900</td></tr>
<tr><td colspan="2">Fuel Adjustment</td><td>8,000,000</td><td>$0.03626</td><td>12</td><td>$290,080</td></tr>
<tr><td colspan="2">Tax Adjustment</td><td colspan="3">State Operated - Tax Exempt</td><td>$0</td></tr>
<tr><td colspan="5">Total Annual Energy Costs</td><td>$559,100</td></tr>
</table>

1. Based on Entergy Gulf States, Inc. Rates as of September 2010

While the cost of electricity already represents the major O&M expense, the escalation of electric power costs is an unknown variable; however, they are rising more rapidly than other costs. Industry estimates of the increase over the next twenty years range from a low forecast of 1.5 times the existing rate to a high of almost 3 times, with a medium forecast of doubling. The above power costs are based upon the current rates from the energy provider and the largest component is the Fuel Adjustment charge. As discussed above, the cost of natural gas fluctuates significantly, but is on a generally upward trend that is predicted to escalate faster than the rate of inflation. Thus, the electrical operating cost will comprise a greater proportion of the overall O&M cost over time and could eventually surpass all of the other costs, including the capital investment, over the life of the station.

4.10.2.5 WATER USAGE REQUIREMENTS

The standard industrial device utilized to achieve a partial vacuum is a rotating positive displacement pump, which uses water to seal the space between the ends of the rotary impeller vanes and the pump housing. These pumps require a continuous flow of freshwater to maintain a liquid ring that is held in place by the centrifugal acceleration applied by the rotary vanes. The impeller shaft is placed off center in the pump housing, so that the air space between the eccentric impeller and the liquid ring is variable. This is what provides the vacuum and compression action required to move air from one side of the pump to the other. On the suction port side, the air space between the impeller vanes and the liquid ring is thin, so a relatively small volume of air is ingested. As the vanes rotate the liquid ring is pulled away from the impeller shaft, so the space increases in volume, which creates a partial vacuum. As the impeller completes the rotation cycle, the space again decreases in volume as the liquid ring thickens, approaches the impeller shaft, and compresses the air for expulsion through the discharge port side.

Liquid ring vacuum pumps can operate continuously with volumetric air flowrates of up to 23,000 cubic feet of per minute (cfm), reducing pressures by up to 14.24 psi, or 32.9-ft water. The liquid ring vacuum pump was patented in 1914 to Lewis Nash of the Nash Engineering Company, which is still one of the leading manufacturers. Such units are regularly used for municipal drainage systems, where the intent is to provide a static lift of water up a length of conveyance piping to prime a pump. In practical applications, where frictional losses occur in the actual system, these types of vacuum pumps can create up to 27-ft of vertical water lift.

As discussed in Section 4.10.2.1, the TDH required to lift the water from river stage 0-ft to elevation 36-ft, at a sufficient flowrate to fill the pipes, would require very large pumps and motors. Using vacuum units that can raise the water level to 27-ft, reduces the static lift during such extreme low water events to only 9-ft, enabling the use of much smaller pumps and motors. During higher river stages, the vacuums can lift the water all the way to the top of the levee, thus completely eliminating the static lift requirement of the pumps. At river stages of 11.5-ft and above, once water flow is established it will be self-sustaining via siphoning. While the pumps could be used to do this, it would be very inefficient; the vacuum units alone can establish the siphon under such conditions.

The size of vacuum pump required for a given application is a function of the volumetric flowrate and vacuum pressure desired. For the subject application, the intent is to be able to prime a single discharge pipe within 30 minutes. From a river stage of 0-ft to the pipe crest at the crown of the levee at 37-ft, each 6-ft diameter pipe runs 375 linear feet and encloses approximately 10,500 cubic feet (cf) [$(\pi\ (6\text{-ft})^2/4) \times 375\text{-ft} \approx 10{,}500\ \text{ft}^3$]. To remove this volume of air within 30 minutes requires an average evacuation rate of 350 cfm. Initially, when the discharge pipe is at atmospheric pressure (no vacuum), the vacuum pump will operate at a much higher capacity, removing air significantly faster than 350 cfm. As the unit pulls a vacuum in the pipe, the pump will have to work harder to remove additional air and the volumetric throughput at the end of the 30 minutes will be much lower. The average power input requirements over the priming period will only be 18 hp; however, the pump must have enough power at the end of the cycle to achieve the total required vacuum. To create sufficient vacuum to lift the river water 27-ft up the pipe will require a vacuum pump with a 50 hp motor and a constant supply of 7 gpm of potable water.

Two vacuum pumps of this size are recommended, enabling either the priming of two pipes simultaneously or a lead-lag arrangement on the operation of the units. Additionally, a duplex arrangement provides redundancy in case one pump fails and requires service. Parallel to these two units, a second duplex vacuum set will also be required for intermittent operation during siphoning to remove entrained air and out-gassed vapors which would otherwise accumulate in the discharge pipe. If these vacuum pumps are sized to match the priming vacuum pumps, four units would be available for initial priming of the system. The advantage to this set-up is that the entire system could be primed from a low river stage in about 1 hour (4 vacuum units operating in two cycles, priming 4 of the 8 pipes within 30 minutes during each cycle). The drawback to this configuration is that all four vacuum units would have to be large enough to prime the pipes, whereas two smaller units could be used for the intermittent operations. The smaller units would consume less power when running and they would run less frequently, but for longer periods. One option to prevent frequent cycling would be to use a vacuum storage tank so that the units only operate every other day on a rotating basis.

The water used to feed the liquid ring must be free of solids; it could be domestic potable water or filtered river water. Use of domestic water would not require the use of a feed pump or filer, but it would result in a monthly utility bill. Use of filtered river water would require a feed pump and filtering system, both of which would require periodic maintenance. Construction of a re-circulating water supply system could reduce the need for continuously filtering raw river water on a flow-through basis. However, neither of these measures are required because of the relatively minor cost to supply the water needs of the station, as discussed below.

4.10.2.6 WATER SUPPLY COST

As discussed in Sections 4.10.2.2 and 4.10.2.5 above, the water requirements of the pump station will be the summation of the following: 1) the volume required by the vacuum units to prime the pumps on average twice per month, 2) the volume required by the vacuum pumps to maintain the siphoning operations, and 3) miscellaneous minor usage, such as station wash-down, etc. The water will be supplied by connecting to the St. John the Baptist Parish domestic water supply system, which services the residential community directly across River Road from the proposed pump station site.

The billing rate schedule for the Parish-supplied water is as follows: \$4.26 / thousand gallons for the first 4,000 gallons; \$3.48 / thousand gallons for the next 2,000 gallons; and \$3.38 / thousand gallons for additional usage above 6,000 gallons. The water usage and costs for operation of the pump station for each month, along with the total annual values, are tabulated in the following table.

Table 10 – Pump Station Water Consumption and Cost

Month	Water Consumption (gallons)	Cost
January	3,492	$14.88
February	5,067	$20.75
March	6,642	$26.17
April	6,642	$26.17
May	6,642	$26.17
June	5,067	$20.75
July	3,492	$14.88
August	3,492	$14.88
September	3,492	$14.88
October	3,492	$14.88
November	3,492	$14.88
December	3,492	$14.88
Total Annual	**54,504**	**$224.16**

4.10.2.7 TOTAL UTILITIES COSTS

As Table 10 above shows, the water consumption cost to operate the pump station is well less than 1% of the electrical power cost and is therefore essentially negligible in the overall cost analysis. The total estimated cost of the two utilities is $559,334. Applying a 15% contingency results in annual estimated cost of approximately $645,000.

4.10.3 LABOR COSTS

The second major O&M expense category includes the cost of labor to operate the station and to perform routine preventative maintenance on the subject facilities. The pump station will be designed to operate as automatically as possible by using various sensors throughout the system. Some examples include:

- Level sensors placed in the river will signal the control system to turn pumps on at a target low river stage and shut them off once the river rises above a certain level.
- Pressure sensors in the vacuum bleeder tank will signal the system to turn the vacuum pumps on and off as needed.

- Level sensors in the diversion channel will operate the throttling valves in the discharge lines to regulate the water level at the head of the channel.
- A SCADA system will be used to record and report any faults in the system.

Despite the system's automation, there are still numerous variables affecting the station operation, including, but not limited to: variations in river stage, seasonal changes in water requirements of the swamp, the inherent tendency of equipment to lose calibration, the fact that components fail unexpectedly, etc. These and other issues require regular attention to keep the station operating correctly. The following sections discuss the O&M manpower requirements and costs.

4.10.3.1 OPERATIONAL MAN-HOURS

Even with a sophisticated control system, the operation of such a large pump station will require daily attention by a trained operator to ensure that the automated components are operating correctly and to manually override the system when necessary. The operator must check that the level sensors in the river and channel are reporting reasonable numbers, all pipes are discharging the proper flows, the pumps are running smoothly, the motors are not overheating, the vacuum units are maintaining the correct pressures, the throttling valves are in the correct positions, and a myriad of other items. These inspections will become routine as the operator learns the nuances of the station, none-the-less, they will require time simply because of the number of components to be examined.

In addition to simply inspecting the system status, oftentimes adjustments will have to be made, which will require varying amounts of additional time, depending upon the nature of the change required. Finally, monitoring and control equipment is only as reliable as its calibration and thus must be periodically re-calibrated to insure continued accuracy. This also takes operator time.

As previously discussed, the pump station will be designed to operate around-the-clock for approximately 360 days per year. It is anticipated that the station operator will spend on average approximately 4 hours per day examining, adjusting, and testing all of the various operating equipment components. Therefore, the annual operator labor requirements are estimated to be 1440 hours per year.

4.10.3.2 MAINTENANCE MAN-HOURS

A skilled laborer will also be required to perform routine preventive maintenance on the various system components at least once monthly. This includes activities, such as pump bearing lubrication, valve stem re-packing, and replacement of electrical fuses, for example. The exact scope of maintenance work will be dictated by the manufacturer's requirements as described in their O&M manuals issued with all of the equipment purchased for the station. Fault troubleshooting, along with minor repairs and/or replacement of failed or damaged components, will also be included in the maintenance activities. For the purpose of this cost estimate, the maintenance person is assumed to be the pump station operator or someone of equal skills, capable of maintaining the equipment.

Another maintenance responsibility is to clean the trash screens and dispose of the collected debris. With each high river, the flotsam washed down river is subject to deposition against the trash racks. As the water levels drop after the annual high water season, the flotsam that remains in place must be manually removed. The operator, or a separate crew, can use a long arm excavator to dislodge the debris and dispose of it either back into the river or into the river batture (downstream of the pump station). This work is anticipated to require one work day per year plus excavator mobilization and rental costs.

It is anticipated that the routine monthly maintenance activities will require one person two full work days each month, or 192 hours per year. Trash cleaning will require another two days, but only once a year, adding another 16 hours. Thus, the annual maintenance requirements total 208 man-hours. Non-routine maintenance due to major equipment failures, as well as major service overhauls, such as removal and servicing of the pumping units, is anticipated to be handled by outside crews hired for repair, replacement, and/or rehabilitation, as required, of the specific component.

4.10.3.3 LABOR RATES

Estimation of labor rates for the required operating and maintenance personnel must include their base salaries plus a percentage for fringe benefits such as Social Security, health insurance, vacation, etc. The average pump station operator hourly pay rate is approximately $28/hour; with an additional 40% for benefits, the loaded hourly cost is roughly $40/hour. The average laborer rate is approximately $16/hour, or a loaded rate of about $22/hour including benefits.

4.10.3.4 TOTAL LABOR COSTS

Using the operational and maintenance man-hours and labor rates discussed above, Table 11 was created to estimate the total annual labor for the pump station.

Table 11 – Pump Station Annual Labor Costs

Labor Hours	
Operator Hours / Year	1440 hours
Maintenance Hours / Year	208 hours
Labor Rates	
Operator Loaded Hourly Rate	$40.00/hr
Laborer Loaded Hourly Rate	$22.00/hr
Annual Total Labor Cost	**$62,176**
Maintenance Supplies	
Annual Cleaning Supplies, Lights, Lubricants, Hardware, Miscellaneous	**$3,000**
Equipment	
Annual Long Arm Excavator Rental	**$2,000**
Total Annual Labor, Supplies, and Equipment Parts Costs	**$ 67,176**

As a check on the total annual labor, supplies and equipment costs, a rule of thumb, based on recent quotes from vendors for stations of this complexity is $8/hour of pump operation. The subject pump station, with 24-hour operation for 360 days per year, will operate 8640 hours annually. At $8/hour this equates to $69,120 per year, which compares closely to the estimated value.

4.10.4 EQUIPMENT COSTS

In addition to the Utilities and Labor costs described above, the third and final significant O&M cost element is the Equipment Replacement costs. This cost includes replacing those major mechanical and electrical components comprising the system which have a service life less than the twenty-year analysis period. Estimating those expenses involves determining the service life of the various equipment items along with the costs associated with their removal and replacement.

4.10.4.1 EQUIPMENT REPLACEMENT SCHEDULE

Determination of the replacement schedule for the various major system components includes estimating the useful service life of the pumps, motors, vacuum units, valves, control system components, etc.

The pump repair and replacement costs comprise the most significant equipment replacement expense. The inspection and maintenance intervals are recommended by the manufacturer to insure that the design service life of their pumps is obtained under the operating conditions encountered. For the high-flow, low-head, axial-flow pumps of the subject pump station, the typical inspection interval required for each pump is 10 years and the average period between major overhauls is 20 years. For the 20 year cost analysis, it is assumed that two of the eight pumps will need to be pulled, serviced and reinstalled during the analysis period. It is also assumed that all eight pumps will require rehabilitation at the end of the 20-years.

Electrical motors and control panels require little maintenance or repair during their service life, provided the cooling system for the motor itself is maintained. The typical replacement interval for motors and control panels is every 20 years. For the 20-year cost analysis, it is assumed that two motors will need to be replaced during the service period, two control panels will need to be rebuilt and all motors and control panels will be replaced at the end of the 20-year service period. Vacuum pumps require little maintenance and have an expected service life of 20 years. For the 20 year cost analysis, it is assumed that one motor will require replacement, one control panel will require replacement, one pump will require replacement and both duplex vacuum pump systems will need to be replaced at the end of the 20-year service period.

Throttling valves will need to be serviced to maintain precise control of the flow through the discharge pipes during pumping and siphon flow. For the 20-year cost analysis, it is assumed that two of the 6-ft diameter valves will need to be removed and serviced once during the 20 year period and that all eight of the valves will be service at the end of 20 years. During each valve servicing event, the actuators will be replaced, resulting in replacement of 10 actuators over the 20 years.

The vacuum priming system will consist of duplicate header systems, both of which will be connected to the eight discharge pipes on one end and the four vacuum pumps on the other. This will require a total of thirty-six 6-in or smaller actuated valves. For the 20-year cost analysis, it is assumed that all of these actuated valves will be replaced twice, for a 10-year service life. It is also assumed that 18 of these valves will fail within the analysis period and therefore require replacement.

The discharge piping is expected to remain in place after the 20 year cost analysis period, however it is reasonable to assume that exposed portions of the piping will require a replacement coating of cold tar epoxy once during the 20 years. With sedimentation of the major solids prior to pumping and the fact that the river water itself is non-corrosive, it is expected that there will minimal erosion or corrosion problems on the pipe interior, thus no costs for internal pipe damage are included.

4.10.4.2 EQUIPMENT REPLACEMENT COST

The twenty year analysis period was used at the direction of OCPR, which is approximately the expected service life of most of the mechanical and electrical equipment. At the end of twenty years, a major overhaul of the station would be performed. This typically consists of completely replacing the electrical components both due to deterioration as well as the introduction of more sophisticated and efficient electronic devices. The pump motors are also generally replaced, but the pumps themselves are often in satisfactory condition and thus require only refurbishment, not replacement. Other smaller mechanical system and valves will be replaced at the end of the 20-year service life. Estimation of the removal and reinstallation cost of the major components.

The equipment cost of replacing a failed 250 cfs conventional vertical pump with a new one, excluding the cost of removal and reinstallation, is simply the current capital cost of a single pump, which is $1,800,000. The average inspection and rebuild cost for the same pumping unit is on the order of $100,000, including the cost of removal and reinstallation. When used in conjunction with an FSI, the cost increases about 10% due to the absence of a tail bearing, which results in more shaft and bearing wear than found in a conventional vertical pump that does have a tail bearing. An additional allowance of approximately $35,000 is needed for the use of a barge for removing, servicing and re-installing the pumps.

4.10.4.3 EQUIPMENT REMOVAL & REINSTALLATION COST

Removal and reinstallation of the pumps will require barge crane access, which can be accommodated in the intake channel. Electrical motors and control panels can be removed and replaced with a medium service crane operating from the batture area between the river and the levee. If complete replacement is not required, the control panels can be rebuilt on-site without removal. Vacuum pump assemblies can be removed and replaced with a small crane from the bridge deck. The throttling valves can be removed and reinstalled by a crane located on the protected side of the levee, along River Rd. The 6-in or smaller throttling valves for the vacuum system are small enough that they can be removed by man-power without the need for any lifting equipment and they can be readily transported by a light duty truck.

4.10.4.4 TOTAL EQUIPMENT COSTS

The total O&M Equipment Costs are tabulated in Table 12.

Table 12 – Total O&M Equipment Costs

O&M Services	Items Serviced / Replaced	Unit Cost (Parts)	Unit Cost (Installed)	Total Cost
Pump Service				
Intermittent Pump Failure	2 Pumps	N/A	$150,000	$300,000
20-Yr Service Life Rehabilitation	8 Pumps	N/A	$150,000	$1,200,000
			Total	**$1,500,000**
Pump Motor Service				
Intermittent Motor Failure	2 Motors	$300,000	$400,000	$800,000
20-Yr Service Life Replacement	8 Motors	$300,000	$400,000	$3,200,000
			Total	**$4,000,000**
Control Panel Service				
Intermittent Rebuild	4 Control Panels	$100,000	$125,000	$500,000
20-Yr Service Life Replacement	8 Control Panels	$100,000	$125,000	$1,000,000
			Total	**$1,500,00**
Vacuum Pump Systems				
Intermittent Motor Failure	1 Motor	$5,000	$6,500	$6,500
Intermittent Control Failure	1 Control Panel	$10,000	$12,500	$12,500
Intermittent Pump Failure	1 Vacuum Pump	$20,000	$26,000	$26,000
20-Yr Service Life Replacement	2 Duplex Systems	$60,000	$80,000	$160,000
			Total	**$205,000**
Throttling Valves				
Intermittent Valve Service	2 Valves	$100,000	$125,000	$250,000
Intermittent Actuator Failure	2 Actuators	$35,000	$45,000	$90,000
20-Yr Service Life Rehabilitation	8 Valves	$100,000	$125,000	$1,000,000
20-Yr Service Life Replacement	8 Actuators	$35,000	$45,000	$360,000
			Total	**$1,700,000**

Table 12 – Total O&M Equipment Costs (Continued)

O&M Services	Items Serviced / Replaced	Unit Cost (Parts)	Unit Cost (Installed)	Total Cost
Air System Actuated Valves				
Intermittent Valve Failure	18 Valves	$1,500	$2,000	$36,000
20-Yr Service Life Replacement	36 Valves	$1,500	$2,000	$72,000
10-Yr Service Life Replacement	36 Valves	$1,500	$2,000	$72,000
			Total	**$180,000**
Discharge Pipe Coating				
20-Yr Service Life Coating	41,500 SF Piping	$0.65 / sf	$2.30 / sf	$95,450
			Total	**$95,450**
			Sub-Total	**$9,180,450**
			Contingency (30%)	**$2,754,135**
			Total 20-Yr O&M Equipment Costs	**$11,934,585**

4.10.5 O&M COST OPINION

The total probable Operating and Maintenance cost over the 20-year analysis period, in 2010 dollars is approximately $24.5 million. These costs are tabulated below.

Table 13 – Total Probable 20-Year O&M Costs

O&M Cost Category	Annual Cost	Total 20-Year Costs
Electrical Power	$559,100	$11,182,000
Water Supply	$ 224	$ 4,480
Labor	$ 67,176	$ 1,343,520
Equipment	$596,729	$11,934,585
	Total Probable 20-Year O&M Costs	**$24,464,585**

5. CONCLUSIONS

URS performed a statistical analysis on 57 years of historical Mississippi River data, defining the "average river year", for which the mean stage was approximately 9-ft. The Gated Culvert structure proposed in the 30% Design was shown in the accompanying Preliminary Design Report to be capable of achieving the target 2,000 cfs at river stage 9.12-ft and sustaining that flowrate for an average of six-months per year. It was also shown to be able to provide lower flows during most of the remaining months.

TM-1 presented an analysis of an 8-pipe (72-in) Vacuum Siphon system, which showed that the river stage would have to be 17.14-ft for it to deliver 2,000 cfs, indicating that such a capacity would never be reached during the typical year, with an average maximum stage of 15-ft. At the 9-ft stage the Vacuum Siphon could only deliver 1,275 cfs. However, the siphons cannot be primed at this river stage - they can only sustain flow once priming has been established. The analysis also showed that the Vacuum Siphon system would only be operable from mid-February through mid-July, or approximately 168 days during the year, with a peak flow of 1,980 cfs for only 8 days. TM-1 thus demonstrated that a Vacuum Siphon system will not meet the goals of the Maurepas project.

TM-2 presented a conceptual design of a Pumped Siphon system, including a forebay and sedimentation basin in the batture; a pump station with eight 72-in pumps, throttling valves and pipes; a vacuum assist system; a levee and roadway crossing; and an access bridge to the station. The significant variation in river stage required for year-round operation greatly complicates the station design and operation. Four different operating scenarios were presented for various river stages: 1) Pump Flow for < 9.12-ft, 2) Siphon Flow for > 16.11-ft, 3) Throttled Flow for 9.12-ft to 11.50-ft, and 4) Mixed Flow for 11.50-ft to 16.11-ft. A sophisticated control system would be required to operate the vacuum assist units, turn the pumps on/off, add/remove pipes from service, and control the throttling valves.

The construction costs of the Pumped Siphon were estimated at $35,346,982, the major components being the pump station at $26,860,000 and raising of River Road at $3,980,000. The total Pumped Siphon construction cost was estimated to be $8,138,754 (30%) more than that of the Gated Culvert system, which was estimated in the Preliminary Design Report to be $27,208,228.

To supplement the capital cost estimate the work under TM-3 involved estimating the O&M costs for the Pumped Siphon alternative. The O&M costs were divided into: 1) utilities, 2) labor, and 3) equipment. The total probable O&M cost over the 20-year analysis period, in 2010 dollars was estimated as approximately $24.5 million, comprised primarily of $11 million for electrical power and $12 million for equipment repair and replacement. On an annualized basis, the average yearly O&M costs were found to be approximately $1,225,000.

Based on the reduced flow deliver capacity of both siphon systems along with the additional complexity, construction cost, and O&M cost of the Pumped Siphon, URS recommends continuance of the Gated Culvert structure as presented in the 30% Design.