CAPITAL ASSESSMENT REPORT: Wastewater Treatment Plant

TOWN OF RIDGWAY

Final Report
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WASTEWATER TREATMENT PLANT ASSESSMENT

Background

The original wastewater treatment plant was constructed in about 1976. It consisted of two earthen lined, approximately 8' deep lagoons. Initially the first cell operated as a partially mixed aerated lagoon and the second cell as a polishing pond. There was no influent flow measurement. A tablet feeder provided chlorine disinfection. When the plant was constructed it met the design requirements current at the time. By the early 90's, the scientific literature and CDPHE experience was concluding that lagoon cells needed to be deeper than 8 ft to control algae and that a minimum of 3 cells was recommended to provide more aerated detention time.

The hydraulic grade line of the interceptor into the plant is very flat, which complicated influent flow measurement and did not allow for the ponds to be raised much without adding a lift station at the plant. The Town did raise the cells a few inches by adjusting the height of the cell overflow weirs to improve the cell depth a little. This change causes a back-up of the hydraulic grade of the sewage to the point where it submerges the manhole immediately upstream of first cell and almost submerges the manhole to the south of that. The Town thought about deepening the cells to improve performance; however, that would require draining the lagoons, allowing them to dry and then excavating and relining. The latter was impractical especially with only two cells. Adding a lift station that increased the depth of the water would cause the ponds to leak. For the most part, the plant was in compliance and with the challenges above and CDPHE did not ask the Town to bring the plant up to the updated design standards during the mid-1980's.

In the mid 1980's the Town had a number of issues with the chlorine tablet feeder. The tablets would get trapped in the feeder tube and insufficient chlorine would get into the effluent leading to effluent coliform violations. As a result, the Town converted to feeding hypochlorite in solution for disinfection. Although all treatment plants were supposed to be measuring both influent and effluent flow starting in the late 1980's, due to the flat hydraulic grade, the Town was granted a temporary waiver by CDPHE. When the plant was expanded in the late 1990's the waiver expired and the town installed an extra-large trapezoidal flume to monitor the influent flow because the extra-large trapezoidal flume can measure flows on a relatively flat slope. A V-notch weir was installed at the chlorine contact chamber outfall to measure the effluent flow.

In response to compliance issues and algal overgrowth in the early 1990's, the Town added a baffle curtain to separate the second cell into two separate treatment areas, creating functionally a three celled system. The upper (southern) two thirds ran as a partially mixed aerated cell and the downstream (northern) third was quiescent and functioned as the polishing pond. For a period of time, this polishing pond area was covered with shade cloth to try to better control algal growth.

As the Town continued to grow through the 1990s, plant capacity started to present a challenge. There were days in the summer in the late 1990's when the plant exceeded 80% of the plant's 0.1 million gallons per day (MGD) capacity. The plant remained mostly in compliance but capacity challenges were looming. In the late 1990's the Town began planning for a plant expansion. In the 1999 timeframe the plant was expanded. The expansion included installing the new extra-large trapezoidal influent flume, adding a polypropylene lined third cell, a new chlorine contact-chamber with solution chlorination and dechlorination facilities, a new V-notch effluent flume and piping that allows the plant to run in series, parallel, or to bypass a cell. There is also a recirculation pump that can recirculate some of the effluent from the 3rd cell back to the front end of the first cell. The effluent from the 3rd cell that is not recirculated is disinfected with sodium hypochlorite (strong bleach), and if needed, can be dechlorinated and then discharged to the Uncompahgre River east of the plant. Piping for the plant expansion included provisions to add one more lagoon cell to the west of the current Cell 3, although flow into the cell to the west would come from cell 2 and the existing 3rd cell would become the 4th cell in the flow schematic. Figure WW-10 is the piping plan for the facilities, including existing conditions as well as showing the future full-build out with the fourth lagoon.

Cell 1 is the southern most cell. The 1999 expansion added sufficient aeration and mixing in the first cell for it to run as a complete mix pond. Note that at least in theory treatment is more rapid in a complete mix environment than in a partially mixed one but requires more mixing and thus requires more horsepower. Cell 1 at the high-water line is about 156’ long by 106’ wide by about 7-8’ deep. The pond has the physical and electrical assets to run up to four each 15 horse power (hp) surface splashers. The plant operators have determined that running two, 10 hp aerators (20 hp total) at a given time is adequate to keep the cell mixed and meet oxygen needs, as recently as 2018. Table WW-10 is list of the conduits and conductors to each of the cells.

Cell 2, to the north of cell 1, and hydraulically downstream of it, has overall normal high-water line dimensions of about 140 feet long, 106 feet wide, and about 7-8 feet deep. As noted above there is a baffle curtain about two thirds of the way toward the north end of the cell. Cell 2 has the electrical and physical capacity for two 10 hp aerators upstream of the baffle curtain and one 10 hp aerator downstream of the baffle. There is also a conduit with a pull cord already installed to run additional wiring as needed from the motor control center to the edge of Cell 2. Currently there is a 10 hp aerator in the upstream side of the baffle curtain and a 7.5 hp aerator downstream of the baffle curtain that is only operated part of the time, which meets current plant demand. Both sections of the second cell are designed and operated as partial mix. There is adequate electrical infrastructure for the treatment regime to be changed so that either or both sections of the second cell could operate as fully mixed, although more horsepower will be required and that added horsepower might result in substantial bank erosion.

Cell 3, the northern most cell, was constructed in 1999. It is hydraulically downstream of Cell 2. Unlike the two original cells which have earthen liners, the newer cell has a synthetic, reinforced polypropylene liner. At the normal high water line, it is about 243 feet long, 135 feet wide, and about 12 feet deep. There are two baffle curtains in Cell 3. Upstream of the baffle curtains in Cell 3, the pond is designed and operated in a partial mix mode. The area within the baffle curtains is designed for plug flow, meaning that a drop of water enters the area and flows through it in the order it entered. Plug flow provides little BOD removal by microbial action; instead it is the section of the treatment system where the water is moving slowly enough (not mixed) that the solids have time to settle out. The design aeration pattern in Cell 3 is to have subsurface aspirator-type aerators in diagonal corners and a standard surface splasher in the middle. The volume of the baffled area was based on having adequate polishing pond for the plant if the 4th and final cell was constructed to the west of current Cell 3, thus there is more quiescent (quiet, slow flow) area than is recommended for the current plant capacity. To limit the quiescent capacity (and the consequential algal growth and low oxygen levels at night), on the downstream side of the first baffle curtain, there is a single 10 hp surface splasher to keep the upstream half of the baffled area partially mixed and keep the volume that is quiescent appropriate to the current flows.

In addition to allowing for settling of the solids, the quiescent area also tends to allow for more algal growth. Note that excess algal growth increases the oxygen demand in the cell at night and also reduces oxygen levels in the river when it is discharged. One way to limit the algal growth is to shade the pond. At the Ridgway plant as currently operated, during the summer months when algal growth is most problematic, duckweed growth can provide shade, once it begins to cover the polishing pond section of cell 3. However, the duckweed growth is a biological process and staff has limited control over how early in the spring duckweed growth occurs or how dense the growth is. Some years it does not cover the pond soon enough to prevent an algal bloom in late spring or early summer. Other times it can be so dense that it creates shortages in dissolved oxygen. The other issue with duckweed is that it has a high organic and nutrient content and when it dies, which happens when it freezes, if not before that, it can put a significant load on the plant. Instead of letting the duckweed die in the pond, staff should remove the duckweed in the fall each year, which is currently being done. There are concrete structures on the northeast and southeast ends of Cell 3 to facilitate duckweed removal.

Lagoons are required to be lined so that they do not leak into the ground below and so that they not gain water from the groundwater table. The newer cell is lined with reinforced polypropylene and unless the material is torn, it is not likely to leak in the 20-25 year expected design life of the material which would mean the 2020 - 2025 range, perhaps even longer. The original two cells have a clay / earthen liner. Quality control during installation is critical to insuring a water tight seal with earthen materials. If the earthen materials are properly installed, the seal should remain functional unless the pond area is dried out or the earthen materials are disturbed. An example of the latter could be the result of removing vegetation that is rooted in the earthen liner material.

The Town's 2013 permit renewal required that the Town demonstrate that the existing facilities meet the leakage requirement (of less than 10^{-6} cm/sec or 0.034 inch per day). The Town was likely required to provide the documentation due to the discrepancies between the influent and effluent flow measurements. It should be noted that the flow measuring gauges are only required to be accurate to within 10% of their range, whereas the leaking requirement requires far more accuracy. In addition, the flat grade into the influent flume further reduces the reliability of the data from the influent flume. The Town staff prepared a report for CDPHE to demonstrate that the ponds were meeting the limits required by CDPHE and in early 2016 CDPHE confirmed that the plant was meeting their requirements. A copy of the documentation is provided in Appendix XXX. There is a possibility that a future permit may require an update to that documentation. The original study will hopefully be useful as guide for future requests.

If the cell liners remain adequate until about 2025 or later, before doing any liner rehabilitation, the Town will likely want to determine how it will meet the anticipated increasingly stringent nutrient effluent requirements that are anticipated in 2027 (see below and Appendix XXX). It is possible that the nutrient requirements could be so stringent that a lagoon system would not be able to meet the new requirements and the entire plant may be subject to an upgrade, possibly requiring a mechanical plant. If there appear to be problems with the liner sooner than that, the Town will want to weigh the costs of fixing the liner issues versus modifying the overall treatment process in light of the changes in stream standards expected in 2027.

As mentioned above and as can be seen on Figure WW-10, the design for the 1999 expansion included the potential for one more lagoon cell that under the then current regulations, would increase the plant capacity by 50%. Hydraulically the additional cell is designed to be placed between the current cell 2 and cell 3. The construction in 1999 included pipe stubs to add in the additional cell. Although adding in the cell would be a relatively inexpensive way to significantly increase the plant capacity, there are a few reasons why that option might not be viable. The first is that if the effluent regulations become so stringent that a lagoon system cannot treat the waste sufficiently to meet the new requirements. Another is that the existing site and the new cell are close to current and proposed future development. To expand the plant would require getting site approval from CDPHE for the changes in plant design and capacity which given the development around the plant might be a challenge as the existing plant and proposed pond are within CDPHE recommended setbacks. Both potential effluent requirement changes and setbacks are discussed in more detail below.

Capacity

The treatment plant has a design capacity of 0.194MGD and 400 pounds per day (PPD) of biochemical oxygen demand (BOD), which is a measure of the organic content of wastewater. Note that different uses generate different wastewater loads. For design purposes loading is typically compared to a typical "single family residence". There are a number of commercial uses (eg: restaurants, drinking establishments) that have higher BOD concentrations. User fees for wastewater are based on both the flow and assumed BOD loads as different uses create different demands on the system, impacting system capacity. Table WW-11 is a summary of the last several years of discharge monitoring results. Because monitoring results are typically collected on a single day to represent a given month, the results, especially the real influent quality data, can fluctuate significantly from what is measured in the single sample each month; however flow is measured almost continuously so is more accurate than the loading. Table WW-11 has rows showing percent of design capacity for influent flow and influent BOD on the monthly basis for each year.

Capacity as measured by influent flow is typically less than half the plant design capacity. Influent loading (pounds per day of BOD) is around half of the design capacity based on the single monthly samples. However there have been several months in the last 5 years where the loading was over 75% of the design capacity. These are likely a result of BOD being a little high during the time the wastewater was sampled for the month.

Figure WW-11 is a graph of the influent flow over the last 5 years compared to the design capacity. One can see that the flow is typically less than half the design flow capacity. Figure WW-12a shows the influent BOD in mg/l (concentration) and Figure WW-12b shows BOD loading in ppd, respectively. BOD seems to fluctuate significantly month to month. Typically, domestic sewage is in the 250-300 mg/l range. When there is substantial infiltration and inflow (I/I) the influent BOD is often in the 100 - 150 mg/l range. Looking at the Ridgway influent BOD data in the Figure S-12, BOD concentration is the 250-300 range and has been for several decades suggesting very low rates of infiltration or inflow. This is to be expected given the collection system is almost exclusively PVC with water tight joints. Most of the I/I seems to be related to rain events, when the Town does sometimes see a spike in influent flow. In response the Town has tried to locate the sources of the inflow and correct as needed. It is recommended that the Town continue to trace I/I sources as time and weather conditions allow, and follow up with the appropriate remedies and repairs.

There have been a few months with even higher BODs, most recently in the spring of 2017. The Town and lab staff at first thought the numbers in 600 mg/l range were a testing anomaly, but when a repeat test in the 500 mg/l range confirmed the results, the Town began trying to trace the source of the loading upstream. However, by the time the Town became aware of and confirmed the very high BOD and knew the problem was real after collecting the additional samples, the influent BOD concentration was back to normal, making it very difficult if not impossible to identify the source of the discharge to the system. The fact that the BOD was high for more than a week, suggests that the discharge of high concentration waste was not a single discharge or a single sample, but something that went on for several days. Note that because it takes 5 days to get results from BOD tests, it is difficult to locate the source a short-term high concentration discharge; however it is recommended that the town try to find the source when spikes occur by taking samples in manholes at dividing points in the collection system as soon as the a spike is identified rather waiting to confirm that the spike is real. The Town might also want to measure total suspended solids (TSS) if that is higher than normal, in part because that is a much faster test, which would make it easier to catch the location of the discharge before the discharge ceases. It is also recommended that when the influent BOD in a given month is above 350 mg/l the Town resample influent BOD to see whether the first sample is representative of the month as a whole. Currently the organic capacity is based on 4 samples taken over a single 8 hour period for the whole month. Sampling over a 24-hour period and more than once a month would provide a higher level of confidence in the true plant loading.

The existing plant is running at about half of its rated capacity. It appears that BOD loading rather than flow will dictate when additional capacity is needed unless more testing of the influent BOD indicates that past sampling has given higher BOD results than are observed with 24-hour sampling. The Town needs to keep in mind that the Town's discharge permit from CDPHE requires the Town to begin planning for plant expansion when the plant reaches 80% of design capacity and implement the expansion plan to increase capacity before reaching 95% of capacity, or impose a moratorium on new construction. Note that the current plant has a design capacity of 0.194 MGD and 400 ppd (pounds per day) BOD. Looking at the data from the last 5 or so years, the plant is not approaching the 80% threshold, but if the Town continues to grow consistent with the more aggressive forecasts, the plant could reach 80% BOD capacity within the next 10-15 years. The Town also needs to be tracking proposed changes in regulations that could require the Town to meet more stringent effluent limits and could make the existing plant obsolete before the flows and loading into the plant reach 80% design capacity. This is not insignificant and is addressed in this assessment in the Regulatory subsection below.

Mechanical items at the plant include (but are not limited to) aerators, recirculation pumps, flow monitoring equipment and data loggers, backup genset (which currently does not run), and chlorine feed equipment. The life expectancy of most of the mechanical equipment is +/- 20 years. Some of the aerators purchased prior to the plant expansion were reused with the new plant. The generator was purchased used and has not worked well. All the other mechanical equipment was purchased with the plant upgrade in 1999. The flow measurement equipment and recirculation pump were replaced in the last 2-3 years. The aspirator style aerators, although purchased in the late 1990's have had issues as have some of the old surface splashers. Several of the older surface splashers have been rebuilt to good operating condition; however, the rebuild / shaft replacement on the aspirators did not last.

For the last 5 years or so, the Town has been considering whether to change to sub-surface air (which is discussed in detail below). Given that the plant has operated adequately without an aspirator due to shaft issues and that the Town is considering changing the aeration system (see Aeration sub section below), that aspirator has remained out of service.

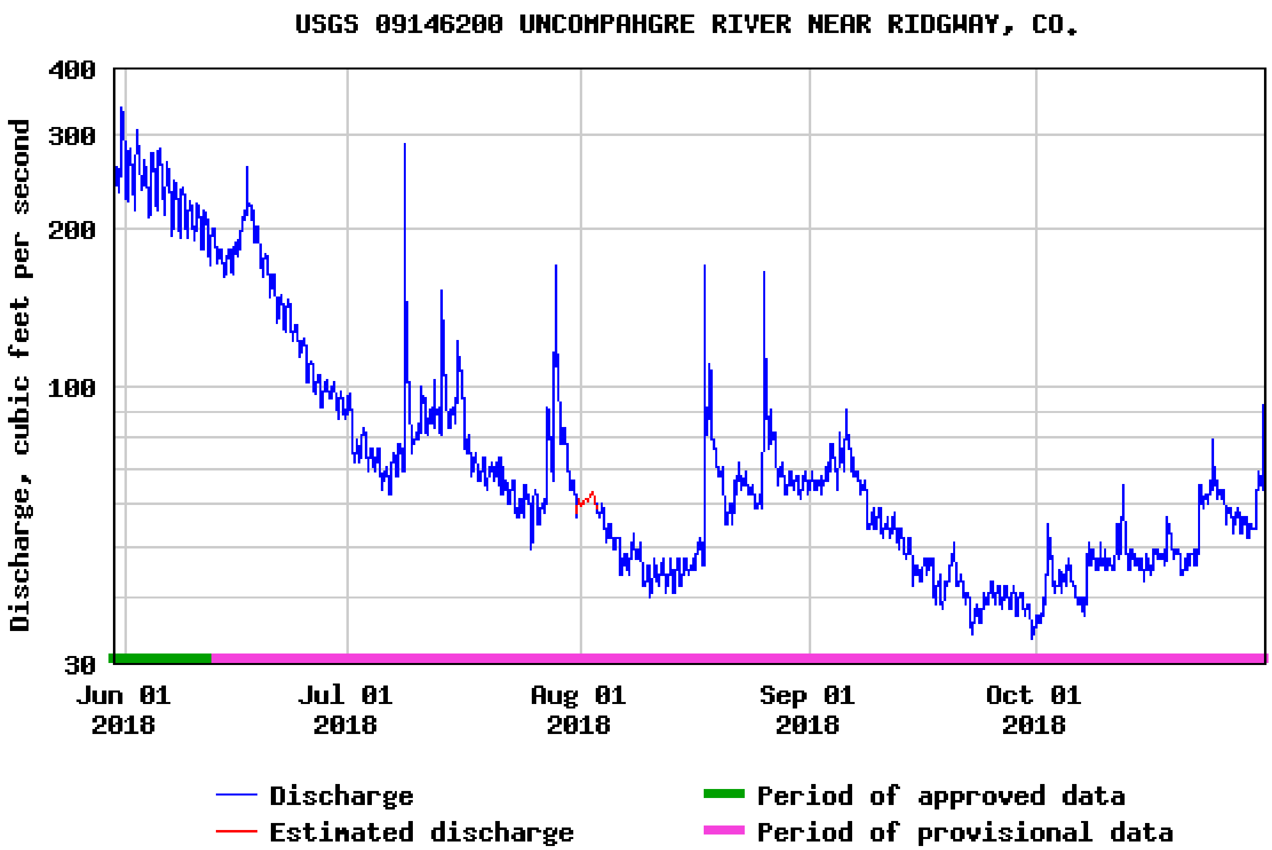
The Town should continue to track purchase and maintenance details for all the mechanical items at the plant. It is recommended that the Town's asset management system list each of the components individually for specific budgeting. In general, the Town should be setting aside money to replace the equipment as it wears out to the point that it is no longer cost effective to repair.

The existing facilities include a backup generator and the motor control center which is set up so that the Town can control which loads run on backup power. The generator was purchased used and has not operated since it was purchased. Typically, power outages at the plant have not been longer than a few hours, most are under an hour. The lagoons can go without air for a few hours on rare occasions without causing problems. Similarly, the recirculation pump and flow monitoring equipment can be idle for hours without adverse impacts. The one load that would be better to not have off line is disinfection, but it is hard to justify the cost of tens of thousands of dollars for a full genset for the plant to run the chemical metering pump for disinfection. Instead is it recommended that the Town consider a portable generator for the chlorine pump.

In the process of treating wastewater, biosolids are generated. With a mechanical system those are removed on a regular basis (as frequently as daily depending on the treatment system). With a lagoon system the solids (sludge) are only removed every 5-10 or so years. The Town last removed sludge in 2014 at a cost of approximately $85,000 for 95 dry tons. The Town should expect that it will be necessary to remove sludge again in the next 3 - 8 years, at significant expense. The last two times biosolids were removed the Town hired a contractor not only remove and haul the solids but also find a long-term disposal site. Prior to that the Town had arranged for the disposal site. The Town switched because of difficulty in find a site and the liability for the Town. In 2018, the Mautz Brothers in Olathe have indicated they plan to improve the Thunderbird Raceway property to accept not only green waste but eventually restaurant waste and biosolids, such as lagoon solids, for composting, and they have been communicating with the Town of Telluride to this end as a pilot location. This may be a good opportunity for the Town in the future as locations for sludge relocation can be difficult to find and expensive; however the Town should make sure that any facility with whom they work with meets CDPHE requirements and does not create any new liability for the Town. A facility such as the raceway property is likely more appropriate for intermittent lagoon removal than for a long-term frequent removal as would be needed for a mechanical plant. For a mechanical plant we suggest that the Town have long term control of the land on which the biosolids are applied.

Regulatory Matters

Treatment requirements and effluent limits can change every five years when the Town's CDPHE-issued discharge permit is renewed or when the Town opts to make a substantial change at the plant which triggers an off-cycle permit update. The current permit which expired the end of May of 2018 was a General Permit for wastewater facilities with a dilution factor of greater than 100:1 comparing the low flow of the receiving stream to the design flow of the treatment plant. Note that if the dilution is less than 100:1, the Town's plant would need a site-specific permit. The site-specific permit for a minor treatment plant like Ridgway's is not that different from the general permit, so that change is not a real concern. Having less dilution though could impact discharge requirements.



In 2013 when the last permit was issued, CDPHE determined that low flow in the river was 36 cfs using the CDPHE DFLOW model for the period from 2001 to 2013 as measured at the stream gauge upstream of Ridgway Reservoir. That resulted in a dilution of 120:1. Note that DFLOW provides a more conservative flow than the actual flows directly measured. Given the record-breaking low flows in the Uncompahgre in 2018 (see at left), if CDPHE uses the 2018 data, the dilution could drop below 100:1 and require a site-specific permit; however, even a dilution of a little less than 100:1 is a reasonable amount of dilution and water-quality based effluent limits (WQBEL) might still not be needed. The permit writer at CDPHE would need to check, and the Town will want to confirm CDPHE's assessment. Parameters that would be evaluated for potentially more stringent limits if dilution is less than 100:1 include ammonia, chlorine, selenium, temperature, and potentially other nutrients.

What is likely to be the most significant change in discharge requirements in the next decade could come in 2027-28 timeframe, when it is expected that the permits will have stringent limits for Phosphorus (TP), Total Inorganic Nitrogen (TIN), and total nitrogen. Nutrient limits were mandated by EPA about 10 years ago. Initially the Water Quality Control Commission (WQCC) only included the stricter limits for larger mechanical plants, with the thought that the smaller plants have less impacts on the receiving streams and that requiring all the wastewater treatment facilities to come into compliance at once would "bankrupt" the funding stream for plant upgrades. EPA did not agree with that plan and the State of Colorado has agreed with EPA that they will start to require nutrient limits as part of the effluent limits in almost all discharge permits starting in about 2027.

The State is aware that it takes a number of years to go through the permitting process, planning, design, environmental review, public input, and construction of a new or upgraded facility. They are planning to include a 5-year time frame (compliance schedule) for plants that receive nutrient limits for the first time in or after 2027 to come into compliance, which means dischargers should have until 2033 or a bit later depending on how soon after 2027 each discharge permit is renewed that include the more stringent nutrient limits. The compliance schedule in the 2027+ permits will include milestones during the 5 years to make sure that the permittees are on track to be able to meet the more stringent limits within the 5 years compliance schedule timeframe. However, CDPHE is aware that the timeframe for these sorts of upgrades is a slow process and with cause often allows for extra time. Note that the Colorado Water Quality Control Commission will hold a public hearing in 2027 to review and potentially adopt the anticipated changes. At that time, the limits adopted could be more or less stringent than the ones currently anticipated.

CDPHE still has concerns about how to review and / or fund that many plant upgrades all at once and about whether all entities will be able to meet the 5-year timeframe. At the same time, EPA and the State of Colorado would like to see some progress toward nutrient removal in the nearer term. CDPHE is currently offering a Voluntary Incentive Program (VIP). A permittee can sign up to test for phosphorus and / or nitrogen monthly and every month that the results meet the new standards and meets some associated criteria, CDPHE will add some time to the compliance schedule. To qualify for the incentive, the permittee needs to submit a nutrient reduction plan to CDPHE before 12/31/19. There is a template for the plan on the VIP website currently at:

https://www.colorado.gov/pacific/cdphe/nutrients/nutrients-incentive-program.

This voluntary compliance does not actually require a specific plan, just that the discharger sign up and to get the added time, meet certain criteria. Facilities need to achieve less than 1 mg/L of total phosphorus (TP) on an annual median to earn credit for phosphorus and less than 15 mg/l for total inorganic nitrogen (TIN). At a minimum, a monthly composite sample must be taken at the plant outfall to be eligible for credit. Credits are given on a sliding linear scale. Once a facility's annual median drops below 15 mg/L TIN and 1 mg/L TP, the facility starts earning incentive credits. The closer (or below) 7 mg/L TIN or 0.7 mg/L TP, the more incentive a facility may earn. More years operating at low levels also increases the incentive.

Signing up for the program is relatively easy and if the plant meets the limits demonstrating nutrient reduction, it would earn the Town additional time to meet the more stringent limits that are expected to be incorporated in the permits in about 10 years. It is not clear whether Ridgway's discharge would qualify for any incentive credits without significant changes. It is recommended that the Town sign up for the incentive program and start testing. If the results look like the Town is qualifying for incentive credits, it would pay to continue to sample monthly. If the results are significantly above the levels that would qualify for the incentive credits it is recommended that the Town consider whether it is worth the time and cost of the additional monitoring, but as will be discussed below, knowing the nutrient levels in the discharge has other value to the Town.

Because the dilution in the Uncompahgre is relatively high, it is likely the nutrient effluent limits to which the Town will be subject will be considerably higher than the stream standards that the Commission is expected to adopt in 2027. The Town will have a better idea of the amount of dilution in the receiving stream when they receive the pending discharge permit renewal in the coming months. It is recommended that the Town use the dilution in the new permit to calculate the mass balance for total nitrogen and total phosphorus to get a guestimate of how stringent the effluent limits might be in next decade. The Town will need to sample effluent TP and TN and have background stream levels (from CDPHE and local Riverwatch efforts and/or by sampling the river upstream of the plant) in order to calculate the mass balance, but having a better idea of the nutrient effluent limits would give the Town a better idea of what will be required going forward.

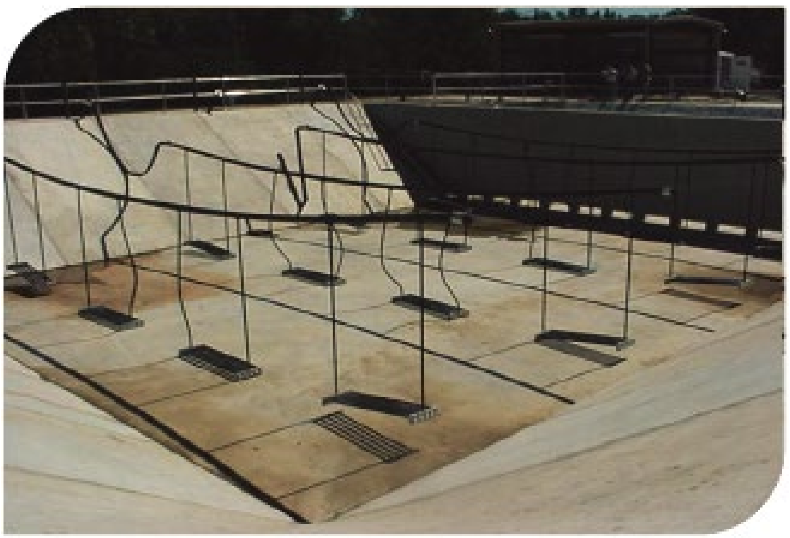
Aeration

The existing aeration system, as noted above, is comprised of surface splashers and aspirators. As currently operated, for the most part, all installed aerators are run full-time. The power costs for the plant are in the $45,000 per year range, representing the Town's largest energy demand and expense by far with the water plant being the next largest municipal energy demand and expense, estimated at $10,000 for 2018 and budgeted at $12,000 in 2019. The Town has been concerned about the amount of power consumed at the wastewater plant for a number of years. To this end the Town has on several occasions attempted to do energy assessments of the wastewater plant to determine if changes to the aeration system would meet treatment needs and if the investment in the alternative system could be partially paid back by the energy savings realized. The Ameresco audit included recommendations for energy conservation but also determined that the energy savings for the full package of improvements would not alone pay for the improvements.

One option the Town explored was whether installing a solar PV system that would offset the energy demand and determine a payback time frame for the investment with the energy savings; however, the payback was excessive (decades) and the Town determined the investment would not be beneficial for cost savings over time. Instead the Town invested in the San Miguel Power Association solar farm project through a Power Purchase Agreement in 2014.

During the process of interviewing firms for the energy audits, the Town learned that subsurface aeration might improve energy efficiency. The theory is that the surface aeration loses oxygen (and heat) in the splashing process; whereas subsurface air is compressed which adds some heat and, if there is sufficient depth, allows for significant oxygen transfer, and can improve the amount of air added to the water per kilowatt. In addition, as discussed below, the setback requirements between the wastewater facilities and habitable structures is less with subsurface aeration than with surface splashers, which from a land use perspective is beneficial for the Town and has been an expressed desire from prior elected and appointed officials.

It is recommended that the Town consider some modifications to the aeration system. In the short term, the Town should consider adding continuous dissolved oxygen monitoring to each cell and adjusting the run time of the aerators to match the need for oxygen in the cell. For instance, in the summer, dissolved oxygen levels in the cells can be super saturated, meaning that adding more air is not adding to the air in the pond and the mixing and splashing from the aerators may actually be reducing the oxygen level in the pond. There is a need during the day for some mixing, but typically the cells do not require full time aeration to provide the needed mixing. Monitoring the dissolved oxygen levels would allow the Town to adjust when aerators run (adjusting the timers) which could reduce the power consumption. To monitor dissolved oxygen, the Town should purchase dissolve oxygen sensors and data loggers and set them about 8' out from the outlet boxes in the 1st 2 cells and in Cell 3, set probes perhaps offset from the baffle curtains or duckweed boxes.



Typically, sub-surface aeration requires a water depth of about 10 feet or more to allow time for the air bubbles to diffuse into the water column and requires draining the ponds to set the diffusers. The Town's first two cells are only about 7-8 feet deep so would have limited time for oxygen transfer. The new cell is 10 - 12 ft deep and could accommodate standard sub-surface air, but the Town was hesitant to have blowers and sub-surface air in one cell and surface aeration in the older cells because of the additional O&M requirements.

As part of the investigation to try to reduce power consumption at the plant, staff looked into a few kinds of sub-surface aeration that used fine-bubble diffusion, which has more efficient oxygen transfer into the water column thus requiring less water depth. Because draining the cells to install air lines and diffusers on the cell floors seems impractical and could result in damage to the earthen and/or synthetic liners, staff focused on aeration systems that could be installed without draining the cells.

Because it did not require draining the cells, the Town requested pricing and design calculations for a proprietary Biolac system from Parkson. The Biolac system consists of diffusers suspended in the water from a cable system (see left). Air is delivered through air lines that are part of the support system. The system requires blowers to supply the air for the diffusers and if one wants to provide air based on oxygen demand in the lagoon one needs to include a dissolved oxygen monitoring system to the improvements package. The 2016 cost estimate for the equipment to convert all 3 cells to a Biolac system was in the $400,000 range.

We received an updated estimate for the Biolac system in early 2019. The cost for the system only increased about $15,000. Looking at the updated proposal, the total blower horsepower (hp) required for the system is around 62 hp. What is still not clear from the proposal is how much of the time that horsepower would run. If it runs full time it would result in a comparable power demand to current system. Parkson also provided a proposal for just using Cell 3 for treatment and limiting the Biolac aeration system to just that cell. That system had a price tag closer to $300,000 but would also require a clarifier and some piping changes. That is still expensive and that type change would require new CDPHE site approval. It's not clear whether CDPHE would allow the entire treatment process to be in a single basin. If the Town has interest in pursuing such a change, discussion with CDPHE staff is recommend.

There were no estimates of the resultant power saving from such a system and with the uncertainty regarding nutrient limits, the Town decided to wait a little longer before determining whether changing to sub surface air was warranted.   It is recommended that the Town monitor for TP and TN and see whether the existing facilities will be capable of meeting the new limits. If, with the dilution assumed in the new permit, which is likely to be issued in 2019, the effluent limits for the plant for TP and TN look to be achievable with minor modifications to the existing system, then conversion of the aeration system to something like the Biolac system could improve operational efficiency and be cost effective. If it looks like the lagoon system will not be able to meet the expected limits, then it probably makes sense to continue with the current aeration system until the Town determines how best to meet the new treatment requirements. While it is unlikely that the energy savings from any system improvements, either a PV solar system and/or a subsurface aeration system, will completely offset the cost of the investment, the Town can expect some resultant energy savings from one or both systems that could offset a portion of the upfront cost over time.

<u>Setbacks</u>

The guidance for Regulation 22 of the Water Quality Control Commission, which governs where one can place a wastewater treatment plant, includes a section (22.3(2)(e)) entitled "Guidance Specific to Odor, Noise, and Aerosol Mitigation from Domestic Wastewater Treatment Works" (a copy of this section is included in the Appendices). Several sections of Regulation 22 require that the Division review proposed treatment plant sites and any changes to existing plant sites to minimize foreseeable potential adverse impacts on public health, welfare and safety. The policy lists four factors to consider including:

1. Addressing potential concerns of neighboring property owners

2. Reducing the likelihood of public nuisance complaints from the operation and maintenance of the facilities including odors, noise, and aerosols,

3. Minimize the potential of airborne pathogens to be transmitted from the facility to neighboring habitable structures, and

4. Provide guidance if setback requirements cannot be met and mitigating factors must be incorporated into the design to mitigate potential odor, noise, and aerosol concerns.

Design of the treatment works and evaluation of the treatment type, process and location is required to include consideration of potential odor, noise, and aerosol issues. The regulatory guidance includes distances from treatment works to habitable structures that the Division will consider adequate. The following list if from that policy:

1. Non-aerated lagoons: ¼ mile
2. Aerated lagoons less than two (2) total surface acres (all basins combined) with no surface aeration: 250 feet
3. Aerated lagoons greater than two (2) total surface acres (all basins combined) with no surface aeration: 500 feet
4. Aerated lagoons less than or equal to two (2) total surface acres (all basins combined) with surface aeration: 500 feet
5. Aerated lagoons greater than or equal to two (2) total surface acres (all basins combined) with surface aeration: 1,000 feet
6. Mechanical plants 2,000 gpd maximum month capacity to less than 50,000 gpd capacity: 250 feet
7. Mechanical plants 50,000 gpd capacity to less than 100,000 gpd capacity: 500 feet
8. Mechanical plants 100,000 gpd or greater: 1,000 feet
9. All enclosed mechanical plants and lift stations: 100 feet
10. Lift stations 150,000 gpd capacity to less than 215,000 gpd capacity: 250 feet
11. Lift stations greater than 215,000 gpd capacity: 500 feet

Note that the existing facilities as operated would fall in category 4 above and should be 500 ft from habitable structures. If the plant changes to sub surface aeration, it would fall under category 2 and only need 250 ft. Figure WW-13 is a graphic showing the 250' and 500' setbacks from the existing lagoons. Given the regulatory changes that are likely in the next decade, the next upgrade or plant replacement could require a mechanical plant which if not enclosed would have 1000 ft setback. If the facilities are totally enclosed, the distance drops to 100 ft. Given the value of land in Ridgway, it is likely that a mechanical plant would need to be fully enclosed.

It is important to note that the distances above are what CDPHE uses as a default. If one can not meet the distances, CDPHE's review would be specific to the facility assessing the specific impacts the facilities might have. There are some ways to mitigate distances less than listed above.

This topic has come before the Planning Commission and Town Council many times over the years as land values and the cost of development increase. The appointed and elected officials have expressed a desire to minimize the setbacks as is reasonable to meet the mitigation requirements of the Town’s CDPHE-issue permit. In the late 1990s the River Park Industrial Park was sited and regulated to be proximal to the lagoons and to prohibit residential uses both in the Town Code and on the River Park PUD plat map, solely because of these setback requirements and mitigation required. The Town now has the lagoon setbacks in GIS and can readily identify properties subject to such setbacks and mitigation requirements. It is recommended that the Town continue to adhere to the setbacks as any reduction without mitigation may require the Town to absorb the investment and maintenance cost of the mitigation. Alternatively the Town could consider requiring development to absorb the cost of investment and maintenance for any future private development that proposes to encroach into the setback areas. However, that would be difficult to enforce, especially long term. Continuing to prohibit habitable uses in the setback is recommended.

Relocation or “Stay in Place” options

When the existing facilities were initially constructed in the early 1970's they were on the north end of Town. There were a few homes near the plant off the corner Lena and Otto, but nothing to the north, south, or east of the plant. In the decades since, then there has been considerable development to the west and north of the plant, placing the existing facilities now closer to the perceived center of town although the facilities have obviously not moved since initially installed. In the early 90's, the land owner to the north of the plant showed Town staff his property and suggested that there might be room to relocate the existing plant to a section of his property by the 40-acre BLM property along the east side of the Uncompahgre River. Staff had concerns about the proximity to the floodplain and the challenges with the size and access to the site as well as whether the sewage could reach the site without pumping. Without an urgent need to relocate the plant and no funding to do so, the Town opted not to proceed at that time.

In the late 1990's the property north of the existing facilities changed hands and the new owners began developing the properties to the west and north of the existing plant (River Park PUD and Ridgway Business Park). During the subdivision and zoning process for the River Park development, the areas within the recommended wastewater treatment plant setback were zoned Industrial 2 which restricts habitable structures and the River Park plat notes include some additional restrictions. With the development around the plant, the plant now seems to be in town rather than on the perimeter of Town resulting in an increased interest in relocating the treatment plant.

Looking downstream of the existing plant, there is not a lot of space that is out of the floodplain and to which the sewage could flow by gravity. Figure WW-14 shows the topography of the area downstream of the existing plant. There is an area on the east side of the bikepath just north of the bikepath trestle bridge (the old railroad bridge), about 1500 feet downstream of the existing plant, that is relatively flat and lower than the existing plant by about 6'. The bikepath (the old railroad grade) likely protects the area from the floodplain as the FEMA map (Figure WW-15) shows the area as zone C, an area of minimal flooding risk. Note: the FEMA map is out of date and needs updated. Depending on how close one could get to the bikepath, there might be 3+ acres at that site. There is also a residence about 300 feet to the northeast of this site. This site is quite a bit lower than the bikepath. A treatment plant at that this site would either need a very high privacy fence or dense, tall vegetation to keep the plant from being visible (eyesore) to those enjoying the bikepath. Conversations with the land owner will need to be had.

The Town has discussed acquiring the BLM parcel, however; it is likely in the floodplain given that it is quite a bit lower than the railroad grade and not protected by it. That site maybe a good acquisition for the Town for other purposes, but protecting the site from flooding would require raising the wastewater treatment improvements and a lift station for the sewage to reach the raised improvements.

A contrarian site considered is the gravel pit site to the northeast of the River Park Subdivision. Gravel was extracted from that site and used for road construction in the River Park Subdivision. The developers have yet to backfill the site and still have gravel stockpiled on the site. The potential site as shown on Figure WW-14 could be up to 9 acres. The reason this would be a contrarian site is because it is about 60 feet higher than the elevation of the collection system as it enters the existing plant site. Pumping the sewage to the gravel pit site would likely require at least 20 hp pumps and annual pumping costs would be on order of $8,000-10,000 for flows in the range expected in about 10 years. Capital costs would include a large lift station, a force main from the lift station to the site and a pipe line from the site back to the river for discharge. Although not clear from the topo on Figure WW-14, the google earth image in Figure WW-16 more closely reflects the extent of the gravel excavation. The excavation could provide an opportunity for a treatment basin or it could create a challenge in terms of siting improvements on the site. Figure WW-14 shows a hatched area on the west toward the north end which is less than 500 feet from the school. The site is large enough that improvements could exclude that encroachment. However, the school ballfields are even closer to this potential site and there could be some residences in the River Park Subdivision that could be closer than 500' from the site. Conversations with the land owner will need to be had.

Plant relocation requires finding a site, acquiring it and ultimately constructing the new facilities. In addition, before the plant can be relocated or even designed, the relocation requires new CDPHE site approval, anti-degradation review (a review of the impact on the river), and effluent limits from CDPHE. If the new facilities would be something other than a lagoon, the Town would also need to find a way to handle the biosolids that are routinely removed from the treatment process. In most cases on the west slope, the biosolids are put to beneficial use on the agricultural lands. Because the Town would need to use the site for the life of the mechanical plant, would need to make use of the site on frequent basis, and because the Town is responsible for the impacts of biosolids on the land in perpetuity, it is recommended that if the Town constructs a mechanical plant, the Town acquire agricultural land for biosolids disposal. It is estimated that the time frame from start to finish for a new, mechanical plant, including the steps generally described above, could take a decade or longer depending on how complicated the project becomes. Finding the right location by itself could take considerable time.

It is also possible to continue to treat the Town's sewage at the existing site. If lagoon treatment remains viable and the Town wants to reduce the recommended setback, converting to subsurface aeration and pond area of less than 2 acres would reduce the recommended setback to 250 feet. Should conversion to a mechanical plant become necessary to meet effluent requirements or be the preferred option for the Town, there is very likely sufficient room at the plant site to construct the facilities in the area where the future lagoon cell was planned to go. All mechanical plants require 1000-foot setback unless they are enclosed or otherwise significantly mitigated. At the existing site, a mechanical plant would likely need to be enclosed. If the facilities remain at the existing site, the existing motor control building, chlorination and de-chlorination buildings, chlorine contact chamber, and effluent line could be re-used. This would be a savings of +/-$100,000.

There are a number of different types of mechanical plants. All are more complicated, time consuming, and expensive to operate than a lagoon system. All mechanical systems also require bio-solids handling which will require not only treating the removed solids but also having a disposal site. At the existing site, one would want to select a type of mechanical system, that not only could meet longer term projected effluent requirements, was relatively easy to operate, but also with a relatively small footprint to reduce enclosure costs. Mechanical systems tend to be designed with a series of unit processes each with specific functions and specific mechanical equipment and controls. Because the systems are a series of components it is more common to expand, supplement, or replace components than to replace the whole system, although there are circumstances where replacing a full mechanical plant makes sense. Most equipment in a mechanical plant has a design life of +/- 20 years. With the rapid improvements in control systems, it is likely that it will be advantageous to upgrade treatment system controls more frequently, perhaps in 10 years or so.

Wastewater Revenues and Expenses

The revenue and expense comparison is based on a "typical year" using the budgets from 2017, 2018, and 2019 to determine what would be normal revenues and expenses. Table WW-12 lists the assumed values for both revenues and expenses. On the revenue side there are the monthly charges for service, as well as penalties, investment income and tap or system investment fees. The expenses include standard day to day expenses as well as much larger capital investments, reserves for major future expenditures, etc.

As of early 2019, sewer rates for a single-family house was $35 per month and in most cases $35 per month for each additional residential unit. The major exception to that is that the charge for accessory dwelling units is $25.20 (70% of the primary residential rate). There are a number of commercial uses (eg: restaurants, brewing facilities, drinking establishments etc.) that have higher BOD concentrations. User fees for wastewater are based on both the flow and assumed BOD loads because different uses create different demands on the system, impacting system capacity. For non-residential units, the fee is $35 per month per unit plus $1.75 per 1,000 gallons of water used per month over 6,000 gallons per month. There is also a surcharge for any non-residential users' discharging wastewater with BOD and/or TSS concentrations greater than those of the typical residential user (over 250 mg/l) of $0.80 per pound BOD.

For a quick and simplistic look at revenue versus expenses one can determine a cost per thousand gallons treated and compare that with the rates. Looking at typical operating expenses the cost per thousand treated is about $12.20 per thousand. The median water use for a residential unit in the winter between 2016 and 2018 is about 3,000 gallons per month and the average is about 4,000 gallons per month. If one assumes that most of the winter usage does get to the sewer, it seems reasonable to assume a typical usage of about 3,000 per month per residential sewer user which at $12.20 per thousand comes out to $36.60.

A more detailed look at revenues versus expenses should look at fixed costs, the costs that the Town needs to be pay whether or not there is much usage and the variable costs, the costs that are proportional to the actual treatment costs. The total expenses are broken into fixed in variable costs in the far-right columns in Table WW-12.

<u>Summary</u>

The existing treatment facilities are operating at about 50% of design capacity in terms of organic load and less than 50% of the design hydraulic load. The plant typically meets effluent limits and can likely continue to do so for several more years. With population projected to increase by about 50% by 2038, it is anticipated that the Town will need to begin planning for additional capacity in about 2030. This timing should work well with the anticipated regulatory changes anticipated in 2027 and having more certainty regarding the impacts of those changes on discharge permits in the following years. As noted above, if the cell liners should fail before 2027, the Town should try to determine what treatment changes will be required to meet the more stringent nutrient standards before investing in liner replacement which would like cost in the $250,000 range plus the cost to drain and clean the cells which could add another $100,000 to the costs.

The Town removed biosolids several years ago (2014) and will likely need to do so again in the next 3-8 years. This is a significant expense and the Town should budget for it. The cost in 2014 was $85,111. Many of the aerators are likely approaching the end of their useful life so the Town should budget for replacement of several of them in the coming years. The regulatory changes coming in 2027 could have a very significant impact on treatment requirements and may mandate that the Town construct a mechanical plant. Before the Town makes any significant investment in the existing facilities, it is recommended that the Town work with CDPHE to determine what effluent limits are likely to result from the adoption of stringent nutrient stream standards expected in 2027 timeframe.

Revisions: JULY 2000, RECORD DRAWINGS

TOWN OF RIDGWAY
WASTEWATER TREATMENT PLANT

PROCESS PIPE, UTILITIES & STRUCTURE LAYOUT

Consolidated Consulting Services
P.O. Box 738 Delta, CO 81416

Drawn By	RON ALEXANDER	Date	MAY 1999
Approved By	JOANNE FAGAN	Dwg No	C:\R_\WWTP\SITE.DWG

Drawing No 5

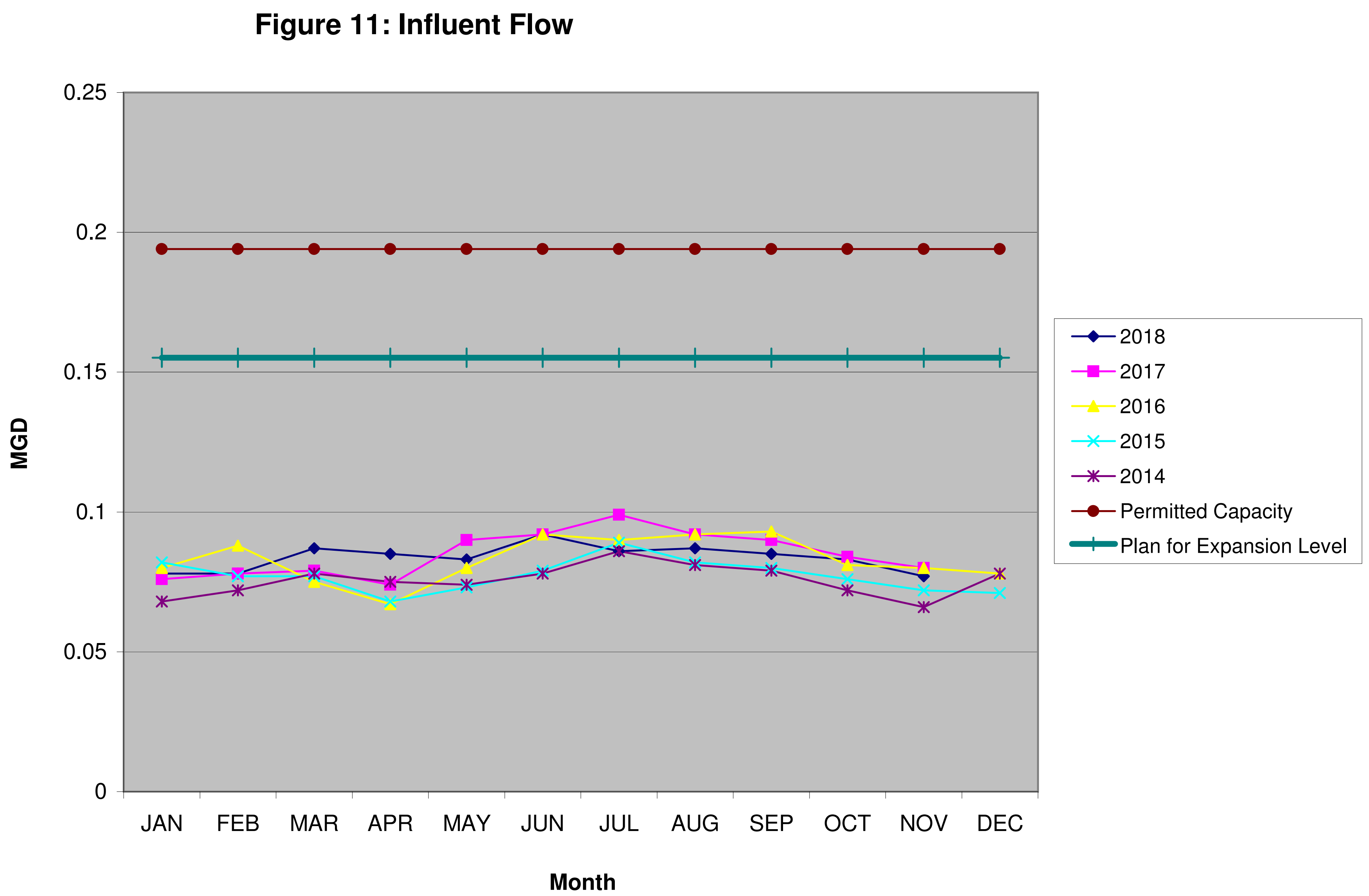
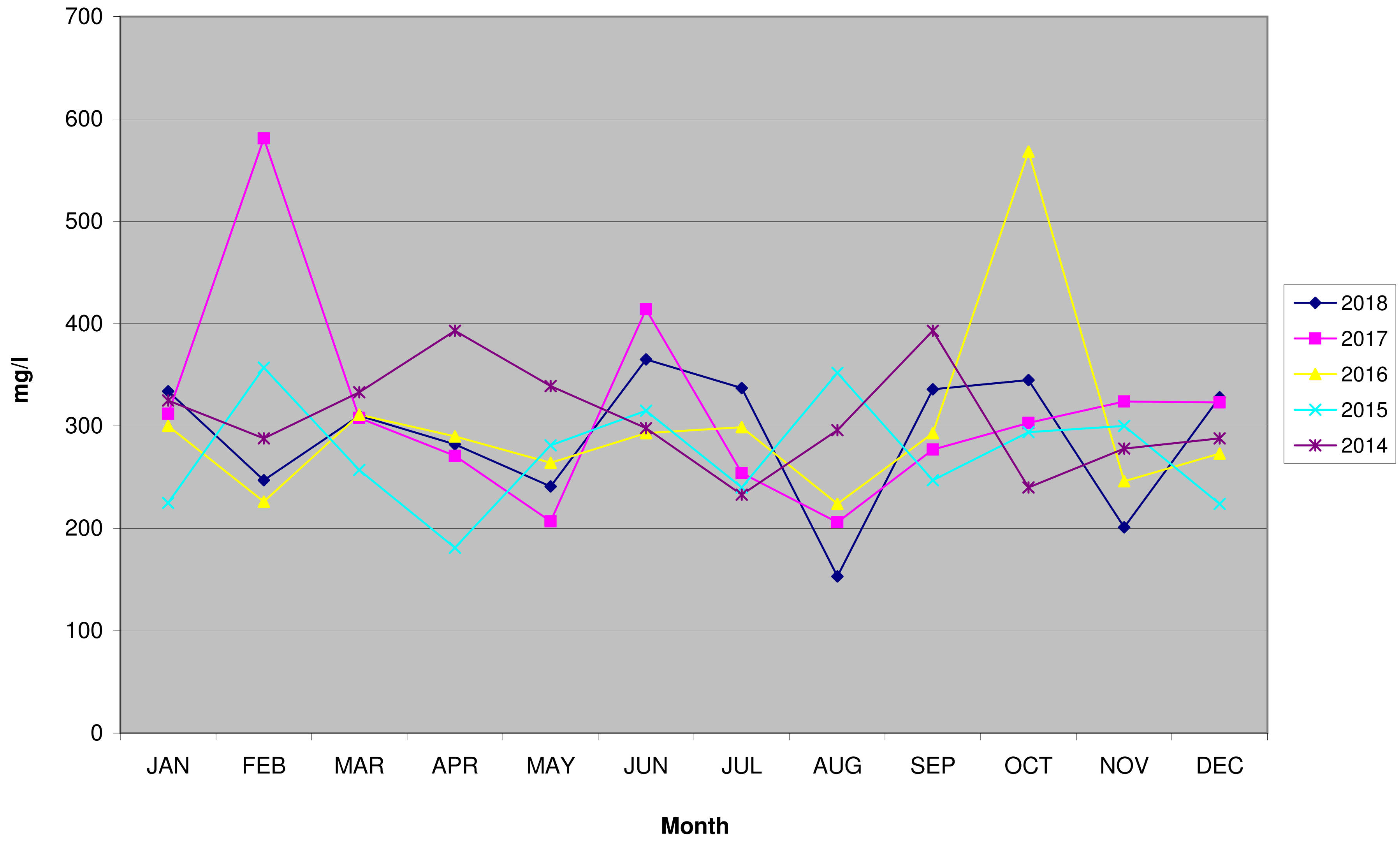


Figure 11: Influent Flow
MGD
0.25
0.2
0.15
0.1
0.05
0
JAN
FEB
MAR
APR
MAY
JUN
JUL
AUG
SEP
OCT
NOV
DEC
Month
2018
2017
2016
2015
2014
Permitted Capacity
Plan for Expansion Level

Figure WW 12a: Influent BOD Concentration



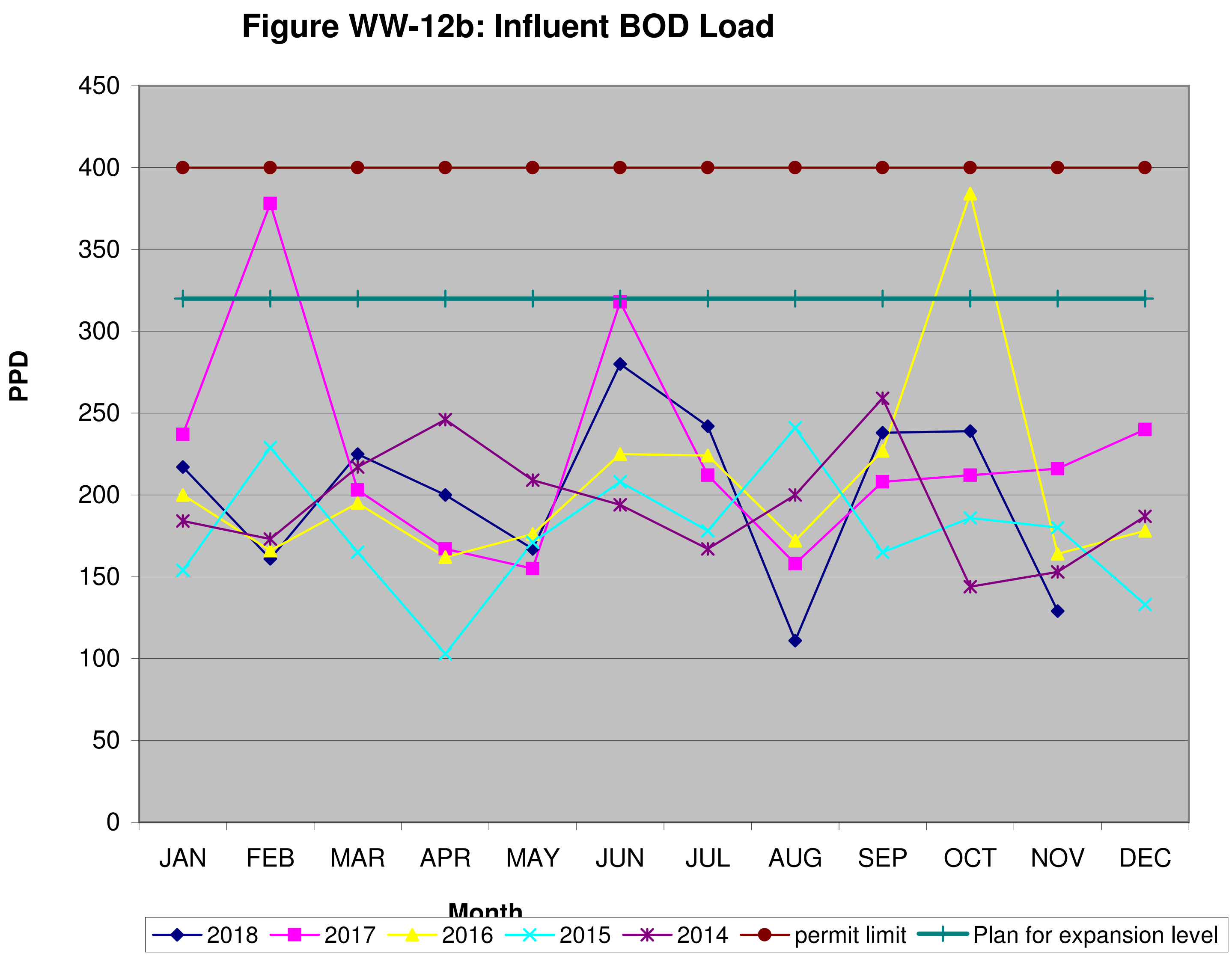
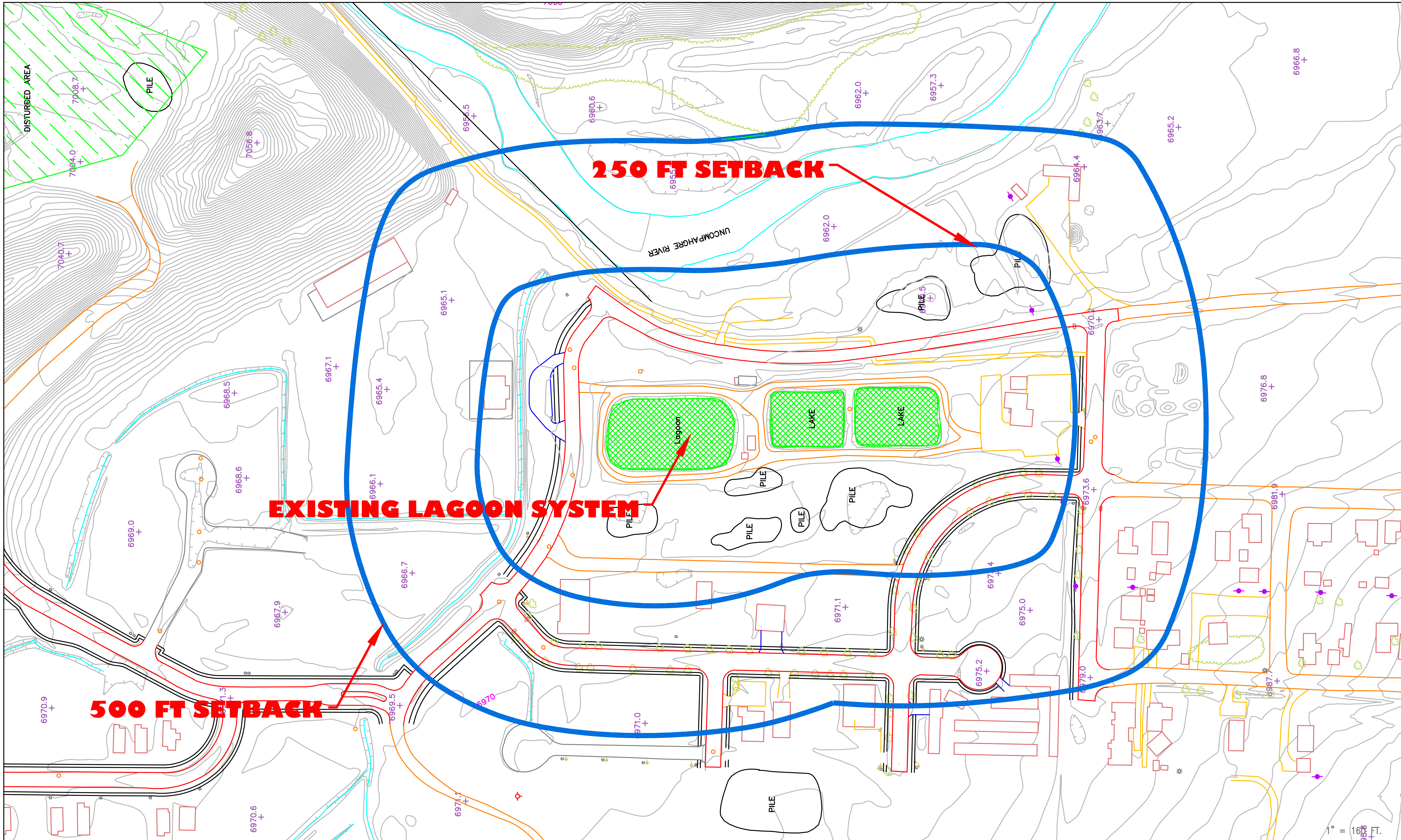


Figure WW-12b: Influent BOD Load
PPD
450
400
350
300
250
200
150
100
50
0
JAN
FEB
MAR
APR
MAY
JUN
JUL
AUG
SEP
OCT
NOV
DEC
Month
2018
2017
2016
2015
2014
permit limit
Plan for expansion level



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Last Modification Date:	12/28/18	Initials:	RA
Full Path:	C:\Ridg\cip\R_cip wwtp		
Drawing File Name:	R_cip ww ds topo		
Acad Ver.: R2000	Scale: NONE	Units:	English

Sheet Revisions

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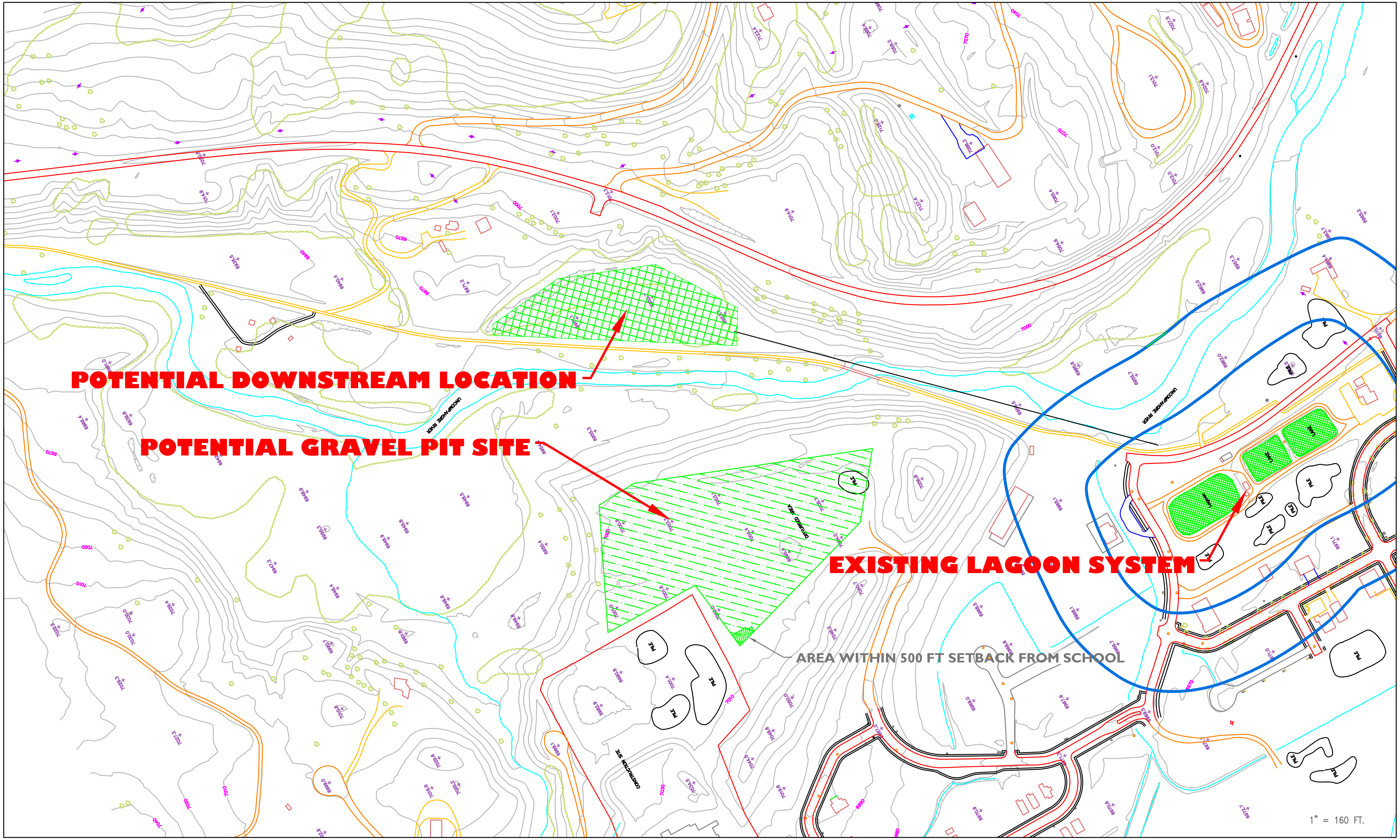
As Constructed
No Revisions:
Revised:
Void:

Ridgway Capital Assessment
Wastewater Treatment Plant Setbacks
Designer: jf
Sheet Subset:
Subset Sheets: 2 of 4

Project No./Code
XXX XXXX-XXX
XXXXX
Sheet # Fig-WW-13



Computer File Information			
Creation Date:	4/4/18	Initials:	RA
Last Modification Date:	12/28/18	Initials:	RA
Full Path:	C:\Ridg\cip\R_cip wwtp		
Drawing File Name:	R_cip ww ds topo		
Acad Ver.: R2000	Scale: NONE	Units:	English

Sheet Revisions

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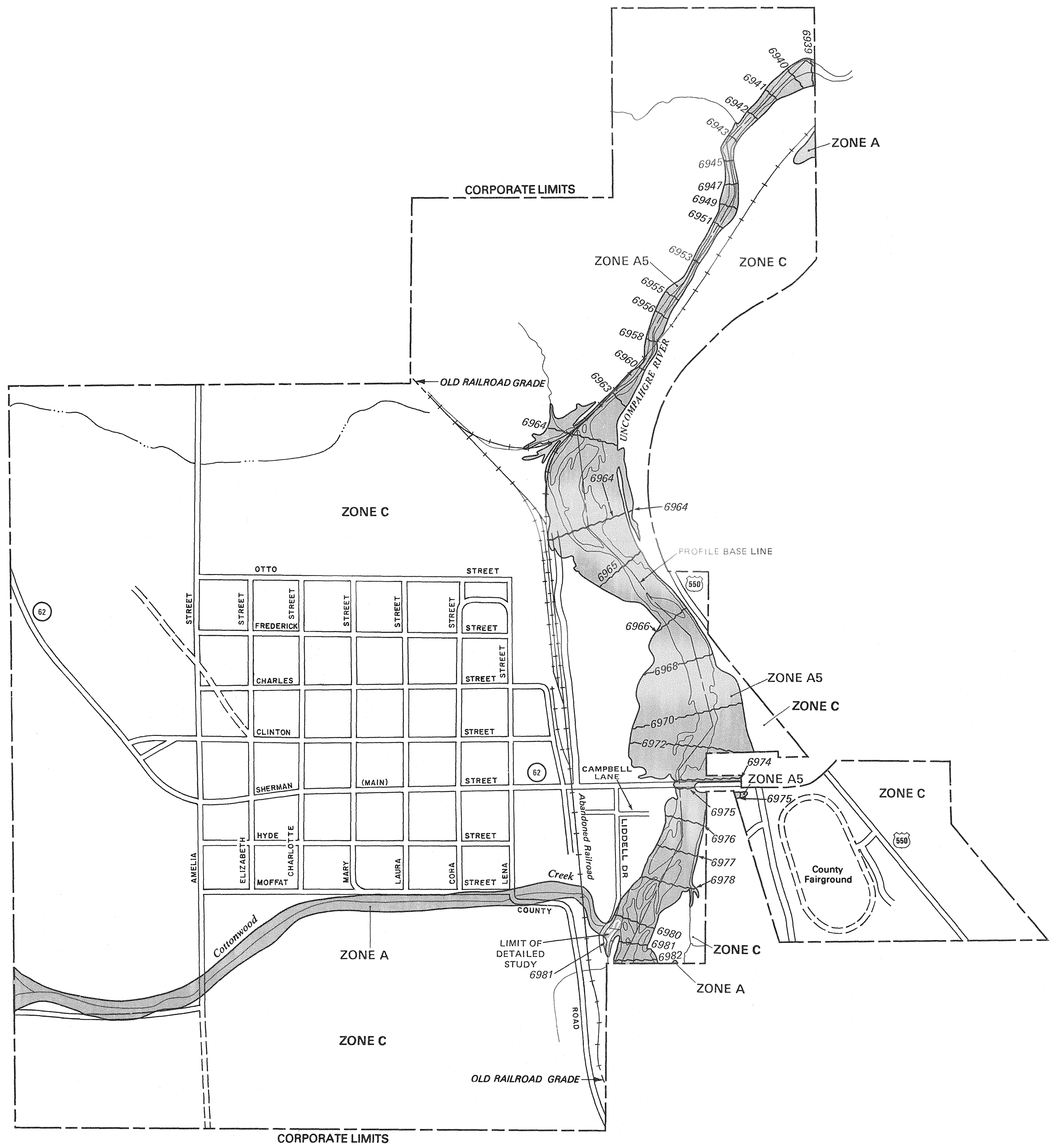
As Constructed
No Revisions:
Revised:
Void:

Ridgway Capital Assessment
Wastewater Treatment Plant Downstream Options
Designer: RA
Sheet Subset:
Subset Sheets: 2 of 4

Project No./Code
XXX XXXX-XXX
5' contours
Sheet # Fig-WW-14

FIGURE WW-15: FEMA Floodplain



KEY TO MAP

500-Year Flood Boundary	ZONE B
100-Year Flood Boundary	ZONE A1
Zone Designations	ZONE A5
100-Year Flood Boundary	ZONE B
500-Year Flood Boundary	
Base Flood Elevation Line With Elevation In Feet**	~~~~513~~~~
Base Flood Elevation in Feet Where Uniform Within Zone**	(EL 987)
Elevation Reference Mark	RM7×
Zone D Boundary	— — —
River Mile	•M1.5

**Referenced to the National Geodetic Vertical Datum of 1929

EXPLANATION OF ZONE DESIGNATIONS

ZONE	EXPLANATION
A	Areas of 100-year flood; base flood elevations and flood hazard factors not determined.
A0	Areas of 100-year shallow flooding where depths are between one (1) and three (3) feet; average depths of inundation are shown, but no flood hazard factors are determined.
AH	Areas of 100-year shallow flooding where depths are between one (1) and three (3) feet; base flood elevations are shown, but no flood hazard factors are determined.
A1-A30	Areas of 100-year flood; base flood elevations and flood hazard factors determined.
A99	Areas of 100-year flood to be protected by flood protection system under construction; base flood elevations and flood hazard factors not determined.
B	Areas between limits of the 100-year flood and 500-year flood; or certain areas subject to 100-year flooding with average depths less than one (1) foot or where the contributing drainage area is less than one square mile; or areas protected by levees from the base flood. (Medium shading)
C	Areas of minimal flooding. (No shading)
D	Areas of undetermined, but possible, flood hazards.
V	Areas of 100-year coastal flood with velocity (wave action); base flood elevations and flood hazard factors not determined.
V1-V30	Areas of 100-year coastal flood with velocity (wave action); base flood elevations and flood hazard factors determined.

NOTES TO USER

Certain areas not in the special flood hazard areas (zones A and V) may be protected by flood control structures.

This map is for flood insurance purposes only; it does not necessarily show all areas subject to flooding in the community or all planimetric features outside special flood hazard areas.

INITIAL IDENTIFICATION:
NOVEMBER 8, 1974

FLOOD HAZARD BOUNDARY MAP REVISIONS:
JANUARY 23, 1976

FLOOD INSURANCE RATE MAP EFFECTIVE:
MARCH 18, 1977

FLOOD INSURANCE RATE MAP REVISIONS:
Map revised October 13, 1981 to change corporate limits, to add new special flood hazard areas, streets names, and streets.

Map revised September 27, 1985 to change flood plain boundaries, zone designations, base flood elevations, corporate limits, scale, cultural features, or map format.

To determine if flood insurance is available in this community, contact your insurance agent, or call the National Flood Insurance Program, at (800) 638-6620.

N

APPROXIMATE SCALE

600 0 600 FEET

NATIONAL FLOOD INSURANCE PROGRAM

FIRM
FLOOD INSURANCE RATE MAP

TOWN OF
RIDGWAY,
COLORADO
OURAY COUNTY

ONLY PANEL PRINTED

COMMUNITY-PANEL NUMBER
080138 0001 D

MAP REVISED:
SEPTEMBER 27, 1985

Federal Emergency Management Agency

Figure WW-16 Google Earth Image of Areas downstream existing treatment plant

Table WW-10

CONDUIT/CONDUCTOR SCHEDULE

CONDUCTORS SIZED PER TABLE 430–72B NEC–99

DESIGNATION	ORIGINATION	TO	CONDUIT	CONDUCTOR	LOAD	APPROX CONDUIT LENGTH	APPROX LOAD LENGTH (ft)
UTIL–1	POWER SOURCE	TRANSFORMER	6" PVC	PULL CORD (WIRE BY SMPA)		410'	
UTIL–2	TRANSFORMER	MAIN DISCONNECT	4" PVC	8 – #3/0, #3 GND	200 A	24'	30
UTIL–3	MAIN DISCONNECT	TRANSFER SWITCH	4" GRC	8 – #3/0, #3 GND	200 A	8"	3
MCC–1	TRANSFER SWITCH	MCC	4" GRC	8 – #3/0, #3 GND	200 A	11"	5
GEN–1	GENERATOR	TRANSFER SWITCH	2" PVC	PULL CORD		17'	
GEN–2	TRANSFER SWITCH	GENERATOR	3/4" PVC	PULL CORD		16'	
CV–3	TRANSFER SWITCH	MCC	3/4" EMT	4 – #12		12"	
CV–4	PANEL A	TRANSFER SWITCH	3/4" EMT	2 – #12, #12 GND		16"	
GEN–3	GENERATOR	AUTO DIALER	3/4" PVC	PULL CORD		28'	
GEN–4	PANEL A	GEN HEAT, CHG.	1" PVC	PULL CORD		18'	
CP–1	MCC	TRANSFORMER	1 1/4" GRC	3 – #4, #10 GND	40 A	4"	3
CP–2	TRANSFORMER	PANEL A	1 1/4" GRC	3 – #4, #10 GND	80 A	40"	6
CL2–1	PANEL A	CL2 PANEL	1 1/2" PVC	3 – #4, #6 GND	60 A	215'	219
MP–1A	MCC	MP–1A	1 1/4" PVC	3 – #8, #10 GND	16 A	332'	450
MP–1B1	MCC	MP–1B1	1 1/4" PVC	3 – #8, #10 GND	16 A	261'	346
MP–1B2	MCC	MP–1B2	1 1/4" PVC	3 – #8, #10 GND	16 A	261'	410
MP–1C	MCC	MP–1C	1 1/4" PVC	3 – #8, #10 GND	16 A	218	370
MP–2A	MCC	MP–2A	1 1/2" PVC	PULL CORD		200'	350
MP–2B	MCC	MP–2B	1" PVC	3 – #10, #10 GND	11 A	150'	245
MP–2C	MCC	MP–2C	1" PVC	3 – #10, #10 GND	11 A	100'	220
MP–2D	MCC	MP–2D	1" PVC	3 – #10, #10 GND	11 A	50'	160
MP–3A	MCC	MP–3A	1" PVC	PULL CORD		107'	
MP–3B1	MCC	MP–3B1	1" PVC	PULL CORD		154'	
MP–3B2	MCC	MP–3B2	1" PVC	PULL CORD		154'	
MP–3C	MCC	MP–3C	1 1/4" PVC	PULL CORD		245'	
MP–4A	MCC	MP–4A	1" PVC	3 – #10, #10 GND	16 A	82'	145
MP–4B1	MCC	MP–4B1	1 1/4" PVC	3 – #8, #10 GND	16 A	173'	325
MP–4B2	MCC	MP–4B2	1 1/4" PVC	3 – #8, #10 GND	16 A	173'	195
MP–4C	MCC	MP–4C	1 1/4" PVC	3 – #8, #10 GND	16 A	254'	400
DW–1	MCC	DEWATER SUMP	1 1/2" PVC	3 – #6, #10 GND	22 A	378'	393
RC–1	CL2 PANEL	RECIRC CONTROL	3/4" PVC	2 – #10, #12 GRD	15 A	24'	32
FR–1	FLOW RECORDER	FLOW SENSOR	3/4" PVC	SIGNAL (PROVIDED BY OTHERS)		22'	
SP–1	MCC	OUTSIDE BLDG	1 1/4" PVC	PULL CORD		9'	
SP–2	MCC	OUTSIDE BLDG	1 1/4" PVC	PULL CORD		9'	
SP–3	MCC	OUTSIDE BLDG	1 1/4" PVC	PULL CORD		9'	
SP–4	MCC	OUTSIDE BLDG	2" PVC	PULL CORD		9'	
SP–5	MCC	OUTSIDE BLDG	2" PVC	PULL CORD		9'	
RC–2	RECIRC CONTROL	RECIRC SUMP	1 1/2" GRC	PROVIDED BY OTHERS		5'	

PLACE 3/8" NYLON PULL CORD IN EMPTY CONDUITS
TIE END TO MOORING POST & LEAVE 18" MIN END IN MCC

Table WW-11 Discharge Monitoring Results 2014-2018

RIDGWAY DMR	LIMIT	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	Avg
2018														
Influent Flow Ave	0.194	0.078	0.078	0.087	0.085	0.083	0.092	0.086	0.087	0.085	0.083	0.077		0.084
Max		0.084	0.091	0.1	0.092	0.09	0.1	0.091	0.1	0.1	0.1	0.085		
% Design Flow Capacity		40%	40%	45%	44%	43%	47%	44%	45%	44%	43%	40%	0%	
Effluent Flow Ave		0.088	0.082	0.081	0.082	0.078	0.09	0.09	0.088	0.088	0.087	0.073		0.084
Max		0.09	0.095	0.09	0.099	0.085	0.095	0.1	0.1	0.099	0.1	0.081		
Influent BOD		334	247	310	282	241	365	337	153	336	345	201	328	289.9
Effluent BOD avg		25	17	5	29	33	24	11	24	19	19	23	21	
Effluent BOD max		25	17	5	36	36.6	24	11	45	19	19	23	21	
BOD % Removal		93	93	98	87	86	93	97	85	94	94	88	94	
BOD ave inf. ppd	400	217	161	225	200	167	280	242	111	238	239	129		
% Design BOD Capacity		54%	40%	56%	50%	42%	70%	61%	28%	60%	60%	32%		50.2%
Influent TSS		241	354	272	298	173	276	253	164	267	381	258	295	
Effluent TSS		35	22	12	32	33	12	16	52	35	40	21	23	
Ecoli		1	150	40	253	210	1248	115	257	1927	960	952	1788	
TRC Max		0.35	0.3	0.3	0.3	0.3	0.21	0.29	0.25	0.21	0.22	0.38	0.35	
pH min		7.8	7.7	7.8	7.8	7.7	7.8	7.8	7.9	7.8	7.7	7.8	7.9	
pH max		8.2	8	8.1	8.1	8	8	8.1	8.2	8.1	8	8.1	8	
Influent NH3		47	33	49	53	51	48	49	33	52	54	50	44	46.92
Effluent NH3		35	38	43	37	18	1	1	6	0.08	0.08	1.1	26	
Eff NH3 Persigo					38.1		<0.2		0.661		<0.2		18.4	
NO3+NO2					1.27		45.8		35.9		33.4		17.1	
Kjeldahl N					44.9		7.86		7.48		5.6		25.5	
Kjeldahl P					6.45		8.18		8.57		7.17		6.31	
TDS raw				202			160			126			204	
TDS Eff				464			710			504			478	

RIDGWAY DMR	LIMIT	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	Avg
2017														
Influent Flow Ave	0.194	0.076	0.078	0.079	0.074	0.09	0.092	0.099	0.092	0.09	0.084	0.08	0.089	0.0853
Max		0.1	0.095	0.11	0.1	0.1	0.11	0.1	0.11	0.1	0.11	0.089	0.097	
% Design Flow Capacity		39%	40%	41%	38%	46%	47%	51%	47%	46%	43%	41%	46%	
Effluent Flow Ave		0.085	0.08	0.078	0.078	0.087	0.094	0.1	0.095	0.092	0.087	0.09	0.094	
Max		0.099	0.1	0.088	0.1	0.094	0.1	0.11	0.1	0.1	0.09	0.096	0.1	
Influent BOD		312	581	308	271	207	414	254	206	277	303	324	323	315
Effluent BOD avg		18	15	28	82	19	19	15	17	15	22	26	21	
Effluent BOD max		18	15	28	132.6	19	19	15	17	15	22	26	21	
BOD % Removal		94	98	91	70	91	95	94	92	95	93	92	94	
BOD ave inf. ppd	400	237	378	203	167	155	318	212	158	208	212	216	240	
% Design BOD Capacity		59%	95%	51%	42%	39%	80%	53%	40%	52%	53%	54%	60%	56.3%
Influent TSS		278	291	333	230	231	237	357	239	244	263	261	241	
Effluent TSS		44	28	42	26	18	22	11	22	7	17	18	25	
Ecoli		8	4	129	225	1045	236	1393	560	896	1919	1190	1940	
TRC Max		0.34	0.3	0.24	0.26	0.23	0.28	0.2	0.2	0.2	0.21	0.2	0.16	
pH min		7.8	7.9	7.9	7.8	7.9	7.6	7.4	7.6	7.9	8	7.9	7.6	
pH max		8.1	8.2	8.2	8.2	8.3	7.8	7.8	8	8.1	8.3	8.1	7.9	
Influent NH3		44	36	46	35	47	36	61	48	44	49	55	46	
Effluent NH3		35	37	41	29	0.11	0.53	0.44	0.49	0.02	1	12	27	
Eff NH3 Persigo		28.5	37.6	36.9		<0.2		0.42		<0.2		8.44		
NO3+NO2		6.64	1.77	1.22		36.5		45.8		34.6		19.8		
Kjeldahl N		36.7	44.6	41.3		7.44		5.68		5.33		14.7		
Kjeldahl P		5.97	5.73	5.74		6.83		7.75		6.78		6.09		
TDS raw				202			190			160			206	
TDS Eff				552			746			608			488	

RIDGWAY DMR	LIMIT	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	Avg
2016														
Influent Flow Ave	0.194	0.08	0.088	0.075	0.067	0.08	0.092	0.09	0.092	0.093	0.081	0.08	0.078	0.083
Max		0.089	0.1	0.086	0.08	0.091	0.1	0.1	0.1	0.12	0.088	0.088	0.085	
% Design Flow Capacity		41%	45%	39%	35%	41%	47%	46%	47%	48%	42%	41%	40%	
Effluent Flow Ave		0.082	0.093	0.079	0.074	0.082	0.1	0.094	0.085	0.095	0.084	0.086	0.082	
Max		0.093	0.1	0.1	0.09	0.1	0.12	0.12	0.1	0.11	0.088	0.09	0.085	
Influent BOD		300	226	311	290	264	293	299	224	293	568	246	273	298.92
Effluent BOD avg		28	18	14	8	29.5	21	12	15	28	42.7	26	30	
Effluent BOD max		28	18	14	8	31	21	12	15	28	42.7	26	30	
BOD % Removal		93	92	95	97	89	94	96	94	90	92	89	89	
BOD ave inf. ppd	400	200	166	195	162	176	225	224	172	227	384	164	178	
% Design BOD Capacity		50%	42%	49%	41%	44%	56%	56%	43%	57%	96%	41%	45%	51.5%
Influent TSS		167	218	243	332	352	255	341	201	287	374	216	243	
Effluent TSS		29	26	18	15	12	13	18	19	17	34	31	39	
Ecoli		1804	119	208	28	43	70	362	181	1208	1137	1880	20	
TRC Max		0.25	0.22	0.25	0.26	0.2	0.21	0.21	0.28	0.22	0.25	0.28	0.25	
pH min		7.9	7.9	7.8	7.8	7.9	7.8	7.8	7.9	7.8	7.8	7.7	7.8	
pH max		8.2	8.1	8.3	8.1	8.1	8.2	8.2	8.1	8.2	8	8	8.1	
Influent NH3		43	31	38	37	46	47	44	39	45	45	48	43	
Effluent NH3		37	27	44	41	26	1	0.5	1	1	1	0.07	19	
Eff NH3 Persigo								0.218		<0.1		0.116		
NO3+NO2								39.1		33.3		30.2		
Kjeldahl N								6.13		5.53		6.41		
Kjeldahl P								8.52		7.51		6.08		
TDS raw				212			94			178			212	
TDS Eff				556			692			568			486	

RIDGWAY DMR	LIMIT	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	Avg
2015														
Influent Flow Ave	0.194	0.082	0.077	0.077	0.068	0.073	0.079	0.089	0.082	0.08	0.076	0.072	0.071	0.077
Max		0.094	0.088	0.097	0.074	0.085	0.094	0.1	0.09	0.085	0.088	0.083	0.08	
% Design Flow Capacity		42%	40%	40%	35%	38%	41%	46%	42%	41%	39%	37%	37%	
Effluent Flow Ave		0.089	0.085	0.081	0.073	0.079	0.081	0.094	0.088	0.087	0.081	0.081	0.075	
Max		0.1	0.1	0.091	0.08	0.098	0.089	0.1	0.1	0.099	0.092	0.084	0.081	
Influent BOD		225	357	257	181	281	315	240	352	247	294	300	224	
Effluent BOD avg		30	17	8	28	26	24	23	22	22	22	8	17	
Effluent BOD max		30	17	8	28	40	24	23	22	22	22	8	17	
BOD % Removal		87.0%	95	97	85	86	92	90	94	91	92	98	92	
BOD ave inf. ppd	400	154	229	165	103	171	208	178	241	165	186	180	133	176.08
% Design BOD Capacity		39%	57%	41%	26%	43%	52%	45%	60%	41%	47%	45%	33%	44.0%
Influent TSS		254	266	220	180	245	309	266	327	212	271	336	218	
Effluent TSS		40	27	444	42	38	14	30	27	25	24	31	32	
Ecoli		593	7	7	7	485	583	224	182	243	920	40	16	
TRC Max		0.2	0.22	0.28	0.2	0.18	0.2	0.22	0.21	0.24	0.2	0.25	0.2	
pH min		7.6	7.6	7.8	7.7	7.7	7.8	7.8	7.9	7.8	7.8	7.8	7.8	
pH max		8	8.2	8	7.9	8.1	8.1	8.2	8	8	8.1	8.1	8.1	
Influent NH3		33	37	37	37	43	41	52	63	51	44	41	38	
Effluent NH3		38	42	44	35	1	1	1	1	1	1	7	20	
TDS raw				220			100			84			194	
TDS Eff				444			580			528			468	

RIDGWAY DMR	LIMIT	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	Avg
2014														
Influent Flow Ave	0.194	0.068	0.072	0.078	0.075	0.074	0.078	0.086	0.081	0.079	0.072	0.066	0.078	
Max		0.084	0.088	0.086	0.089	0.1	0.093	0.1	0.09	0.092	0.081	0.086	0.094	
% Design Flow Capacity		35%	37%	40%	39%	38%	40%	44%	42%	41%	37%	34%	40%	
Effluent Flow Ave		0.079	0.082	0.081	0.1	0.08	0.081	0.088	0.085	0.084	0.08	0.073	0.084	
Max		0.1	0.1	0.099	0.15	0.013	0.09	0.091	0.1	0.1	0.096	0.092	0.094	
Influent BOD		325	288	333	393	339	298	233	296	393	240	278	288	
Effluent BOD avg		16	16	7	23	55	39	29	22	23	5	24	12	
Effluent BOD max		16	16	7	23	55	47	35	22	23	5	24	12	
BOD % Removal		95.1%	94.4%	97.9%	94.1%	83.8%	86.9%	87.6%	92.6%	94.1%	97.9%	91.4%	95.8%	
BOD ave inf. ppd	400	184	173	217	246	209	194	167	200	259	144	153	187	
% Design BOD Capacity		46%	43%	54%	62%	52%	49%	42%	50%	65%	36%	38%	47%	48.6%
Influent TSS		380	303	233	412	209	302	227	228	258	244	310	81	
Effluent TSS		19	17	13	27	33	23	22	16	31	15	23	22	
Ecoli		580	734	20	50	22	3	87	33	72	718	116	584	
TRC Max		0.25	0.2	0.32	0.21	0.27	0.25	0.25	0.27	0.25	0.25	0.25	0.25	
pH min		7.8	7.9	8.1	7.9	8.1	8.2	7.9	8	8.1	7.5	7.5	7.9	
pH max		8.1	8.3	8.3	8.1	8.3	8.4	8.2	8.2	8.3	7.7	7.7	8.2	
Influent NH3		32	44	44	44	32	52.6	33	39	34	33	50	33	
Effluent NH3		33	41	41	36	31	3	1	1	1	1	2.6	23	
TDS raw				92			152			130			208	
TDS Eff				400			568			506			498	

Table WW-12: Typical Revenues and Expenses				
	Typ Total Values	Fix %	Fixed	Variable
		NEEDS UPDATED		
BEGINNING SEWER FUND BALANCE				
REVENUES				
Sewer Service Charges	**310,000**	0.85	263500	46,500
Penalty Fees on Sewer Charges	**2,500**	1.00	2500	0
Transfer Fees - sewer	**500**	0.50	250	250
Material/Labor Reimbursement	**2000**	0.75	1500	500
Tap Fees - sewer	**50,000**	0.90	45000	5,000
Other - sewer	**0**	0.50	0	0
Investment Income - Desgn Res	**8,000**	0.75	6000	2,000
TOTAL SEWER FUND REVENUES	**373,000**		318,750	54,250
TOTAL AVAILABLE RESOURCES				
EXPENDITURES				
PERSONNEL				
Sewer Wages	**102,000**	0.92	93840	8,160
Sewer-Seasonal Wages	**3600**	0.80	2880	720
Employer Tax Expense	**7,803**	0.92	7178.76	624
Health Insurance	**20,000**	0.92	18400	1,600
Retirement Fund	**4,080**	0.92	3753.6	326
Workers Compensation Insuran	**4,500**	0.92	4140	360
ADMINISTRATIVE EXPENSE				
Insurance (Property & Casualty)	**7,400**	0.95	7030	370
Workshops & Training	**1,500**	0.95	1425	75
Consulting & Engineering Servic	**7,500**	0.50	3750	3,750
IT Services	**850**	0.80	680	170
Auditing Services	**2,900**	0.85	2465	435
Legal Services	**2,500**	0.85	2125	375
Wellness Program	**1,650**	1.00	1650	0
OFFICE EXPENSE				
Office - misc	**2,500**	0.65	1625	875
Dues & Memberships	**400**	1.00	400	0
Filing Fees/Recording Costs	**100**	0.65	65	35
Office Supplies	**2,000**	0.75	1500	500
Utilities	**45,000**	0.67	30150	14,850
Telephone	**1,600**	0.90	1440	160
Computer	**2,000**	0.90	1800	200
Records Management	**150**	0.90	135	15
Office Equipment - Leases	**500**	0.90	450	50
Office Equipment - Maint & Rep	**250**	0.67	167.5	83
Postage - sewer	**2,200**	0.90	1980	220
GIS Mapping - sewer	**4,000**	0.95	3800	200

THIS Page and next will be updated with addit6ional data and more accurate typical values

OPERATING EXPENSE							
Maintenance & Repairs		**32,000**		0.75	24000	8,000
Supplies & Materials		**10,000**		0.70	7000	3,000
Tools		**1,000**		0.70	700	300
Testing & Permits		**4,600**		1.00	4600	0
Other - sewer		**500**		0.50	250	250
Safety Equipment		**1,600**		0.80	1280	320
Plant Improvements						
Weed Control		**500**		1.00	500	0
VEHICLE EXPENSE						
Gas & Oil		**4,000**		0.75	3000	1,000
Vehicle & Equipment Maint & R		**6,000**		0.75	4500	1,500
DEBT SERVICE						
Equipment Leases - CAT Equipment						
Debt Service - DOLA		**15,915**		1.00	15915	0
CAPITAL OUTLAY						
Office Equipment Purchase		**500**		0.75	375	125
Equipment Purchase		**40,000**		0.75	30000	10,000
Bio-Solid Removal		**10000**				
Retirement & Severance Payout		**5000**		0.90	4500	500
Emergency Reserves		**20000**		0.70	14000	6,000
TOTAL SEWER FUND EXPENDITU		**378,598**			303,450	65,148
Net Income/Loss with Taps Fees		**-5,598**				
Net w/o Tap Fees		**-55,598**				
Net w/o Tap Fees & debt service		**-39,683**				
Net w/o Tap Fees, debt service,		**317**				
Total WW treated (1000's of gal		**31025**				
Cost per thousand total expense	**$**	**12.20**				
Cost per thousand total expense	**$**	**9.76**				
CAPITAL PROJECT - Line Extension RAMP Proj						
TOTAL TRANS. TO CAPITAL PROJECTS						

Table WW-13 Summary of Needs			
Wastewater Treatment Plant			
Description	Priority	Urgency	Est Cost
Catalog all mechanical equipment (in asset management software)	3	2-24 months	Mostly staff time + software
24 Hour Composite Sampler - Add to better measure influent loading, consider sampling more than once per month to get more accurate information on loading	4	2-24 months	$5,000
	2	when it happens	Mostly staff time + Lab costs
Sample influent BOD more than once per month especially when concentrations to see whether the single sample is representative	2	when it happens	Mostly staff time + Lab costs
Dissolved Oxygen Monitoring & Aeration control	2	6-18 months	$ 7,500
Misc WWTP Mechanical Equipment	2	5-10 yrs	$ 50,000
Backup power - replace generator	4	When power is needed at all times	$ 80,000
Portable generator for chlorine metering pump	3	1-2 years	$ 5,000
Review and comment on Discharge permit renewal	1	As soon as received	$ 1,500
Modify Aeration system - if determined to be worth the change before needing to meet more stringent nutrient limits			$ 425,000
Biosolids Removal	2	3-8 yrs	$ 100,000
Begin planning to upgrade the treatment system when loading reaches 80% of approved design capacity (0.194 MGD and 400 ppd of BOD.)	1	+/- 10 years	$ 100,000
Look for a new plant site farther from the Town core	3		$$$$$$$
Look for land where biosolids could be put to beneficial use	3		$$$$$$$
Note: estimates of costs to address the needs in the report are highly conceptual, based on very limited information and should be used as order of magnitude estimates.			