City of Rohnert Park

Emergency Sewer Pond and Wet Well Lining Project
Condition Assessment Report

January 2018

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1. Introduction

The City of Rohnert Park (City) contracted with GHD to assess the condition of critical assets at its J. Rogers Lane facility, including the wet wells, grinder pit, and diversion structure associated with its two pump stations and for the concrete sewer pond. The concrete sewer pond and Pump Station #1 (PS1) were constructed with the original treatment plant in the 1950s at the site. Pump Station #2 (PS2) was constructed several years later in 1992, with the intent to provide operational redundancy for sending all City wastewater to the regional treatment plant (following decommissioning of the City's treatment facility).

All wastewater from the City is conveyed to the J. Rogers Lane facility. A failure of equipment, infrastructure, or lack of reliable operations from a wet well or storage pond could result in a negative environmental impact. Figure 1 provides an overview of the facility and summary of recommended improvements (see Section 3 for a description of the recommended improvements).

PS1 is located in the northern part of the facility. The PS1 wet well has a waterproof lining suspected to consist of a coal tar bitumen coating that has failed. PS2 is located to the south of PS1 and was constructed in 1992. The wet well in PS2 does not have a liner or coating, exposing concrete surfaces to a corrosive atmosphere.

PS2 includes a grinder vault, which contains hydraulically-powered grinders to chop up wastewater solids, upstream from the wet well. The grinder vault in PS2 also does not have a liner or coating, except for the gate equipment, and is also susceptible to concrete damage caused by a corrosive environment from sewer flow.

An unlined concrete diversion structure adjacent to PS1 conveys the sewer flow from the pipeline entering the pump station site and distributes it to PS1 and PS2. The diversion structure contains two cast iron 18" x 18" sluice gates, one each controlling the wastewater flow to PS1 and PS2. The gates have deteriorated over time and show exfoliating corrosion over their surfaces.

1.1 Project Objectives

The primary objective of the project is to upgrade the sewer pond and two pump stations to current industry standards and extend the useful service life for another 20 years. In order to identify necessary improvements at the site, GHD conducted a condition assessment for the following, existing facility components:

- Sewer pond concrete surface and subsurface;
- Sewer pipelines located between the sewer pond and PS2;
- Concrete structure and reinforcement at Pump Station #1 (wet well and diversion structure);
- Concrete structure and reinforcement at Pump Station #2 (wet well and grinder vault); and
- Ventilation and odor control options at Pump Station #2.

The sewer pond is currently used as a siltation basin for vac/con waste, water storage for force main pigging operations, raw sewage storage, and short term emergency overflow from the pump station. The City expressed interest in eliminating the extraneous features - that were used when the pond was part of the WWTP, but are no longer used - from the pond, thus creating a facility focused on the storage of wastewater for various City needs.

The drain system in the middle of the concrete pond has clogged due to buildup of debris and needs to be redesigned. The filter system in the middle of the pond is difficult to access and the drain pipe should be modified to eliminate a low point that can clog. Incorporation of an access road into the pond that is suitable for a backhoe should be considered to improve pond maintenance.

Operation and maintenance access to PS1 is limited to confined space entry, which makes the assessment of the wet well difficult for City staff, as well as maintenance of the catwalk, railings, and other below grade fixtures.

There are no specific operation and maintenance concerns for PS2 and the grinder vault, other than concern over the condition of the concrete surfaces due to the lack of a coating or lining. The ventilation system in the wet well should also be evaluated to bring the ducting above the high water mark and spray zone.

The south sluice gate for the diversion structure has corroded and should be evaluated for either rehabilitation or replacement.

2. Condition Assessment

A condition assessment of the existing infrastructure was conducted in October and November 2016 to identify the general condition of the facility, and to lay the groundwork for improvements. The assessment is based on the review of physical conditions and functionality of the sewer pond, pump stations wet wells, diversion structure, and grinder vault. Assessment included: review of record drawings, discussions with City staff, visual observation, field testing for concrete in the pond and concrete structures, closed circuit television (CCTV) inspection for pipelines, and laboratory tests. GHD and its sub-consultants performed the condition assessment tasks as described herein.

For the purpose of organizing and prioritizing the recommendations, a matrix with categories and criticality ratings was applied for each evaluated functional component. Based on this matrix, the recommendations for each evaluated component are categorized as being related to safety, code/standard, or maintenance deficiencies, or categorized under optimization/rehabilitation. Further, they are prioritized according to the following criticality ratings:

- **Immediate** – should be addressed within the next year.
- **Urgent** – should be addressed between 1 year and 2 years from now.
- **Short-term** – could be addressed between 2 years and 5 years from now.
- **Long-term** – could be addressed more than 5 years from now.

Assignment of a criticality ranking was subjective, including associated timeframes, but based on engineering judgment. In general, functional components that were found to have a safety or code/standard deficiency were ranked as Immediate. A majority of functional or operational components were ranked as Urgent or Short-term. Modifications were recommended based on equipment age, function, or if a component needs to be updated by code. In regards to deficiencies related to health and safety, current EPA and NFPA standards were utilized.

2.1 Sewer Pond

The existing sewer pond was built in 1975 as the sedimentation and equalization pond. It was originally designed for a total effective volume of 0.36 MG. The pond is nine (9) feet deep with 2H:1V side slopes and made from concrete originally lined with 2.5-inch thickness gunite except for the center sump, which consists of 4-inch thick reinforced concrete. When functioning as a sedimentation and equalization pond, wastewater was conveyed from the east to the pond through an 18-inch influent ductile iron pipe (DIP) from PS1. An effluent box located at the southwest corner of the pond transferred the wastewater out of the pond through a 12-inch reinforced concrete pipe (RCP). The pond is also equipped with a 4’x4’ scum removal box at its northeast corner in order to maintain the removal of scum and vegetation. The pond is currently used as a siltation basin for vac/con waste, water storage for force main pigging operations, raw sewage storage, and short term emergency overflow from the pump station.

The City has expressed interest in removing the influent, effluent and scum removal features from the pond, thus creating a facility focused on the storage of wastewater for various City needs. A vehicular access road into the center of the pond is also required to improve the operation and maintenance of the storage pond.

The pond’s existing drain pipe is located between the center of the pond and PS2. According to record drawings and discussion with the City, this drain can clog if fouled with excessive debris. The City has requested that the p-trap design for the existing drain pipe be modified to allow for free flow of wastewater from the pond towards the pump stations. Modification of the drain pipe should also include replacement of the drain pipe within the pond limits where concrete will be removed and replaced.

The condition assessment survey for the pond included the following components:

- Visual assessment performed by GHD;
- Measurement of reinforcing steel-to-soil potential performed by GHD;
- Delamination survey using acoustical sounding techniques performed by GHD;
- Measurement of in-situ concrete compressive strength performed by GHD;
- Hardness test and petrographic examination of concrete cores performed by Tourney Consulting Group LLC and Braun Intertec Corporation; and
- Geotechnical, environmental, and corrosivity evaluations of soil performed by Miller Pacific Engineering Group, Analytical Sciences, and Cooper Testing Laboratory.

From a geotechnical standpoint, the pond is located on relatively flat ground. Geologic mapping indicates the site is underlain by older alluvial fan deposits, which consists primarily of clay with interbedded layers of coarser-grained alluvium. Seasonal groundwater levels vary and water may be very near or even above the pond bottom during particularly wet periods.

Visual assessment confirmed that the concrete sewer pond has gradually degraded over time and shows fine to moderate cracking and several locations of exposed reinforcing steel. The presence of vegetation and plant growth in various locations confirmed that cracks likely extend through the concrete matrix.

Four concrete core samples – C-1 through C-4 – were taken through the existing concrete liner and subsequently three to five feet into the underlying soil layers. Concrete core samples were sent for analytical testing of soil and concrete at the following locations as shown in Figure 1:

- Location 1 about halfway up the slope – at location for potential new vehicular access road;
- Location 2 near toe of the slope – at location for potential new vehicular access road;
- Location 3 north of the pond midpoint;
- Location 4 near scum removal box.

The concrete thickness at all locations was measured to be more than 2.0 inches, which is generally consistent with the design plans that note 2.5-inch thick concrete. Subsurface conditions below the concrete liner generally included moist to wet, soft, high plasticity clayey and clayey sand soils. No free water was observed in the borings, but they were backfilled soon after excavation and a stabilized depth to groundwater may not have been observed. It should be noted that seasonal groundwater levels will vary and water may be very near or even above the pond bottom during particularly wet periods.

The following environmental laboratory testing was performed on Core Samples C-1 and C-3:

- Total Petroleum Hydrocarbons Diesel and Motor Oil by EPA Method 801B;
- Total Petrolum Gydrocarbones Gasoline by EPA Method 8015B;
- Mercury by EPA Metohd 7471A;
- CAM 17 Metals by EPA Method 6010B.

There were no detectible hydrocarbons (diesel, motor oil, or gasoline) and low levels of metals were encountered in the samples including Barium, Chromium, Cobalt, Copper, Lead, Mercury, Nickel, Vanadium and Zinc according to EPA standards.

It should be noted that the sampling process was not performed under “normal” environmental protocols because of the time of the requested tests by the City. Therefore, sampling equipment was not “cleaned”

for environmental testing. However, since there was no measurable contamination, inadequate cleaning of the sampling equipment or unsuitable storage of material did not result in a “false positive” in the tests. It could be possible that storing samples in plastic bags might allow volatile hydrocarbons to dissipate prior to laboratory testing resulting in the non-detectible lab levels, but the short duration of keeping the samples within the plastic bags eliminates that concern. No hydrocarbon odors were noted in the samples as they were collected.

Reinforcing steel potential was measured in four (4) locations where reinforcing steel strands were exposed. Measurements were taken to assess the potential corrosion of the reinforcing steel in concrete. Per ASTM C876 “Standard Test Method for Corrosion Potentials of Uncoated Reinforcing Steel in Concrete”, reinforcing steel potentials (with respect to the portable reference electrode) more negative than -350 mV indicates a probability of greater than 90% of ongoing corrosion activity. Potentials more negative than -350 mV were measured in locations 1, 2 and 4. Average correlated compressive strength was measured to be more than 3,000 psi at locations 1, 2 and 3. The delamination survey did not reveal significant areas of delamination of concrete from reinforcing steel.

Initial microscopic analysis did not show any problematic issues in the concrete samples; therefore, a detailed microscopic analysis including petrographic analysis was completed on specimens from Cores C-2 and C-4 as well as hardness testing on specimens from Cores C-1 and C-3. Hardness testing was conducted on a polished faces of specimens and at three regions through the depths of the specimens. Results from Mohs Scale of Hardness testing indicated that the surface hardness was at 6.5 and the hardness of the other three depth regions were at 7.5. Therefore, the concrete samples were rather hard in depth with a slightly lower hardness near the surface.

Petrographic examination indicated that the top ends of concrete samples exhibit similar degradation, deterioration, and loss of the original surface, consistent with field reports. Existing depths of degraded and softened cement paste – initial composition of cement, water and aggregates - are generally less than 3 mm (0.12 in.), below which the cement paste become significantly harder with increased depth. It should be further noted that the full depth of concrete in both core locations (C-2 and C-4) has undergone significant alteration, including reduction of pH at and near the top and bottom sides, and significant leaching of calcium from the cement paste without visible evidence of physical distress. No coarse aggregate, residual fly ash, or other supplementary cementitious materials were observed in the concrete samples which makes the concrete more a grout-like material.

Both corrosivity and resistivity tests indicated that the analysed soil is considered to be moderate to highly corrosive. However, the pH of the boring samples tested ranged from 7.4 to 7.6, which are correlated with negligible corrosivity.

Detailed condition assessments for the sewer pond including detailed procedures, test methods, analytical data, and results are described in Appendices A-1 and A-2 and summarized in Table 2.1.

Table 2.1 Sewer Pond Assessment

Component Deficiency	Criticality
__ Safety	__ (A) Immediate
__ Code / Standard	__ (B) Urgent
✓ Maintenance	✓ (C) Short term
✓ Optimization/Rehabilitation	__ (D) Long term / As-needed

Based on the results of the condition assessment, it is concluded that the relatively poor performance of the existing concrete pond is due to the very thin concrete layer and small gauge reinforcing steel. The soft and saturated subgrade conditions, including potentially expansive soils, are a contributing factor and could be a more significant factor in poor performance if heavy equipment needed to operate in the pond interior.

It should be noted that the soil borings were only a few feet deep and they didn't get to solid earth below the pond. Conducting deeper soil borings for the potential pond access road would provide more comprehensive information for design.

2.2 Sewer Pipes

Miksis Services Inc. (Miksis) conducted CCTV inspection for the four (4) pipelines on site as shown in Figure 1, to identify any defects in the existing onsite conveyance system. Manholes were not inspected. The pipelines connect the sewer pond to the manhole located immediately upstream of PS2. However, there is a blind junction point in the pipe alignment located west of the sewer pond, in the vicinity of the operations building. The condition of the pipeline between the sewer pond and this junction point is unknown, though a portion of that pipeline is recommended to be replaced along with other improvements for the sewer pond. CCTV inspection also included the 18-inch pipeline located south of “Manhole 2” that once functioned as the basin return line from the Wastewater Detention Basin #1 of the former treatment plant. The pipeline extends off of the City’s property to the south. See Section 3 for more information.

GHD received and reviewed the CCTV videos and video logs prepared by Miksis to evaluate the existing condition of the sewer lines. The CCTV videos and logs provided manhole identifications and locations, flow direction, pipe diameter and material, general line condition, and specific information regarding pipe joints, cracks, root intrusion and inflow and infiltration. A comparison of the videos and logs was conducted, comparing visual evidence against known pipe lengths and diameters to assess the existing condition of the sewer and to provide a basis of understanding for rehabilitation and/or replacement of the sewer. It should be noted that the horizontal locations and diameters of the pipelines shown in the topographic survey (Figure 1) are different than what is shown in the CCTV report.

In general, all four pipelines appear to be in good structural condition for DIP installed in 1975. Also, there does not appear to be any deficiencies that would affect hydraulics of the pipes. Detailed CCTV assessments for sewer pipes are included in Appendix B and summarized in Table 2.2. Please note that the CCTV inspection notes the pipeline material to be asbestos cement pipe (ACP). Record drawings and topographic survey note the pipeline material to be DIP.

Table 2.2 Sewer Pipes Assessment

Component Deficiency	Criticality
__ Safety	__ (A) Immediate
__ Code / Standard	__ (B) Urgent
__ Maintenance	__(C) Short term
✓ Optimization/Rehabilitation	✓ (D) Long term / As-needed

2.3 Pump Station #1 (PS1) Wet Well

GHD contracted with V&A Consulting Engineers Inc. (V&A) to perform confined space entry and condition assessment for the wet wells at both pump stations, as well as the diversion structure and grinder vault located upstream of the pump stations. Condition assessment for PS1 included physical evaluation on the

interior concrete surfaces and visual and qualitative evaluation of coatings and other metallic features within the wet well space. Detailed condition assessments for PS1 wet well are described in Appendix C and are summarized in Table 2.3.

Evaluation of the interior concrete surfaces of PS1 wet well indicated that the concrete is generally in good condition (VANDA Level 1), with a shallow depth of degradation over most of the surface area. There was an area of exposed aggregate in the northeast corner of the wet well, near the main 18-inch inflow pipe which showed a slightly greater depth of degradation (VANDA Level 2). Details regarding the VANDA rating scale are provided in Section 2 of the V&A report in Appendix C.

The interior coating appeared to be in fair to good condition below the water line, which was approximately at the springline of the suction bells at the time of the assessment. However, V&A noted that the coating had failed completely above the apparent normal water line.. Also, the level of lead concentration found in the existing coating exceeded 600 ppm, which requires the implementation of workers safety measures per CCR Title 8, §1532.1 for future physical assessment or projects for the lining materials.

Access to the PS1 wet well is poor with respect to confined space entry and rescue, where confined space entry is required for any access to the wet well. In addition to this general condition, V&A found two corroded structural frames inside the wet well that inhibit access and cleaning for the wet well. The I-beam across the ceiling of the wet well and its brackets and hardware were also corroded.

V&A conducted SPR (surface penetration radar) scanning at three locations within the PS1 wet well to measure the depth and scaping of reinforcing bars crossing the scan path. SPR scanning found that the minimum depth of concrete cover over the reinforcing steel at the SPR scan locations was 2.5 inches, which is more than the minimum depth of 2 inches recommended as a guideline for water-retaining structures. It is possible that the thickness of the concrete cover is less at locations that were not scanned.

All four suction bells, flanges, nuts and bolts appeared to be in good condition without any evidence of significant corrosion.

Table 2.3 PS1 Wet Well Assessment

Component Deficiency	Criticality
✓ Safety	__ (A) Immediate
✓ Code / Standard	✓ (B) Urgent
✓ Maintenance	__ (C) Short term
__ Optimization/Rehabilitation	__ (D) Long term / As-needed

2.4 Pump Station #2 (PS2) Wet Well

Evaluation of the interior concrete surfaces of PS2 wet well indicated that the concrete is generally in fair condition (VANDA Level 2), with medium diameter exposed aggregate, bug holes and rock pockets in several locations. Visible areas of efflorescence and cracks were observed on the ceiling and the bottom of the walkway. Efflorescence is a by-product of hydration of concrete and/or CMU grout. The salts remain within the concrete until water is present to carry them out.

The interior surface also appeared to have been overlaid with mortar which was degraded and coming loose where it was present.

It should be noted that the safety chains at the ends of the walkway had corroded links, resulting in not all of the chains being functional.

SPR scanning confirmed that the minimum depth of concrete cover over the reinforcing steel at the SPR scan locations was 2.7 inches, which is more than the minimum depth of 2 inches recommended as a guideline for water-retaining structures. It is possible that the thickness of the concrete cover is less at locations that were not scanned, and also less at locations with noted defects (bug holes, rock pockets, etc.).

All four suction bells, flanges, nuts and bolts appeared to be in good condition without any evidence of significant corrosion.

Detailed condition assessments for PS2 wet well are described in Appendix C and summarized in Table 2.4. Overall, the component deficiency is noted as “Safety” and “Maintenance”, since there do not appear to be any imminent needs for improvements within the wet well. However, the safety chains at the end of the walkway should be addressed immediately.

Table 2.4 PS2 Wet Well Assessment

Component Deficiency	Criticality
✓ Safety	__ (A) Immediate
__ Code / Standard	__ (B) Urgent
✓ Maintenance	✓ (C) Short term
__ Optimization/Rehabilitation	__ (D) Long term / As-needed

2.4.1 Ventilation at PS2 Wet Well

GHD conducted a visual assessment of the supply and exhaust fans for the PS2 wet well. The wet well is served by a supply fan mounted on grade and an exhaust fan located in the roof well. During the visual assessment, the fan located on grade and those fans located in the roof well were in running condition and appeared to have been well maintained and/or repaired. The exhaust fan serving the wet well, EF-1 as noted on available record drawings, has been replaced with a Dayton fan per discussion with the City. It had a collection of water in its discharge area which is most likely condensate from the high humidity of the wet well exhaust air. The supply and exhaust ductwork in the wet well appeared to be in good condition with minor functional issues. The lower supply discharge grille is located directly in the path of the wall washing sprinkler and so is constantly wet. The exhaust duct intake grille has fallen out and the bottom plane of the duct had a layer of water in it which is most likely condensate from the humidity in the room.

The estimated volume of the PS2 wet well with the minimum liquid level in the well is approximately 1,000 cubic feet. Ventilation of 12 ACH for a room of this size is 2,200 CFM. This matches the supply design airflow and the exhaust flow, 2,400 CFM in this case, should be slightly larger than the supply to maintain the negative pressurization. Therefore, the ventilation rate complies with the NFPA 820 recommendations.

Field observation indicated the ventilation equipment and ductwork has reached or is close to reaching the end of its useful life.

2.5 Diversion Structure at PS1

Evaluation of the interior concrete surfaces of the diversion structure at PS1 indicated that the concrete is generally in fair condition with some localized defects such as exposed aggregate and rock pockets (VANDA Level 2).

The diversion structure is open to the air with grating spanning across the opening at grade level. Corrosion is intended to be mitigated by this open-air contact, thus the concrete surface does not have a protective coating.

Corrosion was noted over the two cast iron sluice gates, each controlling the flow into PS1 and PS2. The steel plate plugging the pipe connection from the north was also corroded.

The inflow pipe from the east is a RCP that protrudes into the structure several inches. The pipe exhibited a crack or spall around the circumference of the end of the pipe. Detailed condition assessment for the diversion structure at PS1 is provided in Appendix C and summarized in Table 2.5.

Table 2.5 Diversion Structure at PS1 Assessment

Component Deficiency	Criticality
__ Safety	__ (A) Immediate
__ Code / Standard	__ (B) Urgent
✓ Maintenance	✓ (C) Short term
✓ Optimization/Rehabilitation	__ (D) Long term / As-needed

2.6 Grinder Vault at PS2

Evaluation of the interior concrete surfaces of the grinder vault at PS2 indicated that the concrete is generally in good condition (VANDA Level 1) with some isolated areas of spalling, efflorescence, and cracks. Similar to the diversion structure, the grinder vault does not have a coating or lining, with the intent to utilize the open-air ventilation to mitigate for a potentially corrosive atmosphere.

A more dominant crack was observed in the approximate center of the north wall.

The grating and grating supports appear to be in good condition, except for the nuts and anchor bolts holding the support angles in place, which appeared to be moderately to significantly corroded.

Detailed condition assessment for the grinder vault at PS2 is provided in Appendix C and summarized in Table 2.6. The Safety deficiency noted in the table regards replacement for the nuts and anchor bolts.

Table 2.6 Grinder Vault at PS2 Assessment

Component Deficiency	Criticality
✓ Safety	__ (A) Immediate
__ Code / Standard	✓ (B) Urgent
✓ Maintenance	__ (C) Short term
✓ Optimization/Rehabilitation	__ (D) Long term / As-needed

3. Recommendations

This section summarizes the improvements that are recommended for the sewer pond, pump stations, diversion structure, and grinder vault to extend their useful life, improve safety, increase reliability, and/or incrementally improve performance. The recommended improvements and rehabilitation methods described herein are based on GHD's condition assessments described in Section 2 and are tentative because the results of the City's review may change the character and scope of different components of the project. Recommended improvements are also noted on Figure 1.

3.1 Sewer Pond

Rehabilitation of the concrete sewer pond will minimize the potential for pond leakage or the influence of groundwater into the pond. With pond rehabilitation, extraneous features from the original aeration basin design will be eliminated, thus creating a facility focused on the storage of wastewater for various City needs. Since the existing concrete surfacing is less than 2.5-inches thick and supported by clayey soil and a high, seasonal groundwater table, removal and replacement of the concrete surface is recommended. At a minimum, the concrete should be replaced in areas subject to vehicular loading, which includes the bottom of the pond and the new access road. For economy of scale for earthwork and concrete work, removal of all concrete may be advantageous.

The following improvements are recommended for the sewer pond:

- Remove existing influent pipe, effluent structure, and scum box removal;
- Remove existing thin concrete lining;
- Clear old concrete and vegetation/organic materials;
- Protect existing perimeter sub-drain pipe;
- Excavate any loose soil to expose firm "undisturbed" soils and grade the pond to match existing pond slopes and establish grade for a new access road;
- Scarify the subgrade surface to a depth of 12 inches, with moisture content to 2 percent above the optimum moisture content;
- Compact the subsurface to 90% (95% for vehicle access ramp);
- Provide two layers of geotextile fabric under the new fill at the bottom of the pond;
- Fill the slopes with approved material no steeper than 2.5:1 ratio (40% slope);
- Construct the vehicular access road 12-feet wide with maximum longitudinal slope of 10 percent and cross slope of 2 percent from the southeast corner of the pond at the top of the pond to west side of the pond at the bottom;
- Install a 6-inch thick layer of ¾-inch crushed rock on the bottom of the pond that will be subject to vehicular loading;
- Pour a concrete slab on grade with minimum thickness of four inches and reinforcing with #3 steel bars at 12 inches on center or relatively heavy-gauge welded wire mesh;
- Install trench drain at the vac/con dump site to capture and convey runoff to the pond; and
- Remove and replace the existing drain pipe between the center of the pond and the northern edge of pond improvements (i.e., limit of concrete removal and replacement). The new pipe should be designed to at least match the diameter of the existing pipeline. The remainder of the existing

pipeline between the new pipeline and the portion CCTV inspected by Miksis (see Figure 1) should be investigated once construction has begun and the pipe is exposed. The City may determine that the pipeline should be rehabilitated or otherwise improved at that time, including the addition of a junction structure beyond the valve shown on Figure 1.

3.2 Sewer Pipes

Based on review of CCTV records, all four pipelines appear to be in good condition and there is no immediate concern. However, given that the pipelines are DIP and installed in 1975, cured-in-place pipe (CIPP) lining is recommended to improve system reliability. The CIPP liner should be designed to provide adequate structural integrity that can take forces from groundwater, soil loading, and other pressures such as live loads over the exiting sewer pipe. All the pipes should be cleaned and free of debris and obstruction prior to CIPP lining. The pipe between MH1 and the Headworks MH – outside of grinder vault at PS2 - showed some obstructions, which should be removed immediately.

Bypass pumping will not be considered a significant work since both wet wells can be utilized to handle the backup flows during the CIPP lining.

3.3 Pump Station #1 (PS1) Wet Well

The following improvements are recommended for the PS1 wet well:

- Remove and dispose of the existing coating. The abatement procedure per CCR Title 8, §1532.1 should be followed due to the elevated level of lead concentration;
- Resurface the existing concrete by high-pressure water jetting to remove debris and deteriorated concrete;
- Install protective epoxy (100% solid epoxy) coating on the interior surface of the resurfaced concrete. The coating quantity in the opinion of probable cost includes the wet well walls and ceiling;
- Renew corrosion inhibiting paint on the pump station suction bells;
- Rehabilitate the beam running across the ceilings and frame structures inside the wet well to improve corrosion resistance;
- Improve access to the wet well. Provisions can be made by
    - Adding a rigidly-fixed landing to prevent personnel from falling
    - Constructing an aluminium stairway
    - Removing part of the landing and adding a hatch in the landing; and
- Install a sluice gate inside the wet well on the crossover pipe to isolate PS1 from PS2. Note – only install in one of the two pump stations' wet wells.

3.4 Pump Station #2 (PS2) Wet Well

The following improvements are recommended for the PS2 wet well:

- Repair safety chains at both ends of the catwalk;
- Resurface the existing concrete by high-pressure water jetting to remove debris and deteriorated concrete;

- Install protective epoxy (100% solid epoxy) coating on the interior surface of the resurfaced concrete; and
- Renew corrosion inhibiting paint on the pump station suction bells; and
- Install a sluice gate inside the wet well on the cross over pipe to isolate PS1 from PS2. Note – only install in one of the two pump stations' wet wells.

3.4.1 Ventilation at PS2 Wet Well

The following improvements are recommended for ventilation at the PS2 wet well:

- Install a go/no go warning system for proper ventilation inside the wet well with a visual red/green light at wet well entrance;
- Install CGD (combustion gas detector) and tie it to the go/no go warning system;
- Install fan proving switches and tie them to the go/no go warning system;
- Replace supply and exhaust air ductwork with corrosion resistant duct. New duct will be located above the sprinkler level and include high and low diffusers and grilles to increase air mixing and removal of lighter and heavier than air gases. Duct to be sized for an increased airflow for 20 ACH.
- Replace supply fan with non-sparking unit driven by an explosion resistant motor. Increase airflow to 3,500 CFM;
- Replace exhaust fan with non-sparking unit driven by an explosion resistant motor. Increase airflow to 3,850 CFM; and
- Test and balance the new fans to match recommended air flows.

3.5 Diversion Structure at PS1

The following improvements are recommended for the diversion structure at PS1:

- Replace existing corroded brackets and associated lifting components with 316 stainless steel material for the gates;
- Soda blast the corrosion from the two sluice gates and frames and recoat with a corrosion inhibiting paint; and
- Monitor the interior surface concrete for signs of degradation and consider coating the structure in the future.

3.6 Grinder Vault at PS2

The following improvements are recommended for the PS2 wet well:

- Replace existing corroded anchor bolts and nuts with 316 stainless steel anchor bolts and nuts;
- Seal the north wall vertical crack by grout injection; and
- Monitor the interior surface concrete for signs of degradation and consider coating the structure in the future.

3.7 Additional Recommendations

The following improvements are also recommended to increase the efficiency of operation and maintenance of pump stations as they were assessed by V&A in Appendix C:

- Replace damaged exterior and interior lighting fixtures in PS1 and PS2. Interior light fixtures will be explosion proof wall-mounted fixtures. Exterior lighting will be implemented with industrial-type fluorescent fixtures.
- Replace corroded conduit brackets in both PS1 and PS2 wet wells;
- Replace corroded pipe supports in PS1 and PS2; and
- Renew the coatings on all pipes in PS1 and PS2.

3.8 Collection System Evaluation

GHD has recently prepared a hydraulic evaluation of the sanitary sewer collection system located downstream of the proposed Press Democrat development – APN 143-040-135. The purpose of the evaluation was to compare existing and proposed collection system hydraulics to the City's hydraulic design standards for existing 12-inch pipelines located just west of J. Rogers Lane (the two pipelines immediately upstream of the diversion structure in J. Rogers Lane).

Results from the modeling as presented in Appendix E indicate that changing the last segment of pipeline from SSMH #849 to SSMH #1 from 12-inch to 15-inch is appropriate to serve the proposed Press Democrat development.

Other pipe segments located upstream of SSMH #849 may require modification based on future development plans. Hydraulic assessment and design should be completed for those pipelines along with planned future development.

4. Constructability

4.1 Priority Recommendations

The project lends itself to be completed in more than one phase based on the City's available funding and to maintain full operation of the facility during construction. GHD recommends upgrades to be performed in the order of the criticality groups, addressing items that require immediate attention first, as follows:

1) Rehabilitate the PS1 wet well interior concrete and coating and improve access to the wet well;
2) Replace existing corroded anchor bolts and nuts for grating at the PS2 grinder vault with stainless steel anchor bolts and nuts;
3) Rehabilitate the sewer pond, including the drain pipe within the pond limits;
4) Rehabilitate the PS2 wet well interior concrete and repair the safety chains;
5) Rehabilitate the diversion structure;
6) Rehabilitate the PS2 wet well HVAC system; and
7) Sewer cleaning and CIPP rehabilitation of four evaluated pipelines on site.

Note that although the various components are separated into different criticality categories, there is an advantage to perform the improvements by discipline and/or through larger project scopes in order to take advantage of construction costs associated with economies of scale. Construction sequencing and details on phasing of projects were not taken into consideration at this level, and therefore it is important to note that while an item may require both an "urgent" and "long-term" action, the City should determine whether both actions are necessary.

4.2 Project Schedule and Implementation

Construction of all improvements is anticipated to take eight months to complete, from initial site mobilization and demolition to construction closeout activities. The City plans to bid the project in early summer of 2017, which would lead to construction beginning in fall/winter 2017. Construction is anticipated to occur in one construction season (no seasonal shut downs). Timing of construction during wet weather months should be further evaluated during the design phase given the selected scope of work (i.e., improvements to the pump station wet wells is recommended to occur during dry weather months to minimize the quantity of flow to be handled by each pump station when the other pump station is out of service).

The anticipated project schedule for the bidding and construction phases for the recommended improvements is provided in Table 4.1.

On August 9, 2017, GHD staff met with City staff. It was confirmed that the overall project scope of improvements at the Main Pump Station site is intended to be split into three phases, as follows:

- Phase A: Rehabilitation of the PS#1 wet well and diversion structure, and renewal and replacement of pipelines located upstream of PS #1;
- Phase B: Rehabilitation of the Pump Station 2 (PS#2) wet well and grinder pit;
- Phase C: Rehabilitation of the emergency sewer pond, including associated renewal and replacement of pipelines located between PS#2 and the pond.

The City's current project schedule includes the design and construction of all phases in three consecutive years, from 2018 through 2020.

Table 4.1 Summary of Overall Project Schedule

Project Phase	Timeframe
Submit Final Condition Assessment Report	Early January 2018
Phase A - 50% Design Submittal	Early January 2018
Phase A - City 50% Submittal Review (~2 weeks)	Late January 2018
Phase A - 90% Design Submittal	Mid-February 2018
Phase A - City 90% Submittal Review (~2 weeks)	Late February 2018
Phase A - Final PS&E Submittal	Mid-March 2018
Phase A - Bid Phase	April – May 2018
Phase A - Construction	June – November 2018
Phase B - Design and Construction*	2019
Phase C - Design and Construction*	2020

*May change based on City's future development plans.

5. Estimate of Probable Construction Cost

Construction costs are based on a Class 4 (concept evaluation, predesign) estimate of probable cost as defined by the Association for the Advancement of Cost Engineering, International (AACE). AACE defines the "Class 4" estimate as follows:

> *Generally prepared based on limited information and subsequently have fairly wide accuracy ranges. They are typically used for project screening, determination of feasibility, concept evaluation, and preliminary budget approval. Typically, engineering is from 1% to 15% complete. Class 4 estimates are prepared for a number of purposes, such as but not limited to, detailed strategic planning, business development, project screening at more developed stages, alternative scheme analysis, confirmation of economic and/or technical feasibility, and preliminary budget approval or approval to proceed to the next stage. The typical accuracy range for this class estimate are -15% to -30% on the low side, and +20% to +50% on the high side, depending on the technical complexity of the project, appropriate reference information, and the inclusion of an appropriate contingency determination.*

Construction costs are based on the January 2017 Engineering News Record Construction Cost Index (ENR CCI) for San Francisco, CA (11609.44). Estimates of probable cost are provided in Appendix D and are summarized in Table 5.1 for each of the project components. These Class 4 estimates are appropriate for comparing alternatives and estimating funding requirements for budgeting purposes. Assumptions used for preparation of the costs are also included the appendix. The costs associated with Sections 3.7 and 3.8 are not included in the estimated project costs. The exact breakdown of soft costs for these Sections will be determined with further project planning and development.

Note: Contingency (set at 30-percent) is not directly related to the stated accuracy range for a Class 4 estimate. Determination of construction cost contingency is intended to cover unforeseen aspects of construction, including changes in quantities of work, which have not been completely evaluated during this preliminary investigation.

Project costs include allowances for design phase and construction phase services (i.e., "soft costs"). There is no cost associated with easement or land acquisition and permits.

Table 5.1 Summary of Estimated Project Costs

Reference Section	Project Component	Engineer's Estimate
3.3	Rehabilitate PS1 wet well	$456,000
3.6	Rehabilitate PS2 grinder vault	$49,000
3.1	Rehabilitate sewer pond	$1,440,000
3.4	Rehabilitate PS2 wet well	$470,000
3.6	Rehabilitate PS1 diversion structure	$70,000
3.4.1	Rehabilitate PS2 wet well HVAC system	$114,000
3.2	Rehabilitate sewer pipes	$108,000
	Total	$2,707,000

6. Design Services

Phase A of the project will include the following improvements, as discussed with the City prior to finalizing the Condition Assessment Report:

- Wet Well at PS#1:
  - Replace the existing protective coating on the interior surfaces of wet well;
  - Replace exterior and interior light fixtures;
  - Rehabilitate other features such as pump suction bells and metal frames in the wet well;
  - Replace aluminium grating and ladder with stainless material (note – structural improvements for the catwalk are not included); and
  - Modify piping system as needed and replace exposed portion of the drain pipes with ABS material or similar.
- Diversion Structure:
  - Remove the south gate;
  - Replace the west gate and electrical actuator; and
  - Install protective coatings on the walls to approximately 10-feet above the structure base.
- Sewer Pipelines:
  - Upsize 37 linear feet of existing 12-inch pipe to 15-inch between SSMH #849 and SSMMH#1, located upstream of the diversion structure in J. Rodgers Lane; and
  - Cured-in-Place-Pipe (CIPP) rehabilitation of the influent pipe between the Diversion Structure and the PS#1 wet well.

Figures

Appendices

Appendix A-1 - Evaluation and Preliminary Repair Recommendations Sewer Pond Rehabilitation; Miller Pacific Engineering Group; December 21, 2016

December 21, 2016
File: 1206.179altr.doc

GHD
2235 Mercury Way, Suite 150
Santa Rosa, California, 95407
Attn: Mr. Matt Winkelman, PE

Re: Evaluation and Preliminary Repair Recommendations
Sewer Pond Rehabilitation
201 J. Rogers Lane
Rohnert Park, California

Dear Mr. Winkelman:

Introduction and Project Description

Following our discussions, site evaluation, subsurface exploration and laboratory testing, this letter presents our preliminary evaluation along with laboratory test results and repair recommendations for the existing sewer pond at the City of Rohnert Park's pump station facility at 201 J. Rogers Lane in northwestern Rohnert Park. Our services were performed in accordance with our Sub-Consultant Agreement dated September 27, 2016.

The existing sewer sedimentation and equalization pond was reportedly constructed in the early 1970's and the concrete interior liner is performing poorly with many open cracks and areas of vegetation growth. A new concrete liner is desired by the City, along with repairs and upgrades to existing wet wells, pipes and other features at the site. A new "ramp" into the pond will also likely be constructed to allow for easier cleaning of the pond bottom.

The existing pond is roughly square in plan view with +/- 120 foot long sides and a depth of about 15 feet. The pond was apparently constructed by excavating the middle of the pond and placing fill around the perimeter to form embankments that are about five feet above the original ground surface. The interior pond slopes vary in inclination and are steeper near the top of the slope and "rounded" where they transition to a flat pond bottom. Plans prepared in 1970[1] indicate a 4-inch diameter asphalt pipe subdrain is located below the concrete liner and around the perimeter of the pond bottom. Based on the plans, this subdrain discharges at the northeast corner of the pond.

Geologic Setting

The site is located in northwestern Rohnert Park on relatively flat ground. Geologic mapping[2] indicates the site is underlain by older alluvial fan deposits, which consists primarily of clay with interbedded layers of coarser-grained alluvium.

Site Exploration and Laboratory Testing

During our site exploration on November 8, 2016, we excavated four cores through the existing concrete pond liner and then we subsequently hand-augured three to five feet into the underlying soil layers. One core was excavated in the middle to lower portions of the sloping perimeter walls

[1] Yoder-Trotter-Orlab Associates, "Water Pollution Control Plant Modifications," City of Rohnert Park, Job. No. 1763, Sheet G-.3
[2] Clahan, K.B., et al, 2003, Geologic Map of the Cotati 7.5' Quadrangle, Sonoma County, California: A digital database.: California Geological Survey, Preliminary Geologic Maps, scale 1:24,000.

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and three cores were located in the bottom and middle of the pond bottom as shown on Figure 1. After hand-auguring and sampling, the holes were backfilled and patched with concrete. Concrete thickness at the four explored locations is summarized in Table A and reinforcing seemed to be limited to very light gage wire (1/16 to 1/8-inch diameter) that will have limited tensile capacity. The spacing of the wire mesh was impossible to determine in our cores.

Table A
Concrete Core Thickensses
Rohnert Park Sewer Sedimentation Pond

Location	Concrete Thickness (in.)
Core 1	2.25
Core 2	2.0
Core 3	2.0
Core 4	2.0

Notes: Core thickness rounded to the nearest ¼-inch. Cores were retained by GHD for additional structural analysis, including possible strength testing.

Subsurface conditions below the concrete liner generally included moist to wet, soft, high plasticity clayey and clayey sand soils. Core 1, located approximately halfway up the slope at the southeast corner of the pond, encountered high moisture content, soft, high plasticity clay to the maximum (three foot) explored depth. Cores 2, 3 and 4, generally located on the bottom of the pond, exposed a few inches of coarse, saturated sand fill over saturated, soft, high plasticity, clayey sand/sandy clay to the maximum explored depth of about three feet. No free water was observed in our borings, but they were backfilled soon after excavation and a stabilized depth to groundwater may not have been observed. We note that seasonal groundwater levels will vary and water may be very near or even above the pond bottom during particularly wet periods.

We collected samples of the clayey soils from the four cores and laboratory testing included moisture content, dry density, unconfined compression, percent passing the No. 200 sieve and plasticity index. Laboratory test results are summarized in Table B:

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Table B
Geotechnical Laboratory Test Results
Rohnert Park Sewer Sedimentation Pond

Location/ Depth (ft.)	Moisture Content (%)	Dry Density (pdf)	Plasticity Index	%-passing #200 sieve	Unconfined Strength (psf)
C1; 0.5'	34.5	85			600
C1; 1.0'	28.8	92			2500
C1; 2-3'	30.5				
C2; 0.5-1.5'	39.1			59.8	
C2; 2-3'	30.5			43.1	
C2; 4-5'	23.5				
C3; 1'	26.1	97	36	31.5	1800
C3; 2-3'	27.5				
C4; 2-3'	27.7				

Environmental Laboratory Testing

As an amendment to our original scope of services, we were asked to perform laboratory (environmental) testing on composite samples from our subsurface exploration. This request was made a few hours after we had performed our field work so "normal" environmental sampling protocols, such as cleaning sampling equipment between borings locations and storage of samples in laboratory-supplied (sterile) containers was not followed. As discussed below, since no significant contamination was detected, not following established protocols is not believed to have resulted in cross-contamination or inaccurate results. Composite samples from the clayey soils in our Cores 1 and 3 were tested for the following environmental properties:

- Total Petroleum Hydrocarbons Diesel and Motor Oil by EPA Method 8015B
- Total Petroleum Hydrocarbons Gasoline by EPA Method 8015B
- Mercury by EPA 7471A
- CAM 17 Metals by EPA Method 6010B

Laboratory testing was performed by Analytical Sciences of Petaluma and the complete test results and chain of custody forms are attached in Appendix A. For Cores 1 and 3, there were no detectible hydrocarbons (diesel, motor oil or gasoline). Low levels of Metals were encountered in the samples including Barium, Chromium, Cobalt, Copper, Lead, Mercury, Nickel, Vanadium and Zinc as follows:

- Barium levels ranged from 100 to 120 mg/kg
- Chromium levels ranged from 15 to 31 mg/kg
- Cobalt levels measured at 8.4 mg/kg
- Copper levels ranged from 15 to 17 mg/kg
- Lead levels ranged from 9 to 11 mg/kg
- Mercury levels ranged from non-detectable to 0.20 mg/kg
- Nickel levels ranged from 18 to 29 mg/kg

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- Vanadium levels ranged from 13 to 31 mg/kg
- Zinc levels ranged from 20 to 31 mg/kg

Metals testing also included testing for the presence of Antimony, Arsenic, Beryllium, Cadmium, Molybdenum, Selenium, Silver and Thallium which were non-detectable in either of the samples. As discussed previously, "normal" environmental testing protocols were not followed during exploration or sample collection and the samples were stored for a few hours in "Ziploc" baggies until they were transferred to laboratory-supplied containers late in the day when they were collected.

We note that only what would be considered as "background" levels of metals were measured in our laboratory tests so we judge significant contamination is not expected at the site based on this preliminary testing.

Laboratory Corrosion Testing

Laboratory corrosion testing on select samples was performed by Cooper Testing Laboratory of Palo Alto and the full test results are attached as Appendix B. A summary of the results is included in Table C and we understand GHD will evaluate soil corrosion and its potential effect on site piping, concrete structures and other items at the site:

TABLE C
LABORATORY CORROSION TEST RESULTS
CITY OF ROHNERT PARK
POND REHABILITATION PROJECT
ROHNERT PARK, CALIFORNIA

Boring	Depth (ft)	Resistivity (Ohm-cm)[1] As Received	Resistivity (Ohm-cm)[1] Saturated	Chloride (mg/kg)[2]	Sulfate (mg/kg)[2]	pH[3]
C1	0.0-1.0	729	709	<2	103	7.4
C2	0.5-1.5	1,271	1,271	4	33	7.6
C3	0.5-1.0	1,373	1,337	2	23	7.4
C4	0.5-1.5	1,492	1,431	6	37	7.5

Notes:
(1) Resistivity determined in accordance with ASTM G57
(2) Chlorides and Sulfates determined in accordance with ASTM D4327
(3) pH determined in accordance with ASTM G51

Geotechnical Opinions

We conclude the relatively poor performance of the existing concrete shell is due to the very thin concrete layer and small gauge reinforcing steel. The soft and saturated subgrade conditions, including potentially expansive soils, are a contributing factor and could be a more significant factor in poor performance if heavy equipment needed to operate in the pond interior.

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Provided on-site soils are maintained in a moist to saturated condition throughout the calendar year (by covering with a concrete shell), expansive behavior should be a relatively minor concern at the site and a concrete liner that is only subjected to water loads could be relatively thin. The planned access ramp and areas of the pond interior that will be subjected to heavy equipment loads, and particularly rubber-tire loads, should be carefully designed to reduce risks of poor performance due to underlying soft soils.

Other geologic hazards that could affect the pond are likely limited to strong ground shaking during earthquakes and possible liquefaction and associated settlements in the mapped alluvium. To evaluate the potential for liquefaction damage during strong seismic shaking, deeper auger borings or cone penetration tests would be required. If significant liquefaction risks exist at the site, ground improvement or a thicker concrete liner could be considered to mitigate the hazard.

Geotechnical Recommendations

Based on our discussions, we understand the existing (thin) concrete liner will be removed as part of the pond repairs. Demolition should be relatively easy due to the apparent thin layer of existing concrete and “light” reinforcing. As previously noted, a relatively thin new concrete or shotcrete shell should be appropriate in portions of the pond that will not be subjected to vehicle loads. The planned access ramp and bottom of the pond should be specifically designed for vehicle loads due to the soft underlying clay soils.

We understand the new ramp will likely be constructed by new cuts and/or fill slopes which are thought to be more cost effective than new retaining walls within the pond interior. Preliminary plans suggest the ramp surface will be 10 feet wide and will extend along the western and southern sides of the pond so that equipment can access the pond bottom. Our limited scope of hand augering suggested that soft clayey soils exist throughout the project area to depths exceeding three feet so we recommend additional exploration to determine the final depth that will be required for keyways and benches for new fill slopes.

Preliminary recommendations for pond repairs and improvements follow:

Site Grading Recommendations

Site grading within the pond area will be generally limited to removal of the existing pond liner, and construction of a new fill slope to create the new ramp into the bottom of the pond. Site grading should be performed in accordance with the following recommendations.

1. Surface Preparation – Clear all old concrete and vegetation or organic materials that have accumulated in the slab cracks. Roots and small stumps larger than about two inches in diameter should be removed and the excavated areas should be restored with properly moisture conditioned and compacted fill as described in the following sections. Any loose soil at the exposed subgrade surface will need to be excavated to expose firm “undisturbed” soils or they should be recompacted to at least 90% prior to new concrete placement. Debris, rocks larger than six inches, and vegetation are not suitable for structural fill and should be removed from the site. Alternatively, vegetation strippings may be used in landscape areas.

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The existing perimeter subdrain may offer some resistance to “buoyancy” of a new concrete liner so we generally recommend the subdrain be protected and left in place and possibly improved with new pipe and permeable material if the pond will be emptied during periods of high exterior groundwater.

Where fills or other structural improvements are planned on level ground such as the pond bottom, the subgrade surface should be scarified to a depth of about twelve inches, moisture conditioned to at least 2% above the optimum moisture content, and compacted to a minimum of 90% relative compaction (ASTM D-1557). Relative compaction should be increased to a minimum of 95% at the subgrade elevation where the new access ramp or vehicle-loaded areas are planned. Relative compaction, maximum dry density, and optimum moisture content of fill materials should be determined in accordance with ASTM Test Method D 1557, "Moisture-Density Relations of Soils and Soil-Aggregate Mixtures Using a 10-lb. Rammer and 18-in. Drop." If soft, wet or otherwise unsuitable materials are encountered at the subgrade elevation during construction, we will provide supplemental recommendations and/or field directives to address the specific condition.

2. Excavations – Site excavations for new foundations, underground utilities, and other improvements will generally encounter soft to medium stiff sandy clays. Based on our subsurface exploration, we judge the majority of site excavations can be performed with “traditional” grading equipment, such as small to medium-size dozers, excavators, and backhoes.

   All excavations in excess of 5-feet deep will need to be sloped or braced in accordance with Cal/OSHA regulations. Based on our exploration, onsite soils will be prone to raveling and collapse in open excavations and should be considered “Type C”. The project Contractor should anticipate caving conditions and, where needed, provide a shoring system capable of providing immediate support to the walls of the excavation to reduce the risk of caving and collapse. While groundwater was not encountered during our subsurface exploration, seepage may occur in deeper excavations and may require pumping. Seepage can further de-stabilize vertical excavations, so construction in the later summer or fall should result in more stable conditions.

3. Slopes – Compacted fill slopes and permanent cut slopes in soil should be no steeper than 2.5:1 due to the high-plasticity clays on site. If fill slopes are constructed, and where fill is placed on existing slopes steeper than about 8:1, the slopes should be “keyed” into firm materials and constructed as described below. A schematic detail of hillside fill slope construction is shown on Figure 2. As noted above, the depth of keyway excavations will be deeper than the three feet we generally explored to so for preliminary design, we estimate a keyway depth of five feet will be required. This estimated depth should ideally be confirmed based on future subsurface exploration. Subdrains are not recommended at the bottom of the keyway due to potential high water and subsequent need for continual pumping.

   If firm soils are not encountered at the keyway or benching excavation depth, a geotechnical fabric may be needed to stabilize a yielding subgrade. For preliminary design and cost estimating, we suggest an allowance be provided for two layers of Mirafi 2XT (or approved alternative) in the lower lifts of new (ramp) fills.

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For temporary slopes and excavations, the Federal Occupational Safety and Health Administration (OSHA) has promulgated rules for Excavations. OSHA dictates allowable slope configurations and minimum shoring requirements based on categorized soil types, as discussed above. The Contractor may elect to use a variety of shoring and temporary slope configurations, but his operations must conform to Federal and State OSHA regulations. The safety of excavations, slopes, construction operations, and personnel are the sole responsibility of the Contractor.

4. Fill Materials – Soils generated from on-site excavations could be used for new fills but will likely need to be dried so that the required compaction may be achieved On-site soils will also exhibit some expansive potential so imported select fill would be desirable to improve performance of new fills. Imported fill material should consist of soil and rock mixtures that: (1) are free of organic material, (2) have a Liquid Limit less than 40 and a Plasticity Index of less than 15, and (3) are well-graded with a maximum particle size of four inches.

5. Fill Compaction – Following subgrade preparation in accordance with the above recommendations, on-site fill materials should be conditioned to at least 2% above the optimum moisture content. Imported fill soils should be moisture-conditioned to near optimum. They should then be placed in loose horizontal lifts not exceeding 8-inches in thickness, and subsequently compacted to a minimum of 90% relative compaction. Where vehicle-loaded areas are planned, compaction should be increased to 95% minimum at the subgrade elevation.

Concrete Slabs-on-Grade

For the new pond liner, and in areas not subjected to vehicle loads, we recommend a minimum concrete thickness of four inches and reinforcing with #3 steel bars at perhaps 12 inches on center or relatively heavy-gauge welded wire mesh. Thickening the slab to five inches would improve performance and reduce risks of slab cracking and should be considered if the project budget allows. We judge the new concrete could be placed directly on an approved subgrade as recommended above. A four to six inch thick layer of ¾-inch crushed rock could be placed on the bottom of the pond to further improve performance of the new concrete shell and reduce risks of “buoyancy” if the pond is empty and exterior groundwater levels are high (note the crushed rock would need an “outlet” such as the existing subdrain system).

Where vehicles will operate on the liner surface (except low ground pressures equipment that has apparently historically been used to clean the pond), additional thickness should be planned to reduce risks of cracking and poor performance. For the soft clayey soils at the base of the pond, a new slab can be structurally-designed for the expected vehicle loads based on an 80 pci Modulus of Subgrade Reaction. For the new ramp, which will likely be supported on stiffer, compacted soil, a 140 pci Modulus of Subgrade Reaction should be appropriate for design.

Conclusions

As noted above, additional (deeper) subsurface exploration is suggested to determine the extent of soft soils and keyway depths for the new ramp to the bottom of the pond, as our current exploration did not extend into firm materials. As the ramp geometry and extent of new fill

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becomes known, we should also consult with GHD and review plans to confirm that the intent of our recommendations has been incorporated.

We trust that this letter includes the information you require at this time. Please do not hesitate to contact us should there be any questions or concerns.

Yours very truly,
MILLER PACIFIC ENGINEERING GROUP REVIEWED BY:

Nathan Klemin
Civil Engineer No. 83411
(Expires 3/31/17)

Michael Morisoli
Geotechnical Engineer No. 2541
(Expires 12/31/18)

Attachments: Figures 1 and 2, Appendix A and B

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APPENDIX A

LABORATORY ANALYTICAL TEST RESULTS

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Analytical Sciences

Report Date: November 10, 2016

Laboratory Report

Mike Morisoli
Miller Pacific Engineering - Novato
504 Redwood Blvd., Suite 220
Novato, CA 94947

Project Name: **GHD Rohnert Park Pond** **1206.179**

Lab Project Number: **6102812**

This 10 page report of analytical data has been reviewed and approved for release.

Mark A. Valentini, Ph.D.

Laboratory Director

P.O. Box 750336
Petaluma, CA 94975-0336
Telephone: (707) 769-3128

110 Liberty Street
Petaluma, CA
94952

TPH Gasoline

Lab#	Sample ID	Compound Name	Result (mg/kg)	RDL (mg/kg)
6102812-01	**Boring 1, 0-1 Ft**	Gasoline	ND	1.0

Date Sampled:	10/25/16	Date Analyzed:	10/31/16	QC Batch: B016286
Date Received:	10/28/16	Method:	EPA 8015B	

TPH Gasoline

Lab#	Sample ID	Compound Name	Result (mg/kg)	RDL (mg/kg)
6102812-02	**Boring 3, 0-1 Ft**	Gasoline	ND	1.0

Date Sampled:	10/25/16	Date Analyzed:	10/31/16	QC Batch: B016286
Date Received:	10/28/16	Method:	EPA 8015B	

TPH Diesel & Motor Oil

Lab#	Sample ID	Compound Name	Result (mg/kg)	RDL (mg/kg)
6102812-01	**Boring 1, 0-1 Ft**	Diesel	ND	5.0
		Motor Oil	ND	50

Date Sampled:	10/25/16	Date Analyzed:	11/01/16	QC Batch: B016295
Date Received:	10/28/16	Method:	EPA 8015B	

TPH Diesel & Motor Oil

Lab#	Sample ID	Compound Name	Result (mg/kg)	RDL (mg/kg)
6102812-02	**Boring 3, 0-1 Ft**	Diesel	ND	5.0
		Motor Oil	ND	50

Date Sampled:	10/25/16	Date Analyzed:	11/01/16	QC Batch: B016295
Date Received:	10/28/16	Method:	EPA 8015B	

CAM Metals

Lab#	Sample ID	Compound Name	Result (mg/kg)	RDL (mg/kg)
6102812-01	**Boring 1, 0-1 Ft**	Antimony (Sb)	ND	5.0
		Arsenic (As)	ND	1.5
		Barium (Ba)	120	2.0
		Beryllium (Be)	ND	0.50
		Cadmium (Cd)	ND	0.50
		Chromium (Cr)	15	1.5
		Cobalt (Co)	8.4	1.5
		Copper (Cu)	15	2.0
		Lead (Pb)	9.0	3.0
		Molybdenum (Mo)	ND	1.0
		Nickel (Ni)	18	2.0
		Selenium (Se)	ND	5.0
		Silver (Ag)	ND	1.0
		Thallium (Tl)	ND	5.0
		Vanadium (V)	13	2.0
		Zinc (Zn)	20	5.0

Date Sampled:	10/25/16	Date Analyzed:	11/07/16	QC Batch: B016255
Date Received:	10/28/16	Method:	EPA 6010B	

CAM Metals

Lab#	Sample ID	Compound Name	Result (mg/kg)	RDL (mg/kg)
6102812-02	**Boring 3, 0-1 Ft**	Antimony (Sb)	ND	5.0
		Arsenic (As)	ND	1.5
		Barium (Ba)	100	2.0
		Beryllium (Be)	ND	0.50
		Cadmium (Cd)	ND	0.50
		Chromium (Cr)	31	1.5
		Cobalt (Co)	8.4	1.5
		Copper (Cu)	17	2.0
		Lead (Pb)	11	3.0
		Molybdenum (Mo)	ND	1.0
		Nickel (Ni)	29	2.0
		Selenium (Se)	ND	5.0
		Silver (Ag)	ND	1.0
		Thallium (Tl)	ND	5.0
		Vanadium (V)	31	2.0
		Zinc (Zn)	31	5.0

Date Sampled:	10/25/16	Date Analyzed:	11/07/16	QC Batch: B016255
Date Received:	10/28/16	Method:	EPA 6010B	

Mercury

Lab#	Sample ID	Compound Name	Result (mg/kg)	RDL (mg/kg)
6102812-01	**Boring 1, 0-1 Ft**	Mercury (Hg)	ND	0.10

Date Sampled:	10/25/16	Date Analyzed:	11/02/16	QC Batch: B016219
Date Received:	10/28/16	Method:	EPA 7471A	

Mercury

Lab#	Sample ID	Compound Name	Result (mg/kg)	RDL (mg/kg)
6102812-02	**Boring 3, 0-1 Ft**	Mercury (Hg)	0.20	0.10

Date Sampled:	10/25/16	Date Analyzed:	11/02/16	QC Batch: B016219
Date Received:	10/28/16	Method:	EPA 7471A	

Quality Assurance Report

TPH Gasoline

Analyte	Result	Reporting Limit	Units	Spike Level	Source Result	%REC	%REC Limits	RPD	RPD Limit	Notes
Batch B016286 - EPA 5030 GC										
Blank (B016286-BLK1)				Prepared & Analyzed: 10/27/16						
Gasoline	ND	1.0	mg/kg							
LCS (B016286-BS1)				Prepared & Analyzed: 10/27/16						
Benzene	0.025	0.005	mg/kg	0.0250		102	70-130			
Toluene	0.024	0.005	mg/kg	0.0250		97	70-130			
Ethylbenzene	0.025	0.005	mg/kg	0.0250		100	70-130			
Xylenes	0.075	0.015	mg/kg	0.0750		100	70-130			
LCS Dup (B016286-BSD1)				Prepared & Analyzed: 10/27/16						
Benzene	0.025	0.005	mg/kg	0.0250		102	70-130	0.3	20	
Toluene	0.024	0.005	mg/kg	0.0250		97	70-130	0.4	20	
Ethylbenzene	0.024	0.005	mg/kg	0.0250		98	70-130	2	20	
Xylenes	0.074	0.015	mg/kg	0.0750		99	70-130	0.7	20	

TPH Diesel & Motor Oil

Analyte	Result	Reporting Limit	Units	Spike Level	Source Result	%REC	%REC Limits	RPD	RPD Limit	Notes
Batch B016295 - EPA 3550B GC										
Blank (B016295-BLK1)				Prepared & Analyzed: 11/01/16						
Diesel	ND	5.0	mg/kg							
Motor Oil	ND	50	mg/kg							
Matrix Spike (B016295-MS1)		**Source: 6102812-01**		Prepared & Analyzed: 11/01/16						
Diesel	190	5.0	mg/kg	226	ND	84	65-135			
Matrix Spike Dup (B016295-MSD1)		**Source: 6102812-01**		Prepared & Analyzed: 11/01/16						
Diesel	183	5.0	mg/kg	224	ND	82	65-135	3	30	

CAM Metals

Analyte	Result	Reporting Limit	Units	Spike Level	Source Result	%REC	%REC Limits	RPD	RPD Limit	Notes
Batch B016255 - EPA 3050B										
Blank (B016255-BLK1)				Prepared & Analyzed: 10/14/16						
Antimony (Sb)	ND	5.0	mg/kg							
Arsenic (As)	ND	1.5	mg/kg							
Barium (Ba)	ND	2.0	mg/kg							
Beryllium (Be)	ND	0.50	mg/kg							
Cadmium (Cd)	ND	0.50	mg/kg							
Chromium (Cr)	ND	1.5	mg/kg							
Cobalt (Co)	ND	1.5	mg/kg							
Copper (Cu)	ND	2.0	mg/kg							
Lead (Pb)	ND	3.0	mg/kg							
Molybdenum (Mo)	ND	1.0	mg/kg							
Nickel (Ni)	ND	2.0	mg/kg							
Selenium (Se)	ND	5.0	mg/kg							
Silver (Ag)	ND	1.0	mg/kg							
Thallium (Tl)	ND	5.0	mg/kg							
Vanadium (V)	ND	2.0	mg/kg							
Zinc (Zn)	ND	5.0	mg/kg							
LCS (B016255-BS1)				Prepared & Analyzed: 10/14/16						
Antimony (Sb)	24.4	5.0	mg/kg	25.0		98	70-130			
Arsenic (As)	24.9	1.5	mg/kg	25.0		100	70-130			
Barium (Ba)	26.0	2.0	mg/kg	25.0		104	70-130			
Beryllium (Be)	25.5	0.50	mg/kg	25.0		102	70-130			
Cadmium (Cd)	27.0	0.50	mg/kg	25.0		108	70-130			
Chromium (Cr)	26.3	1.5	mg/kg	25.0		105	70-130			
Cobalt (Co)	27.0	1.5	mg/kg	25.0		108	70-130			
Copper (Cu)	25.5	2.0	mg/kg	25.0		102	70-130			
Lead (Pb)	25.6	3.0	mg/kg	25.0		102	70-130			
Molybdenum (Mo)	26.1	1.0	mg/kg	25.0		105	70-130			
Nickel (Ni)	26.4	2.0	mg/kg	25.0		106	70-130			
Selenium (Se)	24.6	5.0	mg/kg	25.0		98	70-130			
Silver (Ag)	6.26	1.0	mg/kg	6.25		100	70-130			
Thallium (Tl)	27.2	5.0	mg/kg	25.0		109	70-130			
Vanadium (V)	25.4	2.0	mg/kg	25.0		102	70-130			
Zinc (Zn)	27.6	5.0	mg/kg	25.0		110	70-130			

CAM Metals

Analyte	Result	Reporting Limit	Units	Spike Level	Source Result	%REC	%REC Limits	RPD	RPD Limit	Notes
Batch B016255 - EPA 3050B										
LCS Dup (B016255-BSD1)				Prepared & Analyzed: 10/14/16						
Antimony (Sb)	24.5	5.0	mg/kg	25.0		98	70-130	0.4	20	
Arsenic (As)	25.4	1.5	mg/kg	25.0		102	70-130	2	20	
Barium (Ba)	26.2	2.0	mg/kg	25.0		105	70-130	0.5	20	
Beryllium (Be)	25.6	0.50	mg/kg	25.0		102	70-130	0.4	20	
Cadmium (Cd)	27.1	0.50	mg/kg	25.0		109	70-130	0.5	20	
Chromium (Cr)	26.4	1.5	mg/kg	25.0		106	70-130	0.6	20	
Cobalt (Co)	27.2	1.5	mg/kg	25.0		109	70-130	0.7	20	
Copper (Cu)	25.6	2.0	mg/kg	25.0		102	70-130	0.4	20	
Lead (Pb)	25.7	3.0	mg/kg	25.0		103	70-130	0.7	20	
Molybdenum (Mo)	26.4	1.0	mg/kg	25.0		106	70-130	0.9	20	
Nickel (Ni)	26.5	2.0	mg/kg	25.0		106	70-130	0.3	20	
Selenium (Se)	24.6	5.0	mg/kg	25.0		98	70-130	0.1	20	
Silver (Ag)	6.30	1.0	mg/kg	6.25		101	70-130	0.6	20	
Thallium (Tl)	27.4	5.0	mg/kg	25.0		110	70-130	0.8	20	
Vanadium (V)	25.5	2.0	mg/kg	25.0		102	70-130	0.3	20	
Zinc (Zn)	27.6	5.0	mg/kg	25.0		110	70-130	0.2	20	

Mercury

Analyte	Result	Reporting Limit	Units	Spike Level	Source Result	%REC	%REC Limits	RPD	RPD Limit	Notes
Batch B016219 - EPA 7471A Prep										
Blank (B016219-BLK1)				Prepared: 10/05/16	Analyzed: 10/06/16					
Mercury (Hg)	ND	0.10	mg/kg							
Matrix Spike (B016219-MS1)		**Source: 6093002-01**		Prepared: 10/05/16	Analyzed: 10/06/16					
Mercury (Hg)	0.330	0.10	mg/kg	0.309	ND	107	60-140			
Matrix Spike Dup (B016219-MSD1)		**Source: 6093002-01**		Prepared: 10/05/16	Analyzed: 10/06/16					
Mercury (Hg)	0.340	0.10	mg/kg	0.300	ND	113	60-140	6	20	

Notes and Definitions

RDL	Reporting Detection Limit
ND	Analyte NOT DETECTED at or above the reporting detection limit (RDL)
RPD	Relative Percent Difference
NR	Not Reported

Analytical Sciences
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110 Liberty Street, Petaluma, CA 94952
(707) 769-3128
Fax (707) 769-8093

CHAIN OF CUSTODY

LAB PROJECT NUMBER: 6102812
CLIENT'S PROJECT NAME: GAD ROHWERT PARK POND
CLIENT'S PROJECT NUMBER: 1206.179

CLIENT INFORMATION

COMPANY NAME: MILLER PACIFIC ENG. GROUP
ADDRESS: 504 REDWOOD BLVD ST 220
NOVATO CA 94947
CONTACT: MIKE MORISOLI
PHONE#: 415-382-3444
FAX #: 415-382-3450

MMORISOLI@MILLERPAC.COM

TURNAROUND TIME (check one)

MOBILE LAB ____
SAME DAY ____ 24 HOURS ____
48 HOURS ____ 72 HOURS ____
5 DAYS ____ NORMAL X

COOLER TEMPERATURE ____ °C

COC
PAGE 1 OF 1

ANALYSIS

ITEM	CLIENT SAMPLE I.D.	DATE SAMPLED	TIME	MATRIX	# CONT.	PRESV. YES/NO	TPH/GAS/BTEX & MTBE EPA 8015M/8020	TPH DIESEL EPA 8015M	OXYGENATED FUEL ADDITIVES EPA 8260M	VOLATILE HYDROCARBONS EPA 8260	CHLORINATED SOLVENTS EPA 8010	TRPH SM 5520F	SEMI-VOLATILE HYDROCARBONS EPA 8270	TOTAL LEAD	5 LUFT METALS	CAM 17 METALS	Gas, diesel oil-TPH.			COMMENTS	LAB SAMPLE #
1																					
2	BORING 1,	10-25-16														X	X			6102812	-01
3	0-1 ft																				
4	BORING 3	10-25-16														X	X				-02
5	0-1 ft.																				
6																					
7																					
8																					
9																					
10																					
11																					
12																					

SIGNATURES

RELINQUISHED BY: [signature] — SIGNATURE
10-28-16 — DATE
TIME

RECEIVED BY LABORATORY: [signature] — SIGNATURE
10/28/16 — DATE
1510 — TIME

APPENDIX B

LABORATORY CORROSION TEST RESULTS

DRAFT

Corrosivity Tests Summary

CTL # 081-293 **Date:** 11/7/2016 **Tested By:** PJ **Checked:** PJ
Client: Miller Pacific **Project:** GHD- Rohnert Park Treatment Pond **Proj. No:** 1206.179
Remarks:

Sample Location or ID			Resistivity @ 15.5 °C (Ohm-cm)			Chloride	Sulfate		pH	ORP (Redox)		Sulfide	Moisture	Soil Visual Description
			As Rec.	Min	Sat.	mg/kg Dry Wt.	mg/kg Dry Wt.	% Dry Wt.		E_H (mv)	At Test Temp °C	Qualitative by Lead	At Test %	
Boring	Sample, No.	Depth, ft.	ASTM G57	Cal 643	ASTM G57	ASTM D4327	ASTM D4327	ASTM D4327	ASTM G51	ASTM G200		Acetate Paper	ASTM D2216	
C1	-	0-1	729	-	709	<2	103	0.0103	7.4	-	-	-	28.4	Dark Gray Sandy CLAY
C2	-	.5-1.5	1,285	-	1,271	4	33	0.0033	7.6	-	-	-	42.4	Olive Brown Sandy CLAY
C3	-	.5-1	1,373	-	1,337	2	23	0.0023	7.4	-	-	-	33.2	Olive Brown Sandy CLAY
C4	-	.5-1.5	1,492	-	1,431	6	37	0.0037	7.5	-	-	-	28.1	Olive Brown Sandy CLAY

Corrosivity Tests

CTL Job No:	081-293	Project No.:	1206.179	IC Ions to test for:	Both
Client:	Miller Pacific	**Date:**	11/3/2016		
Project Name:	GHD- Rohnert Park Treatment Pond	**By:**	PJ		

Boring:	C1	C2	C3	C4		
Sample:						
Depth, ft:	0-1	.5-1.5	.5-1	.5-1.5		
Soil Description:	Dark Gray Sandy CLAY	Olive Brown Sandy CLAY	Olive Brown Sandy CLAY	Olive Brown Sandy CLAY		
EXTRACTION						
Extraction Flask No.						
Wt. of wet soil (g)	100.04	105.62	102.60	104.32		
Vol of water (ml)	300	300	300	300	300	300
% H_2O of Extracted Sample:						
Pan No.						
Pan wt. (g)	22.37	22.32	22.41	22.20		
Total wet wt. (g)	122.58	144.82	109.35	114.78		
Total dry wt (g)	100.42	108.36	87.69	94.48		
ORP / SULFIDE TESTS						
Beaker No.						
ORP, E_H (NHE) (Rmv)						
ORP Test Temp, ^oC						
Sulfide						
ASTM RESISTIVITY **- As Received**						
Small Dial Reading	100	1,000	1,000	1,000		
Large Dial Reading	6.64	1.16	1.24	1.35		
Temp. ^oC	19.4	19.8	19.8	19.7		
ASTM RESISTIVITY **- 100% Saturation**						
Bowl No.						
Small Dial Reading	100	1,000	1,000	1,000		
Large Dial Reading	6.21	1.11	1.10	1.18		
Temp. ^oC	21.2	21.3	24.1	24.0		
pH TEST						
pH measurement #1	7.47	7.58	7.39	7.54		
pH measurement #2	7.42	7.58	7.44	7.53		
pH measurement #3	7.40	7.57	7.40	7.48		
CHLORIDE AND SULFATE TESTING						
IC Ions to test for:	Both	Both	Both	Both	Both	Both
Vial No.						
CHLORIDE						
Meas. conc(mg **Cl^-**/L)	0.3	0.8	0.5	1.5		
SULFATE						
Meas. conc(mg **SO_4^{-2}**/L)	25	7.3	5.5	9.4		
Comments:						

Corrosivity Tests

CTL Job No:	081-293	Project No.:	1206.179	IC Ions to test for:	Both
Client:	Miller Pacific	**Date:**	11/3/2016		
Project Name:	GHD- Rohnert Park Treatment Pond	**By:**	PJ		

Boring:						
Sample:						
Depth, ft:						
Soil Description:						
EXTRACTION						
Extraction Flask No.						
Wt. of wet soil (g)						
Vol of water (ml)	300	300	300	300	300	300
% H_2O of Extracted Sample:						
Pan No.						
Pan wt. (g)						
Total wet wt. (g)						
Total dry wt (g)						
ORP / SULFIDE TESTS						
Beaker No.						
ORP, E_H (NHE) (Rmv)						
ORP Test Temp, °C						
Sulfide						
ASTM RESISTIVITY **- As Received**						
Small Dial Reading						
Large Dial Reading						
Temp. °C						
ASTM RESISTIVITY **- 100% Saturation**						
Bowl No.						
Small Dial Reading						
Large Dial Reading						
Temp. °C						
pH TEST						
pH measurement #1						
pH measurement #2						
pH measurement #3						
CHLORIDE AND SULFATE TESTING						
IC Ions to test for:	Both	Both	Both	Both	Both	Both
Vial No.						
CHLORIDE						
Meas. conc(mg Cl^-/L)						
SULFATE						
Meas. conc(mg SO_4^{-2}/L)						
Comments:						

Corrosivity Tests

CTL Job No:	081-293	Project No.:	1206.179	IC Ions to test for:	Both
Client:	Miller Pacific	**Date:**	11/3/2016		
Project Name:	GHD- Rohnert Park Treatment Pond	**By:**	PJ		

Boring:						
Sample:						
Depth, ft:						
Soil Description:						
EXTRACTION						
Extraction Flask No.						
Wt. of wet soil (g)						
Vol of water (ml)	300	300	300	300	300	300
% H_2O of Extracted Sample:						
Pan No.						
Pan wt. (g)						
Total wet wt. (g)						
Total dry wt (g)						
ORP / SULFIDE TESTS						
Beaker No.						
ORP, E_H (NHE) (Rmv)						
ORP Test Temp, ^oC						
Sulfide						
ASTM RESISTIVITY **- As Received**						
Small Dial Reading						
Large Dial Reading						
Temp. ^oC						
ASTM RESISTIVITY **- 100% Saturation**						
Bowl No.						
Small Dial Reading						
Large Dial Reading						
Temp. ^oC						
pH TEST						
pH measurement #1						
pH measurement #2						
pH measurement #3						
CHLORIDE AND SULFATE TESTING						
IC Ions to test for:	Both	Both	Both	Both	Both	Both
Vial No.						
CHLORIDE						
Meas. conc(mg **Cl^-**/L)						
SULFATE						
Meas. conc(mg **SO_4^{-2}**/L)						
Comments:						

Appendix A-2 - Rohnert Park Sewer Pond Concrete Condition Assessment; GHD; December 9, 2016

Memorandum

December 9, 2016

To:	Parastou Hooshialsadat, GHD	Ref. No.:	11125110,Ph.20,Tsk.2
From:	Jeff Knauer, GHD	Tel:	510.420.3312
cc:	Matt Winkelman, GHD		
Subject:	**Rohnert Park Sewer Pond Concrete Condition Assessment**		

1. Introduction

GHD conducted a condition assessment survey of the City of Rohnert Park's Sewer Pond facility with the objective of quantifying the present condition of the reinforced concrete structure with respect to corrosion related degradation and concrete degradation related to environmental exposure based on service and soil exposure. This information is intended for use in facilitating identification of rehabilitation, repair or replacement alternatives.

The condition assessment survey included the following components:

- Visual assessment
- Measurement of reinforcing steel-to-soil potential
- Delamination survey using acoustical sounding techniques
- Measurement of in-situ concrete compressive strength
- Evaluation of soil corrosivity
- Petrographic examination of concrete cores

2. Description of Test Methods

Visual assessment included identification of concrete spalls and associated exposed reinforcing steel, cracks, stains, bleeding, calcification and other indications of concrete damage or degradation. Reinforcing steel potential was measured in four (4) locations corresponding to the four (4) concrete core locations which exposed reinforcing steel strands. A portable copper/copper sulfate reference electrode was placed in contact with the soil and a high

impedance digital multi-meter was used to measure the potential of the reinforcing steel with respect to the portable reference electrode. Per ASTM C876 "Standard Test Method for Corrosion Potentials of Uncoated Reinforcing Steel in Concrete"

A delamination survey using acoustical sounding techniques in accordance with ASTM D4580 was conducted in four (4) selected regions of approximately two hundred square feet each. Delamination surveys are used to identify areas, not visibly identifiable, where the concrete matrix has separated from the reinforcing steel due to corrosion of the reinforcing steel. Delamination is typically a precursor to concrete spalling.

Measurement of in-situ concrete compressive strength using a rebound hammer in accordance with ASTM C805 "Standard Test Method for Rebound Number of Hardened Concrete" was conducted in four (4) locations. A total of eight (8) measurements were recorded in each location and used in developing an opinion of average reading.

Soil corrosivity evaluation was conducted on four (4) soil borings obtained corresponding with the concrete core locations. Soil samples were analyzed for as-received resistivity, saturated (or minimum) resistivity, chloride ion content, sulfate ion content, and pH.

Petrographic laboratory examination of two core sample specimens was conducted in accordance with ASTM C856 "Standard Practice for Petrographic Examination of Hardened Concrete". The examinations included assessment using a stereo-microscope at magnification up to 45x.

3. Findings

Visual assessment revealed widespread fine to moderate diameter cracking and several locations of exposed reinforcing steel. Presence of vegetation and plant growth in various locations confirmed that cracks extended through the concrete matrix. Photo 1 depicts observed cracks, crack repairs and presence of vegetation.

PHOTO 1

Reinforcing steel potential was measured in four (4) locations corresponding to the four (4) concrete core locations which exposed reinforcing steel strands. A portable copper/copper sulfate reference electrode was placed in contact with the soil and a high impedance digital multi-meter was used to measure the potential of the reinforcing steel with respect to the portable reference electrode. Per ASTM C876 "Standard Test Method for Corrosion Potentials of Uncoated Reinforcing Steel in Concrete".

TABLE 1
REINFORCING STEEL-TO-SOIL POTENTIAL

Test Location	Reinforcing Steel-to-Soil Potential vs. $CuSO_4$ Reference Electrode
1	-367 mV
2	-406 mV
3	-319 mV
4	-366 mV

Per ASTM; reinforcing steel potentials more negative than -350 mV indicates a probability of greater than 90% of ongoing corrosion activity. Potentials more negative than -350 mV were measured in three (3) of the four (4) locations tested.

The delamination survey did not reveal significant areas of delamination of concrete from reinforcing steel. The use of a rebound (Schmidt) hammer was deployed in order to assess the in-place uniformity of concrete, to delineate variations in concrete quality, and to estimate in-place strength. For a given concrete mixture, the rebound number is affected by factors such as moisture content of the test surface, the type of form material or type of finishing used in construction of the surface to be tested, vertical distance from the bottom of a concrete placement, and the depth of carbonation. Relationships between rebound number and concrete strength were used only to provide indications of relative concrete strength at different locations in the pond structure. Table 2 presents the correlated data.

TABLE 2
IN-SITU COMPRESSIVE STRENGTH MEASUREMENTS

Test Location	Average Correlated Compressive Strength
1	3,600 psi
2	3,000 psi
3	3,400 psi
4	<1,800 psi

Soil testing was conducted in order to quantify and classify the environment that the reinforced concrete structure has been exposed to. A summary of the soil corrosivity analysis is presented in Table 3. Evaluation is based on review of soil data and review of project parameters. A summary of the corrosion control recommendations for future restored, rehabilitated, or new reinforced concrete and buried piping is listed included in the Conclusions Section.

TABLE 3
SUMMARY OF SOIL CORROSIVITY ANALYSIS

Test Method	Level of Corrosivity
Laboratory Soil Sample As-Received Resistivity Analysis	Moderate to High
Laboratory Soil Sample Minimum Resistivity Analysis	Moderate to High
Laboratory Soil Sample Chloride Ion Analysis	Negligible
Laboratory Soil Sample pH Analysis	Negligible
Laboratory Soil Sample Sulfate Analysis	Negligible

Understanding of the local electrochemical characteristics of the soil is a key issue when evaluating the corrosivity of an environment. Table 4 includes the results of the laboratory soil analysis.

TABLE 4
LABORATORY SOIL ANALYSIS DATA

Sample No.	Boring No.	As-Received Soil Resistivity (ohm-cm)	Minimum Soil Resistivity (ohm-cm)	Chemical Data		
				pH	Sulfate (ppm)	Chloride (ppm)
1	C1	729	709	7.4	103	<2
2	C2	1,285	1,271	7.6	33	4
3	C3	1,373	1,337	7.4	23	2
4	C4	1,492	1,431	7.5	37	6

Soil resistivity is inversely proportional to the soil's ability to conduct current (conductivity). Increased moisture content in soil generally results in a decrease in soil resistivity. Corrosion of a metal in soil is a function of current flow to and from a metal through the adjacent medium. Therefore, corrosion activity of metal in soil normally increases as soil resistivity decreases. Table 5 correlates resistivity values with corrosivity of the soil.

TABLE 5
EFFECTS OF SOIL RESISTIVITY ON THE CORROSIVITY OF SOIL

Soil Resistivity (ohm-cm)	Degree of Corrosivity
0 - 500	Very High
500 - 1,000	High
1,000 - 2,000	Moderate
2,000 - 10,000	Mild
Above 10,000	Negligible

The minimum resistivities from the laboratory soil analysis, shown in Table 5, ranged from 709 ohm-cm to 1,431 ohm-cm. The soils analyzed are considered to range from *moderately corrosive* to *highly corrosive* based on the minimum measured soil resistivities.

Chloride ions found in soils tend to break down otherwise protective surface deposits, and can result in corrosion of buried metallic piping and reinforcing steel in concrete. Table 6 lists the effects of chloride ion concentrations on the corrosivity of the soil.

TABLE 6
EFFECTS OF CHLORIDE IONS ON THE CORROSIVITY OF SOIL

Water-Soluble Chloride Concentration (ppm)	Degree Of Corrosivity
Over 2,000	Very High
1,000 - 2,000	High
500 - 1,000	Moderate
100 - 500	Mild
Below 100	Negligible

The water-soluble chloride levels ranged from less than 2 ppm to 6 ppm. This range is considered *negligibly corrosive*.

Lower pH (more acidic) environments will result in a greater degree of corrosivity with respect to buried metallic and concrete structures. When pH increases above 7.0 (the neutral value) the conditions become increasingly more alkaline. In alkaline environments, steel forms a protective passive film layer on its surface. Table 7 correlates the effect of pH on the soil corrosivity with respect to exposed metallic or concrete structures.

TABLE 7
EFFECTS OF PH ON THE CORROSIVITY OF SOIL

pH	Degree Of Corrosivity
< 5	Very High
5-6	High
6-6.5	Moderate
6.5-7	Mild
> 7	Negligible

The pH of the boring samples tested ranged from 7.4 to 7.6. These levels are considered *negligibly corrosive*.

Sulfate ions found in soil can cause degradation of concrete structures. Sulfates in soil can be aggressive to portland cement concrete by combining chemically with certain constituents of the concrete which results in expansion and potential degradation of the concrete matrix causing softening of the concrete surface near the soil air interface. Table 8 correlates the effect of sulfates on the corrosivity of soil for concrete structures.

TABLE 8
EFFECTS OF SULFATE IONS ON THE CORROSIVITY OF SOIL

Water-Soluble Sulfate Concentration (ppm)	Degree Of Corrosivity
> 2,000	Very High
1,500-2,000	High
1,000-1,500	Moderate
500-1,000	Mild
<500	Negligible

The water-soluble sulfate levels ranged from 23 ppm to 103 ppm. This range is considered *negligibly corrosive*.

Concrete cores were obtained at four (4) selected locations for laboratory assessment and petrographic examination. Each core was approximately three (3) inches thick. Petrographic examination of concrete cores revealed the following summarized relevant findings:

- The cement binder of the concrete exposed along the top surface is degraded, softened, and eroded; exposing fine aggregate particles
- The concrete has undergone significant alteration including reduction of the pH and significant leaching of calcium from the cement paste
- No coarse aggregate was used in the concrete.
- No residual fly ash or other supplementary cementitious materials are observed in the cement paste.

The complete petrographic analysis report is attached as an appendix to this memorandum

4. Conclusions

The sewer pond is constructed of what may be considered a grout-like material; differencing from traditional concrete due to the absence of course aggregate; with low compressive strength. The average thickness is concluded to be approximately three (3) inches with reinforcing steel depth of cover ranging from 0 to 1.5 inches. The reinforcing steel is small gauge steel. Degradation, including cracking, is anticipated to continue to progress in present exposure scenarios based on surface-side and soil-side mechanisms of deterioration.

The reinforcing steel-to-soil potential measurements obtained were considered in parallel with other data collected such as soil chloride ion content, depth of concrete carbonation, delamination survey findings, assessment of environmental exposure conditions, in order to formulate conclusions concerning corrosion activity of embedded steel and its probable effect

on the service life of a structure. Although potentials obtained suggest a high probability of reinforcing steel corrosion, corrosion of steel reinforcement is concluded not to be the primary mechanism of degradation experienced to date. This is supported by the results of the delamination survey.

The secondary objective of the soil corrosivity evaluation is to evaluate the soil corrosivity at the project site with respect to the various buried utility and foundation material alternatives associated with the facility repurposing project. The corrosion control recommendations have been developed with the objective of imparting a cost effective, low maintenance means of providing corrosion control to the exterior of the buried reinforced concrete structures and buried metallic and reinforced concrete piping and pipe appurtenances. Future corrosion control design should focus on extending the infrastructure service life with minimal accrued costs due to unscheduled maintenance, repair, and replacements. Based on the soil resistivity at the project location, the following methods are recommended to protect exterior pipeline surfaces from corrosion:

- A bonded dielectric coating (steel pipe only)
- Cathodic protection (steel and ductile iron pipe only)
- Mortar coating or concrete encasement

The primary corrosion control mechanism for the reinforcing steel is the high pH (alkalinity) of the concrete that passivates the steel. The minimum concrete cover specified in the Uniform Building Code will prevent migration of aggressive ions through the concrete to attack the reinforcing steel. A maximum water to cement ratio of 0.45 should be used for all concrete in contact with native soils. Table 9 summarizes recommendations for future corrosion control.

TABLE 9
SUMMARY OF BASELINE CORROSION CONTROL RECOMMENDATIONS

Structure	Recommendation
Buried Reinforced Concrete Structures	• A water to cement ratio not to exceed 0.45 for structures in contact with native soils • Minimum concrete cover of reinforcing steel in accordance with Uniform Building Code (UBC) guidelines
Buried Steel and Ductile Iron Pipe	• Cathodic protection or mortar coating/concrete encasement for steel and ductile iron piping • Bonded dielectric coating or mortar coating/concrete encasement for steel piping • Joints, flanges and flexible couplings bonded with insulated copper cable for electrical continuity

5. Appendices

1. Tourney Consulting Group Letter Report
2. Cooper Laboratories Soil Corrosivity Analysis Report
3. Braun Intertec Petrographic Examination Report

November 23, 2016

Jeff Knauer, P.E.
GHD
5900 Hollis Street
Suite A
Emeryville, CA 94608

Subject: Rohnert Park, CA Water Pollution Control Plant Core Analysis

Dear Jeff:

Tourney Consulting Group (TCG) is pleased to submit the findings on an analysis of four concrete cores from the Sedimentation Basin Slabs. The cores received were labeled C-1, C-2, C-3, and C-4. Each core was approximately six inches in diameter and about three inches in thickness.

TCG cut each of the cores twice to obtain the center sections for analysis. All cores were polished on one cut face and determined to have very similar initial visual observations. The core observations were determined to be very similar under magnification. TCG was originally going to conduct a basic microscopic investigation on all four cores. Instead, TCG determined to test hardness with respect to depth into the section on two core specimens and two would have a "detailed" microscopic analysis that includes thin sections and a conventional petrographic analysis.

Braun Intertec was contracted to conduct the detailed microscopic analysis on specimens C-2 and C-4, and this report is attached. TCG prepared the specimens for all the work and conducted relative hardness testing on specimens C-1 and C-3.

Hardness Testing:

Hardness testing was conducted on a polished faces of both C-1 and C-3 specimens. It is usual to see hardness results between 6 and 8 for quality concretes. TCG tested the hardness at four regions through the depth of the specimen. The surface hardness was consistently at 6.5 on both specimens. The results of the other three depth regions on both specimens were consistently a hardness of 7.5. Therefore the concrete slab was rather hard and the surface hardness was a slight bit lower. This makes sense from the observed degradation of the surface from chemical-related exposure. There is nothing surprising or unexpected in these results.

Braun Intertec Corporation
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Minneapolis, MN 55438

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Web: braunintertec.com

November 22, 2016 Project B1610633

Mr. Paul Tourney
Tourney Consulting Group, LLC
3401 Midlink Drive
Kalamazoo, Michigan 49048

Re: Petrographic Examination of Concrete Samples
From the City of Rohnert Park, CA Water Pollution Control Plant
TCG Project No. 16118

Dear Mr. Tourney:

Purpose

The purpose of this investigation is to assess and document the condition of concrete samples taken from the sedimentation basin slabs at the above referenced water-water treatment plant.

Background

Two concrete samples, labeled C-2 and C-4, were received on November 3, 2016 from Mr. Paul Tourney of Tourney Consulting Group (TCG), LLC, Kalamazoo, Michigan. Mr. Tourney stated that the samples are saw-cut portions of larger drilled cores extracted from the concrete slab of a sedimentation – equalization basin at a water pollution control plant in Rohnert Park, California. Figures 1 and 2 document the samples in as-received condition. According to a drawing of a portion of the facility also provided by TCG, the sedimentation basin was constructed in or about 1970. The cores were extracted from the structure and submitted for laboratory analysis as part of a condition assessment of the basin slabs. Reportedly, the surfaces of the slabs exhibit some apparent deterioration. Both cores were taken from deteriorated areas of the slabs.

Petrographic examinations (ASTM C856) were requested by Mr. Tourney for each of the two provided samples to assess and document the composition and condition of the concrete and to ascertain the probable cause(s) for observed distress on the concrete surface.

Procedures

Petrographic examination of the concrete samples was conducted in general accordance with ASTM C856, "Standard Practice for Petrographic Examination of Hardened Concrete." Each of the two core segments was initially subjected to visual examination to document the as-received condition of the concrete. Detailed petrographic (microscopical) examinations were conducted on saw-cut, polished, and broken surfaces of the concrete samples. The examinations were also conducted with the aid of a stereo-microscope at magnifications up to 45x.

Two thin sections were produced from each sample, including a cross section of the top and bottom ends and the remaining body of concrete. To produce such thin sections, small, rectangular blocks (or "billets") were saw-cut from the upper portions of each core, and one side of each billet was ground and polished to a flat smooth finish. These surfaces were then vacuum-impregnated with clear laboratory epoxy, excess material was struck off, and the epoxy allowed to set. After the epoxy hardened, the surfaces were re-polished and attached to separate glass microscope slides with clear epoxy, then later reduced in thickness to approximately 20 to 30 µm (0.0008 to 0.0012 in.), and a coverslip was applied. The resulting thin sections were examined using a Nikon Optiphot-Pol petrographic (polarized-light) microscope at magnifications up to 400x.

Results

Findings of the petrographic examinations confirm that the top ends of both concrete samples exhibit similar degradation, deterioration, and loss of the original surface, consistent with field reported. Figures 1 through 3 illustrate the deterioration evident along the top surfaces of each sample. Cement binder of the concrete exposed along the top surface is degraded, softened, and eroded, exposing many still-attached fine aggregate particles in low to moderate relief. Although such degradation and deterioration is fairly severe and has resulted in the loss of the original finish and top surfaces and of the slab in the sampling locations, it is mostly localized to the surface and near-surface regions. Existing depths of degraded and softened paste are generally less than 3 mm (0.12 in.), below which the cement paste become significantly harder with increased depth. The depth of lost surface paste or concrete form the original finished surface is unclear from the petrographic examination, but due to the fairly level profile and textural featured of the existing top surfaces, the depth of lost concrete/paste is believed to be fairly shallow and the depth of the slabs in these locations does not appear to be appreciably lessened.

Aside from the deterioration evident along the top surface, the body of concrete represented in each core appears to be in good physical condition with no evidence of significant cracking or visible distress. Figures 4, 5, and 6 show the condition of the concrete in vertical profile and the limited depth of surface degradation and deterioration.

However, it should be further noted that the full depth of concrete in both core locations has undergone significant alteration, including reduction of pH at and near the top and bottom sides, and significant leaching of calcium from the cement paste without visible evidence of physical distress. The following salient findings and features relative to destructive and non-destructive alteration of the concrete are observed in the provided cores:

1. Both cores exhibit significant reduction of pH in the concrete at and near both the top and bottom sides, as indicated by application of pH-indictor solutions to cut, polished, and broken surface of the samples (Figure 7). Most of this reduction in pH correlates and is attributed to carbonation of the cement paste. Depth of carbonation along the top surface of the concrete extends to depth of up to 13 mm (0.5 in.) in Core C-2 and 10 mm (0.4 in.) in Core C-4.

2. Significant reduction in concrete pH, up to 10 mm (0.4 in.) deep, is also noted along the underside of the two core samples (Figure 7); however, the reduced pH is only partially attributed to carbonation.

3. Both cores exhibit evidence of reduced crystallinity and chemical leaching of the cement paste along the immediate top surface and in the remaining, non-carbonated body of concrete, the latter without reduction of pH below approximately 10, as indicated by the pH stain pattern (Figure 7).

    a. Microscopical examination of the surface concrete in thin section reveals remnants of a thin zone of isotropic (non-crystalline), discolored, weak cement paste along the exposed top surface of each core (Figures 8, 9, and 10). This thin zone of altered cement paste is attributed to severe but very localized chemical attack and leaching of calcium from the reduced-pH, carbonated paste. This chemical attack is likely related to exposure to aggressive, likely acidic, solutions associated with the sewage water and/or biological growth on the surface of the concrete. Petrographic examination did not reveal the chemistry of aggressive solution of biological matter responsible for the chemical attack.

b. Microscopical examinations also revealed that non-carbonated cement paste in the body of the concrete also exhibits evidence of reduced crystallinity and likely chemical leaching of the cement hydrate (Figure 11). This leaching is apparently not associated with a significant degradation of the physical properties of the affected concrete. Despite the alteration, the cement paste remains fairly hard and paste-aggregate bond fairly tight. Some secondary deposits, mostly of ettringite (calcium sulfo-aluminate hydrate) are observed in some small voids of the cement paste in each core, but the amounts are not excessive or indicative of deleterious chemical attack. These effects are likely related to long-term mild leaching by movement of pore solution through the concrete.

4. An abundance of secondary calcium carbonate, some fairly coarsely crystalline, is noted in the reduced-pH, lower portion of each core (Figure 12). The carbonates appear to have partially replaced some of the cement paste and has filled many pores and voids. Such deposits are likely precipitated from a calcium-rich pore solution and leachate that removed the calcium from the body of the concrete.

5. The examinations of both samples have also revealed evidence of the development of very minor alkali-silica reaction (ASR) in in the concrete. However, only meager amounts of ASR gel are seen and no clear evidence of ASR-related distress or cracking is observed in either core sample.

Concrete Composition and Features

The following additional, salient findings and descriptions of the concrete are based on the petrographic examinations of Cores C-2 and C-4. Additional details of the petrographic examination are given in the attached petrography reports.

1. Concrete represented by each core is composed of siliceous natural sand bound in a non-air-entrained paste of portland cement. No coarse aggregate was used in the concrete. Figures 4, 5, 6, and 11 further document the appearance and composition of the concrete.

2. No residual fly ash or other supplementary cementitious materials are observed in the cement paste. Small amounts of small glassy particles are noted in the cement paste in thin section, but the findings are unclear as to whether this represent evidence of the presence of a slag-like additive, or fines associated with the aggregate, which contains some rocks with glassy and devitrified glass constituents. (Evidence suggest the latter more likely applies.)

3. No mix design information was provided, therefore, the examination could not confirm whether the constituents of this concrete are generally consistent with job specifications.

4. The concrete is not air entrained, based on the scarcity and low volume of small, spherical void (less than 1 mm) in the cement paste (Figures 6). Air contents are estimated to be in the range of 1 to 3 percent by volume of concrete and attributed to the presence of minor amounts of small entrapped air voids

5. Based on the findings of the physical properties and microscopical features of the cement paste, the water-cement ratio (w/c) of the body of concrete is interpreted to be moderate. However, due to the continued and in-situ advanced hydration of the cement paste, extensive leaching of the paste, and possible effects of the wastewater may have had on paste microstructure, more defined estimation of the w/c is deemed as speculative and is not given.

Additional Comments

Petrographic examination reveals that the concrete represented by each core exhibit evidence of both destructive alteration, including localized chemical attack of the top surface, as well as non-destructive alteration, such as carbonation and long-term leaching of the concrete. Carbonation is a normal and expected phenomenon in many if not most portland cement concrete exposed to air and moisture in service. Carbon dioxide in air reacts with hydration products in the cement paste to produce carbonate compounds in the paste, which is also accompanied by a lowering of the pH of the paste, typically from 12+ to less than 9. The depth of carbonation generally increases over time and the rate is dependent on the permeability of the cement paste. The depth of carbonation in these cores is not believe to be exceptional or abnormal for the reported age of the slab(s). However, the reduced pH associated with carbonation is potentially problematic if it extends to or beyond the depth of embedded steel reinforcement. By reducing the passive corrosion resistance that the high pH of concrete provided to embedded steel, carbonation could promote corrosion problems. The two provided cores exhibit embedded sections of welded wire fabric (mesh) that exhibit mostly minor surface corrosion of wires. Depth of mesh embedment in the cores is further described in the attached petrographic data sheets.

Leaching observed in the concrete samples is likely attributed to long-term exposure of the basin slabs to mildly aggressive compounds or conditions in the wastewater that have resulted in the breakdown, mobilization, and precipitation of calcium, and possibly other constituents in the cement paste. Some forms of biological growth or matter (bacteria, moss, etc.) may also contribute aggregate excretions. However, identification of these biological agents is beyond the scope of this petrographic study. Additional chemical analysis of the waste water and surface concrete would be required to further investigate these issues and the aggressivity of the water.

Although the petrographic examinations did not reveal apparent significant distress in the bodies of the provided concrete samples, aside from the shallow deterioration of the top surfaces, the existing quality or adequacy of the concrete for continued service is unclear, particularly given the shallow thickness (less than 3 in.) of the slab. The effects of the chemical leaching of the cement paste on the existing strength and durability of the slab were also not fully assessed from petrography alone and additional physical testing of the slab concrete may be warranted.

General Remarks

Findings of this evaluation are based solely on examination and analysis of the provided sample and may not necessarily represent the materials and condition of materials elsewhere in the same structures.

In performing its services, Braun Intertec used that degree of care and skill ordinarily exercised under similar circumstances by reputable members of its profession currently practicing in the same locality. No warranty, express or implied, is made.

The core sample will be retained for 30 days from the date of this report unless additional instructions for longer sample retention are provided

If you have any questions or concerns, please do not hesitate to contact me at 612.280.6505.

Sincerely,

BRAUN INTERTEC CORPORATION

Ronald D. Sturm
Senior Petrographer

Alfred J. Gardiner
Principal, Laboratory Director

Attachments:
Figures
Petrographic Examination Data Sheets

1a. Exposed, top end of sample.

1b. Side of core, exposed top end facing up.

1c. Bottom end of sample.

Figure 1. Top, side, and bottom of concrete sample C-2, as received for petrographic examination. Note the deterioration or erosion of the top surface.

2a. Exposed, top end of sample.

2b. Side of core, exposed top end facing up.

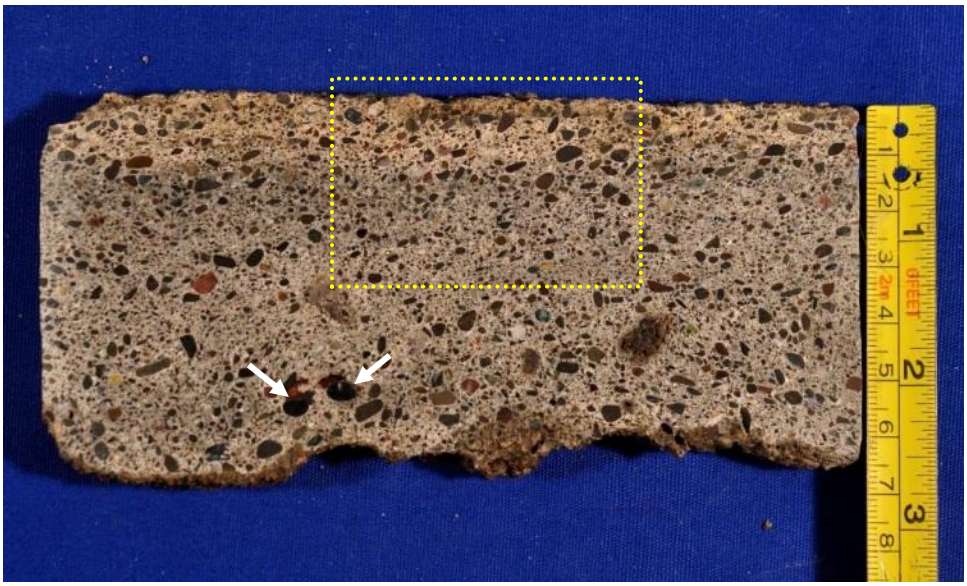
2c. Bottom end of sample.

Figure 2. Top, side, and bottom of concrete sample C-4, as received for petrographic examination. Note the deterioration or erosion of the top surface.

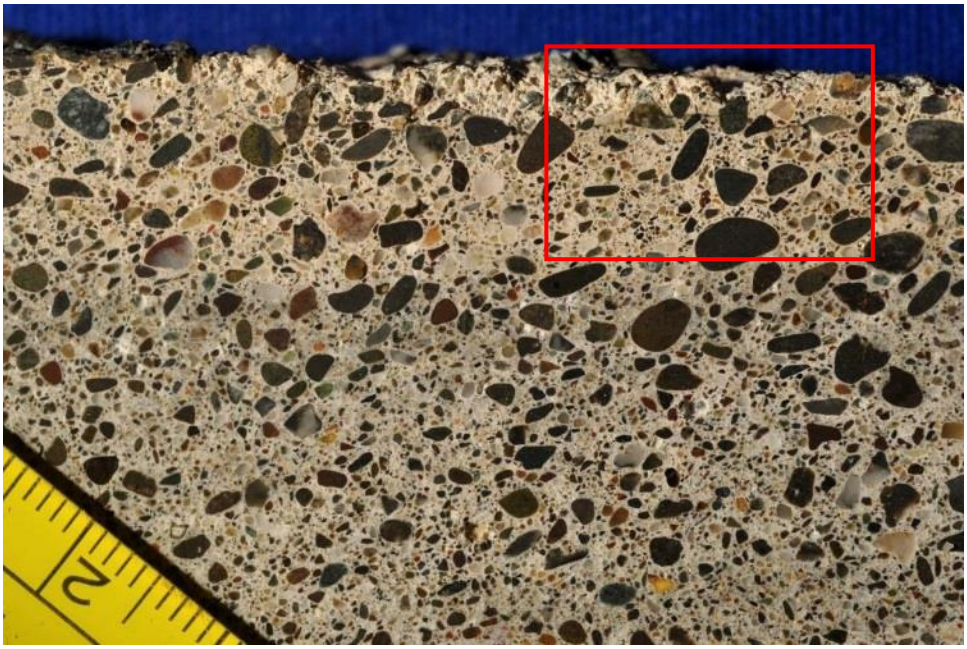
3a. Sample C-2.

3b. Sample C-4.

Figure 3. **Close-up views of representative portions of the top surfaces of the two concrete samples show the deterioration or erosion of the concrete. Note the deterioration of the cement binder has roughened the surface and exposed aggregate in low to moderate relief.**

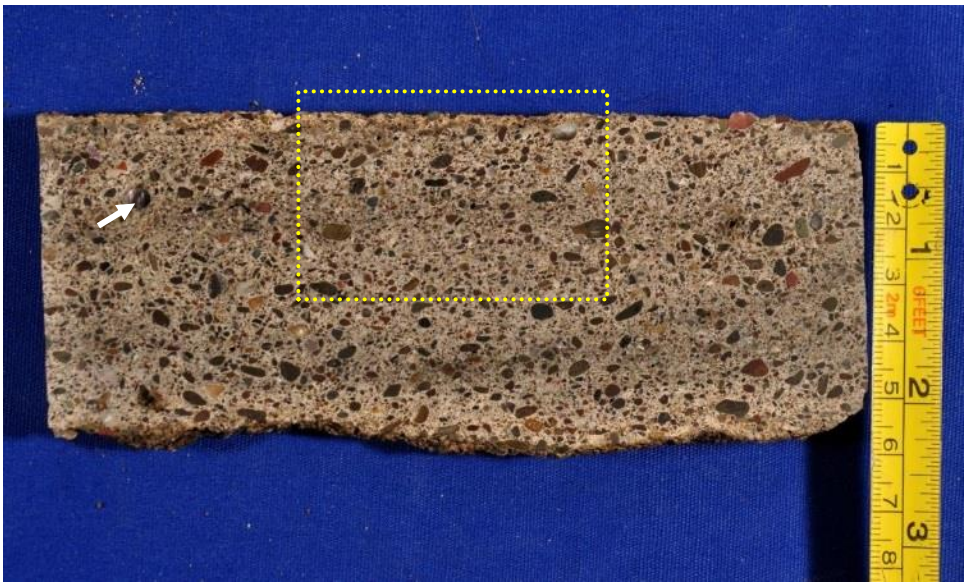


4a. Polished, full-depth vertical cross section.

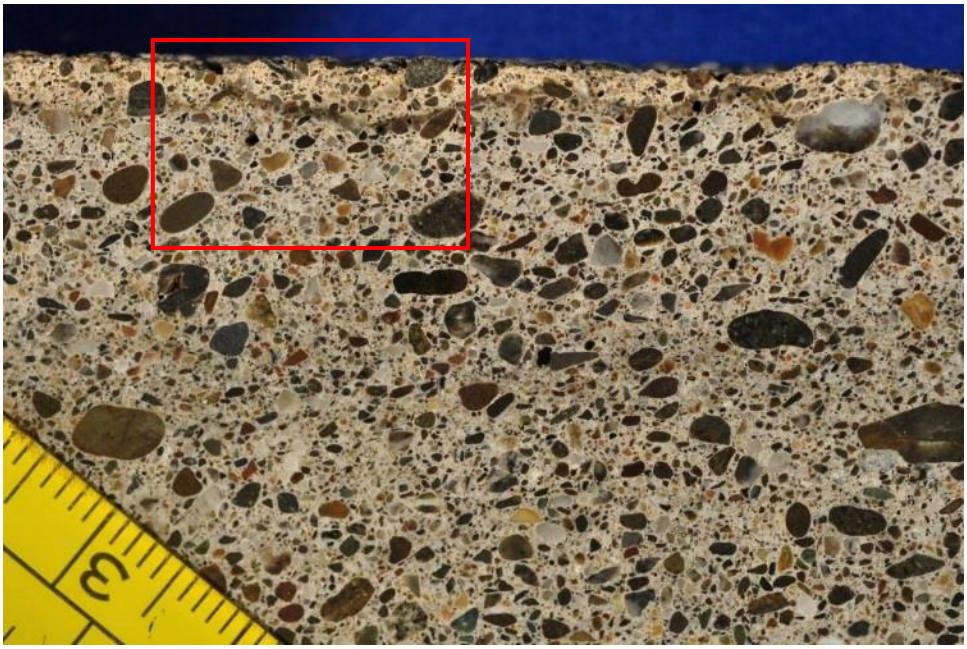


4b. Close-up of outlined area above.

Figure 4. **Polished, vertical cross section of Sample C-2 shows the existing thickness of the slab and general appearance and condition of the body of concrete. Note the small size of the aggregate used in the concrete suggesting placement of a grout-like mixture. Arrows mark cross sections of steel mesh wires. The area outlined in red in the lower image is further documented in Figure 6.**

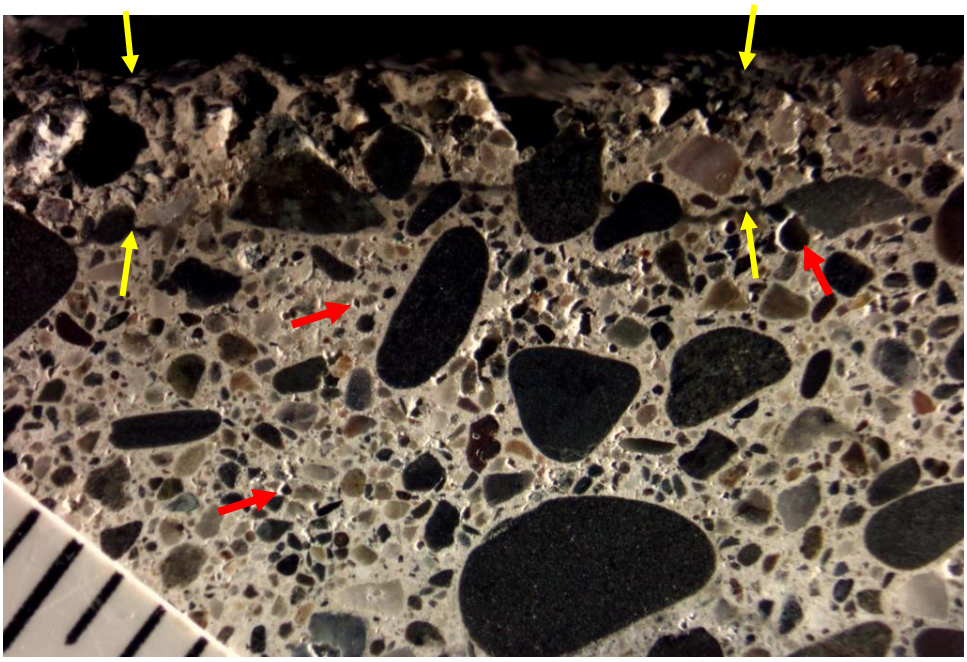


5a. Polished, full-depth vertical cross section.

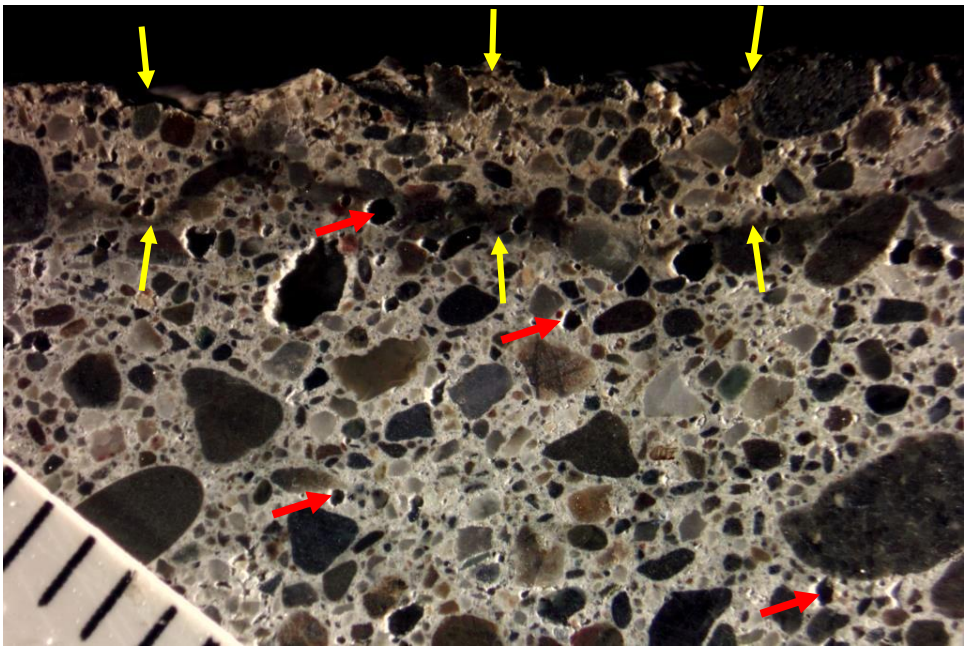


5b. Close-up of outlined area above.

Figure 5. **Polished, vertical cross section of Sample C-4 shows the existing thickness of the slab and general appearance and condition of the body of concrete. Again, note the small size of the aggregate used in the concrete suggesting placement of a grout-like mixture. Arrow marks a cross section of steel mesh wire. The area outlined in red in the lower image is further documented in Figure 6.**

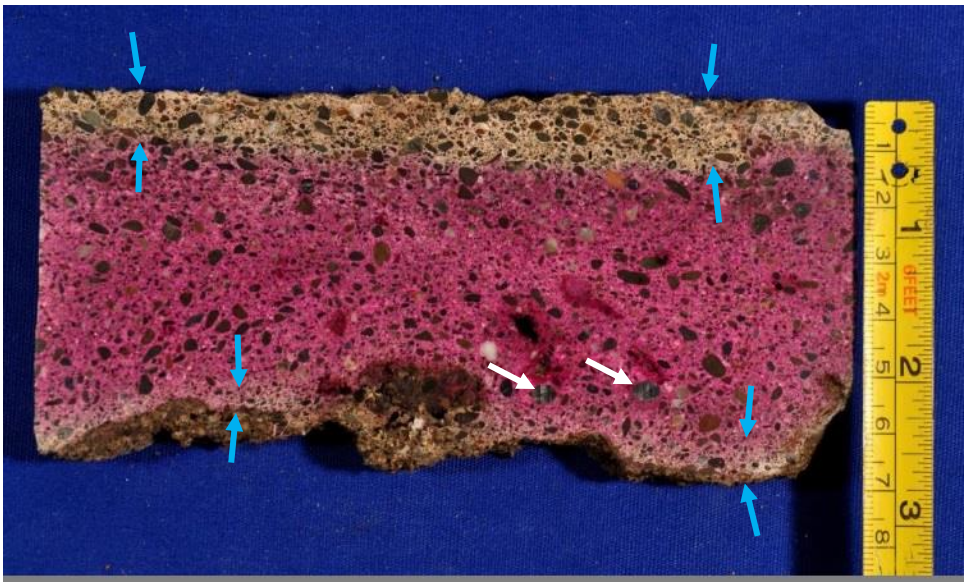


6a. Near-surface cross section of Core C-2.

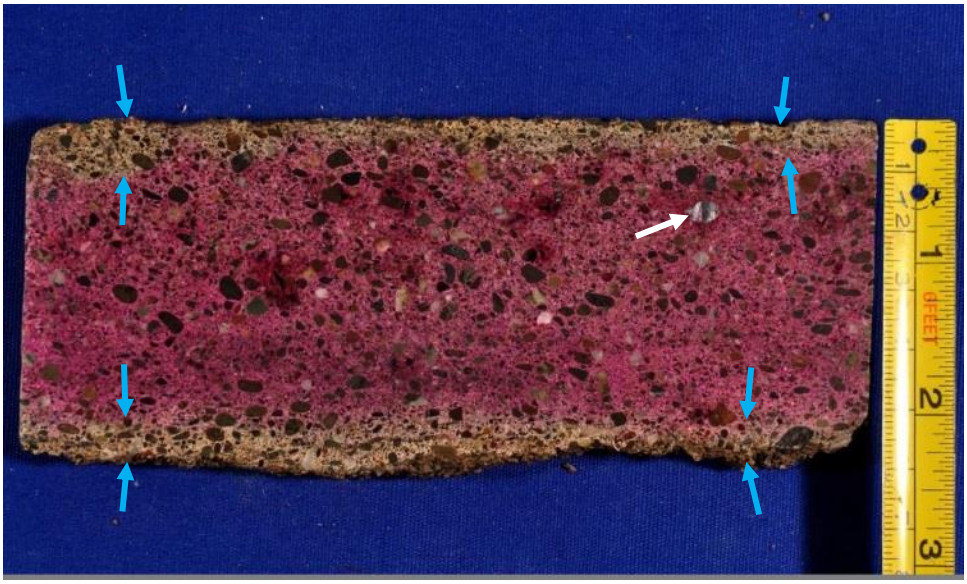


6b. Near-surface cross section of Core C-4.

Figure 6. **Polished, vertical cross sections of the near-surface concrete in Samples C-2 and C-4, previously depicted in Figures 4b and 5b, show a shallow zone of discolored, carbonated, and weakened concrete (marked between yellow arrows) along the top surface of each. Red arrows mark sparse small voids in the non-air-entrained concrete. Scales are marked in millimeter increments.**

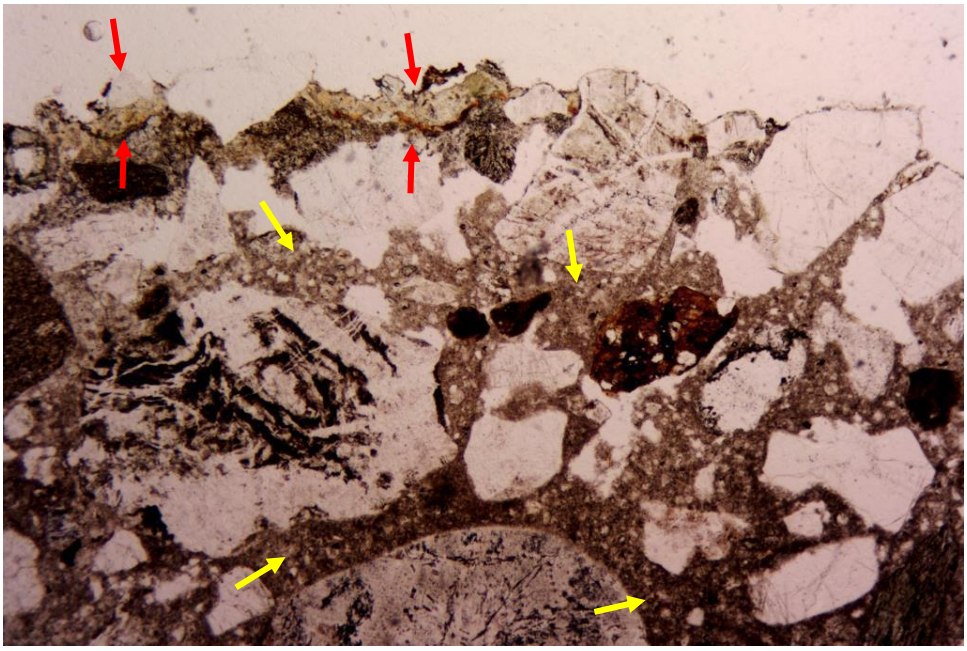


7a. Core C-2.

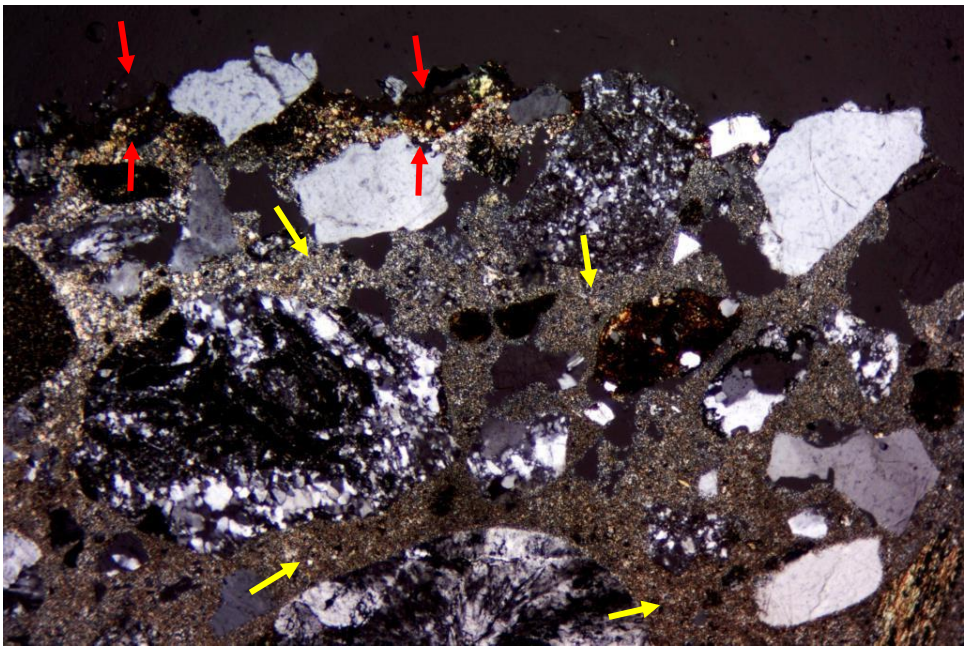


7b. Core C-4.

Figure 7. **Stained vertical cross sections of Cores C-2 and C-4 show the depth and pattern of pH reduction in the concrete at the top and bottom ends of each core partially attributed to paste carbonation. Location of steel mesh wires are marked with white arrows. Each surface was treated with pH-indicator (phenolphthalein) solution that imparts a magenta stain to high-pH, non-carbonated cement paste, but leaves no stain or produced light to no red stain in reduced-pH and carbonated paste (marked between blue arrows).**

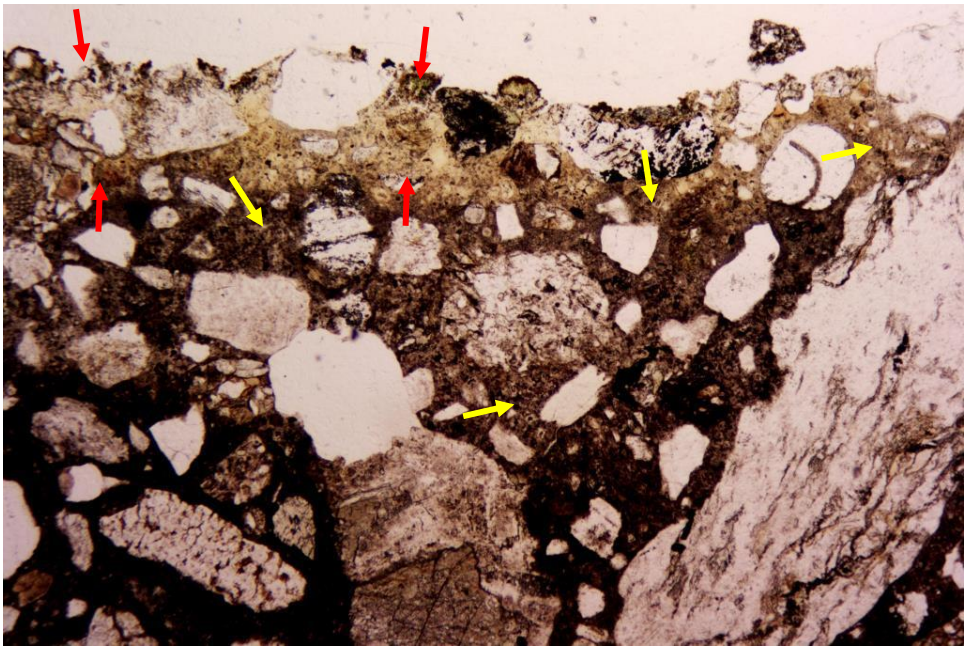


8a. Transmitted, plane-polarized light.

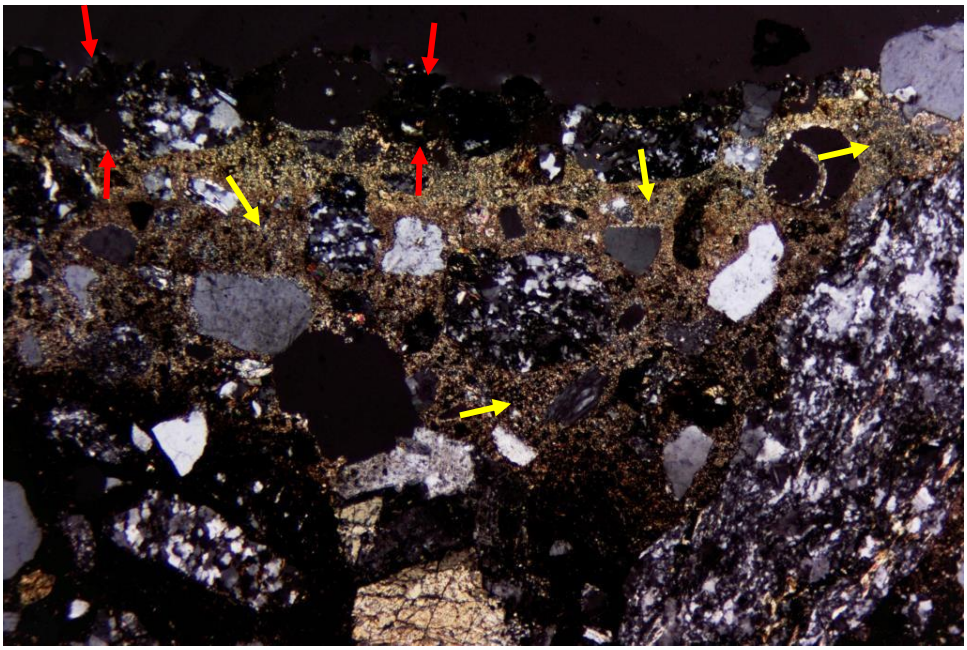


8b. Same area as above, viewed with partially-crossed, polarized light.

Figure 8. **Photomicrographs of the upper approximate 2 mm (0.08 in.) of Core C-2 concrete in thin section shows carbonation of the cement paste (marked with yellow arrows) and a thin zone of intensely leached paste (red arrows). Also note the profile of the top surface with erosion of the cement paste between exposed sand aggregate grains. Length of field, left to right, is approximate 3 mm (0.12 in.).**

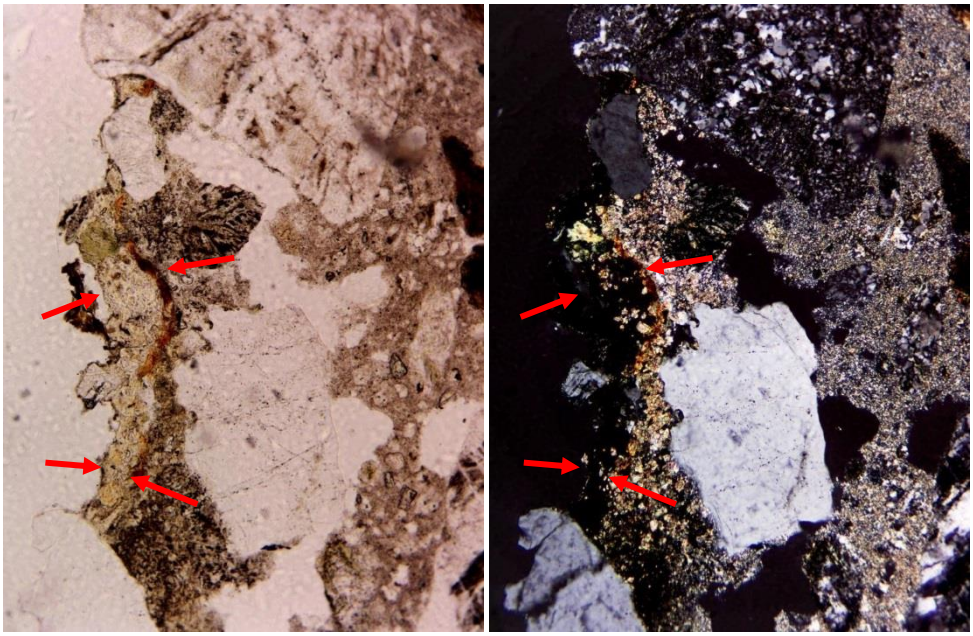


9a. Transmitted, plane-polarized light.

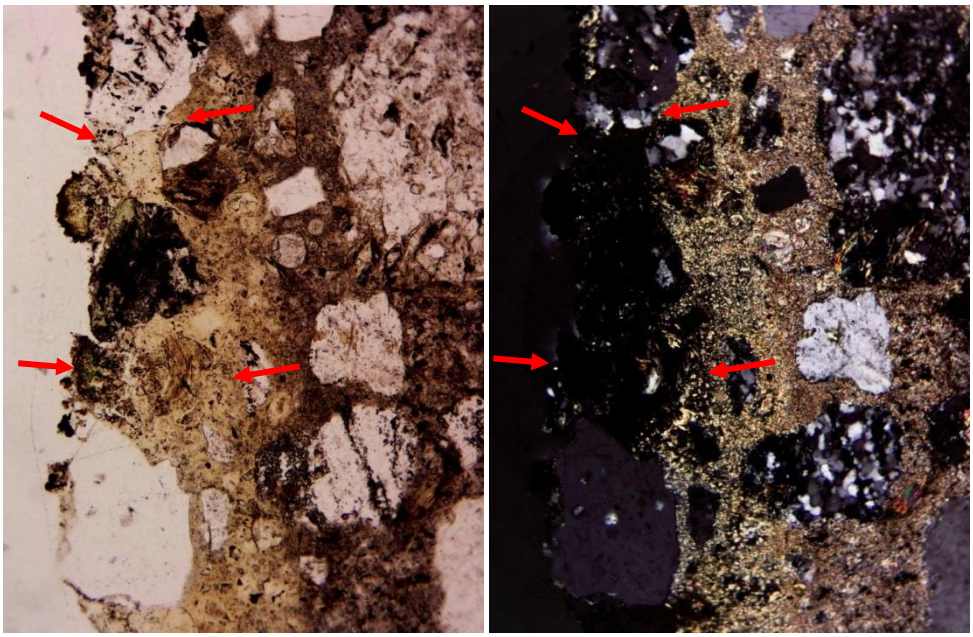


9b. Same area as above, viewed with partially-crossed, polarized light.

Figure 9. **Photomicrographs of the upper approximate 2 mm (0.08 in.) of Core C-4 concrete in thin section shows carbonation of the cement paste (marked with yellow arrows) and a thin zone of intensely leached paste (red arrows). Also note the profile of the top surface with erosion of the cement paste between exposed sand aggregate grains. Length of field, left to right, is approximate 3 mm (0.12 in.).**

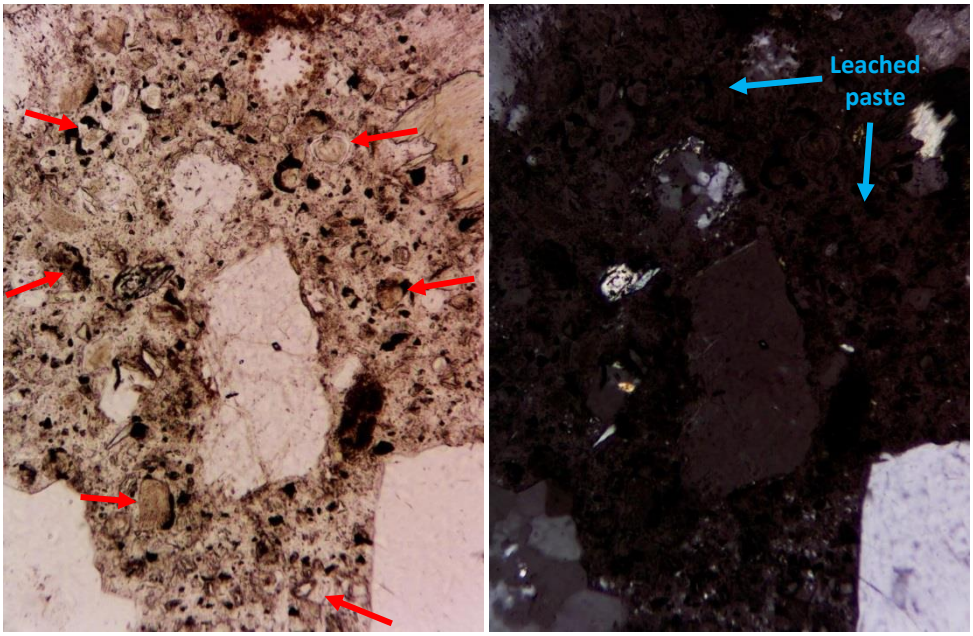


10a. Core C-2. 100x magnification.

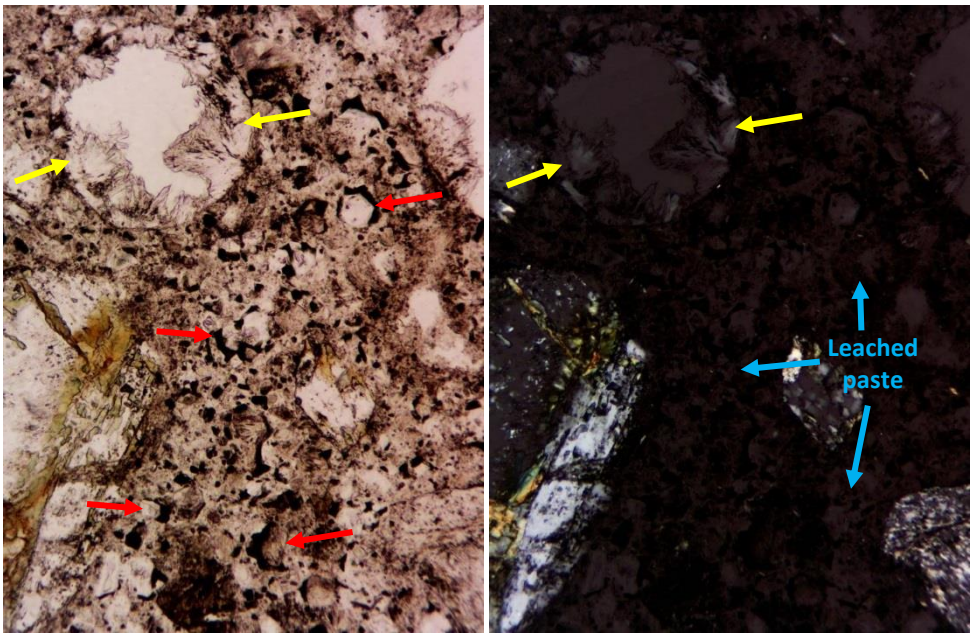


10b. Core C-4. 100x magnification.

Figure 10. **Thin section photomicrographs showing the intensely leached surface paste (red arrows) and underlying carbonated paste previously depicted on Figures 8 and 9 at greater magnification. This surface paste is severely degraded and soft. Each image pair is shown in plane-polarized light (left) and partially crossed polarized light (right). Length of field, left to right, for each image is 0.8 mm (0.03 in.).**

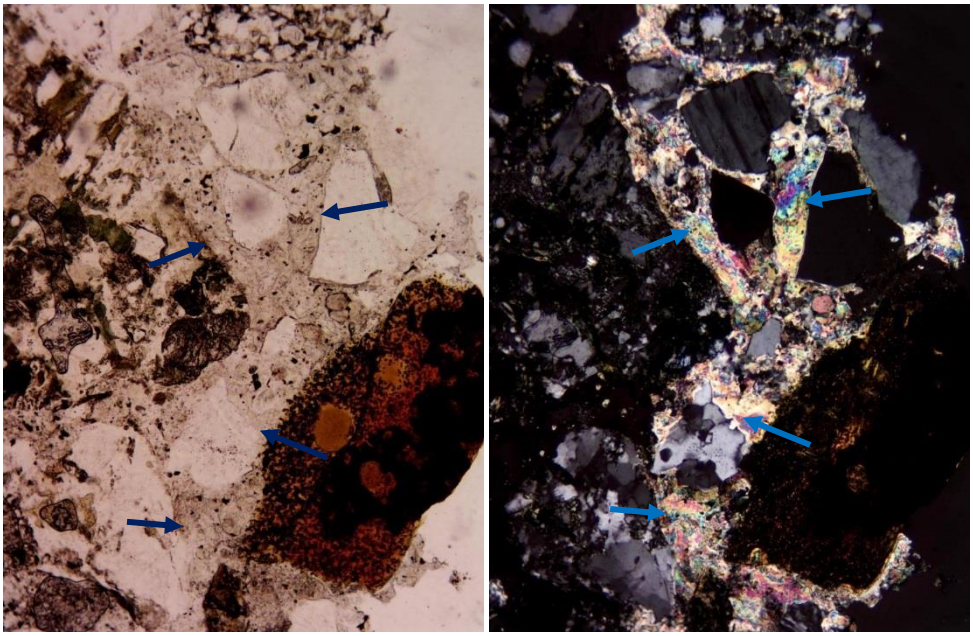


11a. Core C-2. 200x magnification.

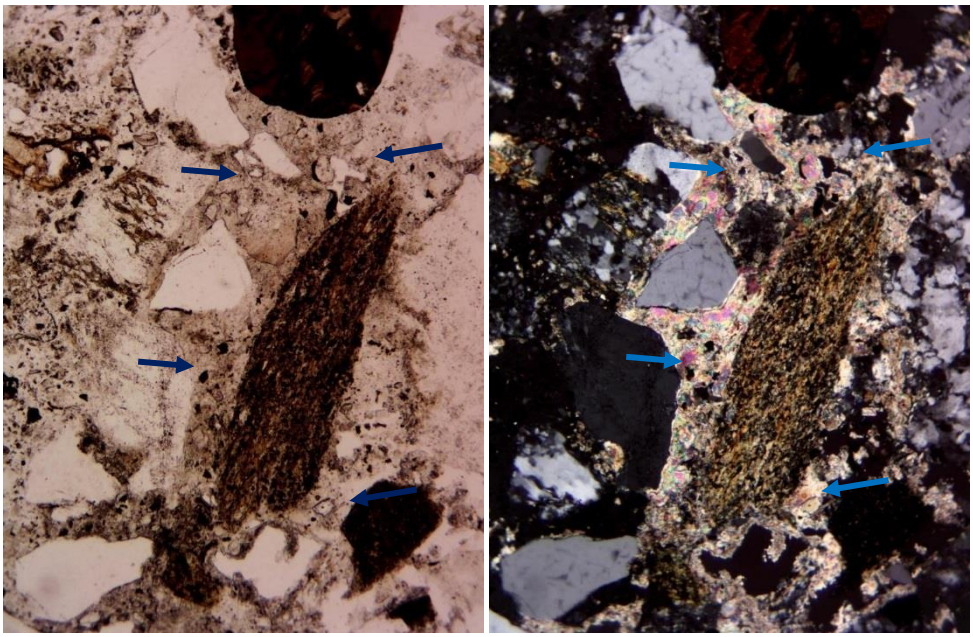


11b. Core C-4. 200x magnification.

Figure 11. **Thin section photomicrographs of cement paste in the body of each core that exhibits low crystallinity and is probably leached of soluble calcium. Despite the leaching, paste properties do not appear to be significantly degraded and the paste still exhibits high pH. Red arrows mark "ghost relics" of hydrated portland cement clinker particles. Yellow arrows mark secondary deposits of ettringite in a void. Each image pair is shown in plane-polarized light (left) and partially crossed polarized light (right). Length of field, l. to r., for each image is 0.4 mm (0.016 in.).**



12a. Core C-2. 100x magnification.



12b. Core C-4. 100x magnification.

Figure 12. **Thin section photomicrographs of concrete near the bottom end of each core that shows localized replacement of the cement paste with coarsely crystalline calcite (blue arrows). Each image pair is shown in plane-polarized light (left) and partially crossed polarized light (right). Length of field, l. to r., for each image is 0.8 mm (0.03 in.).**

Petrographic Examination of Hardened Concrete - ASTM: C 856

Client:
Tourney Consulting Group
Kalamazoo, MI

Project Description:
Petrographic Examination of Concrete Cores from Sedimentation Basin Slabs, City of Rohnert Park, CA WWTP
TCG Project No. 16118

Sample Information — Sample: Core C-2

Sample Location: Not reported.

Placement Date: 1970.

Reported Distress Observed: Deterioration and erosion of top surface. Condition assessment of concrete.

General Observations

Sample Dimensions:

Analysis was performed on a saw-cut, full-depth portion of a 143-mm (5.6-in.) diameter, cylindrical core. The piece is a slice, approximately 50 to 56 mm (2.0 to 2.2 in.) wide and 61 to 70 mm (2.4 to 2.7 in.) deep saw-cut roughly from the middle of the core (Figure 1).

Surface Conditions

Top End: Level but rough and eroded concrete surface. Deterioration and erosion of the cement paste exposes numerous fine aggregate particles in low to moderate relief. Exposed cement paste is deteriorated and varies in hardness from moderately hard to soft and friable. Some of the cement paste is discolored (brown). Exposed grains of sand are intact to broken and also exhibits some brown discoloration. This discoloration is likely due to exposure to biological matter in the waste water/effluent.

Bottom End: Somewhat rough and uneven surface of concrete apparently placed over granular subbase (sand). The core was received with some sand still attached to the surface. Exposed cement paste varies in hardness from moderately hard to moderately soft. Some loss of concrete and possible deterioration is noted along the bottom edges and surface.

General Physical Conditions:

Aside from the aforementioned deterioration and erosion of the top surface, the body of concrete appears to be in fairly good condition, with no visible cracks or large entrapped air voids. The concrete appear well consolidated.

Reinforcement:
A section of welded wire fabric (mesh) is embedded in the concrete in the lower portion of the sample, with 49 to 52 mm (1.9 to 2.0 in.) of cover from the existing top end of the core. Freshly exposed surfaces of the wires exhibit minor surface corrosion and production of brown, ferruginous corrosion products.

Aggregate

Coarse: No coarse aggregate present.

Fine: Natural sand grains composed of a variety of siliceous igneous, metamorphic and meta-sedimentary rock types, including variety of fine-grained altered, and silicified volcanic rocks, quartzite, gneiss and schist, clastic arenites, micro- and cryptocrystalline quartz (probably chert and/or meta-chert), basalt, glassy and devitrified volcanic glass, serpentinite, grains of feldspar and quartz, and other siliceous rocks and minerals. Fine aggregate particles are rounded to sub-angular, mostly equant to slightly elongate, and exhibit smooth to semi-rough surfaces. Sand appears slightly coarse but otherwise fairly well graded to an observed top size of 3 mm (0.12 in.). Grains are somewhat unevenly distributed in the concrete, with localized patterns of grain alignment and possible relic flow patterns.

Paste

Depth of Carbonation	10 to 13 mm (0.4 to 0.5 in.) along the top end of the core. Up to 10 mm (0.4 in.) of reduced pH and possible carbonation along the bottom end. Cement paste in the latter case appears to have been altered and partially replaced with coarsely crystalline calcium carbonate.
Air Content	Estimated 1 to 2 percent, averaged in the body of the core. The concrete does not appear to be intentionally air entrained, based on the scarcity and low volume of small, spherical voids, less than 1 mm, in the cement paste.
Paste Color	Mottled light-medium grey throughout most of the body of the core; light buff-gray in most of carbonated upper 10 to 13 mm (0.4 to 0.5 in.); beige to light brown in top 3 to 5 mm (0.12 to 0.20 in.) and along the outer surface.
Paste Hardness	Moderately hard throughout most of the body of the core; gradually softening in carbonate upper region to moderately soft to soft and friable in top 1 to 3 mm (0.04 to 0.12 in.). Also non-uniformly softer in bottom 1 to 3 mm (0.04 to 0.12 in.).
Paste Absorbency	Moderate throughout body of core. Drops of water are readily accepted onto freshly broken surfaces of the concrete and absorbed into the cement paste at a slow to moderate rate. Cement paste become increasingly more absorbent in the upper approximate 10 mm (0.4 in.) to very absorbent in the top 1 to 3 mm (0.04 to 0.12 in.) and along top surface.
Paste-Aggregate Bond	Moderately tight in the body of the core; surfaces of freshly fractured concrete pass through and around fine aggregate particles. Bond become increasingly weaker in the upper approximate 10 mm (0.4 in.) to weak in the top 1 to 3 mm (0.04 to 0.12 in.).

Residual Cement	Small amounts of residual (unhydrated and partially hydrated) portland cement clinker particles are observed microscopically in the cement paste in thin section. However, "ghost" relics of suspected in-situ hydrated cement clinker are common to abundant. The cement paste appears to be in an advanced state of hydration.
Supplementary Cementitious Materials	No fly ash observed. A few cement-sized particles of glass-like material are observed in the cement paste; however, given the presence of glassy rock in the aggregate, it is unclear as to whether such particles represent constituents of the fines or dust in the aggregate or a ground slag-like material (probably the former).
Calcium Hydroxide	Portlandite crystal are absent in both non-carbonated and carbonated cement paste.
Water-Cement Ratio (w/cm):	Estimated moderate, based on some observed physical properties and microscopical features of the concrete and cement paste, most notably the hardness of the cement paste. However, due to the apparent continued hydration and possible alteration and leaching of the cement paste, a more defined estimate of w/c is not given.
Microcracking	Microcracks are generally sparse in the cement paste in the body of the core, but more common in the weaker concrete at both the bottom and top ends of the core.
Secondary Deposits	Minor amounts of ettringite (calcium sulfo-aluminate hydrate) are noted as partial linings in some air voids in the non-carbonated portion of the core. Coarsely crystalline calcium carbonate is observed in the bottom few millimeters of concrete, apparently as both secondary deposits and as a replacement for some of the cement paste. Traces of clear gel-like material, probably alkali-silica reaction gel, is observed in small voids and gaps in the cement along periphery of a few siliceous aggregate particles.

Examination By:

Ronald D. Sturm
Senior Petrographer

Petrographic Examination of Hardened Concrete - ASTM: C 856

Client:
Tourney Consulting Group
Kalamazoo, MI

Project Description:
Petrographic Examination of Concrete Cores from Sedimentation Basin Slabs, City of Rohnert Park, CA WWTP
TCG Project No. 16118

Sample Information — Sample: Core C-4

Sample Location: Not reported.

Placement Date: 1970.

Reported Distress Observed: Deterioration and erosion of top surface. Condition assessment of concrete.

General Observations

Sample Dimensions:

Analysis was performed on a saw-cut, full-depth portion of a 143-mm (5.6-in.) diameter, cylindrical core. The piece is a slice, approximately 52 mm (2.0 in.) wide and 55 to 63 mm (2.2 to 2.5 in.) deep saw-cut roughly from the middle of the core (Figure 2).

Surface Conditions

Top End: Level but rough and eroded concrete surface. Deterioration and erosion of the cement paste exposes numerous fine aggregate particles in low relief. Exposed cement paste is deteriorated and varies in hardness from moderately hard to soft and friable. Some of the cement paste is discolored (brown). Exposed grains of sand are intact to broken and also exhibits some brown discoloration. Again, this discoloration is likely due to exposure to biological matter in the waste water/effluent.

Bottom End: Somewhat rough and uneven surface of concrete apparently placed over granular subbase (sand). The core was received with some sand still attached to the surface. Exposed cement paste varies in hardness from moderately hard to moderately soft. Some loss of concrete and possible deterioration is noted along the bottom edges and surface.

General Physical Conditions:

Aside from the aforementioned deterioration and erosion of the top surface, the body of concrete appears to be in fairly good condition, with no visible cracks or large entrapped air voids. The concrete appear well consolidated.

Reinforcement:
A section of welded wire fabric (mesh) is embedded in the concrete in the upper portion of the sample, with 10 to 13 mm (0.4 to 0.5 in.) of cover from the existing top end of the core. Freshly exposed surfaces of the wires exhibit minor surface corrosion and meager production of brown, ferruginous corrosion products.

Aggregate

Coarse: No coarse aggregate present.

Fine: Natural sand grains composed of a variety of siliceous igneous, metamorphic and meta-sedimentary rock types, including variety of fine-grained altered, and silicified volcanic rocks, quartzite, gneiss and schist, clastic arenites, micro- and cryptocrystalline quartz (probably chert and/or meta-chert), basalt, glassy and devitrified volcanic glass, serpentinite, grains of feldspar and quartz, and other siliceous rocks and minerals. Fine aggregate particles are rounded to sub-angular, mostly equant to slightly elongate, and exhibit smooth to semi-rough surfaces. Sand appears slightly coarse but otherwise fairly well graded to an observed top size of 3 mm (0.12 in.). Grains are somewhat unevenly distributed in the concrete, with localized patterns of grain alignment and possible relic flow patterns.

Paste

Depth of Carbonation	5 to 10 mm (0.2 to 0.4 in.) along the top end of the core. Up to 10 mm (0.4 in.) of reduced pH and possible carbonation along the bottom end. Cement paste in the latter case appears to have been altered and partially replaced with coarsely crystalline calcium carbonate.
Air Content	Estimated 1 to 2 percent, averaged in the body of the core. The concrete does not appear to be intentionally air entrained, based on the scarcity and low volume of small, spherical voids, less than 1 mm, in the cement paste.
Paste Color	Mottled light-medium grey throughout most of the body of the core; light buff-gray in most of carbonated upper 5 to 10 mm (0.2 to 0.4 in.); beige to light brown in top 3 to 5 mm (0.12 to 0.20 in.) and along the outer surface.
Paste Hardness	Moderately hard throughout most of the body of the core; gradually softening in carbonate upper region to moderately soft to soft in top 1 to 2 mm (0.04 to 0.08 in.). Also non-uniformly softer in bottom 1 to 2 mm (0.04 to 0.08 in.).
Paste Absorbency	Moderate throughout body of core. Drops of water are readily accepted onto freshly broken surfaces of the concrete and absorbed into the cement paste at a slow to moderate rate. Cement paste become increasingly more absorbent in the upper approximate5 mm (0.2 in.) to very absorbent in the top 1 to 2 mm (0.04 to 0.08 in.) and along top surface.

Paste-Aggregate Bond	Moderately tight in the body of the core; surfaces of freshly fractured concrete pass through and around fine aggregate particles. Bond become increasingly weaker in the upper approximate 5 to 10 mm (0.2 to 0.4 in.) to weak in the top 1 to 3 mm (0.04 to 0.12 in.).
Residual Cement	Small amounts of residual (unhydrated and partially hydrated) portland cement clinker particles are observed microscopically in the cement paste in thin section. However, “ghost” relics of suspected in-situ hydrated cement clinker are common to abundant. The cement paste appears to be in an advanced state of hydration.
Supplementary Cementitious Materials	No fly ash observed. Again, a few cement-sized particles of glass-like material are observed in the cement paste; however, given the presence of glassy rock in the aggregate, it is unclear as to whether such particles represent constituents of the aggregate fines or a ground slag-like material (probably the former).
Calcium Hydroxide	Portlandite crystal are absent in both non-carbonated and carbonated cement paste.
Water-Cement Ratio (w/cm):	Estimated moderate, based on some observed physical properties and microscopical features of the concrete and cement paste, most notably the hardness of the cement paste. However, due to the apparent continued hydration and possible alteration and leaching of the cement paste, a more defined estimate of w/c is not given.
Microcracking	Microcracks are generally sparse in the cement paste in the body of the core, but more common in the weaker concrete at both the bottom and top ends of the core.
Secondary Deposits	Minor amounts of ettringite (calcium sulfo-aluminate hydrate) are noted as partial linings in some air voids in the non-carbonated portion of the core. Coarsely crystalline calcium carbonate is observed in the bottom few millimeters of concrete, apparently as both secondary deposits and as a replacement for some of the cement paste. Traces of clear gel-like material, probably alkali-silica reaction gel, is observed in small voids and gaps in the cement along periphery of a few siliceous aggregate particles.

Examination By:

Ronald D. Sturm
Senior Petrographer

Detailed Petrographic Analysis:

The petrographic report speaks well to the condition of the concrete. Overall, the concrete is more associated as a mortar or grout. No coarse aggregates were present. The degradation was more associated to an acidic and/or chemical attack from plant processes. The paste of this grout was attacked but the attack/softening was rather shallow (<3mm) but likely progressive. The slab is very thin and the continued degradation will lead to structural performance of the slab for the purpose intended. There seems to be a decalcification from leaching over time. This is more apparent near the surface and extends deep into the slab.

Conclusion:

The slabs are experiencing degradation from chemical attack. The attack seems to be slow. The grout is rather hard and sound.

Thank you for allowing Tourney Consulting Group to conduct these condition assessment services. Please call me at (269)384-9980 or my mobile at (269)370-1054 to discuss any issue.

Sincerely,

Paul G. Tourney, P.E.
Tourney Consulting Group

Appendix B - CCTV Investigation Report; Miksis Services Inc.; October 31, 2016

Emergency Sewer Pond and Wet Well Lining Project - CCTV Inspections

Pipe Segment	CCTV Video File	Video Duration	Start HM	Stop MH	Camera Direction	Pipe Diameter (in)	Pipe Material	Surveyed length (ft)	Structural Defects		Maintenance Defects			Notes
									Joint Seperatio	Surface Damage	Deposit	Grease	Root	
1	01 MH 1_Headworks_Downstream_20161031_2429	3:22	MH 1	Headworks	Downstream	18	ACP	53.5	X		X			1. Continious grease and/or silt attached at invert from MH to 11 ft. 2. There is an obstruction at pipe invert near 11 ft, could be a steel wheel. 3. Pipe joint at 15.5 ft - small gap. 4. Pipe joint at 33.5 ft.
2	02 MH 2_MH 1_Upstream_20161031_2430	5:24	MH1	MH2	Upstream	18	ACP	94.1	X					1. Pipe joint at the beginning, 19 ft, 36.9 ft, 54.5 ft, 72.4 ft and 86.6 ft. - all with small gap.
3	03 Pond Return_MH 2_Upstream_20161031_2431	11:21	MH2	Pond Return	Upstream	18	ACP	441.2	X	X		X	X	1. The camera is pulled backward inside the pipe. Pipe Joint at 439.5 ft, 435.1 ft, 430.6 ft, 418.1 ft, 416.6 ft, 406.0 ft, 394.5 ft, 382.3, 370.4 ft, 359 ft, 346 ft, 334.6 ft, 310.8 ft, 298.8 ft, 286.6 ft, 274.9 ft, 262.9 ft, 251.3 ft, 238.7 ft, 232.3 ft, 227.0 ft, 215.0 ft, 207.8 ft, 182.6 ft, 170.6 ft, 159.1 ft, 146.9 ft, 135.3 ft appears to be fine. and 2. surface deterioration at 437.8 ft. 3. Pipe left bend at 427.0 ft. 4. The water mark is getting higher after 360.7 ft toward upstream until 310.8 ft ft, which appears to be a small sag in the pipeline. 5. The water mark is getting higher after 251.3 ft toward upstream until 232.7 ft, which appears to be a medium sag in the pipeline. 6. Pipe right bend at 216.0 ft. 7. Pipe joint at 206.7 ft, 190 ft, 98.9 ft, 91.0 ft, 79.0 ft, 42.5 ft, 24.8 ft and 9.1 ft - small offset. 8. Pipe joint at 201.5 ft - small offset with root intrusion and grease deposit. 9. Pipe joints at 123.4 ft and 60.8 ft - small gap 10. Continiuous root intrusion annd surface damage at 115.4 ft - medium.
4	04 Valve by office_MH 2_Upstream_20161031_2432	2:47	Valve by office	MH 2	Upstream	8	ACP	64.5	X					1. Pipe joint at 5.0 ft, 22.9 ft, 40.9 ft and 59.1 ft - small offset.

PO Box 591-I, Healdsburg, CA 95448-0590
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www.unplugu.com admin@unplugu.com
10/31/2016

GHD, Inc.
Emergency Sewer Pond & Wet Well Lining
J. Rogers Lane Pump Station, Rohnert Park

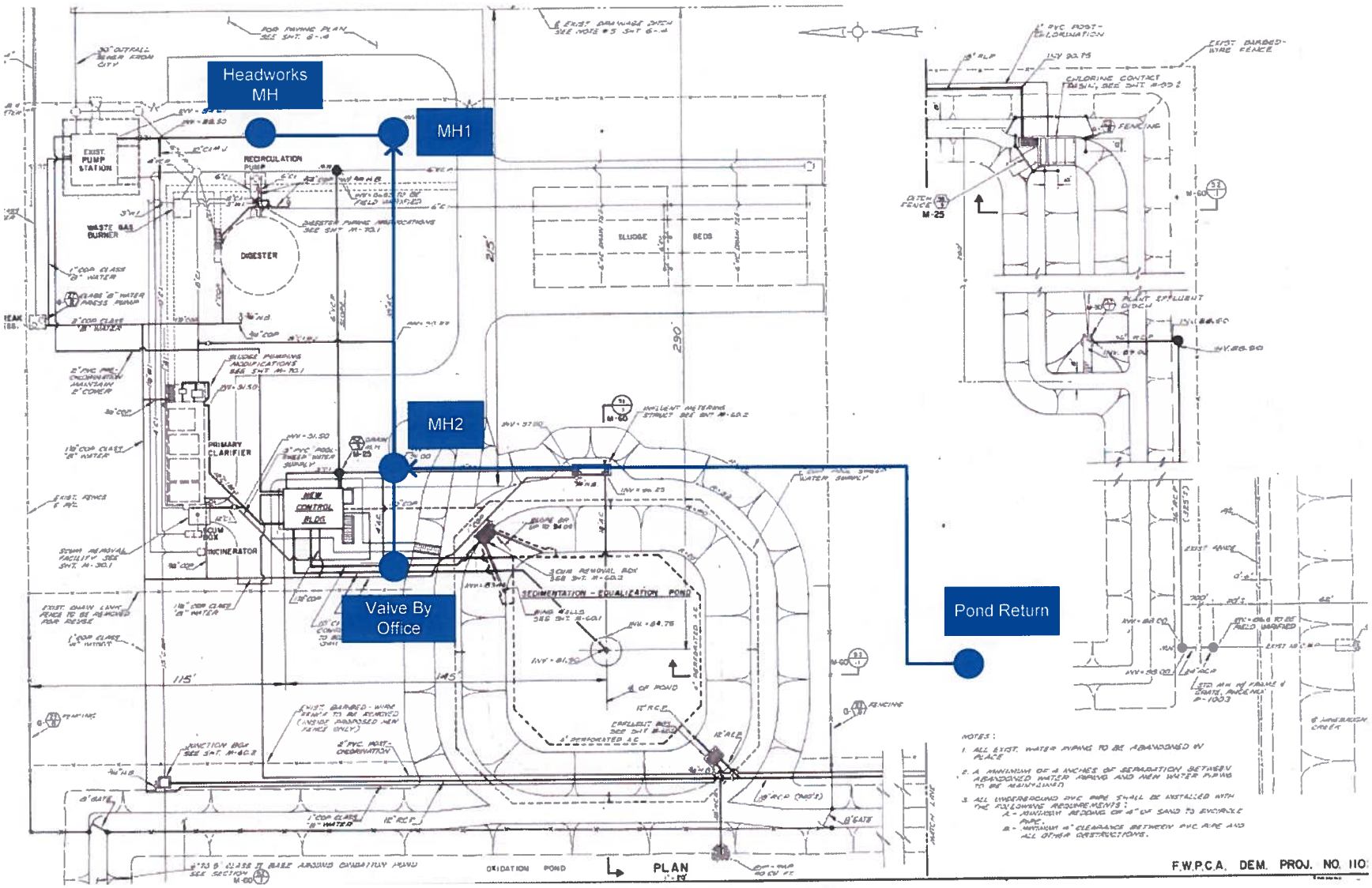
Insp #	Video	Street	US MH	DS MH	Direction of Camera	Size	Mat'l	TV'd Length	Comment
1	**2429**	J Rogers Lane	MH 1	Headworks	D	18	AC	53.5	Complete
2	**2430**	J Rogers Lane	MH 2	MH 1	U	18	AC	94.1	Complete
3	**2431**	J Rogers Lane	Pond Return	MH 2	U	18	AC	441.3	Complete- Camera records inspection during pull back at Pond return to MH2. Pulled back to MH2
4	**2432**	J Rogers Lane	Valve by office	MH 2	U	8	AC	64.5	Complete

MIKSIS SERVICES INC.

GHD, Inc.
Emergency Sewer Pond & Wet Well Lining
J. Rogers Lane Pump Station

Job Site:

201 J. Rogers Lane
Rohnert Park, CA



Map not to scale - for illustration purposes only

MIKSIS SERVICES INC.

PIPELINE VIDEO INSPECTION

Client
GHD, Inc. Emergency Sewer Pond & Wet Well Lining

Date	10/31/2016
Job #	3209
Tech	Greg

Address:	201 J Rogers Ln, Rohnert Park, CA	Pipe Material: AC	Pipe Size: 18"
		Camera Direction: ☐ Upstream	☑ Downstream
Scope:	Inspection 1 Condition Assessment	Camera Type: ☐ Launch	☐ Push
		☐ Cleaned	☑ Mainline Camera

VIDEO Footage		Remarks
0'	2429	Start inspection at MH1 heading downstream to Headworks MH. Silt visible in pipe.
9'		End of silt
11'		Debris in pipe, possible steel wheel
53'		Inspection complete at Headworks MH. Staining in manhole.

Status/Recommendations:

MIKSIS SERVICES INC.

GHD, Inc.
Emergency Sewer Pond & Wet Well Lining

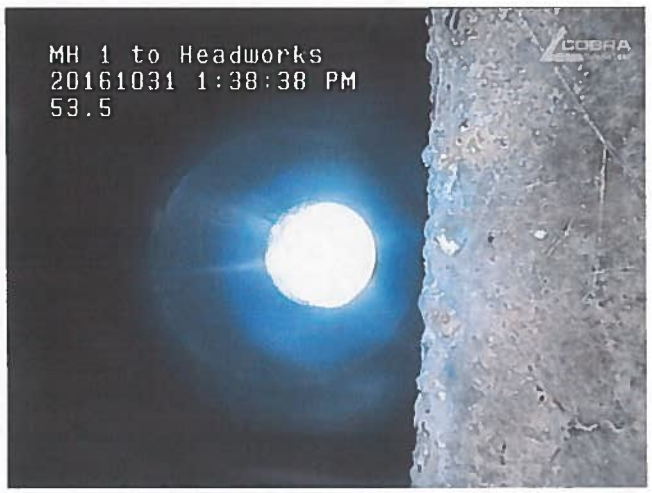
201 J Rogers Ln, Rohnert Park, CA
10/31/2016

Pipe Segment: MH 1 to Headworks

Pipe Size: 18"	Material: Asbestos	Direction: DS	Total Length: 53.5'

0.0' Silt on bottom of pipe

11' Possible steel wheel in pipe



53.5' Completed inspection at manhole

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MIKSIS SERVICES INC.

PIPELINE VIDEO INSPECTION

Client		
GHD, Inc. Emergency Sewer Pond & Wet Well Lining	Date	10/31/2016
	Job #	3209
	Tech	Greg

Address:	201 J Rogers Ln, Rohnert Park, CA	Pipe Material: AC	Pipe Size: 18"
		Camera Direction: ☑ Upstream	☐ Downstream
Scope:	Inspection 2 Condition Assessment	Camera Type: ☐ Launch	☐ Push
		☐ Cleaned	☑ Mainline Camera

VIDEO Footage		Remarks
0'	2430	Start inspection at MH1 heading upstream towards MH2.
94.1'		Inspection complete at MH2.

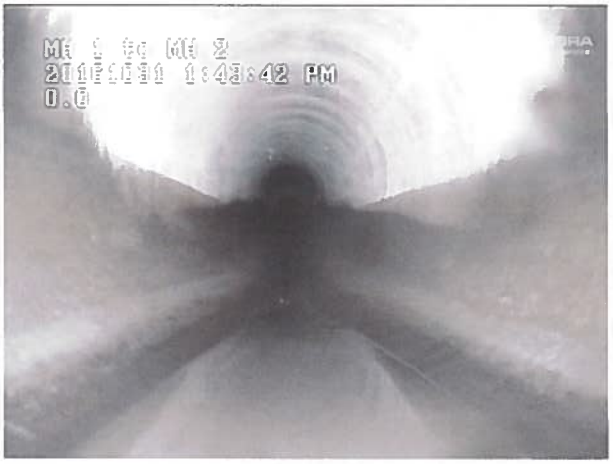
Status/Recommendations:

GHD, Inc.
Emergency Sewer Pond & Wet Well Lining

201 J Rogers Ln, Rohnert Park, CA
10/31/2016

Pipe Segment: MH2 to MH1

Pipe Size: 18"	Material: Asbestos	Direction: US	Total Length: 94.1'



0.0' Start inspection

19.0' Stains noted at the joints throughout the inspection

94.1' Completed inspection at Manhole 2

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MIKSIS SERVICES INC.

PIPELINE VIDEO INSPECTION

Client		
GHD, Inc. Emergency Sewer Pond & Wet Well Lining	Date	10/31/2016
	Job #	3209
	Tech	Greg

Address:	201 J Rogers Ln, Rohnert Park, CA	Pipe Material: AC	Pipe Size: 18"
		Camera Direction: ☑ Upstream	☐ Downstream
Scope:	Inspection 3 Condition Assessment	Camera Type: ☐ Launch	☐ Push
		☐ Cleaned	☑ Mainline Camera

VIDEO Footage		Remarks
441.3'	2431	Camera entered pipe from MH2 and ran up to Pond Return location. Inspection started at 441.3' at Pond Return location recording video as camera was pulled back to MH2.
437'		Surface spauling 12 o'clock- stain at joint
428'		Pipe bends left
361'-307'		Minor sag in pipe, 5%-10% standing water level
260'		Stain at joint at 10 o'clock
250'-226'		Sag in pipe, standing water levels fluctuate from 5%-25%
218'		Pipe bends right
206'		Offset joint
201'		Unknown obstruction in joint at 5 o'clock
116'		Fine roots in pipe
114'		Faulty seal intruding into joint and root growth at faulty seal. Alignment change.
7.4'		End of inspection at camera preset near MH2.

Status/Recommendations:

MIKSIS SERVICES INC.

GHD, Inc.
Emergency Sewer Pond & Wet Well Lining

201 J Rogers Ln, Rohnert Park, CA
10/31/2016

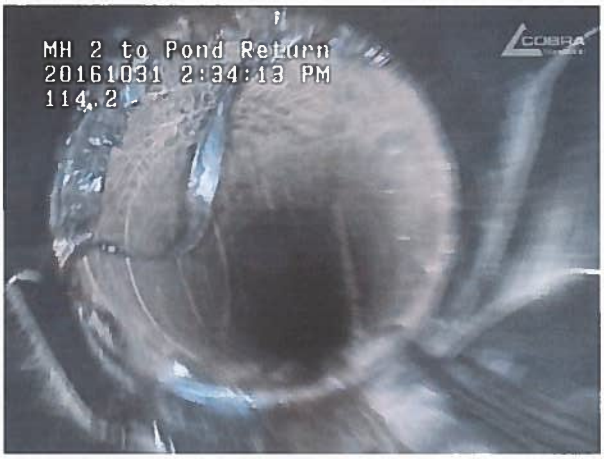
Pipe Segment: MH 2 to Pond Return

Pipe Size: 18"	Material: Asbestos	Direction: US	Total Length: 441.3'

438.1' Start inspection pulling camera backwards from MH 2

237.2' Standing water

206.1' Offset Joint



114.2' Faulty seal at the joint with roots at joint

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MIKSIS SERVICES INC.

GHD, Inc.
Emergency Sewer Pond & Wet Well Lining

201 J Rogers Ln, Rohnert Park, CA
10/31/2016

Pipe Segment: MH 2 to Valve near Office

Pipe Size: 8"	Material: Asbestos	Direction: US	Total Length: 64'

5.0' Stained pipe

14.2' Stained pipe

59.3 End of Inspection

MIKSIS SERVICES INC.

PIPELINE VIDEO INSPECTION

Client		
GHD, Inc. Emergency Sewer Pond & Wet Well Lining	Date	10/31/2016
	Job #	3209
	Tech	Greg

Address:	201 J Rogers Ln, Rohnert Park, CA	Pipe Material: AC	Pipe Size: 8"
		Camera Direction: ☑ Upstream	☐ Downstream
Scope:	Inspection 4 Condition Assessment	Camera Type: ☐ Launch	☐ Push
		☐ Cleaned	☑ Mainline Camera

VIDEO Footage		Remarks
0'	2432	Start inspection at MH2 heading upstream towards Valve by Office
64'		Inspection complete at Valve by Office

Status/Recommendations:

Appendix C - Rohnert Park Pump Station Wet Well Condition Assessment; V&A Consulting Engineers, Inc. December 8, 2016

ROHNERT PARK PUMP STATION
WET WELL CONDITION ASSESSMENT

Prepared for: Matt Winkelman, P.E.
GHD
2235 Mercury Way, Suite 150
Santa Rosa, CA 95407

Date: December 8, 2016

Prepared by: V&A

V&A Project No. 16-0205

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APPENDICES

Appendix A. Documentation of Metallic, Mechanical, Electrical, and Other Items

1.0 INTRODUCTION

V&A Consulting Engineers, Inc. (V&A), was retained by GHD to perform a condition assessment of the wet wells at the Rohnert Park Pump Station. The Rohnert Park Pump Station is located within the City of Rohnert Park, California (City), and it collects all of the City's wastewater flow for transmission to its wastewater treatment plant.

The Rohnert Park Pump Station consists of two individual wet well/dry well pump stations adjacent to one another on the pump station site. Pump Station 1 (PS1) is located in the northern part of the facility and was constructed in the 1950s. The interior surfaces of the wet well are concrete, which was originally lined. Pump Station 2 (PS2) is located to the south of PS1 and was constructed in the early 1990s. PS2 additionally includes a grinder chamber, which contains hydraulically powered grinders to chop up wastewater solids, upstream from the wet well. The interior surfaces of the wet well and grinder chamber are unlined concrete. A diversion structure adjacent to PS1, also constructed of unlined concrete, was also evaluated. The PS1 diversion structure takes in flow from the sewer line entering the pump station site and distributes it to PS1 and PS2.

The intent of V&A's condition assessment activities was to determine the condition of the wet well concrete and pump suction bells with respect to corrosion and to develop recommendations for rehabilitation. The following methods were employed for the condition assessment:

- Visual and qualitative assessment to document overall conditions.
- Concrete pH measurement, concrete penetration depth testing, and concrete soundings to assess degradation of the concrete surfaces.
- Surface penetrating radar scans to measure the depth of concrete cover over the reinforcing bars.
- Ultrasonic thickness testing to determine the remaining wall thickness of the pump suction bells, if necessary per the evaluator's discretion.
- Rating conditions using the VANDA Concrete and Metal Condition Indices.
- Analysis of a coating sample to determine if hazardous materials abatement procedures may be required.
- Documentation of metallic, mechanical, electrical, and other items within the wet wells and PS2 grinder chamber. This also included documentation of the interior of the diversion structure just upstream from PS1.

V&A conducted confined space entries into the PS1 wet well on October 24, 2016, and the PS2 wet well and grinder chamber on November 2, 2016.

2.0 METHODS AND PROCEDURES

2.1 Access and Confined Space Entry

V&A conducted a confined-space dropdown entry into the lower portion of each structure, below the walkway or landing that was present in each. The confined space entry evaluations were made using precautions including permit procedures, gas monitoring equipment, and appropriate personal protective equipment. A davit arm, winch, and self-retracting lifeline were used for descent and fall protection. City staff ensured that the wet well was drained and cleaned prior to entry. V&A and City staff also conducted lockout/tagout (LOTO) procedures prior to entry to ensure that hazardous sources of energy, such as influent flows, were deactivated as required. The atmospheric conditions were constantly monitored for oxygen, carbon monoxide, hydrogen sulfide, and the lower explosive limit throughout the entries.

2.2 Visual/Qualitative Evaluation

The primary investigative method was to conduct visual examinations supplemented with digital photographs. The visual assessment focused on the condition of reinforced concrete surfaces, coatings, and the pump suction bells. Defects such as cracks, spalls, exposed aggregate, corrosion of reinforcing steel, and other concrete defects were documented with digital photographs. The assessments are subjective in nature and are based on V&A's extensive experience evaluating concrete and steel structures in the water and wastewater industry.

2.3 Concrete pH Measurement

Within the wet wells and grinder chamber, V&A conducted several in-situ pH measurements to determine the pH of the concrete exposed to the wastewater environment.

Cementitious mortars are generally made from a combination of aggregate, sand and Portland cement. The Portland cement in mortar has a pH of approximately 13.5 after curing. This elevated pH level provides corrosion control for the steel. Steel will transform from a state of active corrosion to a state of passivity, which is characterized by a thin layer of iron oxide that protects the steel from corrosion, when the steel surface is exposed to a pH greater than 10. At a pH less than 10, corrosion is possible. V&A has developed a table correlating the effect of the pH of the environment on the rate of corrosion of concrete. The data in Table 2-1 is derived from past experience and review of literature, such as American Concrete Institute (ACI) 201.2R-92, "Guide to Durable Concrete."

Table 2-1. pH and Corrosivity Correlation for Reinforced Concrete

pH	Degree of Corrosivity
< 7	Severe
7 to 9	Moderate
9 to 11	Mild
>11	Negligible

2.4 Soundings

Soundings are performed using a chipping hammer to strike the concrete surfaces of a structure. The sound from the strike can indicate discontinuities within the surface. The sound returned from sound concrete without subsurface voids is a solid "ping" noise. A "hollow" sound generally means that a void or discontinuity exists beneath the sounding location. A soft "thud" typically results from deteriorated concrete and soft cement paste. V&A conducted soundings at several locations on the interior walls of the wet wells and grinder chamber to listen for concrete surface delaminations. Soundings were also performed in areas of concern such as visible cracks and spalls.

2.5 Penetration Depth Measurements

Penetration depth measurements involve applying a constant force from a chipping hammer to the concrete surface, until sound, hard material is reached, and then measuring the depth of the resulting cavity. The cavity depth provides quantitative data on the integrity and condition of the concrete surfaces. Typically, as concrete deteriorates, the cement paste begins to lose integrity and becomes soft.

2.6 Surface Penetrating Radar (SPR)

A surface penetrating radar (SPR) unit was used to measure the depth and spacing of reinforcing steel at selected locations within each wet well. At each location, a minimum of two perpendicular scans, typically between 3 and 5 feet long each, were performed. The portable wheel-mounted unit is rolled across the surface to be investigated. A radar beam scans up to 16 inches into the concrete and generates a 2-dimensional image of the underlying concrete member. The distance scanned is plotted on the x-axis and the depth scanned on the y-axis. Figure 2-1 shows a sample graphical image of the SPR scan. Automatic and manual procedures were used to locate the depth of the reinforcing steel on the resulting plots.

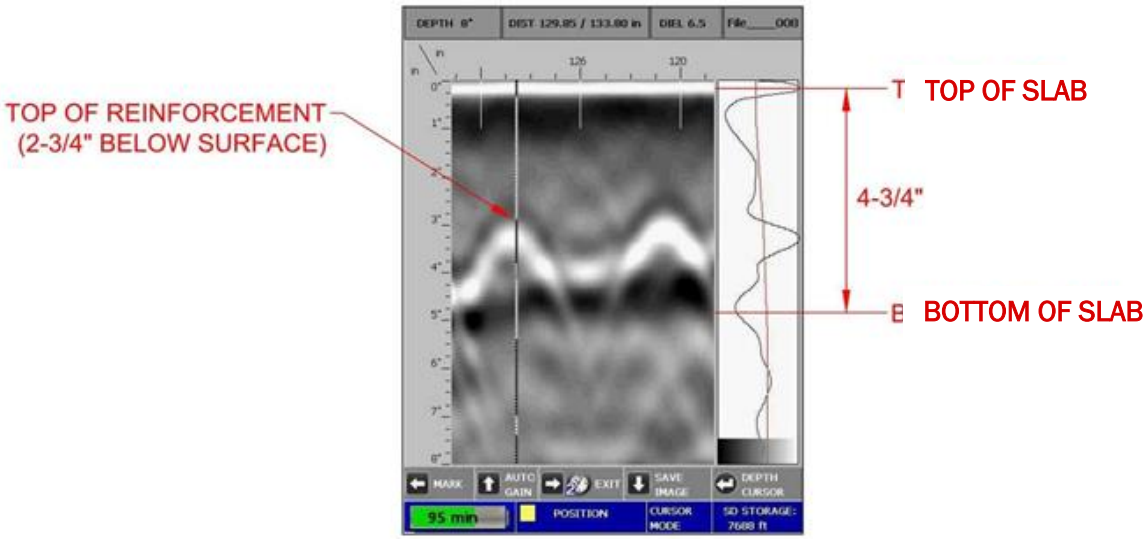

Figure 2-1. Sample SPR Scan

Concrete cover depth is an important element in corrosion protection of reinforced concrete structures. The greater the thickness of concrete cover, the less likely that corrosive constituents have reached the embedded reinforcing steel.

According to ACI 350-06, "Code Requirements for Environmental Engineering Concrete Structures," the minimum depth of concrete cover for corrosion protection of reinforcing steel in water-retaining structures should be 2 inches. In formed concrete surfaces exposed to earth, water, sewage, weather, or in contact with the ground, the minimum depth to reinforcing steel should also be 2 inches. Thus for the structures that were evaluated, the minimum depth of concrete cover over reinforcing steel should be 2 inches.

2.7 Coating Sample Analysis

A coating sample was collected from the interior of the PS1 wet well during the assessment (the PS2 wet well is not coated). The sample was tested for 17 heavy metals, including lead, in accordance with EPA Method 6010B, at Xenco Laboratories, Stafford, Texas. The metals that were tested are listed in Table 2-2. The total threshold limit concentration (TTLC) from CCR Title 22 §66261.24, Table II, is presented for each metal in parts per million (ppm, equivalent to milligrams/kilograms, or mg/kg). The TTLC values are used as a preliminary screening test to plan for hazardous waste disposal procedures and to determine if lead or other hazardous material abatement procedures will be required. If a paint sample exceeds the TTLC limit, any work that is performed on the coating, such as welding or abrasive blasting, will require hazardous material handling for personnel. If the California lead concentration of 600 ppm limit is exceeded, it will require lead abatement procedures per CCR Title 8, §1532.1.

For the removal of lead-based paint, the CCR Title 8, §1532.1 states that worker safety measures must be implemented when the lead concentration in the coating exceeds 600 ppm or 0.06% by weight. The requirement for worker health and safety measures during the removal of a lead-based paint depends on the amount of airborne lead to which the workers will be exposed during a specified length of time as outlined in CCR Title 8 §1532.1. It states that "...the employer shall not expose an employee to lead at a concentration greater than 30 micrograms per cubic meter of air ($\mu g/m^3$) averaged over an 8-hour period." V&A recommends including lead abatement in coating specifications whenever lead concentrations of existing coatings exceed 600 ppm.

The waste that is generated during the abrasive blasting of the surfaces to be coated will have to be tested per Toxicity Characteristic Leaching Procedure (TCLP) before the waste can be removed from the job site. If the TCLP results from the waste exceed the limits of CCR Title 22 §66261.24, Table I, then it must be classified as hazardous waste, requiring an EPA manifest and permit before being disposed of at a hazardous waste disposal site.

Table 2-2. Metals Tested in Paint Samples

Metal	Total Threshold Limit Concentration (ppm)
Antimony	500
Arsenic	500
Barium	10,000
Beryllium	75
Cadmium	100
Chromium	2,500
Cobalt	8,000
Copper	2,500
Lead	1,000
Mercury	20
Molybdenum	3,500
Nickel	2,000
Selenium	100
Silver	500
Thallium	700
Vanadium	2,400
Zinc	5,000

2.8 Ultrasonic Thickness Testing

V&A planned to conduct ultrasonic thickness testing on the pump suction bells if it was warranted. Based on the visual evaluation, ultrasonic testing was not needed and was not conducted. The following description of the method is provided for completeness.

Ultrasonic thickness testing is a non-destructive evaluation technique that allows for the determination of metal wall thickness. High-frequency sound waves are transmitted through one side of a metal wall from a transducer. When sound waves reach the other side of the metal wall, a fraction of the waves will echo back to the transducer. The metal thickness is determined by recording the time it takes for the sound wave to travel through the metal and return.

V&A planned to use the Olympus Epoch XT ultrasonic flaw detector to obtain ultrasonic thickness measurements (Photo 2-1). This device allows more adjustment of the return echo signal waveform than do other ultrasonic thickness gauges, giving greater confidence in the measurements. Prior to taking measurements, the gauge is calibrated to the velocity of sound in ductile iron or steel, as appropriate.

The type of testing that was planned is known as A-scan, which gives the wall thickness at a single point at each measurement location. Measurement locations were referenced to clock positions as viewed in the downstream direction, as shown in Figure 2-2.

Photo 2-1. Olympus Epoch XT Ultrasonic Flaw Detector

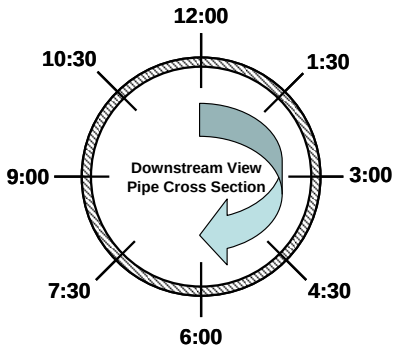

Figure 2-2. Clock Positions on Pipe, Looking Downstream

2.9 VANDA® Concrete Condition Rating System

The VANDA® Concrete Condition Index (Table 2-3) was created by V&A to provide consistent reporting of corrosion damage based on qualitative, objective criteria. Condition of concrete can vary from Level 1 to Level 4 based upon visual observations and field measurements, with Level 1 indicating the best condition and Level 4 indicating severe damage.

Table 2-3. VANDA® Concrete Condition Index Rating System

Condition Rating	Description	Representative Photograph
Level 1	**None/Minimal Damage to Concrete** Hardness: No Loss Surface Profile: No Loss Cracking: Shrinkage Cracks Spalling: None Reinforcing Steel (Rebar): Not Exposed or Damaged	
Level 2	**Damage to Concrete Mortar** Hardness: Damage to Concrete Mortar Surface Profile: Some Loss Cracking: Thumbnail Sized Cracks of Minimal Frequency Spalling: Shallow Spalling of Minimal Frequency, Related Rebar Damage Reinforcing Steel (Rebar): May Be Exposed but Not Damaged	
Level 3	**Loss of Concrete Mortar/Damage to Rebar** Hardness: Complete Loss Surface Profile: Large Diameter Exposed Aggregate Cracking: ¼-inch to ½-inch Cracks, Moderate Frequency Spalling: Deep Spalling of Moderate Frequency, Related Rebar Damage Reinforcing Steel (Rebar): Exposed and Damaged, Can Be Rehabilitated	
Level 4	**Rebar Severely Corroded/Significant Damage to Structure** Hardness: Complete Loss Surface Profile: Large Diameter Exposed Aggregate Cracking: ½-inch Cracks or Greater, High Frequency Spalling: Deep Spalling at High Frequency, Related Rebar Damage Reinforcing Steel (Rebar): Damaged or Consumed, Loss of Structural Integrity	

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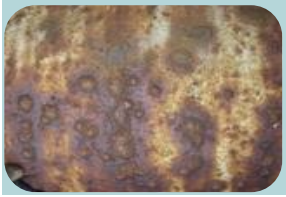
2.10 VANDA® Metal Condition Rating System

The VANDA® Metal Condition Index (Table 2-4) was created by V&A to provide consistent reporting of corrosion damage based on qualitative, objective criteria. Condition of ferrous metal can vary from Level 1 to Level 4 based upon visual observations and field measurements, with Level 1 indicating the best condition and Level 4 indicating severe damage.

Table 2-4. VANDA® Metal Condition Index Rating System

Condition Rating	Description	Representative Photograph
Level 1	**Little or No Corrosion** Loss of Wall Thickness %: None Pitting Depth (as % of Wall Thickness): None to Minimal Extent (Area) of Corrosion: None	
Level 2	**Minor Surface Corrosion** Loss of Wall Thickness %: < 25% Pitting Depth (as % of Wall Thickness): < 25% Extent (Area) of Corrosion: Localized	
Level 3	**Moderate to Significant Corrosion** Loss of Wall Thickness %: 25%-75% Pitting Depth (as % of Wall Thickness): 25%-75% Extent (Area) of Corrosion: 25%-75%	
Level 4	**Severe Corrosion; Immediate Repair/Replacement Needed** Loss of Wall Thickness %: > 75% Pitting Depth (as % of Wall Thickness): 75% or More Extent (Area) of Corrosion: Affects Most or All of Surface	

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3.0 RESULTS

3.1 PS1 Wet Well

3.1.1 pH, Penetration Depth, and Soundings

Physical evaluation was performed on the interior concrete surfaces of the PS1 wet well as shown in Table 3-1. These results indicate that the concrete is generally in good condition, with a shallow depth of degradation over most of the surface area. As described in Section 3.1.2, there was an area of exposed aggregate in the northeast corner of the wet well, near the main inlet. This area showed a slightly greater depth of degradation.

Table 3-1. pH, Penetration, and Sounding Results

Location	Concrete pH	Concrete Penetration Depth (in.)	Soundings	Notes
South wall, above Pump 3 suction pipe, 30 in. above invert, in normally submerged zone	–	Negligible	–	
South wall, above Pump 3 suction pipe, 55 in. above invert, in apparent wet/dry zone	8	1/8	–	pH was 12 at 1/8-in. depth, indicating shallow degradation depth
South wall, between Pump 2 and Pump 3 suction pipes, 24 to 78 in. above invert	–	–	Sound, hard concrete	
North wall, sloped area at bottom of wall, 24 to 48 in. above invert	–	–	Sound, hard concrete	
North wall above landing, outside area of exposed aggregate	–	Negligible	–	
North wall at northeast corner, above landing, above main inlet in area of exposed aggregate	8	1/8	–	Total loss of concrete profile may be up to 1/4 in. deep
Upper walls, as accessible from landing	–	–	Sound, hard concrete	

3.1.2 Visual and Qualitative Evaluation

The interior concrete surfaces of the wet well were in good condition (VANDA Level 1), except for an area near the main inlet (northeast corner) that exhibited medium-diameter exposed aggregate (VANDA Level 2). Above the apparent normal water line, the coating has failed completely, although it is still present on much of the surface area. Below the water line, the coating appeared to be in fair to good condition. Photo 3-1 through Photo 3-4 illustrate typical conditions inside the PS1 wet well.

Photo 3-1. North wall of wet well, showing typical concrete surfaces and failed coating.

Photo 3-2. Exposed aggregate at northeast corner of wet well above main inlet.

Photo 3-3. Concrete surfaces near crossover pipe to PS2 near northwest corner of wet well.

Photo 3-4. Northwest corner, showing typical concrete surfaces.

Additional observations and comments from the PS1 wet well are as follows:

- Access to the wet well was poor with respect to confined space entry and rescue. It was difficult to set up the davit arm in order to use the self-retracting lifeline. The access opening leads down to a landing, which is constructed of grating. To enter the part of the wet well below the landing, a portion of the grating had to be removed. This formed an obstruction to the tag line attached to the entrant, limiting the range of movement inside the wet well. There were other items inside the wet well (see below) that formed additional obstructions.
- The wet well contains two frame structures extending across the wet well (from the north wall to the south wall). These consist of structural angle uprights and cross pieces (Photo 3-5 and Photo 3-6). The purpose of these frames is unknown. The frames, while not necessarily an obstruction to the wastewater flow, obstruct access for personnel and would likely make it more difficult to clean the wet well. These frames were somewhat corroded.

Photo 3-5. Frame extending across wet well.

Photo 3-6. Frame extending across wet well.

- There were a few items of debris at the invert of the wet well. These appeared to be remnants of piping or other appurtenances that had corroded and fallen into the wet well (Photo 3-7 and Photo 3-8). An attempt was made to remove them, but they were too large and heavy, so they were left in place after consulting with the City staff on site. These items did not appear to be an imminent threat to the pumps.
- Otherwise, the invert of the wet well, in the area that was accessible, was clean and free of grit and gravel.

Photo 3-7. Debris at invert of wet well.

Photo 3-8. Corroded item in front of main inlet. This item may fall into the wet well.

- There was a 6-inch-tall by 4-inch-wide beam running across the ceiling near the entry opening (Photo 3-9 and Photo 3-10). The brackets and hardware holding the beam to the walls were corroded, as were the flanges of the beam itself. It is not known if this beam provides a structural function.

Photo 3-9. Corroded hardware, brackets, and flanges of beam at ceiling.

Photo 3-10. Beam extends across ceiling near entry opening.

- Drawings were not provided for the PS1 wet well, so it is unknown if record dimensions were available. The wet well measured approximately 21 feet 5 inches long, 8 feet wide, and 20 feet 8 inches high (from ceiling to invert). The landing was 6 feet wide by 8 feet long (across the full width of the east end of the wet well), at a height of approximately 12 feet 8 inches above the invert. The access opening at grade was 36 inches wide by 66 inches long.

- Additional documentation of metallic, mechanical, electrical, and other items within the PS1 wet well is presented in Appendix A. Appendix A also includes documentation from the interior of the diversion structure just upstream of the PS1 wet well.

3.1.3 Surface Penetrating Radar Scans

SPR scanning was conducted at three locations within the PS1 wet well. Two of the locations were on the south wall between the Pump 2 and Pump 3 suction pipes. One location was on the north wall near the northeast corner, above the landing. The scans measured the depth and spacing of the reinforcing bars crossing the scan path.

Table 3-2 summarizes the depth and spacing of the reinforcing steel as measured at the SPR scanning locations. The minimum depth of concrete cover over the reinforcing steel at the SPR scan locations was 2.5 inches, which is more than the minimum depth of 2 inches recommended as a guideline for this type of structure (see Section 2.6). It is possible that thickness of the concrete cover is less at locations that were not scanned.

Table 3-2. SPR Scan Results for PS1 Wet Well

Location	Bar Direction	Reinforcing Bar Depth (in.)			Reinforcing Bar Spacing (in.)		
		Min.	Avg.	Max.	Min.	Avg.	Max.
South wall, between Pump 2 and 3 suction pipes, approx. 6 ft. above invert	Vertical	3.3	3.4	3.5	3.1	3.6	4.3
South wall, between Pump 2 and 3 suction pipes, approx. 6 to 8 ft. above invert	Horizontal	2.5	2.5	2.6	14.8	14.8	14.8
South wall, between Pump 2 and 3 suction pipes, approx. 3 ft. above invert	Vertical	3.5*	3.7*	3.9*	3.3*	3.6*	4.2*
South wall, between Pump 2 and 3 suction pipes, approx. 2 to 5 ft. above invert	Horizontal	2.5	2.6	2.7	14.7	14.7	14.7
North wall, above landing, approx. 1 to 6 ft. above landing	Vertical	2.6	2.8	3.0	5.5	6.9	8.1
North wall, above landing, approx. 1 to 6 ft. above landing, approx. 5 ft. west of northeast corner	Horizontal	3.2	3.4	3.7	4.9	6.1	7.1
North wall, above landing, approx. 1 to 6 ft. above landing, approx. 1 to 3 ft. west of northeast corner	Horizontal	3.0	3.3	3.5	17.7	18.2	18.9

* Scans in this area were of low quality. These results are approximate.

3.1.4 Pump Suction Bells

The wet well contains four suction bells and pipes. The suction bells were visible during the dropdown entry. The suction bells, rather than being down-turned, are a type that extends horizontally and flares out. The invert of the suction pipes appeared to be close to the invert of the wet well. Two suction bells that were accessible at the dropdown location were evaluated visually and qualitatively. Visually, the suction bells appeared to be in good condition, and evidence of significant corrosion was not found. As a result, ultrasonic thickness testing was not performed on the suction bells. The suction bells appeared to be welded steel, and the weld seams were clearly visible. The flange nuts and bolts appeared to be in good condition beneath the coating or slime layer. It was not possible to distinguish between the slime layer on the suction bells and a coating, but it is anticipated that the coating is in fair condition. The interior of the suction bells may be a cement mortar lining, which appeared to be in good condition. The other two suction pipes in the wet well appeared to be similar to the two that were evaluated directly. Photo 3-11 through Photo 3-14 illustrate the suction bells and pipes.

Photo 3-11. Pump 4 suction bell.

Photo 3-12. Pump 1 and 2 suction bells.

Photo 3-13. Nuts and bolts on Pump 3 suction bell (typical).

Photo 3-14. Interior of Pump 3 suction bell.

3.1.5 Coating Sample Analysis

Table 3-3 presents the concentrations for 17 different metals analyzed from the paint sample collected inside the PS1 wet well. The sample was collected from the concrete wall above the landing. The mercury concentration of 377 ppm exceeds the TTLC of 20 ppm per Title 22 CCR § 66261.24.

Table 3-3. PS1 Wet Well Coating Sample Analysis Results

Metal	Total Threshold Limit Concentration (ppm)	PS1 Wet Well Sample Result (ppm)
Antimony	500	5
Arsenic	500	<1
Barium	10,000	463
Beryllium	75	<1
Cadmium	100	<1
Chromium	2,500	26
Cobalt	8,000	3
Copper	2,500	6
Lead	1,000	115
Mercury	20	**377**
Molybdenum	3,500	1
Nickel	2,000	1
Selenium	100	<3
Silver	500	<1
Thallium	700	<2
Vanadium	2,400	<9
Zinc	5,000	172

3.2 PS1 Diversion Structure

3.2.1 pH, Penetration Depth, and Soundings

A limited physical evaluation (soundings and penetration depth only) was performed on the interior concrete surfaces of the PS1 diversion structure as shown in Table 3-4. These results qualitatively indicate that the concrete is generally in good condition. V&A did not take pH measurements within the diversion structure.

Table 3-4. pH, Penetration, and Sounding Results

Location	Concrete pH	Concrete Penetration Depth (in.)	Soundings	Notes
Four corners of structure as accessible while standing at invert	–	Negligible to 1/16	Sound, hard concrete	

3.2.2 Visual and Qualitative Evaluation

The interior concrete surfaces of the diversion structure were generally in good condition, except there were bug holes and rock pockets in the concrete. The bug holes were widespread, and there were isolated rock pockets in a few locations. These are probably construction defects (poor consolidation), but they tend to reduce the effective cover over the reinforcing steel. The rock pockets in particular may provide a pathway for corrosive substances to reach the reinforcing steel. There were also a few instances of exposed aggregate and deteriorating mortar overlays at lower elevations, near the sluice gates, and in the corners of the structure. Overall, the concrete interior surfaces of the diversion structure are rated VANDA Level 2. Photo 3-15 through Photo 3-18 illustrate typical conditions inside the diversion structure.

The diversion structure is open to the air with a grating stretching across the opening at grade level. This would tend to mitigate the effects of a corrosive sewer atmosphere by allowing gases such as hydrogen sulfide to dissipate. This may explain the relatively minor amount of concrete deterioration that was observed, despite the possibility of there being flow turbulence in the structure under certain operating conditions.

Photo 3-15. General view inside diversion structure. Connection to PS1 is at upper right; connection to PS2 is at upper left; inlet is at lower left.

Photo 3-16. Typical concrete surfaces (east wall); note rock pocket.

Photo 3-17. Typical concrete surfaces in corner of diversion structure.

Photo 3-18. Typical concrete surfaces on lower east wall.

Additional observations and comments from the PS1 diversion structure are as follows:

- The diversion structure contains two cast iron sluice gates, one each controlling the outlet to PS1 and the outlet to PS2. The cast iron gate slides, frames, etc., exhibit exfoliating corrosion products over their surface areas (Photo 3-19). The amount of thickness loss was not

measured. The brackets for the gate lifts, near grade level, are made of steel and also exhibit a moderate amount of corrosion (Photo 3-20).

- The inlet pipe from the east is a reinforced concrete pipe (RCP) that protrudes into the structure several inches. The pipe exhibits a crack or spall around the circumference of the end of the pipe (Photo 3-21).
- The pipe connection from the north is plugged with what appears to be a dished steel plate. At least part of this plate appears to be bare steel and is subject to corrosion, as is the end of the pipe sleeve embedded in the wall (Photo 3-22).

Photo 3-19. Sluice gate at south outlet to PS2.

Photo 3-20. Support bracket for sluice gate lift at west outlet to PS1.

Photo 3-21. Spalling or cracking at end of east inlet pipe.

Photo 3-22. Apparent dished plug inside north pipe connection.

3.3 PS2 Wet Well

3.3.1 pH, Penetration Depth, and Soundings

Physical evaluation was performed on the interior concrete surfaces of the PS2 wet well as shown in Table 3-5. These results indicate that the concrete is generally in good condition, with a shallow depth of degradation over most of the surface area. Penetration depth of 1/4 in. below the water line shows that the concrete may be somewhat soft, or that certain operating conditions may expose the walls in this zone periodically, leading to degradation. A surface pH as low as 6 shows that there may be a minor amount of hydrogen sulfide present in the wet well atmosphere, leading to surface degradation.

Table 3-5. pH, Penetration, and Sounding Results

Location	Concrete pH	Concrete Penetration Depth (in.)	Soundings	Notes
West wall, between Pump 2 and Pump 3 suction pipes, approx. 66 in. above invert	10	1/4	Sound, hard concrete	
West wall, between Pump 2 and Pump 3 suction pipes, between approx. 24 and 72 in. above invert	–	–	Sound, hard concrete	
North wall, near northwest corner, approx. 18 in. above invert, in area of exposed aggregate	12	1/8	–	
West, north, and sloped east walls at northwest corner, within approx. 6 ft. of corner, from invert to approx. 7 ft. above invert	–	–	Generally sound, hard concrete	Some dull sounds (indicative of soft material), possibly due to slime layer
North wall, below northeast hatch opening, approx. 5 ft. above sloped surface, in area of exposed aggregate	8	3/16	Sound, hard concrete	pH was 11 at 3/16-in. depth, indicating shallow degradation depth
North wall, below northeast hatch opening, approx. 5 ft. above sloped surface, outside area of exposed aggregate	6	1/8	Sound, hard concrete	pH was 9 at 1/8-in. depth, indicating minor degradation penetrating into the concrete

3.3.2 Visual and Qualitative Evaluation

In general, the concrete interior surfaces of the wet well were in good condition with respect to corrosion, but there were numerous localized defects that result in an overall rating of VANDA Level 2 for the PS2 wet well. The defects observed within the PS2 wet well were as follows:

- There was medium-diameter exposed aggregate in several locations:
  - On the walls at or just below the apparent normal water line (Photo 3-23).
  - At the bottom of the north wall at the northwest corner of the wet well (Photo 3-24).
  - On the sloped east wall at the south end of the wet well.

Photo 3-23. Exposed aggregate near water line on north wall (typical).

Photo 3-24. Exposed aggregate at bottom of north wall.

- There were bug holes and rock pockets in the concrete. The bug holes (Photo 3-25) were widespread and were more prevalent at lower elevations. The rock pockets were predominantly on the west wall (Photo 3-26), which is the shared wall between the wet well and dry well, particularly in the lowest 2 feet of the wall (Photo 3-27). There was one apparent rock pocket (it may instead have been a hole in the concrete) that was at least 6 inches deep (Photo 3-28). These are probably construction defects (poor consolidation), but they tend to reduce the effective cover over the reinforcing steel. The rock pockets in particular may provide a pathway for corrosive substances to reach the reinforcing steel.
- There were apparent voids in concrete fill around the suction pipes. These may also provide a pathway for corrosive substances to reach the reinforcing steel.

Photo 3-25. Bug holes on north wall near crossover pipe from PS1.

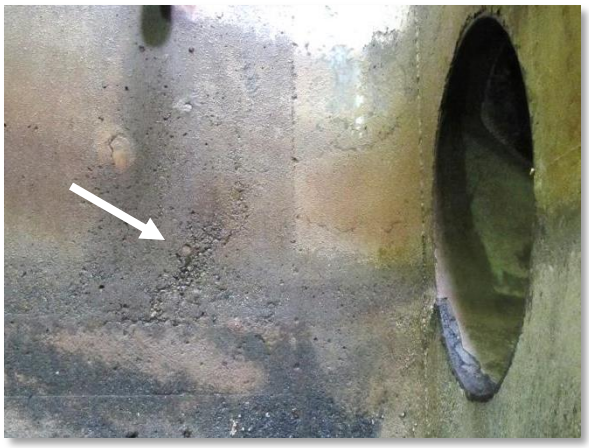


Photo 3-26. Rock pocket on west wall near northwest corner.

Photo 3-27. Rock pocket near bottom of west wall.

Photo 3-28. Ruler inserted 6 inches into rock pocket or hole near base of west wall.

- There was efflorescence and cracking visible on the ceiling (Photo 3-29) and on the bottom of the walkway that runs along the east wall (Photo 3-30). There was also seepage below the wall running along the edge of the walkway (Photo 3-31), and vertical crack in the wall (Photo 3-32). These cracks may also provide a pathway for corrosive substances to reach the reinforcing steel.

Photo 3-29. Cracking and efflorescence on wet well ceiling. Some cracks run across nearly the full width of the wet well.

Photo 3-30. Typical cracking and efflorescence on bottom surface of walkway.

Photo 3-31. Seepage at bottom of wall along walkway. Standing water on the walkway appears to be seeping through. This may be related to the efflorescence.

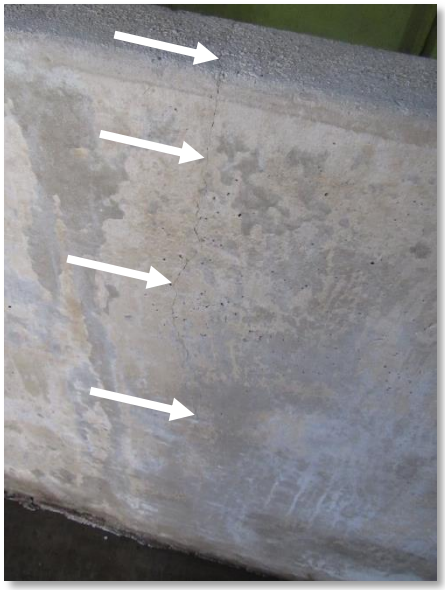


Photo 3-32. Hairline crack in wall along edge of walkway.

- There appeared to have been a mortar overlay that was originally applied to much or all of the interior surface. This mortar overlay was degraded and coming loose where it was present (Photo 3-33).
- There were rough patches in the ceiling in a few locations, and spalling around the hatch frames at the north end of the wet well (Photo 3-34).

- The safety chains at the ends of the walkway had corroded links, resulting in not all of the chains being functional (Photo 3-35).
- The bottom edge of the sloped east wall was rough, as though the formwork had been irregular in this area.
- Additional documentation of metallic, mechanical, electrical, and other items within the PS2 wet well is presented in Appendix A.

Photo 3-33. Degraded mortar overlay at northwest corner.

Photo 3-34. Rough patch in ceiling (below) and spalling of concrete around hatch frame (above).

Photo 3-35. Corroded and non-functional safety chains (typical).

3.3.3 Surface Penetrating Radar Scans

SPR scanning was conducted at two locations within the PS2 wet well. One location was on the north wall near the northwest corner, below the crossover pipe inlet from PS1. The other location was on

the west wall between the Pump 1 and Pump 2 suction pipes. The scans measured the depth and spacing of the reinforcing bars crossing the scan path.

Table 3-6 summarizes the depth and spacing of the reinforcing steel as measured at the SPR scanning locations. The minimum depth of concrete cover over the reinforcing steel at the SPR scan locations was 2.7 inches, which is more than the minimum depth of 2 inches recommended as a guideline for this type of structure (see Section 2.6). It is possible that the thickness of the concrete cover is less at locations that were not scanned. Also, it should be noted that some of the defects (bug holes, rock pockets, etc.) noted in Section 3.3.2 may reduce the effective depth of cover over the reinforcing steel.

Table 3-6. SPR Scan Results for PS2 Wet Well

Location	Bar Direction	Reinforcing Bar Depth (in.)			Reinforcing Bar Spacing (in.)		
		Min.	Avg.	Max.	Min.	Avg.	Max.
North wall, approx. 1 to 5 feet east of west wall, from approx. 1 to 6 feet above invert (see note a)	Horizontal	4.6	5.5	6.4	8.5	11.1	12.3
North wall, approx. 2 feet east of west wall, from approx. 1 to 6 feet above invert	Vertical	3.1	3.9	5.2	11.0	12.1	13.6
West wall, between Pump 1 and Pump 2 suction pipes, approx. 18 in. above invert, below step in wall (see note b)	Horizontal	3.4	3.6	3.8	9.8	9.8	9.8
West wall, between Pump 1 and Pump 2 suction pipes, approx. 4 to 6 ft. above invert, above step in wall (see note b)	Horizontal	4.0	4.3	4.6	11.8	12.6	13.6
West wall, between Pump 1 and Pump 2 suction pipes, approx. 4 to 6 ft. above invert, above step in wall (see note c)	Vertical	2.7	2.9	3.2	11.8	12.0	12.2

a. An additional horizontal bar was found in this area at a depth of 2.9 inches. This bar was not on the same grid as the others.
b. Two additional horizontal bars were found in the vicinity of the step in the wall, at depths of 4.2 and 6.2 inches. These bars were not on the same grid as the others.
c. Additional vertical bars (not included in the table findings) were found adjacent to the pipe penetrations.

3.3.4 Pump Suction Bells

The wet well contains four suction bells and pipes, which were evaluated visually and qualitatively. The suction line for Pump 1 does not appear to be in use. Visually, the suction bells appeared to be in good condition, and evidence of significant corrosion was not found. As a result, ultrasonic thickness testing was not performed on the suction bells. The suction bells appeared to be welded steel, and the weld seams and markings were clearly visible. The flange nuts and bolts appeared to

be in good condition beneath the slime layer. The coating and lining on the suction bells appeared to be in fair condition, with some chipping at edges and seams. Photo 3-36 through Photo 3-39 illustrate the suction bells and pipes.

Photo 3-36. Pump suction bells.

Photo 3-37. Typical bolt on suction bell flange; appears to be in good condition under slime.

Photo 3-38. Coating chipped on edge of Pump 2 suction bell (typical).

Photo 3-39. Interior of Pump 2 suction bell (typical). Note possible chipping or erosion of coating at weld seam.

3.4 PS2 Grinder Chamber

3.4.1 pH, Penetration Depth, and Soundings

Physical evaluation was performed on the interior concrete surfaces of the PS2 grinder chamber as shown in Table 3-7. These results indicate that the concrete is in good condition. The grinder chamber probably experiences a relatively non-corrosive atmosphere because it is open to the air.

Table 3-7. pH, Penetration, and Sounding Results

Location	Concrete pH	Concrete Penetra-tion Depth (in.)	Soundings	Notes
South wall near southeast corner, at apparent water line	–	Negligible		
South wall near southeast corner, above apparent water line	–	Negligible		
South and southeast walls between main inlet and upstream slide gate, below grating	–	–	Sound, hard concrete	
North and northeast walls between main inlet and upstream slide gate, below grating	–	–	Sound, hard concrete	
North wall between upstream slide gate and grinders, above grating, both sides of vertical crack	–	–	Sound, hard concrete	

3.4.2 Visual and Qualitative Evaluation

In general, the concrete interior surfaces of the PS2 grinder chamber were in good condition with respect to corrosion, resulting in an overall rating of VANDA Level 1. Photo 3-40 shows typical conditions inside the grinder chamber channels. However, there were some localized defects:

- There was an isolated area of spalling above the main outlet to the wet well (Photo 3-41).
- The grating and grating supports appear to be in good condition, except for the nuts and anchor bolts holding the support angles in place, which appear to be moderately to significantly corroded (Photo 3-42). This applies to the gratings directly above the channels and also to the gratings at grade level.
- There is a vertical crack in the approximate center of the north wall. The crack extends most of the height of the wall and exhibits some efflorescence (Photo 3-43 and Photo 3-44). There were other minor cracks in a few other locations as well, apparently randomly distributed.

- There was an isolated area of spalling and apparent reinforcing steel corrosion on the north wall at the bottom of the stairway (Photo 3-45). This location is in between the grinder chamber and wet well.
- As with the PS2 wet well, there appeared to have been a mortar overlay that was originally applied to much or all of the interior surface. This mortar overlay was degraded and coming loose where it was present.
- Additional documentation of metallic, mechanical, electrical, and other items within the PS2 grinder chamber is presented in Appendix A.

Photo 3-40. General view looking upstream in south grinder channel.

Photo 3-41. Spalling of concrete above outlet from grinder chamber to wet well.

Photo 3-42. Corroded nuts and bolts holding grating supports to walls (typical).

Photo 3-43. Crack near center of north wall.

Photo 3-44. Crack near center of north wall.

Photo 3-45. Isolated spalling on north wall outside grinder chamber (above bottom of stairway).

3.4.3 Surface Penetrating Radar Scans

SPR scanning was conducted at one location within the PS2 grinder chamber. The scan location was on the south and southeast walls at the southeast corner of the chamber, extending downstream from the main inlet. The scans measured the depth and spacing of the reinforcing bars crossing the scan path.

Table 3-8 summarizes the depth and spacing of the reinforcing steel as measured at the SPR scanning locations. The minimum depth of concrete cover over the reinforcing steel at the SPR scan locations was 2.5 inches, which is more than the minimum depth of 2 inches recommended as a guideline for this type of structure (see Section 2.6). It is possible that thickness of the concrete cover is less at locations that were not scanned.

Table 3-8. SPR Scan Results for PS2 Grinder Chamber

Location	Bar Direction	Reinforcing Bar Depth (in.)			Reinforcing Bar Spacing (in.)		
		Min.	Avg.	Max.	Min.	Avg.	Max.
South wall near southeast corner	Vertical	2.6	2.6	2.7	11.6	12.1	12.6
South wall near southeast corner	Horizontal	3.4	3.6	3.8	12.0	12.1	12.2
Southeast (diagonal) wall at southeast corner	Vertical	2.5	3.2	3.9	15.3	15.3	15.3

4.0 CONCLUSIONS

4.1 PS1 Wet Well

1. The interior concrete surfaces of the wet well were generally in good condition. Based on the physical and visual evaluations, the wet well interior was rated VANDA Level 1, except for the area around the main inlet, which was rated VANDA Level 2. The area around the main inlet exhibited medium-diameter exposed aggregate.
2. The existing coating on the concrete has reached the end of its service life.
3. The coating sample (collected from the concrete wall surface) concentration of 377 ppm mercury was well over the TTLC limit of 20 ppm. The results for the other 16 metals that were analyzed were below the TTLC limits.
4. The pump suction bells appeared to be in good condition. The coatings and linings on the suction bells appeared to be in fair to good condition.
5. The reinforcing steel is adequately protected by the existing concrete cover thickness.
6. The pH of the concrete surface does not indicate high levels of degradation.
7. Access to the wet well was poor with respect to confined space entry and rescue. It was not straightforward to set up a self-retracting lifeline at the entry point, and there are several obstructions inside the wet well, including two frame structures made of structural angles that extend across the wet well. The purpose of these frames is unknown.
8. There were a few large items of debris at the invert of the wet well. These may have been items that were originally installed in the wet well and had corroded away until they fell down to the invert.
9. There was a beam, running across the wet well ceiling near the entry opening, which exhibited corrosion. It is not known if this beam serves a structural purpose.

4.2 PS1 Diversion Structure

10. In general, the concrete interior surfaces of the PS1 diversion chamber were in good condition with respect to corrosion, but there were some localized defects, such as exposed aggregate and rock pockets, that result in an overall rating of VANDA Level 2. Rock pockets and bug holes may serve as a pathway for corrosive substances to reach the reinforcing steel. Conversely, the corrosivity of the structure's internal environment is mitigated by the fact that the diversion structure is open to the air at the top. This is reflected in the low level of degradation that was observed.
11. The two cast iron sluice gates inside the structure exhibit exfoliating corrosion products over their surface areas. The degree of corrosion was not quantified. The brackets for the gate lifts also exhibited corrosion.

12. The inlet pipe is RCP and exhibits a crack or spall around the circumference of the end of the pipe. The pipe connection from the north wall is plugged with what appears to be a dished steel plate, which is subject to corrosion along with the steel wall sleeve.

4.3 PS2 Wet Well

13. In general, the concrete interior surfaces of the wet well were in good condition with respect to corrosion, but there were numerous localized defects that result in an overall rating of VANDA Level 2. The defects included medium-diameter exposed aggregate; bug holes, rock pockets, and voids; spalling around the hatch frames; and cracking and efflorescence, particularly on the ceiling and walkway. A full listing of the observed defects can be found in Section 3.3.2.
14. The depth of concrete cover over the reinforcing steel was adequate in the areas that were scanned. However, some of the defects listed above may serve as a pathway for corrosive substances to reach the reinforcing steel.
15. The pump suction bells appeared to be in good condition. The coatings and linings on the suction bells appeared to be in fair to good condition.

4.4 PS2 Grinder Chamber

16. The concrete interior surfaces of the grinder chamber were in good condition and were rated VANDA Level 1. However, there were some localized defects, including a crack in the north wall, corrosion of the grating anchor bolts, and minor concrete spalling. A full listing of the observed defects can be found in Section 3.4.2.

5.0 RECOMMENDATIONS

5.1 PS1 Wet Well

1. Resurface and recoat the interior of the wet well. Based on the current condition, this is a medium-term (within 5 years) recommendation that will serve to extend the useful life of the structure. Recommendations for a lining system are as follows:
    a. Surface Preparation – Prior to the application of the epoxy novolac or polyurethane coating below, the existing concrete should be prepared by high-pressure water jetting at 5,000 psi to remove soft concrete and debris and abrasive blasting to remove deteriorated concrete. Any exposed, corroded reinforcing steel should be evaluated by a structural engineer to determine if repairs need to be made. Exposed reinforcing steel should be treated with a corrosion inhibitor such as Sika Armatec 110 EpoCem or approved equal. Then the concrete substrate should be resurfaced up to the approximate original surface by spray-applying or hand-applying a repair mortar such as BASF MasterEmaco S488 CI, Tnemec Series 217 Mortarcrete or approved equal. Note that if the repair mortar is spray-applied, the repair surface should be hand-finished to a surface suitable for coating.
    b. Epoxy Novolac Coating System – Epoxy novolac coatings such as Raven Lining Systems' Raven 405, Sauereisen Sewergard 210S, or an approved equal offer excellent resistance to chemicals and sulfuric acid. Polyurethane products such as Raven Lining Systems' Raven 505, GET Endura Flex 1988, or Sherwin Williams Sherflex S are also recommended. The application requires a two-step process after the concrete has been cleaned, abraded and resurfaced:
        - Spray application of an epoxy primer at 2 to 5 mils.
        - Spray application of the epoxy novolac or polyurethane coating at 125 mils.
    c. Require abatement procedures per CCR Title 8, §1532.1, for removal of the existing coating due to the elevated level of mercury that was found. Require the contractor to retain the services of an industrial hygienist and develop an industrial hygiene plan for the removal of the existing coating.
2. Recoat and reline the pump suction bells with a high solids epoxy such as Carboline 890, PPG Amercoat 240, or International Paint Bar Rust 231. Abrasive blast per SSPC SP 10 and apply two coats at a dry film thickness of 5 to 6 mils per coat.
3. Address the other individual issues that were noted. Analysis of the beam running along the ceiling should be prioritized due to the corrosion that was observed.
4. Evaluate the purpose of the frame structures that span the wet well. If they are not necessary, remove them. If they are necessary, consider replacing them with 316 stainless steel in order to address the corrosion that was observed.

5. Improve access to the wet well. Some possibilities include the following:
    a. Add a fixed base for a davit arm near the opening.
       Consider ways to make access below the landing easier:
        - Add a hatch in the landing.
        - Remove part of the landing.
        - Remove other obstructions in the wet well such as the frame structures.
6. Consider providing a gate to shut off the crossover pipe from PS1 to PS2, so that full isolation of each station can be provided when there is work taking place in either wet well.

5.2 PS1 Diversion Structure

7. Consider coating the interior of the diversion structure in order to extend its useful life. The coating systems recommended above for the wet wells above can be used in the diversion structure, although exposure to sunlight may cause the coating to chalk or discolor. This will not prevent the coating from performing its protective function. Coating this structure does not need to be a high priority because of the minimal concrete degradation that was observed, but it may be cost-effective to do in conjunction with one of the wet wells. Coating the structure interior would also be an opportunity to coat the north pipe connection plug and seal the crack in the east pipe inlet to prevent further degradation.
8. If the diversion structure is not coated, reassess the concrete interior surfaces on a 10-year interval, and address the bare steel that is exposed at the north pipe connection. Monitor the east pipe inlet for further degradation on a 2-year interval or more frequently.
9. Consider coating the sluice gates and repairing the corroded brackets, etc., for the gate lifts.

5.3 PS2 Wet Well

10. Resurface and coat the interior of the wet well. Based on the current condition, this is a short- to medium-term (within 2 to 5 years) recommendation that will serve to extend the useful life of the structure. Recommendations for a lining system are as follows:
    a. Surface Preparation – Prior to the application of the epoxy novolac or polyurethane coating below, the existing concrete should be prepared by high-pressure water jetting at 5,000 psi to remove soft concrete and debris and abrasive blasting to remove deteriorated concrete. Any exposed, corroded reinforcing steel should be evaluated by a structural engineer to determine if repairs need to be made. Exposed reinforcing steel should be treated with a corrosion inhibitor such as Sika Armatec 110 EpoCem or approved equal. Then the concrete substrate should be resurfaced up to the approximate original surface by spray-applying or hand-applying a repair mortar such as BASF MasterEmaco S488 CI, Tnemec Series 217 Mortarcrete or approved equal. Note that if the repair mortar is spray-applied, the repair surface should be hand-finished to a surface suitable for coating.
    b. Epoxy Novolac Coating System – Epoxy novolac coatings such as Raven Lining Systems' Raven 405, Sauereisen Sewergard 210S, or an approved equal offer

excellent resistance to chemicals and sulfuric acid. Polyurethane products such as Raven Lining Systems' Raven 505, GET Endura Flex 1988, or Sherwin Williams Sherflex S are also recommended. The application requires a two-step process after the concrete has been cleaned, abraded and resurfaced:

- Spray application of an epoxy primer at 2 to 5 mils.
- Spray application of the epoxy novolac or polyurethane coating at 125 mils.

11. Consider recoating and relining the pump suction bells with a high solids epoxy such as Carboline 890, PPG Amercoat 240, or International Paint Bar Rust 231. Abrasive blast per SSPC SP 10 and apply two coats at a dry film thickness of 5 to 6 mils per coat.
12. Address the other individual issues that were noted. The safety chains at the ends of the walkway should be prioritized for immediate repair.
13. Consider providing a gate to shut off the crossover pipe from PS1 to PS2, so that full isolation of each station can be provided when there is work taking place in either wet well.

5.4 PS2 Grinder Chamber

14. Monitor the interior of the grinder chamber for signs of degradation. At this time, a coating is not recommended because the grinder chamber is open to the air and appears to experience a low degradation rate.
15. Seal the vertical crack in the north wall by injecting a urethane grout.
16. Address the other individual issues that were noted. The replacement of the anchor bolts and nuts for the gratings should be prioritized for immediate repair. The new anchor bolts and nuts should be 316 stainless steel for corrosion resistance. Install insulating gaskets between the bolts/nuts and beams if the existing beams are made of dissimilar metals such as aluminum or galvanized steel.

APPENDIX A. DOCUMENTATION OF METALLIC, MECHANICAL, ELECTRICAL, AND OTHER ITEMS

PS1 Wet Well

Photo A-1. Piping on west wall. Piping and supports exhibit corrosion.

Photo A-2. Piping and ducts at southwest corner. Duct supports and piping exhibit corrosion.

Photo A-3. Piping at ceiling in western portion of wet well. Some piping exhibits corrosion. One pipe exhibits an apparent repair patch.

Photo A-4. Closer view of apparent patch on piping.

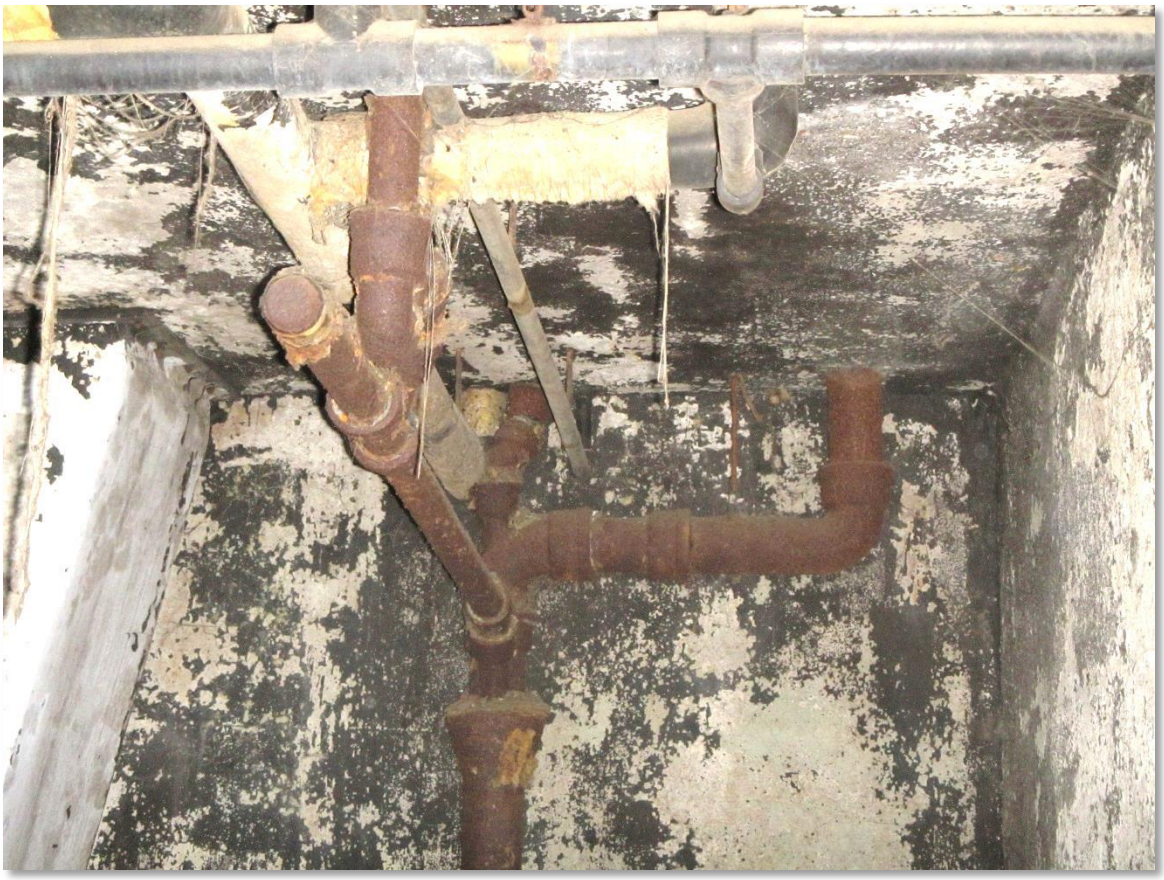


Photo A-5. Additional view of bell-and-spigot iron piping at ceiling of wet well.

Photo A-6. PVC piping and conduits above landing at northeast corner. Penetration for PVC piping exhibits corrosion of the wall sleeve.

Photo A-7. Piping or conduit on south wall.

Photo A-8. Apparently loose pipe support on south wall.

Photo A-9. Corrosion on frame structures extending across wet well (typical).

Photo A-10. Frame structure extending across wet well and supports for grated landing.

Photo A-11. Railing at end of landing, and frame structure crossing wet well.

Photo A-12. Access to lower portion of wet well is by removing a portion of the grating.

Photo A-13. Light fixture on north wall. Fixture is corroded.

Photo A-14. Beam extending across ceiling (see Section 3.1.2). Note corroded and non-functional conduit bracket at right.

Photo A-15. Light fixture at access opening.

Photo A-16. Penetration through ceiling.

Photo A-17. Apparent disused anchor bolts in ceiling.

Photo A-18. Gap or crack between pump station structure (wet well entrance at right) and wall that surrounds area outside entrance.

PS1 Diversion Structure

Photo A-19. East inlet.

Photo A-20. View inside east inlet.

Photo A-21. South outlet to PS2.

Photo A-22. View inside south outlet to PS2.

Photo A-23. Wedges on sluice gate frame at south outlet.

Photo A-24. Sluice gate at west outlet to PS1. A view inside this pipe was not possible because the gate was closed.

Photo A-25. Typical concrete surfaces and stem guides above south outlet.

PS2 Wet Well

Photo A-26. Piping and conduits or hydraulic lines above and inside main inlet to wet well.

Photo A-27. Piping and duct at southwest corner.

Photo A-28. Conduit or piping at invert of main inlet.

Photo A-29. Piping on ceiling. Note corroded hangers and strut channel supports.

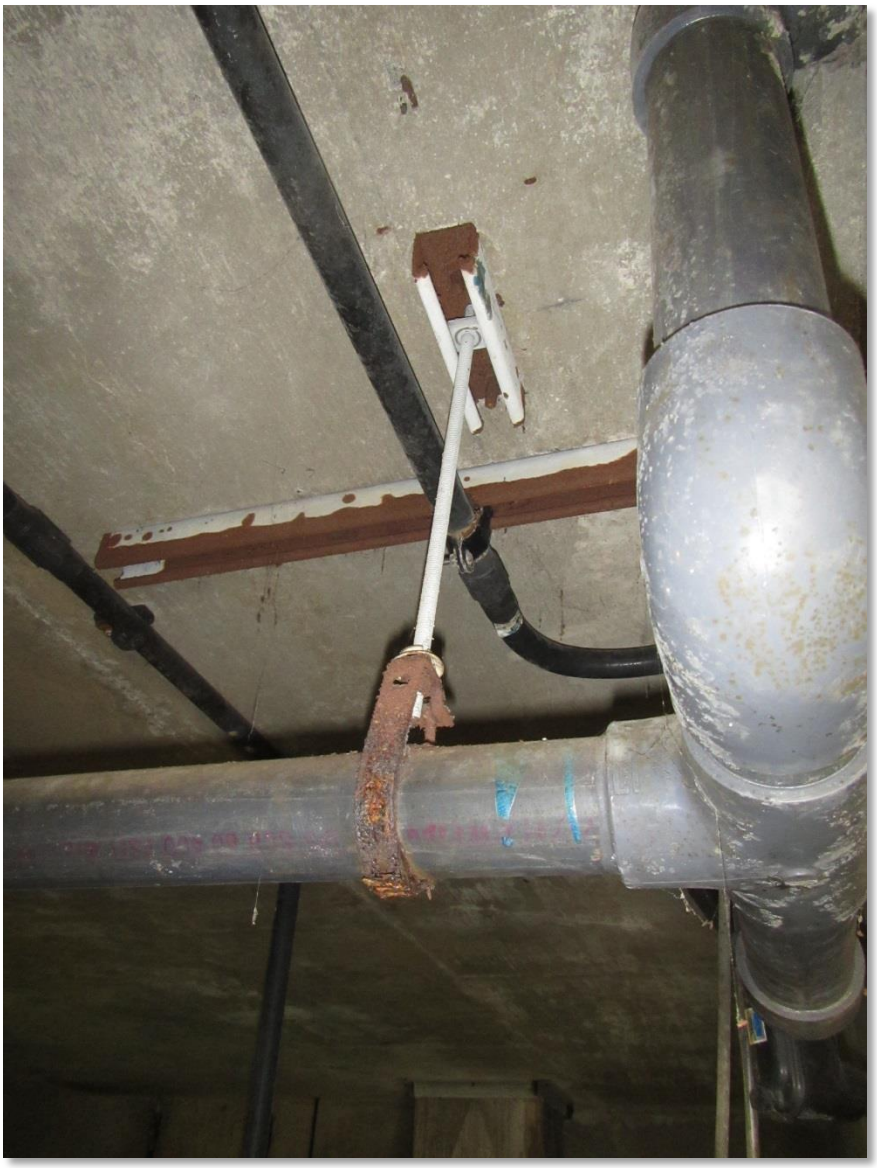


Photo A-30. Piping on ceiling. Note corroded hangers and strut channel supports.

Photo A-31. Corrosion on hydraulic lines and supports.

Photo A-32. Piping and conduits inside wet well. Note corrosion at conduit fittings.

Photo A-33. Non-potable water piping at southeast corner with corrosion at threaded joints.

Photo A-34. Piping and duct at northwest corner.

Photo A-35. Additional view of piping and duct at northwest corner.

Photo A-36. Closer view of duct at northwest corner.

Photo A-37. Duct at southwest corner. Note corrosion of hanger rods.

Photo A-38. Duct at southwest corner has standing water inside.

Photo A-39. Corrosion at conduit threads for light fixture (typical).

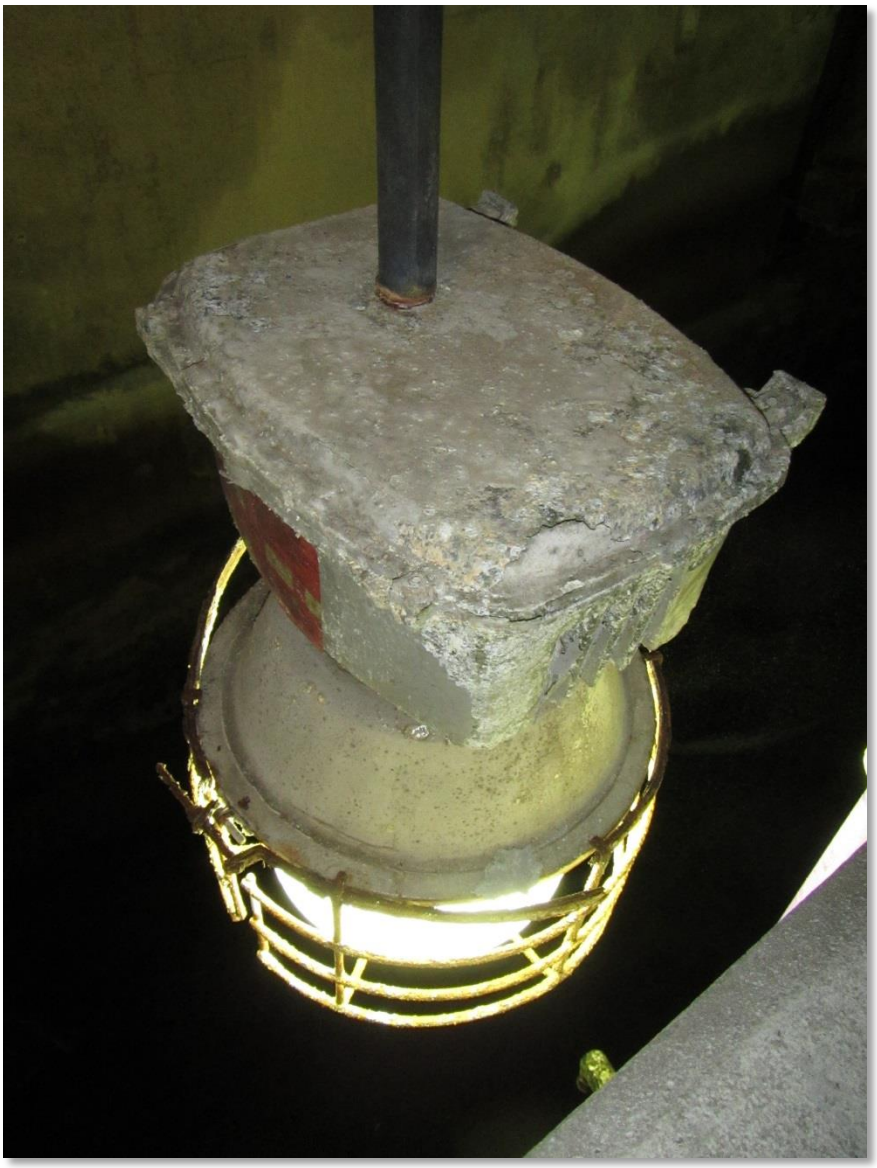


Photo A-40. Light fixture adjacent to platform. Note corrosion at threads on conduit, which is the only support for the fixture. Also note corroded cage around lens. Cage was easily removed by hand by City staff.

Photo A-41. Corroded cage from light fixture after removal by City staff.

Photo A-42. Float at southwest corner.

Photo A-43. Corrosion at welds on access gate for wet well platform. Also note roughened concrete where it was installed.

PS2 Grinder Chamber

Photo A-44. General view, looking east inside grinder chamber. Grating panels at bottom were removed for access to channels.

Photo A-45. Truncated conduits or pipes above main outlet to wet well.

Photo A-46. Piping exiting main outlet to wet well.

Photo A-47. Piping on south wall.

Photo A-48. Disused stop-plate guides (typical). Note that mortar overlay is spalling off of the guides.

Photo A-49. Disused stop-plate guides (typical). Note that mortar overlay is spalling off of the guides.

Photo A-50. Typical slide gate frame (at left). Per City staff, the slide gates and frames are stainless steel but were coated with Line-X. This coating has failed, and is peeling off in large sheets.

Photo A-51. Slide gate and grinder in south channel.

Photo A-52. Hydraulic lines, etc., in north channel.

Photo A-53. Disused bracket on channel wall, reportedly for a former bar screen installation.

Photo A-54. Upstream slide gate in north channel. Note sagging (disbonded) Line-X coating on gate and guides.

Photo A-55. Apparently disused bracket embedded in wall (at left).

Photo A-56. Typical brackets for grinders.

Photo A-57. Downstream slide gate in south channel. Note uneven (disbonded) Line-X coating.

Photo A-58. Main inlet to grinder chamber.

Photo A-59. View inside main inlet to grinder chamber.

Photo A-60. View inside main inlet to grinder chamber.

Photo A-61. Light fixture inside grinder chamber.

Photo A-62. Corrosion at conduit threads (typical). Also note tool marks and coating damage at left.

Appendix D - Planning Level Opinions of Probable Cost

City of Rohnert Park
Project: Emergency Sewer Pond and Wet Well Lining
Sewer Pond

ENR Construction Cost Index:		
	Jan-17	11,609.44

Item Description	Quantity	Unit	Unit Cost	Total Cost
Excavate Top 12-inches of Sewer Pond	1,100	CY	$60	$66,000
Disposal of Excess Material from Sewer Pond to Designated Site	1,100	CY	$50	$55,000
Shoring and Trench Safety	1	LS	$50,000	$50,000
Dewatering	1	LS	$50,000	$50,000
Fill and Grade Sewer Pond (6" thickness)	400	CY	$50	$20,000
Drain Rock for Sewer Pond Foundation (6" thickness)	200	CY	$80	$16,000
Geotextile Fabric for Sewer Pond Foundation	18,400	SF	$3	$55,200
Pond Backfill and Compaction (6" thickness)	400	CY	$150	$60,000
Reshape Pond Side Slopes to Remove Old Pond Features	1	LS	$50,000	$50,000
Concrete Slab (6" thickness)	400	CY	$500	$200,000
Access Road Subgrade and Pond Slope Preparation	1	LS	$25,000	$25,000
Access Road Retaining Curb	1	LS	$25,000	$25,000
Install Trench Drain at Vac/con Dump Site	1	LS	$10,000	$10,000
Replace Existing Drain Pipe within Pond Limits	1	LS	$50,000	$50,000
Construction Subtotal				$732,200
Mobilization/Demobilization (6%)				$43,932
Contractor's Bonds and Insurance (3%)				$21,966
Contractor's Overhead and Profit (15%)				$109,830
Estimated Bid Price				$907,928
Construction Contingency (30%)				$272,378
Total Estimate of Probable Construction Cost				$1,180,306
Engineering/CM				
- Contract Documents (8%)	1	LS		$94,425
- Engineering Support During Construction - Office (4%)	1	LS		$47,212
- Construction Management - Field (10%)	1	LS		$118,031
Grand Total				**$1,440,000**

City of Rohnert Park
Project: Emergency Sewer Pond and Wet Well Lining
Sewer Pipes

ENR Construction Cost Index:		
	Jan-17	11,609.44

Item Description	Quantity	Unit	Unit Cost	Total Cost
Temporary Bypass Pumping	1	LS	$5,000	$5,000
Sewer Cleaning & Pre-Rehabilitaiton CCTV	355	LF	$6	$2,130
Rehabilitate 18-inch Sewer Pipes using CIPP Method	355	LF	$120	$42,600
Post-Rehabilitaiton CCTV	355	LF	$2	$710
Construction Subtotal				$50,440
Mobilization/Demobilization (4%)				$2,018
Contractor's Bonds and Insurance (3%)				$1,513
Contractor's Overhead and Profit (15%)				$7,566
Estimated Bid Price				$61,537
Construction Contingency (30%)				$18,461
Total Estimate of Probable Construction Cost				$79,998
Engineering/CM				
- Contract Documents (10%)	1	LS		$8,000
- Engineering Support During Construction - Office (4%)	1	LS		$3,200
- Construction Management - Field (20%)	1	LS		$16,000
Grand Total				**$108,000**

City of Rohnert Park
Project: Emergency Sewer Pond and Wet Well Lining
PS1 Wet Well

ENR Construction Cost Index:		
	Jan-17	11,609.44

Item Description	Quantity	Unit	Unit Cost	Total Cost
Temporary Bypass Pumping	1	LS	$10,000	$10,000
Remove and Dispose Existing Coating	970	SF	$20	$19,400
Rehabilitate Existing Concrete and Install Protective Epoxy Coating (100% solid)	1,095	SF	$50	$54,750
Renew Coating on Suction Bells	1	LS	$5,000	$5,000
Rehabilitate Existing Beam	1	LS	$15,000	$15,000
Improve Acess to the Wet Well	1	LS	$50,000	$50,000
Lighting Improvements	1	LS	$20,000	$20,000
Motor-Actuated Sluice Gate between PS1 and PS2	1	EA	$50,000	$50,000
Construction Subtotal				$224,150
Mobilization/Demobilization (4%)				$8,966
Contractor's Bonds and Insurance (3%)				$6,725
Contractor's Overhead and Profit (15%)				$33,623
Estimated Bid Price				$273,463
Construction Contingency (30%)				$82,039
Total Estimate of Probable Construction Cost				$355,502
Engineering/CM				
- Contract Documents (10%)	1	LS		$35,550
- Engineering Support During Construction - Office (4%)	1	LS		$14,220
- Construction Management - Field (14%)	1	LS		$49,770
Grand Total				**$456,000**

City of Rohnert Park
Project: Emergency Sewer Pond and Wet Well Lining
PS2 Wet Well

ENR Construction Cost Index:		
	Jan-17	11,609.44

Item Description	Quantity	Unit	Unit Cost	Total Cost
Temporary Bypass Pumping	1	LS	$10,000	$10,000
Repair Safety Chains	1	LS	$2,000	$2,000
Rehabilitate Existing Concrete and Install Protective Epoxy Coating (100% solid)	2,990	SF	$50	$149,500
Renew Coating on Suction Bells	1	LS	$5,000	$5,000
Lighting Improvements	1	LS	$15,000	$15,000
Motor-Actuated Sluice Gate between PS1 and PS2	1	EA	$50,000	$50,000
Construction Subtotal				$231,500
Mobilization/Demobilization (4%)				$9,260
Contractor's Bonds and Insurance (3%)				$6,945
Contractor's Overhead and Profit (15%)				$34,725
Estimated Bid Price				$282,430
Construction Contingency (30%)				$84,729
Total Estimate of Probable Construction Cost				$367,159
Engineering/CM				
- Contract Documents (10%)	1	LS		$36,716
- Engineering Support During Construction - Office (4%)	1	LS		$14,686
- Construction Management - Field (14%)	1	LS		$51,402
Grand Total				**$470,000**

City of Rohnert Park
Project: Emergency Sewer Pond and Wet Well Lining
Ventilation PS2 Wet Well

ENR Construction Cost Index:		
	Jan-17	11,609.44

Item Description	Quantity	Unit	Unit Cost	Total Cost
Temporary Bypass Pumping	1	LS	$5,000	$5,000
Install Go/No Go Warning System at Wet Well Entrance	1	LS	$5,000	$5,000
Install CGD and Connect to the Go/No Go Warning System	1	LS	$5,000	$5,000
Install Fan Proving Switches and Connect to the Go/No Go Warning System	1	LS	$5,000	$5,000
Replace Supply and Exhaust Air Ductwork with Stainless/FRP Ducts	1	LS	$20,000	$20,000
Replace Supply Fan (3,500 cfm) with Non-sparking Unit and Explosion Proof Motor	1	EA	$7,500	$7,500
Replace Exhaust Fan (3,850 cfm) with Non-sparking Unit and Explosion Proof Motor	1	EA	$7,500	$7,500
Construction Subtotal				$55,000
Mobilization/Demobilization (4%)				$2,200
Contractor's Bonds and Insurance (3%)				$1,650
Contractor's Overhead and Profit (15%)				$8,250
Estimated Bid Price				$67,100
Construction Contingency (30%)				$20,130
Total Estimate of Probable Construction Cost				$87,230
Engineering/CM				
- Contract Documents (12%)	1	LS		$10,468
- Engineering Support During Construction - Office (4%)	1	LS		$3,489
- Construction Management - Field (14%)	1	LS		$12,212
Grand Total				**$114,000**

City of Rohnert Park
Project: Emergency Sewer Pond and Wet Well Lining
Diversion Structure at PS1

ENR Construction Cost Index:		
	Jan-17	11,609.44

Item Description	Quantity	Unit	Unit Cost	Total Cost
Temporary Bypass Pumping	1	LS	$20,000	$25,000
Replace Existing Brackets and Lifting Parts with 316 Stainless Steel Items	1	LS	$5,000	$5,000
Renew Protecting Coating on Two Sluice Gates and Frames	1	LS	$5,000	$5,000
Construction Subtotal				$35,000
Mobilization/Demobilization (4%)				$1,400
Contractor's Bonds and Insurance (3%)				$1,050
Contractor's Overhead and Profit (15%)				$5,250
Estimated Bid Price				$42,700
Construction Contingency (30%)				$12,810
Total Estimate of Probable Construction Cost				$55,510
Engineering/CM				
- Contract Documents (8%)	1	LS		$4,441
- Engineering Support During Construction - Office (4%)	1	LS		$2,220
- Construction Management - Field (14%)	1	LS		$7,771
Grand Total				**$70,000**

City of Rohnert Park
Project: Emergency Sewer Pond and Wet Well Lining
Grinder Vault at PS2

ENR Construction Cost Index:		
	Jan-17	11,609.44

Item Description	Quantity	Unit	Unit Cost	Total Cost
Temporary Bypass Pumping	1	LS	$15,000	$15,000
Replace Existing Anchor Bolts and Nuts with 316 Stainless Steel Items	1	LS	$2,500	$3,000
Seal the North Wall Vertical Crack by Grout Injection	1	LS	$5,000	$5,000
Construction Subtotal				$23,000
Mobilization/Demobilization (4%)				$920
Contractor's Bonds and Insurance (3%)				$690
Contractor's Overhead and Profit (15%)				$3,450
Estimated Bid Price				$28,060
Construction Contingency (30%)				$8,418
Total Estimate of Probable Construction Cost				$36,478
Engineering/CM				
- Contract Documents (14%)	1	LS		$5,107
- Engineering Support During Construction - Office (4%)	1	LS		$1,459
- Construction Management - Field (14%)	1	LS		$5,107
Grand Total				**$49,000**

Appendix E – Press Democrat Development Sewer Hydraulic Evaluation

Memorandum

October 25, 2017

To:	Vanessa Garrett, City of Rohnert Park	Ref. No.:	GENL2017-0010
From:	Matt Winkelman and Adam Fisher, GHD	Tel:	707-236-1546
cc:		GHD Project No.:	11151211
Subject:	**Press Democrat Development Sewer Hydraulic Evaluation**		

GHD prepared a hydraulic evaluation of the sanitary sewer collection system located downstream of the proposed Press Democrat development. The purpose of the evaluation was to compare existing and proposed collection system hydraulics to the City's hydraulic design standards for existing 12-inch pipelines located just west of J. Rogers Lane (the two pipelines immediately upstream of the discharge to the City's Main Pump Station).

The scope of services for the evaluation included hydraulic modeling for the local sewer basin (Basin 26) relative to City design criteria, notably the flow depth to pipe diameter (d/D) ratio of 0.9 for assessment of existing pipelines. Hydraulic evaluation did not include proposed pipelines within the Press Democrat property.

1. Proposed Project

The Press Democrat development is an commercial/industrial development that includes warehouse, commercial, and office land uses as shown on Figure 1. Additionally, the parcel due west of the Press Democrat development (APN 143-040-135) is proposed to develop as a 170-unit multi-family development. The developments are assumed to connect to the existing sewer system at the manholes indicated on Figure 1.

Information regarding the planned development was provided by the developer's engineer. Table 1 provides a summary of the planned development.

Table 1 Planned Development Information

Location	Land Use	Description
Building 1	Commercial Service	19,800 sf
Building 2	Commercial Service	19,800 sf
Building 3	Commercial Service	5,000 sf
Building 4	Commercial Service	Existing Building, 60,205 sf
Building 5	Warehouse	34,716 sf
Building 5	Offices	34,716 sf
Parcel 143-040-135	Multi-Family	170 units

2. Model Development

The model used for hydraulic evaluation of the existing sanitary sewer collection system located downstream of the planned development was developed approximately 10 years ago as part of a city-wide sewer model. Pipe information is based on City GIS records, where the data used for the model 10 years ago matched current data provided by the City. Flow monitoring data was used at that time to develop base and peak wet weather flows throughout the City, and divide the collection system into 28 sub-basins. Base flows were distributed proportionally to the average day potable water demands developed for a concurrent water modeling effort (by others provided by the City). Inflow and Infiltration (I/I) was distributed over the collection system by prorating I/I to the pipes based upon five parameters: pipe age, pipe material, hydraulic head on the pipe from groundwater, pipe diameter, and pipe length.

To update the model for this study, GHD started with the model that was recently updated for the proposed Five Creeks development, an area immediately upstream of the Press Democrat development. To update the model for the Five Creeks development, GHD took the old model and clipped out the relevant manholes and pipes within Basin 26. We then calibrated the model to match the newly developed design storm peak wet weather flows calculated by V&A as part of the 2016 sewer flow monitoring project. This created the flow conditions for the existing conditions model scenario. Flows prepared for planned development were then added to the existing conditions model to create the future conditions model scenario.

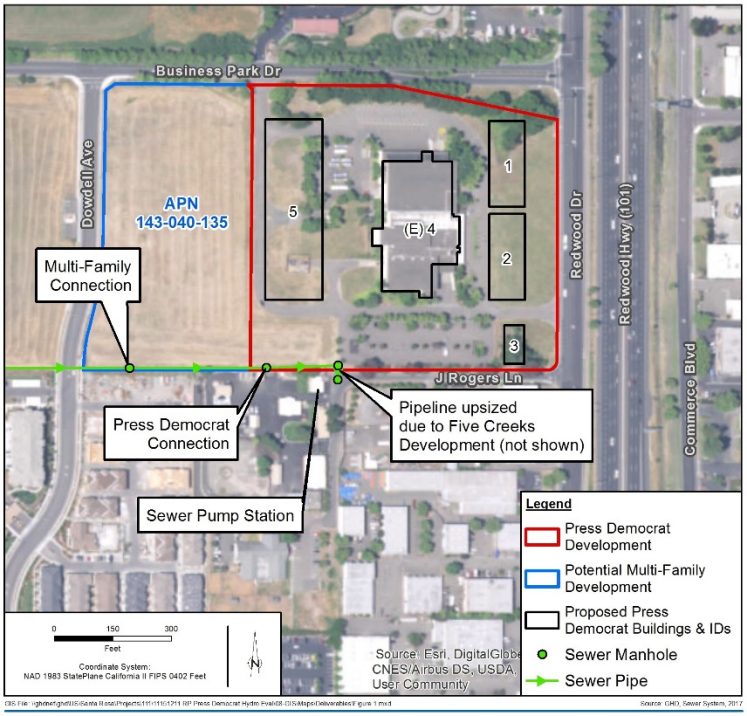


Figure 1: Project Area

3. Flow Analysis

The planned developments and their associated peak wet weather flows are summarized in Table 2 below. Peak flow is calculated using the City's design criteria, as follows:

Peak flow = (Base flow * 2.6 peaking factor) + I/I at 1.4 gallons per minute per acre

Other City design criteria used in the flow analysis include unit flows (average dry weather flows; ADWF) for the different land uses and types of flow, as follows:

- Commercial: 150 gallons per day (gpd) per 1,000 square feet (sf)
- Office: 110 gpd / 1,000 sf
- Warehouse: 40 gpd / 1,000 sf
- Multi-family: 100 gallons per capita per day (GPCD); 2.0 people / unit

See Figure 1 for the locations of the proposed developments and where they would connect to the existing sewer collection system.

Table 2 Flow Data

Development	Land Use/Type of Flow	Description	Flow Data
Press Democrat	Commercial	104,805 sf	Q_{ADWF} = 150 gpd/1,000 sf * 104,805 sf = 15,721 gpd = 10.92 gpm Q_{PDWF} = 10.92 gpm * 2.6 = 28.38 gpm
Press Democrat	Office	34,716 sf	Q_{ADWF} = 110 gpd/1,000 sf * 34,716 sf = 3,819 gpd = 2.65 gpm Q_{PDWF} = 2.65 gpm * 2.6 = 6.89 gpm
Press Democrat	Warehouse	34,716 sf	Q_{ADWF} = 40 gpd/1,000 sf * 34,716 sf = 1,388.64 gpd = 0.96 gpm Q_{PDWF} = 0.96 gpm * 2.6 = 2.51 gpm
Press Democrat	Total	12.71 acres	Q_{PDWF} = 28.38 gpm + 6.89 gpm + 2.51 gpm = 37.78 gpm $Q_{I/I}$ = 1.4 gpm/acre * 12.71 acres = 17.79 gpm Q_{PWWF} = 37.78 gpm + 17.79 gpm = 55.57 gpm **(55.6 gpm)**
Parcel 143-040-135	Multi-Family	170 units	Q_{ADWF} = 100 GPCD * 2.0 people/unit * 170 units = 34,000 gpd = 23.61 gpm Q_{PDWF} = 23.61 gpm * 2.6 = 61.39 gpm $Q_{I/I}$ = 6.53 acres * 1.4 gpm/acre = 9.14 gpm Q_{PWWF} = 61.39 gpm + 9.14 gpm = 70.53 gpm **(70.5 gpm)**

1 sf: square feet

2 GPCD: gallons per capita per day

3 gpd: gallons per day

4 gpm: gallons per minute

5 Q_{ADWF}: Base flow (average dry weather flow)

6 Peaking Factor: 2.6 (per City design criteria)

7 Q_{PDWF}: Peak dry weather flow

8 I/I Factor: 1.4 gallons per minute per acre (per City design criteria)

9 Q_{PWWF}: Peak wet weather flow for the 10-year, 24-hour design storm event

4. Modeling Results

Under existing conditions for peak wet weather flow (PWWF), including the planned upsizing of a pipeline due to the Five Creeks development, but without the new development flows for the Press Democrat or neighboring site, there are no apparent capacity deficiencies in the collection system located downstream of the planned development. The highest d/D in pipelines downstream of the Press Democrat or neighboring site is 0.67.

With the Press Democrat development PWWF added to the model, there are no apparent capacity deficiencies in the collection system located downstream of the planned development. The highest d/D in pipelines downstream of the Press Democrat or neighboring site is 0.70.

With the development of the parcel west of the Press Democrat site (APN 143-040-135) added to the model, there are no apparent capacity deficiencies in the collection system located downstream of the planned development. The highest d/D in pipelines downstream of the Press Democrat or neighboring site is 0.78.

Flow velocity is below the City standard of 2.0 feet per second (fps) for ADWF for all but one pipe segment in the evaluated portion of the collection system. The addition of flow to the existing collection system results in an increase in flow velocity, which is generally beneficial for scouring debris from within the pipe.

5. Recommendations

Simply based on the results of the hydraulic modeling evaluation relative to City design criteria, and with planned upsizing of the pipeline located between the Press Democrat site and J. Rogers Lane due to the Five Creeks development, no modification of the existing collection system is required due to the development of the Press Democrat parcel or to the parcel due west of the Press Democrat parcel.

From:	Parastou Hooshialsadat
To:	Garrett, Vanessa
Cc:	Matt Winkelman
Bcc:	011146434
Subject:	RE: WW Lining at PS1 - follow up
Date:	Friday, August 18, 2017 2:52:00 PM
Attachments:	image002.png

Hi Vanessa,

Sure. We will send you the scope and fee next week.

Have a good weekend,
Parastou

From: Garrett, Vanessa [mailto:VMarin@rpcity.org]
Sent: Friday, August 18, 2017 2:28 PM
To: Parastou Hooshialsadat <Parastou.Hooshialsadat@ghd.com>
Cc: Matt Winkelman <Matt.Winkelman@ghd.com>
Subject: RE: WW Lining at PS1 - follow up

Hi Parastou,
I'd like to just upsize the 37' pipe in preparation for the future development which is looking like it will happen in the near future. The other pipes upstream of that one will need to get funded through the development that benefits from the upsizing, and can be designed at that point.

Does that make sense?

Vanessa Marin Garrett, PE
Civil Engineer
City of Rohnert Park
Direct Line: 707-588-2251
Fax: 707-794-9242

From: Parastou Hooshialsadat [mailto:Parastou.Hooshialsadat@ghd.com]
Sent: Tuesday, August 15, 2017 12:28 PM
To: Garrett, Vanessa <VMarin@rpcity.org>
Cc: Matt Winkelman <Matt.Winkelman@ghd.com>
Subject: FW: WW Lining at PS1 - follow up

Hi Vanessa,

Adam ran the sewer model based on the new information you sent us last week i.e. APN 143-040-135 will host 170 multi-family units. Please refer to the following email for the results. Basically, changing the last segment of 37' long pipe to 15" is fine if we consider 2.0 people/unit for new developments in the model.

If we consider 3.2 people/unit, we need to upsize all the three pipes from red circled MH to the pump station to 15".

Also, if we are certain that other parcels - outlined in yellow and fuschia - will be developed in the near future, we should upsize all pipes downstream of fuschia color parcel.

Please advise how to proceed so I can adjust the scope for design accordingly.

Thank you,
Parastou

From: Adam Fisher
Sent: Friday, August 11, 2017 2:33 PM
To: Parastou Hooshialsadat <Parastou.Hooshialsadat@ghd.com>; Matt Winkelman <Matt.Winkelman@ghd.com>
Subject: RE: WW Lining at PS1 - follow up

Hi Parastou and Matt,

Here are the impacts of the development of the parcel (143-040-135) referenced below.

Residential 2.0 people/unit:
Q_{ADWF} = 100 GPCD * 2.0 people/unit * 170 units = 34,000 gpd = 23.6 gpm
Q_{PDWF} = 23.6 gpm * 2.6 = 61.4 gpm

$Q_{I/I}$ = 6.53 acres * 1.4 gpm/acre = 9.1 gpm
Q_{PWWF} = 61.4 gpm + 9.1 gpm = 70.5 gpm

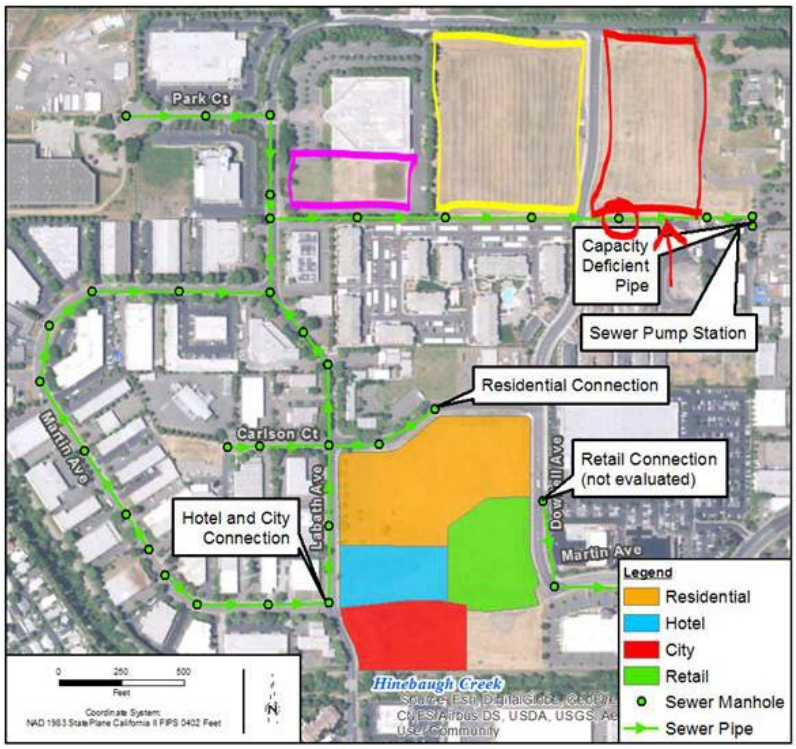
Residential 3.2 people/unit:
Q_{ADWF} = 100 GPCD * 3.2 people/unit * 170 units = 54,400 gpd = 37.8 gpm
Q_{PDWF} = 37.8 gpm * 2.6 = 98.2 gpm
$Q_{I/I}$ = 6.53 acres * 1.4 gpm/acre = 9.1 gpm
Q_{PWWF} = 98.2 gpm + 9.1 gpm = 107.4 gpm

Analysis/Results
See image below, red outline = parcel with 170 MF units, red circle = manhole where 170 MF parcel flow enter system, red arrow point to the impacted pipe, yellow and fuscia outlines indicate additional potential development.

At the residential density rate of 2.0 people/unit, the impact of the 170 MF parcel on the sewer system does not cause any additional surcharging or capacity deficient pipelines, the proposed upsizing of the last segment of 37' long pipe to 15" is still adequate.

At the residential density rate of 3.2 people/unit, the imapact of the 170 MF parcel on the sewer system causes the pipe with the red arrow to have a d/D of 1.0 (q/Q = 1.03). So, that pipe, plus the next one downstream should be upsized to 15". The 37' long pipe being upsized to 15" is still adequate with a d/D of 0.67.

If additional parcels are developed, outlined in yellow and fuschia, downstream pipes would likely need to be upsized.



Please let me know if you need any additional information or analysis.

Thanks,
Adam

Adam Fisher PE
Civil Engineer

GHD
T: 707 523 1010 | F: 707 527 8679 | E: Adam.Fisher@ghd.com
2235 Mercury Way, Suite 150, Santa Rosa, CA 95407, USA | www.ghd.com
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From: Parastou Hooshialsadat
Sent: Friday, August 11, 2017 10:09 AM
To: Adam Fisher <Adam.Fisher@ghd.com>
Subject: FW: WW Lining at PS1 - follow up

From: Garrett, Vanessa [mailto:VMarin@rpcity.org]
Sent: Friday, August 11, 2017 8:48 AM
To: Parastou Hooshialsadat <Parastou.Hooshialsadat@ghd.com>
Subject: RE: WW Lining at PS1 - follow up

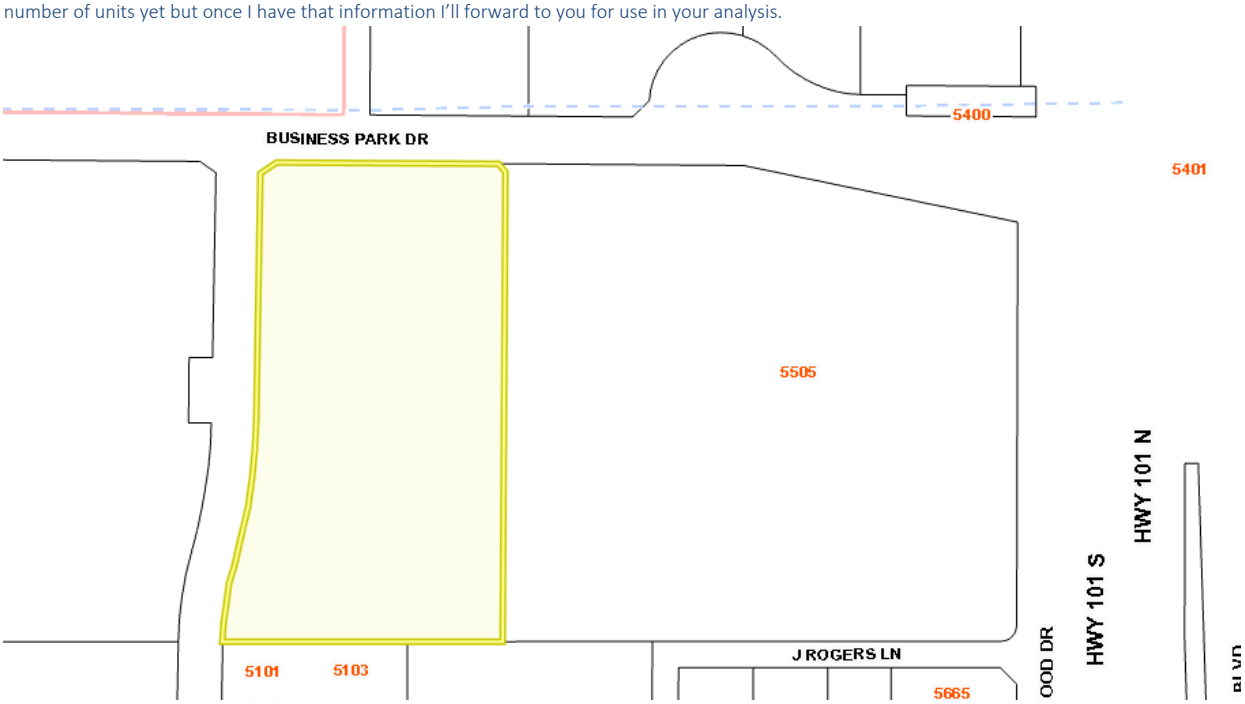
Hi Parastou,
I spoke with our planners and they say APN 143-040-135 will host 170 multi-family units according to the developer.

Vanessa Marin Garrett, PE
Civil Engineer
City of Rohnert Park
Direct Line: 707-588-2251
Fax: 707-794-9242

From: Garrett, Vanessa
Sent: Friday, August 11, 2017 8:34 AM
To: 'Parastou Hooshialsadat' <Parastou.Hooshialsadat@ghd.com>
Subject: RE: WW Lining at PS1 - follow up

Hi Parastou,
Let's wait until I issue a new task order to wrap up the condition assessment. That PO for the investigation was closed out on our end so it will have to happen as part of the design task order.

I've attached the proposed uses for the recent parcel split in that area. There are now 4 separate parcels, and those are all proposed to stay industrial/commercial uses (see attached report).
APN 143-040-135 (6.53 acres), which is just west of this parcel split, is proposing to put residential units. I don't have a conceptual drawing with the number of units yet but once I have that information I'll forward to you for use in your analysis.



Vanessa Marin Garrett, PE
Civil Engineer
City of Rohnert Park
Direct Line: 707-588-2251
Fax: 707-794-9242

From: Parastou Hooshialsadat [mailto:Parastou.Hooshialsadat@ghd.com]
Sent: Thursday, August 10, 2017 11:50 AM
To: Garrett, Vanessa <VMarin@rpcity.org>
Subject: WW Lining at PS1 - follow up

Hi Vanessa,

We didn't talk about finalizing the condition assessment report yesterday. Since we are going to have a new project for the design phase, I would like to include some effort to complete that task and have a final report for the pre-design activities. Please let me know if you have any objections.

Also, please send me the revised flows for Press Democrat property per the updated residential usage.

Thank you,
Parastou

Parastou Hooshialsadat, PE
Project Engineer

GHD
T: 707 523 1010 |D: 707 540 9695 | F: 707 527 8679 | E: Parastou.Hooshialsadat@ghd.com
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Document Status

Rev No.	Author	Reviewer		Approved for Issue		
		Name	Signature	Name	Signature	Date
1	P. Hooshialsadat	M. Winkelman				

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