Longview Regional Water Treatment Plant Constructibility Study

for the

The City of Longview Washington

May 7, 2008

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Prepared for

City of Longview
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Table of Contents

List of Tables

List of Figures

List of Appendices

Executive Summary

Overview

The City of Longview (City) Regional Water Treatment Plant (RWTP) Constructability Study (Study) is intended to provide the City with a decision-making tool to determine whether improvements to the RWTP can be accomplished efficiently and economically. The goal of the study was to identify RWTP deficiencies and needed improvements and develop a cost estimate and schedule for implementing the study's recommendations.

The following criteria were used when preparing the study:

- No consideration was given to alternative supply sources or treatment methods. This study focused solely on restoring and rehabilitating the existing facility.
- A peak day production capacity of at least 14 million gallons per day (MGD) is required from the RWTP while constructing the improvements.
- This analysis included replacement of the 36-inch transmission main from the RWTP to the Hillside reservoirs.

Findings

Summary of Existing Conditions

The evaluation began with a one-day condition assessment of the existing RWTP conducted by a team of engineers possessing specific expertise in similar restorative projects. In addition to visiting the RWTP, the team reviewed previous design plans, engineering studies, and other related data.

The following conclusions were developed after reviewing the information and data:

1. The existing intake structure can be restored to a capacity of 20 MGD with minimum permitting requirements. Construction of a new intake structure at a different location would trigger a more difficult and lengthy permitting process. River training is required to maintain the long-term viability of the existing intake.
2. Flocculation and Sedimentation Basins 2 and 3, constructed in 1945, have exceeded their useful life and must be replaced. Restoration of these facilities is not practical. Basin 1, constructed in 1978, has approximately 10 years of useful life remaining and will require replacement by year 2016.
3. Use of some of existing facilities, such as those constructed as part of the1998 improvements, can continue. These include the new rapid mix basins and filters. The solids handling facilities may require additional equipment.

Recommended Improvements

A phased construction approach will maintain adequate capacity while implementing the needed improvements. Three construction phases are recommended in order to construct 20 MGD of capacity. A fourth construction phase will be necessary if more than 20 MGD of capacity is desired in the future.

- Phase 1: Intake Improvements and River Training Analysis: In the first phase of the project, the existing intake is repaired and upstream modifications to the river flow pattern (river training) are completed. Phase 1 includes studies, design, construction, and construction management. Phase 1 will require about 3 years to complete at a cost of $6.2 million in 2008 dollars.

- Phase 2: New West Treatment Train Addition: The project's second phase includes new grit removal basins, raw water pipeline improvements, a new flocculation/sedimentation basin and filters, solids handling improvements, and a new 36-inch transmission main connecting the RWTP to the Hillside reservoirs. Phase 2, which may be completed concurrently with Phase 1, includes studies, design, construction, and construction management. Phase 2 will require about 4 years to complete at a cost of approximately $26 million in 2008 dollars.

- Phase 3: Demolition and Replacement of Treatment Trains 2 and 3: In the project's third phase, Trains 2 and 3 will be demolished and replaced with two new flocculation and sedimentation basins. The existing filters and clearwell will be retained, with modifications. The construction phase of Phase 3 is anticipated to start about 6 months after completion and startup of Phase 2 improvements. Phase 3 improvements will require about 3 years to complete at a cost of approximately $13.6 million in 2008 dollars.

- Phase 4: Upon completion of Phase 3, Train 1 will be the only remaining unrestored facility onsite. Although the train will reach the end of its useful life by year 2016, it can continue to provide about 7 MGD as a supplemental supply. At that time the City can abandon, upgrade or do nothing with this facility until later.

Conclusion

The constructability evaluation of the RWTP determined that a combination of new and restored facilities may be constructed at the existing site as long as the work is conducted in phases. The work will require about 8 years to complete, and the planning-level cost for the project is approximately $46 million in 2008 dollars.

Section 1: INTRODUCTION AND OBJECTIVES

The City of Longview Regional Water Treatment Plant (RWTP) was constructed in 1945 with additional improvements completed in 1979 and 1998. The RWTP is a conventional plant, treating Cowlitz River water through coagulation, flocculation, sedimentation, filtration, and disinfection. The plant has a maximum day capacity of about 14 million gallons per day (MGD). The City's 2005 Water System Plan (WSP) projects that maximum day demand in year 2025 will be about 17 MGD. To reliably meet the maximum day demand in year 2025, the WSP recommends a 20 MGD capacity for RWTP. The additional 3 MGD of capacity will allow the RWTP to meet maximum day demands even when minor process failures or maintenance events temporarily down-rate the plant.

The WSP also recommends a number of needed improvements to the RWTP, primarily an improved intake facility on the Cowlitz River and a better grit removal system. Recommendations also include repair or replacement of electrical systems, chemical feed, plant piping and pumping, much of which has been in service for more than 60 years.

Since completion of the WSP in 2005, the City has investigated other alternatives for improving the reliability and increasing the capacity of the potable water system. Studies conducted since 2005 have suggested that the RWTP may be at the end of its life. Furthermore, the deterioration of the raw water quality due to increasing amounts of grit and debris in the raw water has negatively affected RWTP operation.

1.1 Previous Studies and Engineering Documentation

The following information was used in preparing this report:

- The City of Longview Water Filtration Plant, John W. Cunningham and Associates, April 1944. Engineering plans for the initial construction of two conventional treatment trains.
- City of Longview Water Treatment Plant Expansion, Robert E. Meyers Consultants, July 1979. Engineering plans for a third treatment train (No. 3). Later changed to Train No. 1.
- City of Longview Regional Water Treatment Plant, Mt. St. Helens Emergency Expansion, Flocculation and Sedimentation Basin No. 2, Robert E. Meyers Consultants, September 1982. Engineering plans for an additional settling basin to remove ash from the Mt. St. Helens eruption.
- City of Longview Water Treatment Plant Upgrade, HDR Engineering Inc, July 1998. Engineering plans for restoring the existing eight filters, two new rapid mix basins, improving the three flocculation basins, and installing a new cyclone degritter. Provisions were made for a future treatment train on the side of the RWTP.
- Water System Plan for the Longview-Kelso Urban Area, Kennedy/Jenks Consultants, October 2005. A comprehensive water planning document for the Longview-Kelso urban area, including the Cowlitz County Public Utilities Department (PUD). The plan indicates that by year 2025, maximum day demand will be about 17 MGD. A RWTP capacity of 20 MGD is required to reliably meet the 2025 peak day demand. The plan identifies a number of RWTP-related improvements, including:
    - Prepare a feasibility study focusing on the alternatives available for repair or replacement of the river intake structure.

- Replace finished water piping manifold and raw water intake pipe; add grit removal facility and better solids handling equipment.
- Construct a new 36-inch transmission main connecting the RWTP to the Hillside reservoirs.

- Source Analysis, City of Longview, Pace Engineering Services, October 2006. Evaluates development of a groundwater supply as an alternative to further RWTP improvements. The analysis proposes a new groundwater supply and treatment plant at the Mint Farm. The new supply would supplement or replace the RWTP.
- Value Engineering Workshop Report, City of Longview, Washington, Water Supply Alternatives, Smith-Culp Consulting, 4 September 2007. Provides the results of a value engineering workshop focusing on the 2006 Source Analysis. The Report suggests that the City consider additional treatment alternatives and options before moving forward with the Mint Farm project. Potential options identified in the workshop include upgrading the existing RWTP or constructing a membrane plant or conventional filter plant at the Mint Farm.

1.2 Objectives

The objective of this analysis is to investigate the suitability and costs of restoring and/or replacing the existing RWTP and intake facility to provide a capacity of 20 MGD. Maintaining sufficient capacity to meet system demands while constructing the improvements is an equally important consideration. No consideration was given to alternative sources of supply or treatment methods as a part of this evaluation.

Two conceptual alternatives were initially considered:

1. Rehabilitating the existing RWTP
2. Demolishing the existing RWTP (or portions thereof) and constructing a new RWTP (or portions thereof) at the existing site.

The following criteria served as the basis for developing alternatives:

- Condition and performance of the surface water intake
- RWTP structural considerations
- Mechanical constraints
- Process and solids handling constraints
- Electrical and telemetry limitations
- Ability to maintain water production during rehabilitation or replacement construction
- Permitting issues that may affect scheduling and construction.
- Cost estimates for the proposed alternatives
- Scheduling of the proposed improvements.

Work on this project began with a field evaluation of the RWTP by a Kennedy/Jenks team of engineers specializing in water treatment plant restoration and expansion. The team inspected the structural, mechanical, process and electrical elements of the RWTP during the one-day evaluation. An engineer with expertise in river intakes and hydraulic issues inspected the intake facility. Treatment plant operating data and design drawings were also examined. Interviews with the RWTP operations staff were conducted to understand the challenges and limitations associated with existing plant.

After completing the field reconnaissance, the team summarized the general condition of the plant. Their assessment provided a basis for developing a strategy to restore and expand the RWTP. Besides the condition assessment, the following factors played a role in the development:

- A required production capacity of no less than 14 MGD throughout summer construction and a 7.0 MGD minimum during low demand periods.
- The need for a new 36-inch transmission main connecting the RWTP to the hillside reservoir.
- The City's desire to use, to the extent possible, improvements made to the RWTP in the most recent upgrades constructed in 1998.

Section 2: SUMMARY OF EXISTING CONDITIONS

A one-day site visit was performed by a Kennedy/Jenks team representing structural, electrical, mechanical and treatment process disciplines and an engineer with intake expertise. The condition assessment presented below is based upon that site visit and a review of construction drawings and relevant previous engineering reports.

2.1 Intake

2.1.1 River Aggradation

Analysis performed by the Army Corps of Engineers (Corps) indicates that the Cowlitz River channel bed is continually aggrading; in fact, the rate of aggradation predicted from the Corps' modeling efforts underestimated the actual rate. The material appears to be generally fine sands and silts containing fine volcanic ash. It is difficult for the existing plant to separate the material from the intake stream. Much of the material settles in the flocculation basins which are not equipped with solids removal equipment.

The City has periodically dredged of the Cowlitz River bed to open the intake area. The most recent activities included removal of a small point of land that protruded into the river channel from the right descending bank line just downstream of the intake structure. The point had caused a large eddy to form in the vicinity of the intake, possibly leading to increased deposition adjacent to the intake face and openings. The City is applying for permits to continue the dredging.

Periodic dredging by the City outboard of the intake since the eruption of Mt. St. Helens in 1980 has enabled the plant to maintain its required withdrawal rate. However, water withdrawal has experienced significant and increasing difficulty due to insufficient submergence of the intake openings above the river bed and the high proportion of fine sediment being drawn into the intake structure. Frequent maintenance dredging using diver and suction dredge is required to clear the narrow opening between the intake sill and the lower intake openings. Without such periodic dredging, only two or perhaps three openings are exposed to river flow, which are occasionally submerged by less than 24 inches in the summer.

City staff noted that the existing intake has a sediment exclusion sill constructed just a few feet riverward from the intake openings. The crest of the sill is well above the original river bed elevation, but it is now regularly overtopped by sediment moving down the river bed, which has aggraded entirely to, and perhaps above, the sill elevation. As sediment fills in between the sill wall and the lowest openings, the openings are progressively closed. The space between the sill and the intake openings must be cleaned regularly by divers using small hydraulic dredges. City staff report that the traveling screens can become so clogged and submerged with sediment that the screens collapse and fall into the pump chamber. The screens must be cleaned frequently, as often as twice per month during the winter and spring months and somewhat less frequently during summer and fall. Sand entrainment is a continual problem. In addition, the depth of flow over the intake openings can decrease to less than 24 inches during low summer flows due to deposition in front of the intake.

Corps of Engineers modeling studies for the Cowlitz River predict continued aggradation of the river bed due to volcanic ash, silts, and sands generated primarily by the eruption of Mt. Saint Helens in 1980. Corps studies from 2002 indicate that the river bed likely will aggrade fully to the present level by about the year 2030, and the rate of aggradation will gradually decrease as the bed level reaches equilibrium with the Columbia River channel. In fact, that predicted level has already nearly been reached, and the river continues to aggrade, although not as quickly as in prior years. A proposed Corps dredging project in the lower 2.5 miles of the Cowlitz River channel was expected to affect some downcutting, but it appears this downcutting will not extend upstream to the vicinity of the existing intake structure.

The existing intake structure is located on the right descending bankline of the Cowlitz River just upstream of the Kelso Bridge. The Cowlitz River watershed drains the northern and western slopes of Mt. St. Helens, an active volcano. The eruption of Mt. St. Helens in 1980, as well as subsequent eruptions and geologic instability associated with this active volcano, has continuously deposited prodigious quantities of coarse sands, pumice and volcanic ash into the Cowlitz River. To combat the sedimentation problem, the Corps has conducted periodic large-scale dredging activities along the Cowlitz and Columbia rivers at the mouth of the Cowlitz to remove sediment blocking the shipping channel on the river.

The value engineering (VE) report also discussed regular dredging using small, barge-mounted suction dredge systems, air scour devices and other means of moving sediment away from the existing intake. The report concluded that it would be most cost effective to use small suction dredges to periodically remove sediment as necessary to keep the intake clear. Reconfiguring the intake as an infiltration gallery in the bed of the river was also evaluated but ruled infeasible due to the large size required to deliver the desired flow requirement, and the inherent unreliability and difficult maintenance of such a system in this environment.

Moving the intake to the Columbia River was also evaluated briefly but summarily excluded because of the anticipated high cost of extending a supply waterline from the existing plant to a new Columbia River site. The cost, in combination with the possibility of contamination by radionuclides and other substances, as well as the necessity of moving Longview's water rights, caused this alternative to be rejected. In addition, the VE team evaluated, but found too expensive, the possibility of moving the intake across the channel to the left descending bank line, where a more consistent channel thalweg appears to have formed in recent years.

2.1.2 River Training Structures

The VE study concluded that it may not be feasible to use river training structures such as rock dike fields due to potentially increased flood stages that may raise concerns about the existing level of protection afforded to the cities of Kelso and Longview by the levees.

2.1.3 Assessment of Intake Condition and Deficiencies

The existing intake suffers from continued and increasing deposition of fine sediment on the river bed adjacent to the intake structure. When the intake is operating, sediment is readily drawn over the top of the sill wall and into the intake openings. There are 8 openings with gates across the width of the structure, each opening approximately 2.5 feet high by about 2.5 feet wide, and separated from each other in elevation and position across the face of the intake (see Appendix).

The lowest two gates have an invert elevation of 1.0 feet, or about 0.5 feet above the intake chamber. There is one gate above the first row, with an invert elevation about 4.3 feet. The fourth and fifth gated openings have invert elevations of 7.6 and 7.8 feet, respectively, while the sixth has an invert elevation of about 11.1 feet. The sixth gated opening is not submerged during summer low flows. The seventh and eighth gated openings are at invert elevations of about 14.4 feet. During low flows in the summer, only the lowermost five gated openings are typically submerged, and the upper three openings are not submerged except during higher winter flows or extreme high tides combined with moderately high river flows. (Although the Cowlitz is at least 50 miles from the Pacific Ocean, the Columbia River and its tributaries are affected somewhat by tide elevations in their lower extents).

The sediment gradation observed in the river channel is predominately sand sizes generally less than 1 mm in diameter, with a large proportion of low-density volcanic ash of finer gradation by volume. Plant operators have observed sediment waves moving down the Cowlitz River channel, as this fine material continues to be transported to the mouth of the river. Material appears to move throughout the year, not just in response to high river flows. Not surprisingly, similar bed movement and bed forms have been observed in other streams adversely affected by volcanic eruption materials.

Remediation of the existing intake, although not specifically evaluated in detail in the previous VE study, may be possible to realize an increase to the desired 20 MGD supply. To do so will require the amelioration of the adverse effects of fine sediment entrained into the intake, screen chamber and treatment plant. Exclusion of these sediments from the intake is the most desirable goal, with separation of the sediment at the RWTP less desirable but perhaps more easily accomplished.

The first and most obvious deficiency noted is the continued aggradation of the river bed and the occasional partial or complete isolation of the intake from the main thalweg of the river channel by large sediment bars and deposits across the river. Past dredging to reconnect the intake to the main thalweg of the channel has had limited and only temporary success.

Physical and/or numerical sediment transport and hydraulic modeling is required to assess a variety of river training structures. Not all of these carry the same risk of increased flood elevations. For example, some success has been accomplished with embedded or adjustable scour-generating structures in the Midwestern United States sand bed streams where navigation interests hold sway. Such structures have also been used to aid in scouring sand beds in the vicinity of intake structures. An investigation of alternative bed scouring devices and river thalweg training structures in either a numerical model or physical scale movable bed model (or both) is required of the Cowlitz River channel in the reach adjacent to the intake.

2.1.4 Fish Protection

The present intake structure does not comply with current fish protection standards. Fish are regularly entrained into the outer intake openings and often enter the screen chamber. There is currently no bypass available to pass entrained fish back to the river from the screen chamber or the intake chamber. The present fish protection guidelines require a screen of firm, non-deflectable wedge wire or profile bar with maximum opening size not to exceed 1.75 mm (0.069 inches) in width for fry (Natural Marine Fisheries Services 1995 and 2000) , with active cleaning mechanisms, semi-passive cleaning systems or passive screen area redundancy. In addition, current standards require that the calculated and measured normal approach velocity to the screen material not exceed 0.40 feet per second (Natural Marine Fisheries Services 1995 and 2000) to prevent impingement of juvenile salmonids and other aquatic vertebrates.

Federal and State resource agencies [Washington Department of Fish and Wildlife (WDFW) and National Oceanic and Atmospheric Administration (NOAA) Fisheries] also discourage the use of systems that entrain fish, even though a bypass may be provided to return entrained fish to the river. The desired intake capacity of 20 MGD [about 31 cubic feet per second (cfs)] therefore requires at least 85 square feet of screen area (including about 10% additional area normally required to account for partial obstruction of an uncleaned screen). The present opening area of the five lowermost gate openings provides up to perhaps 31 square feet below low summer flow water level (31.25 ft^2 for 4 30-inch x 30-inch openings). However, in the case where sediment deposition has submerged the lowermost three gates and only gates four and five are available, the existing available area is less than 13 square feet, and the intake cannot meet the prescribed velocity criterion of 0.4 feet per second (fps); it is currently at approximately 1 fps. Therefore, it would be necessary to create additional openings in the face of the intake structure to provide adequate screen area to meet fish protection standards. Moreover, the current mesh type of the traveling screens does not meet current fish protection standards.

The current configuration of the existing intake, with the bottom-oriented placement of the four intake gates leading to the screen chamber, tends to draw sediment into the screen chamber at all times when it is present inside the intake chamber. The entrainment of sediment into the screen chamber may be somewhat relieved by moving these openings higher in the water column, taking advantage of at least temporary storage of entrained sediment affected by the resulting sump at the bottom of the intake chamber that would be formed by such a modification. In addition, the current dimensions of these four gates result in fairly high velocities leading into the screen chamber, contributing to sediment entrainment and potential "hot spots," or areas of higher than desirable approach velocity on the screen material.

2.2 Treatment Processes

2.2.1 Degritter

A cyclone degritter was installed as part of the 1998 improvements. It removes a fraction of the larger entrained sand from the raw water. However, most of the sediment passes through the degritter and settles in the flocculation basins.

2.2.2 Coagulation

City staff indicate that one rapid mix basin handles two-thirds of the plant flow rate and delivers it to Trains 1 and 2. The other rapid mix basin handles one-third of the flow rate and delivers it to Train 3, with the capacity to also deliver to a future Train 4. RWTP staff previously conducted a flow distribution study that indicates that the flow rate through each of the three flocculation/ sedimentation trains is reasonably close to one-third of the RWTP flow rate.

2.2.3 Flocculation/Sedimentation

City staff report that two of the three sets of cast iron chains for the sedimentation basin chain and flight sludge collectors have been replaced. When a flocculator fails, a local machine shop rebuilds the unit. The flocculation basins lack adequate access for removal of grit deposits.

2.2.4 Filtration

City staff indicate that filters are typically backwashed after 48 to 96 hours of operation, but they may be backwashed every 20 to 24 hours when the RWTP flow rate is high. The operators perform each filter backwash step manually at the eight individual filter control consoles. The supervisory control and data acquisition (SCADA) system records the backwash rate and duration data, but it cannot automatically conduct the filter backwash steps. A printed list of the 25 filter backwash steps is posted at each filter console.

The water levels in the filters and the settled water channel are a control set point. The SCADA system adjusts the operating filters' filtered water control valves to maintain the water level in the settled water channel at the set point. The filter control valves also modulate to maintain the desired filtered water flow rate.

City staff state that the filters tend to go to a declining rate mode when the media are dirty. City staff also indicated that the filters are backwashed when the run time exceeds a set point, the head loss exceeds a set point, or the filtered water turbidity exceeds a set point. However, the filters are usually backwashed based on run time, and the accumulated head loss is generally less than 75% to 80% of the head loss set point when the filter is backwashed. Filters are usually backwashed based on run time because of the tendency to trend to a declining rate operating condition as the dirtier beds slow down and the cleaner beds handle increasingly higher flow rates.

Kennedy/Jenks Consultants observed a filter backwash between 12:00 and 12:30 p.m. on 11 March 2008 (Filter 8). The operator manually conducted each of the open/close valve operations, including manual adjustment of each of the three backwash rates. The initial backwash rate was low enough to operate with the four surface wash units in service.

Kennedy/Jenks Consultants inspected the filter gallery and observed filter-to-waste operation. The filtered water, filter-to-waste, backwash supply, and water filter backwash water valves at each of the eight filters have hydraulic actuators. Each filter has a set of solenoid valves that permit supplying pressurized water to the hydraulic actuators. The existing filter-to-waste discharge pipelines have eight air gaps separating the filter-to-waste discharge from the larger open pipe end on the waste filter backwash pipeline. A butterfly control valve is located immediately downstream of each propeller meter on the filtered water piping. This may have a significant impact on the meter's accuracy.

2.2.5 Disinfection

The City uses chlorine gas as a pre-oxidant and post-chlorinates to maintain a disinfectant residual. The City is considering replacing the Chlortainer 1-ton chlorine container containment vessels. City staff indicated that it requires 20 to 30 minutes to replace one 1-ton container with a full 1-ton container in the containment vessel. City staff indicated that on one occasion, the on-duty operator connected the pigtail from a new 1-ton container to the wrong external withdrawal valve connection; however, the error was corrected.

2.2.6 Disinfection By-Products

During the site visit, Kennedy/Jenks Consultants performed a cursory review of raw and treated water total organic content (TOC) data. The raw water TOC is typically 1.0 to 1.8 milligrams per liter (mg/L), and treated water TOC is <1.0 mg/L. The available trihalomethane (THM) and haloacetic acid (HAA5) lab reports on file at the RWTP indicated that nearly all THM and HAA5 data are below the Stage 1 Disinfection Byproducts Rule maximum contaminant level (MCL) and only one HAA5 quarterly sample (at Willow Grove) was above the HAA5 MCL.

2.3 Structural

2.3.1 River Intake and Pump Station

The river intake and pump station are reinforced concrete structures with the exception of the additional electrical room on the northern side of the pump station, which is a concrete masonry unit block bearing wall structure. Kennedy/Jenks Consultants observed the concrete of the intake from the exterior, and it appeared to be in adequate condition; however, extensive cracking was observed on the southern wall of the intake during the site visit (Photos 1 through 3). Both craze cracks and pattern cracking were observed on the southern side of the structure. Hairline cracking was also observed at regular intervals along the height of the wall. The age of the structure, the condition of the concrete, and the exposure of the reinforcing steel all contribute to continued deterioration of the structure with increasing size of cracks, size and number of spalls, and required repairs of the structure. It is unlikely that the intake and pump station would meet current building codes for soil and earthquake loading.

2.3.2 Degritter (Cyclone Separator) and Rapid Mix Basins

The degritter and rapid mix basin structures are relatively new (installed in 1998), and the reinforced concrete and miscellaneous metals of the structures appear to be in good condition.

2.3.3 Flocculation and Sedimentation Basins

Basins 2 and 3, in the center and western side of the plant, were constructed during the first phases of the plant's construction in 1945.

2.3.4 Flocculation and Sedimentation Basin 1 (East)

Basin 1 was constructed as part of the 1979 expansion. Extensive cracking and some leakage were observed on the eastern side walls of both the flocculation (Photo 9) and sedimentation (Photo 6) basins. Vertical cracks were observed at regular intervals of 4 to 6 feet (Photo 10), portions of the cracks had spalled (Photos 7 and 8), and there was evidence of previous attempts to repair the cracks. Efflorescence in the cracks was also evident (Photo 11). Efflorescence is the deposit of salts, usually white, that forms on the surface after the salt has emerged from solution from within the concrete. Evidence indicated that the cracks have led to corrosion of the reinforcing steel in the walls; corrosion stains (Photo 10) were noted on the walls of the flocculation basin.

Basin 1 will reach the end of its useful life within the next 10 years. Over the next few years, the quantity and size of cracks, spalling and leakage of the tanks are expected to continue to increase as the corrosion cycle destroys the reinforcing steel in the walls of the basins, producing larger cracks, spalls and leakage from the basins.

2.3.5 Flocculation and Sedimentation Basins 2 and 3

Kennedy/Jenks Consultants observed extensive cracking, spalling, and some leakage in the walls of the basin. Extensive cracking and spalling were observed beneath and along the sides of structural corbels supporting the channel that connect Basins 1 and 2 (Photos 13, 14 and 15). Extensive cracking, spalling, and leakage were also observed in the walls of Basin 2 (Photos17 and 18).

The cracking, spalling, and leakage in these basin walls, as well as the history of unsuccessful attempts to repair the cracks in the walls, indicates that the walls of these structures have reached the end of their useful life. In addition, the walls of Basins 2 and 3 are extremely thin and contain reinforcement in size and spacing which, while acceptable for the hydrostatic loads when the basins were designed, would be considered severely deficient under current code requirements for environmental structures.

2.3.6 Filter Basins

The basins appeared to be in acceptable condition, with minimal cracking and leakage.

Photo No. 1 – Intake and Pump Station

Photo No. 2 – Intake and Pump Station

Photo No. 3 – Intake and Pump Station

Photo No. 4 – Degritter and Rapid Mix Basin

Photo No. 5 – Flocculation/Sedimentation Basin 1 (East)

Photo No. 6 – Cracks and Spalls

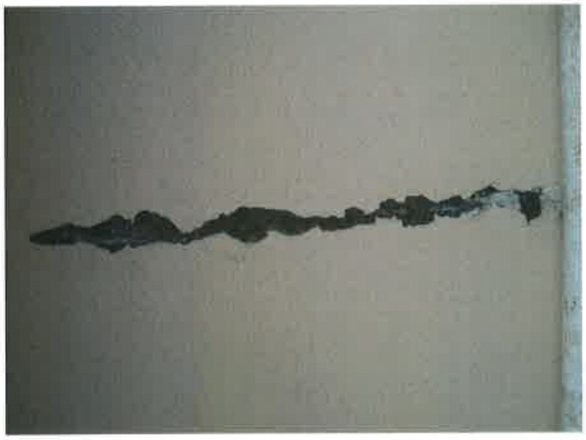


Photo No. 7 – Cracks and Spalls

Photo No. 8 – Closeup of Spall

Photo No. 9 – Flocculation Basin 1

Photo No. 10 – Flocculation Basin 1 Cracks

Photo No. 11 – Closeup Crack and Spalls

Photo No. 12 – Flocculation/Sedimentation Basins 2 and 3 (West)

Photo No. 13 – Flocculation/Sedimentation Basin 1

Photo No. 14 – Flocculation/Sedimentation Basin 1

Photo No. 15 – Flocculation/Sedimentation Basin 1

Photo No. 16 – Flocculation/Sedimentation Basin 1

Photo No. 17 – Flocculation/Sedimentation Basin 2

Photo No. 18 – Flocculation/Sedimentation Basin 2

Photo No. 19 – Flocculation/Sedimentation Basin 3

2.4 Mechanical

2.4.1 Intake Pump Station

The intake pumps appear to have worn out after 3 years of service due to the large quantity of sand, ash and grit being pumped through the system. The City reports that obtaining parts for the variable frequency drives (VFDs) on two of the pumps (GE Spectra series) is problematic with limited availability. The local displays indicated that a VFD was running at 42 Hz and a second VFD was running at 7 Hz, which is below the minimum frequency of approximately 20 Hz [or 360 revolutions per minute (rpm)]. According to City staff, the pump motors have been rebuilt in the last few years.

The condition of the underground electrical cable from the plant to the intake pump station could be an issue and should be investigated further.

2.5 Chemical Systems

2.5.1 Gas Storage

The City reports no problems with the chlorine gas stored in containment cylinders. The City keeps two cylinders onsite, with the resulting onsite storage apparently low. The chlorine supplier has been able to deliver chlorine on time as required.

2.5.2 Liquid Chemical Storage Room

The storage room lacks secondary containment. A code analysis should be performed to determine whether this chemical storage area meets all fire and other code requirements.

2.5.3 Upstairs Dry Chemical Storage

Dry sodium fluorosilicate was the main chemical being fed from this area during the plant visit. A code analysis is required to determine whether ventilation is required in this area.

2.5.4 Finished Water Pump Rooms

The condition of the pipe in the old pump room should be evaluated. Several repair clamps on the piping indicate past attempts to rehabilitate these lines.

The new pump room is in generally good condition. Pump 6 was howling during the site visit and the potential vibration should be investigated to determine whether the equipment is out of alignment.

2.5.5 Solids Handling

The potential for reclaiming the backwash water after solids removal should be investigated as a way to improve plant efficiency.

2.6 Electrical

2.6.1 Influent Pump Station

City staff reported that the motors and VFDs on both pumps have failed, although the cause was not reported. Given the distance from the plant main switchgear, it is possible that the feeder conductors are marginally undersized contributing to the failures. Two motor control centers (MCCs) are present (MCE1 and MCE2), each fed from one-half of the main switchgear. The MCCs appear to be in good condition.

2.6.2 Main Switchgear

The main plant switchgear is dual-fed, each half fed from a separate 3000 KVA utility transformer. The original design provided for a tie-breaker between the two halves of the switchgear, such that the whole plant could be fed from one transformer if the other experienced a power failure. This tie-breaker appears to have been removed, and this redundancy is no longer operational. City staff report that the plant has experienced very few power outages during its service life, and alternative standby power arrangements have not been deemed necessary.

The main switchgear is installed on the ground floor, essentially at road level. It has experienced at least two flood events, with water approximately 2 feet deep in the room. The main switchgear is heavily rusted and beyond its normal service life. The switchgear room sits atop a sump that experiences heavy water infiltration, requiring almost constant pumping to an outside stormwater drain.

2.6.3 Main Electrical Room MCCs

MCCs MCF, MCD, MCB and two lighting panel boards are located in the same electrical room as the main switchgear. The two panel boards appeared to be severely rusted. The MCCs do not show the same external signs of rust. A careful examination of the MCC internals would be required to determine the extent of any damage. With the exception of one solid-state starter that was installed recently, the MCCs and panel boards are more than 30 years old and therefore generally beyond their normal service life.

2.6.4 Other Electrical Room MCCs

Four other MCCs are located in the plant at various locations: MCA feeds the one set of High Lift pumps; MCG1 and MCG2 feed the residuals area; and MCC is in an electrical room on the first floor of the building. With the exception of MCG1 and MCG2, which are quite new and in good condition, the other MCCs are more than 30 years old and therefore generally beyond their normal service life. MCA has had a fairly new Allen-Bradley 1336-VT VFD added to it.

2.6.5 SCADA System

A number of Siemens programmable logic controllers (PLCs) are located throughout the plant and are connected to a SCADA system in the control room. The whole system was custom-provided, and City staff report that spare parts for the PLCs are difficult to obtain. The SCADA system is a stand-alone package, not connected to any other City system.

2.6.6 Electrical Loads

A brief review of the equipment indicates a total motor load of more than 2550 horsepower (HP) at the plant. This suggests that the utility transformers are oversized for the current switchgear arrangement, considering that both transformers must be operational in order for the plant to run at capacity, but they are within a normal range.

The main switchgear should be replaced with new equipment and relocated to avoid future flooding. All MCCs, with the exception of MCG1 and MCG2, are near the end of their useful life.

Section 3: RECOMMENDED IMPROVEMENTS

This section presents a conceptual plan to rehabilitate the aging plant facilities while maintaining a minimum treatment plant capacity of 14 MGD so that the City can meet anticipated summer demands during construction.

A major objective of the plant rehabilitation is to avoid excessive solids in the raw water and the accumulation of solids in treatment processes. The accumulation of sand, ash and grit within the flocculation basins decreases the effectiveness of flocculation, causes excessive plant downtime to remove the solids, and increases plant maintenance costs. Solids removal will be improved by a combination of intake and treatment process improvements.

The plan has been divided into four major phases:

- Phase 1 focuses on intake improvements, dredging to remove solids from the vicinity of the intake, and river training to increase the river flow rate and hence scouring of solids from the intake screens.
- Phase 2 includes construction of a new treatment train west of the existing plant facilities, raw water line improvements, and finished water pipe improvements.
- Phase 3 involves demolition of Trains 2 and 3 and construction of two new 7 MGD treatment trains to replace them.
- In Phase 4, Train 1 will be addressed, and the need for maintaining that train will be evaluated at that time. An overview of the construction phasing is presented in Figure 1.

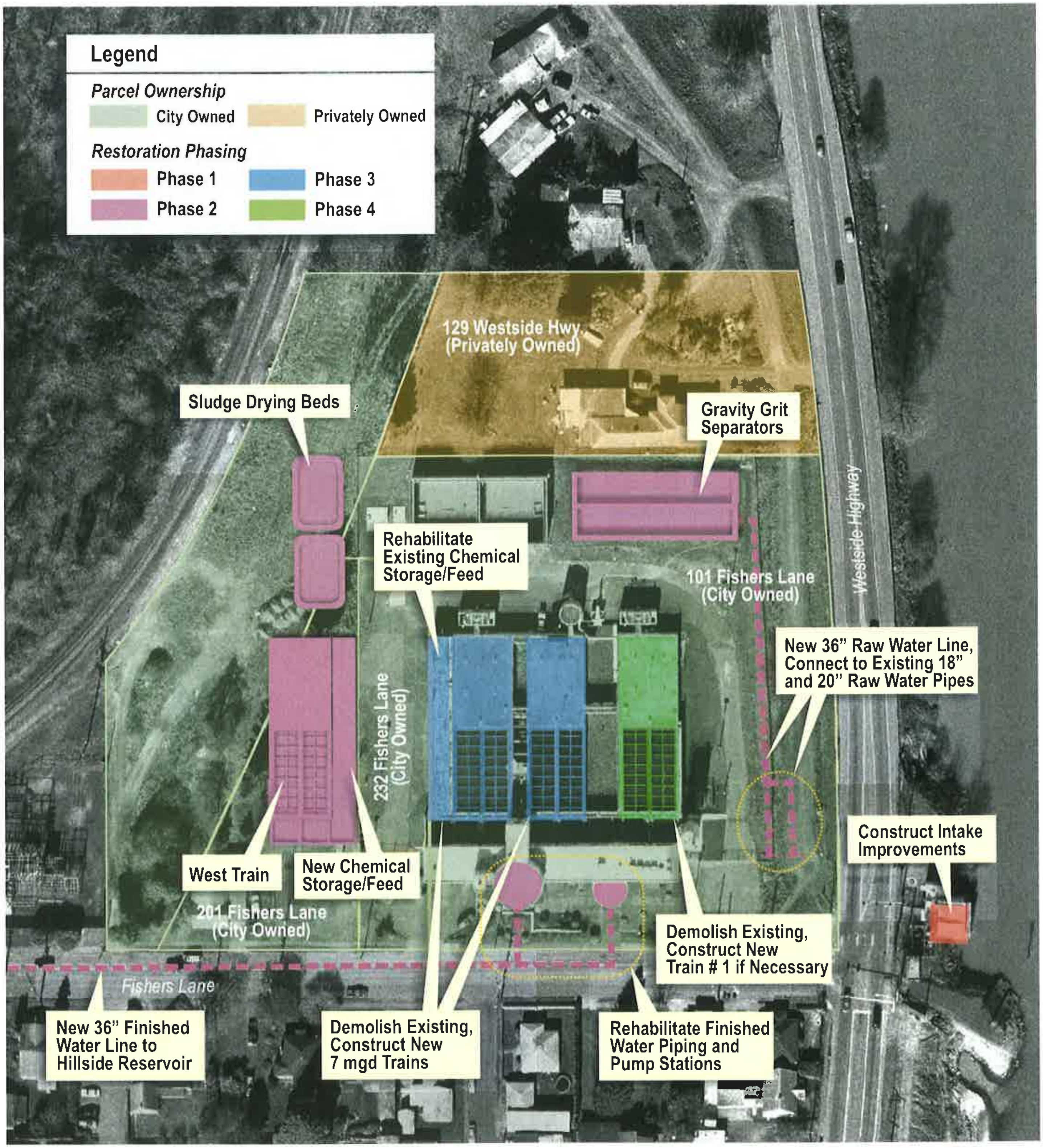
3.1 Phase 1: Intake Improvements and River Training Analysis

3.1.1 Overview

To improve the intake, a periodic dredging program to open the channel thalweg adjacent to the intake structure is required, as well as application of river training/scour generating structures to increase the clear water depth of the water column in the vicinity of the intake.

River training structures have been used with considerable success in numerous applications worldwide in sand-bed streams similar to the Cowlitz River. Embedded scour generating structures, such as Iowa vanes or some derivation thereof, have also been used to encourage less sediment deposition in particularly important areas of some streams, generally also sand-bed in character. Use of either a numerical sediment transport and hydrodynamic model or a physical scale model with moveable bed materials (or both) is required to evaluate and select the best alternative for this application and to verify the flood stage impacts.

The previous VE study cites flood stage impacts as a possible fatal flaw for river training structures. However, it is likely that a combination of training structures and levee improvements can be employed to maintain a clear channel immediately adjacent to the existing intake. Numerous precedents have been set using these techniques and modeling tools on a



Kennedy/Jenks Consultants

CITY OF LONGVIEW, WA
LONGVIEW REGIONAL WATER TREATMENT PLANT
CONSTRUCTIBILITY STUDY

CONSTRUCTION PHASING OVERVIEW

FIGURE 1

variety of applications. Hence, the relative expected success of such measures is reasonably high, if combined in the proper way.

3.1.2 Intake Openings

The existing intake openings do not provide sufficient screen area to meet current screening standards. Therefore, the number or size of openings must be increased to provide the necessary screen area. A more detailed analysis is required to evaluate alternative approaches for increasing the opening area. Alternatives that appear to be feasible are described in the following paragraphs.

The area of the openings could be increased by cutting through the outer wall of the existing intake structure, strengthening the resulting perforated intake structure wall, and installing the necessary screen, porosity control plates and additional control gates as required to provide the required minimum screen area. Based upon available information, 85 square feet of opening area would be required. A larger number and/or size of the openings will be necessary to provide the necessary opening area below elevation 9.5. It may be possible to install additional gates below and above existing gates, but each gate would require a separate operators to individually control each gate.

Alternatively, the entire opening area could be reconfigured by cutting eight larger openings, each about 5 feet wide by about 8 feet high, providing a net screen area of 85 square feet for each 4.25 feet in depth. The remaining concrete intake structure would require strengthening with integral galvanized steel structural framework for the larger openings, bulkhead guides and screen panels. These improvements will also include evaluating and repairing cracks in the concrete structure. Several cracks are apparent on the exterior of the exiting intake structure; however, the extent of interior cracking is unknown.

By using positive sealing bulkheads instead of individual gates, the water level from which intake flow could be drawn could be progressively raised to accommodate increasing sedimentation in the river channel, while providing a maximum intake area across the minimum necessary depth. Bulkheads would be raised and lowered using a small gantry from the existing deck.

In addition, periodic dredging or such river training/scour generating devices and structures are required to maintain appropriate conditions at the intake. The typical low flow water level in the river would provide for complete submergence of the lower row of four openings, according to drawings and river profile information provided by the City. Installation of either vertical wedge wire or profile bar screen material on the individual intake openings and internal air capture and directional vanes, coupled with air burst or mechanical cleaning systems, appears to be feasible. However, to confirm such application, more detailed analysis of the existing structure and the arrangement of the proposed screens, bulkheads and operating mechanisms would be necessary before proceeding with this concept.

3.1.3 Interior Intake Openings and Gates

Relocating the four gated interior openings to a location higher in the rear wall of the intake chamber and expanding them to provide for lower average flow velocity through the openings should be investigated. If the existing openings are not moved and/or expanded, areas of unacceptably concentrated flow and consequent high flow velocity would manifest on the intake opening screens. To attenuate high flow velocities, it is necessary to install head loss panels, typically orifice plates, behind the intake screens to distribute the flow more evenly. Although these panels may be difficult to access if located inside the current intake chamber, it is likely a more suitable location than on the outside of the intake structure. Regular maintenance and inspection is recommended for both the proposed intake screens and the porosity panels.

3.1.4 Traveling Screens

The traveling screen material requires replacement with a stiffer and more durable material, given the evident sediment concentration and deposition problems in the screen chamber. Modular, high-density plastic brick-type traveling intake screens have been used successfully in water intakes to exclude debris, sediment and aquatic vegetation. The City should investigate the application of such updated components for its screen chamber. In addition, a water level and sediment level monitoring system should be installed in the intake and screen chambers and outside the structure to measure the relative hydrostatic head across various components of the system. Warning alarms could be used to transmit the relative condition of the system so that emergency shutdown could be activated prior to structural or mechanical failure of the traveling screen or intake screen assemblies.

3.1.5 Recommendations for Alternatives Evaluation and Selection

A study is required to evaluate and select a river training configuration and verify performance over a range of expected river conditions. The study will include a sediment transport, a hydrodynamic numerical model and validation using a physical scale movable bed hydraulic model.

The first task of the modeling evaluation consists of data review and collection of the most up-to-date channel survey and bathymetry data. This would be followed by a preliminary development and evaluation of a range of possible river training or channel modification alternatives. The evaluation process proposed during this second task would likely include a two-dimensional numerical model of the channel in the vicinity of the intake, followed by a physical scale hydraulic model of the river channel using movable bed materials corresponding to the coarse sands observed in the existing bed material. Both the numerical and physical models would be able to test the ability of a structure or combination of structures to establish and maintain a clear water column in the vicinity of the intake and reduce or eliminate the need for periodic dredging to remove accumulated deposits.

3.1.6 Available Models and Validation Approach

The proposed modeling tools include SMS, a software package used to develop a computational mesh and input for the two-dimensional transport and hydrodynamic model. Based upon the site review and visit, the model would likely include a section of the river extending from approximately 2,000 feet downstream of the intake, near the downstream side of the Allen Street bridge, to approximately 2,000 feet upstream of the intake. The area in the immediate vicinity of the intake and across the river would be represented by a higher resolution model mesh. If necessary, to further separate any boundary effects or to further trace impacts of the modified condition, the model could be extended upstream and downstream as needed using cross-section data from the existing HEC-RAS model. Either the FESWMS or RMA2 code would be used to develop a numerical model. Both are linked with the SMS package, with neither offering an obvious benefit over the other. The resulting model would be tested and compared with the observations and velocity data collected during a channel survey at the site. Adjustments would be made to model parameters, including roughness and turbulence coefficients, to best replicate observed data.

3.1.7 Flood Elevation Evaluation

It will also be necessary to evaluate the effects of flood elevation changes generated by proposed channel modification treatments during the alternatives evaluation. The second task includes development of the channel modification design based upon the findings of the first task. This approach has been used with considerable success in many applications over the past 100 years or more, including countless navigation models, hydropower intake models, and water supply intake models.

In summary, the intake improvement recommendations are as follows:

- Conduct a base channel survey with velocity measurements for calibration purposes.
- Prepare a two-dimensional hydrodynamic and sediment transport numerical model to screen alternatives.
- Develop a list of preferred alternatives for river training structures.
- Verify performance of such river training structures in a physical scale movable bed model.
- Design and construct the intake structure modifications and river training structures.

3.1.8 Permitting Considerations

Information provided by the Portland District of the US Army Corps of Engineers indicates that construction activities that do not involve additional footprint area impacts outside the existing intake footprint do not require a Section 404 or Section 10 permit. However, construction of a new intake would require a Corps permit that would involve extensive negotiation with the consulting agencies (NOAA Fisheries, US Fish and Wildlife Service, Tribes, etc.). Dredging of any kind in the river channel (outside the footprint of the existing intake) requires a Corps Section 10 permit and a phased approach of application, review and consultation with directive, followed by revised and expanded application and associated review, consultation and directive, and again followed by a third round of revision and expansion of the application. Rehabilitation of the existing intake is preferred, because the permitting process is much less onerous than that required for construction of a new intake. Kennedy/Jenks Consultants understands that the City is applying for a dredging permit to remove some of the aggraded silt and ash in the vicinity of the intake.

3.1.9 Summary of Nationwide Permits Applicable to Intake Modification

The Corps' nationwide permit programs currently cover three specific areas of potential proposed actions: 1) Residential fill and removal of material from wetland areas, 2) outfalls and intakes, and 3) maintenance activities on existing riverine structures. Under these nationwide permits, minimal permissible activities may occur without permit, but only if they meet specific and stringent conditions. For example, maintenance activities (nationwide permit 3 above) on existing intake structures may be conducted with no Corps permit or consultation, provided the work, including construction, does not affect or disturb areas beyond the footprint of the existing structure. For all other work that does not qualify under the nationwide permit program, a Corps Section 10 or Section 404 permit is required.

3.1.10 Permitting Process

To qualify for exclusion from permitting under these programs, the proposed activities must meet a particular standard for the type and extent of work to be accomplished. These conditions are extensive. Activities that do not qualify for the nationwide permit (such as dredging more than 25 cubic yards to open a clear channel to the intake) must be permitted under conditions and requirements of Section 404 of the Clean Water Act and/or Section 10 of the Rivers and Harbors Act. Of the potential solutions proposed to resolve the sediment entrainment issues for the Longview municipal intake, it appears that the most time-consuming and potentially contentious issue will likely be dredging activities in excess of 25 cubic yards and replacement of the existing intake structure not located within the existing footprint of the intake. Based upon information provided by the Corps, the expected time frame and required submittals for such work are as follows:

1) Prepare site review and proposed plans for dredging activity.
2) Submit to Corps for agency review.
3) Wait 30 days for comment from consulting agencies from the time the permit application file is opened.
4) Agency review and comment or directive is received following 30-day comment period.
5) Project proponent must then prepare and submit to the Corps for agency review the proposed project plans in more detail, including a sampling and analysis plan for testing of dredged sediments for contaminants.
6) Wait 30 days for comment from consulting agencies from the time the revised permit application file is reopened.
7) Agency review and comment or directive is received following a 30-day comment period.
8) Project proponent must then prepare and submit to the Corps for agency review the proposed project plans in still greater detail with a sediment characterization plan with analysis of proposed dredged material, along with detailed contingency plans for use if contaminants are discovered.
9) Wait 30 days for comment from consulting agencies from the time the revised permit application file is reopened again to include this new information.
10) Agency review and comment or directive is received following a 30-day comment period.

At all times during this review process, the agencies may require special conditions and prescribe specific guidelines or criteria for the proposed dredging program, as well as direct special studies to provide coverage of uncertain conditions or data gaps. Agencies may request that the permit application or reapplication package be resubmitted if it does not include all information as required or directed. The 30-day review time does not commence until the permit application package is judged complete. If the submittals are complete, the Corps will not take more than 30 days in review time.

In addition to the Corps permits and agency consultation, it will be necessary to obtain a Washington Department of Ecology Section 401 Water Quality Certification for construction activities. The customary time frame for submittal of the 401 certification application is after receipt of the Corps of Engineers Section 404 or Section 10 permit approval, and the allowed Ecology comment period is up to 45 days.

A Hydraulic Project Application (HPA) must also be filed with the Washington Department of Fish and Wildlife for any in-water construction activities. Prior to submittal of this application, however, the WDFW must be consulted to determine the necessary conditions or modifications to the proposed design to meet WDFW review staff concerns. The allowed WDFW comment period on the HPA is 30 days.

In addition to State permits from Ecology and WDFW, a local jurisdiction building permit may be required, as well as a shoreline permit for all work proposed below the ordinary high water line. Often these permits are coordinated through the County or City Building Department using a Joint Aquatic Resources Permit Application (JARPA). The local jurisdiction will serve as a clearinghouse for permit comments and conditions submitted by other agencies and parties on the proposed action.

3.1.11 Anticipated Time Frame for Permitting

The intake modifications would be considered work within the existing footprint of the intake, and therefore a streamlined approach is applicable. The intake footprint includes all engineered structures, including the silt dam. A courtesy letter to the Corps is recommended to inform them that the work is being performed even though the Corps would have no jurisdiction over that work. The JARPA process is required. and that would include WDFW review, especially of the screen opening sizes and approach velocities. This process is expected to be 2 to 3 months in duration.

The river training structures would require a more lengthy permitting process, including a Section 404 permit from the Corps, the JARPA process, and review by several agencies. If the submittals are complete and the City initiates prior meetings with agency staff to provide information on the intended project and obtain preliminary feedback, the process can occur in 6 to 12 months.

3.1.12 Construction Considerations

The proposed river training structures will be constructed while operating the existing intake structure to its typical capacity under existing conditions. Some localized turbidity would be expected in the vicinity of the dredging equipment and/or water-based barge and placement equipment, which may have a temporary negative impact on intake performance. Dredging activities would be expected to cause the greatest impacts to turbidity, while pile driving and rock placement would be expected to create less turbid conditions.

Construction of the intake modifications would likely be best accomplished by isolating half of the existing intake structure with an internal diaphragm wall. Two interior gates serving one side