SEWPCC Upgrading/Expansion
Conceptual Design Report

SECTION 7 - Raw Sewage Pumping and Headworks

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7.0 Raw Sewage Pumping and Headworks

The purpose of this section is to evaluate the capability of the existing raw wastewater pumping, screening and grit removal system to meet the future design criteria, and to identify and define any associated upgrading/expansion requirements. Refer to Appendix E for the headworks existing hydraulic profile.

7.1 PLANT INLET WORKS AND RAW SEWAGE PUMPING

The interceptor flows and capacity was identified in Section 3. In this section we will review the influent wet well and associated mechanical to determine if the existing facilities are adequately sized.

The plant inlet works are required to accommodate the current nightly minimum flow of 22 ML/d and proposed peak wet weather flow of 415 ML/d. As part of this work the existing vibration issues with the pumping systems at low flow and how this might affect operation of the pumping facilities for the expanded facility is investigated. Options for modifying the pumping systems to provide the required plant design flows is also looked at.

7.1.1 Influent Interceptor

The 1980 mm diameter St. Mary's interceptor has a maximum capacity of 430 ML/d which roughly corresponds to the maximum proposed pumping capacity at the SEWPCC of 415 ML/d. The design capacity of the treatment and bypass systems were thoroughly discussed and finalized in the PDR. Any system modifications made to increase flow transfer to the SEWPCC would have to be evaluated separately, such as elimination of overflows at the Darcy Lift Station.

Raw sewage is conveyed to the SEWPCC via the St. Mary's Interceptor Sewer, which discharges to the plant inlet chamber. The plant inlet works are suitable to accommodate the proposed current nightly minimum flow of 22 ML/d and peak wet weather flow of 415 ML/d. No improvements are necessary to the inlet chamber.

7.1.2 Plant Influent Equipment and Wet Well

Two transition channels, each equipped with sluice gates at the upstream end, deliver sewage from the inlet chamber to the raw sewage pump well as shown in Figure 7.1. The raw sewage pumping facility is comprised of a wet well and dry pit pumping area. A dividing wall separates the pump well into two compartments namely, the East Wet Well and West Wet Well. Each wet well is fed separately by one of the transition channels. Each wet well compartment can be isolated from the incoming flow for maintenance by closing the corresponding sluice gate at the upstream end of the transition channel.

One of the major issues with upgrading of the piping and equipment attached to the wet well is that the influent gates do not work. We have been informed that these gates are being replaced with new gates as part of the criticality work being undertaken by SNC Lavalin. This work will allow the City to isolate the influent wet well chambers for maintenance and upgrading purposes. A structural inspection of the wet well concrete is recommended at that time in order to determine the condition and any need for remedial work. Concrete cores were taken from the Primary Settling Tank (PST) walls, which were constructed at the same time, and the concrete condition was good.

The need for interconnection between the two wet wells was reviewed. The installation of an interconnecting sluice gate to help equalize flows between chambers was considered. The analysis indicates that the chambers operate close to the Interceptor invert and as such there is minimal variation in wet well levels. This means an interconnecting sluice gate would provide minimal operational benefit. An additional sluice gate also has the potential to cause future operation and maintenance problems.

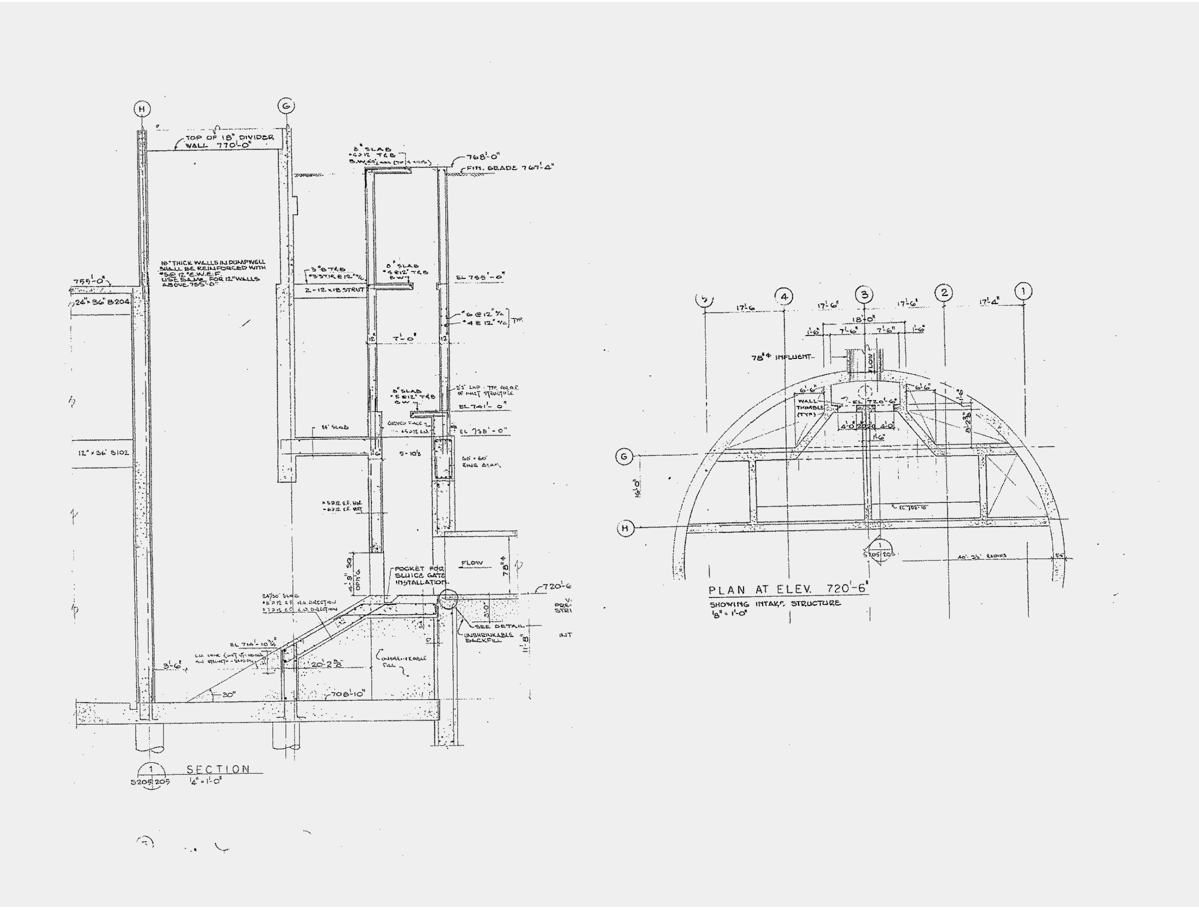


Figure 7.1: Influent Wet Well Plan and Section

There was discussion regarding the wet well configuration and any changes that should be implemented to reduce the possibility of vortexing in the wet well chamber. Currently the wet well chambers handle flows of 22 ML/d to as high as 388 ML/d without operational issues. It was also noted that the design flow of 415 ML/d will occur at high wet well levels, which will minimize any impact on the pump suction hydraulics. In conclusion, the wet well is adequately sized and is capable of handling the design flows.

7.1.3 Raw Sewage Pump Vibration

The Worthington raw sewage pumps have had some vibration issues in the past that required operators to be called out to site during the night to reset the system. A review of the plant records indicates that vibration related pump shutdowns have occurred as shown in Table 7.1.

Table 7.1 - Raw Sewage Pump Vibration Shutdowns

Year	No. of Vibration Related Failures
2005	43
2006	8
2007	4
2008	0

The vibration related pump shutdowns have steadily declined over time. The operators have implemented a variety of system upgrade aimed at reducing the number of vibration alarm callouts. It appears that these changes have been successful. It is believed that the most beneficial upgrade was a change in the vibration sensors from digital to analog in 2005 and an adjustment to the alarm set points. Additionally, the Variable Frequency Drive (VFD) turn down set points were adjusted to minimize vibration and the lowest speed permitted for the Worthington pumps was set to 560 rpm. The SEWPCC has also implemented a daily line flushing program that has likely resulted in a reduction of debris in the suction and discharge lines that could have been affecting flow (line flushing is currently manual and should be automated). The operators also indicated that overnight dry weather flows have increased slightly over the years resulting in fewer vibration issues.

As part of the review of pump vibration, an investigation into what might have been the cause of the vibration and what the potential solutions would be was undertaken. The pump vibration problems appear to be related to the high static and low frictional head losses within the piping system. As the pumps slow down, they come very close to the static head value. Thus speed reduction at low flows can result in the pump being unable to output enough pressure to overcome the static head. A minimum pump speed must be maintained to permit adequate discharge pressure to overcome the static head.

Additional testing was undertaken on April 21, 2008 that included speed reductions below 560 rpm to determine the minimum output of the smaller pumps. Raw sewage pump G101 operated well at 560 rpm and had unacceptable vibration at 540 rpm. The flow at 560 rpm was 27 ML/d. At 550 rpm, vibration output from the pump varied. Raw sewage pump (RSP) G104 operated well at 560 rpm and also at 550 rpm. It had unacceptable vibration at 540 rpm. RSP G104 produced a flow of 22ML/d at 550 rpm.

Through on-site testing the operators have determined that 560 rpm is the minimum speed the small RSP's can be operated without inducing excessive vibration. By setting this as the lowest speed permitted by the Worthington pumps, the vibration problem has been greatly reduced or eliminated. The operators have attempted to vary wet well operating levels to mitigate vibration issues but this was found to be ineffective.

Since there have been no alarms in 2008, it appears the problem has been resolved and no changes are required in the system. Should the vibration problem reoccur with any regularity in the future, additional system changes should be investigated. One option to temporarily increase flow would be to divert the Windsor Park P.S. to the SEWPCC (it currently flows to SEWPCC in summer only). Since we intend to maintain at least one of the existing Worthington pumps, the plant will continue to operate as it currently does at low flows and we do not anticipate any new pump vibration problems. We also understand that the operators have run the facility on a single large pump in the past (incorporating pump stops when flows get too low) so this is an option in future should the small pump(s) be down for maintenance.

If vibration issues are to occur in future when using a large pump as the lead overnight, we recommend:

- Adjust the speed reduction increment as wet well levels drop (programming change) as the pump would be less susceptible to dynamic changes of the flow with reducing speed if the incremental speed reduction were smaller.

- Provide short (30 minutes) shutdowns of all pumps to allow the wet well to fill (the operators currently implement shutdowns occasionally to exercise the pumps). This solution would require program changes including running the pumps at low speed following a shut down (provided wet well levels did not continue to increase) to minimize the number of start/stop cycles.

- Return flow to the wet well during low flows to keep pump speeds high enough that vibration is not a problem (mechanical and programming change). This would be a very inefficient solution.

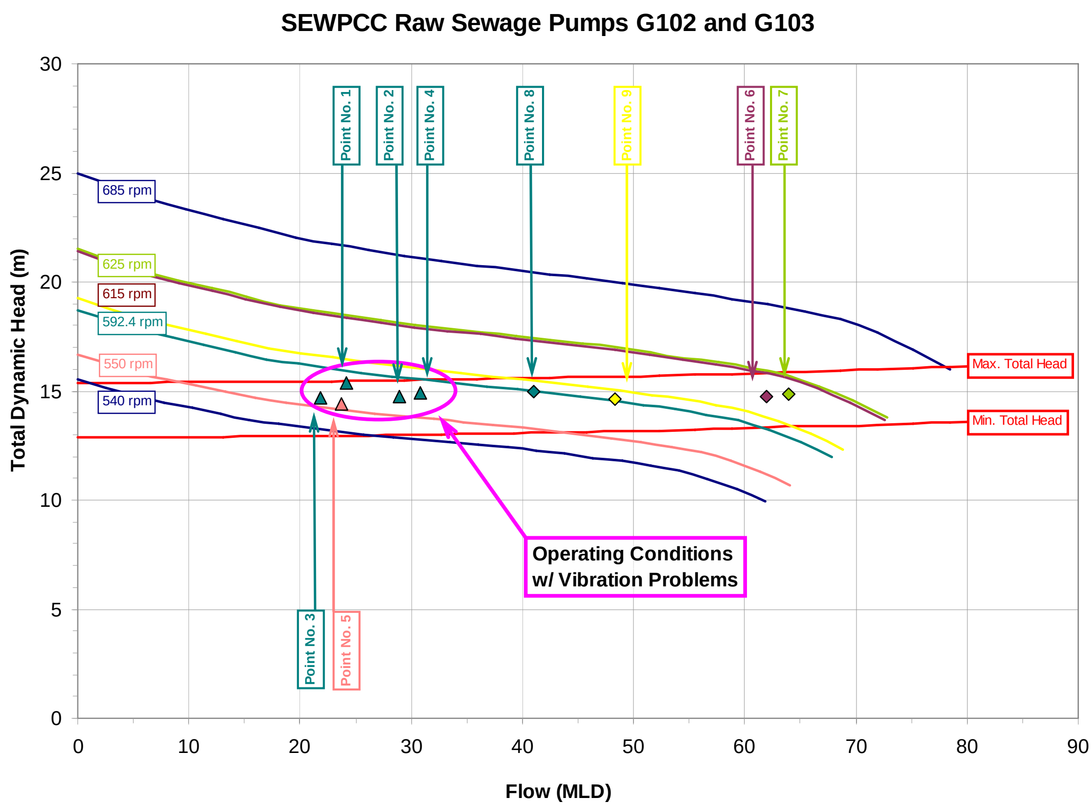


Figure 7.2: SEWPCC Pump Operation and Operation Test Points

7.1.4 Raw Sewage Pumping

There are a total of four (4) vertical centrifugal solids handling raw sewage pumps located in the annular dry well adjacent to the wet well as shown in Figure 7.3. Data obtained from the SEWPCC monitoring system indicates that the maximum pump flows are 114 ML/d for each of pumps G101-RSP and G014-RSP (Fairbanks Pumps); and 80 ML/day for each of pumps G102-RSP and G103-RSP (Worthington Pumps). The pump/system curves indicating flow and head with respect to pump speed are shown in Figures 7.4 and 7.5.

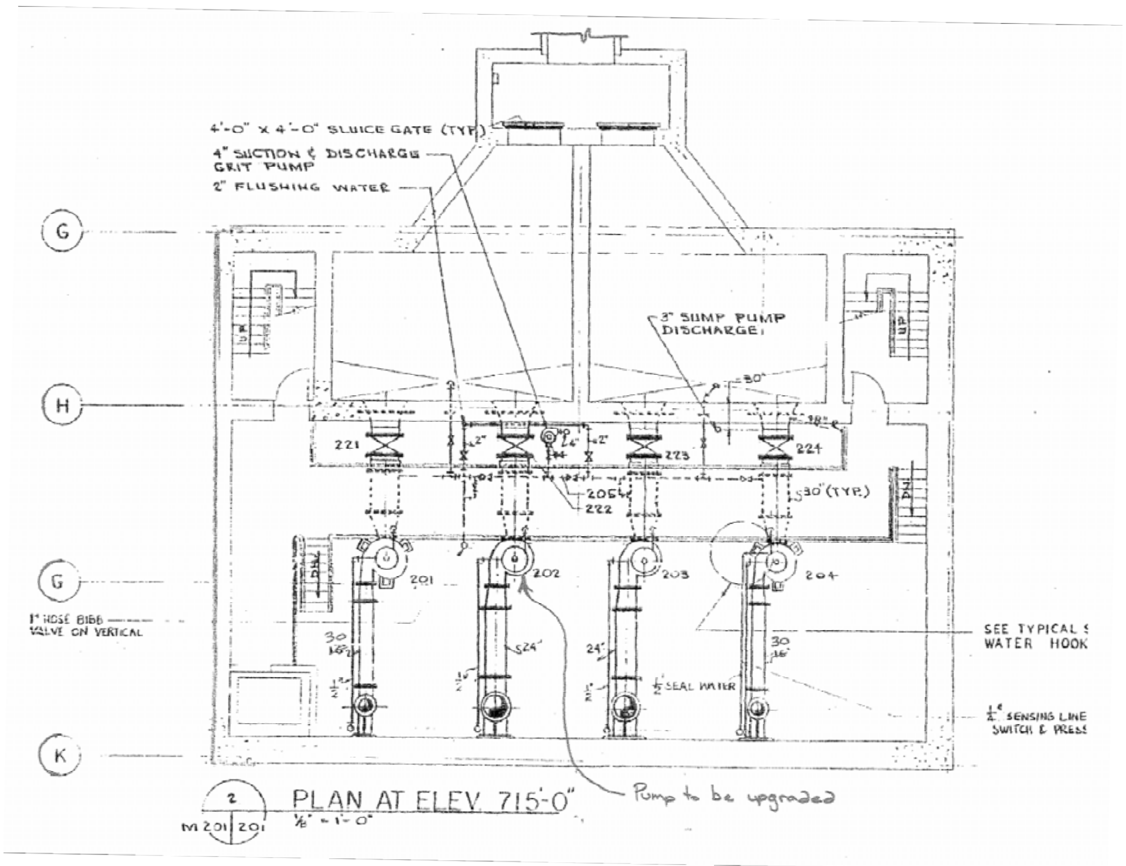


Figure 7.3: Influent Wet Well/Dry Well Existing Mechanical Floor Plan

The total installed pumping capacity (all four pumps in operation) is 388 ML/d and firm pumping capacity, which is defined as the pumping capacity with the largest pump out of service, is 274 ML/d. The design criteria for the raw sewage pumping are to provide a total pumping capacity of 415 ML/d and firm pumping capacity of 300 ML/d. We looked at several options to provide the increased flow.

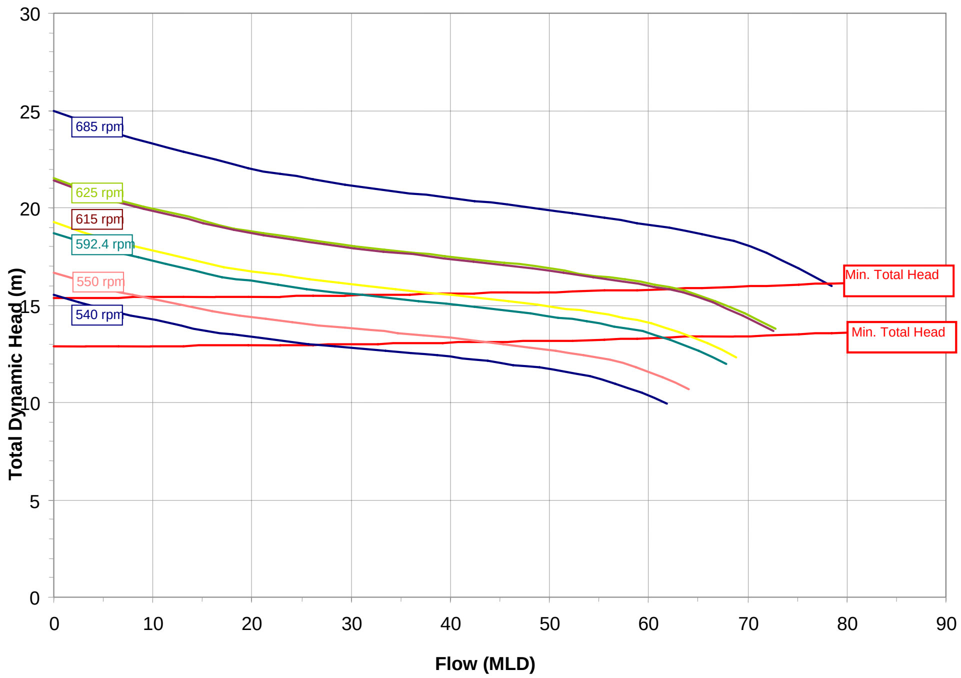


Figure 7.4: SEWPCC Raw Sewage Pumps G102-RSP and G103-RSP

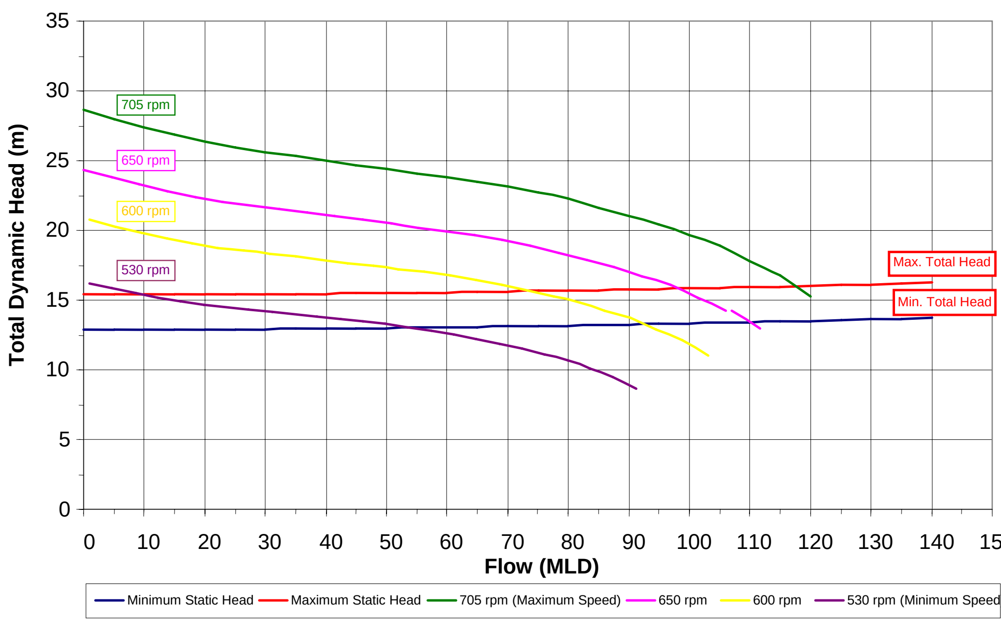


Figure 7.5: SEWPCC Raw Sewage Pumps G101-RSP and G104-RSP

7.1.5 Wet Well Operation and Pump Control Strategy

The SEWPCC operations staff provided information on the level set points and pump operation for the wet well. See Appendix F for information provided by the SEWPCC on pump operation levels and strategies.

Operational issues have been identified with the pump control strategy. One issue identified by the operators is that when a pump starts, it starts at full speed. This happens each time a second or third pump is called to run. It causes brief overflow periods in the plant where a portion of flow is directed to the bypass which affects effluent quality. Resolution of this issue is complex and additional work is recommended. In brief, the pump operation must be modified to either start at a reduced speed or turn down the operating pumps each time a new pump starts to prevent internal bypassing.

During investigation of the pump vibration issues, one potential issue contributing to pump vibration is the rate and size of rpm reduction. This is a control issue that could be resolved by decreasing the permissible rpm reduction increment from the current 10 rpm to 5 rpm. A limitation on how quickly a pump reduces speed should also be implemented. These changes should reduce vibration problems.

The pump shutdowns caused by low wet well level could be reduced by a slightly altered control strategy. During low flow periods, the wet well should be permitted to operate at a higher level without a second pump turning on. This could be accomplished by monitoring the rate of change in wet well level and modifying the control set points based on this rate. When the rate of level change and particularly the rate of level increase is low, the wet well would be allowed to fill to a higher level, thus reducing the number of times larger pumps turn on and the number of times smaller pumps are required to turn off due to low wet well level. Adjusting to this type of operational strategy would increase the efficiency of the system and also result in lower operational costs. Pump operation could be further refined if instantaneous data from the WWS flow meters was available at the SEWPCC.

Further examination of the wet well control strategy is strongly recommended. This will be required to reduce or eliminate internal plant overflows to the bypass channel. It would also reduce the number of pump shut downs due to low wet well levels.

7.1.6 Raw Sewage Pumping Upgrade Options

Several options to provide an increase in total flow of 27 ML/d to achieve the total desired pumping capacity of 415 ML/d were evaluated. The options are summarized as follows:

- Option 1 – Increase impeller size or increase motor size for existing pumps.
- Option 2 – Increase VFD operating speed.

- Option 3 – Replace an existing pump with a larger capacity pump.
- Option 4 – Install an additional new pump in the existing well.
- Option 5 – Construct a new wet well (shallow) and install an additional new pump.
- Option 6 – Defer pump upgrade.

Each option is discussed in detail below.

7.1.6.1 Option 1 - Existing Pump Modifications

Option 1 investigated upgrading of pump impellers, motors or VFD drives for the Fairbanks Morse Pumps or Worthington Pumps. The Fairbanks Morse pump supplier indicated that these pumps were still available but that the current impeller and motor combination was the maximum capable size. The Worthington pump supplier initially indicated that they could possibly be upgraded but later identified that upgradeability was not an option since this model of pump is no longer manufactured.

Option 1 can be discounted.

7.1.6.2 Option 2 - VFD Drive Adjustment

The VFDs were looked at to determine if the speed could be increased to obtain additional pump performance. The supplier responsible for installation of the City of Winnipeg VFDs at the SEWPCC is King's Electric Motors Ltd. They indicated that the drives for the Fairbanks Morse pumps could be adjusted to provide additional rpm output and thus achieve greater flow. Electrically, the VFDs and pump motors are capable of operating at higher speeds. Our preliminary calculations indicated that an increase of 50 rpm for each Fairbanks Morse pumps would provide the required additional flow. It was also noted that the pump was very close to the end of the operating curve when the speed is increased by 50 rpm and this could result in operational issues.

SEWPCC operations personnel undertook pump testing May 5, 2008 to determine the pump reaction (flow, vibration, amperage draw) at varied speeds. The data is shown in Figure 7.6.

Fairbanks Pump G101-RSP

Speed (rpm)	Flow (ML/day)	Wet Well Level (meters)	Vibration (in/sec)	Current (Amps)
705	110	3.93	0.050	320
720	114	4.31	0.113	348
730	92.94	4.41	0.128	343-365
740	95.56	4.34	0.126	345-370
750	95.77	4.31	0.135	368-378
760	98.01	4.31	0.195	378-390
770				
780	103	4.34	0.263	409-418
790	104	4.33	0.271	426-447
800	104	4.33	0.337	445-470

Fairbanks Pump G101-RSP

Speed (rpm)	Flow (ML/day)	Wet Well Level (meters)	Vibration (in/sec)	Current (Amps)
705	110	3.85	0.061	312-318
720	112	3.95	0.061	330-340
730	113	4.18	0.094	340-360
740	111	4.32	0.110	350-375
750	109	4.31	0.171	360-385
760	111	4.25	0.183	380-390
770	112	4.16	0.231	390-413
780	114	4.19	0.271	400-416
790	114	4.20	0.331	412-430
800	116	4.18	0.211	436-450

Fairbanks Pump G104-RSP

Speed (rpm)	Flow (ML/day)	Wet Well Level (meters)	Vibration (in/sec)	Current (Amps)
705	119	4.43	0.016	340
720	118	3.86	0.061	346-364
730	119	3.97	0.108	351-367
740	122	4.15	0.108	359-381
750	125	4.32	0.138	377-396
760	128	4.30	0.176	395-415
770	131	4.18	0.219	422-431
780	132	4.16	0.272	419-440
790	113	4.19	0.297	450-462
800	116	4.27	0.356	460-480

Fairbanks Pump G104-RSP

Speed (rpm)	Flow (ML/day)	Wet Well Level (meters)	Vibration (in/sec)	Current (Amps)

Figure 7.6: VFD Overspeed Test Results for Fairbanks Morse Pumps

The results of the overspeed testing are shown in Figure 7.6 and can be summarized as:

- G101-RSP can provide a maximum flow of 116 ML/d. This pump should provide more output but it is likely under performing due to wear.

- G104-RSP can provide a maximum flow of 132 ML/d. This pump has been recently refurbished.

The existing sewage pumps cannot be adjusted to obtain the additional flow required. It appears that as the pumps and impellers age, their performance is affected and any attempts to operate at higher speeds than those designated results in excessive vibration and is therefore not advisable. Option 2 can be discounted.

7.1.6.3 Option 3 - Pump Replacement

Option 3 looked at provision of a replacement pump for the 80 ML/d Worthington pumps that would provide 114 ML/d. Both the Fairbanks Morse and Flow-Serve (formerly Worthington) pump suppliers are able to provide a pump to meet this revised flow and head requirement. For the replacement pump, we identified that it would be advantageous to have the same pump as already exists for the two large pumps. The positive aspects of this are that it would result in interchangeable parts and no change in O&M procedures. It would also be easier to setup and use based on previous experience with the same pump. The Fairbanks Morse supplier indicated that the existing pump is still available. In order to incorporate this pump into the existing dry pit and attain the required flows extensive discharge piping and valve replacements would be required in order to match head loss values, costing roughly $300,000 in addition to the $400,000 cost of a new pump.

To avoid having to replace the existing piping and valves, pumps that could provide the required flows within the existing configuration were investigated. This has the potential to minimize the costs of upgrading the pump capacity. Both Fairbanks Morse and Flow-Serve have pumps that meet this requirement. The cost of this option is approximately $400,000.

Designing the facility to operate on a single 80 ML/d pump in combination with three larger pumps creates an operational concern during dry weather flows, which exist during much of the year. Currently dry weather flows can be as low as 22 ML/d during overnight periods. Testing has confirmed that the larger pumps can be turned down to a flow of about 27 ML/d at 560 rpm without vibration concerns, but no lower. The City would have two options to operate the facility during overnight dry weather flows utilizing a large pump. These would be to either 1) incorporate periodic pump shut downs (say half hour in duration) or 2) utilize St. Mary's interceptor for equalization starting at some predetermined time in the evening. Both of these options would require that the large pump be programmed to run at minimum speed when it is turned on so it does not automatically pump the wet well down in a short period of time. Operating the facility as described in Option 1 does not present any operational concerns, while the impacts related to odor generation and sedimentation in the interceptor sewer would have to be investigated in more detail for the second option. The City operated the facility for a number of days this spring by implementing the first option without any operational problems.

In order to minimize the potential downtime associated with having to remove the 80 ML/day pump for maintenance, it is recommended that the City keep the second pump that is removed from the system on hand as a spare or for parts.

7.1.6.4 Option 4 - Add a Fifth Pump

Installation of a fifth smaller pump to provide the additional flow of 27 ML/day was investigated. The smaller pump would likely be 100HP and would be installed in a similar configuration to the existing pumps. There is room in the existing dry pit to install a small pump of this nature but it would be a tight fit. The advantages are that the existing pump configuration could be maintained, a fifth pump would provide better handling of low flows, and structural modifications would be minimal. The downside is that it would reduce accessibility to the existing pumps, it would require breaking into the wet well with a new suction line, breaking into the grit channel with a new discharge line, reworking of the wet well grit removal piping, new piping for the pump, and new associated electrical items. Hydraulic and structural evaluations would be required to determine if a new suction line from the wet well would be problematic. The cost for this option is approximately $700,000.

7.1.6.5 Option 5 - New Wet Well and Additional Pump

Installing an additional pump in a new wet well at a higher elevation than the current wet well to provide increased pumping capacity during high flow wet weather events was also considered. It was proposed to set the wet well floor at a higher elevation in order to avoid the extremely high costs associated with installing a deeper wet well in the very challenging groundwater conditions at depth at the site. This pump would only be used during high flows to pump when the interceptor sewer is surcharged. The advantage of this concept is that it would permit ample space for a new pump installation, provide for future pump installations, and not affect the current pumping scenario. The downside is that this requires new structural components in an area already known to have difficult constructability, all new electrical and mechanical would be needed, and it would likely mean going to a submersed type application. It is a similar concept to adding a new pump to the existing dry pit except for the depth of the pump. The cost for this option is approximately $2,500,000. The option 4 scenario provides better functionality and is much less costly. Thus this option is discounted.

7.1.6.6 Option 6 - Defer Pump Installation

One of the options is to defer the addition of pumping capacity to a future date. The present pumping capacity is 388 ML/d, which is very close to the desired value of 415 ML/d. Depending on the budgetary position of the project, this work could be deferred as a cost savings measure. Proceeding with this option would result in the project criteria not being achieved.

7.1.6.7 Electrical Requirements

Options 3, 4 and 5 would all require electrical upgrades for power and they could all be tied into the existing control system. For option 3, pump replacement, the change in motor size would be from the existing 250 hp to a new 500 hp. This is a significant increase in power requirements and would likely require upgrading of the plant power service feed. A new VFD would be required as well as new power cables.

The pumps associated with option 4 and 5 would be smaller than the 500 hp pump required for option 3, but would be new and not a replacement. Thus the overall power increase requirements would be similar to option 3 in terms of facility power service upgrades. New VFD equipment would also be necessary. New power cables and connection to control systems would be required.

The specific plant power upgrades attributed to any of these pump installations have not been analyzed specifically in this section. The overall power requirements of the proposed new equipment is examined in Section 17. All costs associated with the electrical upgrades to the pumping system are included in the general electrical cost and not specifically identified here.

7.1.7 Influent Systems and Raw Waste Water Pumping Conclusion and Recommendation

To achieve both total and firm pumping capacities it is recommended that the City replace one of the Worthington pumps with a larger unit. The least cost option would be to install a pump slightly larger than two existing pumps as this would permit the existing piping configuration to be utilized while incurring a minimal cost increase for the slightly larger pump. This would require minor work to the existing mechanical piping in order to fit the pump in place and upgrades would primarily be for the associated electrical equipment. The slightly larger pump is required to handle the additional head loss through the smaller piping associated with the current Worthington pump mechanical configuration. This option is anticipated to cost $400,000 for a pump replacement. A pump the same as the existing Fairbanks Morse pumps could be installed but the discharge piping would have to be replaced increasing the cost to $700,000.

7.1.8 Dry Well HVAC Upgrades

Calculations were done to determine the ventilation requirements for the dry well. Using NFPA 820 as a guideline, the dry well should be provided with 6 air changes per hour. This works out to a volume of 16,284 m^3. Data was obtained from the existing operation and maintenance manual HVAC schematic that indicate a design air flow of 1464 L/s for the dry well. This roughly works out to 2 air changes per hour. If no work was being undertaken in the dry well, it is likely that no additional work would be required on this issue as it likely met code during the original design.

Unfortunately, replacement of one of the existing RSP's could result in the need for the dry well to meet current code requirements. This would entail adding additional air makeup units and exhaust fans that would triple the current air flow in the room. Additional work would included a heat exchanger, additional heating, and new ventilation. The scope of work was not looked at in detail and will have to be refined. If the pump upgrade is not undertaken, this work will not be necessary.

An allowance has been carried for the cost of this work as it will be required if the recommended pump replacement is undertaken.

7.1.9 Raw Sewage Well Grit Removal System

The raw sewage pump well is fitted with a grit removal system, which is comprised of a single grit pump located in the dry well of the pumping station. The grit pump suction piping is connected directly to each wet well and to the suction piping of each raw sewage pump (six connections in total). The pumped grit is conveyed to the inlet channels of the preliminary treatment facility.

This system is currently not being operated due to deterioration of the mechanical systems. It is our understanding that this system was reviewed by SNC Lavalin and it is recommended in their report that this be upgraded once new sluice gates are installed. At this time we have not included upgrading of this system in our scope of work as it may be included in the Risk and Criticality work being undertaken by SNC. It is reported to have operated adequately in the past prior to deterioration of the mechanical systems. If upgrading of this system is not completed by the time of project implementation, replacement of the current system with one constructed of corrosion resistant materials, likely 304 Stainless Steel, should be considered.

7.1.10 Constructability

The raw sewage pump modifications can be implemented during dry weather flows with little or no interruption in plant operation. To facilitate the pump replacement, the gate valve on the suction piping would be closed to isolate the pump from the wet well. This would facilitate removal of the pump and corresponding suction and discharge piping while ensuring the balance of the existing pump capacity remains in service. Under this condition, the firm and total pumping capacity would be 194 ML/d and 308 ML/d respectively.

The outlet of the pump discharge piping is situated above the HWL in the discharge channel. Provided the actual water level in the pump discharge channel remains below the HWL, there would not be any problems with backflow of sewage from the pump discharge channel to the dry well while the discharge piping is disassembled. However, from a construction safety perspective it is suggested that a blind flange be welded to the outlet of the pump discharge to prevent flooding of the dry well in the event of some unusual or unforeseen circumstances that cause the liquid level in the discharge channel to rise above the HWL.

7.2 HEADWORKS

7.2.1 Preliminary Treatment

Preliminary treatment is typically comprised of screening and grit removal facilities, which are located upstream of the primary and secondary treatment processes. The following provides a description of the unit processes that comprise preliminary treatment at the SEWPCC as well as the proposed modifications to the preliminary treatment facility. The inlet channels that convey sewage through preliminary treatment have been included in this discussion.

7.2.2 Projected Flows

As indicated in the previous section, the raw sewage pumping facility will have firm and total capacities of 300 ML/d and 415 ML/d respectively. It has also been determined that the primary treatment process, located downstream of the preliminary treatment facility, will be designed for a peak wet weather flow of 200 ML/d in order to meet the effluent quality criteria that was established for the project. Minimum flow during winter months is estimated to be in the order of 22 ML/d, with average flows in the 70-90 ML/d range.

It is proposed to design the preliminary treatment facility to accommodate the total pumping capacity of 415 ML/d. Any flows in excess of the proposed primary treatment capacity of 200 ML/d would bypass to the BNR process or outfall sewer at a location immediately downstream of the preliminary treatment facility (and upstream of primary treatment).

7.2.3 Screening Facility

7.2.3.1 Function

Bar screens are typically provided as the first stage of treatment. These screens remove rags, sticks and other oversized debris from the incoming sewage flow and this protects downstream plant equipment against reduced operating efficiency, blockage or damage.

7.2.3.2 Existing Facility

The existing bar screen facility is comprised of the pump discharge channel, three screening channels and one bypass channel. The screening channels are equipped with stop logs on the upstream and downstream end, which allows plant staff to take screens and the corresponding channel out of service for maintenance purposes. The bypass channel is equipped with automatically actuated sluice gates on the upstream and downstream end of the channel to automatically control bypassing of flows around the screening channels in the event of a high liquid level in the pump discharge channel. Operators have indicated that the bypass channel is no longer used.

The screening facility includes three automatically actuated, mechanically cleaned bar screens, located in the respective screening channel. The screens are the Infilco Degremont Inc (IDI), climber type screen with the bar rack having 12 mm (½") clear spacing. The screens have an automatic rake mechanism that cleans the front of the bar rack and deposits screenings into the grit conveyor, which carries and discharges the screenings and grit to a disposal bin in the Grit Truck Bay. The disposal bin is emptied periodically and the screenings and grit are hauled to a landfill for disposal.

Normal operation provides for all three screens to be in service concurrently and flows are evenly distributed between the in service channels. The normal cleaning cycle is initiated based on an operator selectable cleaning cycle to provide for intermittent cleaning of the screens. A continuous cleaning cycle is also initiated when either a high level in the pump discharge channel or a high differential level (between the upstream and downstream side of the screen) occurs. Once the high level or high differential level drops to the corresponding low level set point, the normal cleaning cycle is resumed.

Flow through the bar screen channels is controlled by a weir downstream of the grit tanks that establishes a minimum flow depth of 1.435 m (4.7') in the screen channels. IDI has confirmed the capacity of each existing screen to be 180 ML/d with a headloss of 0.163 m. Therefore, there is sufficient capacity with the three existing screens to accommodate the anticipated peak flow condition of 415 ML/d.

7.2.3.3 Screening Upgrade Options

Screening options were evaluated to provide additional debris removal for the BNR process downstream and floatables control for bypasses to the river.

Conversion to 10 mm (3/8") or 6 mm (1/4") screens were evaluated to provide additional debris removal for all influent flow. The screen manufacturer indicated that the 6 mm spacing would result in an expected 50% blockage and a significant increase in headloss. For this application IDI would not supply 6 mm rake style screens and this means significant structural modifications would be required to implement 6mm screening. Considering the physical installation problems and more importantly the blockage and headloss issues, use of 6mm screens is not feasible.

The reduction in bar spacing to 10 mm reduces the capacity of each screen to 150 ML/d and increases the headloss by 30%. The estimated equipment cost for this conversion is $75,000 (US) per screen. Although 10 mm screens have the capacity to pass the peak flow with all three units in service, the additional headloss, minimal factor of safety resulting from the capacity reduction and cost eliminates this option from consideration. Furthermore, since the majority of flow conditions allow for primary clarification prior to the BNR process, the additional debris removal gained by a finer screening is also accomplished though the primary clarification process.

Because of the primary clarification process, the existing 12 mm screens are adequate for the flow directed through the aerated grit and the treatment process. However, for flow directed to the bypass, additional screening and floatables control is required to protect the downstream processes. The bypass flow is later directed through the bioreactors and as such, there would be a concern regarding blinding of the bypass screens if the flow was not passed through a minimum 6 mm screen upstream.

Hydraulics for the screens had to be determined in order for the manufacturers to provide quotations. The conceptual design includes two channels that will permit installation of two 50% capacity screens, one screen per channel. The two channel design provides better hydraulics leading to the screens. In general each channel is proposed to be 1200 mm wide and approximately 1,500 mm deep. This will provide flow velocity in the channel of 0.6 m/s.

A number of screen types were considered for the bypass application. These included ban screens, bar screens and climber type screens. There can be issues with cleaning of the ban screens and they can have trouble with larger debris so they were not investigated. Because the existing screens are bar type and they are favored by the City, this type was investigated. Climber screens were also investigated and provide the additional benefit of remaining constantly clean and pricing was obtained for these also. Both bar screens and climber screens are acceptable for screening of the bypass flow. They both provide similar head loss (in the range of 200 mm across each screen) and cost approximately $200,000 per screen for supply. The conceptual design identifies a bar screen as the recommended option. However, additional investigation is recommended during the design phase on two fronts. One is to investigate the use of a 4 mm screen. The additional cost is about $10,000 per screen and they provide additional protection for the bioreactor without a significant head loss addition. The second item that should be given additional consideration is the use of a climber screen. This type of screen provides the added benefit of constant cleaning, but the City would have to be comfortable using this type of screen as it differs from the existing style at the facility.

7.2.3.4 Proposed Screening Facility

Two of the existing coarse screens in operation provide adequate capacity for flow rates up to 360 ML/d with one unit serving as a standby unit. Total peak screening capacity is 540 ML/d which is more than sufficient to accommodate the total pumping capacity of 415 ML/d for the proposed SEWPCC upgrade and expansion. Normal operation could continue to be all three screens in service and equal distribution to each of the three screens, but consideration should be given to operating only one or two screens during low flow conditions in order to reduce grit accumulation in the channels. This issue is discussed in further detail in Section 7.2.5. The existing bypass channel gate would be repaired and the channel would be maintained to allow for bypass of the screening facility in the unlikely event of a high level in the pump discharge channel due to extreme debris loading conditions or equipment failure. A static manually cleaned screen with spacing up to 25 mm (1”) could be considered to protect downstream processes in emergency conditions. However, these conditions are not anticipated to occur frequently enough to warrant installation of a manual screen in the bypass channel and thus it is not recommended.

Screenings are currently discharged on to a conveyor system and conveyed with grit to a hopper or truck on the east side of the Screening Room. This room also serves as a service bay for receiving raw sewage pumps and motors pulled for maintenance. The addition of a fourth bar screen would require significant modification to this area and is not recommended.

7.2.4 Conveyor

The existing conveyor that transports screening and grit material to the collection hopper will require replacement. With the new grit equipment, new grit classifiers will be required and are proposed to be placed in the existing screening room. This will require a longer conveyor which will be accomplished by replacing the existing conveyor. The City has had a good experience with the existing conveyor and thus it is proposed to extend the existing conveyor or replace it with a similar type. The conveyor length would be approximately 20 meters long. Potential upgrades such as hardware, a scrapper, and a safety stop switch will be investigated and proposed as options to be included in the specification. The budget price for a new belt is $55,000.

Screenings materials from the new fine screens located in the bypass channel will be captured in new wheeled poly bins located directly behind the new screens. Two options are currently being investigated for disposal of the screenings captured in these bins. The options include separate disposal in the new grit building by including truck access to the room, or by making the pathway to the existing screenings bin fit for an operator to wheel the poly bins to that location for manual dumping into the much larger existing bin. The second option would permit the additional screenings to be disposed of without any additional trucking but would require more labor. These options will be investigated further and specific details presented for review and direction during the Functional Design phase.

7.2.5 Grit Removal

7.2.5.1 Function

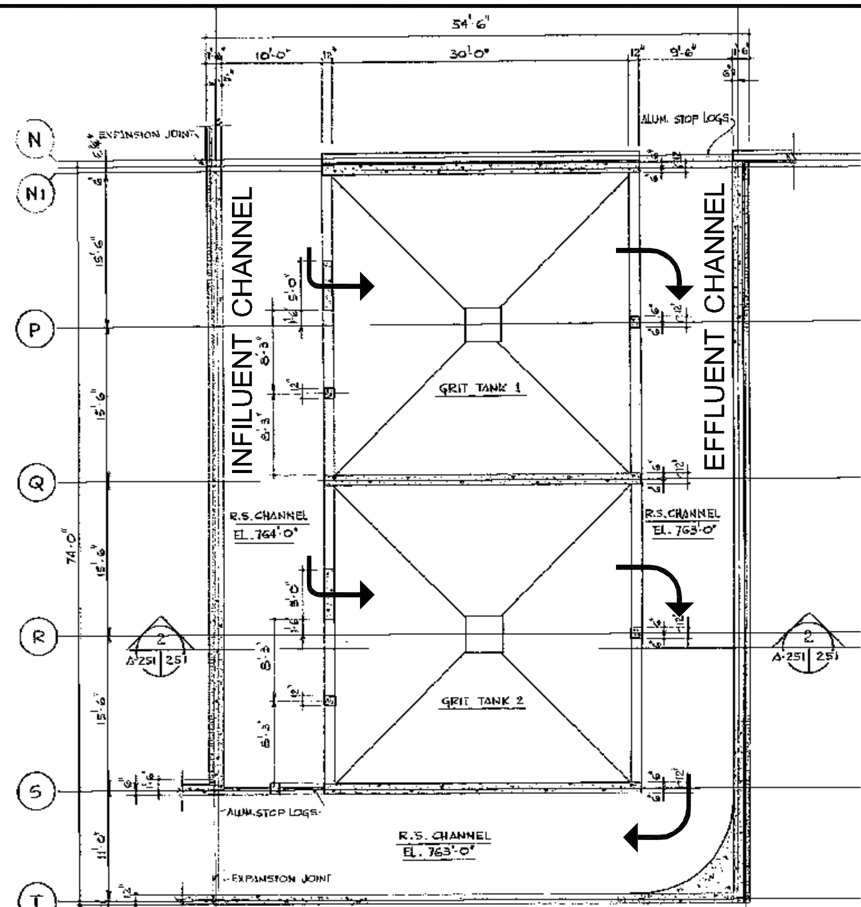
Grit removal is provided in advance of treatment units to prevent undue wear of machinery and unwanted accumulation of inorganic solids in channels, settling tanks and digesters. Typically, grit removal is accomplished by providing aerated grit tanks, vortex grit tanks, detritus tanks and grit channels, which are used less frequently. Grit removal can also be accomplished using centrifugal type separators and stationary screens.

7.2.5.2 Existing Facility

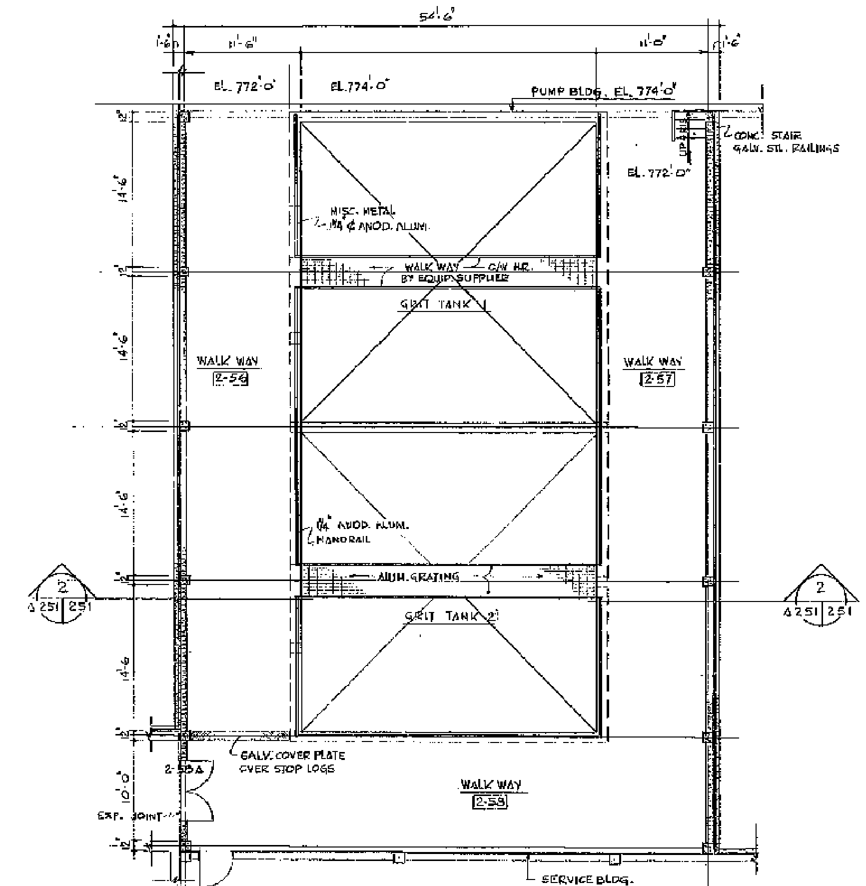
The existing grit removal system is comprised of 2 – 9.1 m square aerated grit tanks with a dedicated air blower, grit pump and classifier for each tank. Air blowers designed to supply air to headworks channels may also be used to supply the grit tanks for emergency conditions. The water level in the bar screen channels and grit tanks are controlled by a weir on the downstream end of the grit tanks. Figure 7.7 illustrates the current configuration of grit facilities.

Screened raw sewage flows through the inlet channel and into the aerated grit tank via a 915 mm x 1525 mm manually operated inlet sluice gate on each tank. There is also an overflow weir between the screened wastewater channel and the grit tanks that permits high flows to bypass the sluice gates. A low pressure air supply is provided to each tank through a 150 mm air header from two blowers. The air is released through a central diffuser to aerate the sewage flowing into the grit tank. The air causes the dense solids and grit to settle to the bottom of the tank while the lighter organic solids remain in suspension. The degritted sewage flows over the downstream weir and discharges into a common channel where it is conveyed to the primary treatment facility. The inlet channels are equipped with sluice gates to permit bypassing of one or both of the grit tanks. Bypass flows can be conveyed to the existing plant bypass or the primary treatment facility.

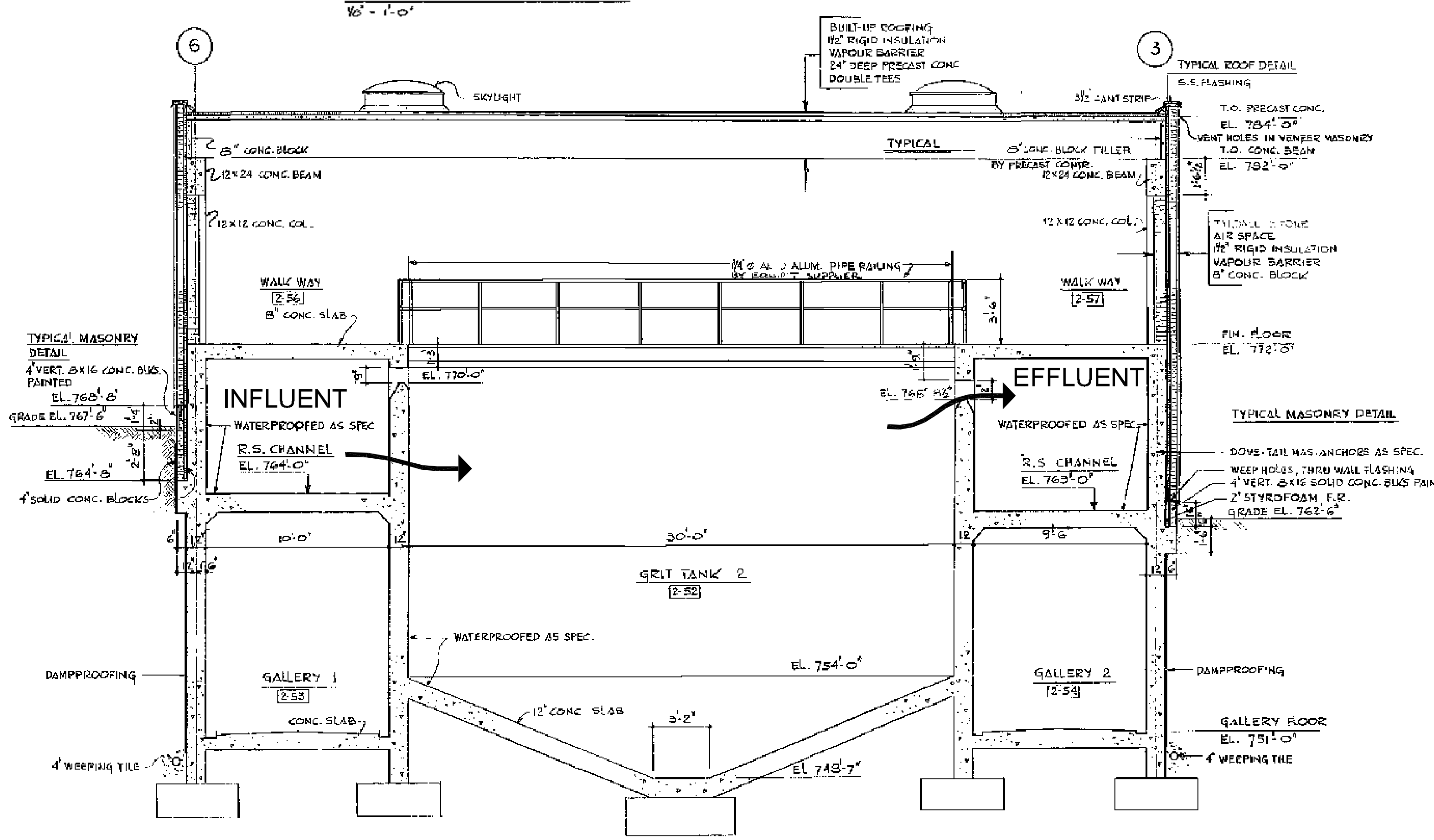
Grit is removed from the hopper in the bottom of the grit tank by two grit removal pumps. The grit slurry is pumped to two classifiers located in the Screen Room. The classifiers separate the



CHANNEL LEVEL
PLAN AT ELEVATION 763'-0"
1/16" = 1'-0"



TOP LEVEL
PLAN AT ELEVATION 772'-0"
1/16" = 1'-0"



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A-251 | 251 1/4" = 1'-0"

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THE CITY OF WINNIPEG
WATER AND WASTE DEPARTMENT

SOUTH END WATER POLLUTION CONTROL CENTRE

EXISTING AERATED GRIT CHAMBER CONFIGURATION

CITY DRAWING NUMBER: FIGURE 7.7

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grit from the liquid stream of the slurry. The liquid and any organics washed from the grit are discharged from the classifier and into the inlet channel downstream of the screens. Grit from the classifier is discharged into a conveyor in the Screen Room. The conveyor discharges grit and screenings to a disposal bin in the Truck Bay. The bin is periodically emptied and the contents are hauled to a landfill for disposal.

The aerated grit tanks are providing removal of grit from the waste stream as there is no evidence of solids carryover to the PSTs from the grit tanks. However, there are problems with the existing grit removal system as excessive quantities of grit are settling in the grit tank and not being removed by the grit pumps. Solids deposition over time in the grit tanks results in less efficient operation of these units. Continuous operation of the grit pumps while the unit is in service is recommended to minimize grit buildup and prevent blockage in grit pipelines. It is also recommended that each grit tank be taken down on an annual basis (after spring melt or in summer) for inspection and cleaning with a vacuum truck.

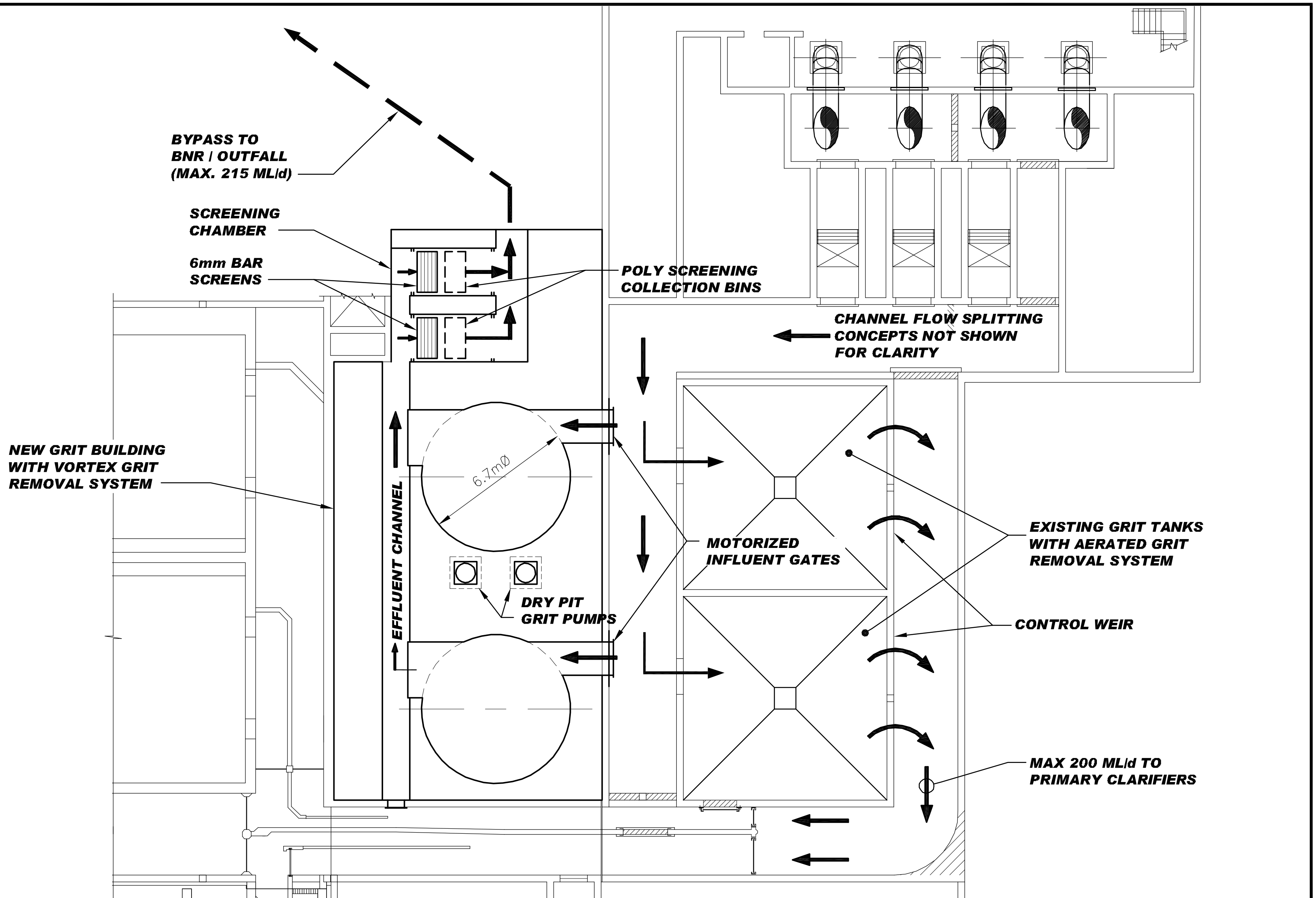
The SEWPCC Functional Design Report (MacLaren Engineers Inc., September 1989) states that the existing aerated grit tank has an effective volume of 752 m^3 and the capacity should be based on a hydraulic retention time (HRT) of 3 to 5 minutes. Based on these figures, the total peak hydraulic capacity of the existing grit removal system would range from approximately 220 ML/d (at 5 minute HRT) to 360 ML/d (at a 3 minute HRT). Earth Tech reported the hydraulic capacity of the existing grit system at 264 ML/d (providing an HRT near 4 minutes). A review of the existing drawings roughly confirms the effective volume and the HRT times are industry standards so the statements in the Functional Design Report seem reasonable. In order to optimize grit removal efficiency, it is recommended that each grit tank operate at a flow range not to exceed 100 ML/d (total for two tanks = 200 ML/d) to allow for a 5.4 minute HRT. One point of note is that standard HRT rates are based on the settling rate of sand particles and not the specific grit characteristics of the SEWPCC influent wastewater.

7.2.5.3 Grit System Upgrade Options

Two options are being considered for the grit system upgrade. Figures 7.8 and 7.9 illustrate conceptual layouts of the two options. With both options, consideration will be given to seasonal variations in flow and flexibility in routing effluent. New grit removal facilities intended to handle high flows bypassing primary treatment are best located to the west of the existing grit tanks. This arrangement simplifies directing excess flow to the new units and ultimately to a new high flow effluent channel/pipeline bypassing the PSTs. Locating the new equipment to the west of the existing grit tanks will simplify diversion of flow above the maximum 200 ML/d to be directed to primary clarification.

<u>Option 1</u>

Option 1 involves utilizing the existing grit tanks and adding additional screening/grit removal facilities to provide the additional capacity to meet the 415 ML/d peak flow requirement.



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DESIGNED BY S.A.B.	CHECKED BY S.A.B.		SOUTH END WATER POLLUTION CONTROL CENTRE
DRAWN BY K.R.	APPROVED BY S.A.B.		PLAN VIEW OF GRIT SYSTEM LAYOUT OPTION I
HOR. SCALE: VERTICAL:	RELEASED FOR CONSTRUCTION:	TENDER NO.	CITY DRAWING NUMBER FIG. 7.8 — SHEET OF
DATE 2008.10.07	DATE		

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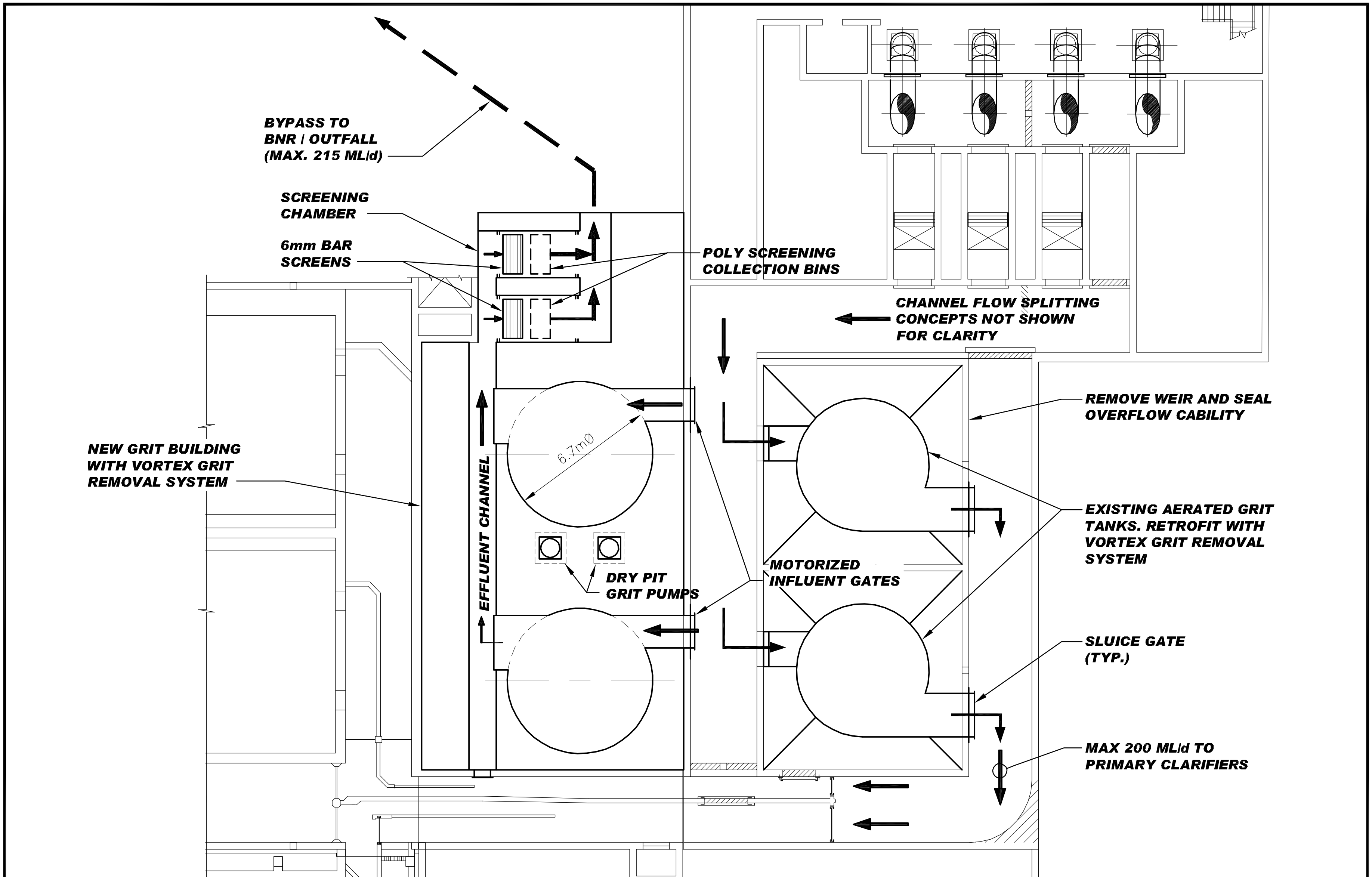

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SOUTH END WATER POLLUTION CONTROL CENTRE

PLAN VIEW OF GRIT SYSTEM LAYOUT
OPTION 2

CITY DRAWING NUMBER

FIG. 7.9

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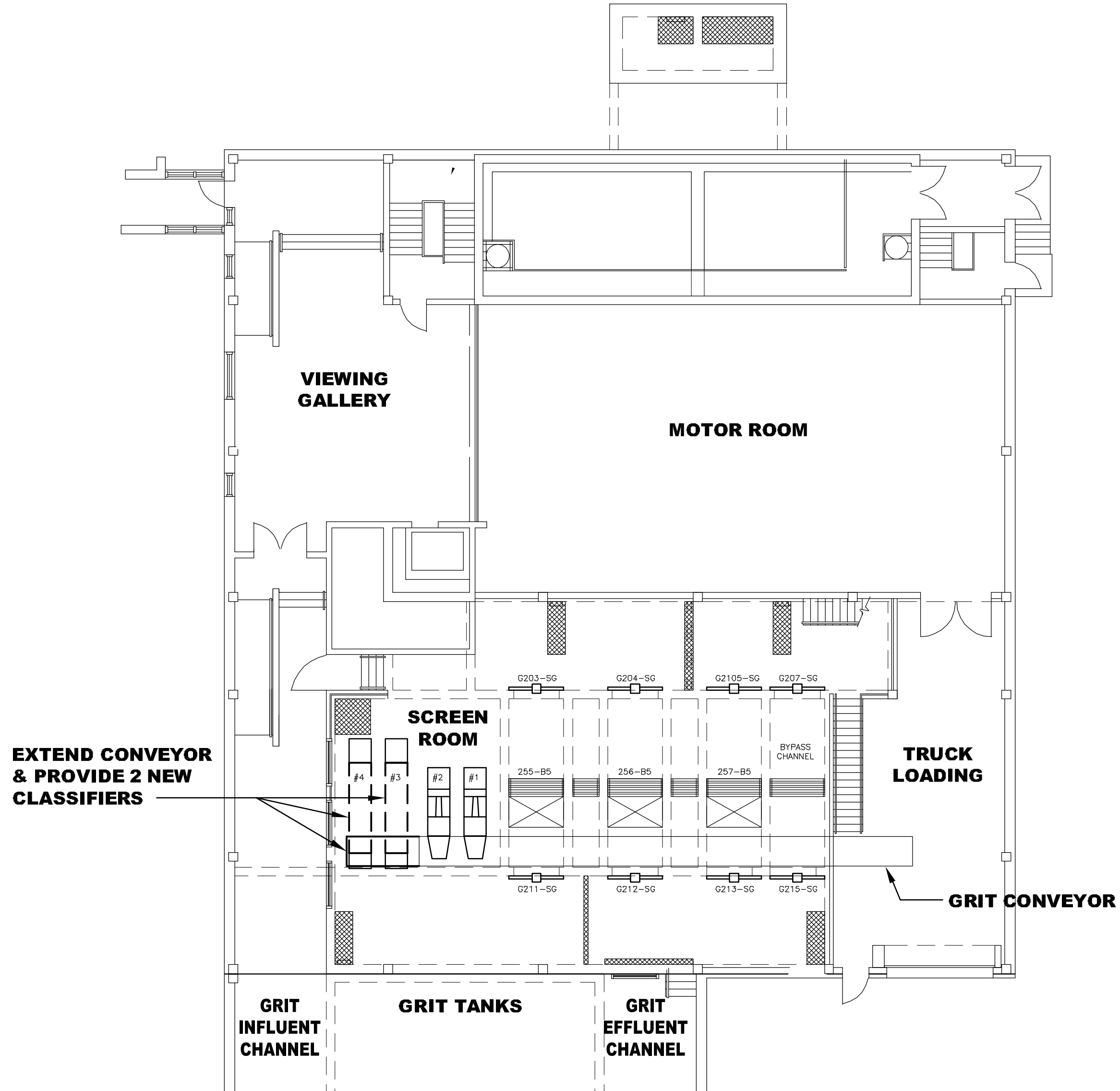
Automated gates will be utilized to divert flow to new units when influent flow to the plant exceeds 200 ML/d.

The additional grit removal facilities would be concrete below-ground vortex grit tanks. Systems from Eutek and Hydro International were used as the baseline in this evaluation. Hydro International's Grit King with Swirl-Cleanse combines screening and grit removal in a self-contained, self-cleaning unit with a low head loss, which is critical to the feasibility of second stage screening. There is no flocculation associated with this system. Other potential screening options evaluated include the 4mm Hydro-Jet screen or the Heliscreen, a powered unit with lower head loss. Both the Swirl-Cleanse and Hydro-Jet screens require a screenings siphon with a free discharge, which cannot be fit into the existing hydraulic profile to divert flow to the bioreactors. Installation of a Heliscreen unit appears to be feasible. Final selection of screen and grit removal components will be accomplished during detailed design. The supplier selected during detailed design will be required to perform a grit characterization to confirm system adequacy. Additional information on these units is provided as Appendix G.

In this option, the coarsely screened wastewater enters and exits the vortex grit tanks tangentially. Grit is removed from a hopper at the bottom of the tank by a centrifugal pump located in an adjacent dry pit and is discharged to a grit classifier. The grit tank may be equipped with a compressed air or water supply to provide a scour effect as the first step in the grit removal cycle. The fluidized grit in the bottom of the hopper is pumped via a centrifugal pump in an adjacent pit to a dewatering classifier. The grit classifier is comprised of a settling chamber with an inclined screw and a spray wash system. Organics are washed from the grit and excess water drains from the grit as it moves up a ramp for discharge into a grit conveyor. The classifiers will be located adjacent to the existing grit classifiers in the existing screenings room as shown on Figure 7.10 and will deposit dewatered grit on a conveyor to the screenings/grit bin. We considered the use of compactors for the new grit system but they are likely unnecessary due to the low volume of grit generated.

A cyclone will also be provided with the classifier to improve the separation of grit and water from the pumped grit slurry. The grit system may be packaged together with associated pumping and grit dewatering classifiers. Refer to Appendix H for product information on grit separation.

The possibility of diverting the primary flow through the new vortex grit tanks was examined. This was determined to be problematic. Since bypass flows > 200 ML/d and less than 300 ML/day are directed to the bioreactors, high efficiency grit removal is required on these flows to protect the bioreactor from grit accumulation and screen blinding. Therefore the vortex grit equipment is better used on the bypass flow and not the primary flows (<200 ML/d).



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DRAWN BY K.R.	APPROVED BY S.A.B.		SCREENINGS ROOM PROPOSED LAYOUT
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DATE 2008.06.20	DATE	TENDER NO.	CITY DRAWING NUMBER FIG. 7.10 — SHEET OF

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Option 2

Option 2 involves retrofitting the existing grit basins with higher efficiency vortex type grit removal units. Under this option, the total hydraulic capacity of the new grit removal system would be 415 ML/d under peak flow conditions. The number of units and available capacity in the existing area is dependent on the grit gradation to be determined during detailed design. Multiple units are recommended to allow the turndown required for low flow rates anticipated in winter months. The existing grit classifiers could be re-used while new grit pumping would be required. This option has several potential advantages, as newer units are more efficient and compact and minimize the need for additional structures. Newer units will also remove a higher percentage of the grit and allow organics to pass through to the BNR system, enhancing process efficiency.

The two new vortex grit removal units retrofitted into the existing tanks would treat flows < 200 ML/d. Gates could be added to the grit channels allowing the larger single unit to be used during low flows if required.

Additional facilities to accommodate peak flows will be enclosed in an expansion to the west of the existing Headworks Building as with option 1. Because blowers would no longer be required, there would be some operational savings. The disadvantage to this option is the initial capital cost, which would likely be in excess of $1 million more than the first option to accomplish retrofits in the existing basins. In addition, flow splitting to PSTs and to the Bioreactors and the river becomes more complex in this option and is therefore not recommended.

7.2.5.4 Grit System Recommendations and Proposed Facility

Based on this evaluation and discussions with plant staff, Option 1 is recommended with flow diversion to limit loading of the two existing units for enhanced removal efficiency. Selection of this option maximizes use of the City's previous investments and saves over $1 million in costs for new equipment and retrofit construction based on preliminary vendor quotations.

To facilitate flow splitting between the PSTs and bypass piping, the capacity of the existing grit chambers will be established at 200 ML/d (100 ML/d per unit). This flow rate results in an HRT of approximately 5.4 minutes at peak flow which will allow the existing units to operate effectively throughout the anticipated range of flows expected. Partitioning of the channels should also be considered to provide scouring velocities at low flows. This is discussed in greater detail in Section 7.2.5. During low flow periods, one (existing) unit will provide adequate grit removal performance and will provide higher velocities in the upstream channels to minimize grit accumulation.

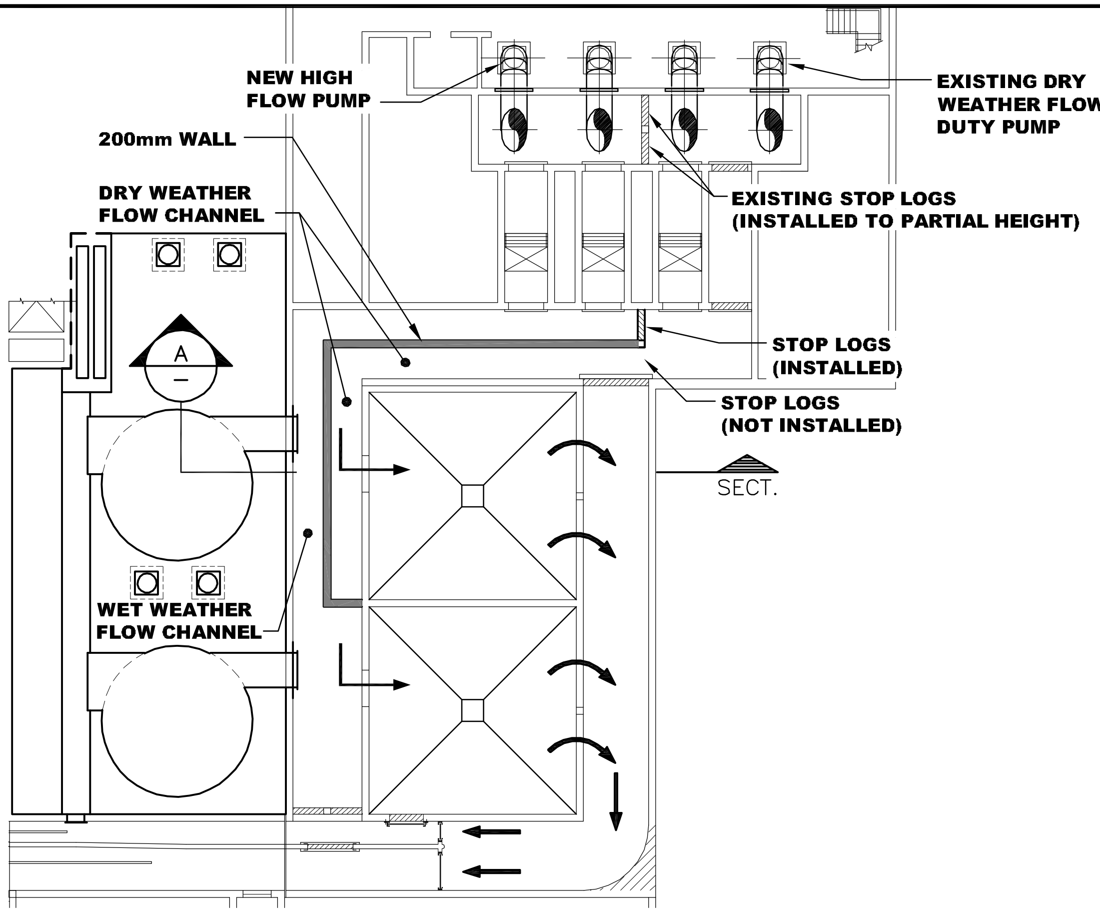
For the grit expansion, two Storm King units are recommended to allow flexibility in operation for the required flow ranges. In order to optimize removal efficiency and sizing of the new unit(s), a gradation analysis should be performed on a grit sample from the headworks during detailed design. Motorized fine screens are recommended on the PST bypass line to provide additional debris and floatables control.

7.2.6 Headworks Channels

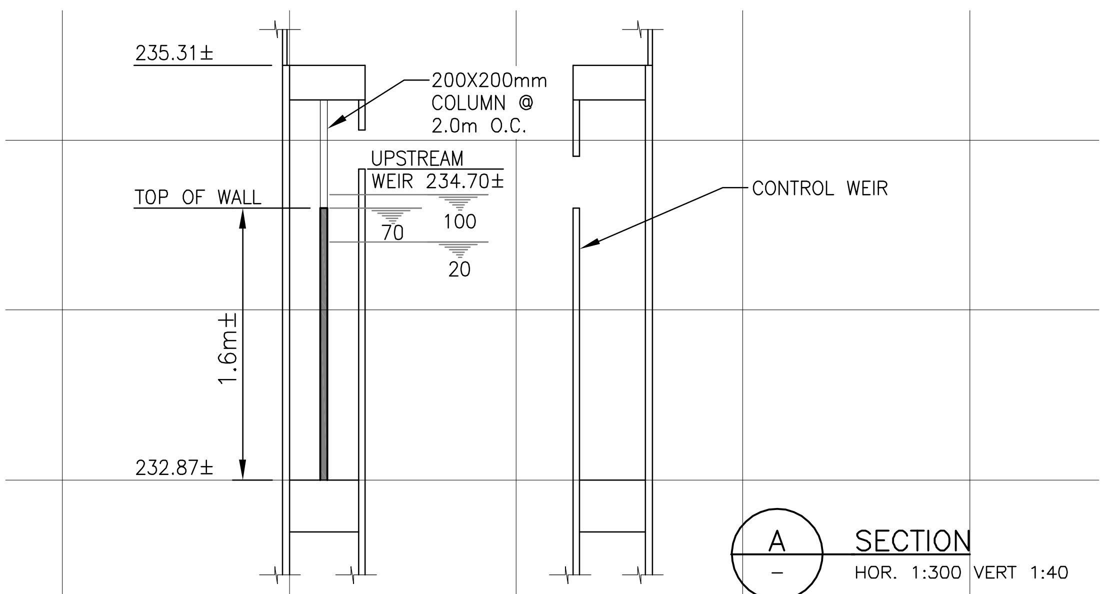
The preliminary treatment facility includes flow channels that convey sewage from the pump discharge, through the unit processes of preliminary treatment and eventually, to the primary treatment facility. Accumulation of grit in the channels is an ongoing maintenance concern and affects the channel capacity, which is a concern at high flows. The existing channels between the pump discharge and inlet to the aerated grit tanks were originally fitted with an aeration system to prevent sedimentation of grit and other inorganic materials in the channels. This piping was subsequently removed due to plugging and other maintenance problems.

The first area of concern is around the screens. Low water velocities in the channels in front of and behind the screens result in grit accumulation immediately upstream and downstream. The screens themselves remain relatively clean due to increased velocity through the screens themselves but accumulation nearby has been known to slough towards the screens causing damage. The solution to this grit accumulation problem is to increase the velocities in these channels to prevent grit deposition. There is existing infrastructure to permit installation of stoplogs in the channels leading up to the screens. Stoplogs can be installed that would direct flow from one small and one large pump through one screen at low flows. The remaining two large pumps would flow through the remaining two screens during high flows. This configuration would increase channel velocities and reduce grit deposition. The stoplogs would only be partially installed to permit overflowing under emergency high flow situations.

The geometry of the channel downstream of the screens does not allow a simple solution to prevent grit buildup. Based on preliminary model runs, velocities in the existing 3 m wide bar screen and grit channels ranges from 0.052 m/s at minimum flow (20 ML/d), to 0.176 m/s at average flow (70 ML/d). Desirable velocities at minimum flow and average flow are 0.30 m/s and 0.60 m/s respectively. Placing a concrete partition in the channel will increase velocities at lower flow rates and reduce grit buildup but could also make cleaning the channels more difficult and may create additional odor issues. A conceptual sketch is presented as Figure 7.11. At a 70 ML/d flow rate (approximate peak daily flow during dry conditions), the velocity in the partitioned channel increases from 0.20 m/s to 0.35 m/s, greatly reducing the potential for deposition in the channels. Periodic cycling of the larger pumps to produce scouring velocities is also recommended to minimize buildup of grit in the channels. This cycling could occur on a daily or weekly basis during higher influent flow conditions. At 100 ML/d, velocities approach 0.50 m/s in the partitioned channel. Provision of water and/or air jetting was considered but should not be necessary if pump cycling and channel partitioning is used to increase velocities and annual inspections/cleaning are performed. Filleting is also recommended at 90º bends and dead-end channels where grit deposition normally occurs.



PARTITIONED CHANNEL CONCEPT PLAN



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Winnipeg

THE CITY OF WINNIPEG
WATER AND WASTE DEPARTMENT

SOUTH END WATER POLLUTION CONTROL CENTRE

HEADWORKS CHANNEL CONCEPTS

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7.2.7 Odor Control

The HVAC system for the expanded preliminary treatment facility would be designed to collect foul air from the facility and convey it via foul air ducts to odor control facilities. The processes used to treat odors will be discussed in more detail in a future technical memorandum.

7.2.8 Constructability

Ease of constructability and maintenance of plant operations have been considered through the preliminary design process. With the exception of the final connections between the new and existing channels, the preliminary treatment facility improvements can be constructed while maintaining operation of the existing plant. Consideration must also be given to maintaining operator access through the preliminary treatment facility to downstream processes.

With respect to the connection between the new and existing grit facilities, most of the new construction can be accomplished with minimal impact to operation of existing facilities. A new cut-in to the existing grit tank influent channel will be made after new facilities are in place. A passive overflow weir or modulating gate is envisioned to facilitate flow control to existing and new facilities. Stop logs may be utilized at the appropriate locations to permit work to proceed under 'dry' conditions should work be necessary in existing channels. Excavation for the new grit facilities adjacent to existing structures must be considered in selecting the size and number of new units. Since most of the high efficiency grit removal units have a diameter to depth ratio of about 1:1, multiple units may actually prove be a more cost-effective option.

7.2.9 Electrical Requirements

The electrical requirements for this new equipment are not significant. We do not have specific motor sizes at this time but they are generally small (in the 10 hp range) and are not significant in the overall plant power usage. Most of the equipment such as the vortex grit equipment and screens are supplied with control panels that include the motor starters and their own PLC control units. Discussions regarding power and control of vendor supplied packages are covered in Section 17 and 18.

An allowance has been carried for the cost of this work as it will be required if the recommended pump replacement is undertaken.