# RM Clayton Water Reclamation Center - Hydraulic Modeling in InfoWorks ICM

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**ABSTRACT**
The City of Atlanta is developing hydraulic models of their Water Reclamation Centers (WRCs), which can be combined with their existing collection system model, to ultimately have a single dynamic model to assess hydraulic capacities and performance of the overall wastewater system. As part of this task, the BGR Joint Venture of Black & Veatch, and Gresham, Smith and Partners, and Rohadfox constructed and calibrated a new model of the 103 mgd RM Clayton WRC using InfoWorks ICM.

This paper summarizes the implementation and calibration of the hydraulic model for the RM Clayton WRC and highlights the challenges and solutions for accurately representing treatment plants using ICM.

**KEYWORDS**
Hydraulic modeling, InfoWorks, ICM, wastewater, calibration.

## INTRODUCTION

The City of Atlanta has developed hydraulic models of their Water Reclamation Centers (WRCs) with the intent to combine with their existing collection system model to have a single dynamic model to assess hydraulic capacities and performance of the overall wastewater system. As part of this task, the BGR Joint Venture of Black & Veatch, and Gresham, Smith and Partners, and Rohadfox constructed a new model of the 103 mgd RM Clayton WRC using InfoWorks ICM. The model was then calibrated and used to determine the maximum hydraulic capacity of each process area of the WRC.

InfoWorks ICM (Integrated Catchment Modeling) has been developed by Innovyze to incorporate both urban and river catchments. This paper will set out how, by careful selection of model components and coefficients, the software has been adapted to enable accurate hydraulic modeling and calibration of a treatment plant.

### Model Overview

The hydraulic model for RM Clayton WRC extends from the influent collection structure to the Chattahoochee River, including all the liquid stream process units. The model was constructed based on record drawings, a site survey, an existing Excel spreadsheet hydraulic model, and feedback from plant operations staff.

The process units included in the model were:

- influent collection structure, headworks with bar screens, vortex grit collectors, multi-tray grit removal units, and drum screens,
- (8) primary clarifiers,
- (2) biological nutrient removal (BNR) process basins,
- mixed liquor pumping station,
- (10) secondary clarifiers,
- (22) effluent filters,
- ultraviolet (UV) disinfection system,
- and outfall diffuser.

All process units were included, along with 6 bypasses. The Return Activated Sludge (RAS) system was also included in a simplified form since RAS is a significant proportion of the plant total flow.

Figure 1 shows the location of these process units and Figure 2 shows a screenshot of the overall model. The model was based on the geographical layout although with some model elements spread out to make model interrogation easier.


**Figure 1. RM Clayton Main Process Units**

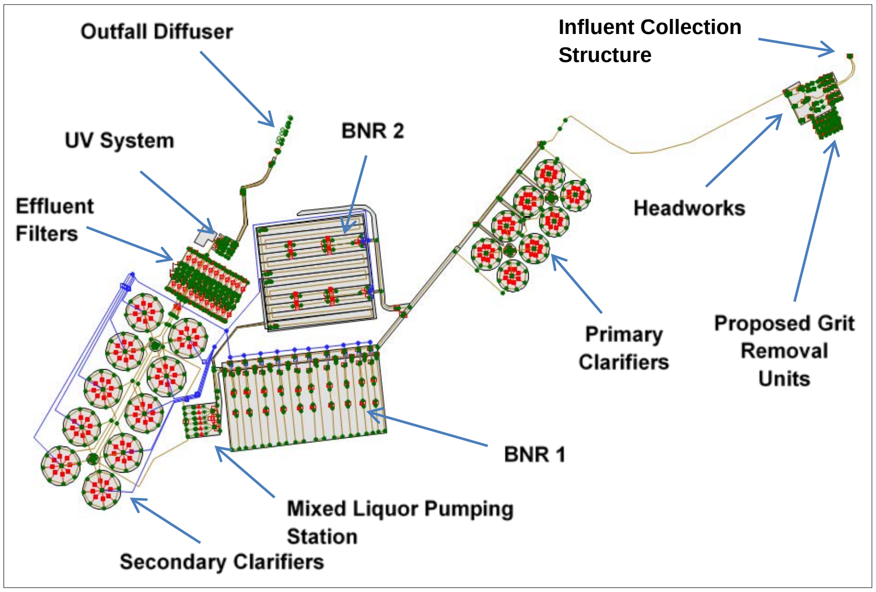



**Figure 2. RM Clayton Hydraulic Model Screenshot**

## METHODOLOGY

### Model Setup

The model was constructed using Innovyze's InfoWorks ICM version 5.5.5. Nine scenarios were created to represent different calibration periods and hydraulic capacity cases, and simulations were performed using inflow files specifying the incoming flow to the plant and level files specifying the Chattahoochee River level at the effluent.

The model was configured to use the Manning's equation to determine the headloss due to friction in all conduits with an '*n*' value of 0.013 used throughout. Sediments were included where this was observed during site survey.

Fixed headloss coefficients were used throughout the model with headloss K values for bends and other fittings based on BGR's recommended design values or determined from separate detailed spreadsheet calculations.

ICM requires a node between every component, for example where a conduit size changes. In order to minimize the effect on routing and time lag through the plant, node areas were set at 1ft$^2$ where chambers are not present in reality.

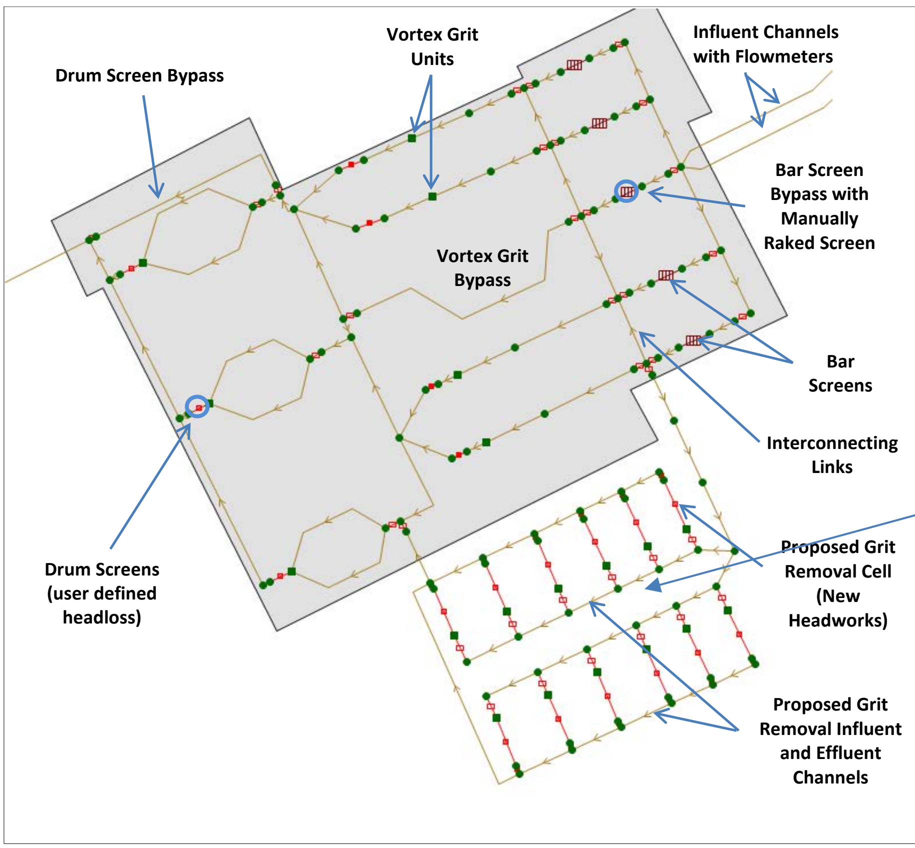
### Main Process Unit Modeling

Influent Collection Structure and Headworks: The upstream end of the model is the influent collection structure which receives flow from the 96" Peachtree Creek trunk sewer, 90" Peachtree relief sewer, 48" Proctor Creek Sewer and Nancy Creek Tunnel Pump Station. Flow then passes to the headworks through two channels containing the plant's influent flowmeters. Figure 3 shows a screenshot of the headworks area of the model, with the key items labeled.

The plant includes four mechanically raked bar screens which were modeled using the built-in

ICM screen component. The screen width entered was the effective width, i.e. the sum of the openings between the bars, subtracting an estimated screen blockage of 30%.

Downstream of the bar screens there are four vortex grit collectors. Based on manufacturer's information, the headloss within the units is less than ¼ inch at the design flow (103 mgd), so these were modeled as storage nodes without any headloss. The headloss associated with flow entering and leaving the grit collectors was included in the headloss coefficients of the adjacent conduits. The 6 ft. wide weirs on the effluent channel from each grit collector were included with higher than typical discharge coefficients as these weirs are always submerged. The standard method used by ICM over-estimates the headloss of drowned weirs and although Villemonte's equation for drowned sharp-crested weirs can be selected in the model simulation parameters, this can cause instability and still over-estimates the headloss of drowned broad crested weirs. The weir coefficients were set manually to obtain the same headloss as determined using P.A. Kolkman's equations in Miller (1994). The same issue also occurs where sluices are operating as submerged weirs, with upstream water level below the top of the gate opening, and the sluice secondary discharge coefficients were manually increased.




During the implementation of the model the headworks was being expanded. This structure was included in the model for future use. The modeling of the new headworks is beyond the scope of this paper.

**Figure 3. Headworks Model Screenshot**

The drum screens were modeled based on curves provided by the drum screen supplier, Bracket Green. It is not possible in ICM to define headloss as a function of depth, therefore five different headloss versus discharge tables were included within the model and the most appropriate table selected based on the closest water level downstream of the drum screens.

Gates were included upstream and downstream of each process unit to enable easy isolation of any stream. In most cases the gates are full channel width and with openings greater than the water depth, and high discharge coefficients were used so they provided negligible headloss in the model. The bypasses to the bar screens, vortex grit collectors, and drum screens were included in the model.

The new multi-tray grit removal system was modeled based on the conformed drawings available during this study but was included in the model for future use. Gates were included on the upstream and downstream ends of the proposed system and closed for simulations of the existing plant. Each multi-tray grit removal unit was modeled using an inlet sluice, manhole with area equivalent to area of unit, user defined headloss and effluent weir. The user defined headloss versus discharge table was based on information provided by the manufacturer, Hydro International, for their Eutek HeadCell®.

**Primary Clarifiers:** Downstream of the headworks, flow passes to the primary clarifiers via a 12 ft. diameter tunnel. Figure 4 shows a screenshot of the primary clarifier area of the model.

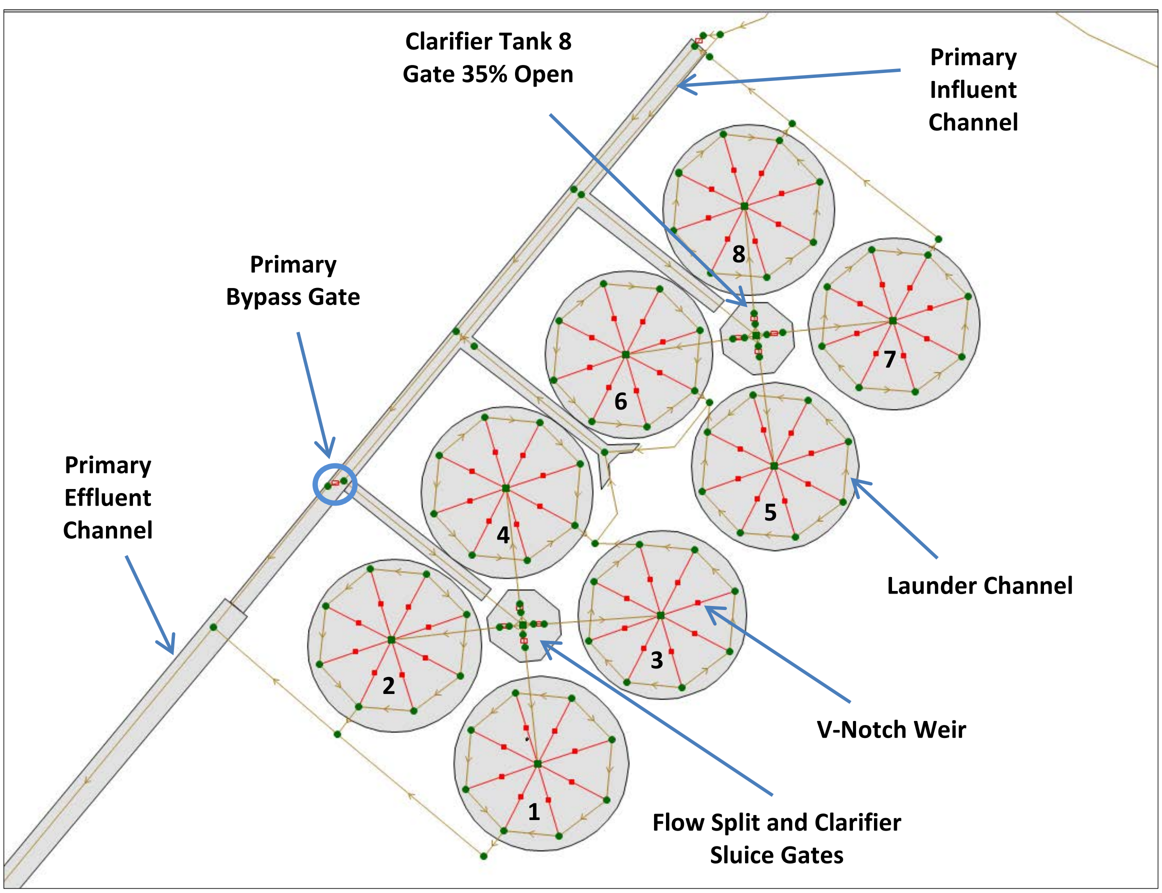
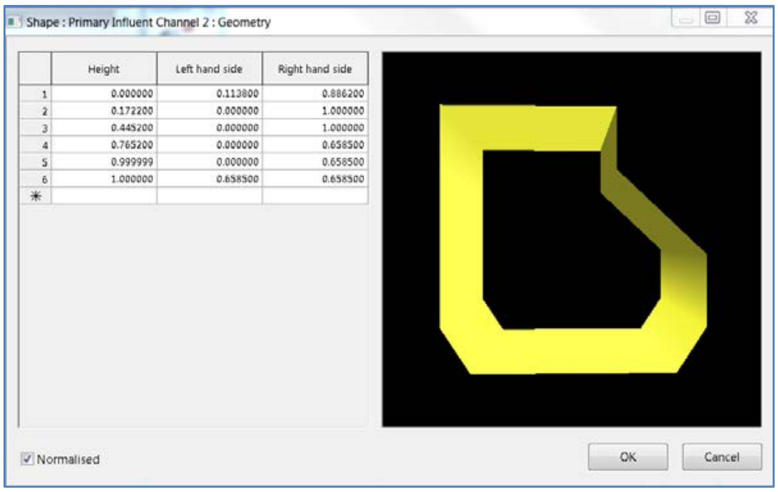



**Figure 4. Primary Clarifiers Screenshot**

The flow split between the primary clarifiers is based on hydraulic similarity of inlet pipework and the clarifier weirs. Based on feedback from operations staff, the gate to clarifier 8 was set 35% open to prevent excessive solids load to this clarifier. The model indicates a good flow distribution between the clarifiers with each set of four clarifiers receiving 50% of flow at 122 mgd (average day, maximum month flow). The model also indicates an identical flow split between clarifiers 1 to 4 due to the identical inlet pipework. With the clarifier 8 gate 35% open, it predicts that this clarifier would receive 7% of flow, with clarifiers 5 to 7 each receiving 14% of 122 mgd. However, ICM does not take into account local 3 dimensional or momentum effects.

The primary clarifier influent channels downstream of the tunnel have a non-standard cross section and were entered into ICM as user-defined shapes as shown in Figure 5. The model celerity ratio was increased to 14.414 to reduce the Preissmann slot to 1% of conduit width and reduce the impact of the slot on surcharged conduits.




ICM cannot simulate flow entering along the length of a channel directly from another model link, so to minimize inaccuracy, the launder channels on each primary clarifier were split into eight sections.

**Figure 5. Primary Clarifiers User Defined Shape**

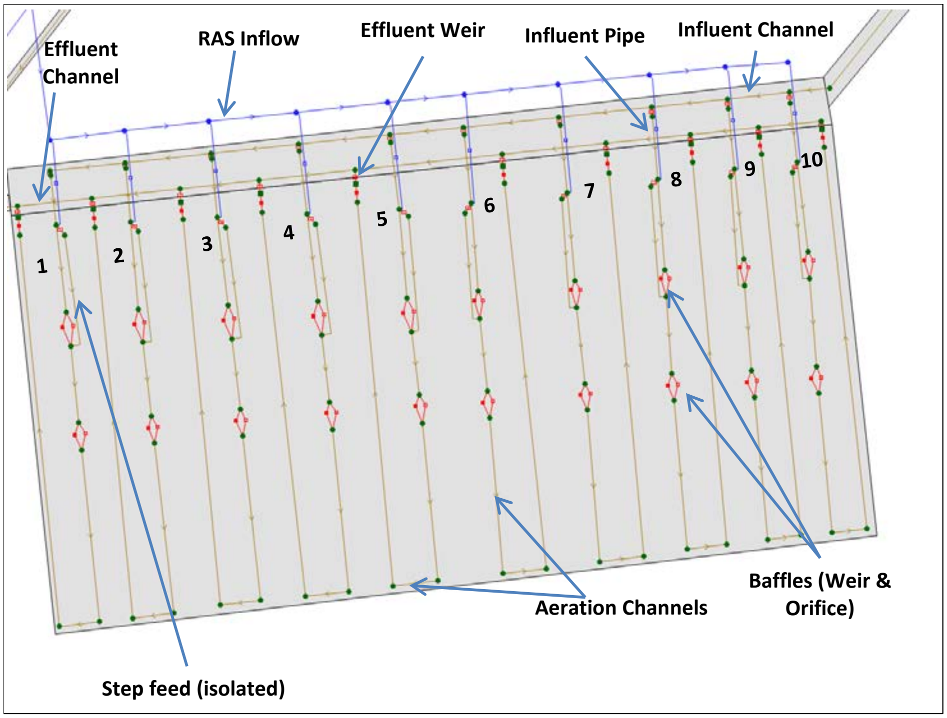
This required the V notch weirs on each clarifier to also be split into eight sections. It should be noted that even with this refinement, ICM cannot account for the energy required to accelerate incoming flow to a channel, so detailed spreadsheet backwater calculation was performed when determining the plant hydraulic capacity.

The primary clarifier bypass was included within the model. A proportion of the flow bypasses the clarifiers when the water level in the clarifier influent channel becomes too high to prevent hydraulic overloading of the clarifiers and site flooding. Automatic control of this gate was replicated using the Real Time Control (RTC) function within ICM, with the gate controlled to target a water depth of 9.5 ft. in the clarifier influent channel.

**Biological Nutrient Removal (BNR) Basins:** RM Clayton includes two BNR basins, with the flow distribution between the basins reliant on hydraulic similarity. Although the BNR trains both have similar effluent weir levels, the total weir lengths and tank inlet arrangements are not the same. Operations staff advised that BNR1 receives more flow than BNR2 and this was confirmed by the ICM model which showed 55% of flow to BNR1 at 122mgd with all trains in service.

There are gates present on the route to each BNR at the channel wye branch which could be used to amend the flow distribution but these are currently kept fully open. The gates are narrower than the adjacent channels and the discharge coefficient for these gates was amended to match the headloss predicted by Idel'chik (2003) for a thick edged orifice.

The model arrangement for BNR1 is shown in Figure 6 with key items labeled. Modeling of the RAS which enters each of the BNR trains is described later in this paper.




BNR1 trains 1 to 6 have 48" inlet pipes whereas trains 7 to 10 have 42" inlet pipes. The headloss along the influent channel is very low, so this results in a slight flow bias to trains 1 to 6.

**Figure 6. BNR1 Screenshot**

Individual trains are isolated using butterfly valves at the upstream end of each inlet pipe. ICM does not include a butterfly valve component and these were represented in the model by variable discharge orifices. The orifice discharge coefficient was set at 2.6 during calibration, which is equivalent to a headloss coefficient, K value of 0.3. It is important to note that the orifice discharge coefficients required in ICM are √2 greater than typical coefficients due to a difference in the equation used. Using variable discharge orifices enabled the limiting discharge to be set to zero using RTC to close individual trains and replicate changes which occurred during the calibration period.

The model arrangement for BNR2 is shown in Figure 7. Both BNR basins include step feed configurations which allow operations staff to divert flow around the first half of the mixing zone of each train. These facilities are not currently used but were included within the model to give that flexibility.

Both BNRs also include baffle walls on each train where flow can pass over a weir or through an opening at the bottom of the weir. These arrangements were modeled with a weir and orifice in parallel and the model automatically determines the flow split between the two. ICM uses dimensionless weir coefficients and these were all adjusted from the default value of 1.0 to 0.57 and 0.544 for thin plate and broad crested weirs respectively.

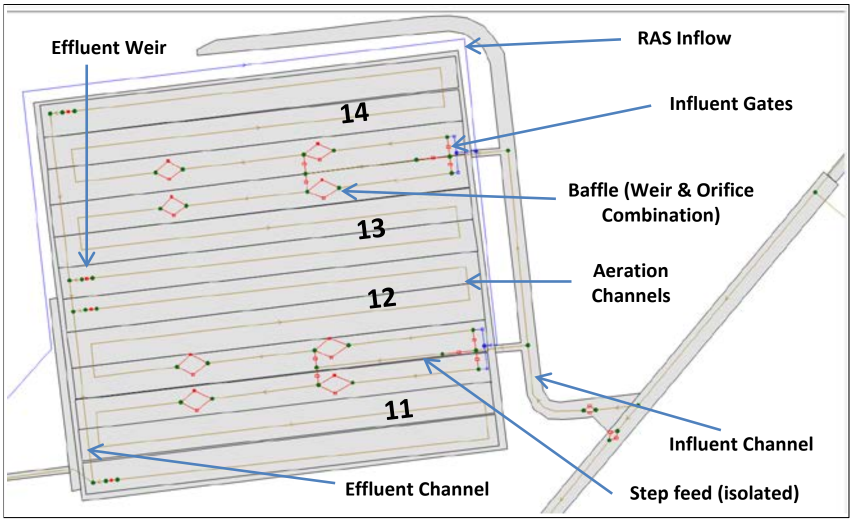



**Figure 7. BNR2 Screenshot**

**Mixed Liquor Pumping Station:** The model arrangement for the mixed liquor pumping station is shown in Figure 8. Flow from both BNRs passes through twin mixed liquor channels to the mixed liquor pumping station. The pumping station comprises six variable speed pumps split across two wet wells, which were modeled as Variable Frequency Drive Pumps (VFDPMPs). Pump speed is automatically controlled in response to the wet well water level with an additional pump brought online if the pump speed is high and a pump stopped if the speed drops. However, in order to provide a simple means of pump control within the model, which would automatically adjust to a wide range of incoming flows, the number of operating pumps was fixed with all pumps controlled to the same speed. Incremental controllers were used in the RTC file to incrementally increase or decrease the pump speeds in response to the water level in wet well.

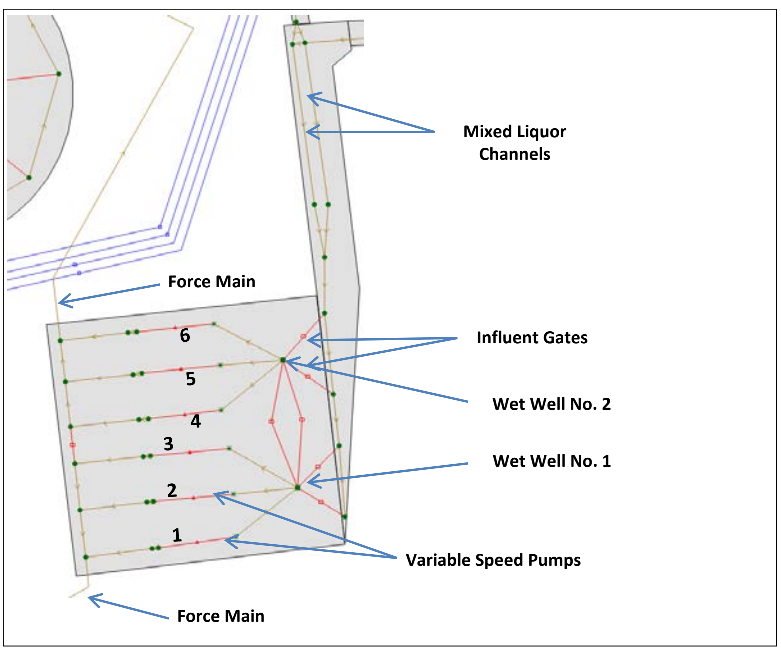
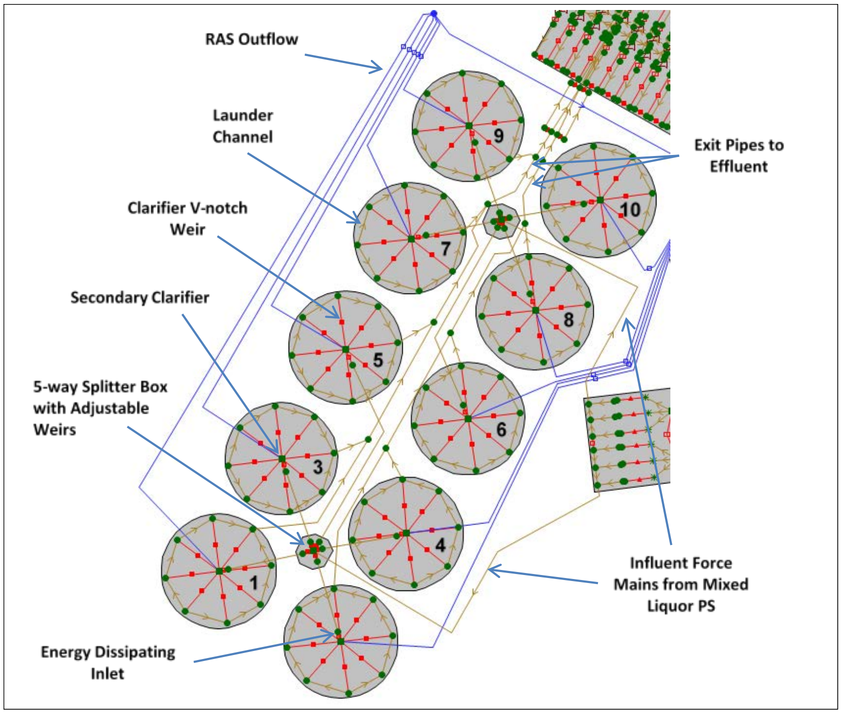



**Figure 8. Mixed Liquor Pumping Station Screenshot**

Slightly different pump curves were used for each pump using measured curves obtained from Operations Staff. Headlosses in the pump suction pipes, discharge manifold and along the length of the force mains were included, with the force mains modeled as pressure conduits.

**Secondary Clarifiers:** Flow is distributed between the ten secondary clarifiers via two splitter chambers. Each splitter chamber has five weir slide gates – one for each secondary clarifier. These gates are adjusted by plant operators in order to manage sludge blanket formation and were therefore modeled as variable crest weirs "VCWEIR", so that their level can be easily amended and clarifiers isolated if required.




Following the same approach as the primary clarifiers, the secondary clarifier weirs and launder channels were split into eight sections on each tank. The model arrangement for the secondary clarifiers is shown in Figure 9. The energy dissipating inlet within each secondary clarifier was modeled using a headloss versus discharge table, based on information from WesTech.

**Figure 9. Secondary Clarifiers Screenshot**

**Effluent Filters:** The model arrangement for the effluent filters is shown in Figure 10. At the upstream end of the filters is an overflow chamber with overflow weirs on each side. Each weir was split into two sections to improve model accuracy in this region. At high water levels flow automatically bypasses the effluent filters, passing through two overflow conduits which join the filter outlet conduits.

Flow distribution between the effluent filters is achieved using a cutthroat flume on each filter inlet and an almost perfect flow distribution was estimated. The flumes were modeled using the in-built ICM flume component which is based on the British Standard (BS) flume. BS flumes have a rounded inlet rather than the tapered inlet provided but the effect on headloss is small. ICM uses a nominal headloss if the flume becomes submerged (downstream depth / upstream depth > 75%) but it was checked that this does not occur at RM Clayton even at high flows.

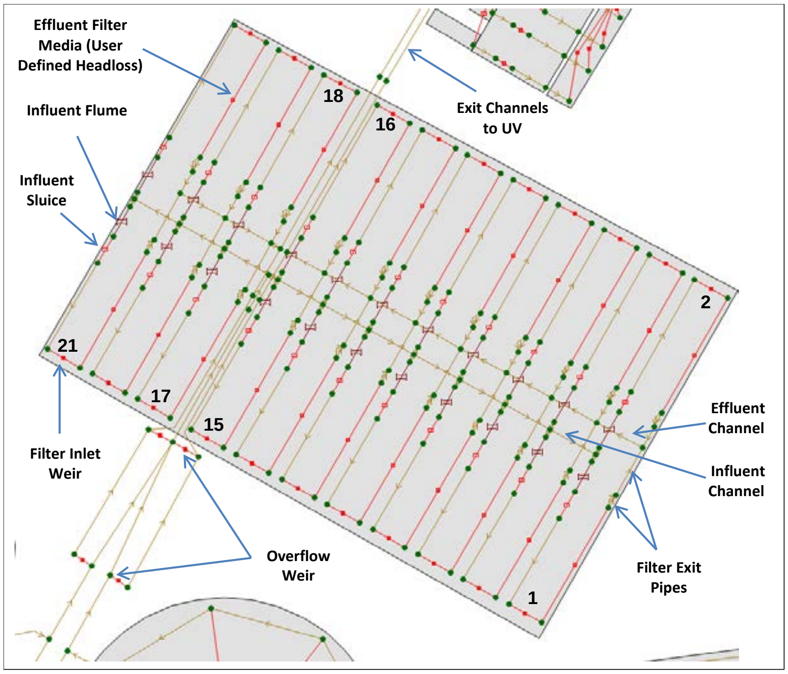



**Figure 10. Effluent Filters Screenshot**

Each of the 22 effluent filters has two cells which are fed from a single inlet and connected by small holes in the dividing wall. The two cells were combined in the model to simplify the arrangement. All filters are normally in service, with the filters backwashed sequentially, with one backwash every 3 hours. To ensure the model was conservative, the base model setup included one of the downstream filters offline. The flume link for the offline filter was removed to enable automatic model initialization.

The headloss through the media in each effluent filter varies with flow rate, but also with the amount of time since the media was last backwashed. A clean media headloss versus flow curve was obtained from the media supplier, De Nora, and was included in the model as a user defined head discharge table. De Nora also confirmed that the terminal headloss for filters with this hydraulic loading rate was about 2 ft. To be conservative, a second user defined headloss table including this constant headloss was used for all filters when assessing the maximum hydraulic capacity of the plant.

**UV System:** The UV system consists of two influent boxes, a long weir feeding the UV common influent channel, and five UV channels. Flow from each channel flows over effluent weirs into the UV Clearwell before passing to the outfall. The model arrangement for the UV system is shown in Figure 11.

The headloss associated with the UV system lamps was modeled using a head versus discharge table based on information obtained from the manufacturer, Trojan Technologies. The fifth UV channel was included in the model but without any lamps in the base scenario and the UV bypass channel was also included.

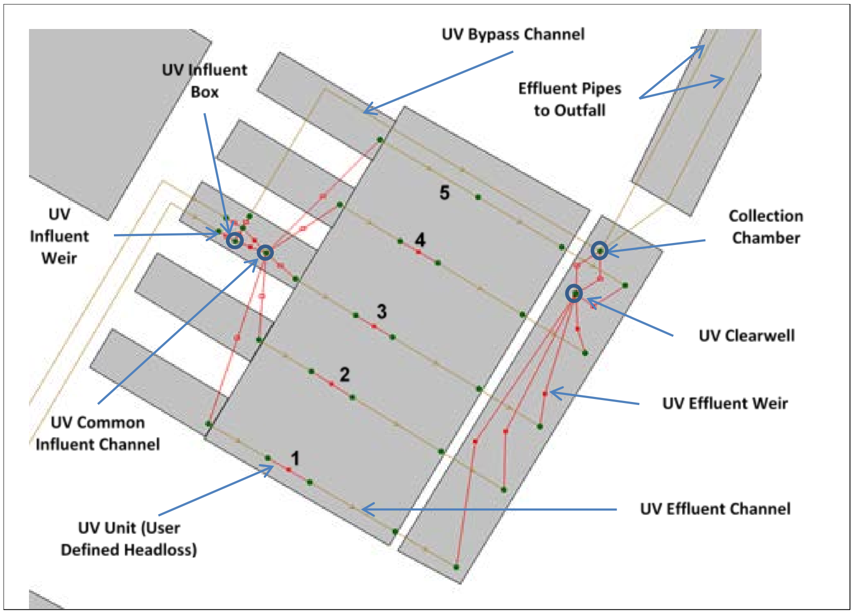



**Figure 11. UV System Screenshot**

**Outfall:** The model arrangement for the outfall is shown in Figure 12 with key items labeled. Flow from the UV system passes through two parallel venturi flowmeters which record the plant effluent flow. These were modeled with user defined head discharge table based on manufacturer's data.

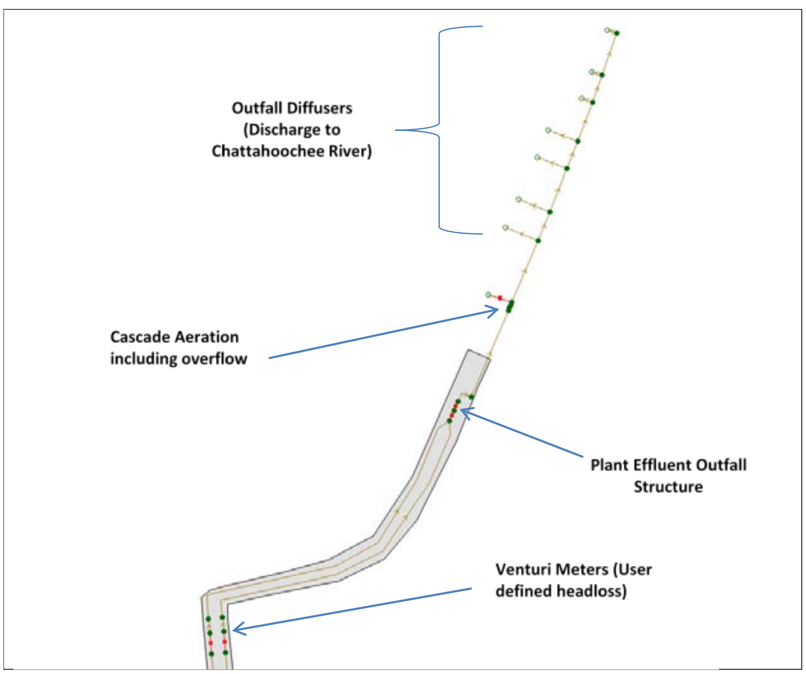



**Figure 12. Outfall Screenshot**

Flow then passes down a number of steps (modeled as weirs) in the plant effluent outfall structure before joining the original plant outfall. Next to the river, flow passes down a cascade aeration structure and into the effluent diffuser.

This diffuser was modeled in detail with each diffuser port connected to an outfall representing the Chattahoochee River. The river level was set via a separate level file so simulations could easily be performed for different river conditions.

**Return Activated Sludge (RAS):** Biowin process modeling for RM Clayton showed the RAS rate is typically around 50% of the incoming flow and therefore considerably increases flows and headlosses between the BNRs where the RAS is returned, and the secondary clarifiers where the RAS is removed. The RAS system was modeled in a simplified form using variable discharge orifices to enable the RAS flow to automatically adjust to changes in incoming flow, enable the proportion of RAS to be easily adjusted, and accommodate isolation of individual clarifiers or BNR trains. The model arrangement can be seen in Figure 13. Defining global variables allowed them to be used by all of the regulators and greatly simplified the RTC file.

In order to improve accuracy, measured RAS flows were used instead of fixed RAS proportion for calibration. This was achieved by setting the RAS proportion in the RTC file to zero and including RAS flows in the inflow file.

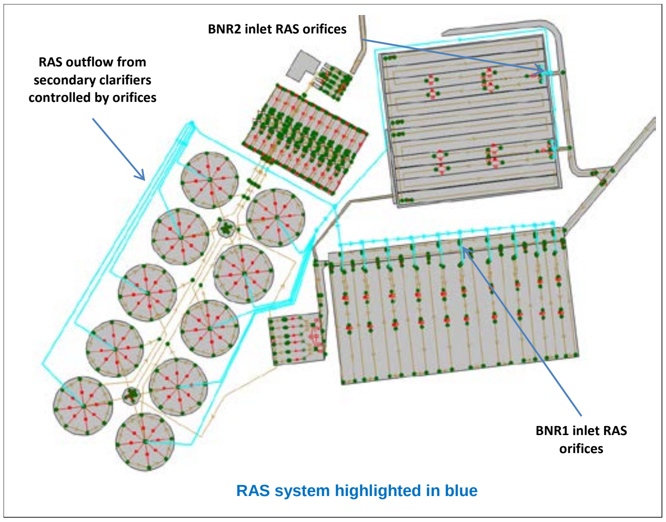



**Figure 13. RAS System**

## Calibration Data

Hydraulic calibration was completed to verify the model under actual flow scenarios experienced at RM Clayton. Flow data was obtained from the WRC SCADA output for the influent channel flow meters, the mixed liquor pumping station and the effluent venturi meters. Flow data was also obtained for the individual BNR1 RAS inflows and the total BNR2 RAS.

The existing system at RM Clayton only records water levels in the influent collection chamber and mixed liquor wet wells and analysis of the data showed these both to be unreliable. The large number of hydraulic breaks within RM Clayton meant that calibrating every section of the plant would have required more instruments than were available, but BGR determined the most important locations for calibration. Eight portable ultrasonic level meters were installed by BGR and the City's flow monitoring team to measure water depths over a four week period in 15-minute increments. During this period site operations staff logged the number of units in service across the plant.

Figure 14 shows the locations of the data used, with permanent locations shown in yellow and temporary monitor points shown in blue.

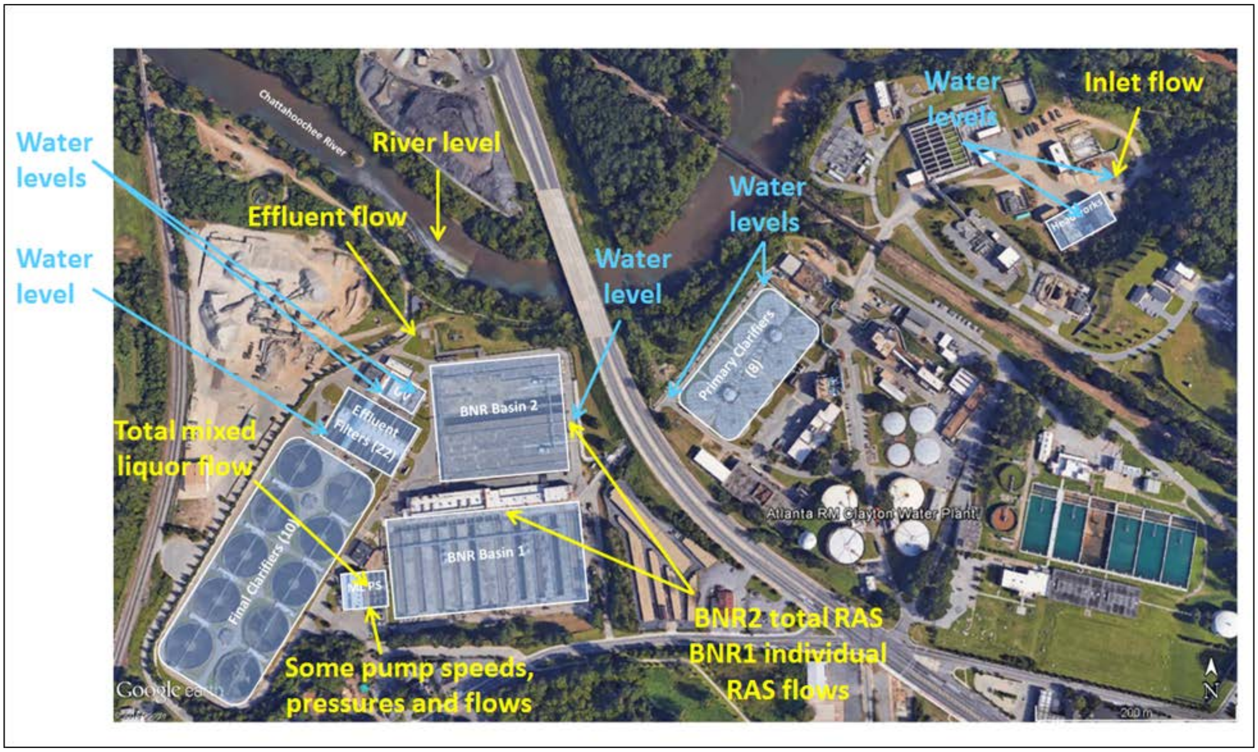



**Figure 14. Permanent and Temporary Instrumentation Used**

A detailed site survey was also completed to obtain dimensions which could not be obtained from record drawings, and to verify key hydraulic information, such as weir levels.

## RESULTS

### Flow Data Analysis

The first task undertaken was to analyze the site flow data. Figure 15 includes flows recorded between 20$^{th}$ February and 24$^{th}$ March 2017. The influent and effluent flowmeter data showed similar peaks although the effluent flow reading was noticeably more variable and generally lower than the influent. The highest plant inflow recorded during this period was 125mgd at 23:30 on 7$^{th}$ March, with the highest effluent flow of 122mgd recorded one hour earlier.

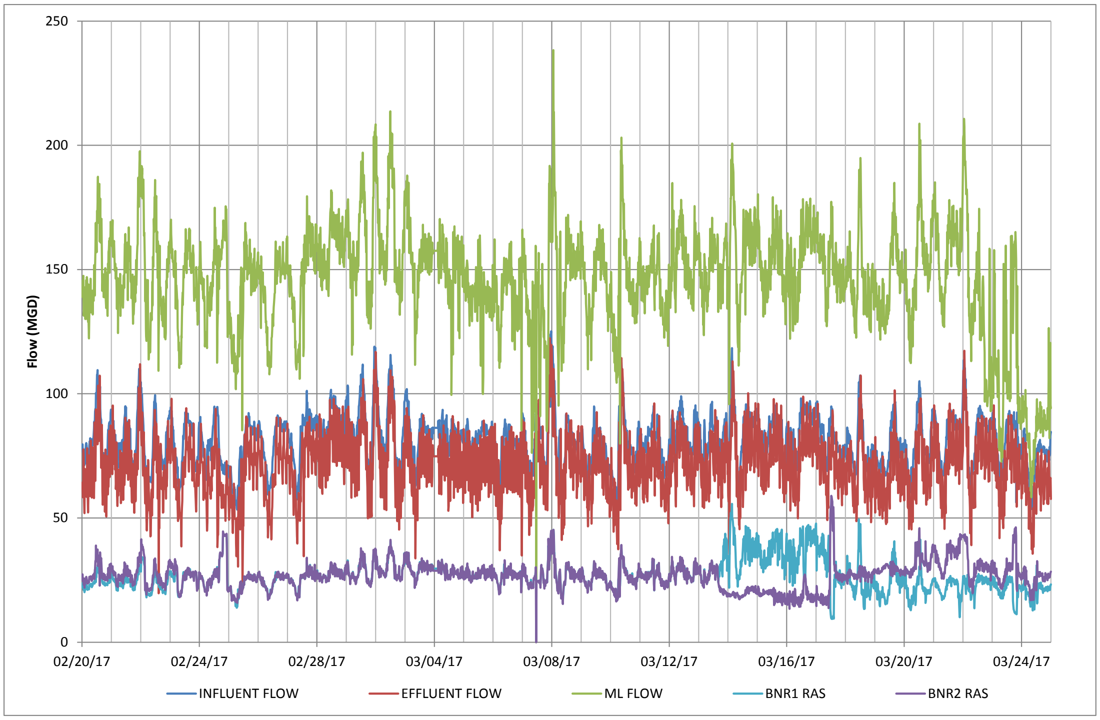



**Figure 15. Flow Data, 20th February to 24th March, 2017**

The effluent flow at the plant is calculated using two parallel venturi meters whereas the influent flow is calculated using two parallel flowmeters retrofitted in channels upstream of the headworks. The pattern of the influent flow matched very well with the measured water elevation in the headworks (see Figure 16) but the total measured volume of influent was 15% higher than the effluent over the four week calibration period.

The influent flow measurement is believed by operations staff to be affected by grit accumulation and therefore the influent flow used for calibration was reduced by 15%. Figure 17 shows the original and adjusted flows for one week during the calibration period. The effluent flow was still noticeably more variable and 'peaky' than the adjusted influent but this is due to operation of the mixed liquor pumping station.

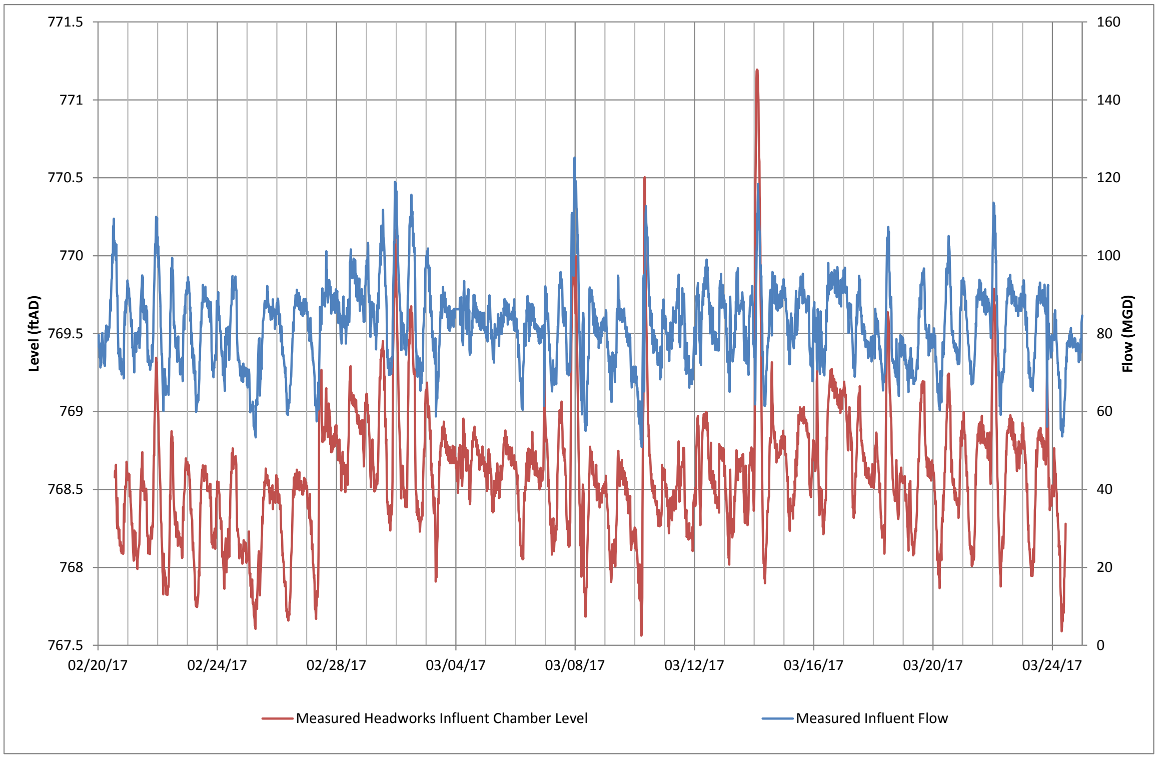



**Figure 16. Influent Flow Data and Headworks Water Level**

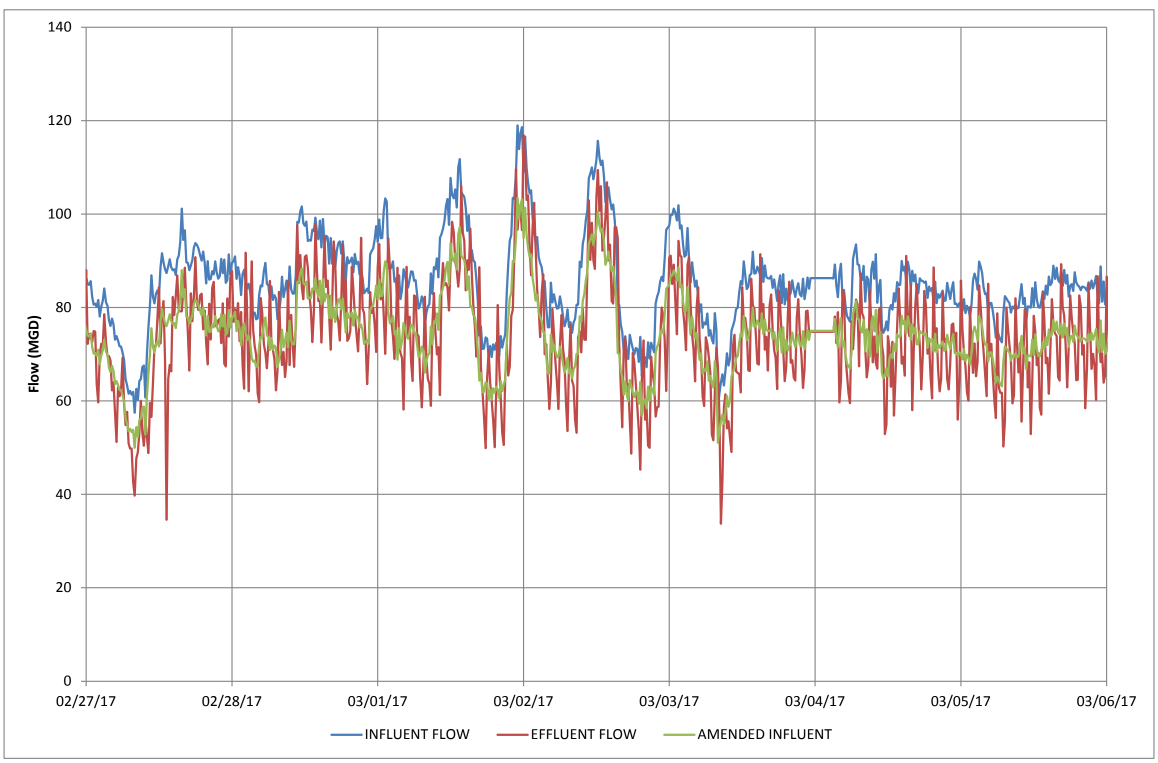



**Figure 17. Influent Flow Adjustment**

## Model Calibration

To verify the model, measured and simulated water levels were compared for four weeks of data, at eight locations across the plant, with the results discussed below.

**UV System to Outfall:** Level data was obtained for the UV Clearwell and the UV common influent channel. Three UV units were generally in service during the calibration period but channel 1 was brought on line for a week in the middle of the calibration period. This caused poor data to be obtained for the UV Clearwell at this time as this water level was measured adjacent to the channel 1 effluent weir.

Figure 18 and Table 1 show the measured and simulated water levels at the two UV locations. A very good calibration was achieved for both locations, and for both 3 and 4 UV units in operation. The measured data showed higher maximum levels and lower minimum levels than estimated by the modeling but this is likely to be due to operation of the mixed liquor pumps which results in more rapidly varying flow than simulated by the model.

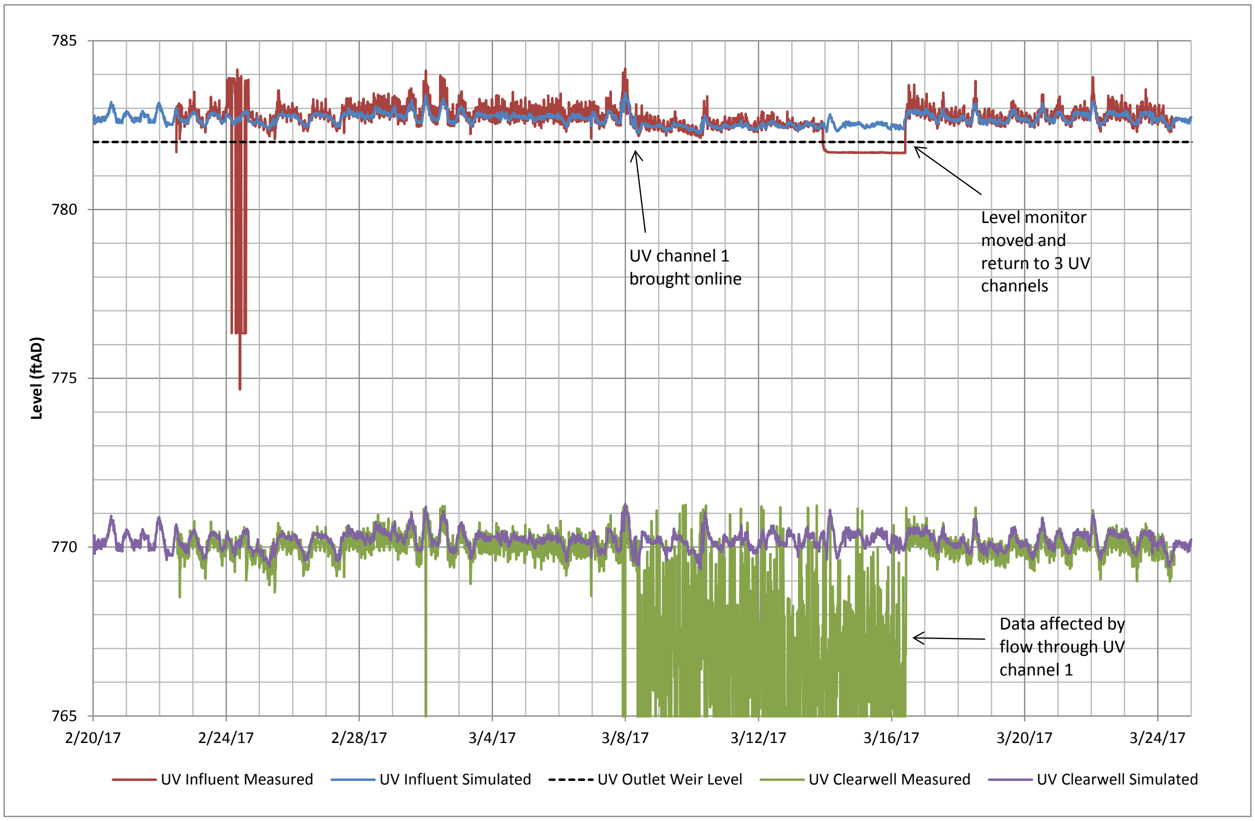



**Figure 18. UV Level Comparison**

**Table 1 UV Calibration Summary**

| Reading Location | Invert Level | Item | Flow Rate (mgd) | Measured Depth (ft) [3] | Simulated Depth (ft)[4] | Difference | |
|---|---|---|---|---|---|---|---|
| UV Common Influent Channel | 776.34ft[1] | Maximum | 108.9 | 7.84 | 7.12 | -0.72 ft | -9% |
| | | Average | 70.9 | 6.42 | 6.33 | -0.09 ft | -1% |
| | | Minimum | 55.7 | 5.75 | 6.15 | 0.40 ft | 7% |

| Reading Location | Invert Level | Item | Flow Rate (mgd) | Measured Depth (ft) [3] | Simulated Depth (ft)[4] | Difference | |
|---|---|---|---|---|---|---|---|
| UV Clearwell | 762ft[2] | Maximum | 100.6 | 9.22 | 9.06 | -0.16 ft | -2% |
| | | Average | 70.9 | 8.02 | 8.20 | 0.18 ft | 2% |
| | | Minimum | 55.7 | 6.66 | 7.72 | 1.06 ft | 16% |
| [1] From record drawings and confirmed by site survey<br>[2] From record drawings<br>[3] Excluding time periods of poor data<br>[4] Maximum and minimum determined for same event as measured depth | | | | | | | |

**Effluent Filters:** Level data was obtained for effluent filter 21. The water level in the filters is determined by the headloss between the filters and the UV inlet weir. This varies with both flow rate and the filter media headloss, and gradually increases between filter backwashes. The measured levels were therefore compared against two simulated levels, based on clean media and the terminal media headloss of 2ft.

Figure 19 shows that the model accurately represented the clean filter water level. The filter headloss was shown to increase rapidly between backwashes as the media has degraded and will be replaced soon, but the figure also shows that the maximum headloss of 2ft. matched well with observations and was suitable for determining the maximum hydraulic capacity of the plant.

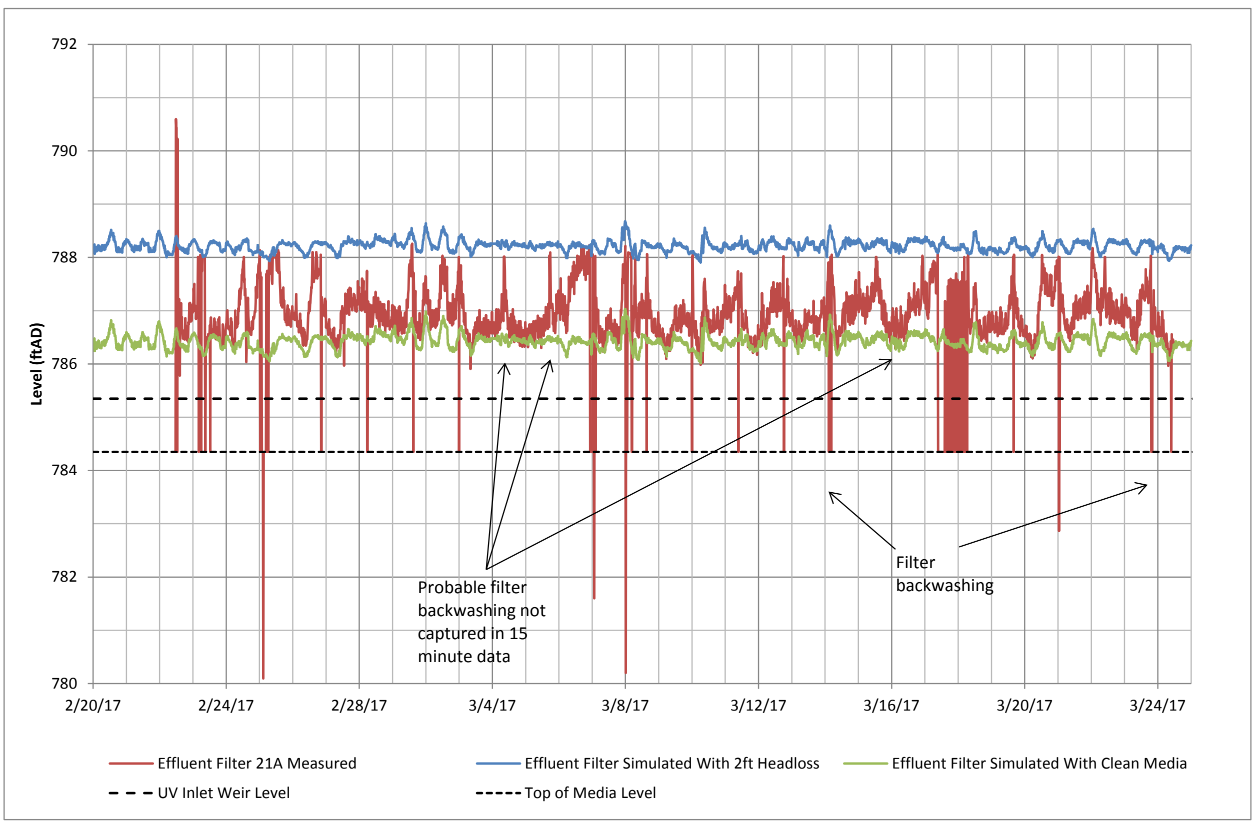



**Figure 19. Effluent Filter Level Comparison**

**Mixed Liquor Pumping Station:** Pump speed, flow, suction pressure and discharge pressures for the mixed liquor pumps are monitored on the City's SCADA system. The pump speed recorded showed a reliable pattern of variation but the pump flows and pressures varied in steps throughout the calibration period. The pump discharge pressures were combined with surveyed gauge elevations in order to give the discharge head in feet above datum. The measured discharge heads for both pumps 1 and 6 varied in steps and the head for pump 1 was below the elevation of the secondary clarifier splitter box weir levels. The simulated head was relatively close to the measured head for pump 6 but the data was deemed not reliable enough to calibrate this area of the model.

**Primary Clarifiers to BNRs:** Level data was obtained for the BNR2 influent channel and the primary clarifier effluent channel. A close match was obtained for the BNR2 influent channel with calibration within 1% based on water depth as shown in Figure 20.

In order to improve calibration in the primary clarifier effluent channel, the headloss coefficients for the BNR1 inlet butterfly valves were reduced from the BGR standard of 0.5/0.7 to a more typical in service value of 0.3.

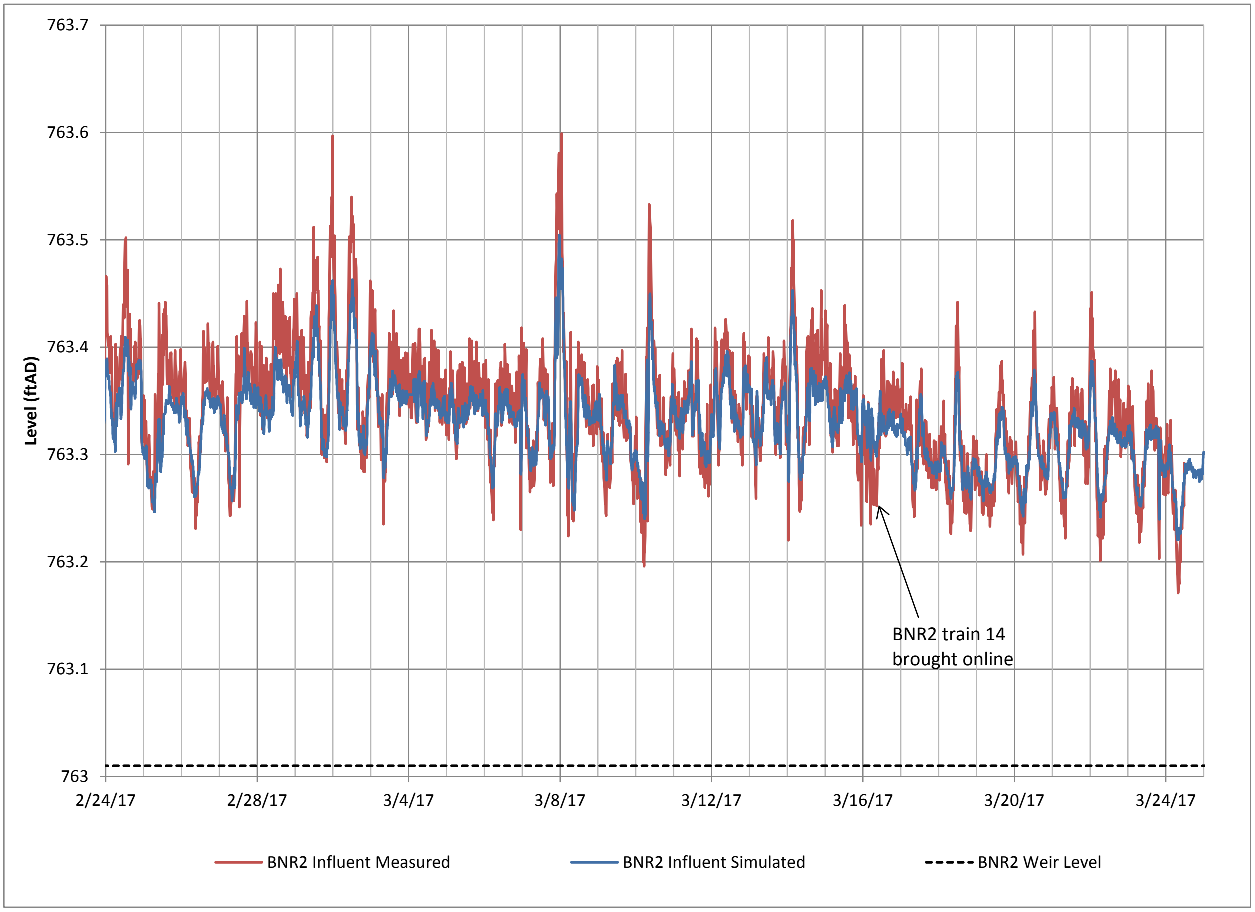



**Figure 20. BNR2 Influent Level Comparison**

**Primary Clarifier Influent:** Water depths were also measured at the upstream end of the primary clarifier influent channel. Initially there was a discrepancy between the measured and simulated levels but this was eliminated following clarification of the invert level at the monitor location and the addition of sediment to the model. A sediment depth of 0.3ft. was observed in the clarifier influent channel during the site survey and this was added to all the channels in this area. Figure 21 shows the measured and simulated water levels for the final model.

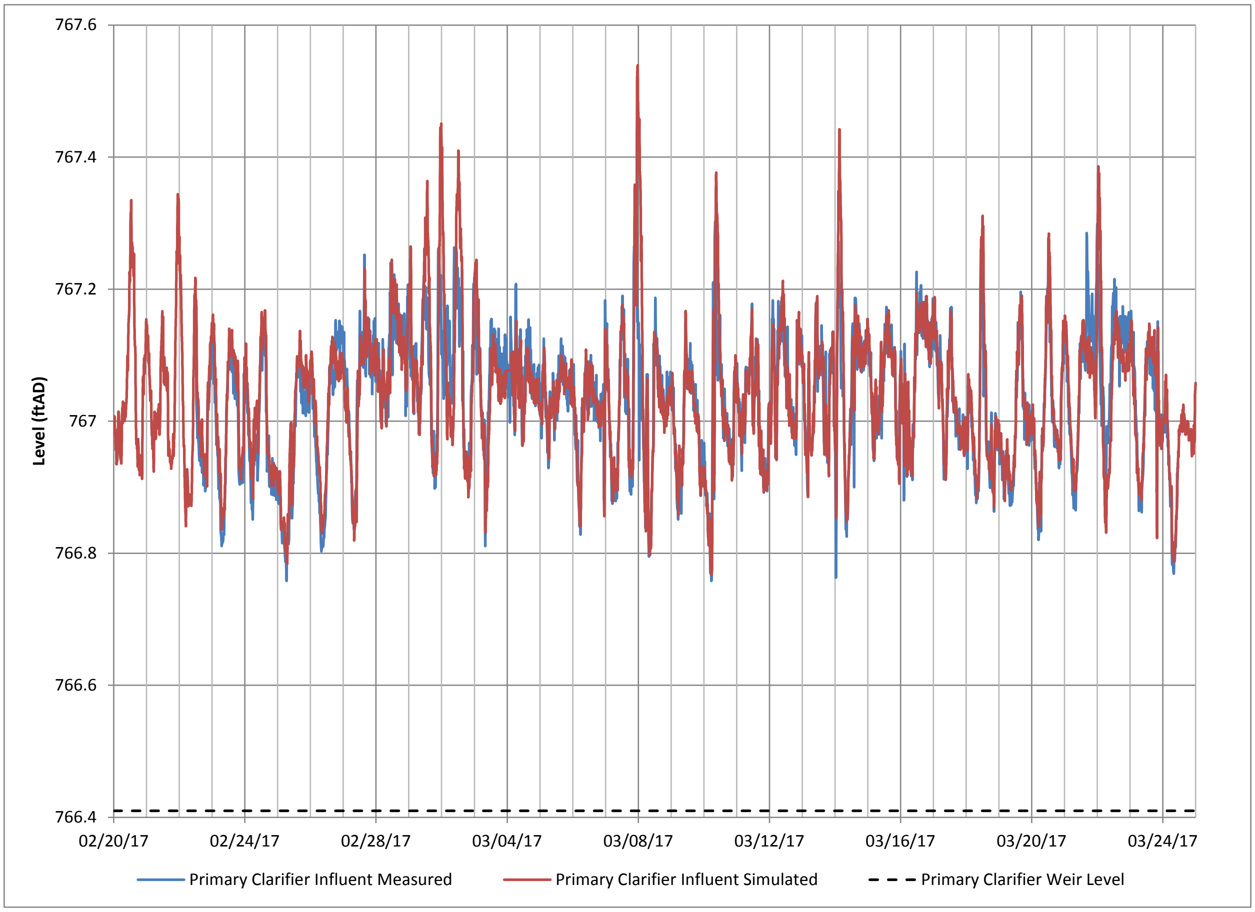



**Figure 21. Primary Clarifier Influent Level Comparison**

**Headworks:** Level data was obtained for the headworks influent collection structure and downstream of bar screen number 2. During the calibration period there were three bar screens online, two vortex grit collectors (three during high flow) and two drum screens. The time periods for the third grit collector being online was not known so three units were kept online in the model for the entire calibration period.

Figure 22 shows the water levels for the headworks. There was more variation in water levels in the headworks compared to other areas of the plant as the headlosses depend on screen blinding as well as flow rate and units in service. Fairly good calibration was achieved for the headworks influent chamber apart from two short duration events where it is likely that the drum screen headloss was higher than estimated. The simulated water elevation downstream of bar screen 2 was noticeably higher than measured throughout the calibration period but this was believed to be due to removal of sediment at this location during screen replacement work.

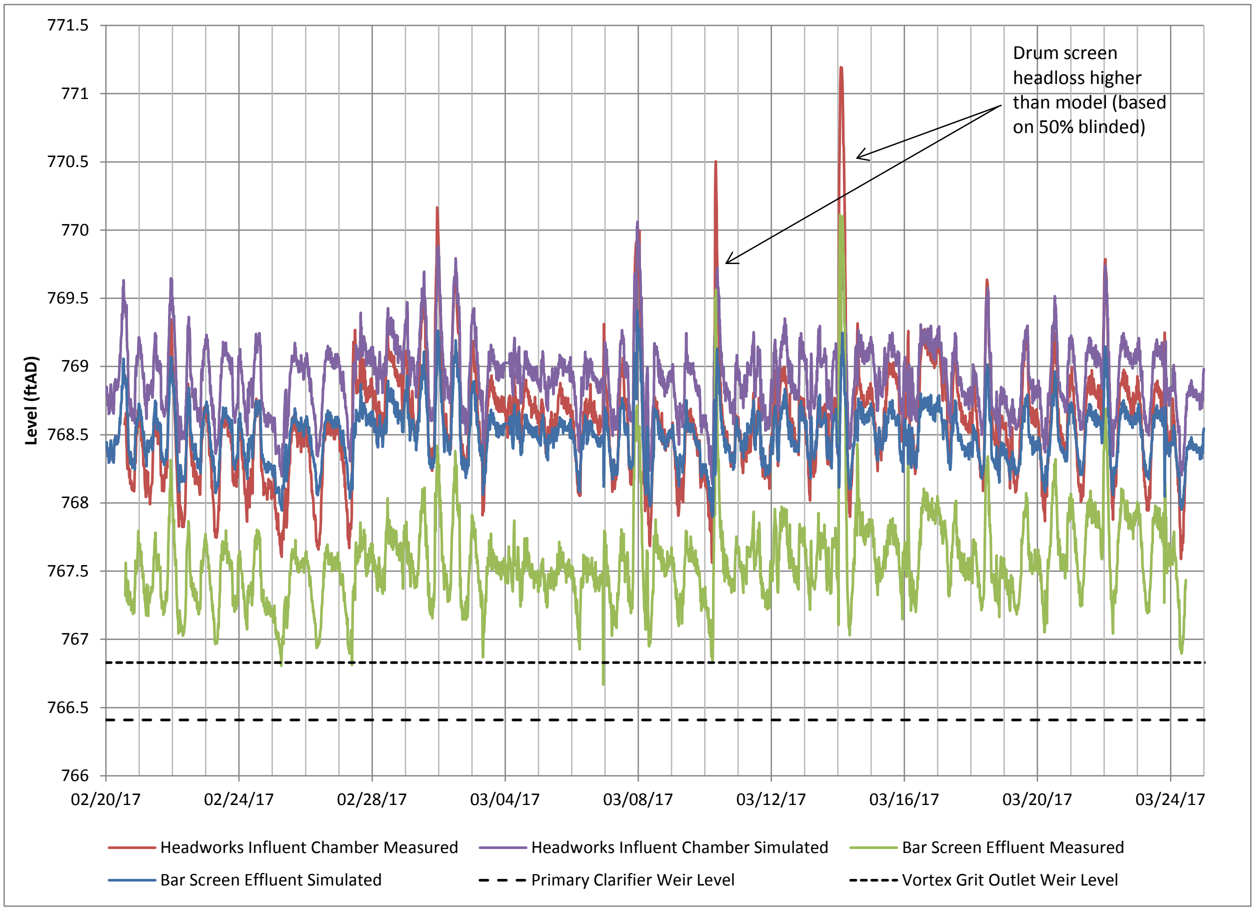



**Figure 22. Headworks Level Comparison**

## Plant Hydraulic Capacity

Following calibration, the hydraulic model was used to estimate the hydraulic capacity of each process area of the plant in order to assist the City of Atlanta in planning future expansion of the WRC. Each area of the plant was simulated individually, with the hydraulic capacity determined as the flow at which submergence of weirs or flooding would start to occur.

Due to low rainfall throughout the four week calibration period, the model was calibrated against flows of only 50-130 mgd. Surcharged headlosses increase in proportion to velocity squared so any additional losses from e.g. sediment deposition not observed during the site survey, could have a large effect on the maximum capacity. The hydraulic capacities calculated therefore generally represent ideal conditions with clean conduits and most units in service.

Table 2 summarizes the hydraulic capacities determined for the different areas of the plant. Using a RAS rate of 50%, the lowest capacity is at the primary clarifiers where only 210 mgd could be passed through the clarifiers before submergence is shown at the V notch weirs. This is also the area of the WRC where flooding would occur first at a flow of 200 to 340 mgd depending on the control of the clarifier bypass gate.

**Table 2 RM Clayton Hydraulic Capacity Summary**

| Process Unit | Units in Service / Case | Hydraulic Capacity | Comments |
|---|---|---|---|
| Outfall | Typical river level | 360 MGD | Before submergence UV effluent weir |
| | 100yr river level | 250 MGD | |
| UV Plant | 3 | 170 MGD | Before submergence UV influent weir |
| | 4 | 230 MGD | |
| | 5 | 280 MGD | |
| Effluent Filters | Before bypass | 270 MGD | Before bypass |
| | Filters | 360 MGD | Before submergence filter 1 influent weir |
| Secondary Clarifiers | Upstream | 510 MGD (inc RAS) | Before submergence splitter box weirs to clarifiers 5 and 6 |
| | Downstream | 330 MGD | Before submergence clarifier 1 and 2 V notch weirs |
| Mixed Liquor Pumps | - | 385 MGD (inc RAS) | Before wet well level not maintained |
| BNRs | Downstream | > 600 MGD (inc RAS) | Before submergence of BNR effluent weirs |
| Primary Clarifiers | Upstream, bypass closed | 200 MGD | Before flooding from channel opposite old grit tanks |
| | Upstream, bypass open | 340 MGD | |
| | Upstream, bypass controlled | 270 MGD | |
| | Downstream | 210 MGD | Before submergence clarifier 7 V notch weirs |
| Headworks | Primary bypass closed | 240 MGD | Before surcharging of upstream catchment |
| | Primary bypass open | 270 MGD | |
| | Primary bypass closed | 340 MGD | Before flooding within headworks |
| | Primary bypass open | > 400 MGD | |

Figure 23 shows the hydraulic profile through the WRC at 210 mgd.

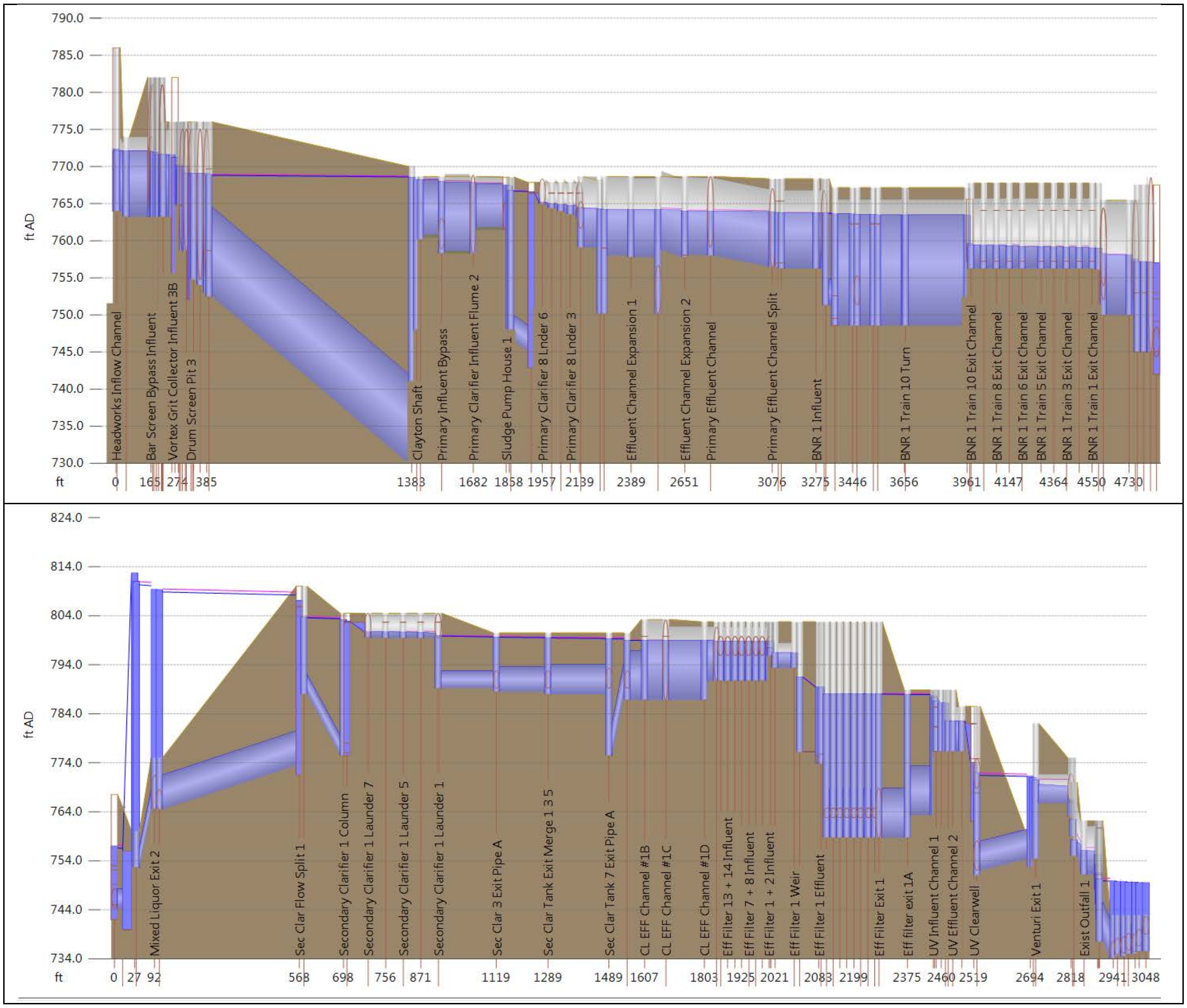



**Figure 23. Model Long Section at 210mgd**

## DISCUSSION

### ICM Adaptations

Although ICM is predominantly used as a catchment modeling tool, accurate modeling of a treatment plant has been achieved by careful adaptation. Key items included:

- Fixed headloss coefficients used for all conduits
- All weir discharge coefficients amended from default values and increased above typical values where weirs are submerged
- All sluice discharge coefficients amended from default values and determined by separate spreadsheet calculations based on empirical data. VSGates used widely to allow sluices to be opened/closed using RTC
- Launder channels split into sections and capacities checked using detailed spreadsheet backwater calculations
- User defined shapes used for non-standard conduit sections

- Pressure conduits used for pipes which are always surcharged and Preissmann slot width reduced for other conduits by amending model celerity ratio
- Node areas set to small values where chambers are not present in reality
- Head versus discharge tables used extensively
- Variable discharge orifices used to represent butterfly valves which need to be opened/closed using RTC

A limitation of the software is that headloss and discharge coefficients within the model cannot be adjusted during a simulation. Fixed values have to be entered for each model scenario. This was not a major problem for modeling RM Clayton, but would be significant if, for example, simulation of sluice operation was required since the required sluice discharge coefficient is a function of opening position.

**Real Time Control**

The RTC function in ICM has been used extensively to simulate pump control, bypass gates and the automatic operation of a new multi-tray grit removal plant. RTC was also used, in combination with variable discharge orifices, to circulate RAS within the works, varying as a fixed percentage of plant inflow.

Sophisticated control can be replicated in ICM although the RTC can only work sequentially which prevents control based on information from previous timesteps or rolling averages. Incorporation of RTC within each model scenario also makes tuning of control laborious as models must be validated and committed before every simulation.

**Calibration**

Calibration of a treatment plant is difficult due to the large number of hydraulic breaks present. No existing level measurement and a limited number of portable instruments limited calibration to eight locations, however these were carefully chosen to maximize their benefit.

Very good agreement was achieved in most locations with very few modifications of the model required. Comparing the results graphically as well as numerically enabled constant offsets to be very easily distinguished from under/over-estimation of headlosses which are flow dependent.

The four week calibration period unfortunately did not include any high flow events and subsequent calibration of the RM Clayton headworks modifications was instead made dependent on high flows, rather than of a fixed time duration. The four week period did though enable calibration for different numbers of process units in service as events occurred and were logged by operations staff. RTC was used to open/close tanks in the model and replicate these real events.

A site survey was extremely beneficial in confirming key elevations across the plant, invert levels at the portable instrument locations and sediment depths present.

## CONCLUSIONS

InfoWorks ICM has been used to produce a detailed hydraulic model of RM Clayton WRC. The model is significantly more flexible and dynamic than the previous spreadsheet model of the plant and will be integrated with the City of Atlanta's existing catchment system model.

Accurate modeling and good agreement with measured data was achieved with careful choice of model components and setting of model coefficients. The model was then used to determine the hydraulic capacity of each area of the plant to assist with planning of future work.

## REFERENCES

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I E Idel'chik (2003) Handbook of Hydraulic Resistance, 3rd edition, Jaico Publishing House