Flood Protection Level of Service Provided by Existing District Infrastructure for Current (2015) Sea Level Conditions and Three Future (2065) Sea Level Scenarios for Golden Gate Watershed – Final Report

FLOOD PROTECTION LEVEL OF SERVICE FOR BIG CYPRESS BASIN: CURRENT AND FUTURE SERVICE IN GOLDEN GATE, COCOHATCHEE, HENDERSON-BELLE MEADE, AND FAKA UNION WATERSHEDS

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1.0 Introduction

The South Florida Water Management District (District) is conducting a system-wide review of its regional water management infrastructure to determine the flood protection level of service (FPLOS) currently provided. The FPLOS describes the level of protection provided by the water management facilities within a watershed under both current and future conditions, where future conditions FPLOS considers sea level rise and future development. This information can be used by local governments, the SFWMD, and other state and federal agencies to identify areas where improvements or upgrades of water management facilities are required, the appropriate entity or entities responsible for making improvements, and funding and technical resources available to support these efforts.

The four watersheds within the BCB Study Area, along with the primary canal network, are depicted on **Figure 1.0.1.** This Final Report incorporates input from the District on two previous draft reports describing current and future (2065) FPLOS in the Golden Gate Watershed. Future conditions modeling is used to identify potential flood protection issues caused by future development and sea level rise. Subsequent draft reports will document the following:

- FPLOS provided within the Cocohatchee, Henderson/Belle Meade, and Faka Union Watersheds under current conditions
- FPLOS provided within the Cocohatchee, Henderson/Belle Meade, and Faka Union Watersheds under future sea level rise and future land use

The flood protection LOS is determined through a number of metrics, the majority of which are derived from the outputs of watershed-scale flood event modeling. The flood protection metrics are defined in **Section 2.0**.

Previous tasks completed as part of this study effort include model conceptualization, model calibration, and model verification. The model tool used for this study is a MIKE SHE / MIKE-11 model originally developed by others for long-term simulations. Because the FPLOS performance metrics rely on model outputs resulting from synthetic short-term high intensity rainfall events, it was necessary to re-conceptualize certain aspects of the model, and to then re-calibrate the model using data from short-term high intensity rainfall events of record. The model calibration/verification effort is documented in the April 2017 report titled *Deliverable 2.5 Model Recalibration*. The re-calibrated version of the BCB MIKE SHE / MIKE-11 model is referred to herein as Version EC-Cal, dated March 2017, was used for the subsequent design storm simulations described in **Sections 3.5** and **5.0** for current and future conditions, respectively.

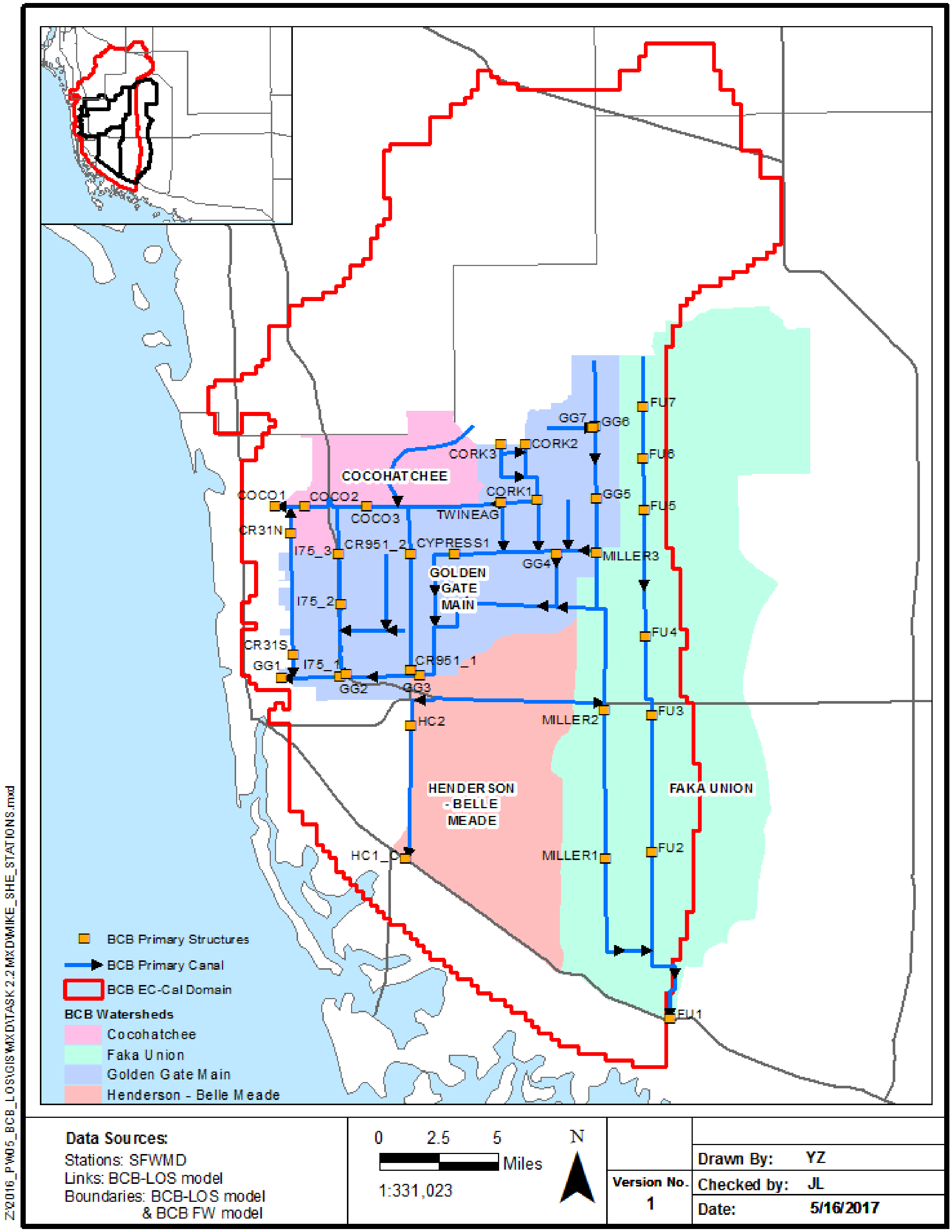

Figure 1.0.1 – BCB Watersheds, Primary Canals, and Primary Structures

2.0 Flood Protection LOS Performance Metrics

The District relies on six (6) formal performance metrics (PMs) to evaluate the flood protection LOS provided by the primary water management infrastructure. These metrics, defined briefly in this section, were derived from the District publication *Flood Protection LOS Analysis for the C-4 Watershed, Appendix A: LOS Basic Concepts* (SFWMD H&H Bureau, December 29, 2015). **Section 4.0** provides the results of the FPLOS evaluation for current conditions, and **Sections 6.0** and **7.0** provide the results of the FPLOS evaluation for future conditions within the Golden Gate Watershed.

<u>PM #1 Maximum Stage in Primary Canals</u> – This is the peak stage profile in the primary canal system. The profile is developed for a range of design storms (5-year, 10-year, 25-year, and 100-year). The largest design storm that stays within the canal banks establishes the FPLOS of the primary canal system.

<u>PM #2 Maximum Daily Discharge Capacity through the Primary Canals</u> – PM #2 is the maximum discharge capacity throughout the primary canal network. Discharge is calculated as areally weighted flow, in units of cubic feet per second per square mile of contributing area. Tidal effects are filtered by using a 12-hour moving average of discharge.

<u>PM #3 – Structure Performance – Effects of Sea Level Rise</u> – This metric shows the effective capacity of a tidal structure. It is comparable to the static design condition assumed in the original design, but compares structure flow over a range of storm surge events and a range of sea level rise scenarios. For the BCB FPLOS evaluation, this metric will be evaluated internally by District staff and will therefore be documented separately.

<u>PM #4 Peak Storm Runoff – Effects of Sea Level Rise</u> – PM #4 is the maximum conveyance capacity of a watershed at the tidal structure for a range of design storms. It shows the maximum conveyance (moving 12-hour average) for a specific design storm and a specific tidal boundary condition. This metric examines the behavior of the system under severe stress and can be used to check if conditions exceed design limits. In evaluating this PM, it is assumed that design rainfall and design storm surge occur simultaneously or with a temporal offset that maximizes stress on the structure.

<u>PM #5 Frequency of Flooding – Stage-based FPLOS for Subwatersheds</u> – In this PM, the flood elevations or depths of overland flooding are evaluated for a range of design storms (5-year, 10-year, 25-year, and 100-year). These flood depths/elevations can then be compared with elevations of build features such as buildings and roadways, where such information exists. For the purposes of this BCB FPLOS evaluation, flood inundation maps were developed from the model output for each storm event.

<u>PM #6 Duration of Flooding</u> – PM #6 quantifies the duration of flooding at specific locations of interest within a watershed. For this metric, potential flood-locations associated with existing or future development are identified, and a representative threshold elevation corresponding to building or roadway flooding is chosen. The length of time the flood elevation is projected to be above that elevation is the duration of flooding.

3.0 Current Conditions Application Model Set-Up

The re-calibrated BCB MIKE SHE / MIKE-11 model, described in the April 2017 report titled *Deliverable 2.5 Model Recalibration* and referred to herein as Version EC-Cal, was used as the basis for the design storm simulations described in this section. Several changes to the EC-Cal model setup were required in order to simulate the synthetic design storm events. These changes included structure operations, tailwater boundary conditions, rainfall, and initial conditions.

Model Parameters and Structure Operations: The current conditions model application incorporated all applicable changes to the model setup and parameterization documented in the Model Recalibration Report. These include hydrologic parameter adjustments (i.e., drain codes, drain levels, drain time constants, detention storage, and paved runoff coefficients), hydraulic parameter adjustments (manning's roughness coefficients), and addition of MIKE-11 branches in the Cocohatchee Canal subwatershed. All recorded structure operations from the calibration model have been replaced with rule-based operations, based on the structure descriptions in the District's *Water Control Operations Atlas: Big Cypress Basin System* dated September 21, 2016 and updated March 31, 2017. The Miller 3 structure was updated to represent the post-2015 configuration (with 3 4x8 dual-leaf gates), as described in the most recent Water Control Operations Atlas. The pump stations associated with the ongoing Picayune Strand Restoration project (on the Miller and Faka Union Canals) are not included in the current conditions model, but will be included in the future conditions modeling of the Faka Union system.

Tailwater Boundaries at Coastal Outfall Structure GG1: Tailwater hydrographs for use in the design storm simulations are documented in a report prepared by the District (ref: South Florida Water Management District, 2017. Flood Protection Level of Service Analysis for the Big Cypress Basin. Appendix C: Preparation of Boundary Conditions at the Tidal Structures. H&H Bureau, SFWMD, West Palm Beach, FL. 34 pp. June 21, 2017). The accompanying spreadsheets included existing conditions storm surge hydrographs for the 5-year, 10-year, 25-year, and 100-year return intervals. Also included were storm surge hydrographs for future sea level rise. For the current conditions model, the columns in the spreadsheet designated "YEAR2015/IPCCAR-MEDIAN", were used, which represent current sea level. **Table 3.0.1** and **Figure 3.0.1** show the peak tailwater levels and water level time series, respectively, for each return interval.

Table 3.0.1 – Peak Tailwater Stages for Current Sea Level at GG1

Return Period	Current Sea Level Conditions Peak Tailwater Stage at GG1 (ft. NAVD88)
5-year	4.93
10-year	5.38
25-year	5.95
100-year	6.89

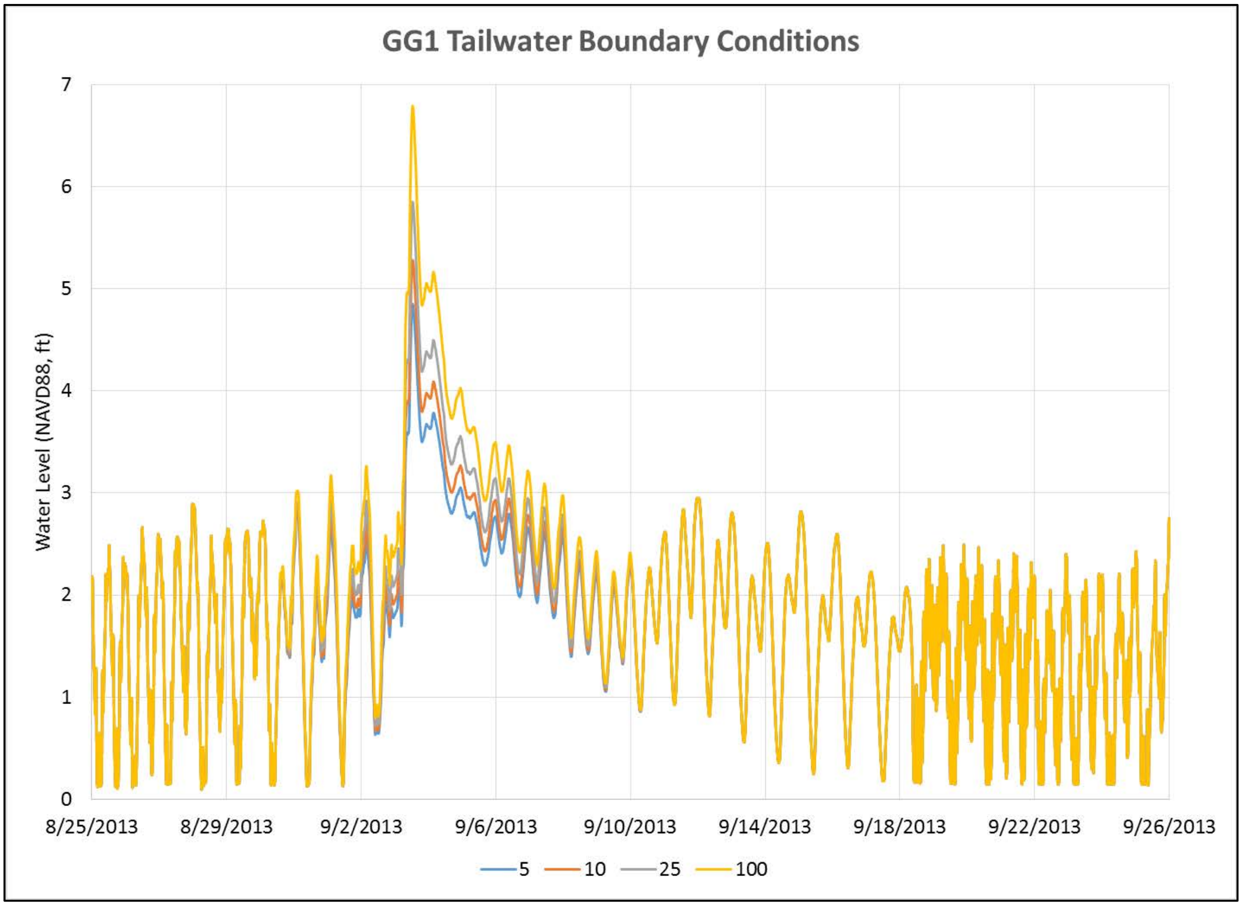

Figure 3.0.1 – GG1 Tailwater Boundary Conditions for Current Sea Level

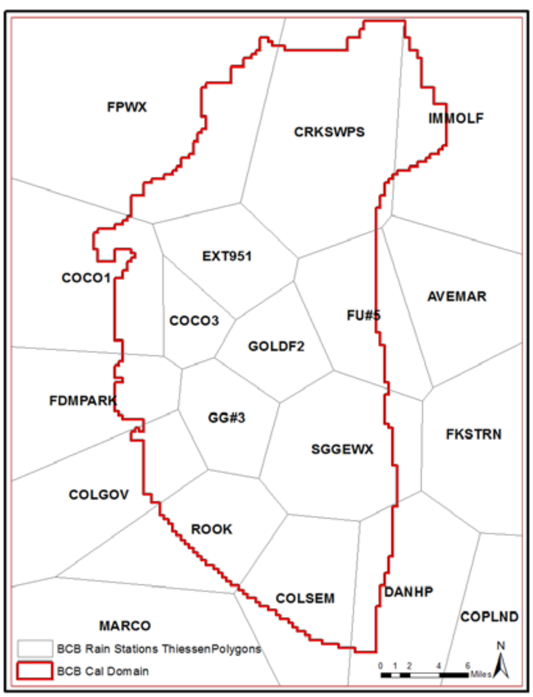

Figure 3.0.2 – Rainfall

Rainfall: In the calibration models, the rainfall was distributed on the 2 km x 2 km NEXRAD radar rainfall grid. For the design storm simulations, this was replaced with a Thiessen-polygon approach, identical to the approach used for the design storm runs in the BCB-FW model developed by Lago Consulting in 2015. The centroid of each polygon corresponds to a rainfall gage location. Rainfall 3-day distribution and totals for each return period were based on the SFWMD Environmental Resource Permit Information Manual Volume IV, Water Resource Regulation Department (July 2010 version). Rainfall totals varied from polygon to polygon based on the position of each centroid relative to the isohyets published in the Manual. Rainfall depths for each Thiessen polygon are listed in **Table 3.0.2**.

Table 3.0.2 – Total Design Storm Rainfall Depths

Rainfall Gage	5-year	10-year	25-year	100-year
COCO1	7.69	9.24	11.33	13.98
COCO3	7.64	9.14	11.09	13.69
COLGOV	7.75	9.63	11.81	14.75
COLSEM	7.68	10.10	12.51	15.91
CRKSWPS	7.37	8.55	9.74	11.84
EXT951	7.60	9.04	10.93	13.28
FKSTRN	6.76	8.47	9.92	13.37
DANHP	7.52	10.03	12.23	15.86
GOLDF2	7.57	9.02	10.94	13.68
IMMOLF	6.65	7.76	8.70	10.71
MARCO	7.81	10.20	12.85	16.10
ROOK	7.75	9.94	12.08	15.37
SGGEWX	7.55	9.13	11.05	14.28
AVEMAR	6.87	8.08	9.29	11.70
FPWX	7.56	9.03	10.73	13.25
GG#3	7.66	9.32	11.33	14.27
COPLND	7.18	9.90	11.97	15.68
FU#5	7.32	8.58	10.02	12.30
FDMPARK	7.74	9.48	11.65	14.43
Area-Weighted 3-Day Rainfall Depth (inches)	7.50	9.10	10.90	13.60

The dates for the rainfall distributions were adjusted so that the peak rainfall intensity lined up with the peak tailwater stage, per the Scope of Work. This resulted in a spin-up period of approximately one week with normal tidal boundaries and no rainfall, and with the peak rainfall intensity and peak tailwater stage occurring on September 3 2013 between noon and 1 PM.

<u>Initial Groundwater Levels and Surface Water Depths</u>: Initial groundwater levels were previously determined using the 90th percentile of levels simulated in the 2-year (2013-2014) model run used in the Curry Canal Structure Evaluation. The resulting initial flows at the Golden were considered to be too high (about 750 cfs); well above the September average flows of about 400 CFS over the most recent 8-year period (see **Figure 3.0.3**). After discussion with District staff, it was decided to try a simulation with lower initial water levels, corresponding to either the 80th or 85th percentile. Interflow used the results of the BCB-FW 7-year simulation (2008 through 2014) in conjunction with the stand-alone "DFSpercentiles" tool to calculate the 80th percentile water level in each model grid cell and within each groundwater layer. The resulting grid files were then used as the starting groundwater elevations. The 80th percentile overland water depths were also computed and used as the starting overland water depth. This modification resulted in initial flows at GG1 of about 400 cfs, which corresponds to the September average flow.

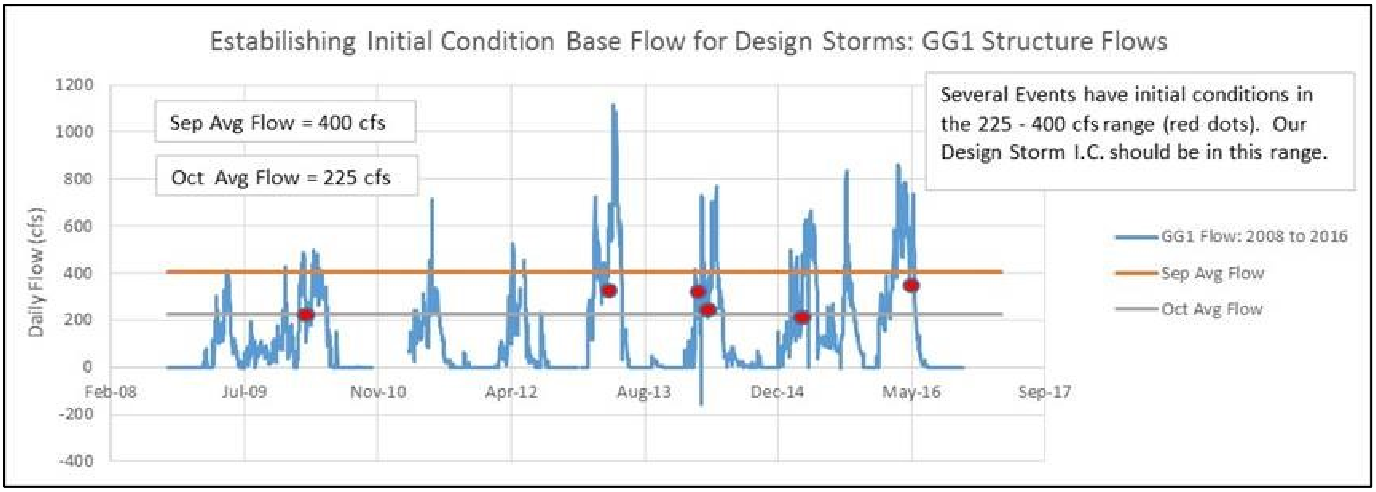

Figure 3.0.3 – GG1 Average September and October Flows, 2009 - 2016

4.0 Flood Protection Level of Service – Current Conditions

Once all of the model setup changes were completed to represent design storm conditions, the model was executed for the 5-year, 10-year, 25-year, and 100-year 3-day storm events. Model results were evaluated for stability and reasonableness prior to proceeding with the FPLOS evaluation. **Appendix A** provides summary model results at primary control structures, while **Appendices B** through **E** provide the complete flow and stage hydrographs for each of the four design storm events. The remainder of this section describes the results of the FPLOS evaluations for all relevant performance metrics, which for current conditions include PM #1, PM #2, PM #5, and PM #6.

4.1 PM #1 – Maximum Stage in Primary Canals

This is the peak stage profile in the primary canal system. The profile is developed for a range of design storms (5-year, 10-year, 25-year, and 100-year). The largest design storm that stays within the canal banks establishes the FPLOS of the primary canal system.

To evaluate this PM under current conditions with the Golden Gate Watershed, peak stage profiles were prepared for all primary canals within or bordering the Golden Gate Watershed. Bank elevations on the profile figures are generally based on the MIKE-11 cross section data. However, in several cases the bank elevations appeared suspect and were later modified (based on the current LiDAR data) for use in preparing the profile plots. Also shown in the Figures are major roadway landmarks, control structures, and primary canal junctions.

Table 4.1.1 summarizes the PM #1 Results shown graphically on **Figures 4.1.1** through **4.1.6**, listing the maximum return period profile that is contained within the canal banks. Although all of the canals contained the 10-year profile along the majority of the bank lengths, the bank elevation was exceeded for the 10-year event in multiple locations for all canals except the Airport Road Canal. It is important to note that based on the LiDAR topographic information, many of the bank exceedances are localized to small areas and the flooding does not extend more than a few hundred feet from the canal bank. PM #5 (frequency of flooding) depicts the extents of inundation for each design storm event, as discussed in **Section 4.3**.

Table 4.1.1 – PM #1 Summary Results

Canal Segment	Figure Number	FPLOS Provided
Golden Gate Main	**4.1.1**	5-year
Airport Road Canal	**4.1.2**	100-year
I-75 Canal	**4.1.3**	5-year
CR 951 Canal	**4.1.4**	<5-year
Cypress Canal	**4.1.5**	5-year
Corkscrew Canal	**4.1.6**	<5-year

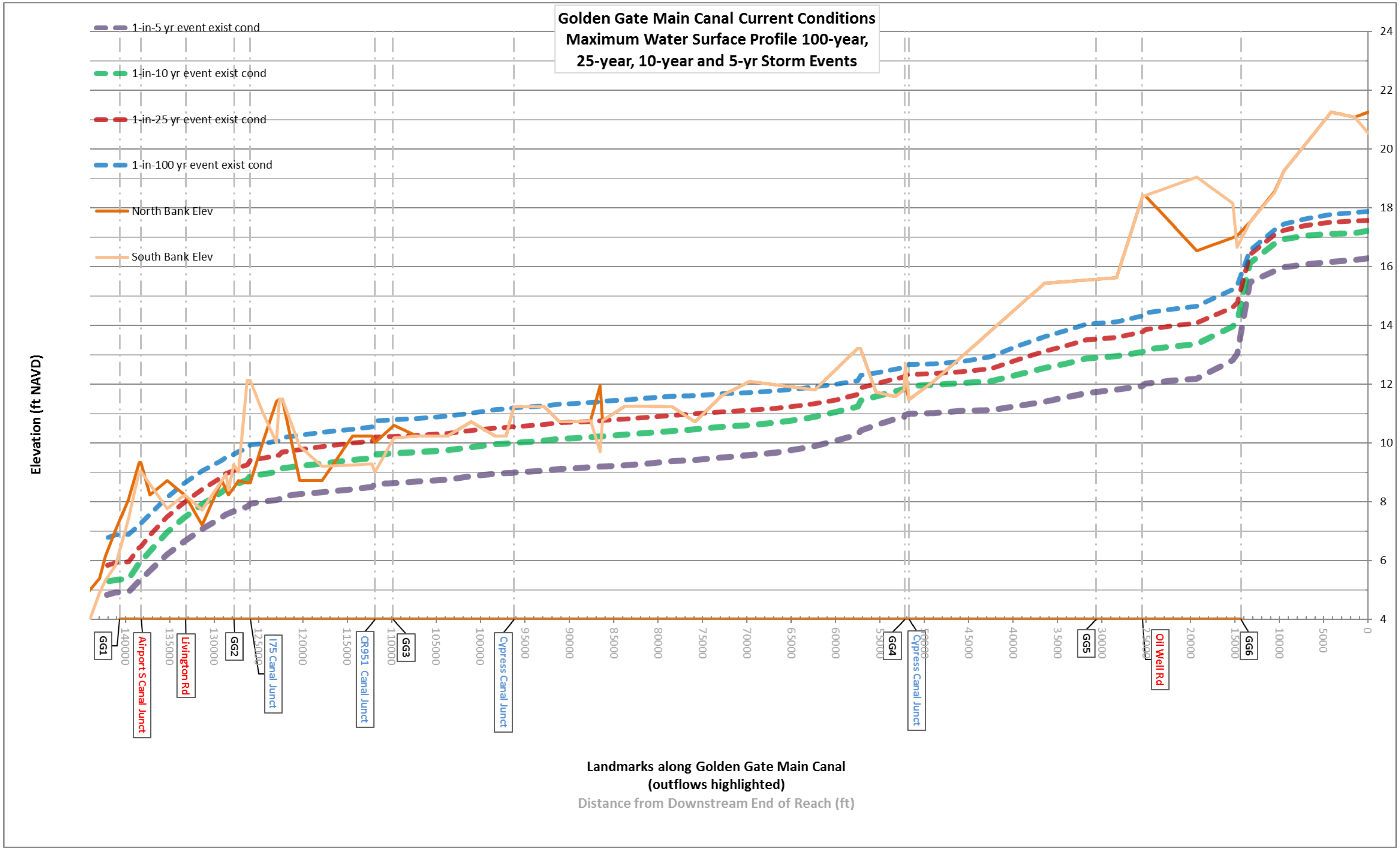

Figure 4.1.1 – Golden Gate Main Peak Stage Profiles

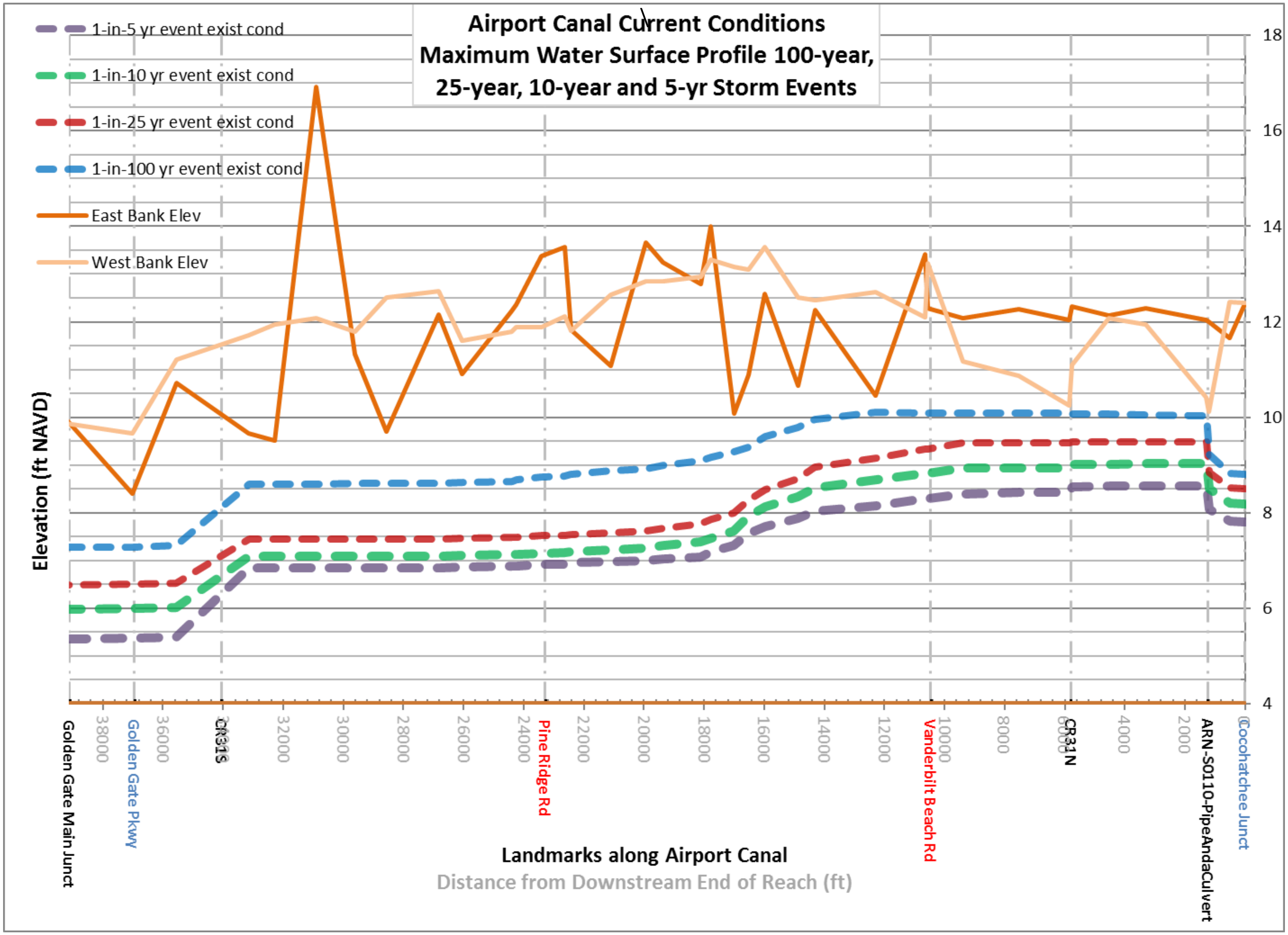

Figure 4.1.2 – Airport Road Canal Peak Stage Profiles

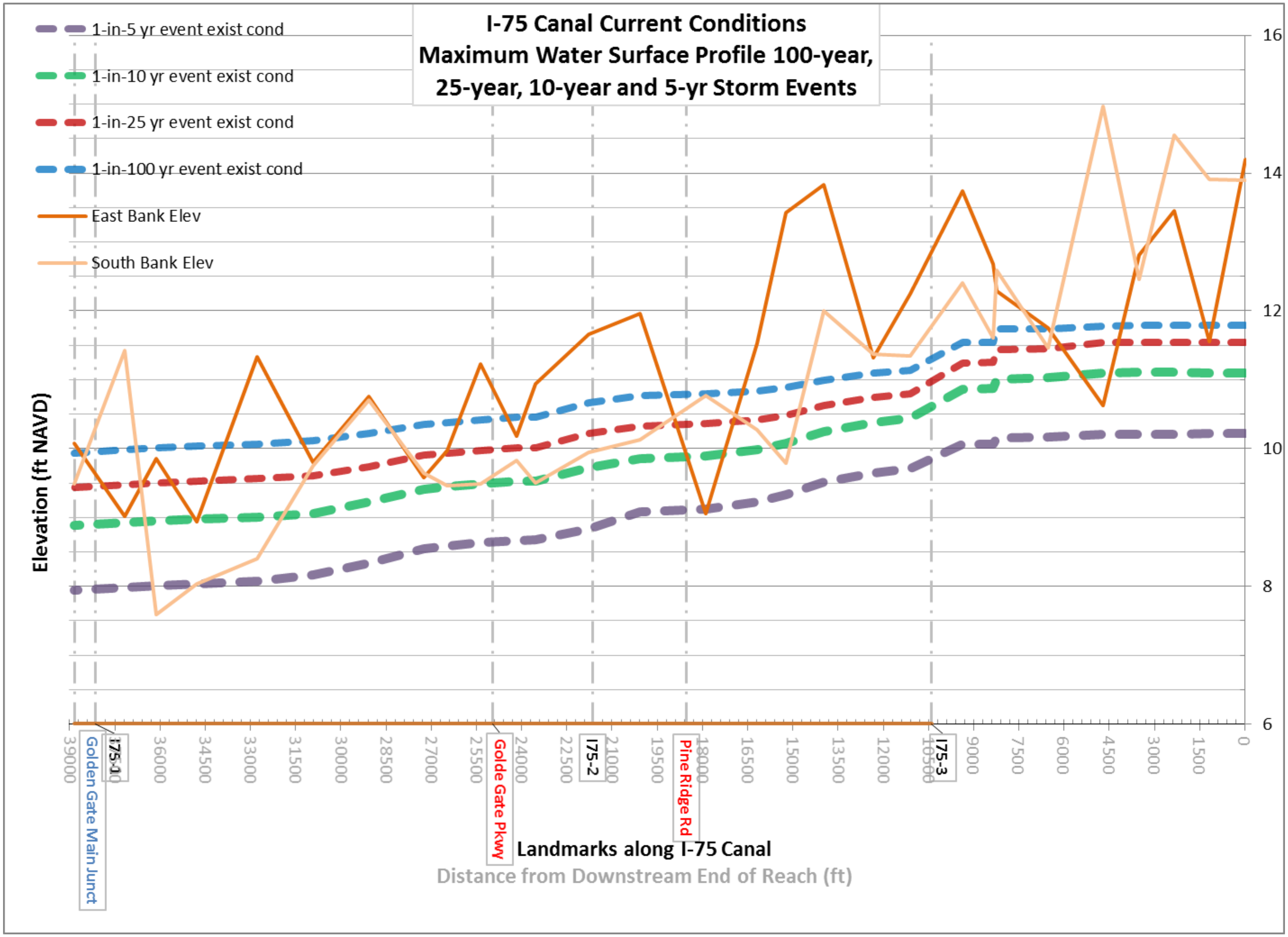

Figure 4.1.3 – I-75 Canal Peak Stage Profiles

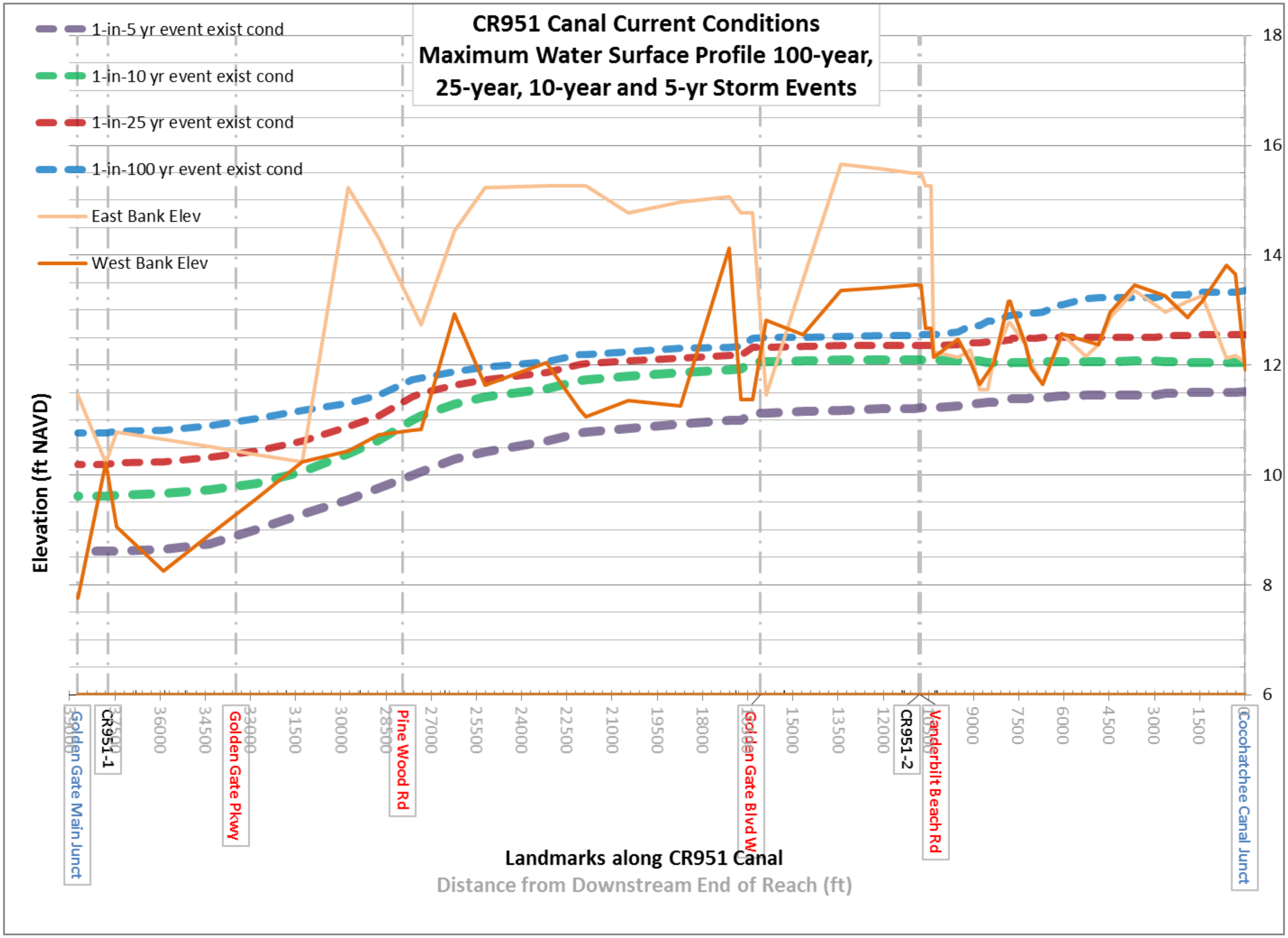

Figure 4.1.4 – CR951 Canal Peak Stage Profiles

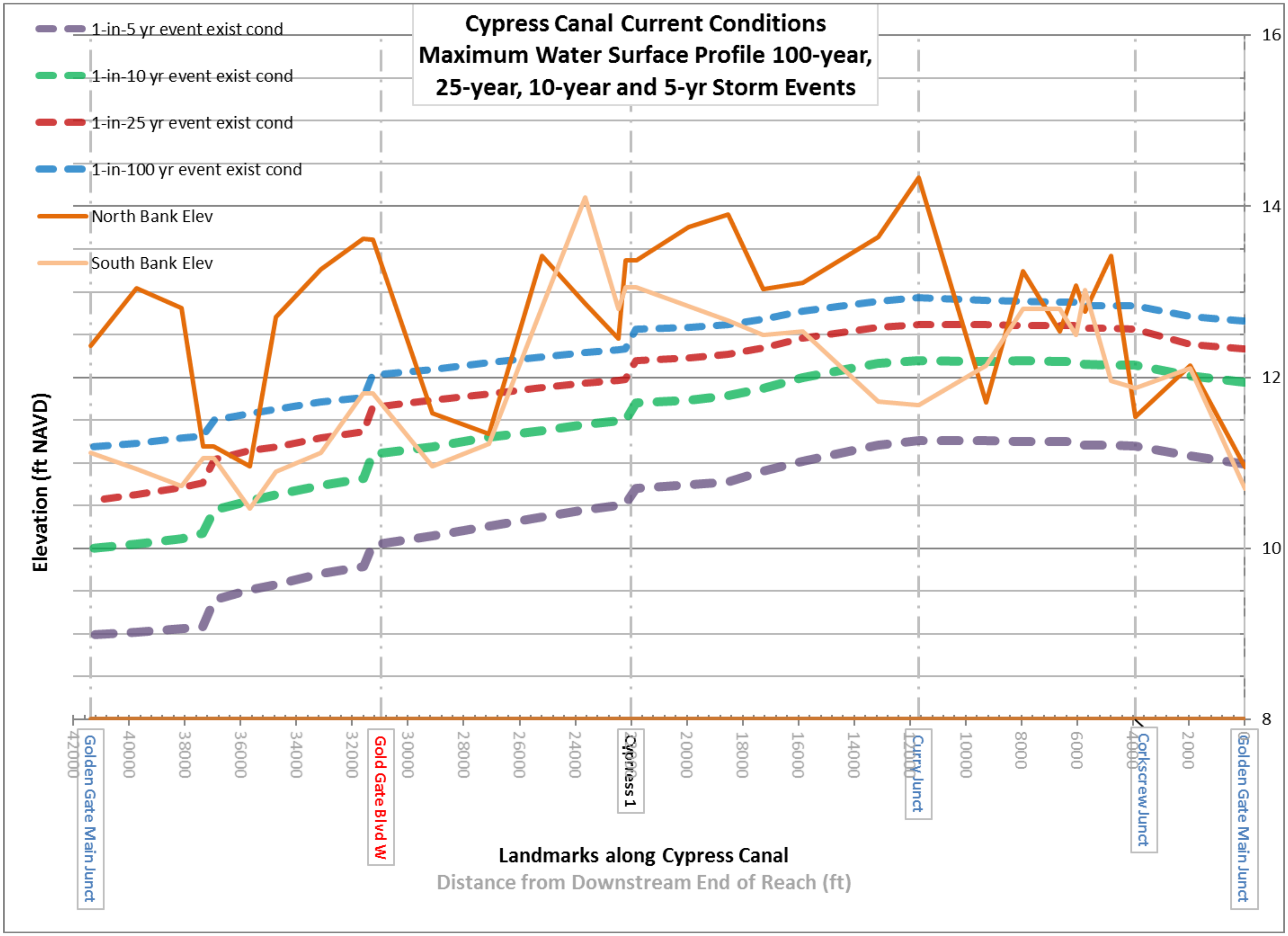

Figure 4.1.5 – Cypress Canal Peak Stage Profiles

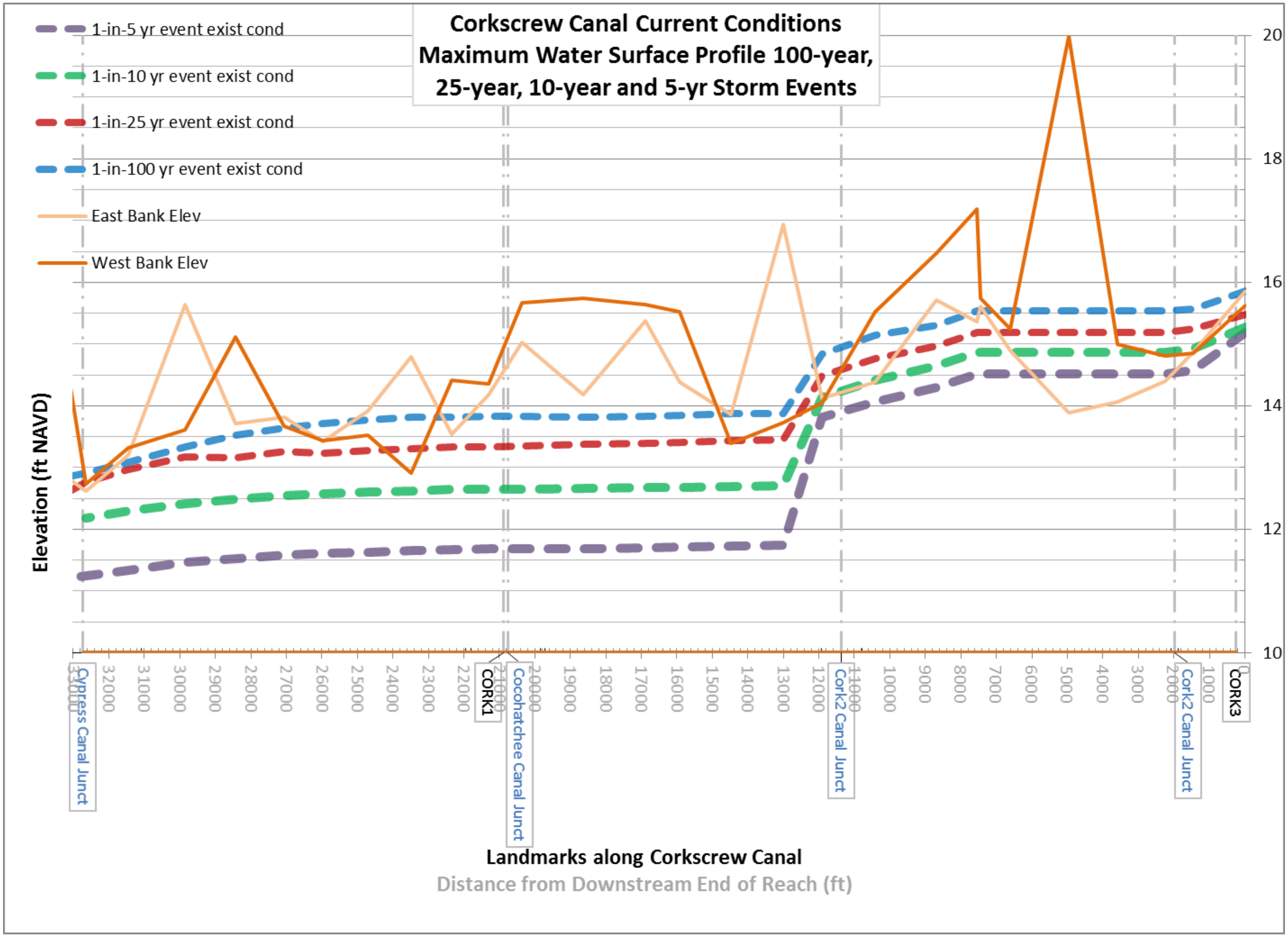

Figure 4.1.6 – Corkscrew Canal Peak Stage Profiles

4.2 PM #2 – Maximum Daily Discharge Capacity through the Primary Canals

PM #2 is the maximum discharge capacity throughout the primary canal network. Discharge is calculated for defined canal segments as areally weighted flow, in units of cubic feet per second per square mile of contributing area. Canal segments are generally those segments between water control structures. For example, the segment associated with structure GG1 is the Golden Gate Main Canal between structures GG1 and GG2, and the contributing area is defined as only the area contributing runoff to that segment. **Table 4.2.1** lists the canal segments identified for this analysis, where each segment is identified by the downstream structure. In the case of the Airport Road Canal, the outflows at the north and south structures were combined and treated as if they were a single structure. The table also identifies the contributing area for each canal segment, and the discharge capacity calculated for each segment associated with the 25-year, 3-day design storm event.

Discharge capacity was calculated by subtracting the hydrographs at all inflow points to each segment from the segment's outflow hydrograph, and then dividing the peak of the resulting net discharge hydrograph by the segment's contributing area. Tidal effects were filtered by using a 12-hour moving average of discharge.

Table 4.2.1 – Water Control Catchment Inflow and Outflow Points and 25-year Discharge Capacity

Structure/ Segment	Inflow Point(s)	Outflow Point(s)	Water Control Catchment Area (sq. mi.)	25- year Peak Discharge Capacity (cfs / sq. mi.)
GG1	GG2, CR31S	GG1	3.04	88
GG2	I75-1, CR951-1, GG3	GG2	3.07	48
GG3	CYPRESS1, GG4, C1-CONNECTOR	GG3	28.95	14
GG4	GG5, BEGINNING OF CYPRESS CANAL	GG4, MILLER3	6.02	36
GG5	GG6, GG7	GG5	8.84	36
GG6		GG6	4.34	38
GG7		GG7	3.33	23
CR31/ Airport Rd		AR1, AR2 (CR31 N & S)	6.28	24
I75-1	I75-2	I75-1	9.71	59
I75-2	I75-3	I75-2	4.62	35
I75-3		I75-3	4.10	61
CR951-1	CR951-2	CR951-1	9.54	25
CR951-2	BEGINNING OF CR951 CANAL	CR951-2	1.41	77
TWINEAGLE		TWINEAGLE	2.07	24
Cypress1	CURRY CANAL, CORKSCREW CANAL, BEGINNING OF CYPRESS CANAL	CYPRESS1	9.81	38
CORK1	CORK2, CORK3, TWINEAG	CORK1, BENGINING OF CURRY CANAL	6.26	27

Figure 4.2.1 shows the contributing areas ("water control catchments") draining to each canal segment. The catchment polygons were taken from the District's Arc Hydro Enhanced Database (AHED), and were used to determine the contributing areas listed in **Table 4.2.1.**

Figures 4.2.2 through **4.2.16** graphically depict the net area-weighted discharge hydrographs for each canal segment, and for each design storm event (5-year, 10-year, 25-year, and 100-year). Negative values on the graphs correspond to times where total inflows exceeded total outflows for each segment. The majority of these instances of negative net discharge result from a difference in timing of peak flows as the main flood wave moves downstream through the canal network.

For the majority of the canal segments, the calculated discharge capacity increases with increasing return period. However, in some cases (GG2, GG4, and Cypress1) the calculated discharge capacity is lower for the 100-year storm than for the 25-year storm. This appears to be a result of backwater effects and associated peak flow timing differences not seen in the smaller events. In the case of GG4 (**Figure 4.2.5)**, and Cypress1 (**Figure 4.2.14**), the stunted initial peak of the 100-year storm event is followed by a second peak that occurs when downstream tailwater levels drop, allowing the release of stored volume.

Although the peak of the net discharge hydrographs for each design storm event are referred to in this section as the calculated discharge capacity for each event, the true capacity of the canal segment is the net discharge corresponding to the largest design flood event that remains within the banks of the canal. For this reason, the results of PM#2 must be evaluated in conjunction with the results of PM#1 (Maximum Stage in Primary Canals) and PM#5 (Frequency of Flooding). From the PM#1 results presented in the previous section, peak stages in most canals exceed the canal banks for the 100-year event. In some cases, a 10-year event is sufficient to cause water levels to exceed the canal banks (Refer to PM#1 **Figures 4.1.3, 4.1.4,** and **4.1.5**), but these generally appear to be localized flooding instances that do not extend far from the canal banks. This is based on an examination of the PM#5 results, which are discussed in the next section.

Interbasin Flows

The discharge capacity at structure CORK1 (**Figure 4.2.16**) is partly a function of inter-basin flows entering from the Corkscrew Swamp. These flows enter the CORK system through structures CORK2 and CORK3, located along the northern boundary of the Golden Gate Watershed. **Figures 4.2.17** and **4.2.18** show the 25-year and 100-year design storm hydrographs at structures CORK2 and CORK3, respectively. Under the design storm conditions these inter-basin flows, totaling several hundred cubic feet per second, are subtracted (along with the Twin Eagles structure discharge) from the CORK1 catchment outflows, to compute the discharge capacity in the CORK1 water control catchment.

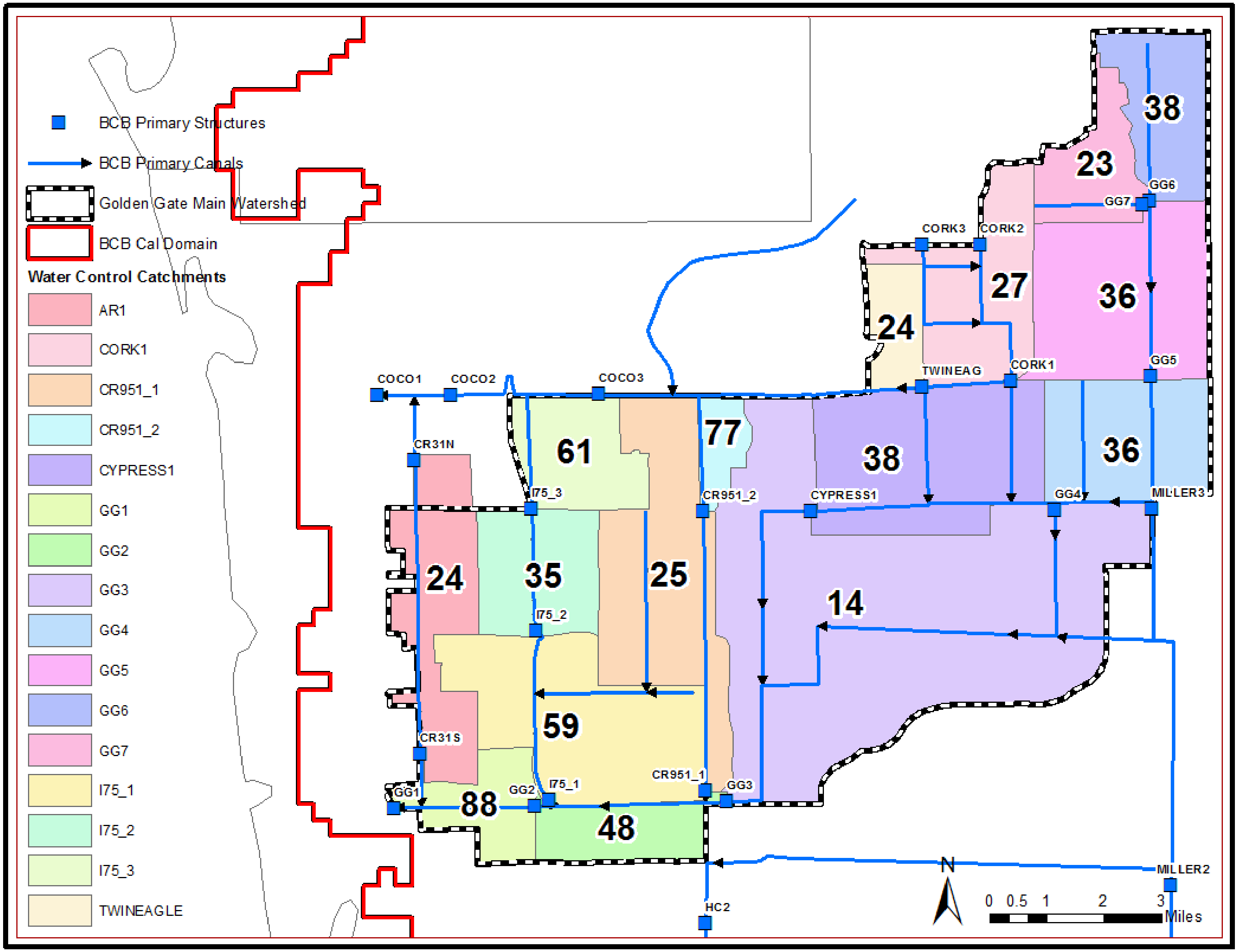

Figure 4.2.1 – Control Structure Catchments in Golden Gate Watershed, for Calculating PM#2

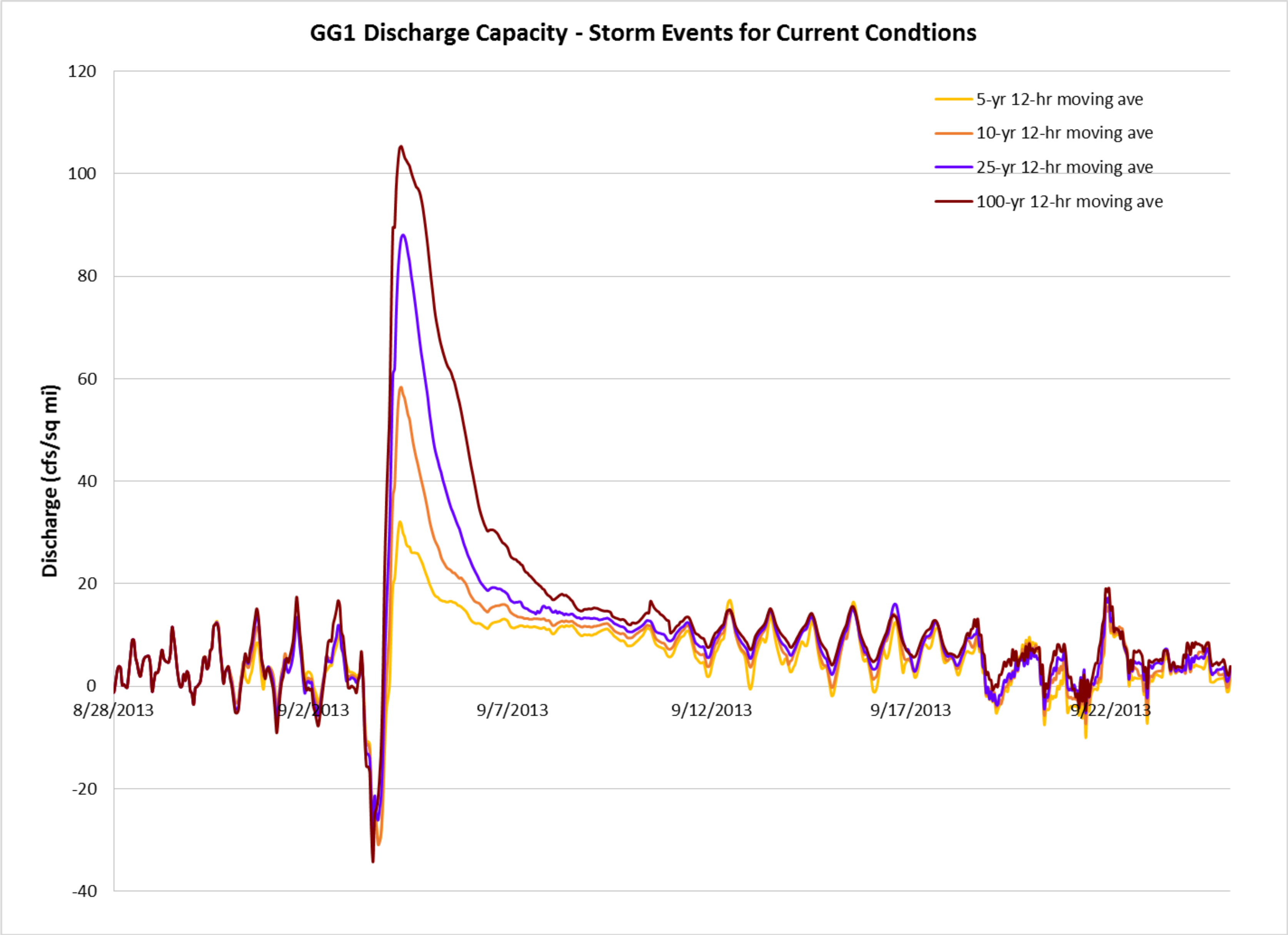

Figure 4.2.2 – Net Area-Weighted Discharge Hydrograph for GG1

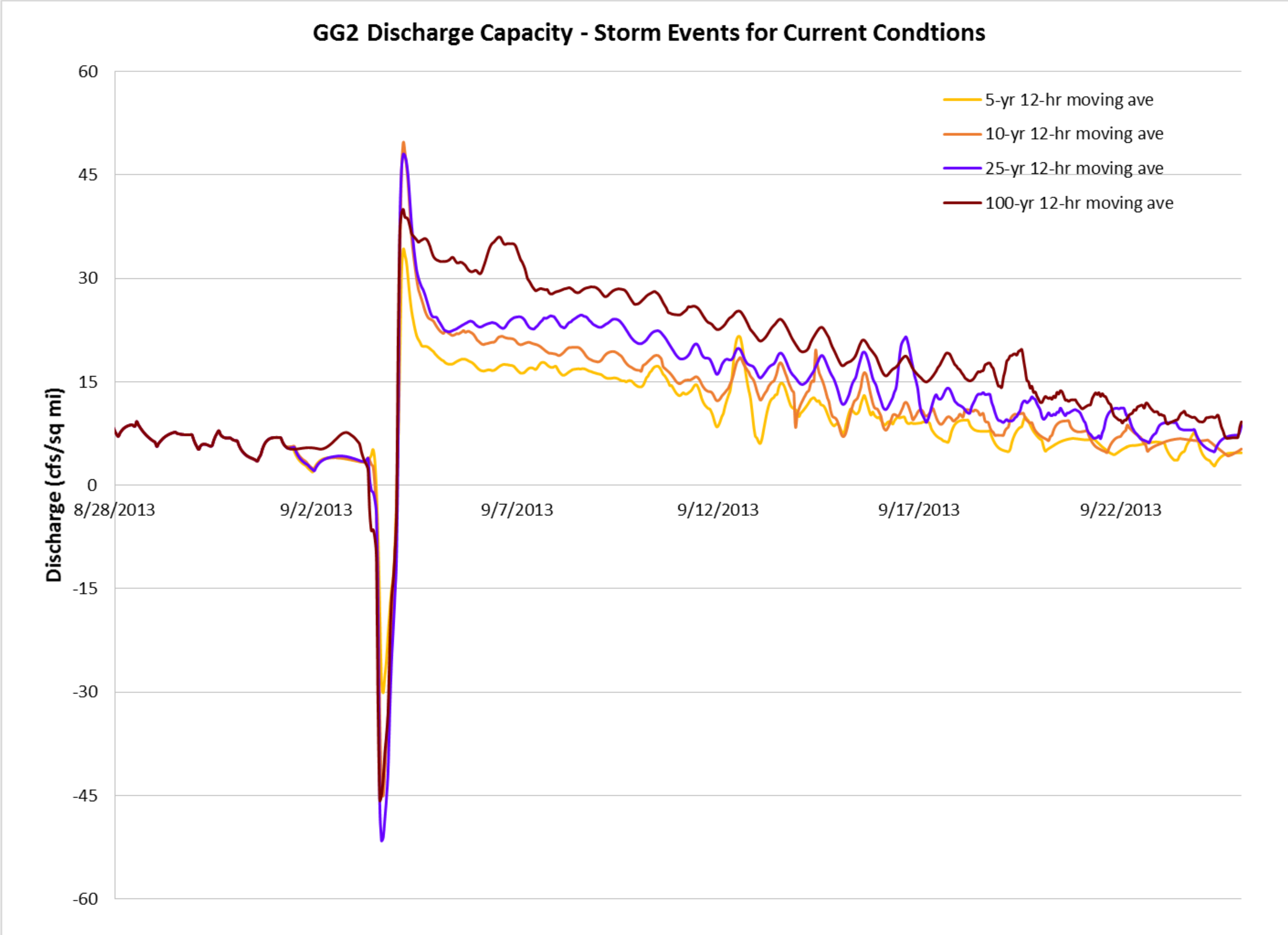

Figure 4.2.3 – Net Area-Weighted Discharge Hydrograph for GG2

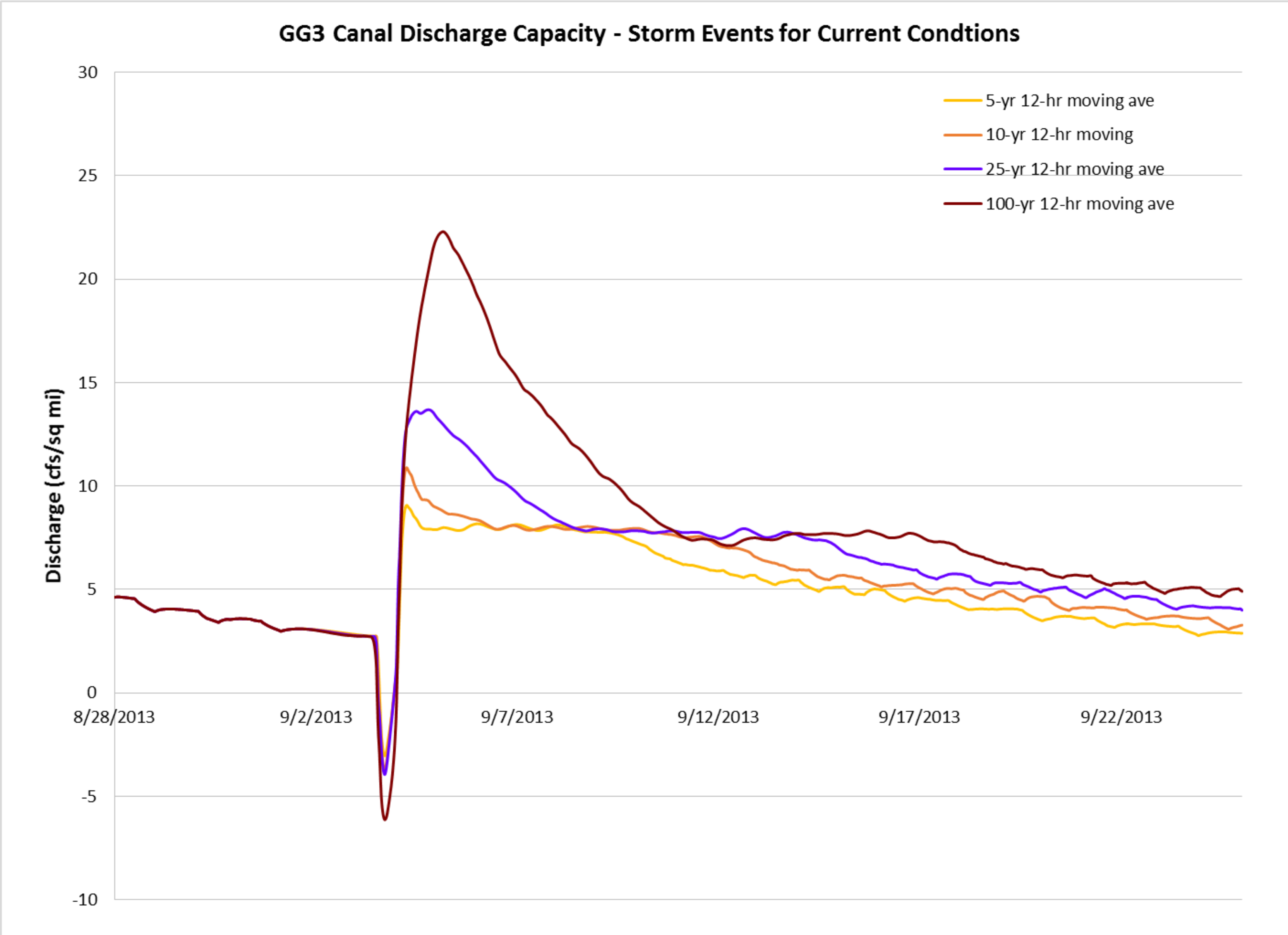

Figure 4.2.4 – Net Area-Weighted Discharge Hydrograph for GG3

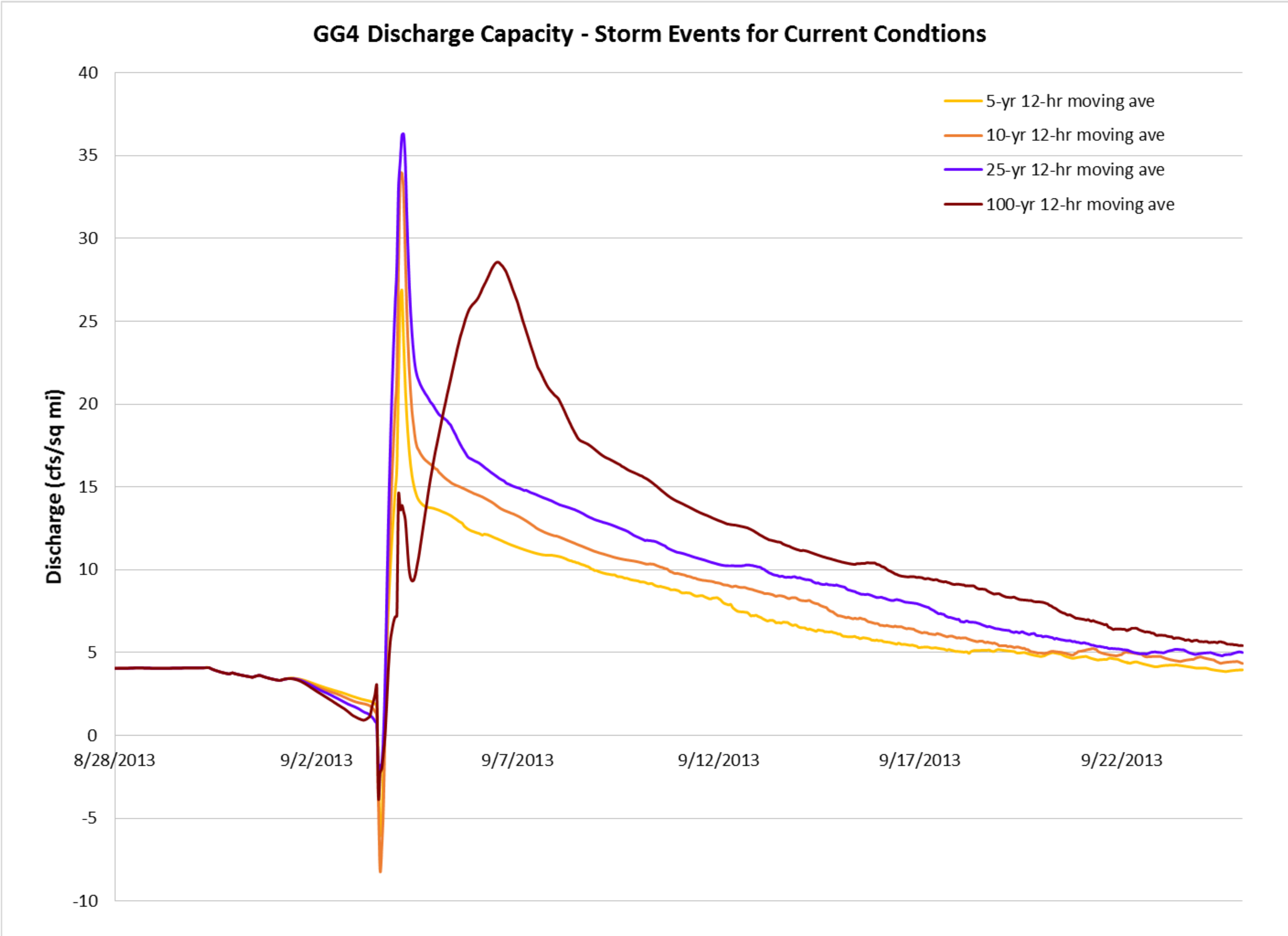

Figure 4.2.5 – Net Area-Weighted Discharge Hydrograph for GG4

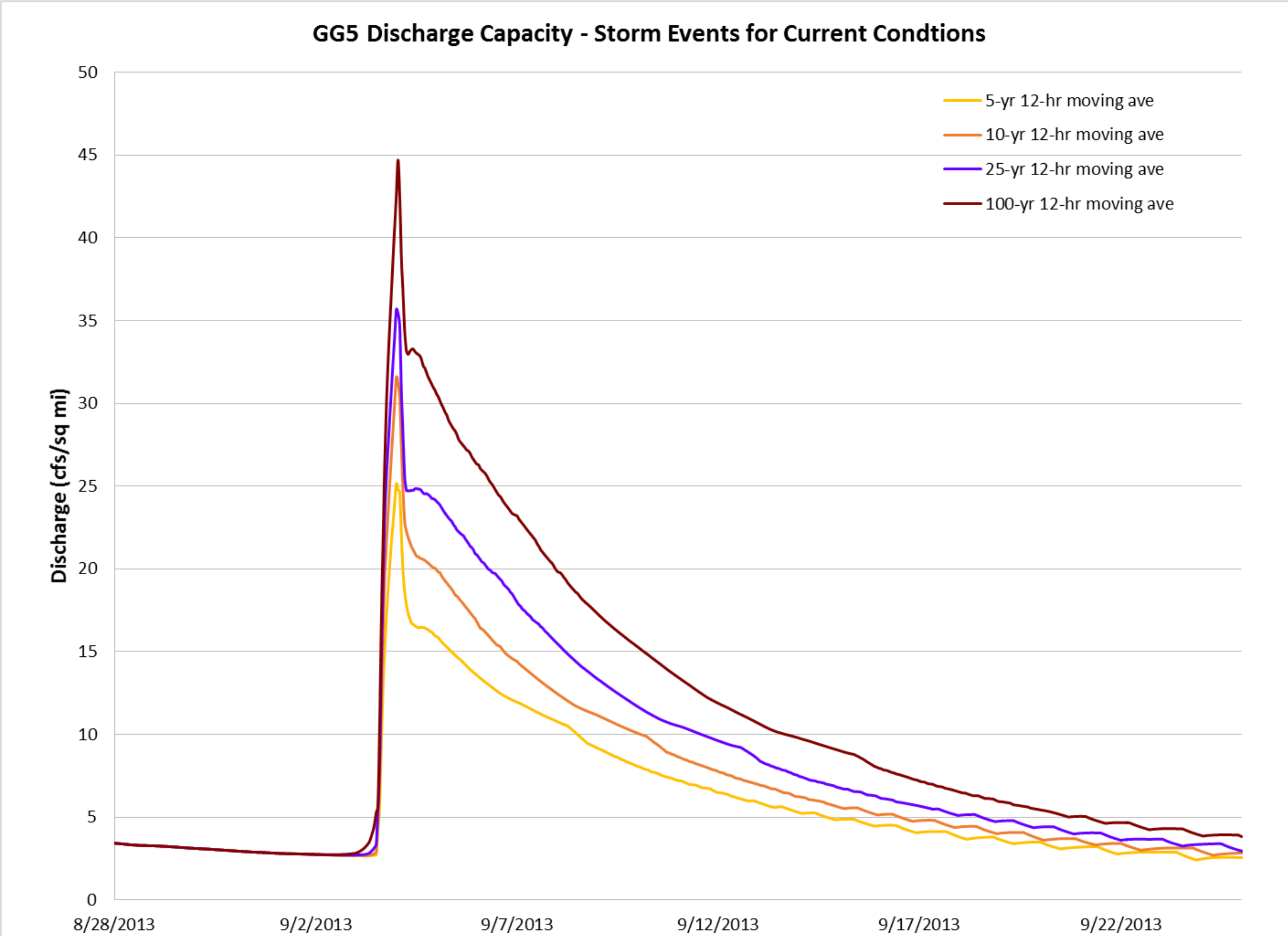

Figure 4.2.6 – Net Area-Weighted Discharge Hydrograph for GG5

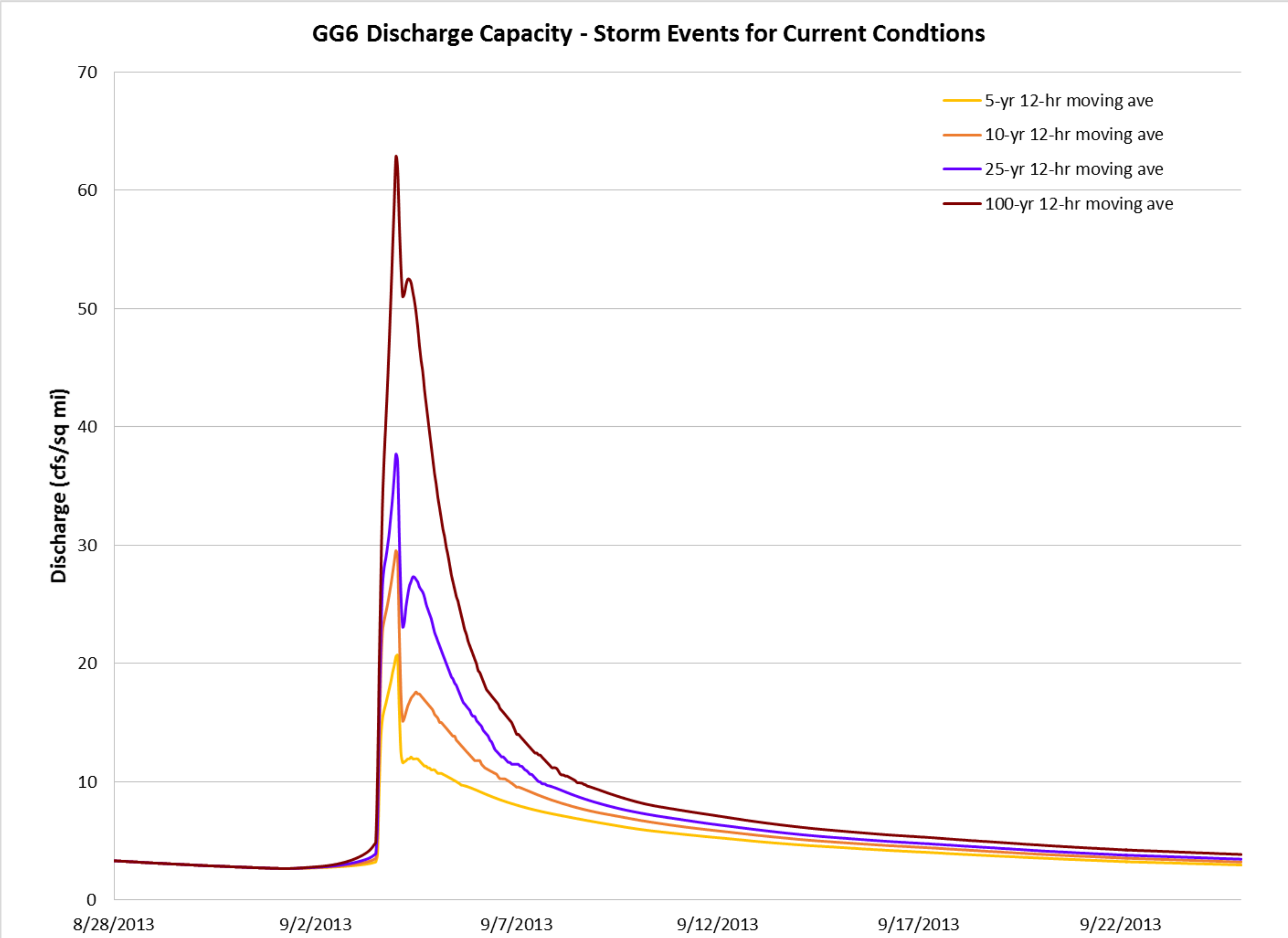

Figure 4.2.7 – Net Area-Weighted Discharge Hydrograph for GG6

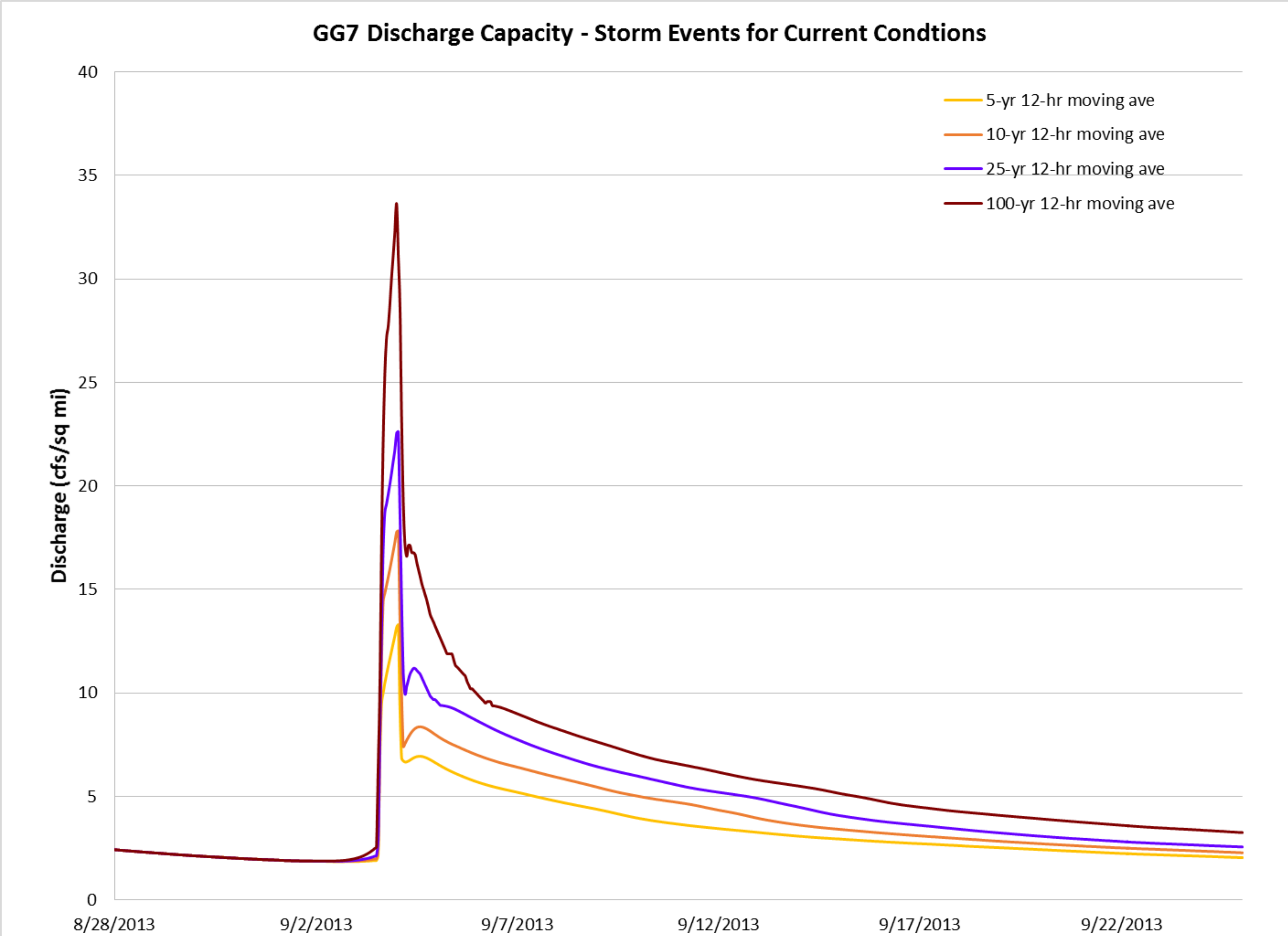

Figure 4.2.8 – Net Area-Weighted Discharge Hydrograph for GG7

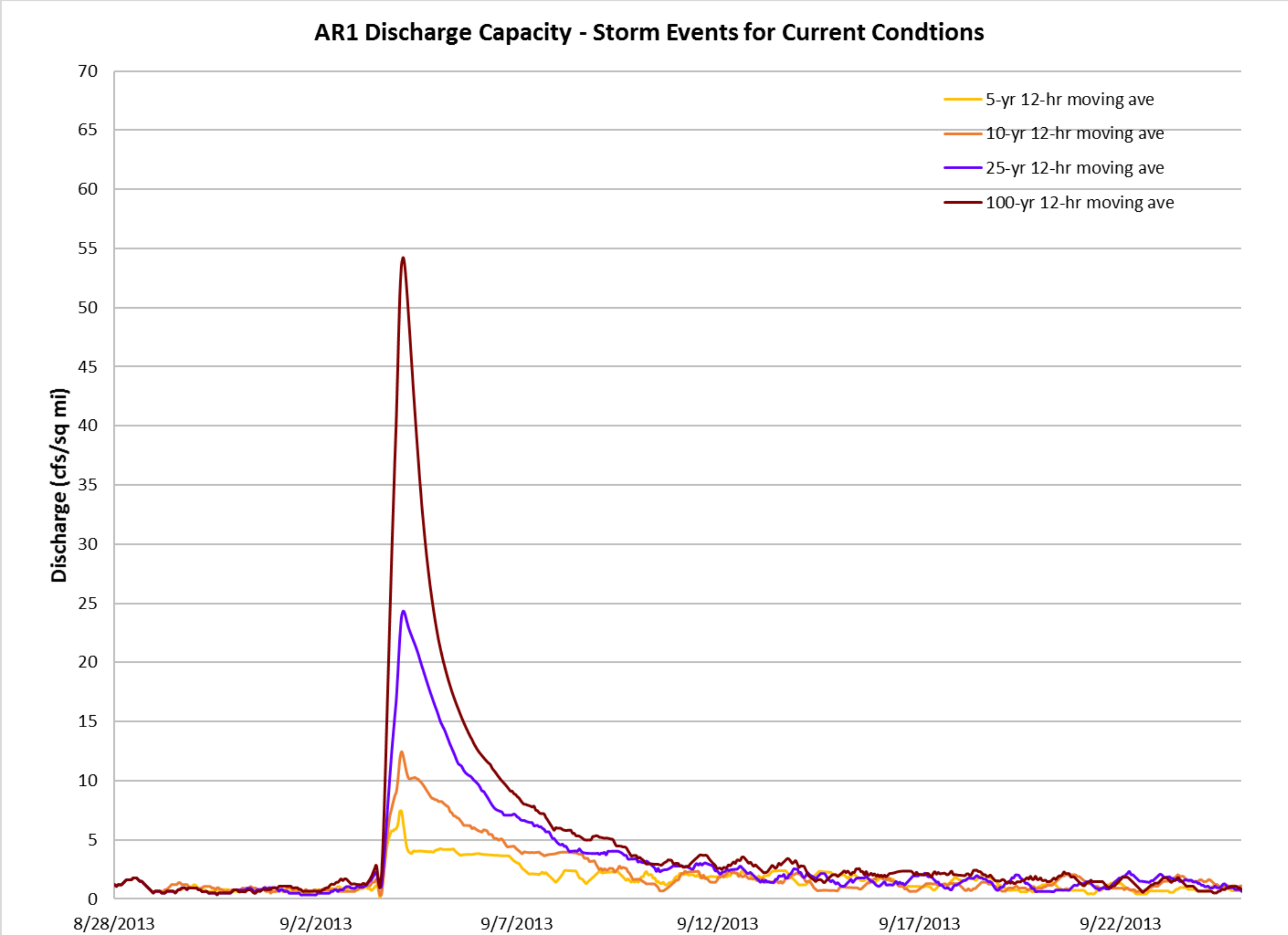

Figure 4.2.9 – Net Area-Weighted Discharge Hydrograph for Airport Road Canal (North and South structures)

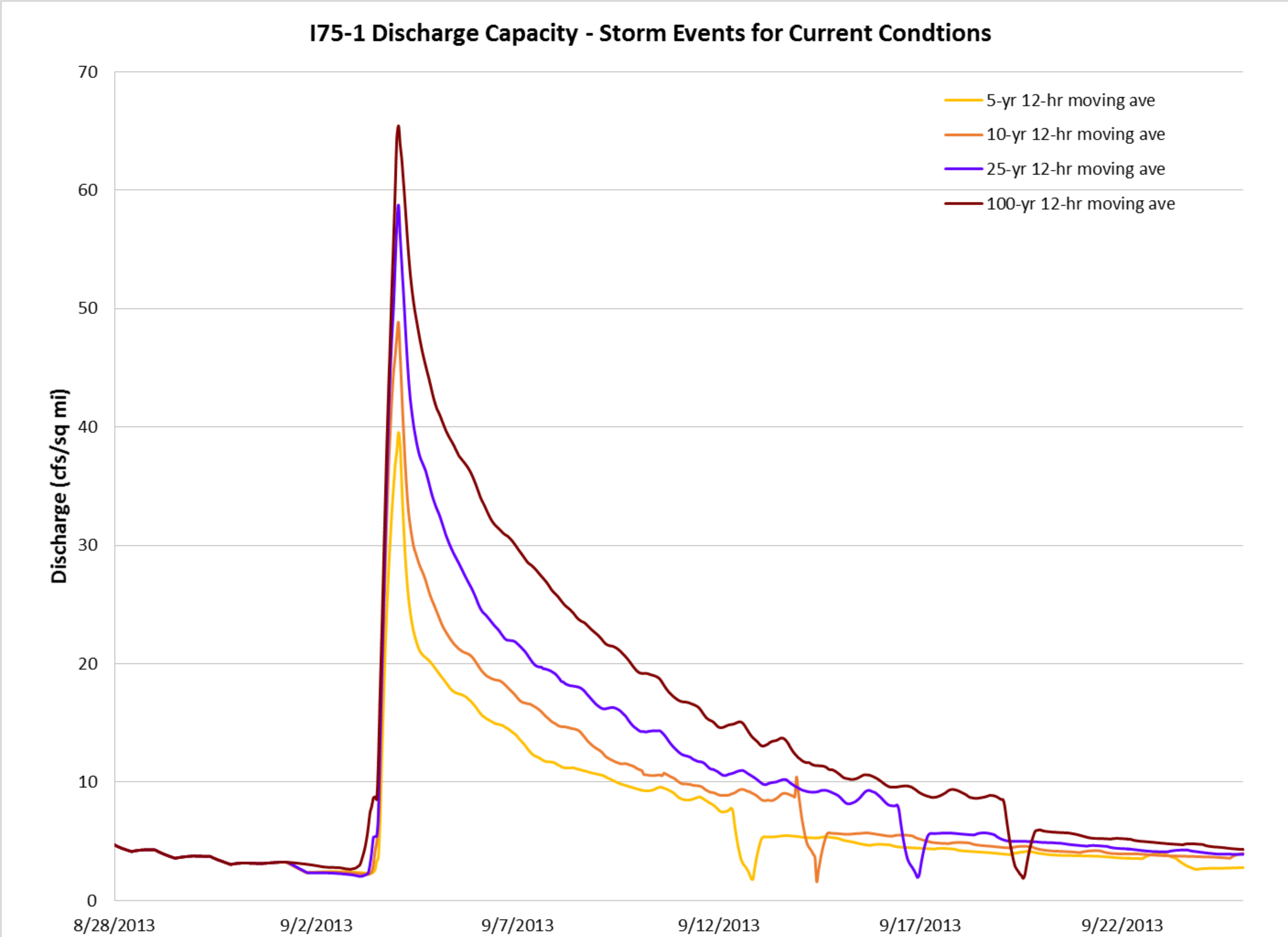

Figure 4.2.10 – Net Area-Weighted Discharge Hydrograph for I75-1

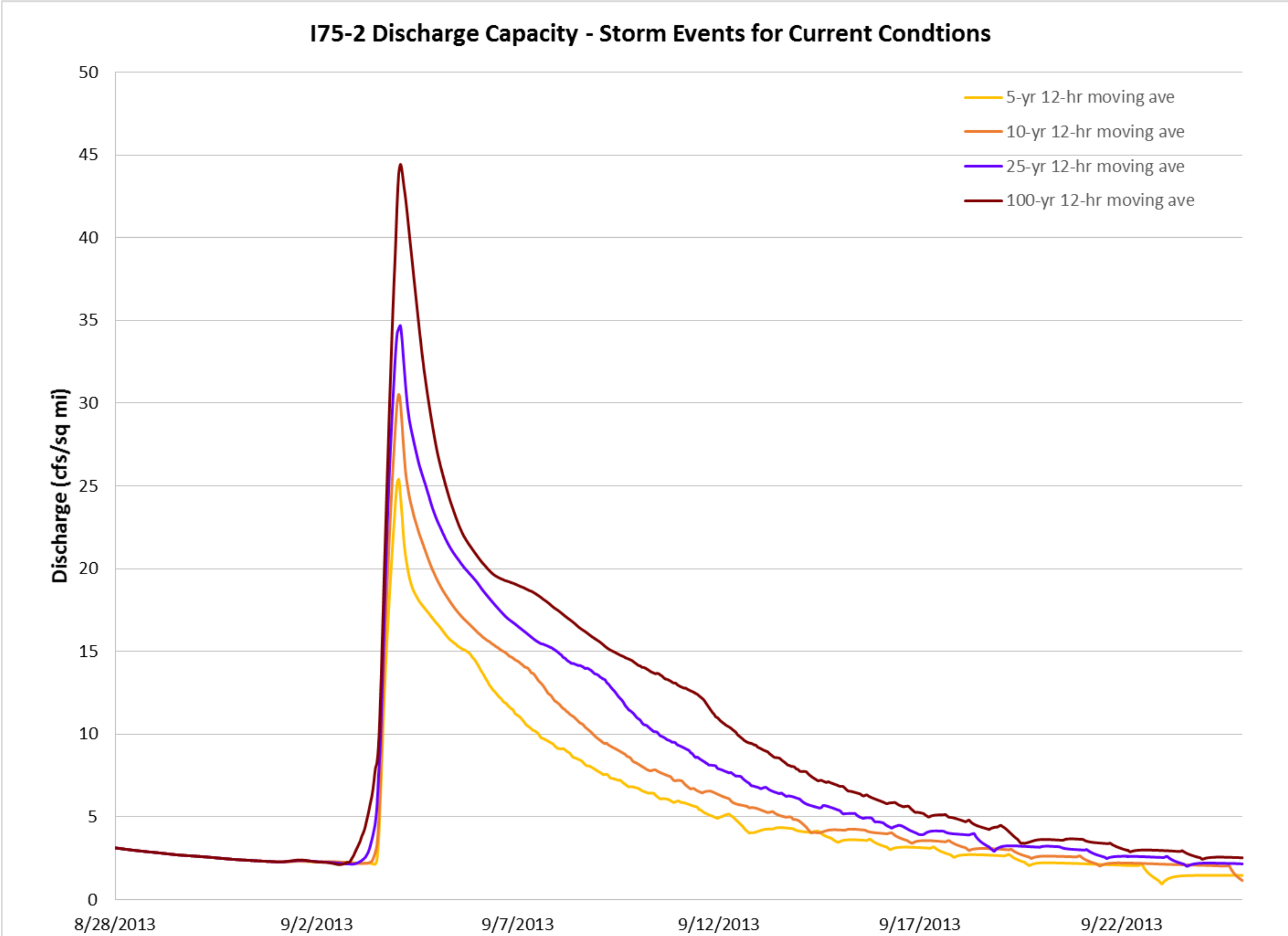

Figure 4.2.11 – Net Area-Weighted Discharge Hydrograph for I75-2

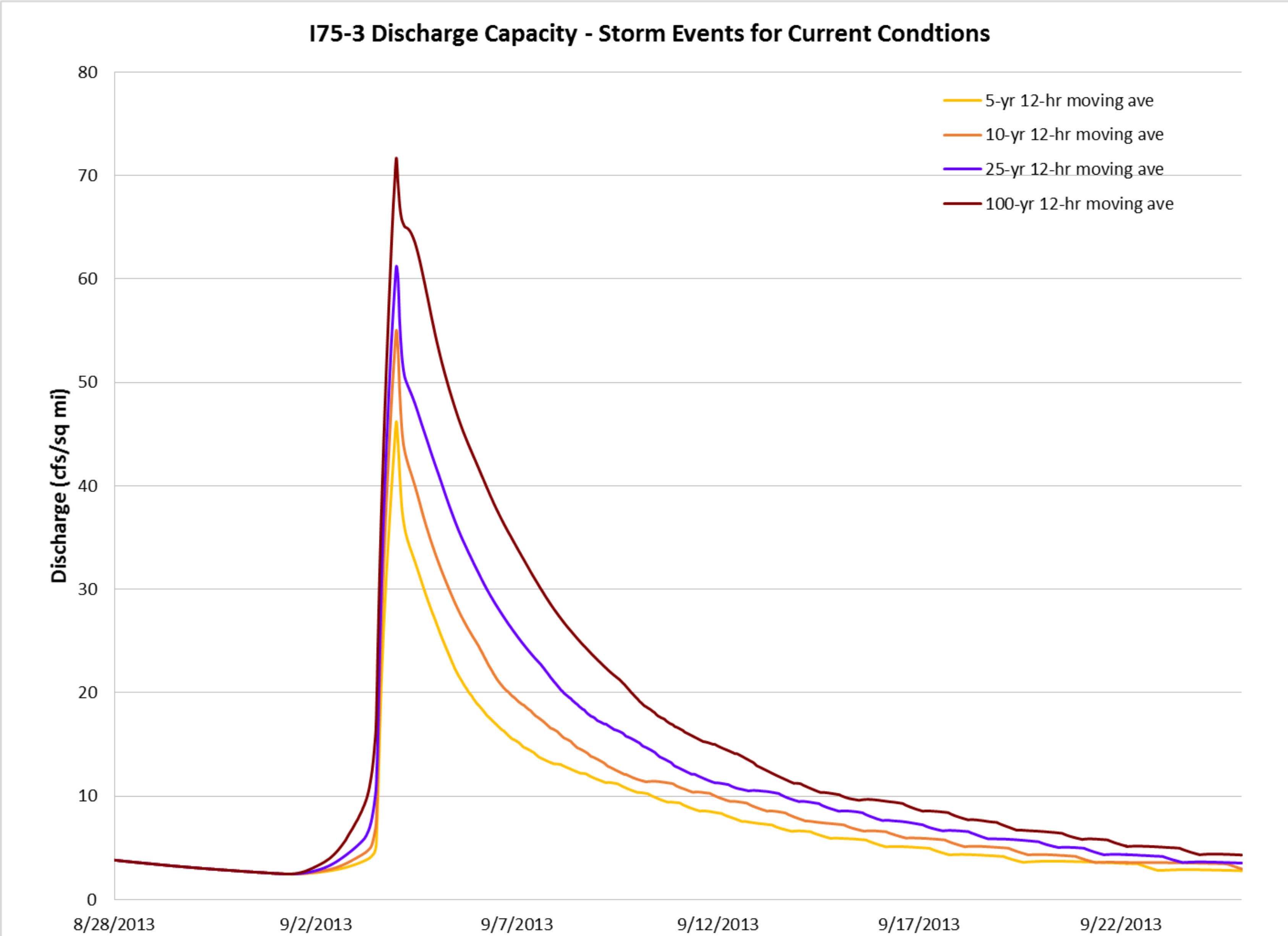

Figure 4.2.12 – Net Area-Weighted Discharge Hydrograph for I75-3

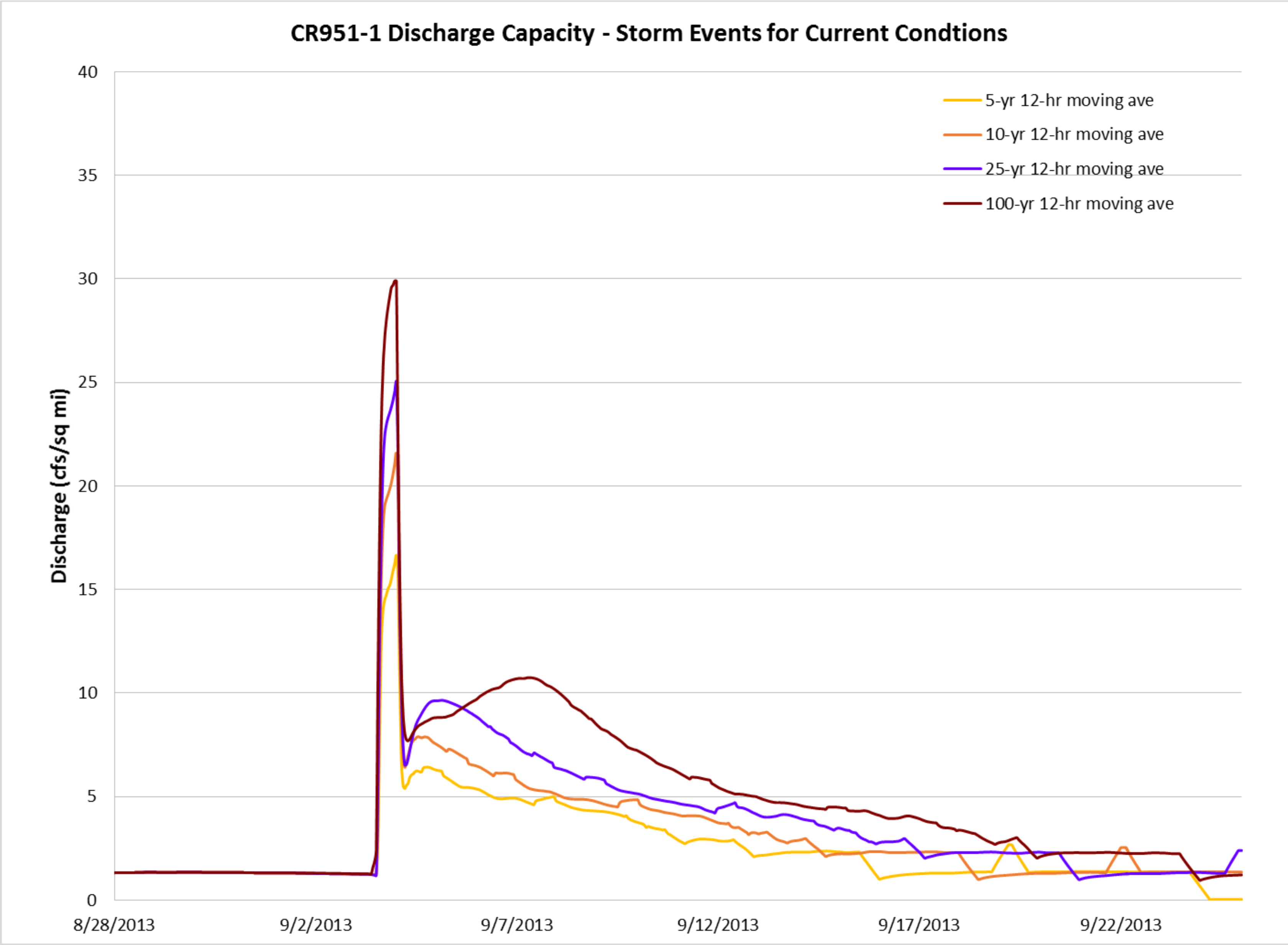

Figure 4.2.13 – Net Area-Weighted Discharge Hydrograph for CR951-1

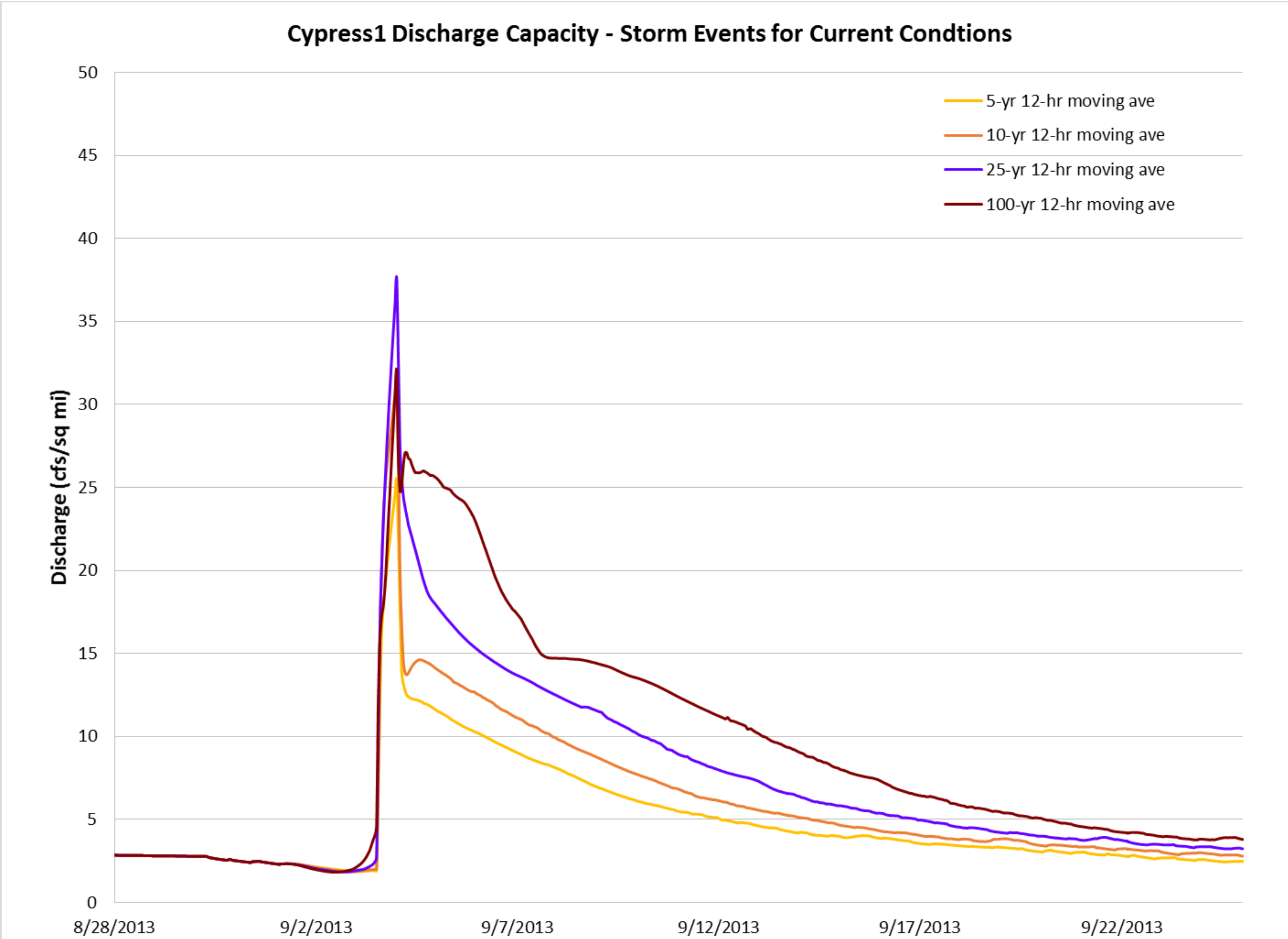

Figure 4.2.14 – Net Area-Weighted Discharge Hydrograph for Cypress1

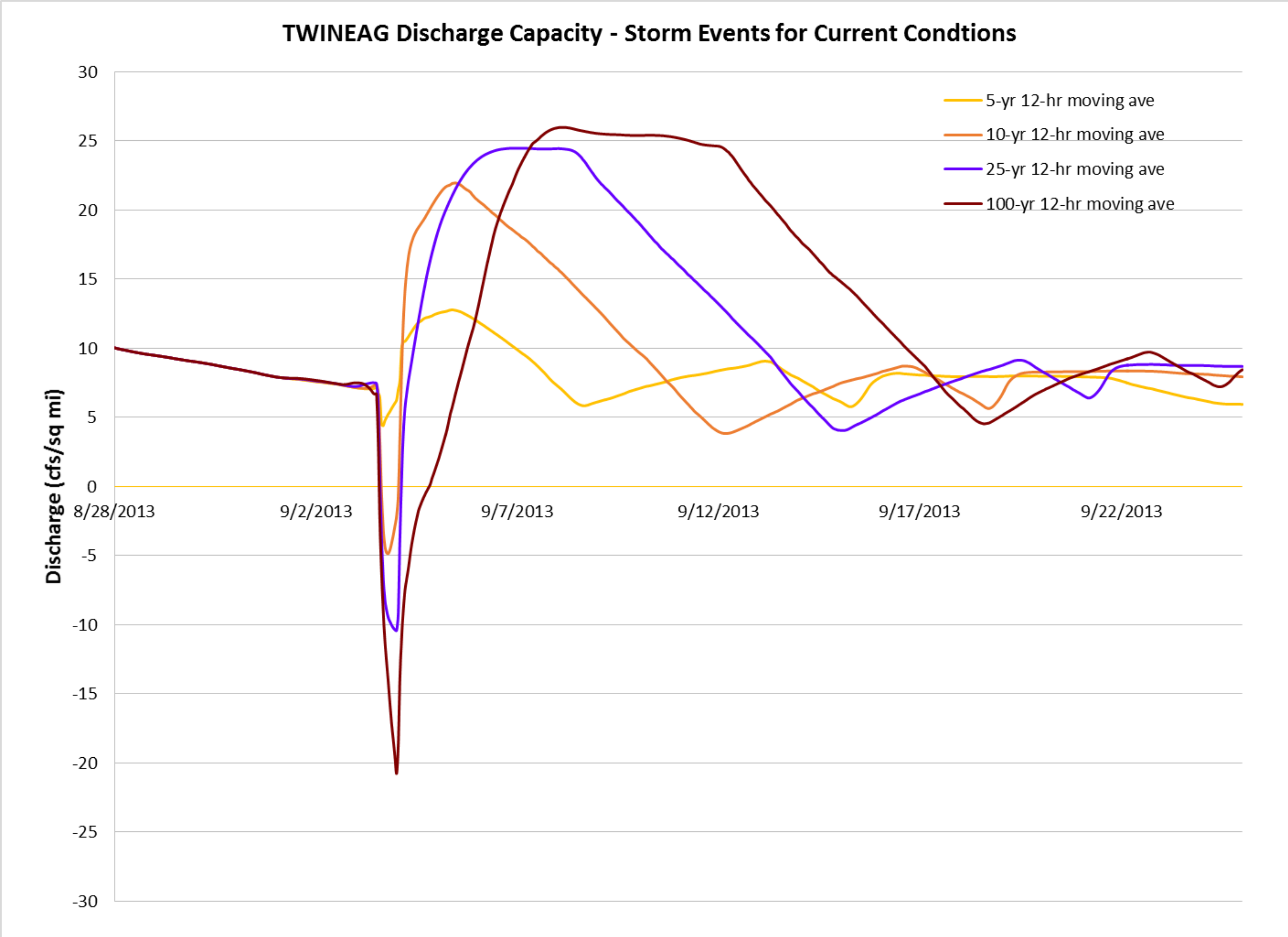

Figure 4.2.15 – Net Area-Weighted Discharge Hydrograph for Twin Eagles Structure

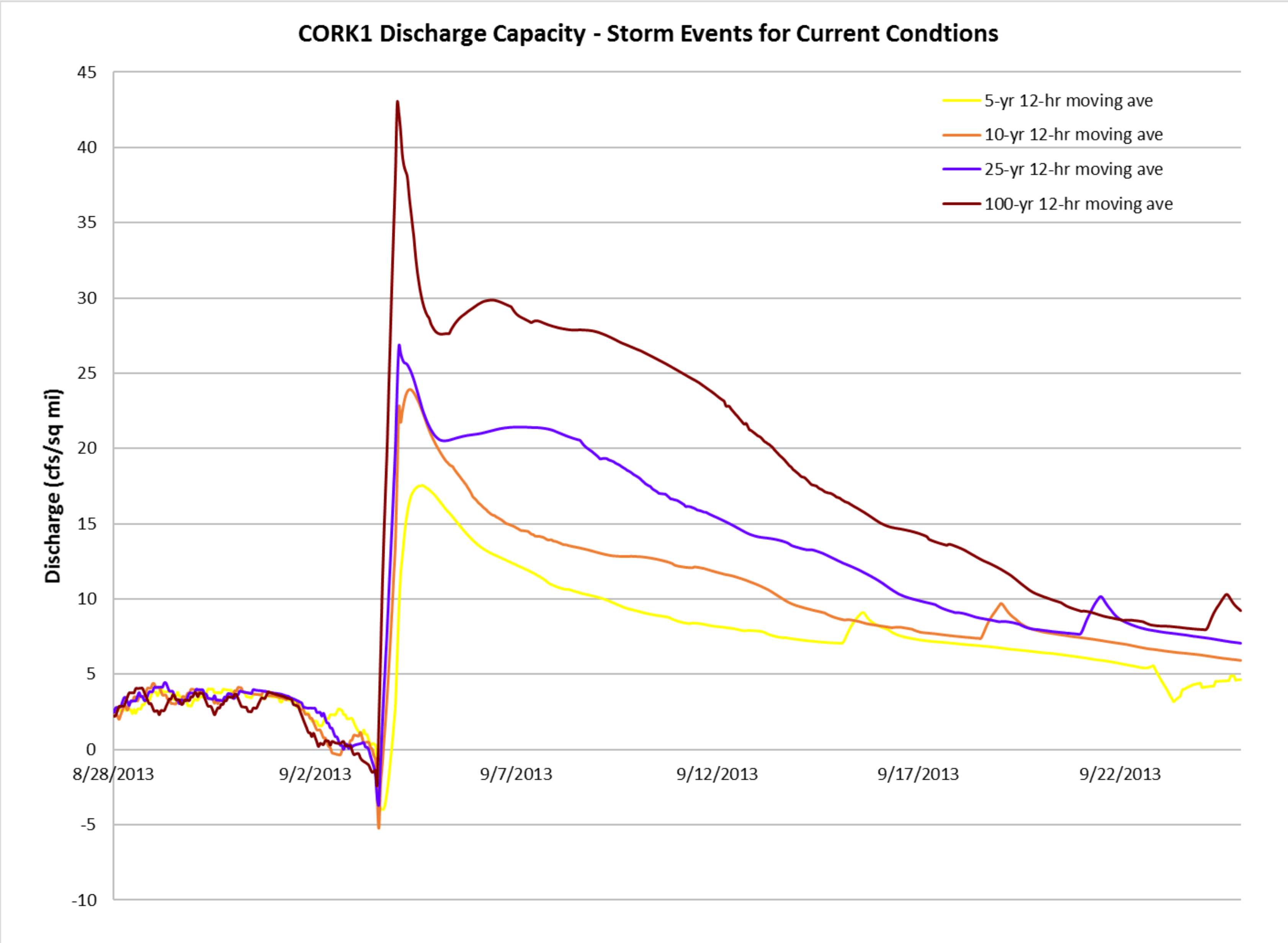

Figure 4.2.16 – Net Area-Weighted Discharge Hydrograph for CORK 1

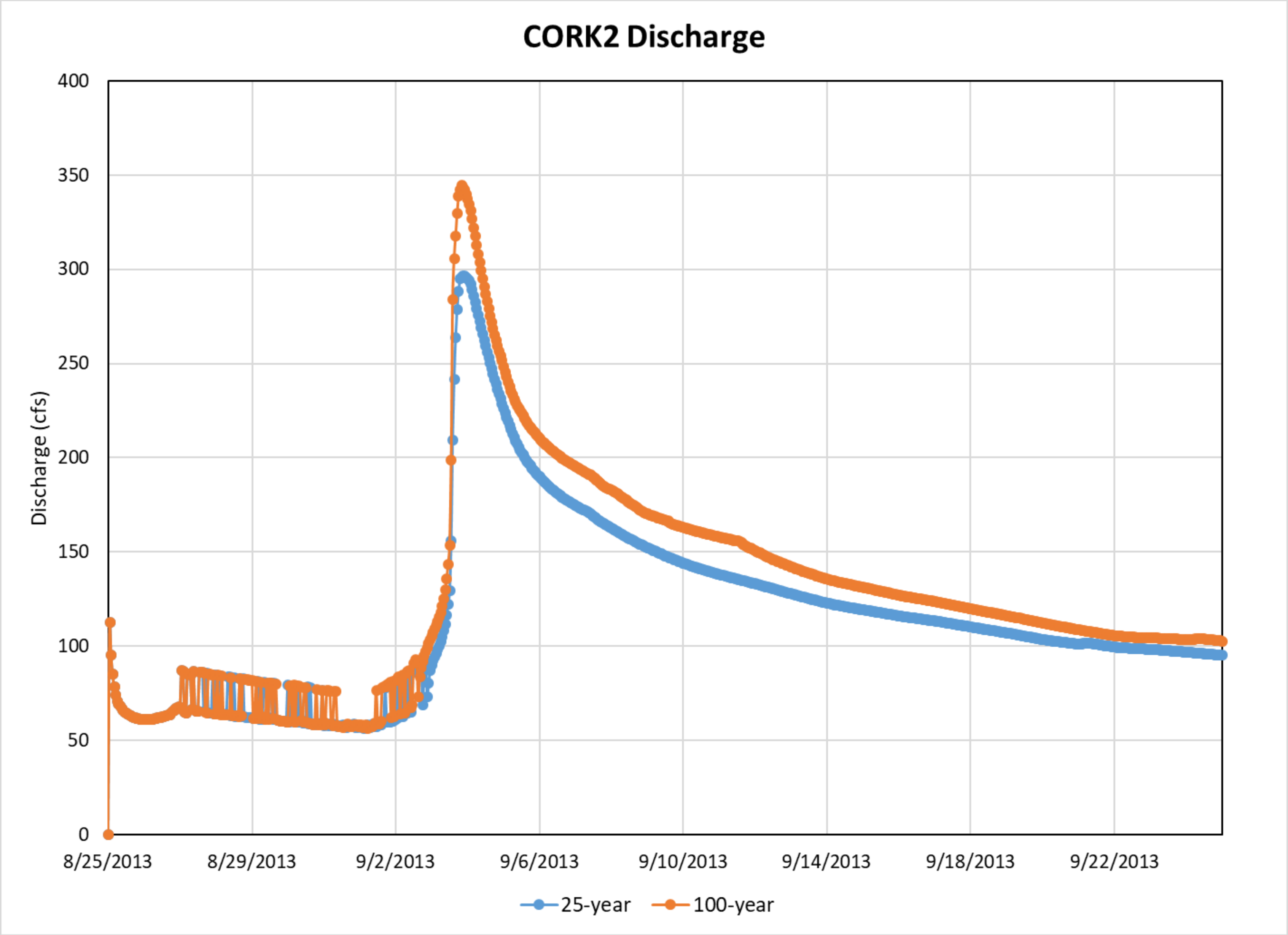

Figure 4.2.17 – 25-Year and 100-Year Discharge Hydrographs for CORK 2

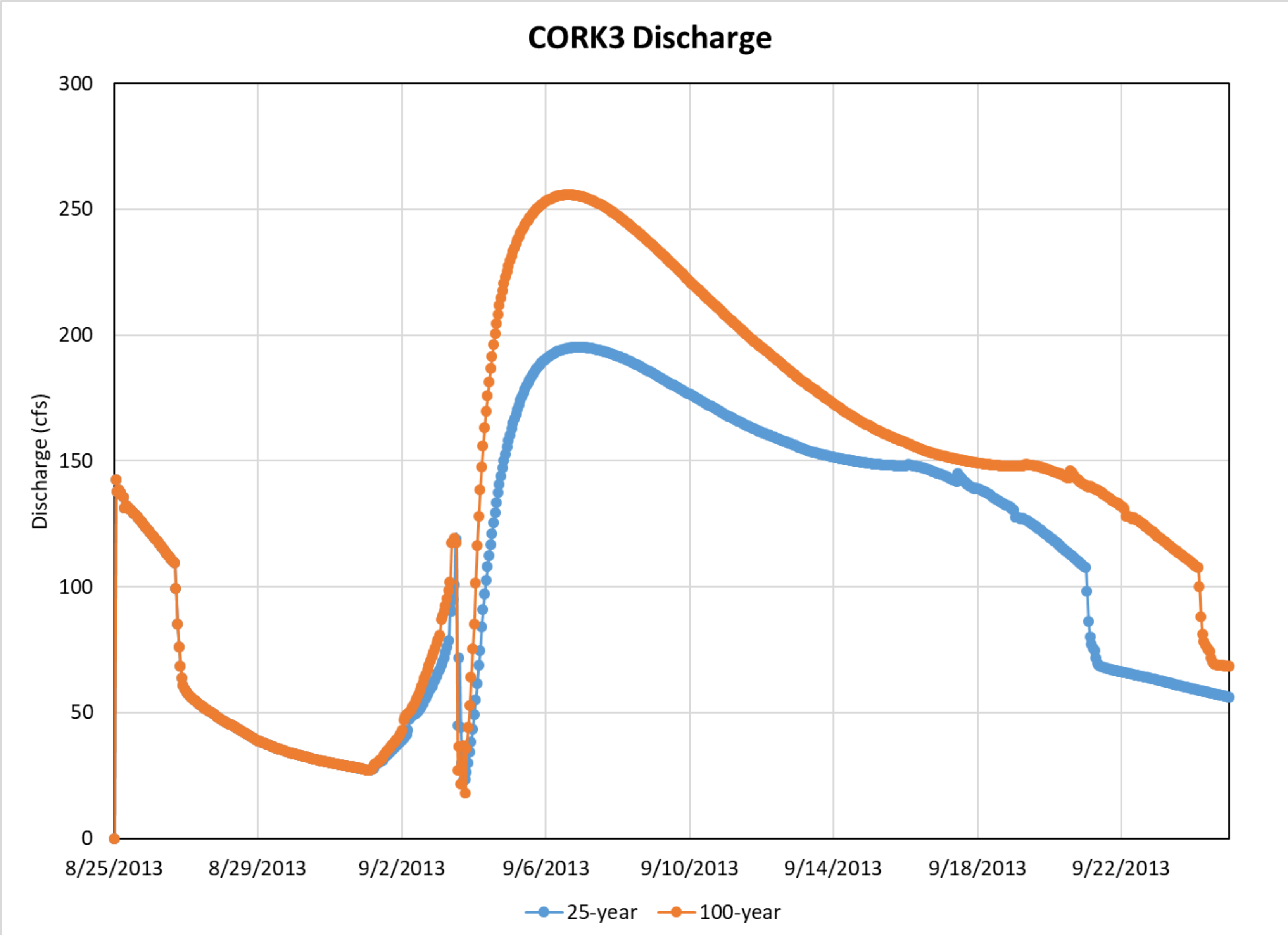

Figure 4.2.18 – 25-Year and 100-Year Discharge Hydrographs for CORK 3

4.3 PM #5 – Frequency of Flooding

In this PM, the flood elevations or depths of overland flooding are evaluated for a range of design storms (5-year, 10-year, 25-year, and 100-year). These flood depths/elevations can then be compared with elevations of features such as buildings and roadways, where such information exists. For the purposes of this BCB FPLOS evaluation, flood inundation maps were developed from the gridded MIKE SHE model output for each storm event, in the form of depth of overland water. These overland water depths, with the 500-foot model grid spacing, were converted into a TIN for presentation purposes. The resulting flood inundation maps are presented on **Figures 4.3.1 through 4.3.4** for each of the four design storm events.

Generally speaking, many of the newer developments (i.e., those with modern stormwater management systems) within the Golden Gate Watershed were predicted to remain relatively flood-free for all storm events. Notable exceptions to this include the Island Walk subdivision (northeast of the intersection of Logan Blvd N. and Vanderbilt Beach Road) and the area around Barron Collier High School (north of Pine Ridge Road and East of Airport Pulling Road). Conversely, Golden Gate Estates was predicted to be mostly inundated for the 100-year storm event with flood depths ranging from 0.25 feet to 2.5 feet. The most severely flooded areas within Golden Gate Estates were predicted to be the area north of Immokalee Road adjacent to the Bird Rookery Swamp, and a broad swath of the development on either side of Golden Gate Blvd West (**Figure 4.3.4**).

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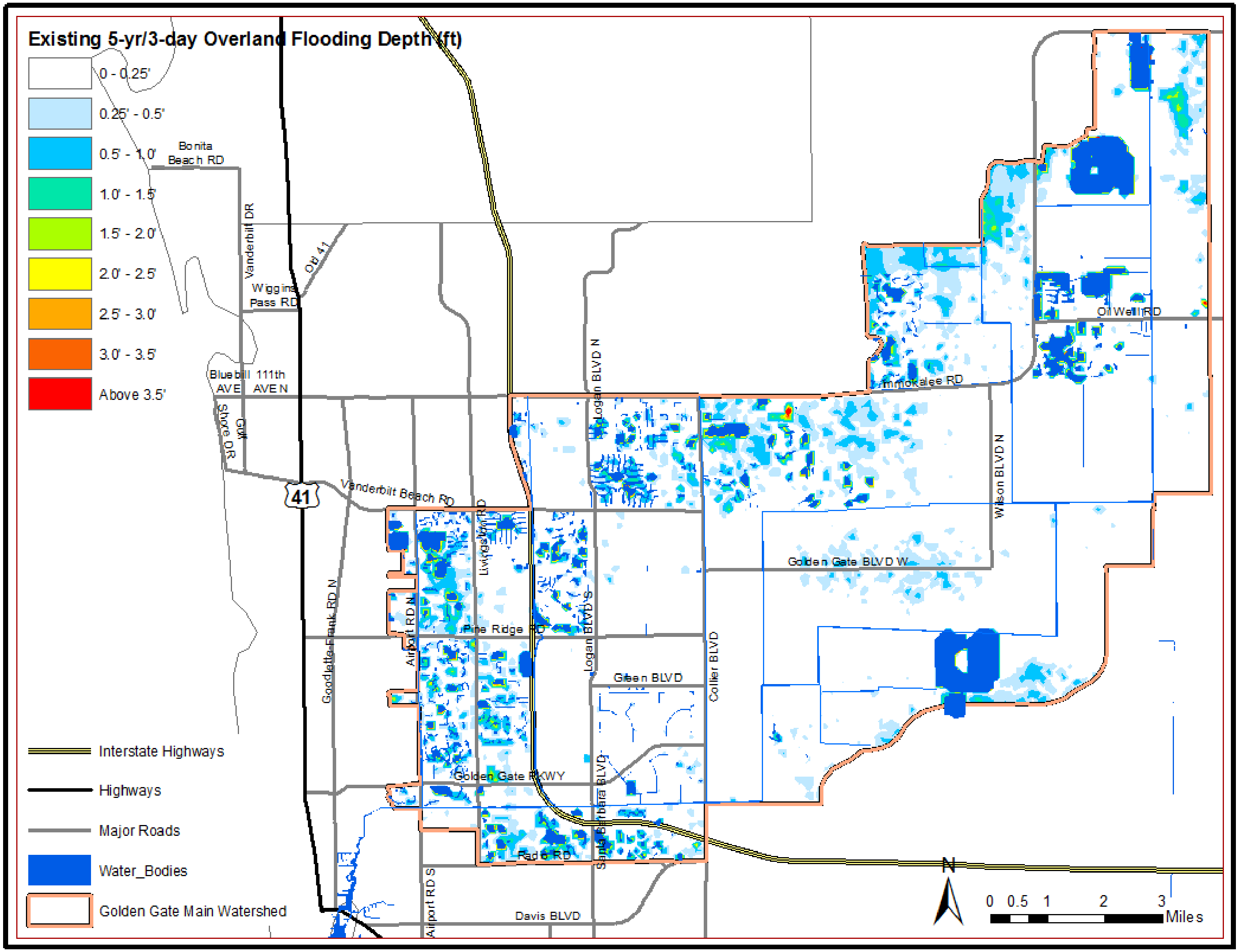


Figure 4.3.1 – Inundation Map for 5-year Design Storm Event, Golden Gate Watershed

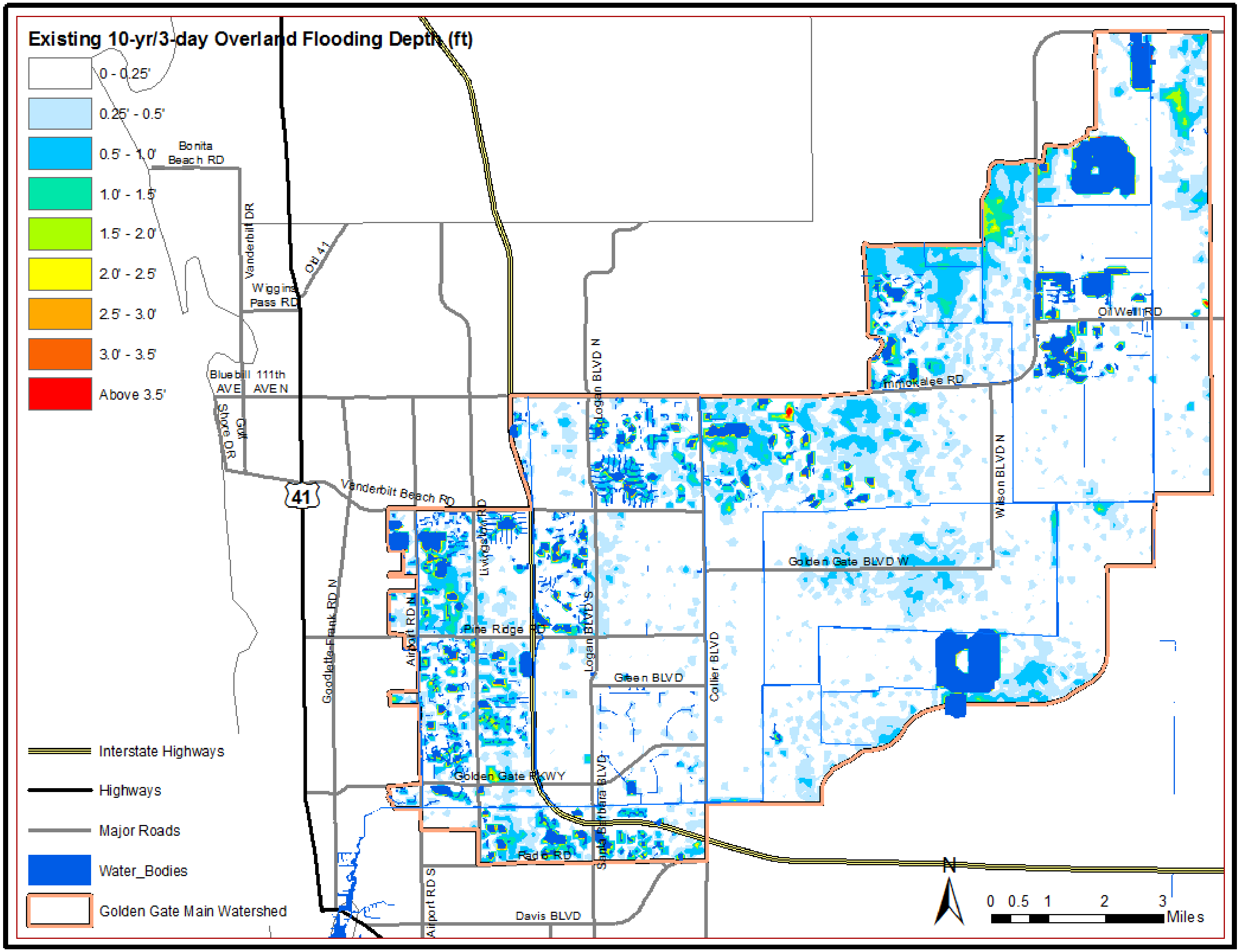


Figure 4.3.2 – Inundation Map for 10-year Design Storm Event, Golden Gate Watershed

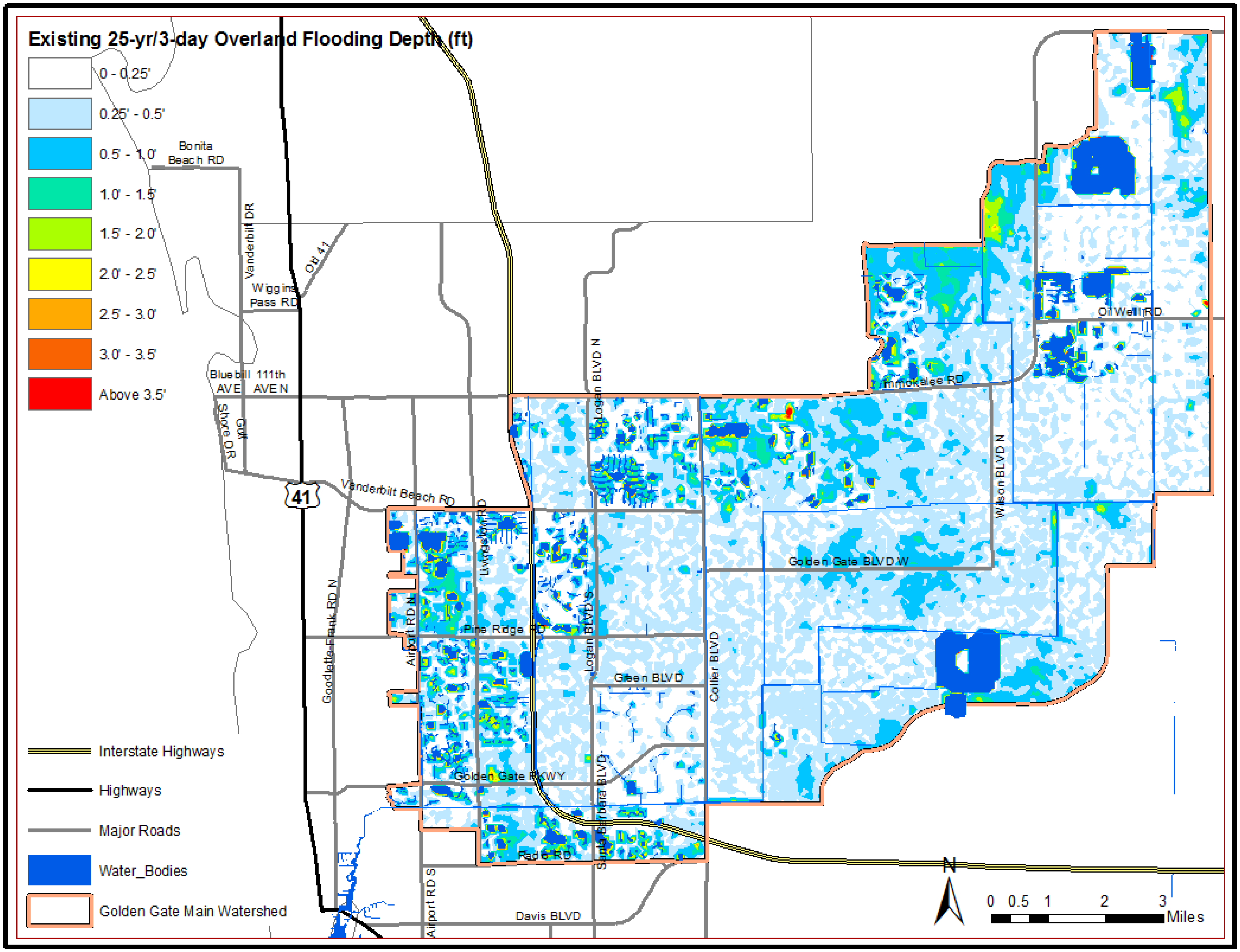


Figure 4.3.3 – Inundation Map for 25-year Design Storm Event, Golden Gate Watershed

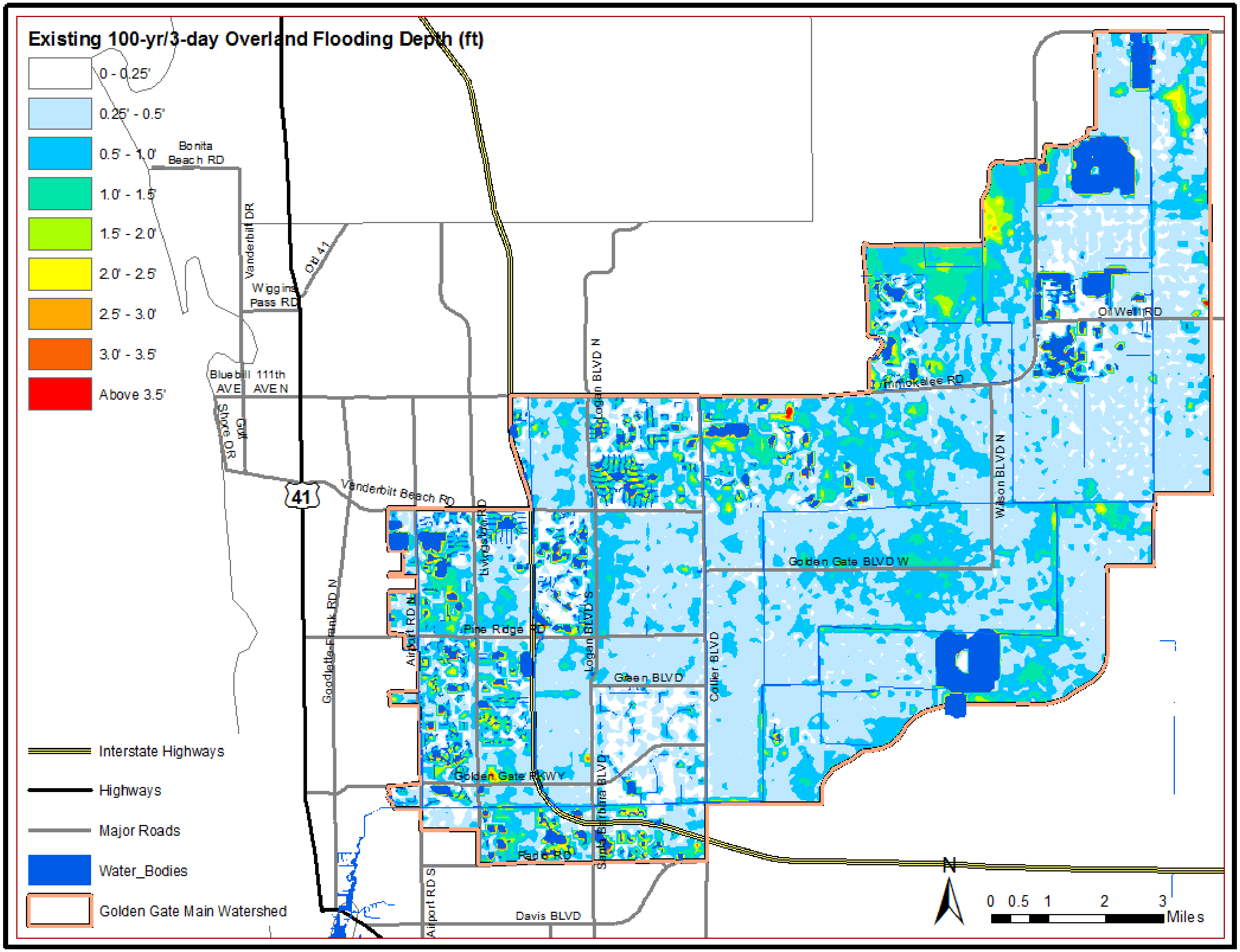


Figure 4.3.4 – Inundation Map for 100-year Design Storm Event, Golden Gate Watershed

4.4 PM #6 – Duration of Flooding

PM #6 quantifies the duration of flooding at specific locations of interest within a watershed. For this metric, potentially flood-prone locations associated with existing or future development are identified, and a representative threshold elevation corresponding to building or roadway flooding is chosen. The length of time the flood elevation is projected to be above that elevation is the duration of flooding. This metric requires the selection of specific areas of interest. For the Golden Gate Watershed, six areas of interest were identified in consultation with District staff. These locations are shown on **Figure 4.4.1**.

At each location of interest it was necessary to identify a target or threshold stage which, when exceeded by water levels in the nearby canal, would cause flooding of streets or buildings. Building pads within the watershed are generally higher than the adjacent roadways, so the target stages represent road and street flooding thresholds. To develop the target stage, several (typically 4) representative road and street locations were identified within the area of interest, and the pavement elevations (estimated from the LiDAR data) were averaged to develop the representative target elevation for each of the six locations. Stage hydrographs from the MIKE-11 output for each storm event were then compared with the target stages, to compute the duration of time target stages were exceeded. The results of the comparisons are depicted graphically on **Figures 4.4.2** through **4.4.25**.

From the figures, it is evident that location 1 is predicted to be flooded for all storm events. Location 1 is immediately upstream of the GG-1 structure. At this location the topography is low and flooding results in part from the high tailwater condition, which at this location is impacted by storm surge. The simulated 100-year tailwater boundary condition at GG-1 is 6.9 feet NAVD88, which is about 2 feet above the surrounding topography. Predicted flood durations here range from 3.5 hours for the 5-year event to almost 4 days for the 100-year event. In contrast, streets and roads at locations 2 and 3 are above the simulated peak flood levels in the adjacent canals (Airport Road and I-75 Canals, respectively) for all storm events.

Location 4 (Golden Gate Estates east of the CR951 Canal) is predicted to have no flooding for the 5-year event, and only minor flooding (duration of 30 minutes) for the 10-year event. For the 25-year and 100-year events, the predicted durations of flooding at location 4 are 2 hours 1.5 days, respectively.

Location 5 is adjacent to the downstream portion of the Cypress Canal, and is also within Golden Gate Estates. No flooding is predicted here for the 5-year and 10-year storm events. However, significant flooding with durations of 2 hours and 2.25 days are predicted for the 25-year and 100-year events, respectively.

Location 6 is adjacent to the Cypress Canal near structure GG-4, in Golden Gate Estates. Because it is a topographically low area in the upstream reaches of the watershed, flooding is predicted to occur for all four design storm events. The predicted durations of flooding at Location 6 range from 1 hour for the 5-year event to 5 days for the 100-year event.

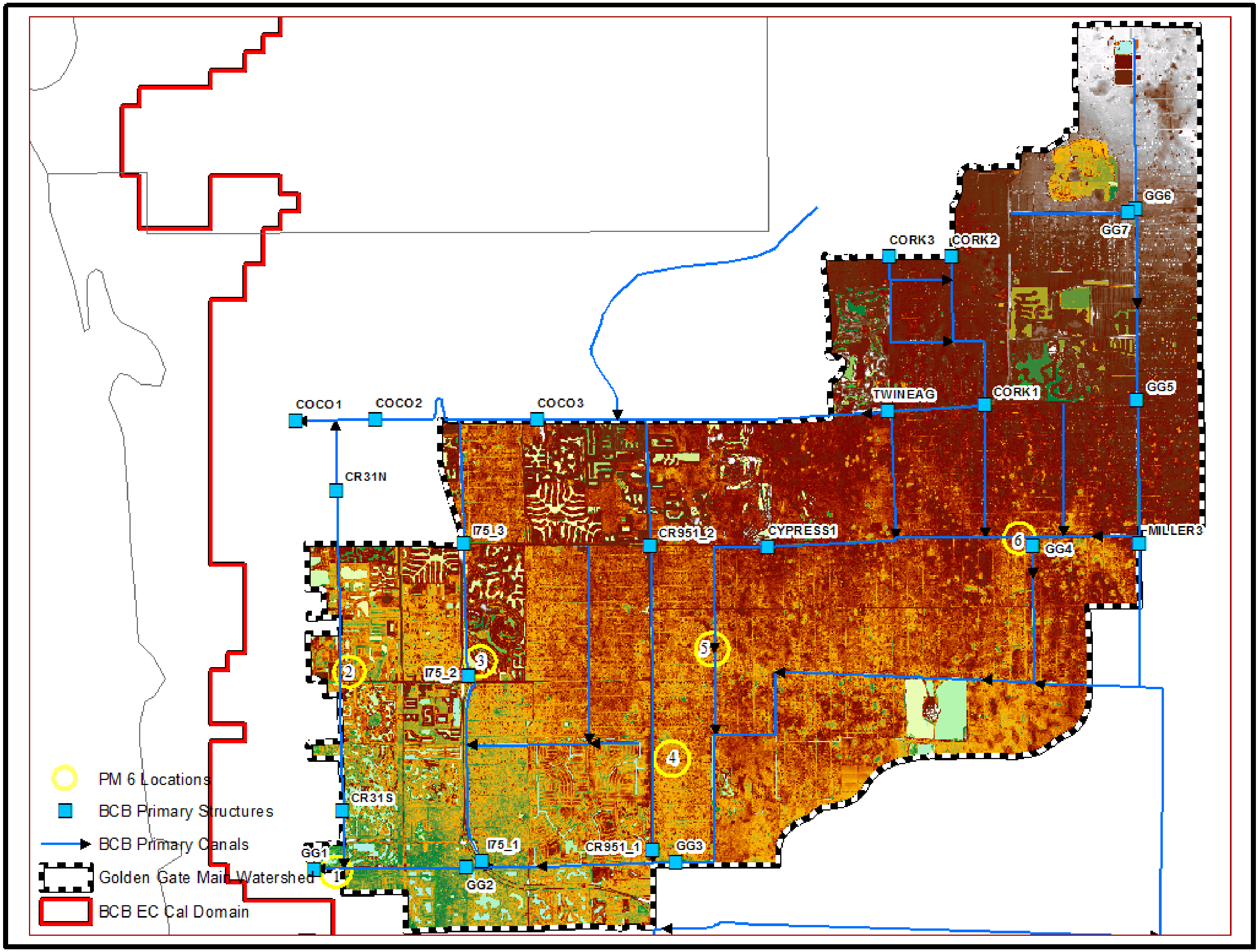


Figure 4.4.1 – Locations of Flood Duration Evaluation Points and LiDAR DEM

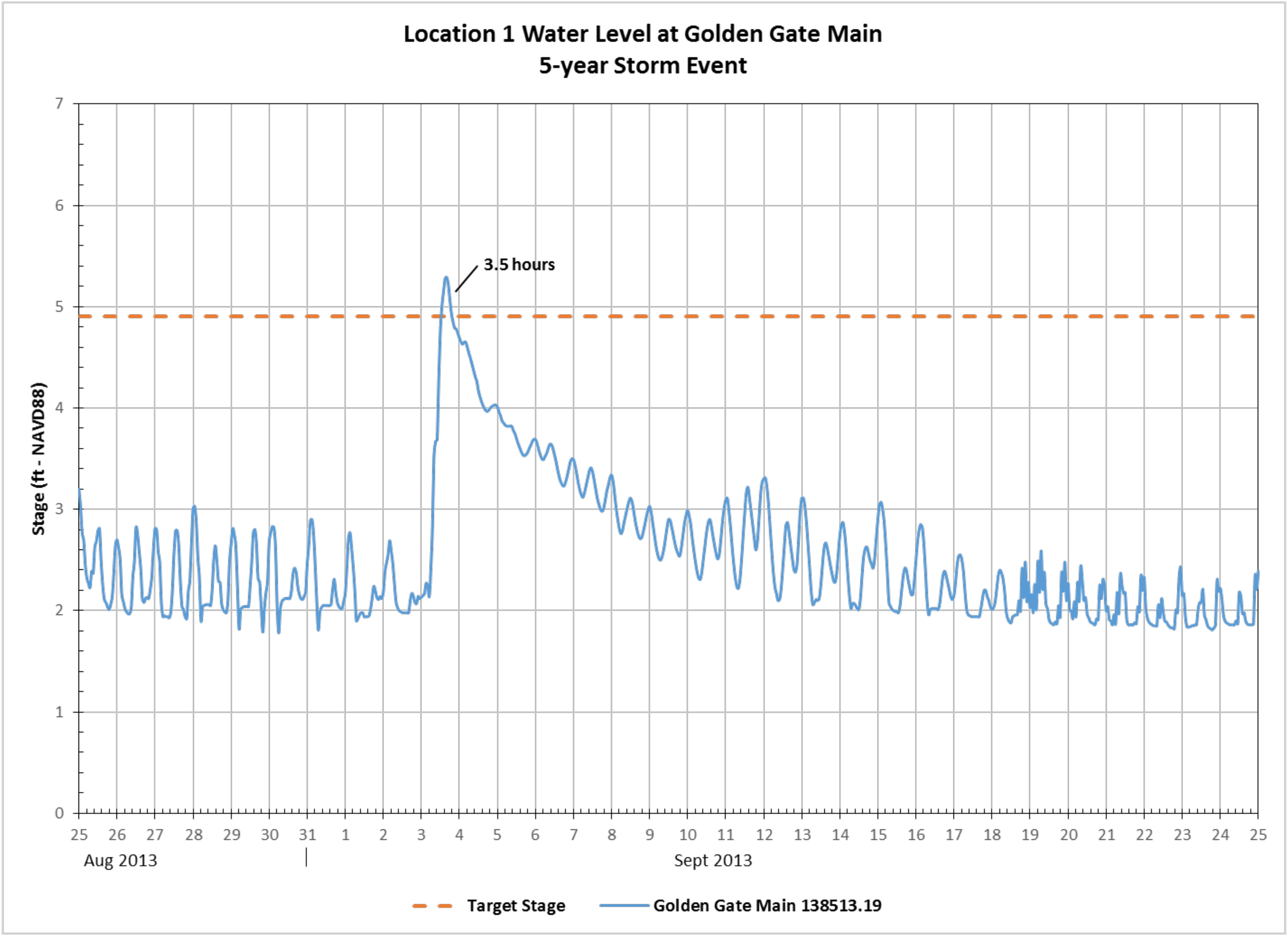


Figure 4.4.2 – Location 1 Duration of Flooding, 5-Year Storm

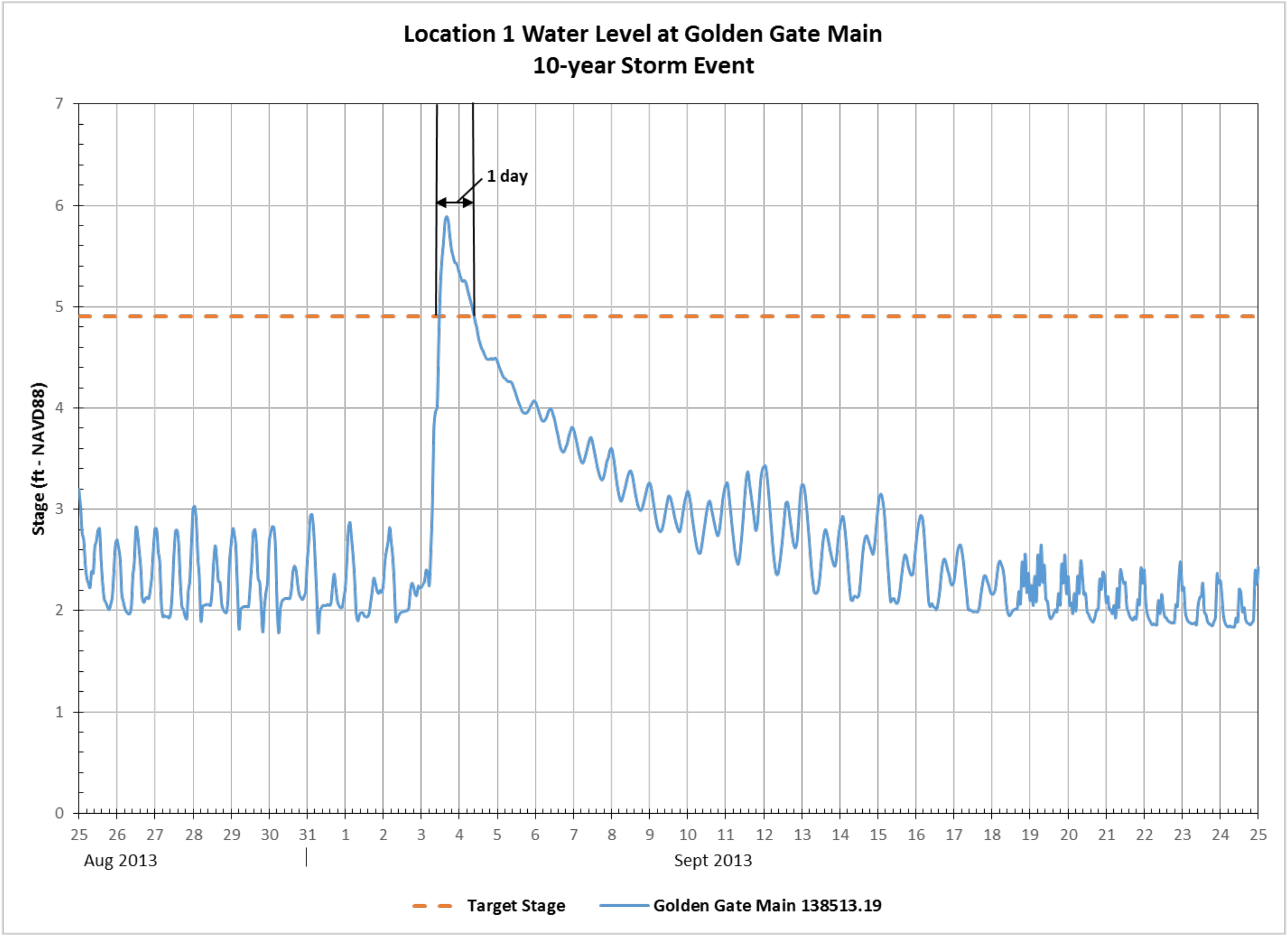


Figure 4.4.3 – Location 1 Duration of Flooding, 10-Year Storm

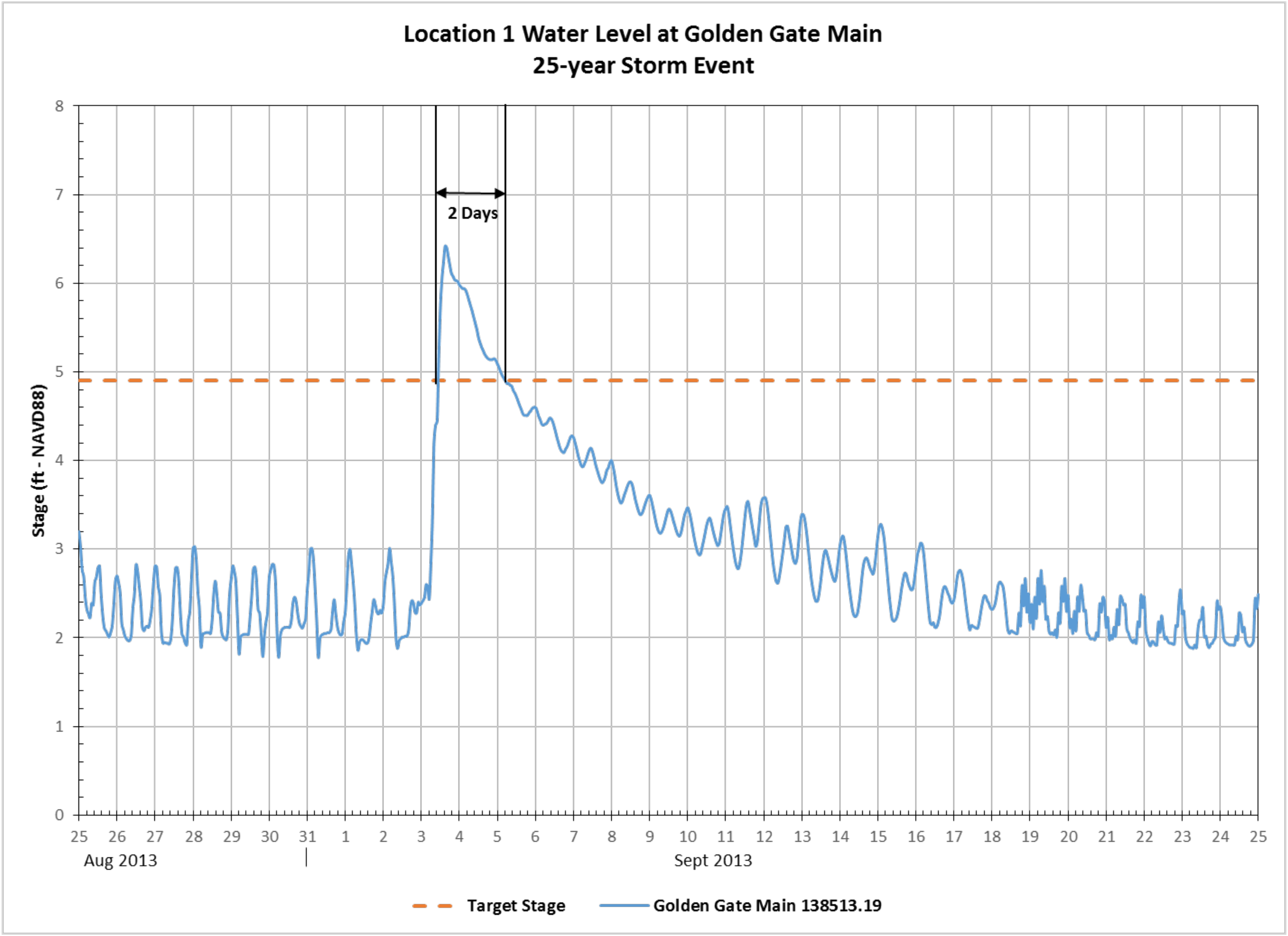


Figure 4.4.4 – Location 1 Duration of Flooding, 25-Year Storm

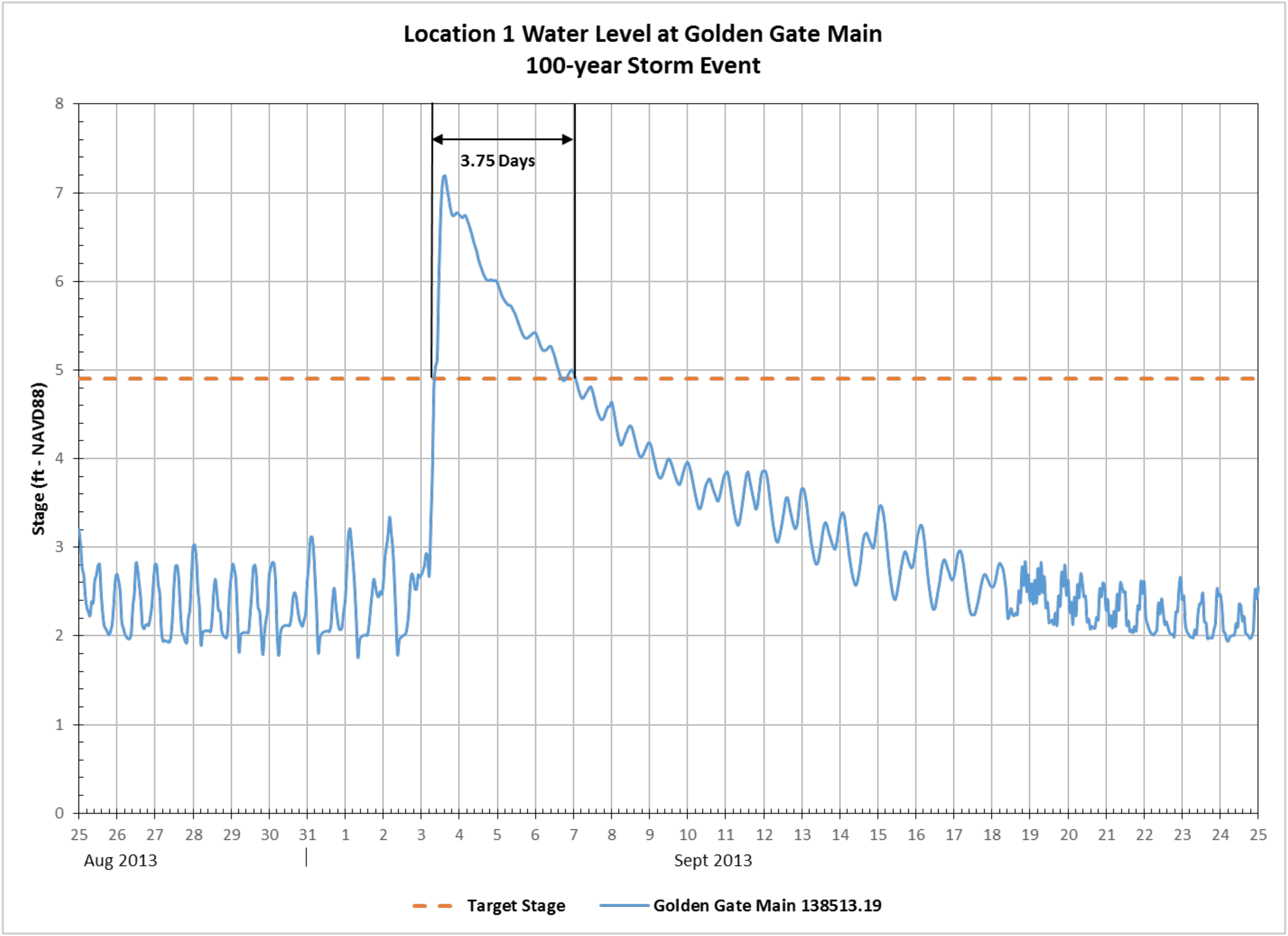


Figure 4.4.5 – Location 1 Duration of Flooding, 100-Year Storm

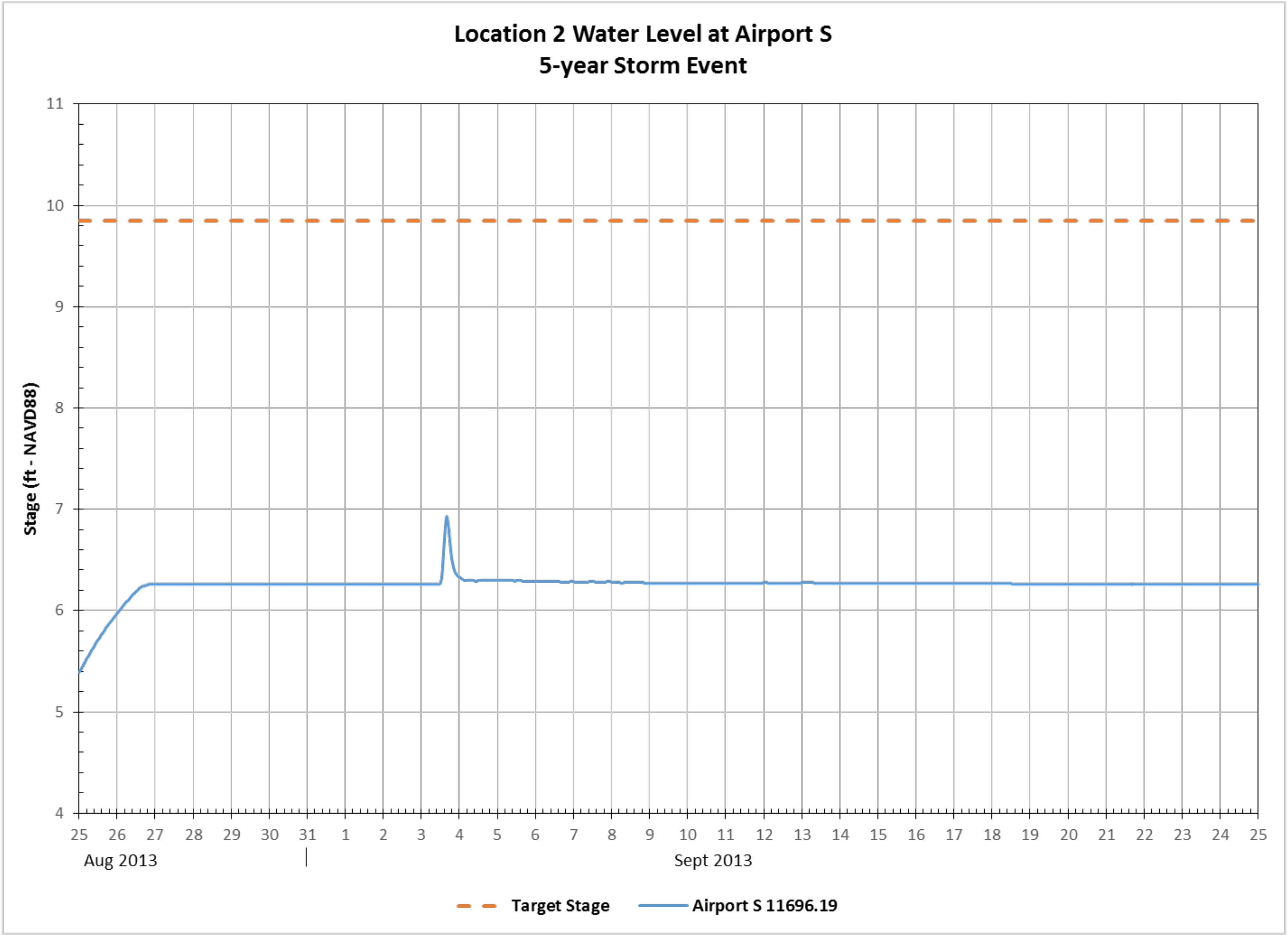


Figure 4.4.6 – Location 2 Duration of Flooding, 5-Year Storm

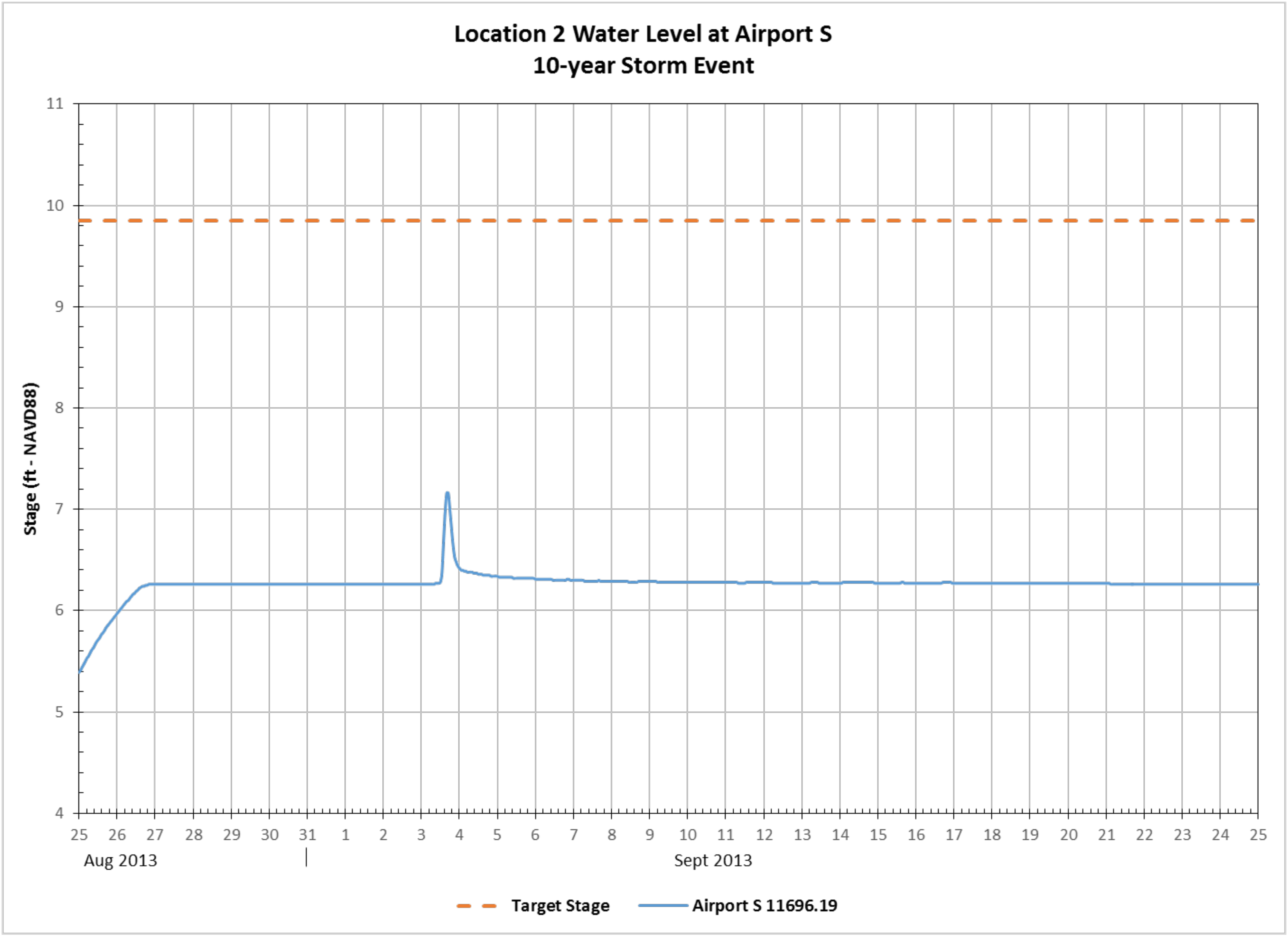


Figure 4.4.7 – Location 2 Duration of Flooding, 10-Year Storm

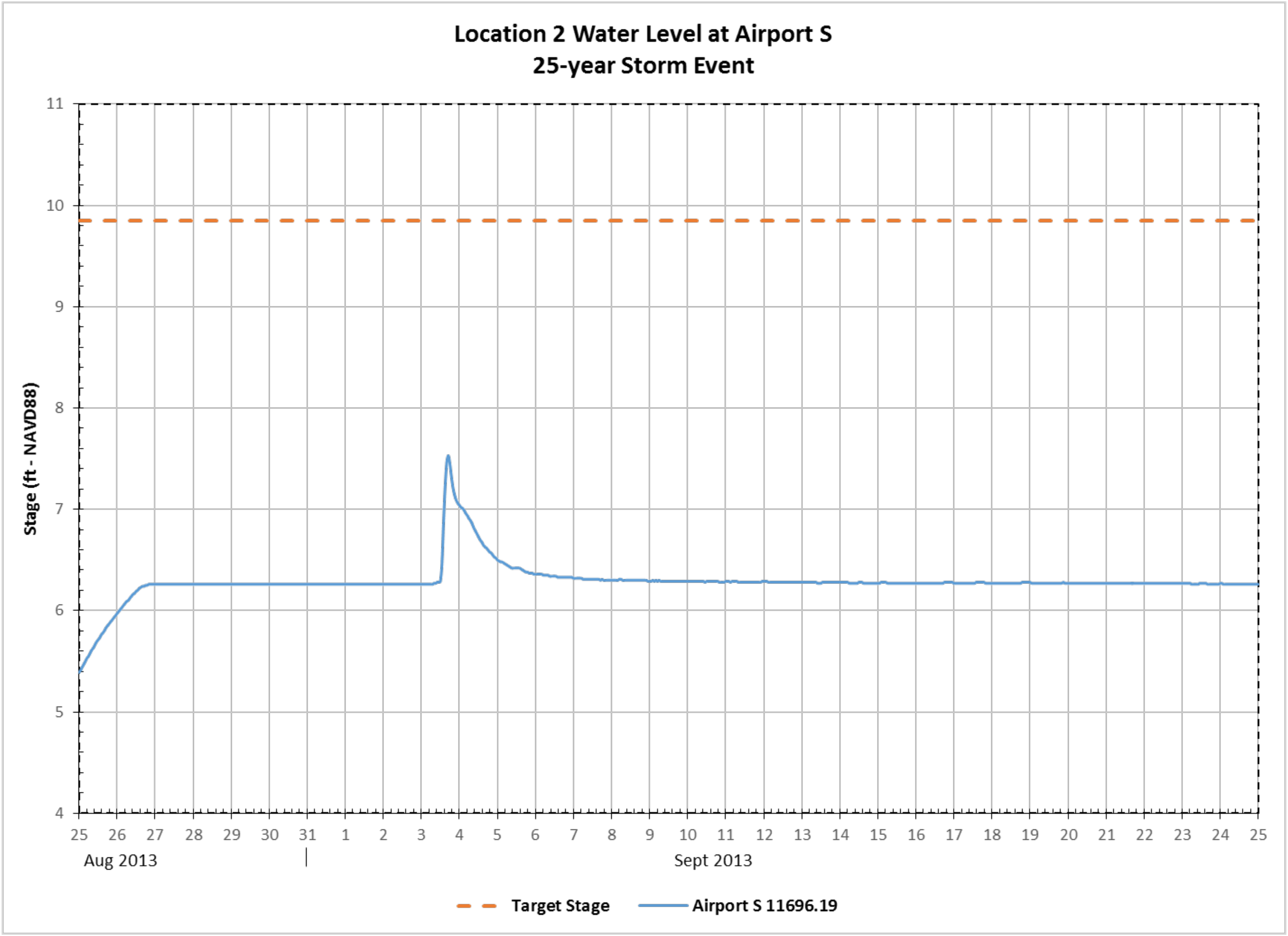


Figure 4.4.8 – Location 2 Duration of Flooding, 25-Year Storm

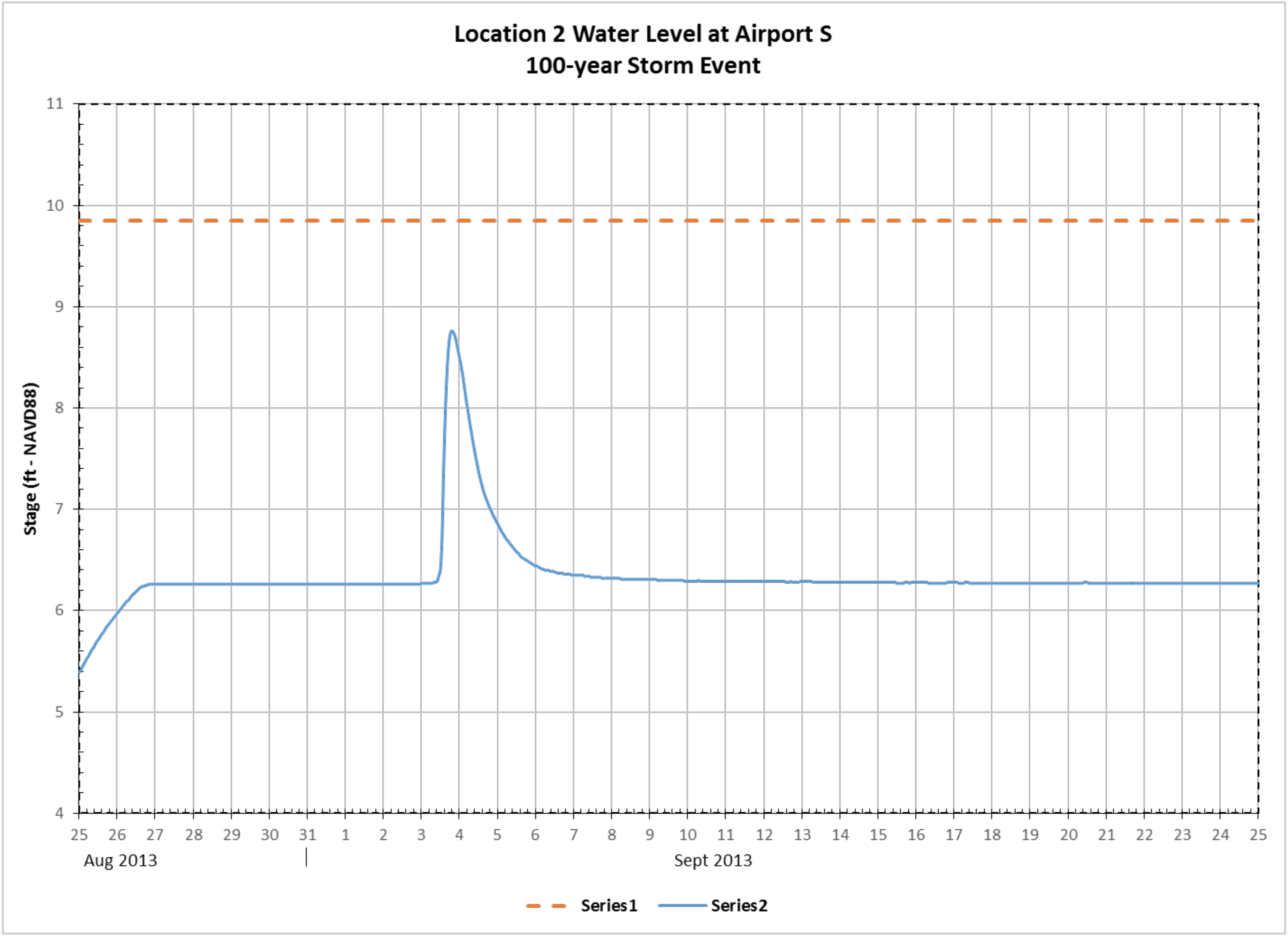


Figure 4.4.9 – Location 2 Duration of Flooding, 100-Year Storm

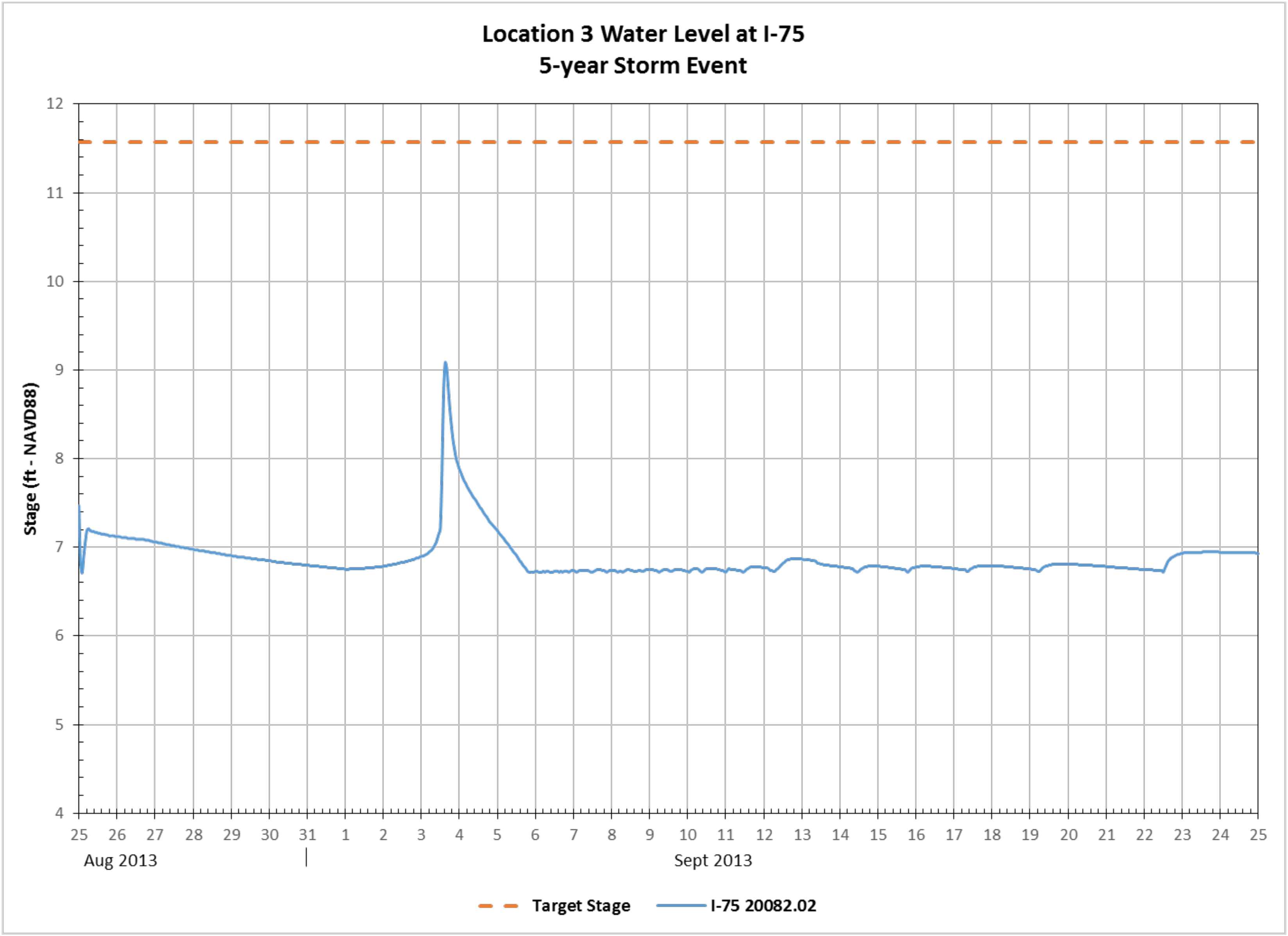


Figure 4.4.10 – Location 3 Duration of Flooding, 5-Year Storm

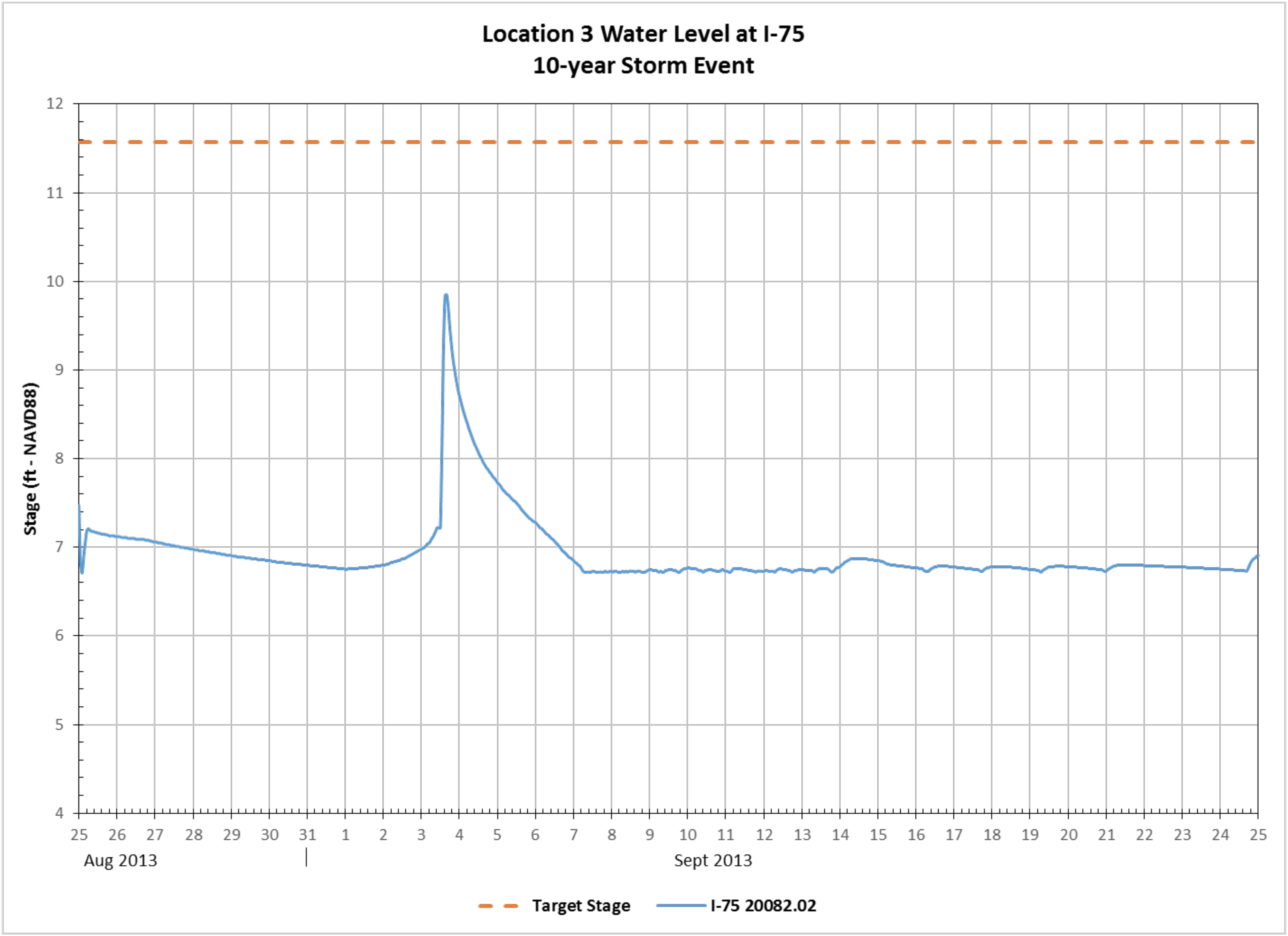


Figure 4.4.11 – Location 3 Duration of Flooding, 10-Year Storm

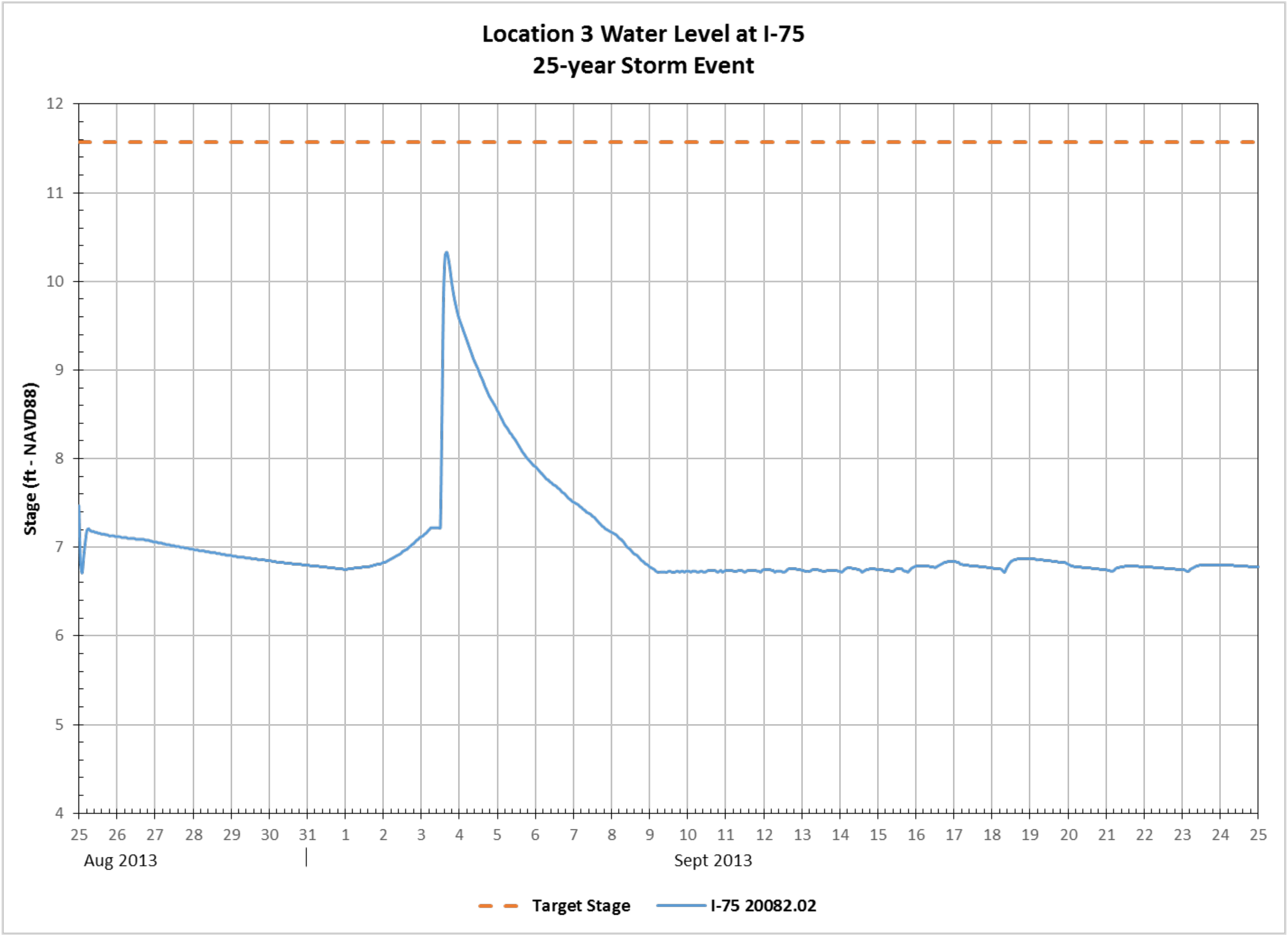


Figure 4.4.12 – Location 3 Duration of Flooding, 25-Year Storm

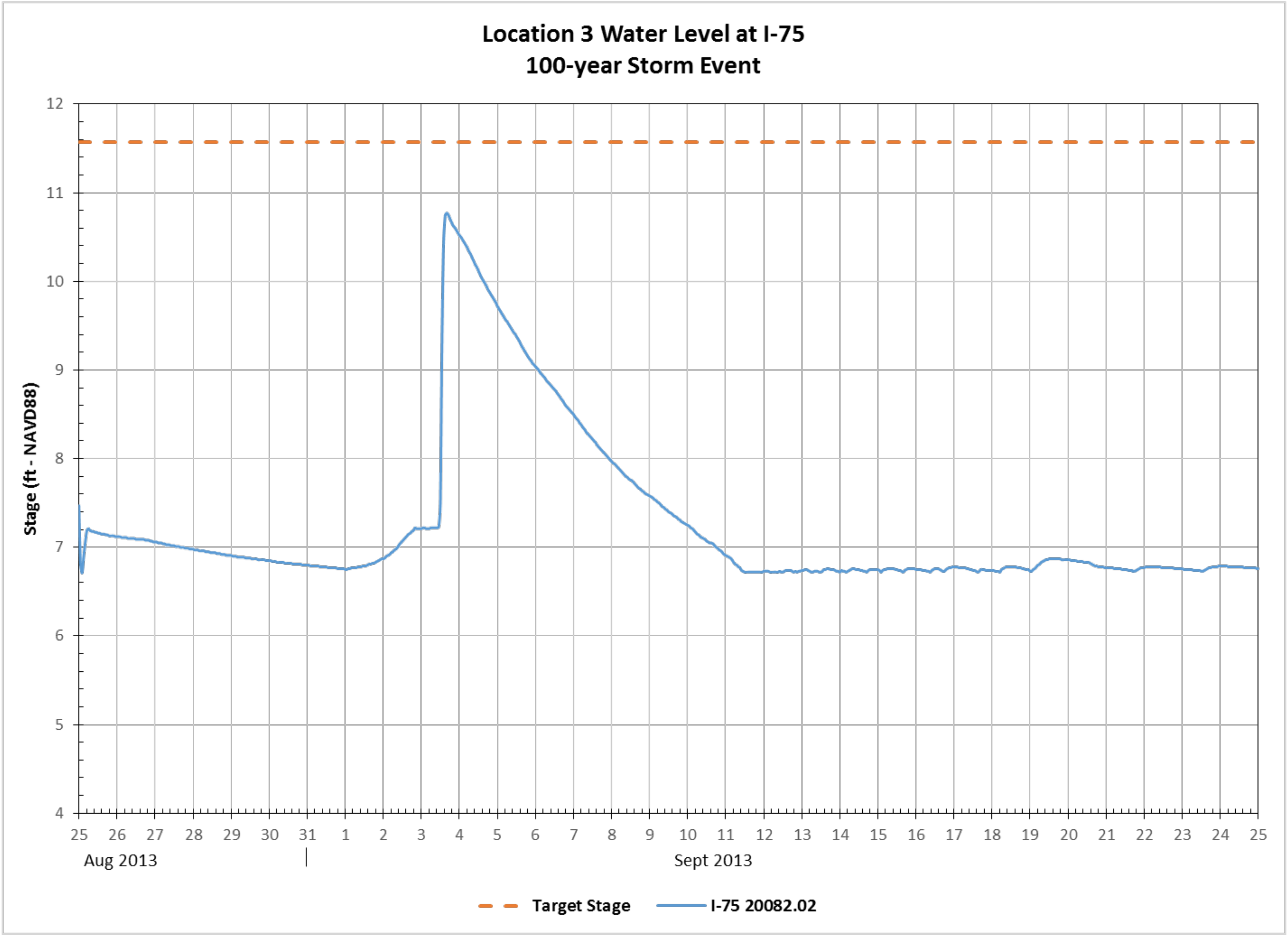


Figure 4.4.13 – Location 3 Duration of Flooding, 100-Year Storm

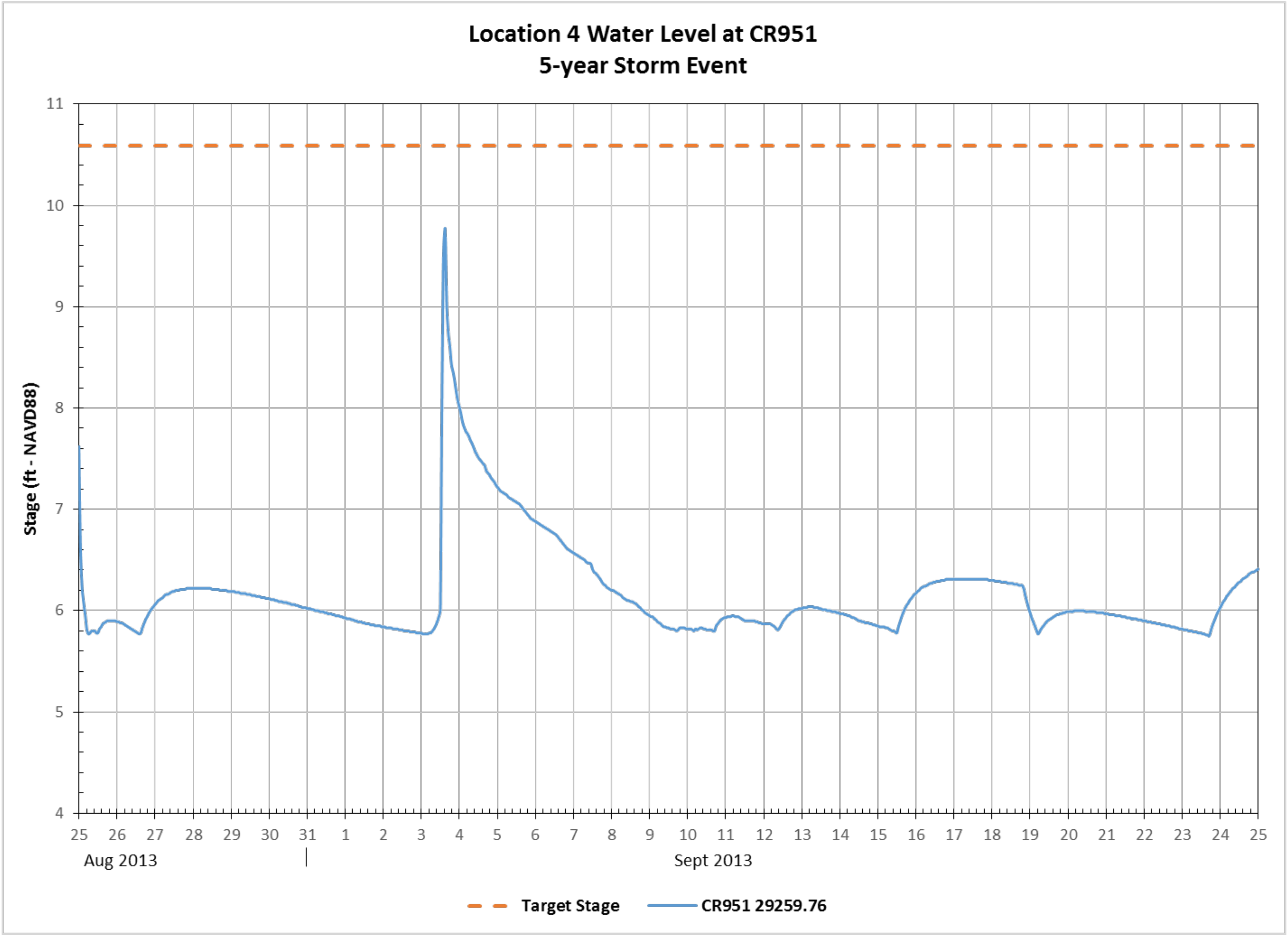


Figure 4.4.14 – Location 4 Duration of Flooding, 5-Year Storm

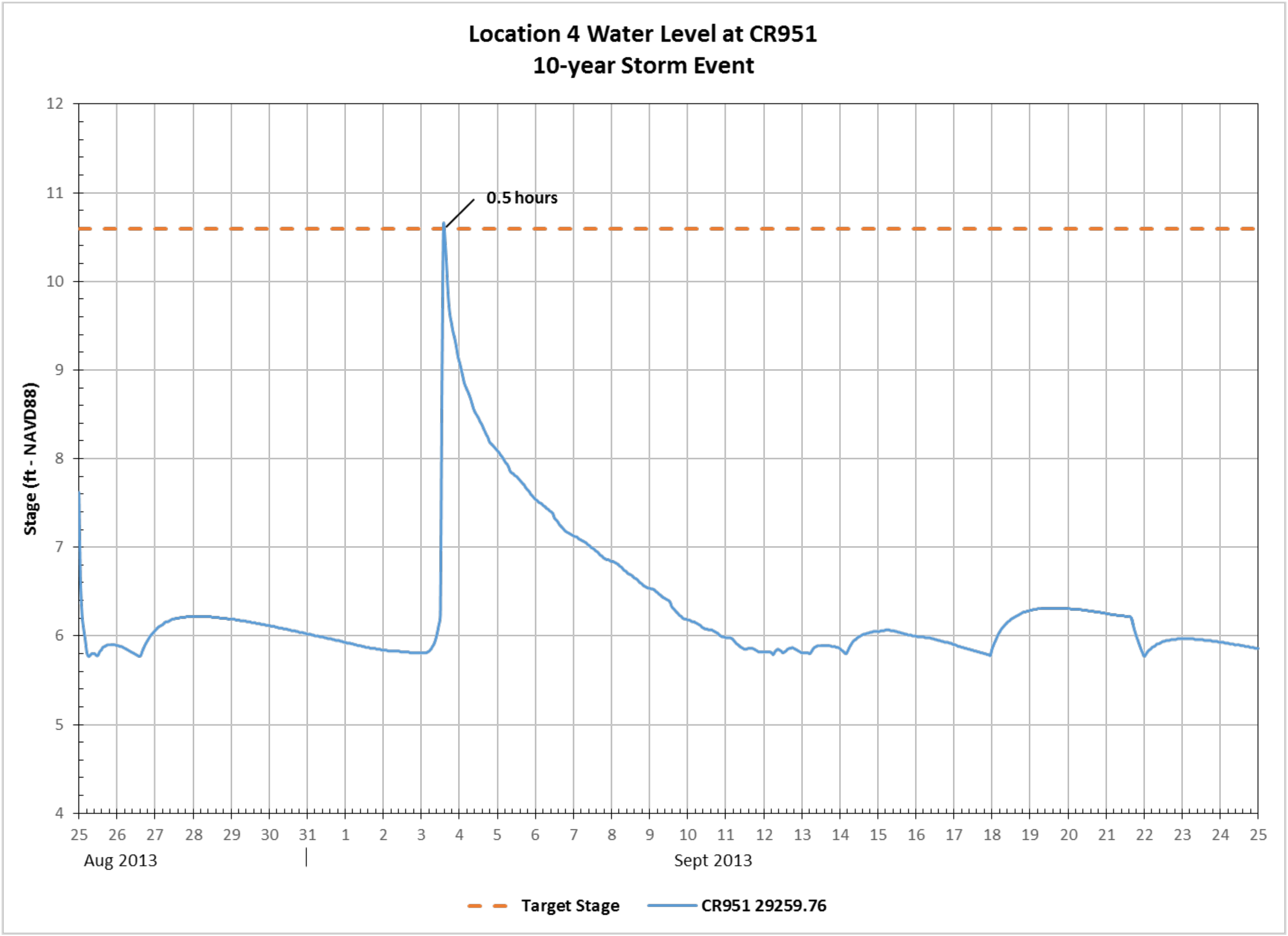


Figure 4.4.15 – Location 4 Duration of Flooding, 10-Year Storm

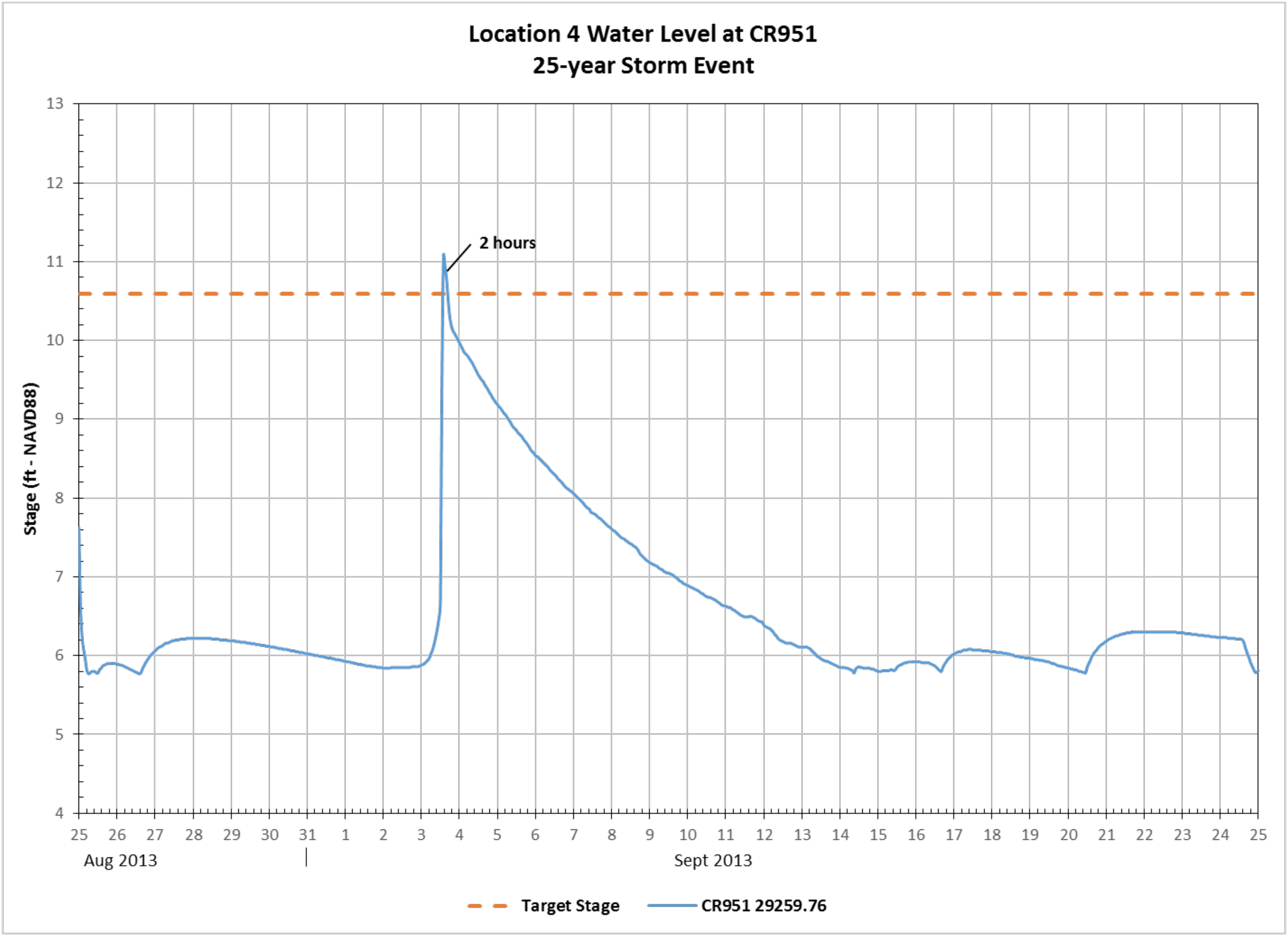


Figure 4.4.16 – Location 4 Duration of Flooding, 25-Year Storm

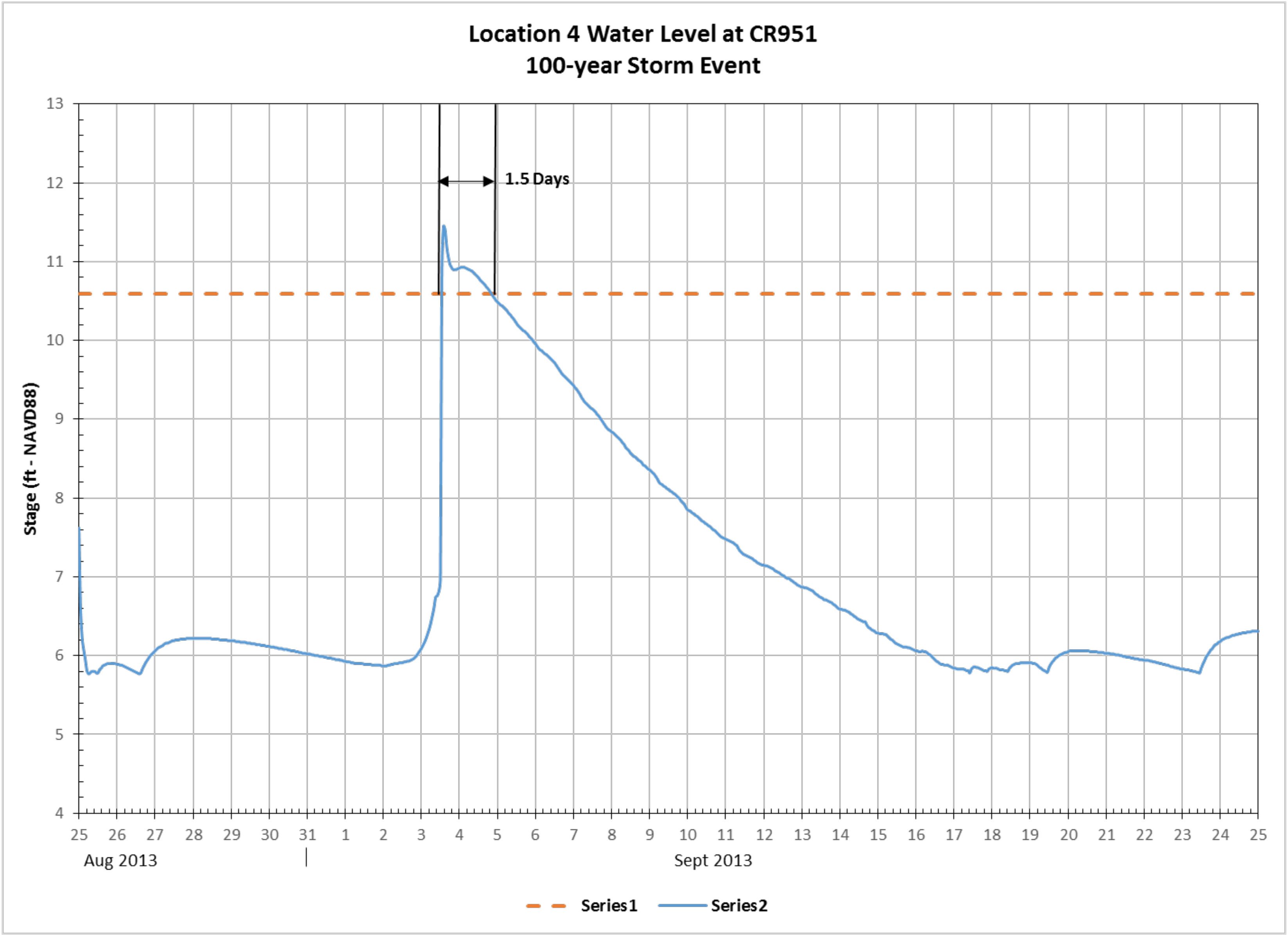


Figure 4.4.17 – Location 4 Duration of Flooding, 100-Year Storm

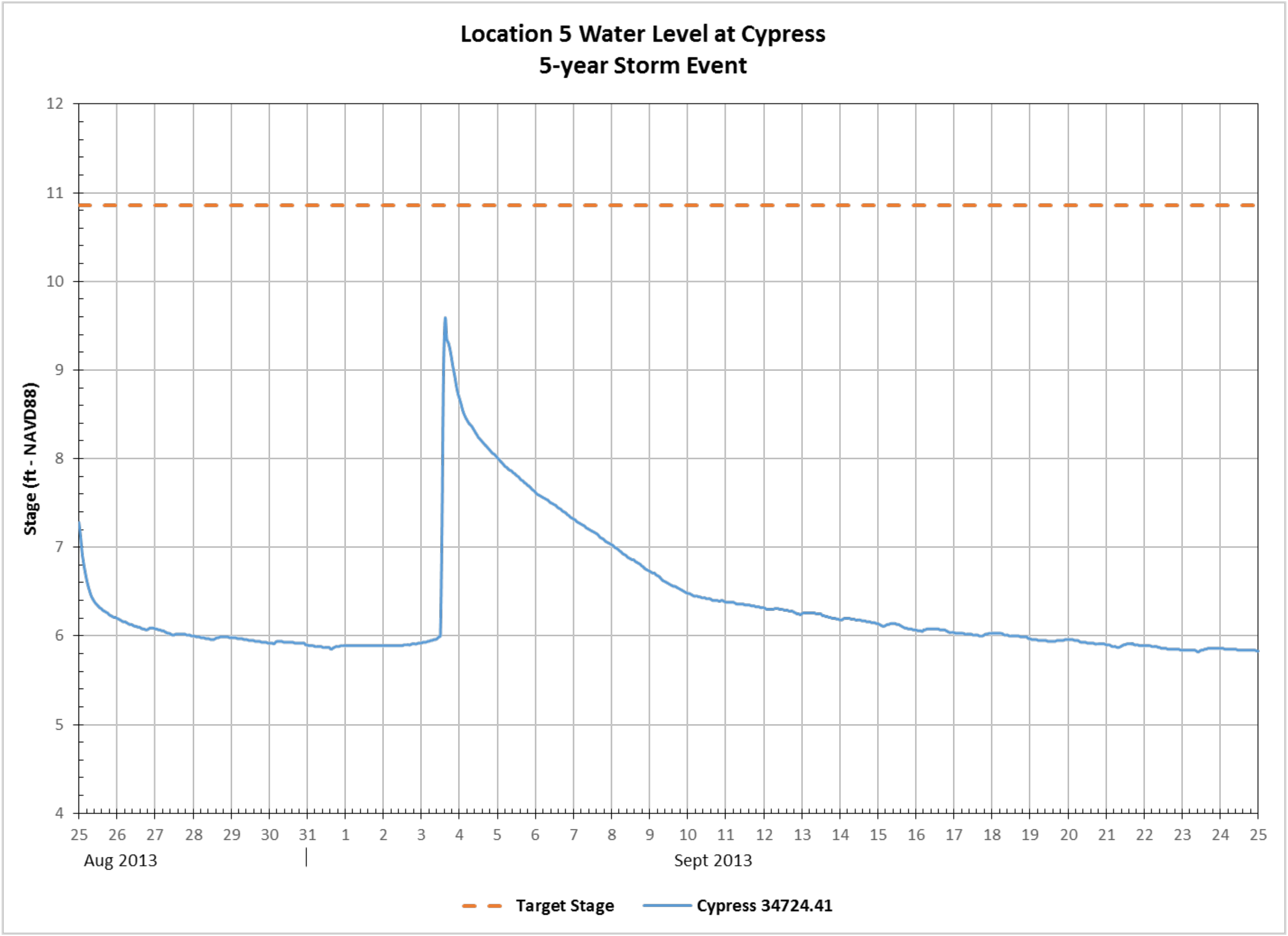


Figure 4.4.18 – Location 5 Duration of Flooding, 5-Year Storm

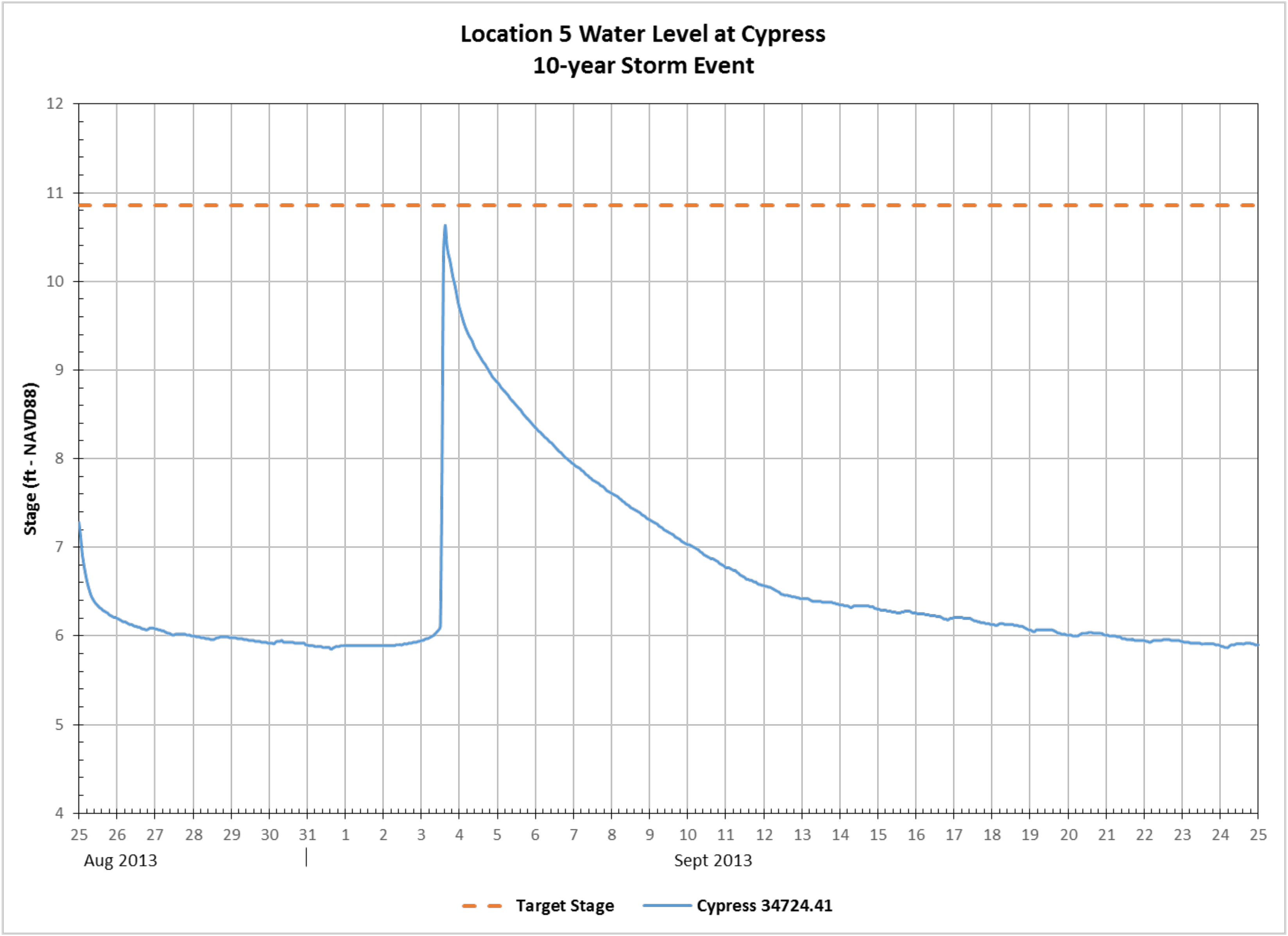


Figure 4.4.19 – Location 5 Duration of Flooding, 10-Year Storm

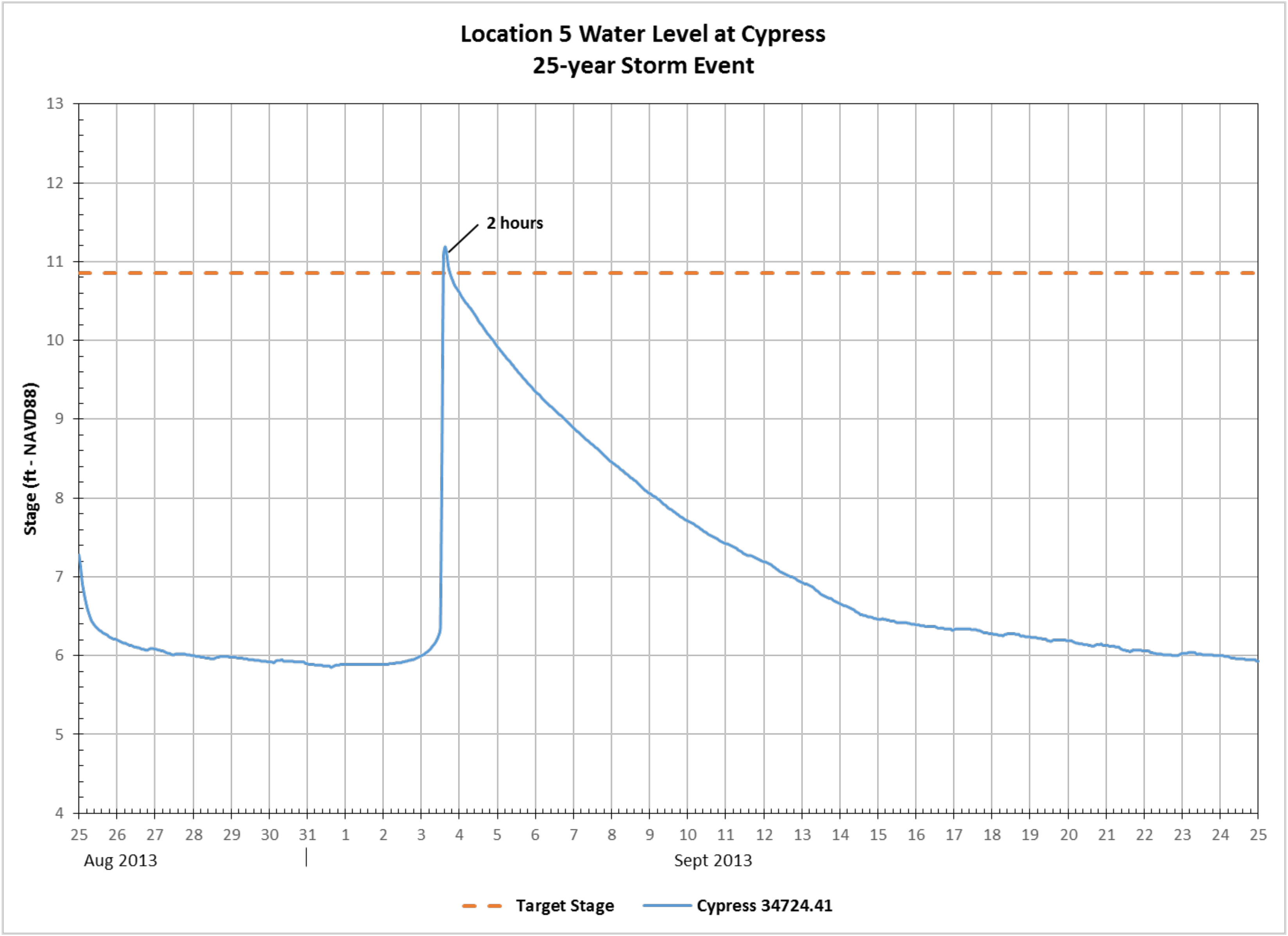


Figure 4.4.20 – Location 5 Duration of Flooding, 25-Year Storm

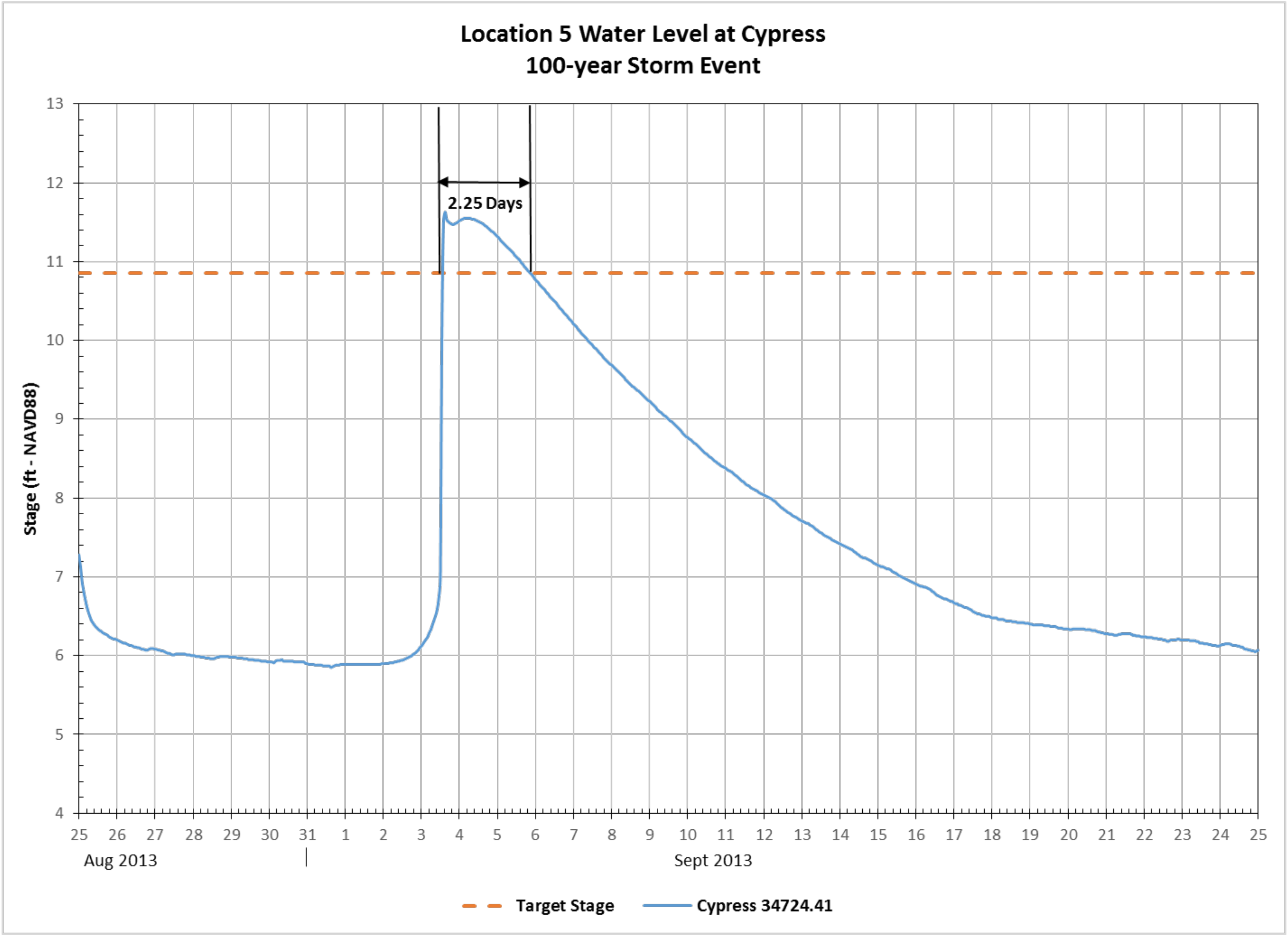


Figure 4.4.21 – Location 5 Duration of Flooding, 100-Year Storm

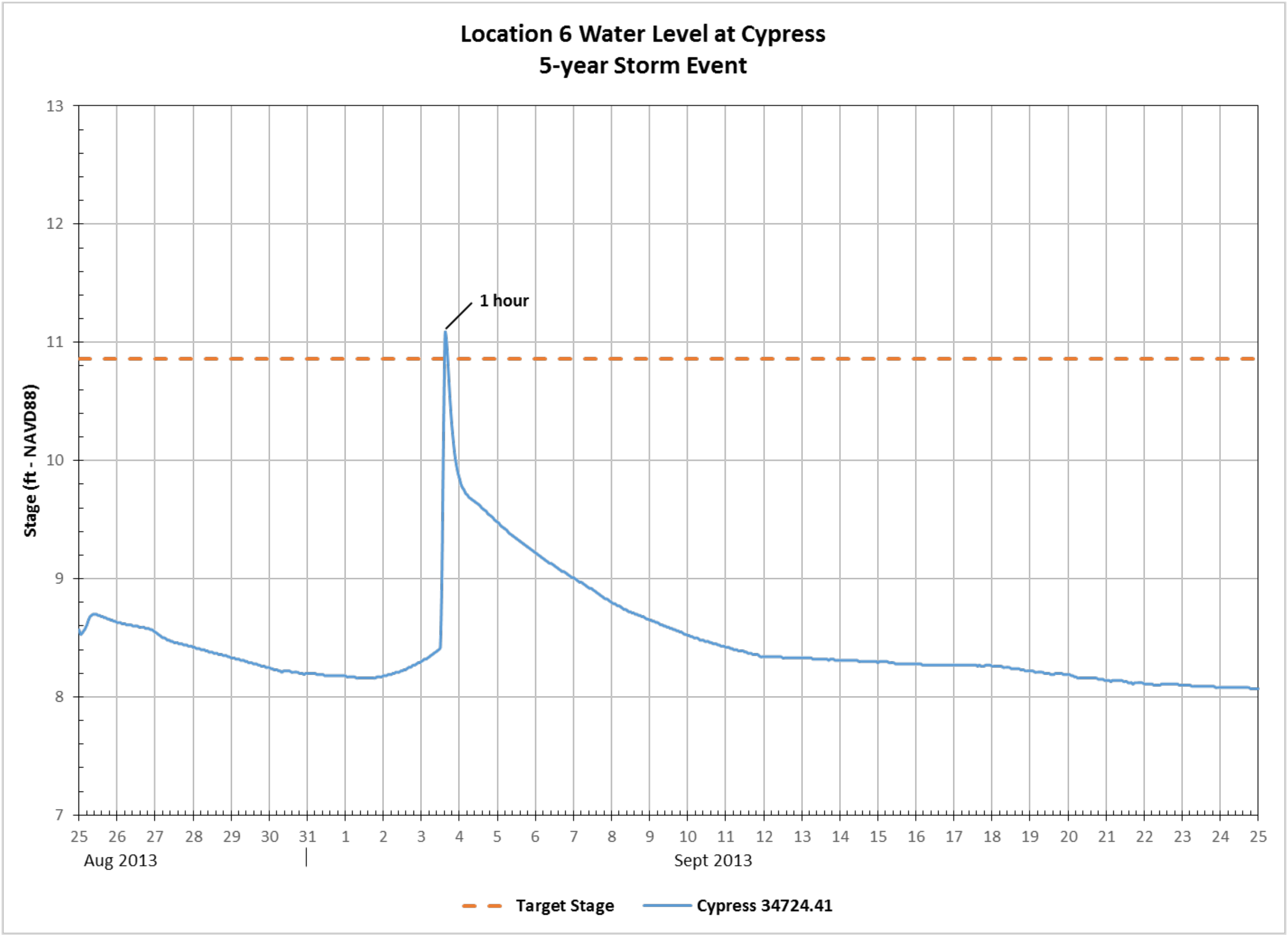


Figure 4.4.22 – Location 6 Duration of Flooding, 5-Year Storm

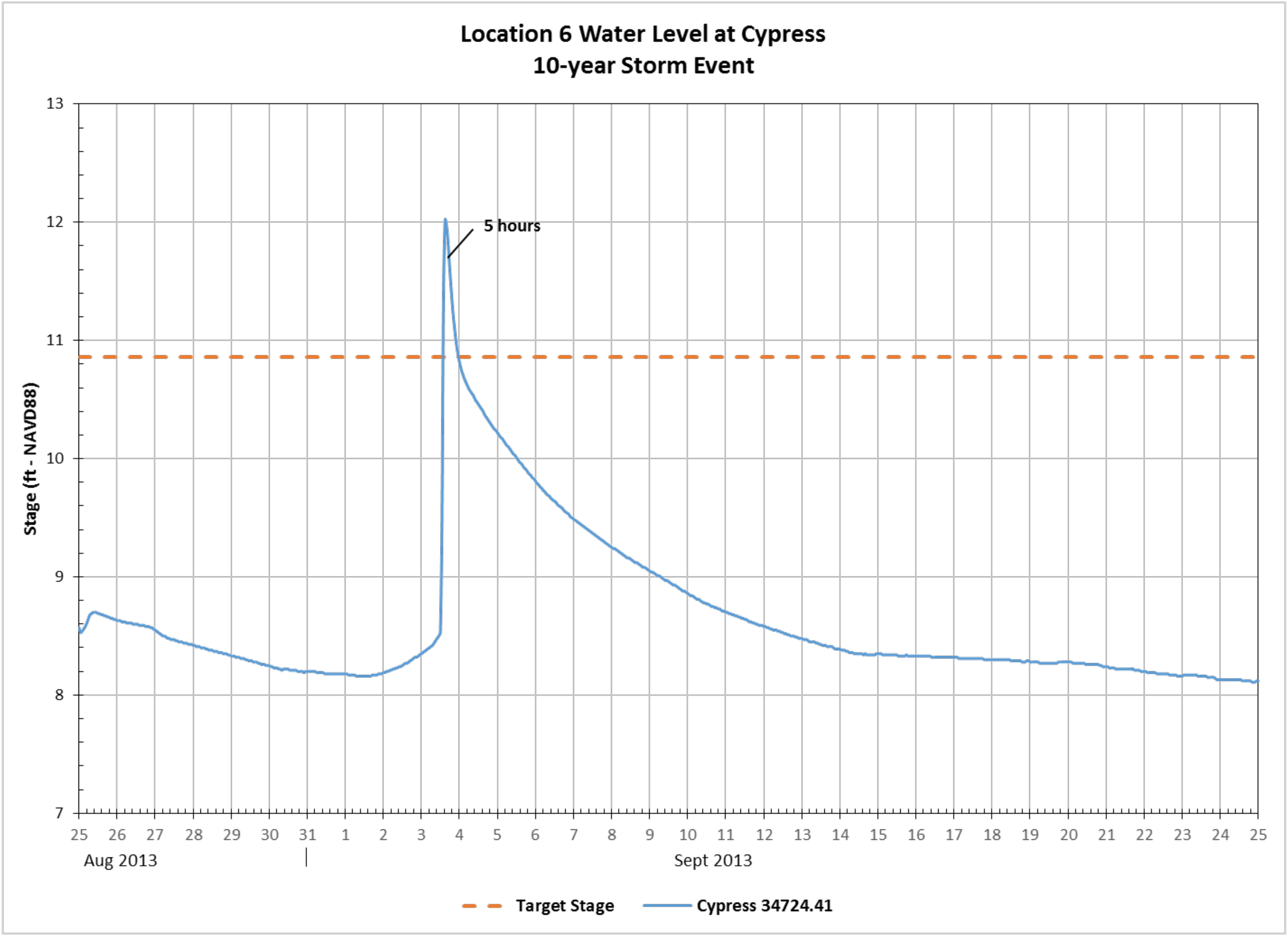


Figure 4.4.23 – Location 6 Duration of Flooding, 10-Year Storm

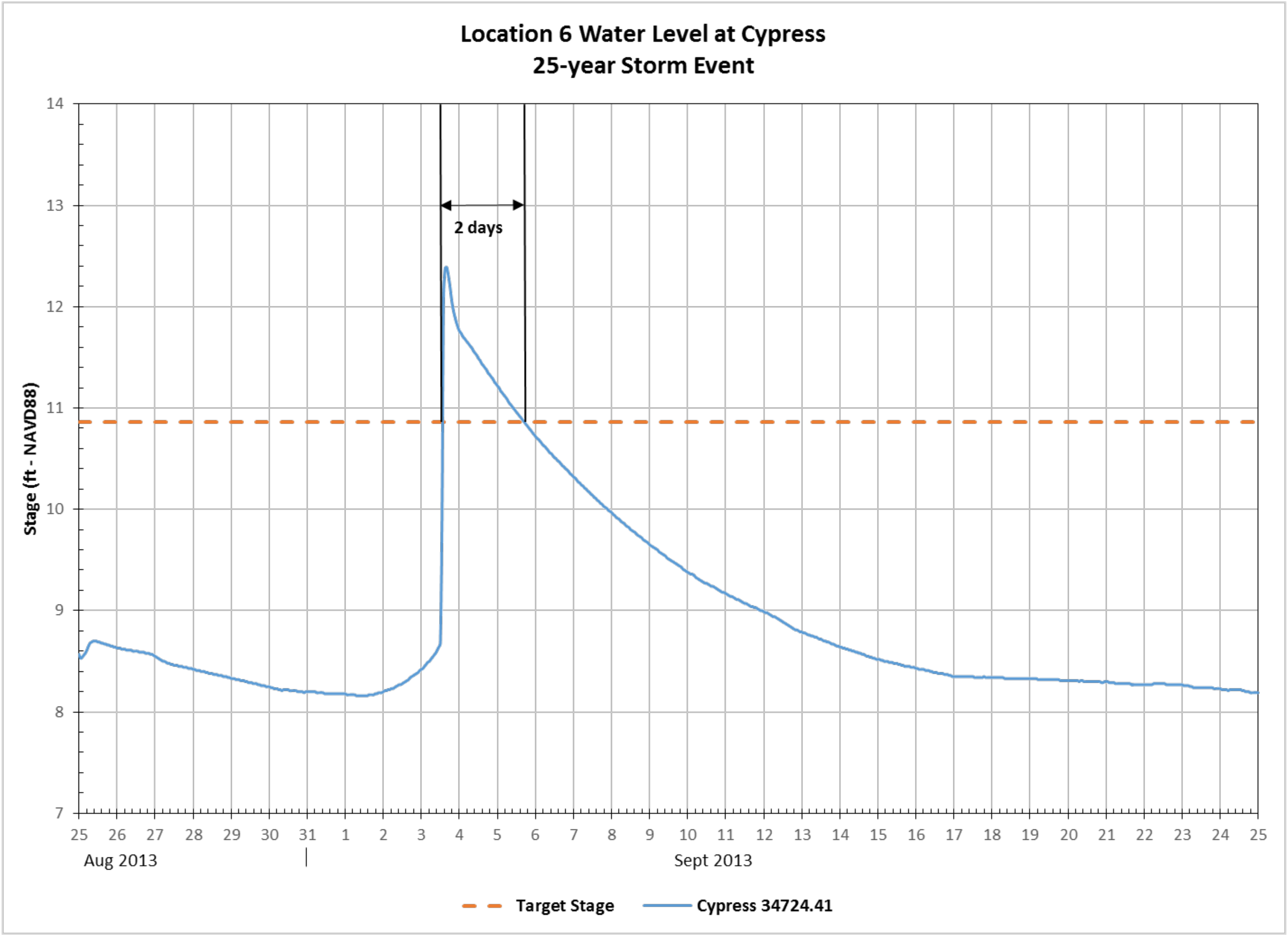


Figure 4.4.24 – Location 6 Duration of Flooding, 25-Year Storm

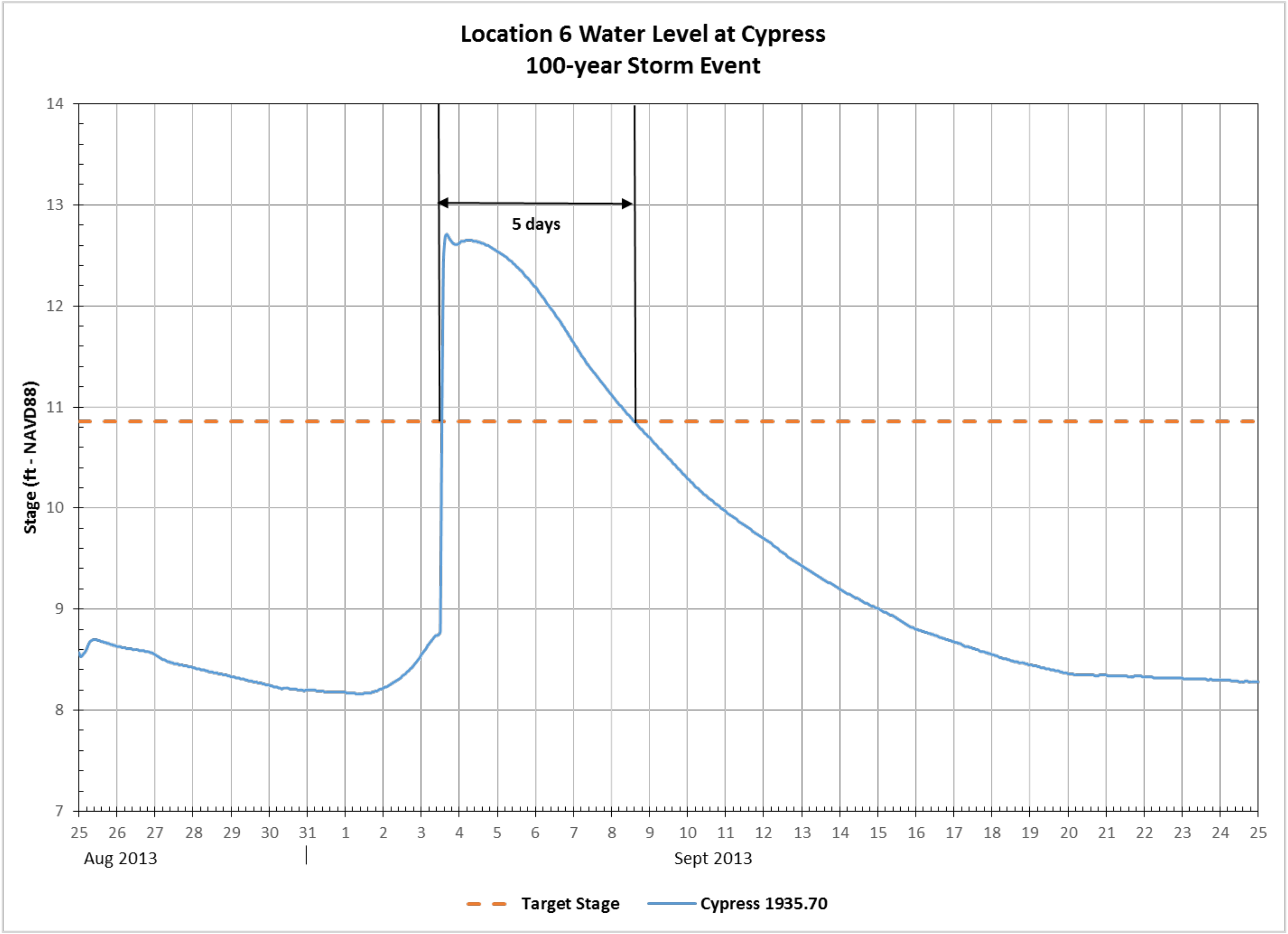


Figure 4.4.25 – Location 6 Duration of Flooding, 100-Year Storm

5.0 Future Conditions Application Model Set-Up

For the Future Conditions model, the "current conditions" model infrastructure, structure operations, and design storm rainfall were carried over, unchanged. The performance of the existing infrastructure was tested under several future scenarios and design storm events.

5.1 Future Land Use

The District provided shape files of future (year 2065) land use projections, which are documented in a December 2016 report prepared by Tim Lieberman of the District titled *Collier County 2065 Landuse for MIKESHE Modeling*. In one of the shape files, the land use was categorized in a fashion identical the MIKE SHE land use classification. Interflow analyzed the shape file to identify areas where urbanization is expected to expand or intensify. **Figure 5.1.1** shows all such areas within the Golden Gate Watershed. In most cases, undeveloped lands are projected to become developed into one of three urban land use categories associated with dwelling unit density (high, medium, and low). In some cases, existing low-density urban is expected to increase to medium or high density urban.

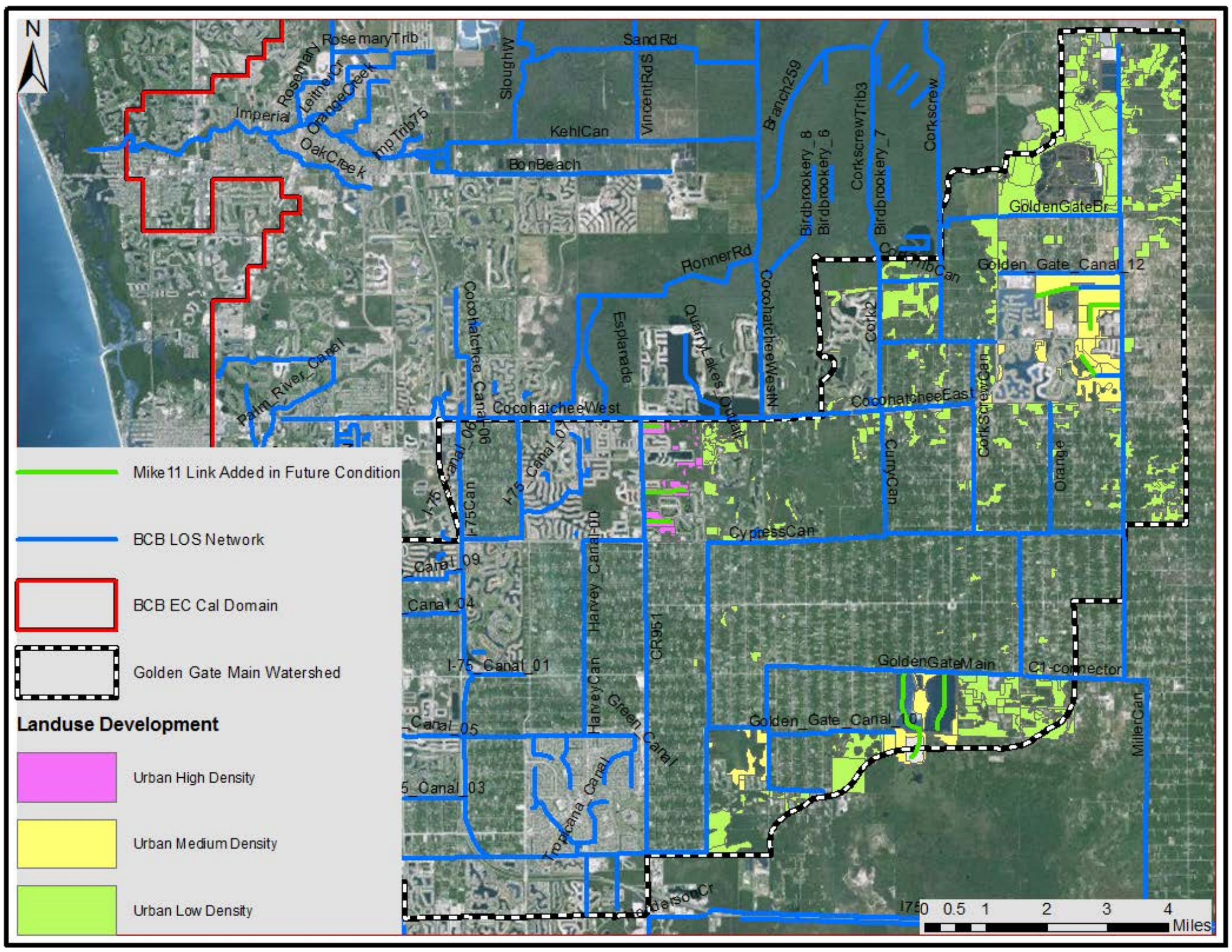


Figure 5.1.1 – Projected Changes in Land Use, 2015 to 2065 (Source: SFWMD)

The polygon-based land use changes were then mapped onto the MIKE SHE Grid. **Figure 5.1.2** shows the grid cells that were changed to represent 2065 Land Use. All land-use based model parameters were changed accordingly for these cells, in order to appropriately represent the potential urbanization. These include the following:

- Paved Area Runoff Coefficient
- Drain Code
- Drain Level
- Drain Time Constant
- Topography (if cell current topo was below FEMA BFE)
- Detention Storage

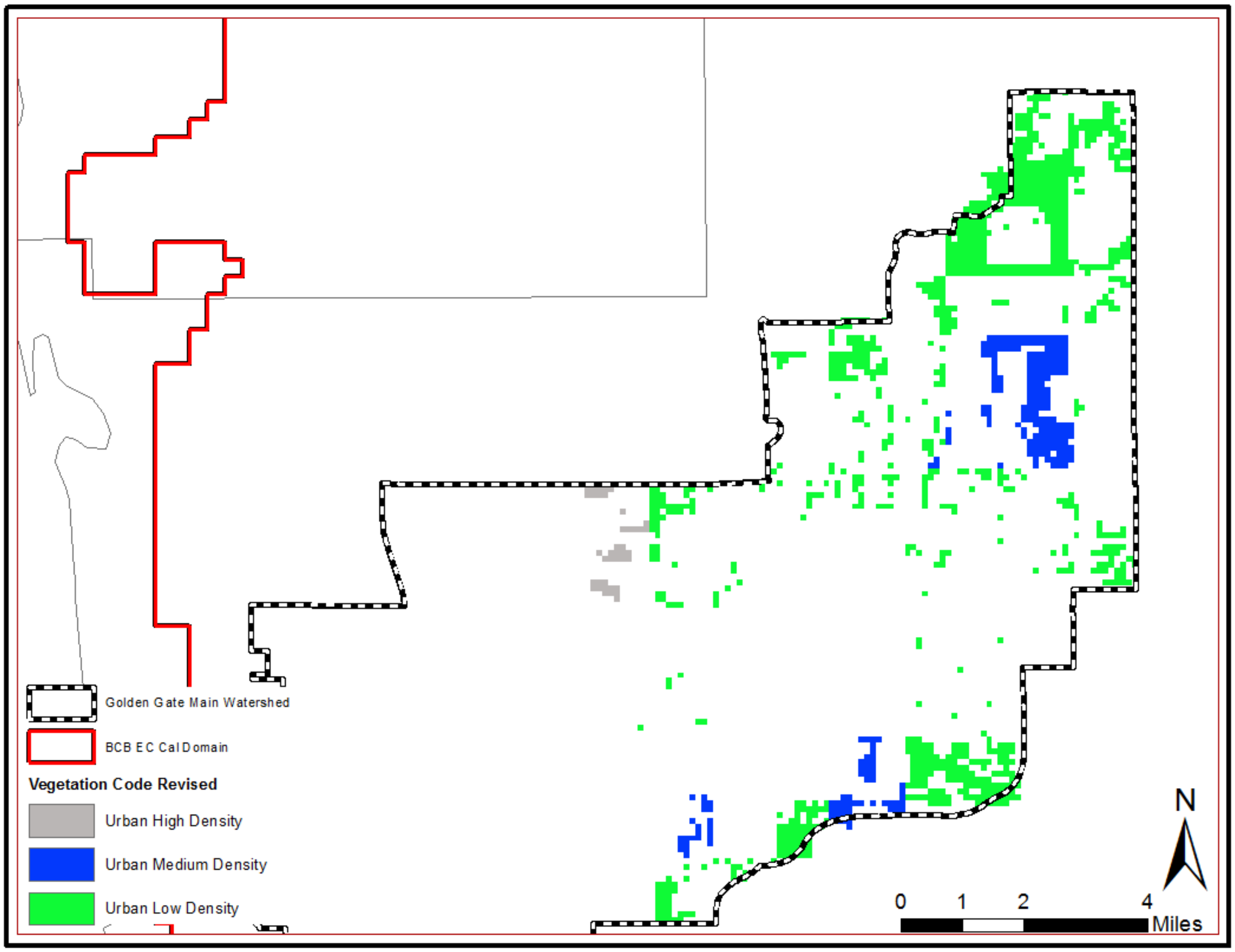


Figure 5.1.2 – Changes in MIKE SHE Vegetation Codes

For larger areas projected to be developed, a procedure similar to that proposed in the August 2016 Report *Task 1.2 Test Bed Model* was applied. In this approach, MIKE-11 branches were added to represent stormwater detention ponds and their associated outfall structures. An iterative process was employed to size the outfall structure to pass the permittable peak discharge rate, based on Collier County's permitting criteria as of 2016/early 2017. The allowable discharge rate for the Golden Gate Watershed was 0.15 cfs per acre (ref: Collier County Ordinance 90-10, as amended in 2000 and 2007) . **Figure 5.1.3** shows an example of one of the areas where this approach was employed. It is noted that Collier County very recently amended their discharge-related permit criteria. However, the future conditions modeling utilized the 0.15 cfs per acre criteria in effect at the time of model development.

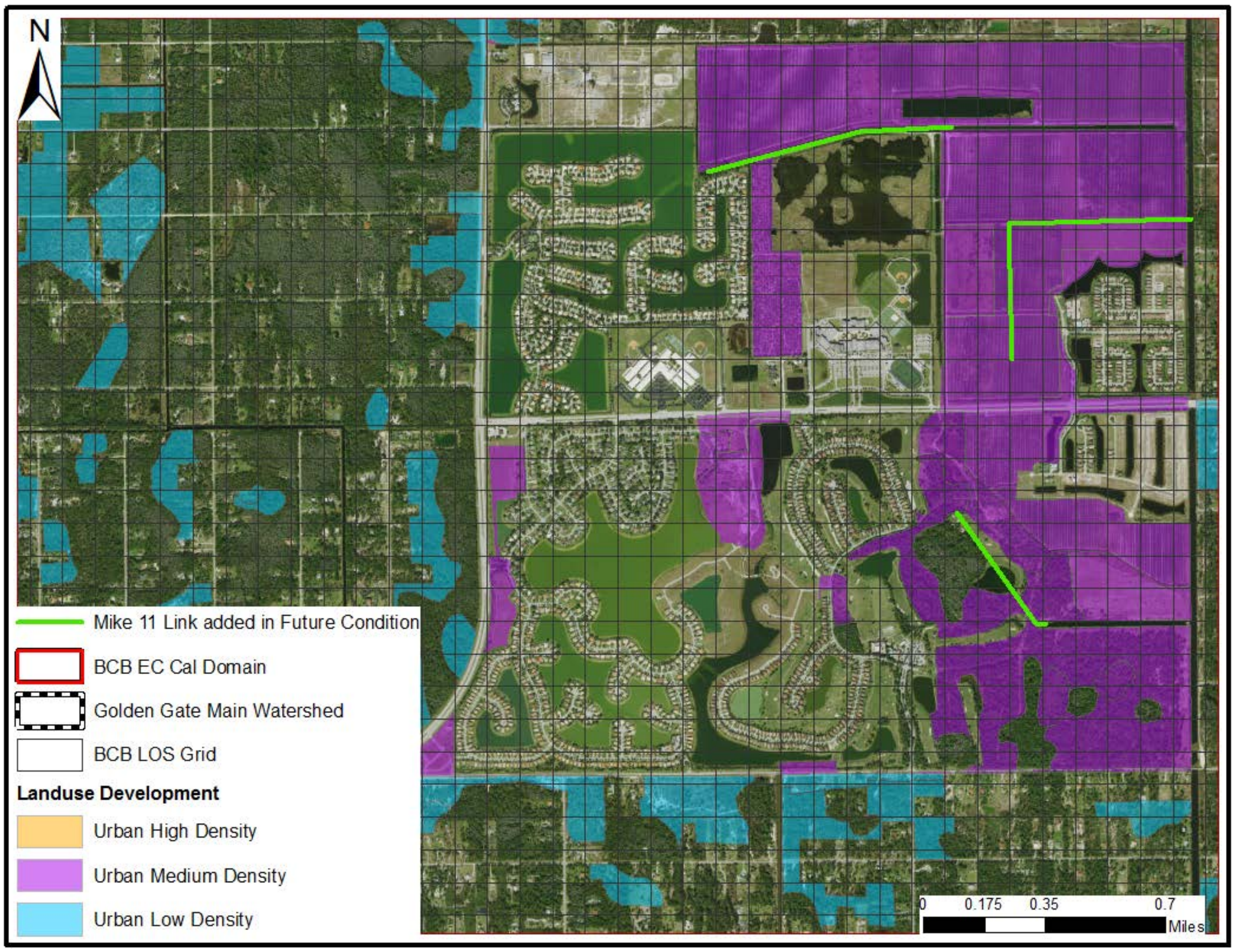


Figure 5.1.3 – Potential Urbanization – Location #1

In some of the areas identified for potential development, land use changes are currently underway. **Figure 5.1.4** shows several smaller developments currently under construction. We obtained the ERP permit application packages for these developments (where available), and used the information on the construction plans to guide the development of model input including pond and control structure dimensions.

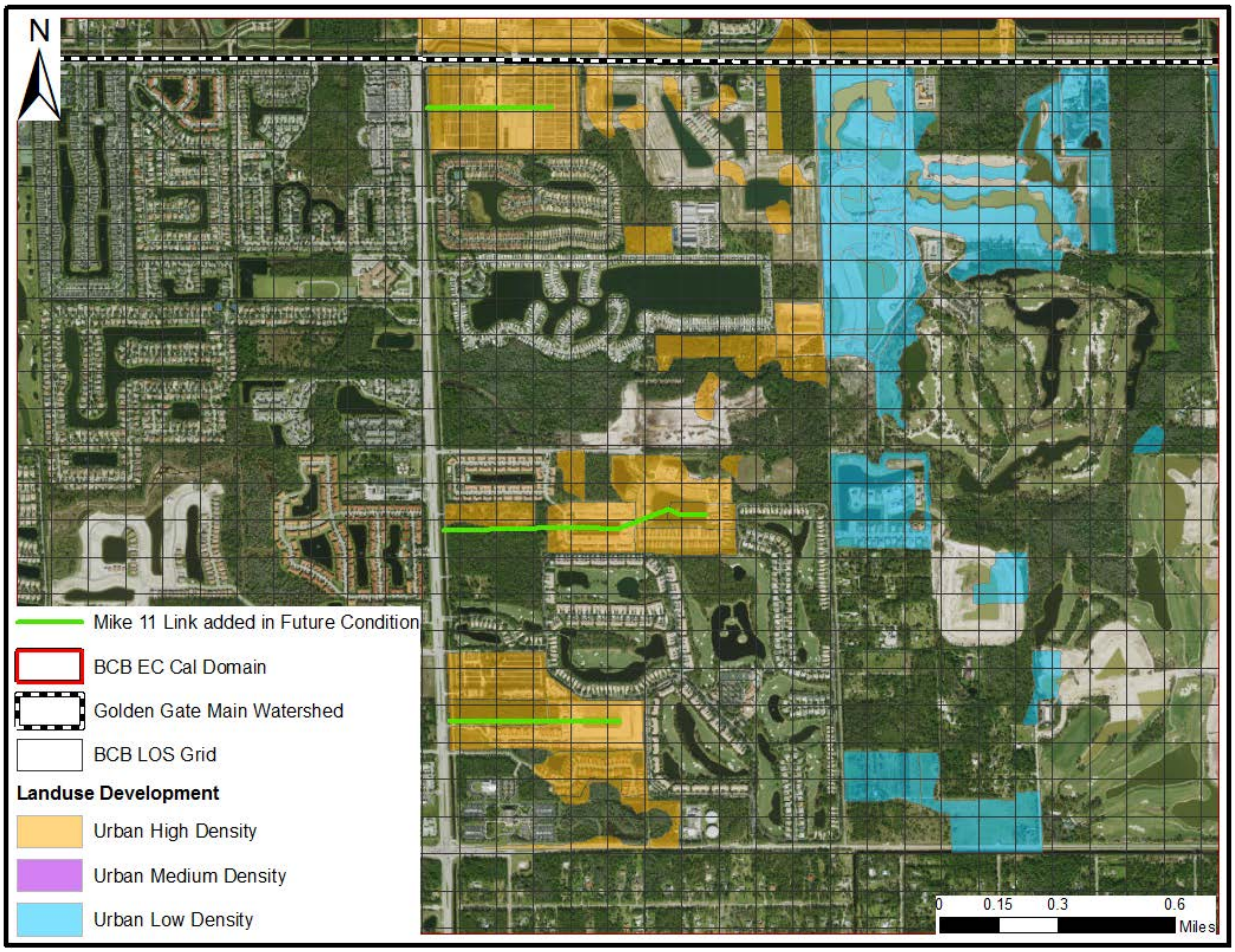


Figure 5.1.4 – Potential Urbanization – Location #2

Figure 5.1.5 shows a complex of mine pits, with surrounding lands identified for potential development. In this area, we assumed the large pits would serve to attenuate stormwater runoff, and we added MIKE-11 branches with conceptual weirs for this purpose. It is important to note that these mine pits were simulated as isolated lakes in the current conditions model. Examination of the detailed DEM revealed that the pits are indeed separated from the adjacent Golden Gate Canal by an elevated embankment. Because the pits are much larger than necessary for stormwater attenuation, connecting the pits to the Golden Gate Canal also provides the potential for lowering flood levels in the canal.

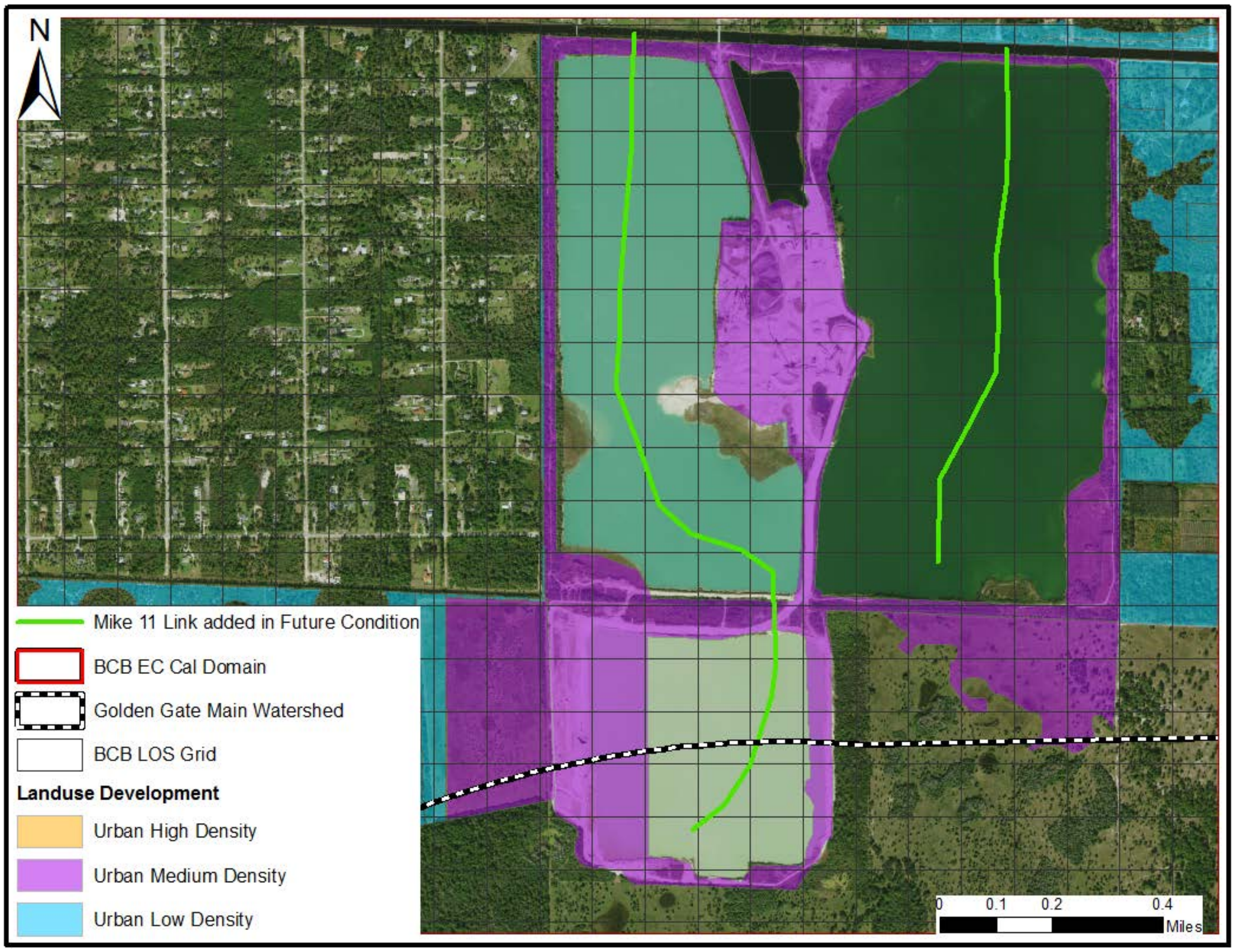


Figure 5.1.5 – Potential Urbanization – Location #3 (Preferred Materials Mine)

Location #4 (**Figure 5.1.6**) shows an area of existing low-density residential, agriculture, and forested lands projected to become medium density urban. Because these areas are relatively small and somewhat isolated an unconnected, it was decided not to use the test-bed model approach here, but rather to increase the detention storage to account for the attenuation provided by stormwater ponds. A similar approach was used in other smaller isolated land use change areas. In areas where the test-bed approach is used, a detention storage of 0.2 inches was used. In the other future urbanized areas, a value of 1.0 inch was used.

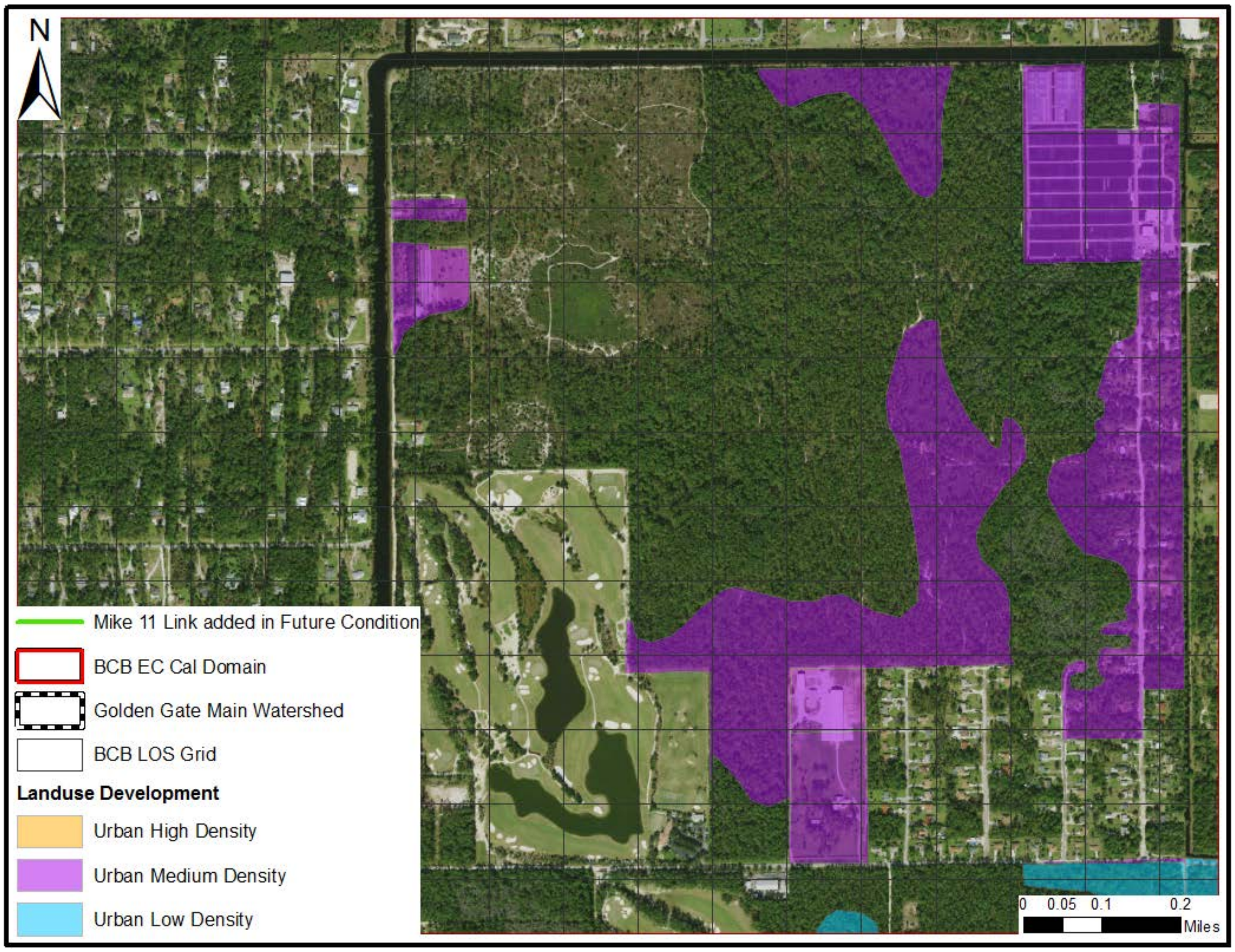


Figure 5.1.6 – Potential Urbanization – Location #4

Topography was changed in selected grid cells, based on a comparison of current topography and the current FEMA Flood Insurance Rate Maps in the areas identified for future development. If the grid cell topography was lower than the FEMA Base Flood Elevation (BFE), then the topographic elevation of the grid cell was raised to be near the FEMA BFE. It was assumed that the ground elevations would be a few tenths of a foot below the BFE, with the understanding that the floor slabs would typically be about one-half of a foot or so above the adjacent grade. **Figure 5.1.7** shows the changes in topography (Future Conditions minus Current Conditions).

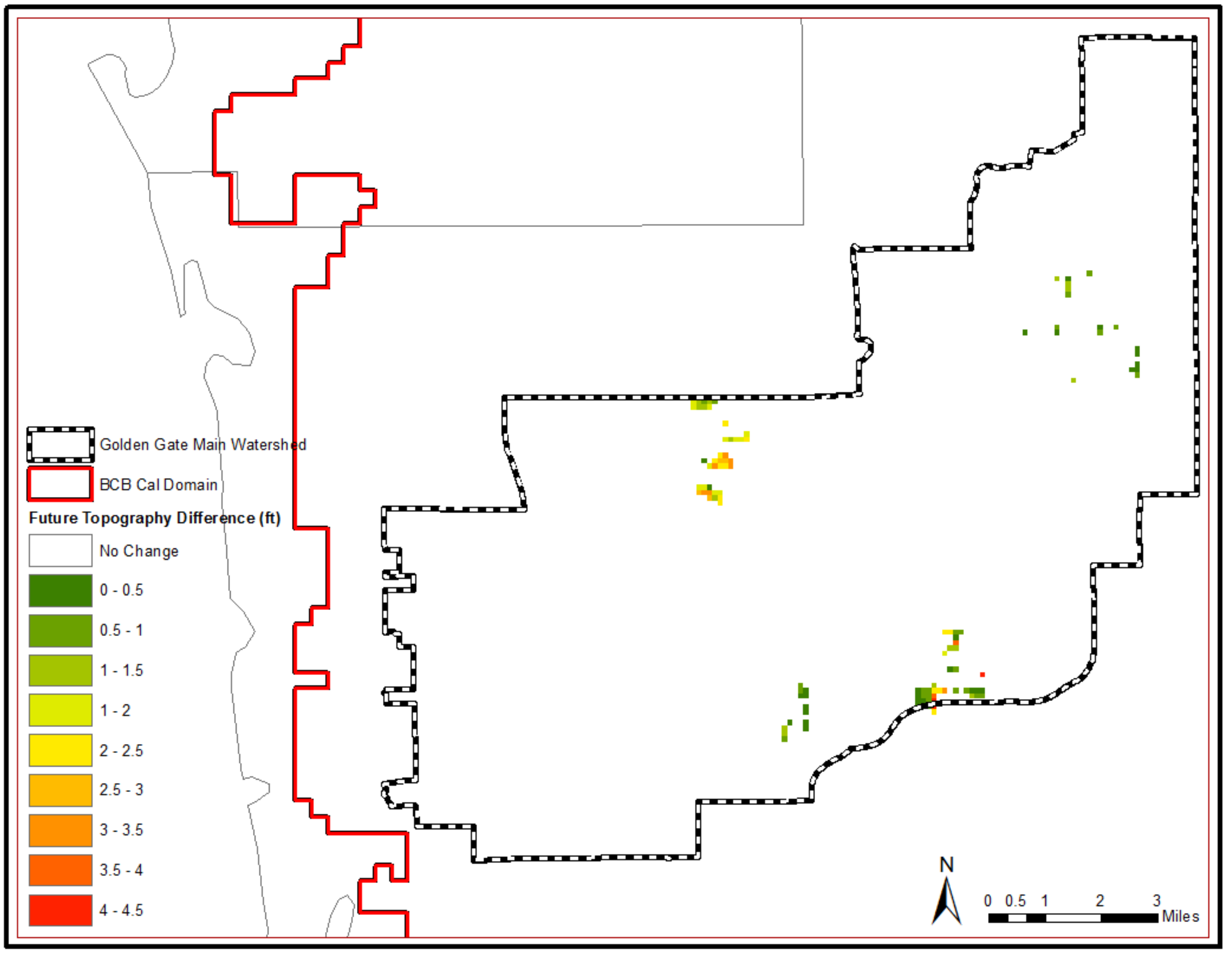


Figure 5.1.7 – Changes in Topography (Future Conditions minus Current Conditions)

Drain codes were set up for the areas identified for urbanization. Some of the drain codes in the current conditions model in this area were negative, which allowed the model to route drainage to local depressions. All new drain codes are positive values, which means that all drainage in the areas of future development are routed to the nearest MIKE-11 h-point. This is based on the expectation that drainage will be improved in these areas to include positive outfalls for new development. **Figure 5.1.8** shows the drain code areas that were added or revised.

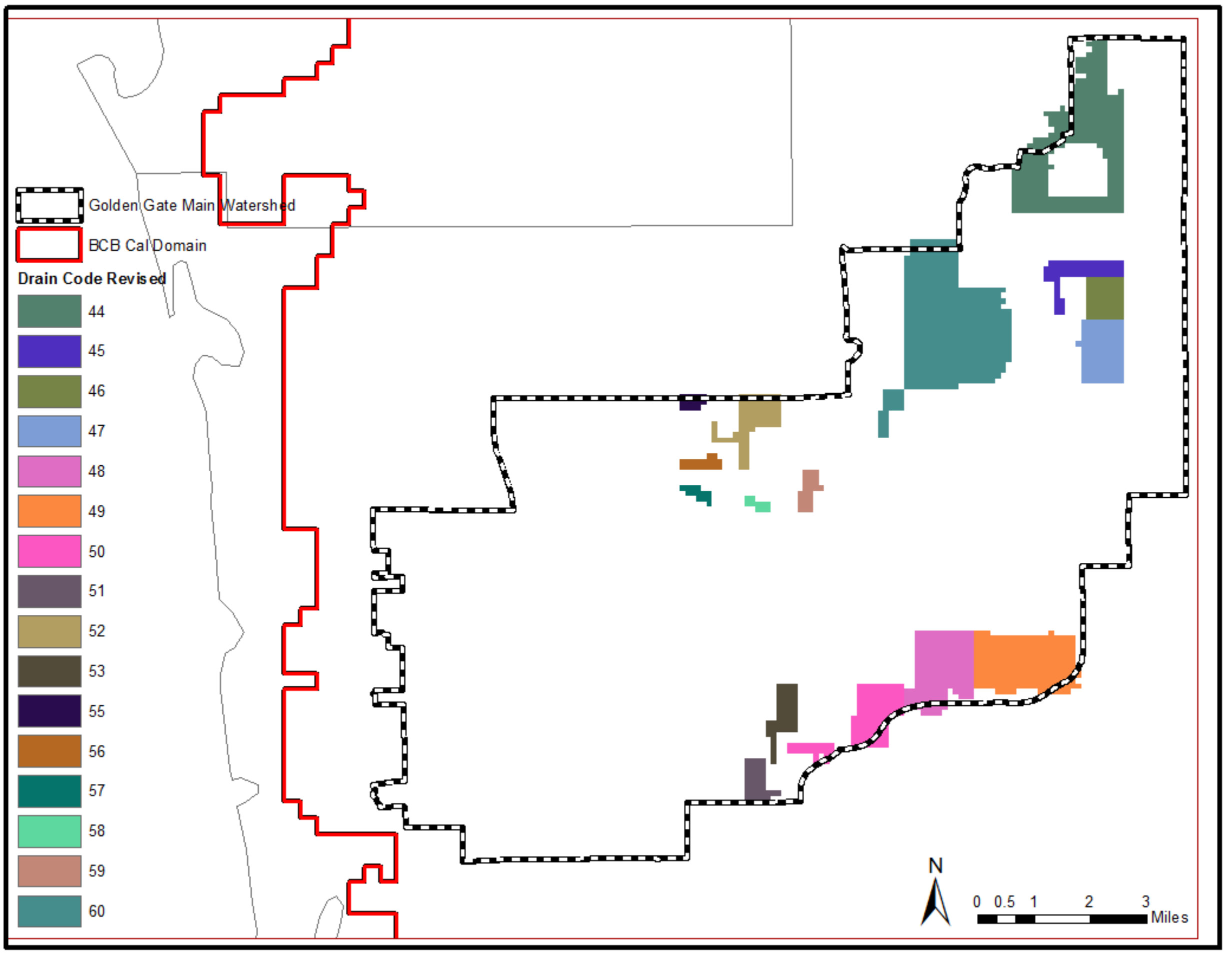


Figure 5.1.8 – Revised Drain Codes

Similarly, separated overland flow areas (SOLFAs) were adjusted to represent the changes in urban land use. **Figure 5.1.9** shows the SOLFAs that were added or modified. SOLFAs were not added to areas projected to be developed into low-density subdivisions, as these subdivisions generally do not entail wholesale changes to the topography. For the low-density developments it was assumed that offsite runoff would be accommodated rather than re-routed as is often the case with medium and high-density developments.

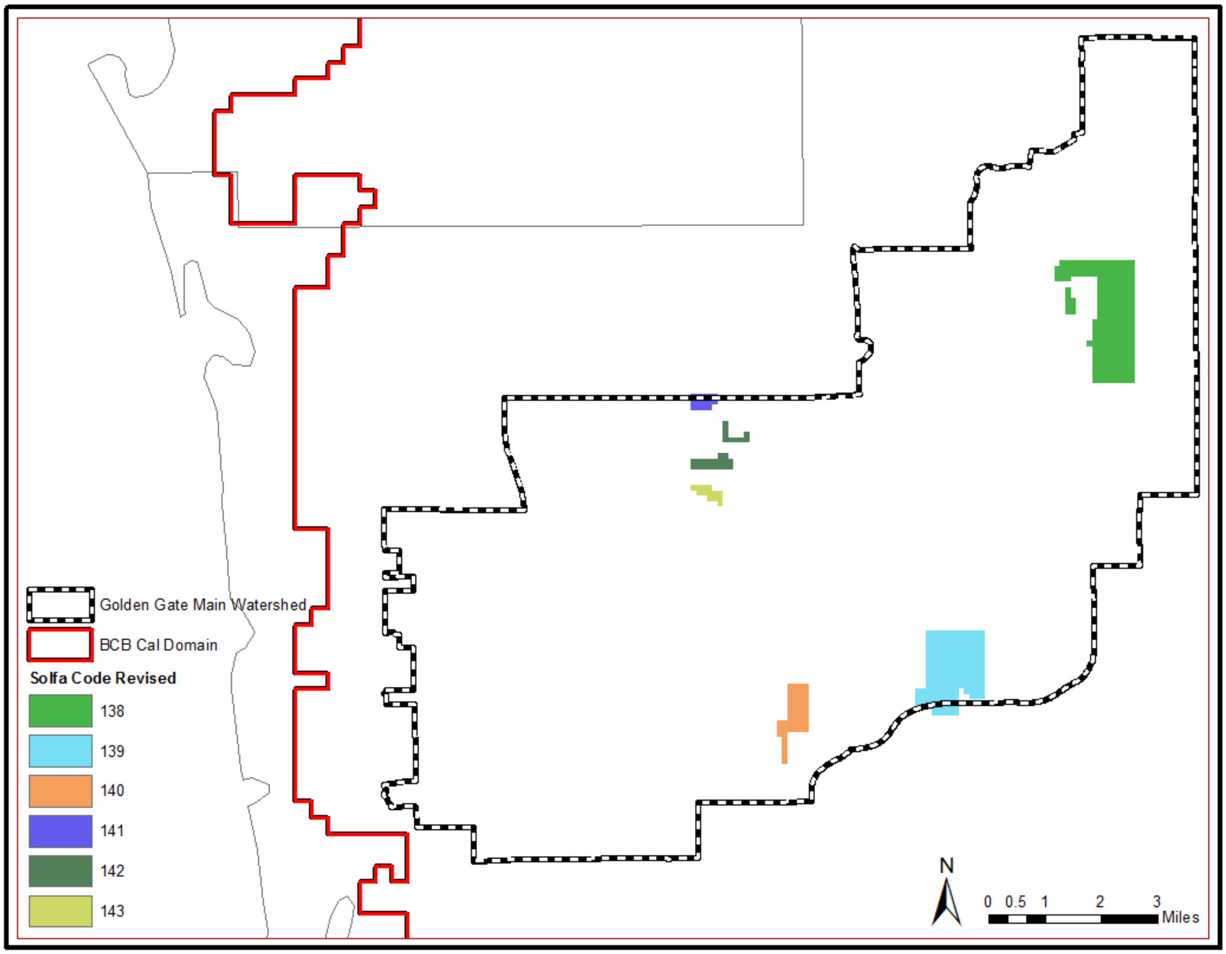


Figure 5.1.9 – Separated Overland Flow Areas (SOLFAs) Added or Revised

5.2 Future Sea Level

Future sea level is represented in the model using a combined approach involving three components of the model setup:

- Surface water (MIKE-11) tailwater conditions
- Groundwater boundary conditions
- Initial water table elevations

<u>MIKE-11 Tailwater Boundaries at Coastal Outfall Structure GG1:</u> Tailwater hydrographs for use in the future conditions design storm simulations were provided by the District in a spreadsheet, and documented in the report: *South Florida Water Management District, 2017. Flood Protection Level of Service Analysis for the Big Cypress Basin. Appendix C: Preparation of Boundary Conditions at the Tidal Structures. H&H Bureau, SFWMD, West Palm Beach, FL. 34 pp. June 21, 2017.* Tailwater hydrographs for three year-2065 sea level scenarios were included; low (SLR1), medium (SLR2), and high (SLR3). Tailwater hydrographs were provided for all four design storm events (5-year, 10-year, 25-year, and 100-year), but it was decided to run all three sea level rise scenarios only for the 25-year event. Separate MIKE-11 boundary files were created for each scenario. **Figure 5.2.1** shows the 25-year tailwater hydrographs for

the three future conditions scenarios, with current conditions shown for comparison. **Table 5.2.1** lists the peak tailwater stages at GG1 for current conditions (SLR0) and the three future sea level scenarios.

Similar to current conditions, the dates for the rainfall distributions were adjusted so that the peak rainfall intensity lined up with the peak tailwater stage. This resulted in a spin-up period of approximately one week with normal tidal boundaries and no rainfall, and with the peak rainfall intensity and peak tailwater stage occurring on September 3 2013 between noon and 1 PM.

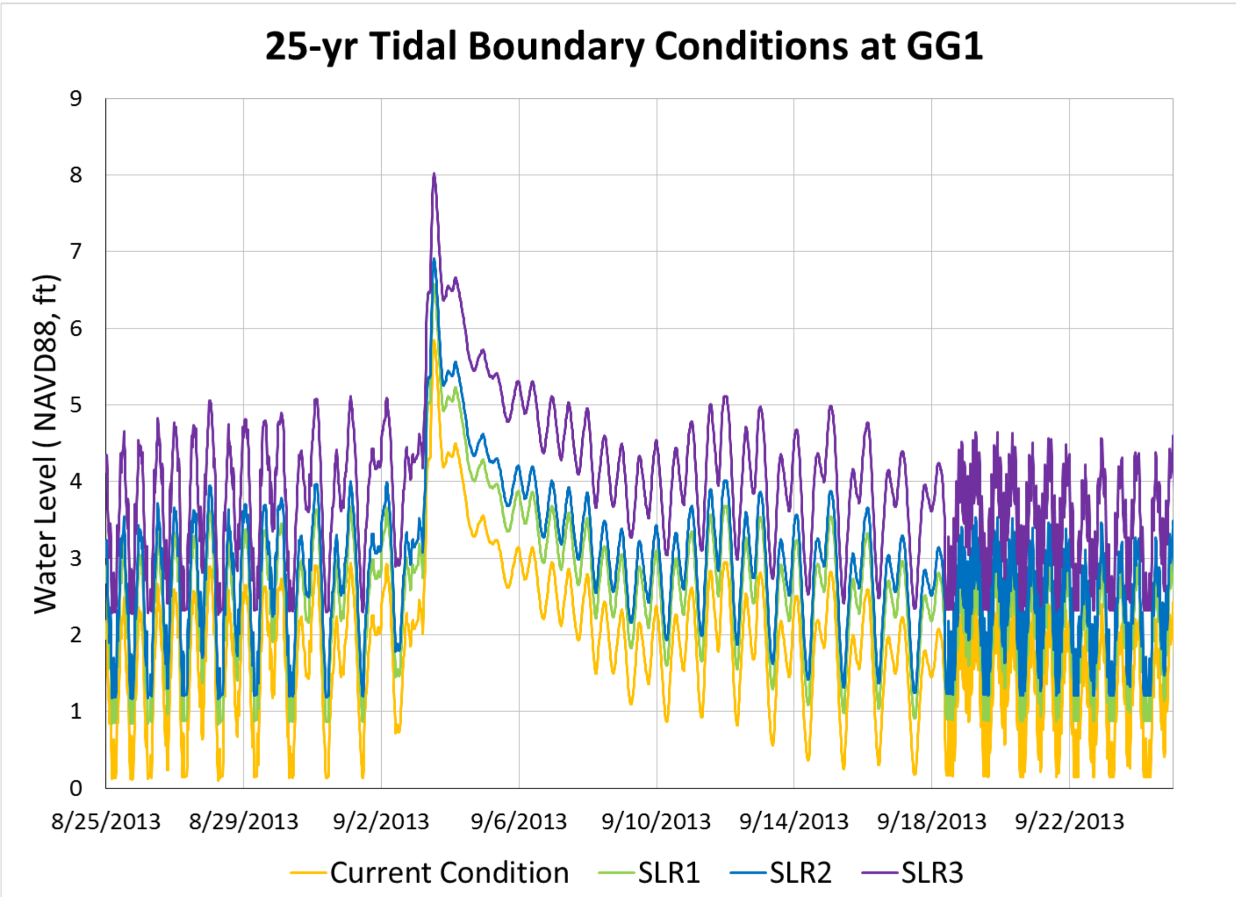


Figure 5.2.1 – GG1 Tailwater Boundary Conditions for Future Sea Level (25-year event)

Groundwater Boundary Conditions: The time-varying Groundwater boundary conditions along the coast for Layer 1 (surficial aquifer) of the MIKE SHE groundwater model were increased for each sea level rise scenario. The increase was equal to the projected sea level rise, relative to current conditions, associated with each scenario. The increase is 0.73 feet, 1.06 feet, and 2.17 feet respectively for SLR1, SLR2, and SLR3.

Table 5.2.1 – Peak Tailwater Stages for Future Sea Level at GG1

Return Period		Future Sea Level Conditions Peak Tailwater Stage at GG1 (ft. NAVD88)		
	SLR0	SLR1	SLR2	SLR3
5-year	4.85	5.58	5.91	7.02
10-year	5.28	6.01	6.34	7.45
25-year	5.85	6.58	6.91	8.02
100-year	6.79	7.52	7.85	8.96

Initial Groundwater Levels: Initial groundwater levels were increased to account for higher tidal boundaries along the coast. Average tide levels are projected to increase, compared to current (year 2015) conditions, by 0.73 feet, 1.06 feet, and 2.17 feet for SLR1, SLR2, and SLR3, respectively. This increase will be manifested some distance inland in the form of elevated water tables. The Project Team discussed the possibility of constructing a separate steady-state model of the surficial aquifer in order to simulate the impact of sea level rise on wet-season water table elevations near the coast. Another option discussed would be to re-run the long-term model with higher coastal groundwater boundary conditions, and then recalculate the 80th percentile groundwater levels using a similar approach to that used in generating the current conditions initial groundwater levels, as described in the Task 3.2 Report (*Flood Protection Level of Service Provided by Existing District Infrastructure for Current (2015) Sea Level Conditions for Golden Gate Watershed*). After some discussion, it was decided that a separate modeling effort would not be feasible within the schedule constraints of the current project, and that a simpler approach is warranted given the uncertainty associated with future sea level rise projections.

Adjustments were made to the current conditions initial water table levels by first determining the average water table level at the cells along the coastal model boundary under current conditions (approximately 0.3 feet NAVD88), and then adding the increase in sea level to determine the minimum coastal water table elevation for each of the three future conditions scenarios. For example, the minimum coastal water table level for SLR3 was set to 0.3 + 2.17 = 2.47 feet NAVD88. To create the initial water table grid (dfs2) file for SLR3, all cells in the initial water table grid with values less than 2.47 were selected, and then those cells were set equal to that value. **Figure 5.2.2** shows the increases in initial water table elevations for SLR3, as compared to current conditions. From the Figure, it is evident that the water table impacts would propagate several miles inland in some areas, particularly in the southern portions of the model domain.

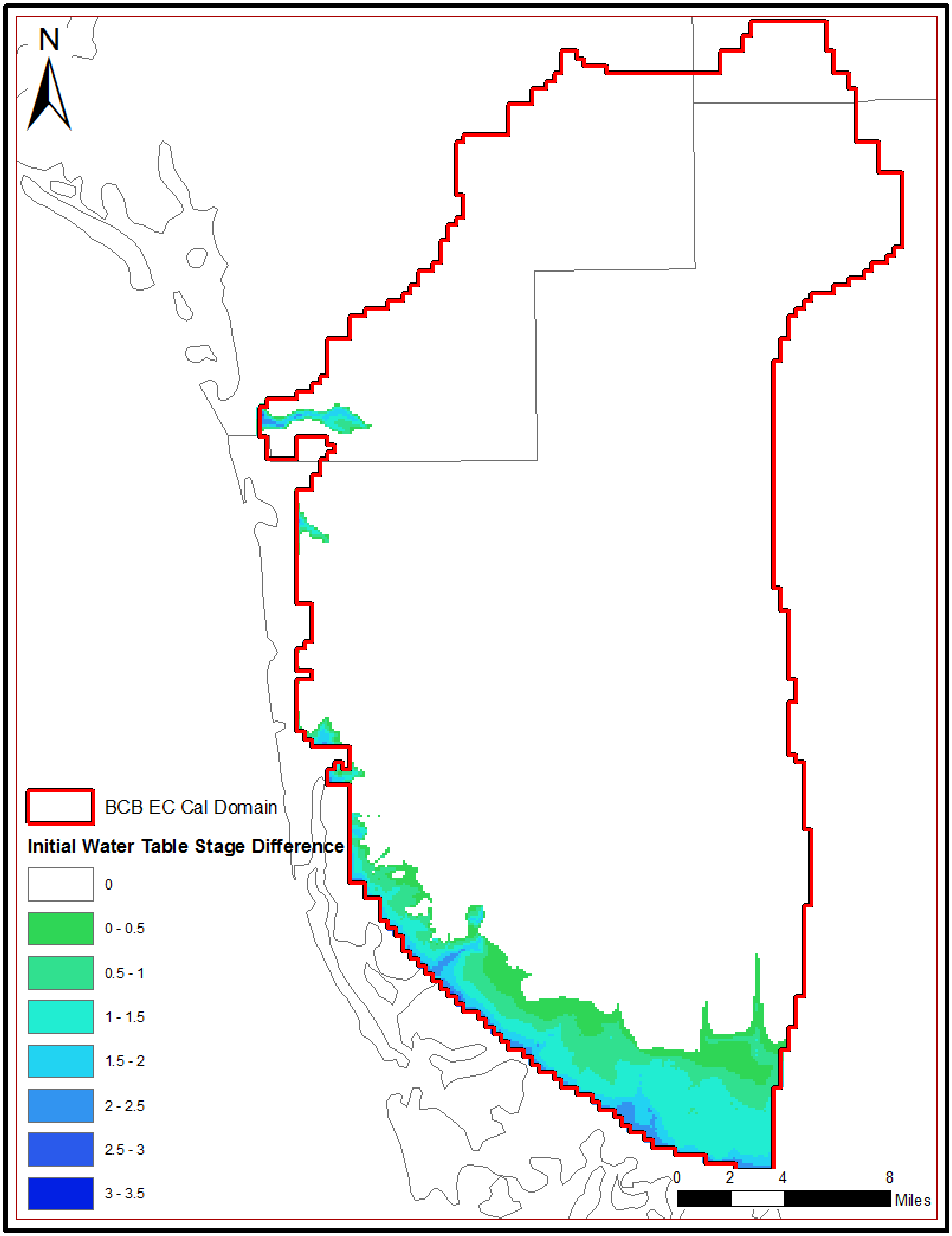


Figure 5.2.2 – Change in Initial Water Table Stages for SLR 3

6.0 Flood Protection Level of Service – Future Land Use

Initial model simulations of the 5-year, 10-year, 25-year, and 100-year, 3-day design were conducted with current sea level conditions and the future land use changes described in **Section 5.1**. **Appendix A** provides summary model results at primary control structures, while **Appendices F** through **I** provide the complete flow and stage hydrographs for each of the four design storm events. The performance of the existing infrastructure with future land use changes was evaluated with respect to the five performance metrics described in the remainder of this Section.

6.1 PM #1 – Maximum Stage in Primary Canals, Future Land Use

This is the peak stage profile in the primary canal system. The profile is developed for a range of design storms (5-year, 10-year, 25-year, and 100-year). The largest design storm that stays within the canal banks establishes the FPLOS of the primary canal system.

To evaluate this PM under future land use conditions within the Golden Gate Watershed, peak stage profiles were prepared for all primary canals within the Golden Gate Watershed. For reference, peak stage profiles are also shown for current conditions. Figures on the following pages depict the peak stage profiles for the following primary canals:

- Golden Gate Main (**Figure 6.1.1**)
- Airport Road Canal (**Figure 6.1.2**)
- I-75 Canal (**Figure 6.1.3**)
- CR 951 Canal (**Figure 6.1.4**)
- Cypress Canal (**Figure 6.1.5**)
- Corkscrew Canal (**Figure 6.1.6**)

Bank elevations on the profile figures are generally based on the MIKE-11 cross section data. However, in several cases the bank elevations appeared suspect and were later modified by Interflow (based on the current LiDAR data) for use in preparing the profile plots. Also shown in the Figures are major roadway landmarks, control structures, and primary canal junctions.

The model results indicate that the potential future land use conversions would affect peak stage profiles in the Golden Gate Canal to a greater extent than in the other canals. Approximately midway along the canal (approximately station 78000), connection of the Preferred Materials mine pits to the Golden Gate Canal as part of the potential urbanization of this area (refer to **Figure 5.1.5**) would provide incidental flood relief in the Golden Gate Canal and the surrounding area. Maximum reductions in peak stage would range from approximately 0.7 feet for the 100-year event to 1.4 feet for the 5-year event at the point of connection. Smaller reductions would occur upstream and downstream, including portions of the Cypress, CR951, and I-75 Canals near their junctions with the Golden Gate Canal. It is important to note that this is a hypothetical development, and as such several modeling assumptions were required. Among them is the assumption that development of the uplands adjacent to the pits and the hydraulic connection of the pits to the Golden Gate Canal would occur together. However, connecting the mine pit storage to the canal could occur independently of development. The degree of flood relief ultimately provided by the mine pit storage would depend in part on how adjacent lands are developed, and on other developments in the watershed.

Upstream of GG4 in the Golden Gate Canal, the maximum flood stages are projected to increase as a result of the future land use conversions. The future development projected to occur in the upper reaches of the Golden Gate Watershed could increase the runoff volumes routed to the canal. In the MIKE SHE model, this increase is simulated in part through increases in the paved area runoff coefficients, and also through the implementation of positive drain codes that route the shallow subsurface drainage directly to the MIKE-11 network in areas of future urbanization. Except for an area in the vicinity of GG4, the majority of the stage increase would be contained within the canal banks.

Although the increased flooding in the areas upstream of GG4 is significant, it’s important to remember that several modeling assumptions were made regarding future development. Actual development will differ from the configurations assumed for modeling purposes. Collier County recently changed the allowable post-development discharge for the Main Golden Gate Canal Basin and the Cypress Canal Basins, from 0.15 cfs/acre to 0.04 and 0.06 cfs per acre, respectively (ref: Collier County Ordinance 90-41). This action alone might be sufficient to mitigate the potential increases in flooding predicted by the land use changes simulated in this study.

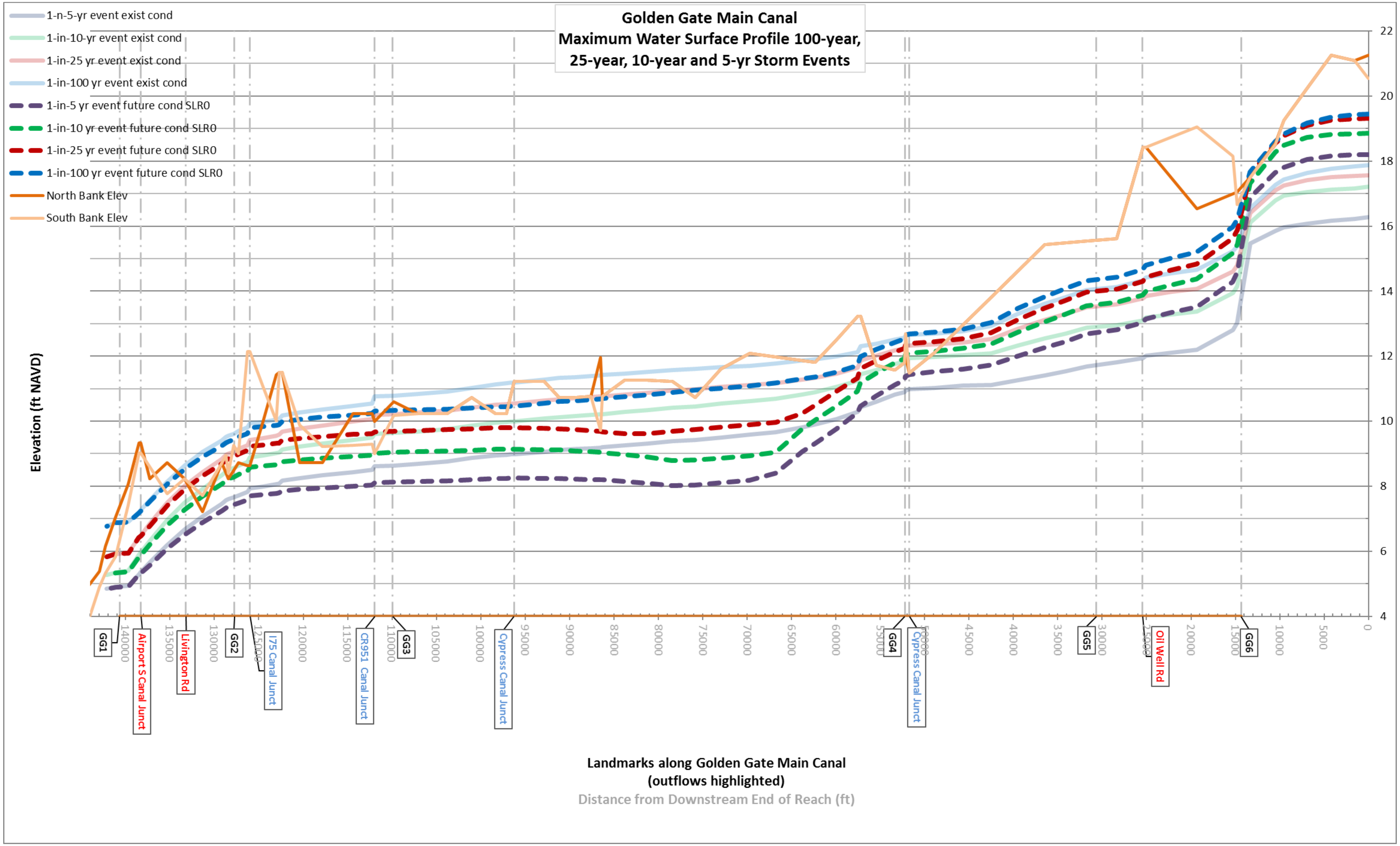


Figure 6.1.1 – Golden Gate Main Peak Stage Profiles, Current and Future Land Use

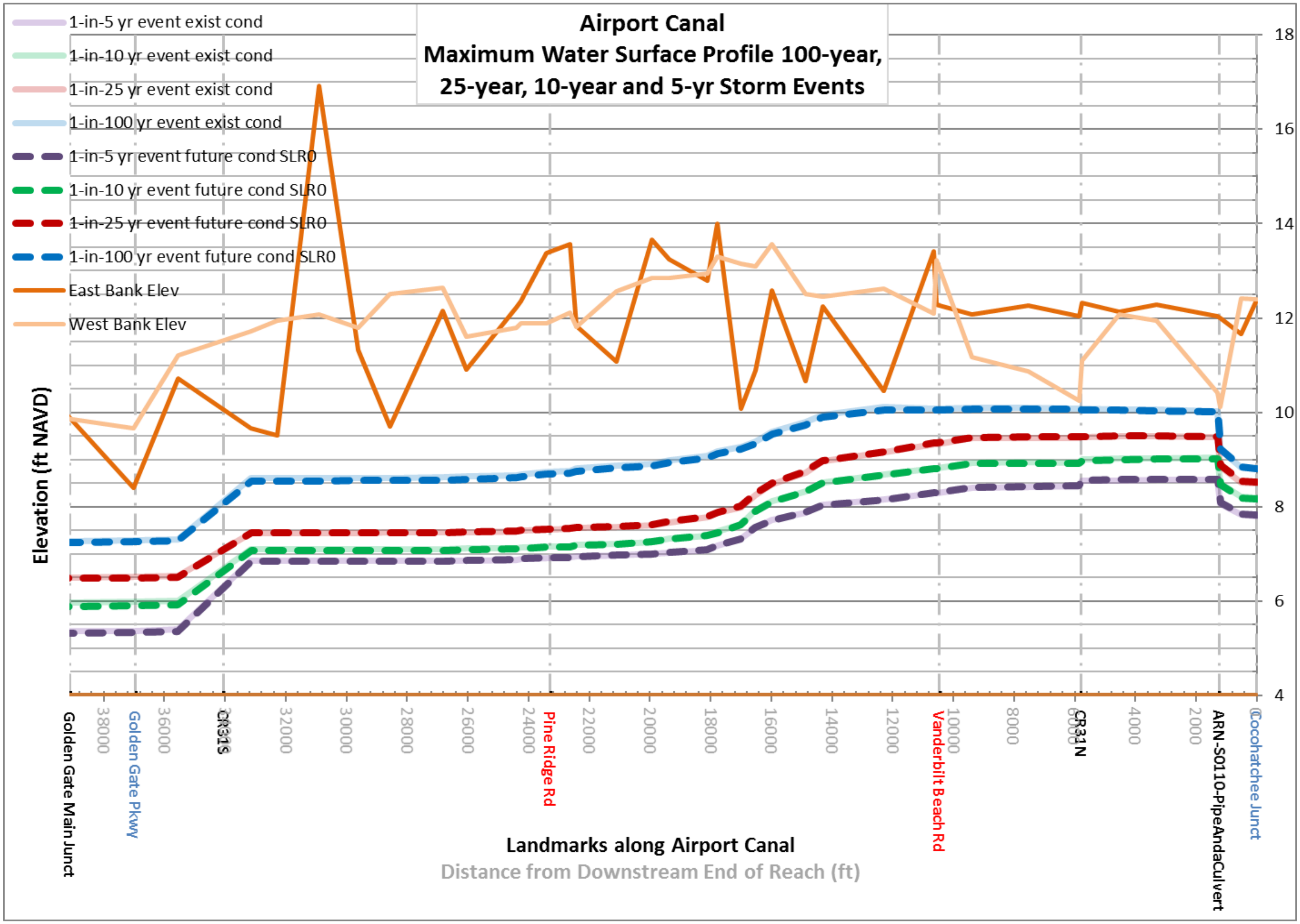


Figure 6.1.2 – Airport Road Canal Peak Stage Profiles, Current and Future Land Use

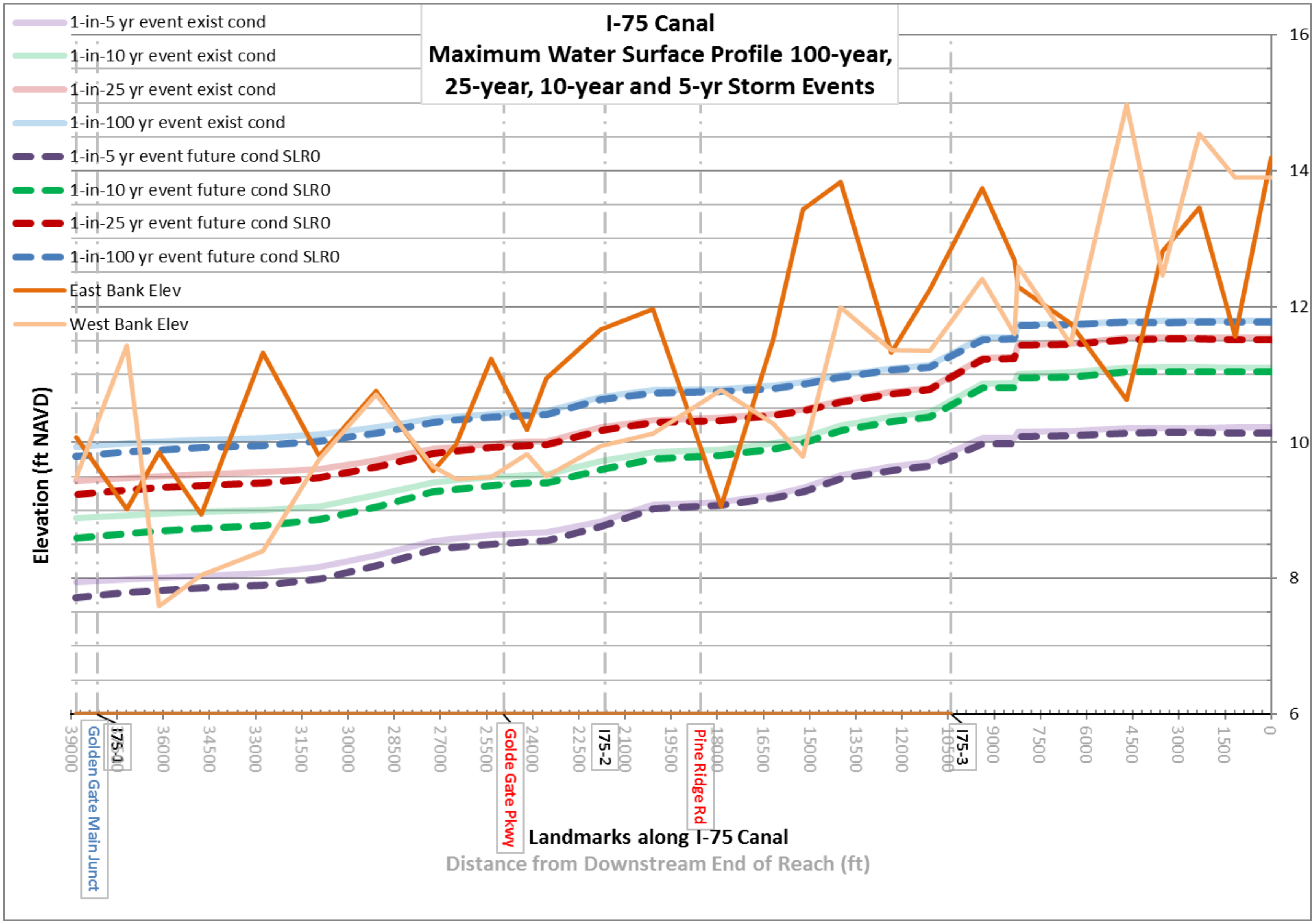


Figure 6.1.3 – I-75 Canal Peak Stage Profiles, Current and Future Land Use

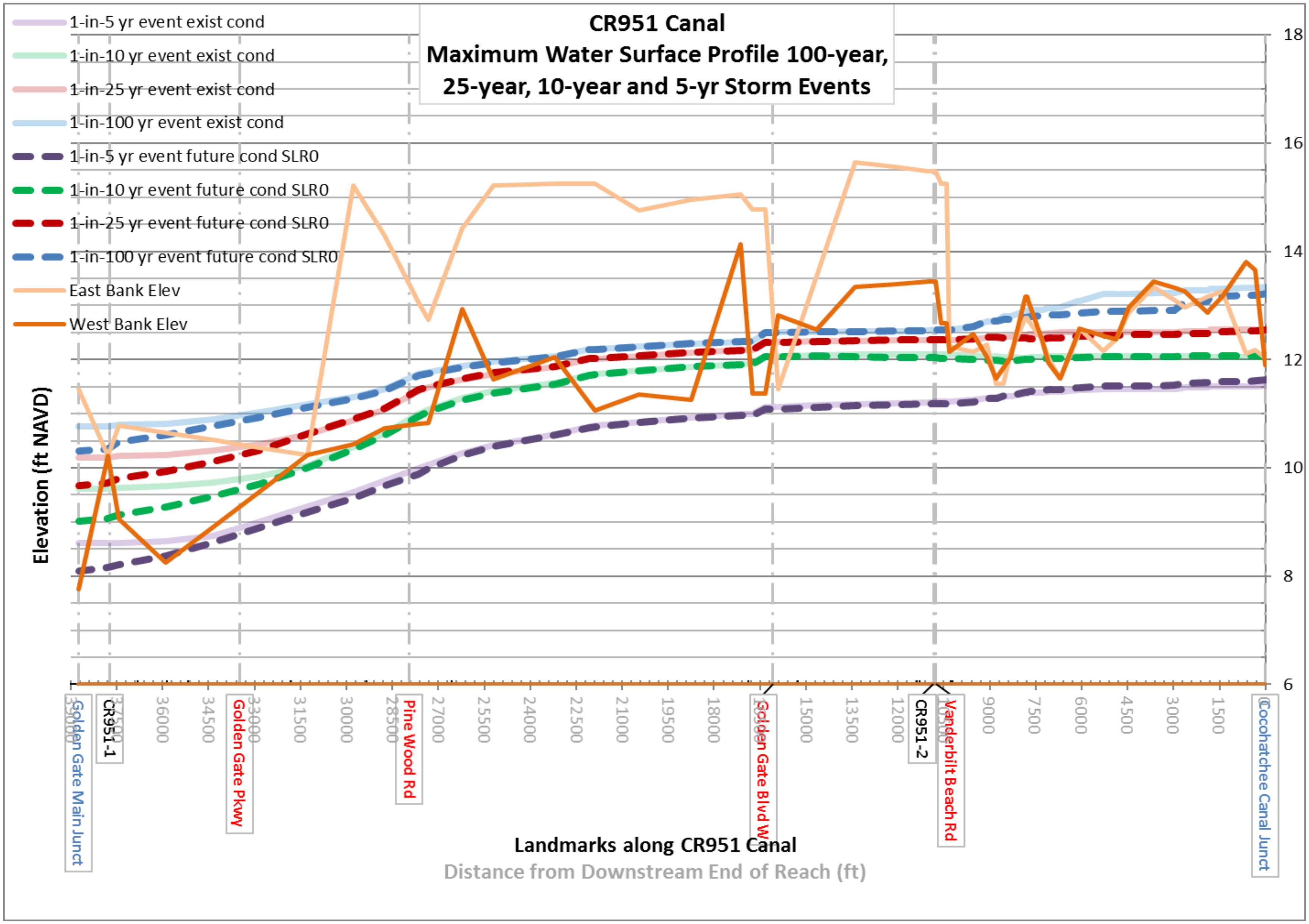


Figure 6.1.4 – CR 951 Canal Peak Stage Profiles, Current and Future Land Use

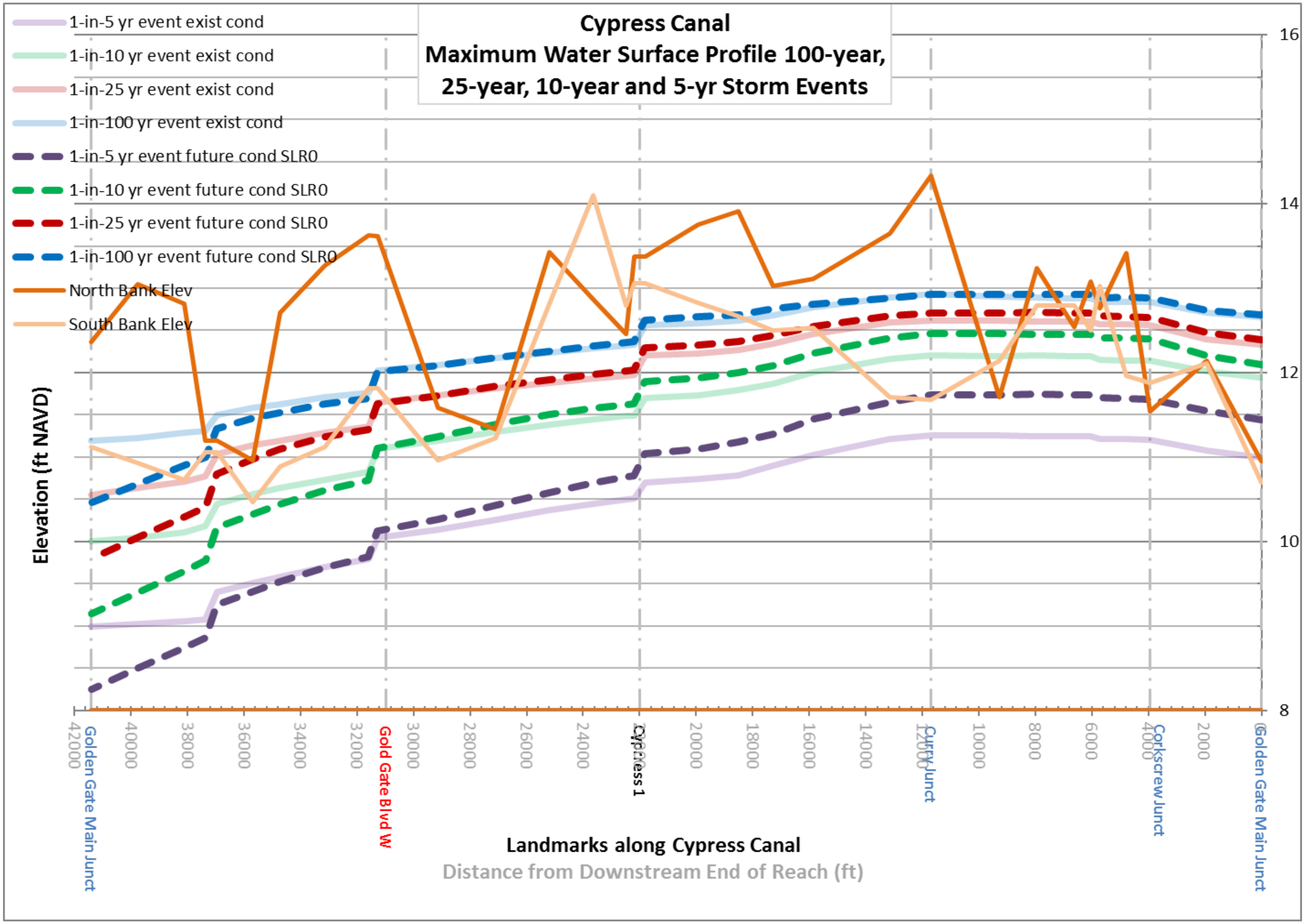


Figure 6.1.5 – Cypress Canal Peak Stage Profiles, Current and Future Land Use

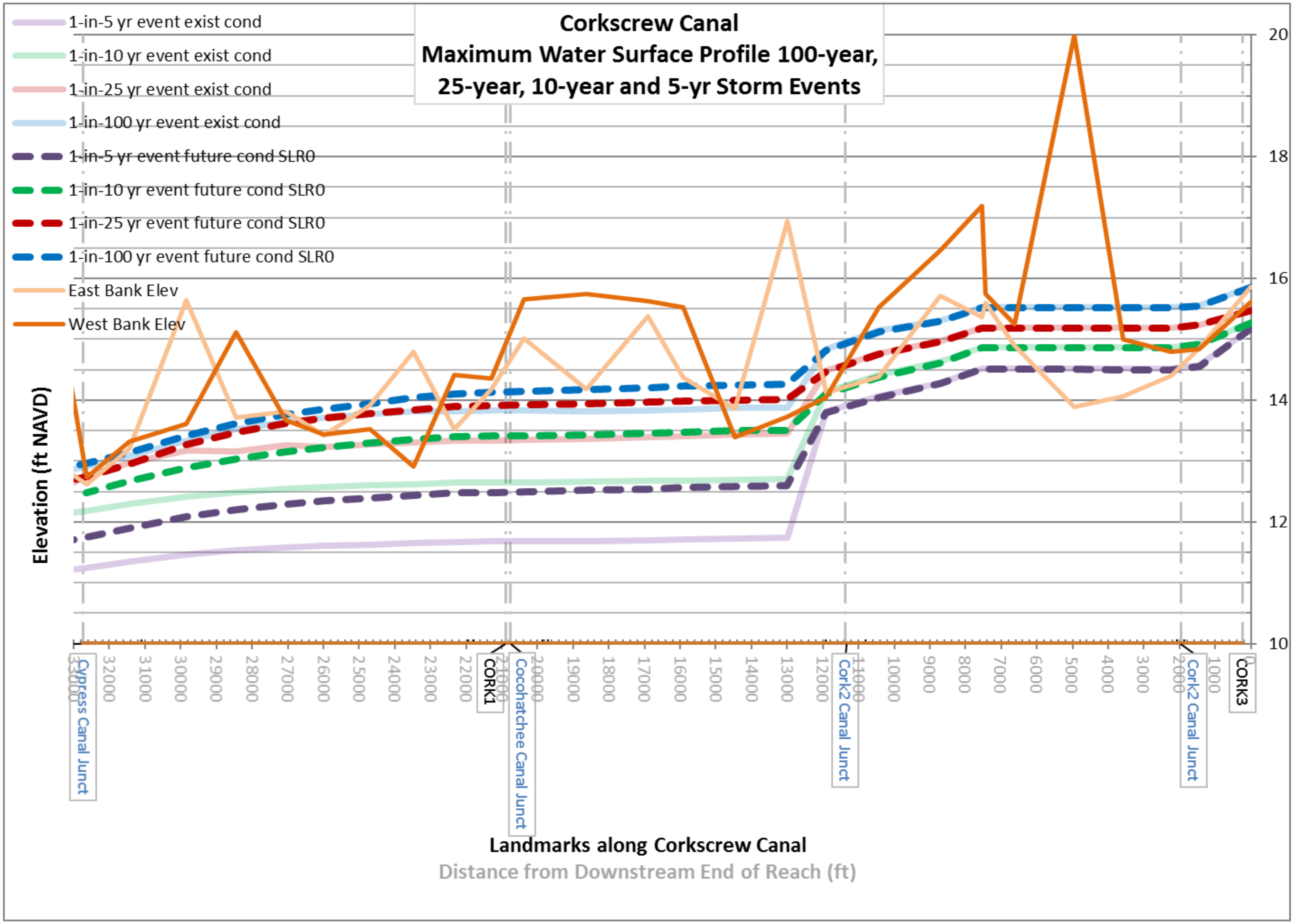


Figure 6.1.6 – Corkscrew Canal Peak Stage Profiles, Current and Future Land Use

6.2 PM #2 – Maximum Daily Discharge Capacity through the Primary Canals, Future Land Use

PM #2 is the maximum discharge capacity throughout the primary canal network. Discharge is calculated for defined canal segments as areally weighted flow, in units of cubic feet per second per square mile of contributing area. Canal segments are generally those segments between water control structures. For example, the segment associated with structure GG1 is the Golden Gate Main Canal between structures GG1 and GG2, and the contributing area is defined as only the area contributing runoff to that segment. **Table 6.2.1** lists the canal segments identified for this analysis, where each segment is identified by the downstream structure. In the case of the Airport Road Canal, the outflows at the north and south structures were combined and treated as if they were a single structure. The table also identifies the contributing area for each canal segment, and the discharge capacity calculated for each segment associated with the 25-year, 3-day design storm event for both current conditions and future land use conditions.

Discharge capacity under current and future land use conditions was calculated by subtracting the hydrographs at all inflow points to each segment from the segment's outflow hydrograph(s), and then dividing the peak of the resulting net discharge hydrograph by the segment's contributing area. Tidal effects were filtered by using a 12-hour moving average of discharge.

Under future land use conditions, the discharge capacity is projected to increase in the canal segments associated with GG1 and GG2. This is a result of the connection of the Preferred Materials mine pits to the Golden Gate Canal upstream of the GG3 structure. The flood attenuation provided by the mine pits would reduce the peak discharge through GG3 and to a lesser extent through GG2. Because the inflow to each of the two canal segments would be reduced, the difference between the peak outflow and peak inflow would increase, resulting in higher discharge capacities.

Canal segments GG6 and GG7 have no inflows, and the outflows would increase due to the potential future land use changes. Thus the computed discharge capacities for these canal segments are higher under future land use conditions. For the segments associated with GG4 and GG5, future land use changes are projected to further increase discharge rates in the canal, also resulting in higher computed discharge capacities for the 25-year event. However, it is important to note that the 10-year, 25-year, and 100-year peak stage profiles in the vicinity of the GG4 structure are out-of-bank, and therefore the true discharge capacity at this structure would be less than the computed 25-year discharge capacity.

At structures I75-1 and CR951-1, lower tailwater conditions in the Golden Gate Canal would create larger head differences across the structures, with corresponding increases in peak discharge rates and discharge capacities under future land use conditions compared to current land use conditions. At CR951-2, higher flows at the northern-most point in the CR951 Canal, coupled with limitations on the capacity of the CR951-2 structure, would result in a reduction in the discharge capacity for this canal segment.

Table 6.2.1 – Water Control Catchment Inflow and Outflow Points and 25-year Discharge Capacity for Current and Future Land Use Conditions

Structure/Segment	Inflow Point(s)	Outflow Point(s)	Water Control Catchment Area	25- year Peak Discharge Capacity	
				Current	Future
			(sq. mi.)	(cfs / sq. mi.)	
GG1	GG2, CR31S	GG1	3.04	88	105
GG2	I75-1, CR951-1, GG3	GG2	3.07	48	76
GG3	CYPRESS1, GG4, C1-CONNECTOR	GG3	28.95	14	13
GG4	GG5, BEGINNING OF CYPRESS CANAL	GG4, MILLER3	6.02	36	51
GG5	GG6, GG7	GG5	8.84	36	45
GG6		GG6	4.34	38	68
GG7		GG7	3.33	23	30
CR31/Airport Rd		AR1, AR2	6.28	24	24
I75-1	I75-2	I75-1	9.71	59	68
I75-2	I75-3	I75-2	4.62	35	39
I75-3		I75-3	4.1	61	64
CR951-1	CR951-2	CR951-1	9.54	25	31
CR951-2	BEGINNING OF CR951 CANAL	CR951-2	1.41	77	49
TWINEAGLE		TWINEAGLE	2.07	24	24
Cypress1	CURRY CANAL, CORKSCREW CANAL, BEGINNING OF CYPRESS CANAL	CYPRESS1	9.81	38	35
CORK1	CORK2, CORK3, TWINEAG	CORK1, BENGINING OF CURRY CANAL	6.26	27	38

Figure 6.2.1 shows the contributing areas ("water control catchments") draining to each canal segment. The catchment polygons were taken from the District's Arc Hydro Enhanced Database (AHED), and were used to determine the contributing areas listed in **Table 6.2.1.**

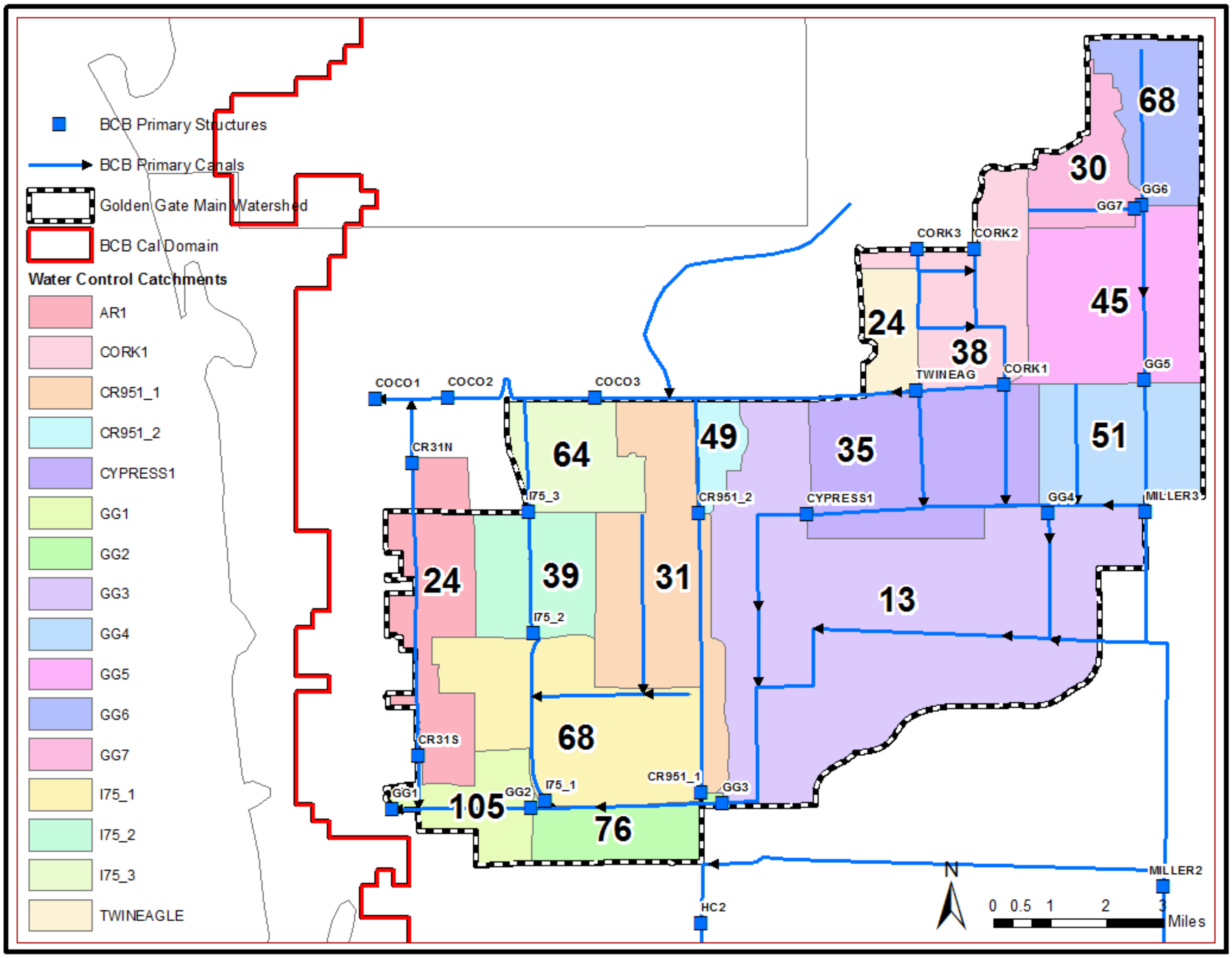


Figure 6.2.1 - Control Structure Catchments in Golden Gate Watershed, for Calculating PM#2, with corresponding 25-year Discharge Capacities for Future Land Use Conditions

6.3 PM #5 – Frequency of Flooding, Future Land Use

In this PM, the flood elevations or depths of overland flooding are evaluated for a range of design storms (5-year, 10-year, 25-year, and 100-year). These flood depths/elevations can then be compared with elevations of features such as buildings and roadways, where such information exists. For the purposes of this BCB FPLOS evaluation, flood inundation maps were developed from the gridded MIKE SHE model output for each storm event, in the form of maximum depths of depth of overland water. These overland water depths, with the 500-foot model grid spacing, were used to develop a TIN for presentation purposes. The resulting flood inundation maps are presented on **Figures 6.3.1 through 6.3.4** for each of the four design storm events.

In order to provide a comparison of the future land use results with the current conditions model results, inundation difference maps were created for the 10-year, 25-year, and 100-year design storm events (**Figures 6.3.5** through **6.3.7**). The current conditions overland flood depths were subtracted from the future land use overland flood depths; positive values mean flood depths would increase and negative values indicate flood depths would decrease under future land use conditions.

In many areas the flood depths are projected to decrease under future land use conditions. This results from a combination of higher topography (refer to **Figure 5.1.7**) and simulated drainage improvements associated with the future land use conversions.

However, there is a low area adjacent to the Golden Gate Canal, upstream of GG4 where the flooding is projected to worsen for the 10-year and 25-year flood events under future land use conditions. This same area was identified in PM #1 (maximum stage profiles) as a portion of the canal where out-of-bank flooding could be exacerbated by future land use changes. As noted in the discussion of the PM #1 results, actual development will differ from the configurations assumed for modeling purposes. Collier County recently changed the allowable post-development discharge for the Main Golden Gate Canal Basin and the Cypress Canal Basins, from 0.15 cfs/acre to 0.04 and 0.06 cfs per acre, respectively (ref: Collier County Ordinance 90-41). This action alone might be sufficient to mitigate the potential increases in flooding predicted by the land use changes simulated in this study.

On **Figure 6.3.7**, an increase in the flood depth within the footprint of the Preferred Materials mine pits is evident; this is a result of floodwaters from the Golden Gate Canal backing up into the mine pits in the future land use 100-year design storm scenario. Reductions in flood levels upstream and downstream of the pits, along and adjacent to the Golden Gate Canal, are also evident on this map.

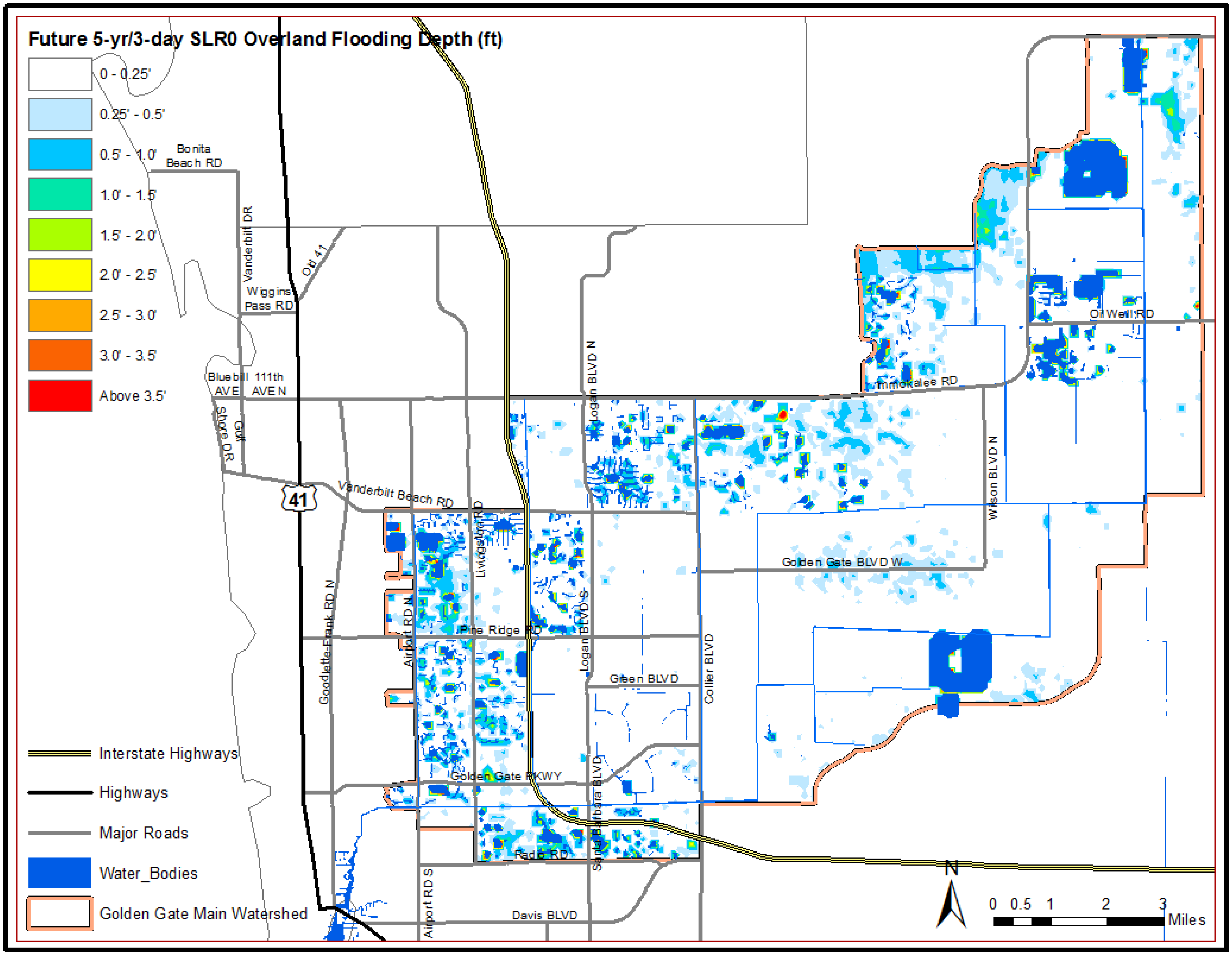


Figure 6.3.1 – Future Land Use Conditions Inundation Map for 5-year Design Storm Event, Golden Gate Watershed

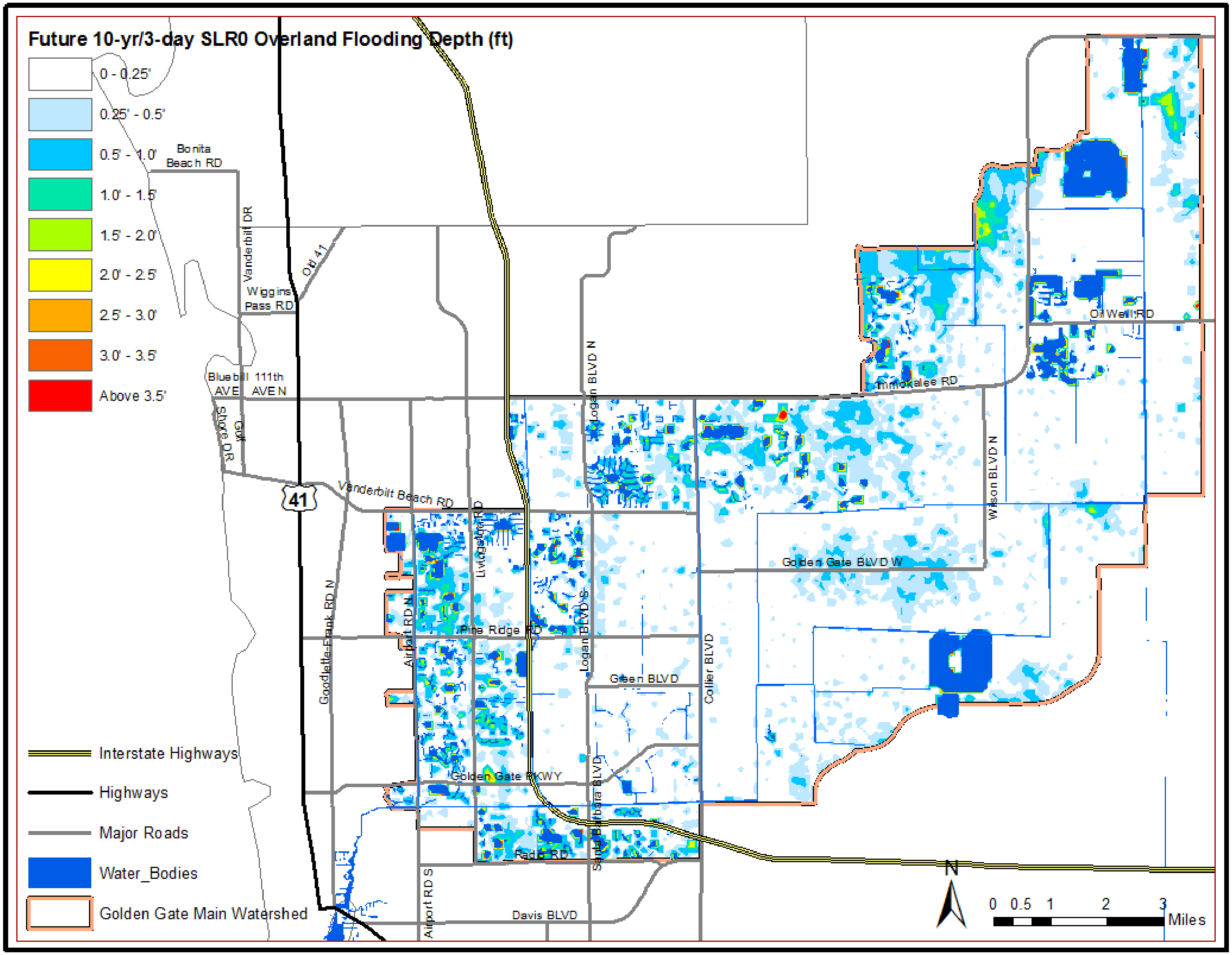


Figure 6.3.2 – Future Land Use Conditions Inundation Map for 10-year Design Storm Event, Golden Gate Watershed

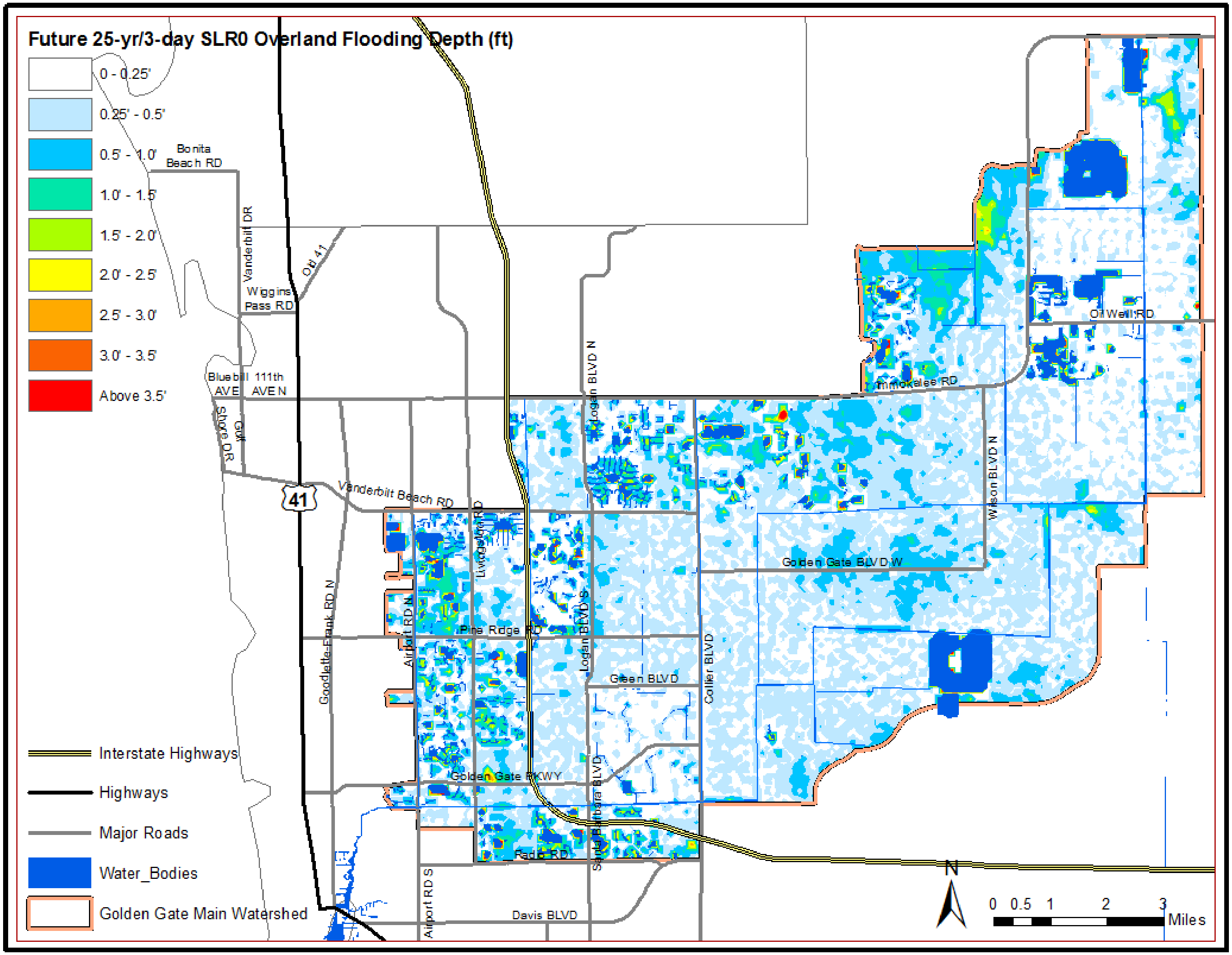


Figure 6.3.3 – Future Land Use Conditions Inundation Map for 25-year Design Storm Event, Golden Gate Watershed

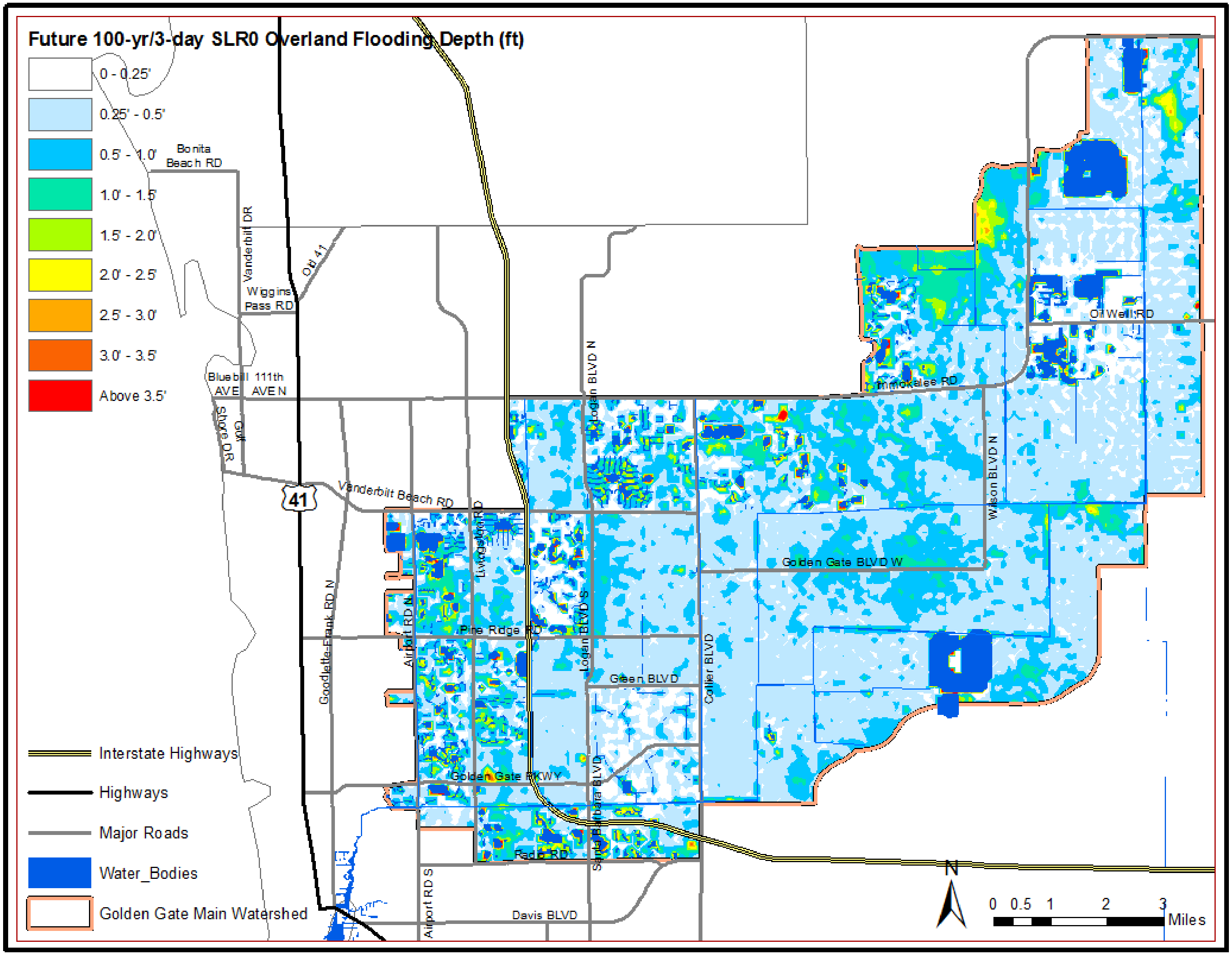


Figure 6.3.4 – Inundation Map for 100-year Design Storm Event, Golden Gate Watershed

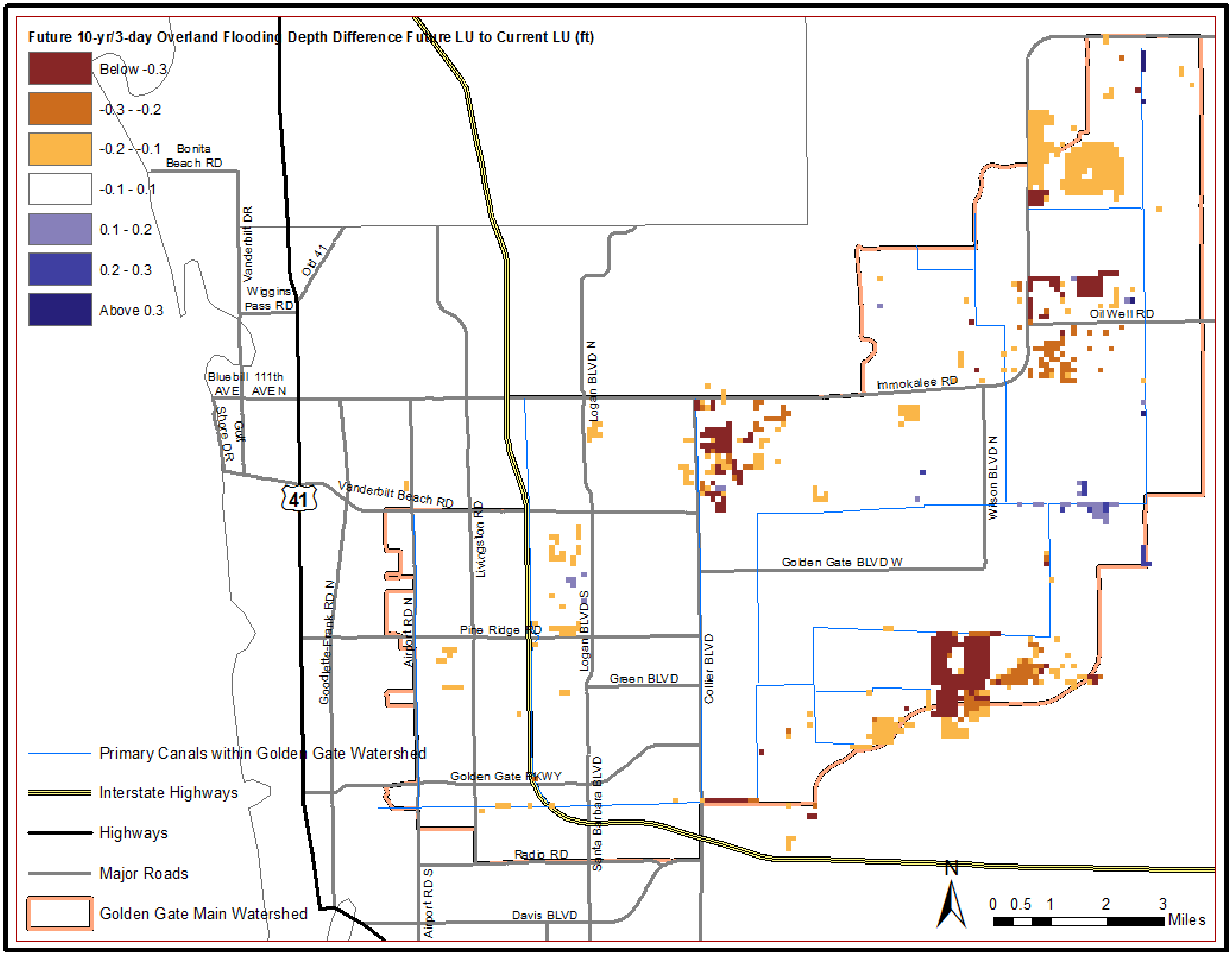


Figure 6.3.5 – Golden Gate 10-Year Inundation Difference Map – Future LU minus Current LU

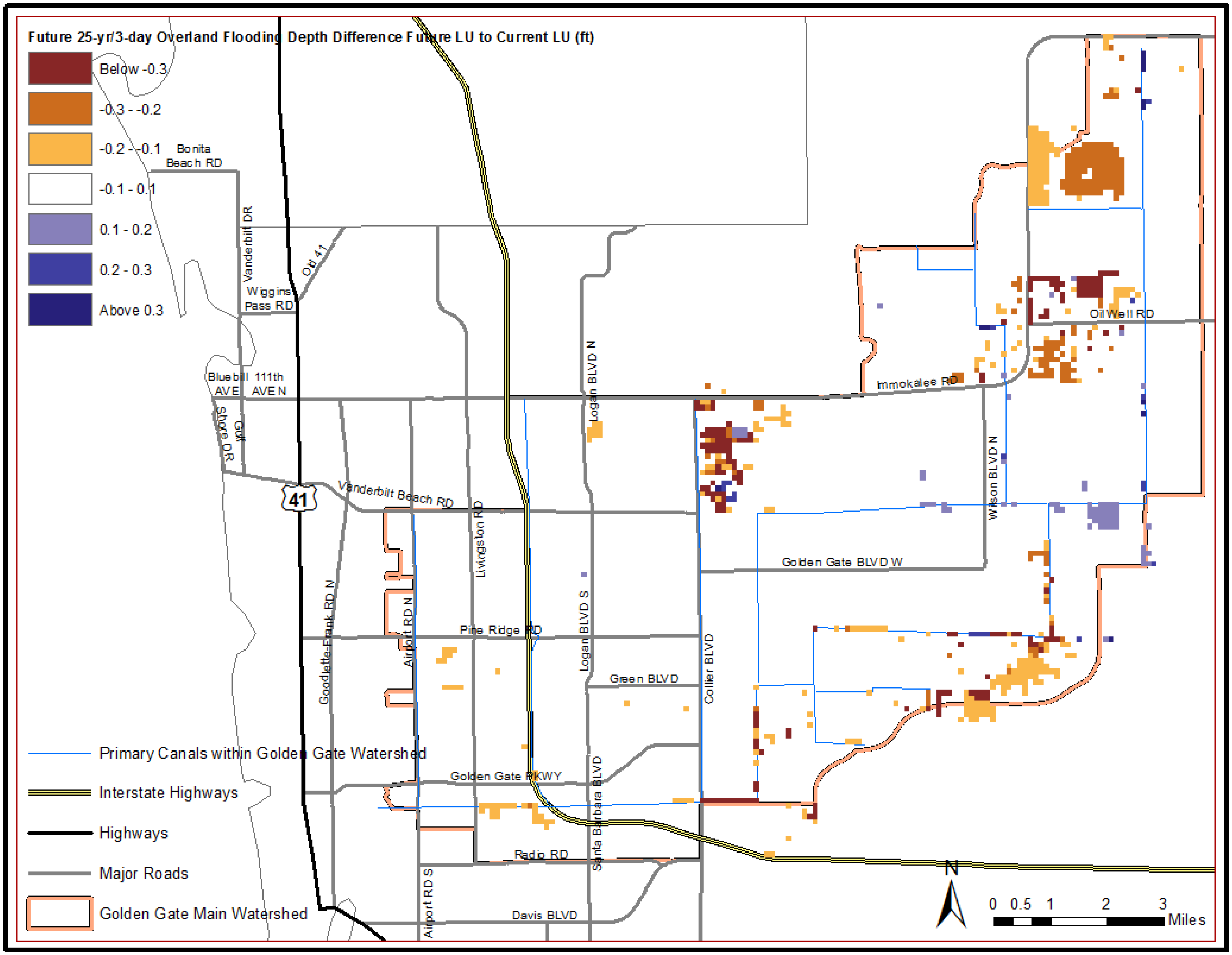


Figure 6.3.6 – Golden Gate 25-Year Inundation Difference Map – Future LU minus Current LU

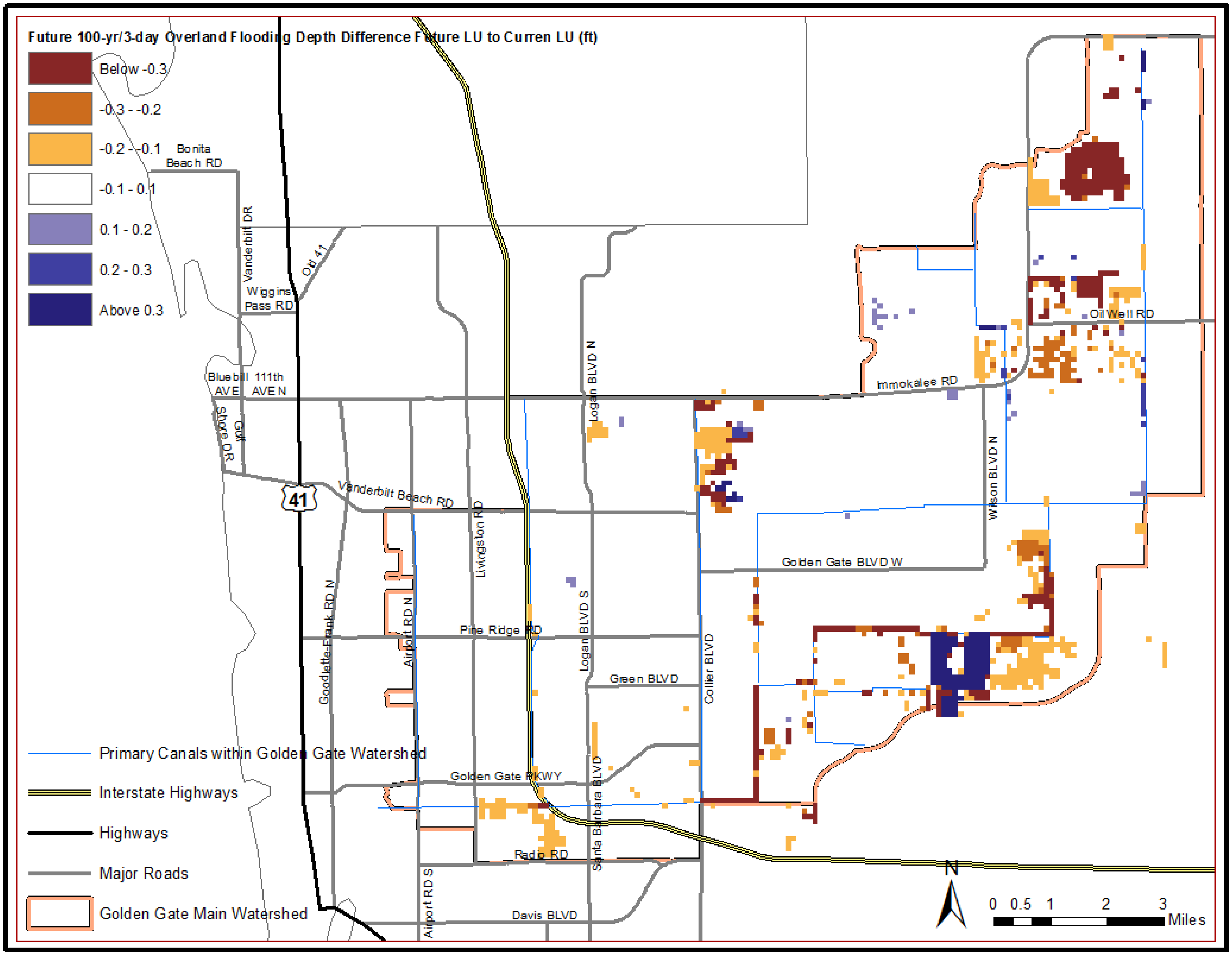


Figure 6.3.7 – Golden Gate 100-Year Inundation Difference Map – Future LU minus Current LU

6.4 PM #6 – Duration of Flooding, Future Land Use

PM #6 quantifies the duration of flooding at specific locations of interest within a watershed. For this metric, potentially flood-prone locations associated with existing or future development are identified, and a representative threshold elevation corresponding to building or roadway flooding is chosen. The length of time the flood elevation is projected to be above that elevation is the duration of flooding. This metric requires the selection of specific areas of interest. For the Golden Gate Watershed, six areas of interest were identified in consultation with District staff. These locations are shown **on Figure 6.4.1**.

At each location of interest it was necessary to identify a target or threshold stage which, when exceeded by water levels in the nearby canal, would cause flooding of streets or buildings. Generally speaking, building pads within the watershed are higher than the adjacent roadways, so the target stages represent road and street flooding thresholds. To develop the target stage, several representative road and street locations were identified within the area of interest, and the pavement elevations (estimated from the LiDAR data) were averaged to develop the representative target elevation for each of the six locations. Stage hydrographs from the MIKE-11 output for each storm event were then compared with the target stages, to compute the duration of time target stages were exceeded. This was done for both current and future land use conditions. The results of the comparisons are depicted in tabular form on **Table 6.4.1**.

Location 1 is upstream of the GG-1 structure and north of the Golden Gate Canal in the Bears Paw development. At this location the topography is low and flooding results in part from the high tailwater condition, which at this location is impacted by storm surge. The simulated 100-year tailwater boundary condition at GG-1 is 6.9 feet NAVD88 for SLR0, which is about 2 feet above the surrounding topography. Predicted flood durations here range from 3.5 hours for the 5-year event to almost 4 days for the 100-year event. In contrast, streets and roads at locations 2 and 3 are above the simulated peak flood levels in the adjacent canals (Airport Road and I-75 Canals, respectively) for all storm events. These durations do not change significantly under future land use conditions.

Location 4 (Golden Gate Estates east of the CR951 Canal) is predicted to have no flooding for the 5-year event, and only minor flooding (duration of 30 minutes) for the 10-year event. For the 25-year and 100-year events, the predicted durations of flooding at location 4 are 2 hours 1.5 days, respectively for current conditions. Under future land use conditions, the 100-year duration is projected to decrease to about 4 hours. This is due to the flood relief that would be provided by connecting the Preferred Materials mine pits to the Golden Gate Canal as part of the conceptual future land use modeling.

Location 5 is adjacent to the downstream portion of the Cypress Canal, within Golden Gate Estates. No flooding is predicted here for the 5-year and 10-year storm events. However, significant flooding with durations of 2 hours and 2.25 days are predicted for the 25-year and 100-year events, respectively. Under future land use conditions, a slight decrease in flood duration is predicted to occur for the 100-year event, also due to connecting the Preferred Materials mine pits to the Golden Gate Canal.

Location 6 is adjacent to the Cypress Canal near structure GG-4, in Golden Gate Estates. Because it is a topographically low area in the upstream reaches of the watershed, flooding is predicted to occur for all four design storm events. The predicted durations of flooding at Location 6 range from 1 hour for the 5-year event to 5 days for the 100-year event. The duration is predicted to increase slightly under future land use conditions due to increased volumes of runoff from upstream development.

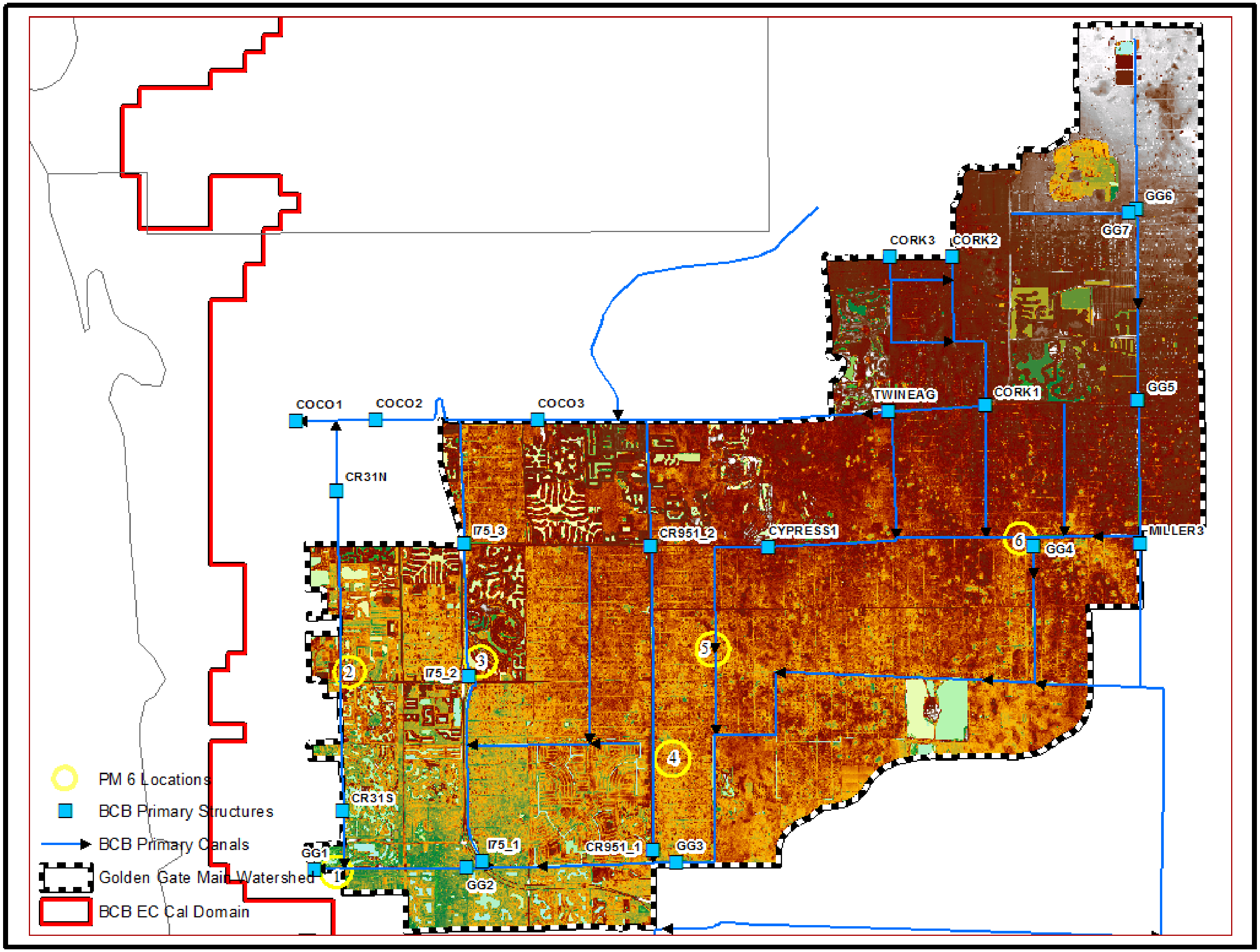


Figure 6.4.1 – Locations of Flood Duration Evaluation Points and LiDAR DEM

Table 6.4.1 – Comparison Table of Flooding Duration (days), Current Land Use and Future Land Use

Location	Canal	Chainage (ft)	5-yr		10-yr		25-yr		100-yr	
			Current	Future	Current	Future	Current	Future	Current	Future
1	Golden Gate Main	138,513	0.15	0.2	1	1.5	2	1.75	3.75	3.75
2	Airport	11,696	0	0	0	0	0	0	0	0
3	I75	20,082	0	0	0	0	0	0	0	0
4	CR951	28,754	0	0	0.02	0.02	0.08	0.08	1.5	0.17
5	Cypress	34,724	0	0	0	0	0.08	0.08	2.25	1.5
6	Cypress	1,936	0.04	0.17	0.2	0.35	2	2.25	5	5.75

7.0 Flood Protection Level of Service – Future Sea Level

Future sea level rise was simulated for the 25-year, 3-day design storm event only. Using the model developed for future land use conditions, three future sea level rise scenarios were simulated (SLR1, SLR2, and SLR3). The model setup for these scenarios was previously described in **Section 5.2**.

Appendix A provides summary model results at primary control structures, while **Appendices J** through **L** provide the complete flow and stage hydrographs for each of the three sea level rise scenarios. The performance of the existing infrastructure with future land use changes and future sea level rise was evaluated with respect to the five performance metrics described in the remainder of this section.

7.1 PM #1 – Maximum Stage in Primary Canals, Future Sea Level

Figures 7.1.1 through **7.1.5** depict the maximum 25-year stage profiles in the primary canals under future land use and future sea level rise conditions. The future land use with current sea level (SLR0) profiles are also provided on the plots for reference. As expected, the largest increases in maximum stages would be under SLR3 conditions at the downstream end of the Golden Gate Canal, with smaller increases at the southern ends of the Airport Road, I-75, and CR951 Canals where they connect to the Golden Gate Canal. Within the Golden Gate Canal, the maximum stage increase is approximately 1.7 feet under SLR3 (immediately upstream of GG1). Significant stage increases under the SLR3 scenario would extend several miles upstream of GG1, to beyond the GG3 structure.

More modest increases in maximum stage are projected to occur under SLR1 and SLR2. Under SLR2, the maximum increase is projected to be approximately 0.5 feet upstream of GG1; this increase rapidly diminishes to approximately 0.1 feet at the GG2 structure. Stage increases associated with the SLR1 scenario are projected to be minimal within the Golden Gate Watershed, as there is almost no difference between the SLR1 and SLR0 profiles.

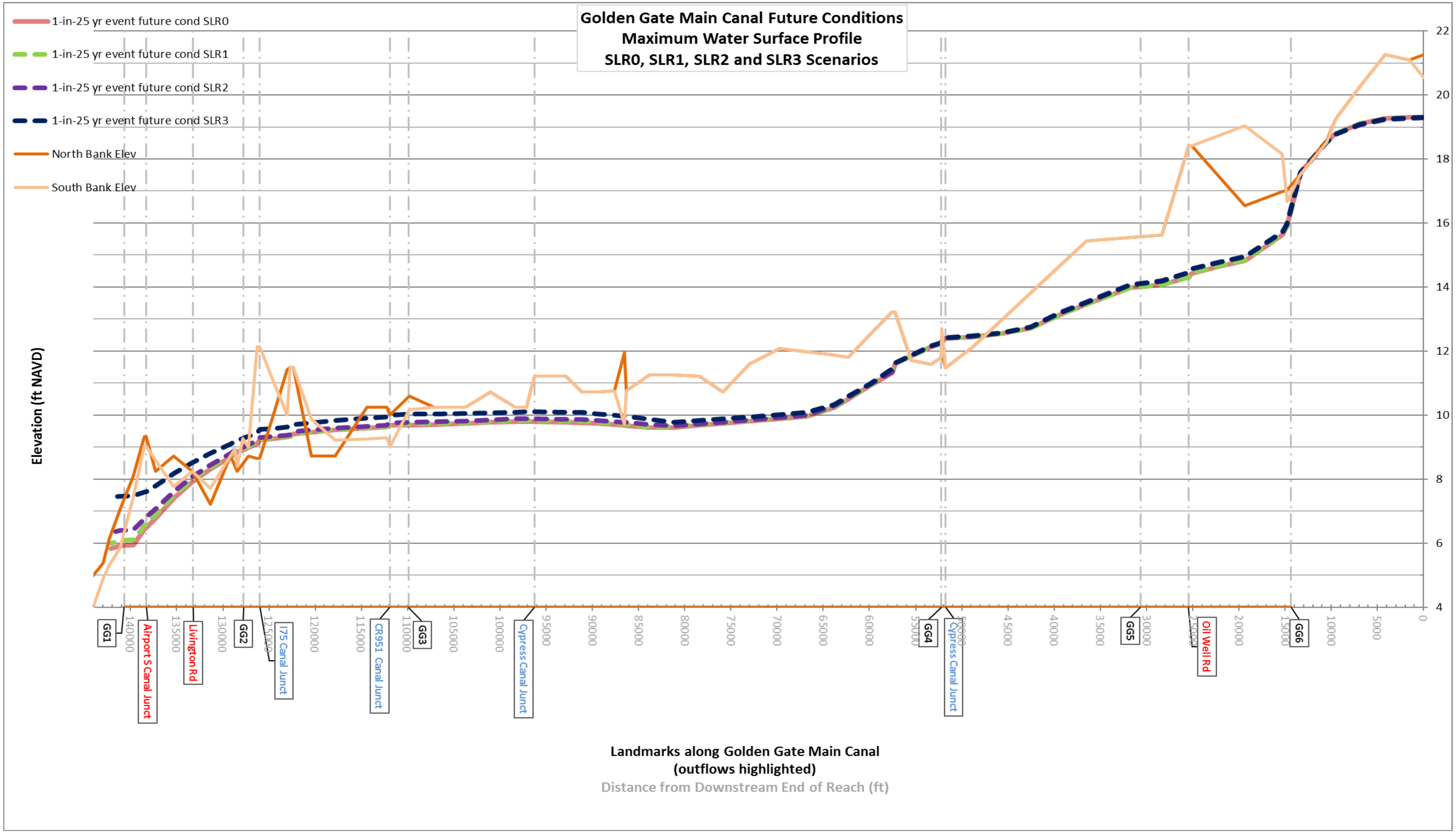


Figure 7.1.1 – Golden Gate Main Peak Stage Profiles – Current and Future Sea Level Scenarios

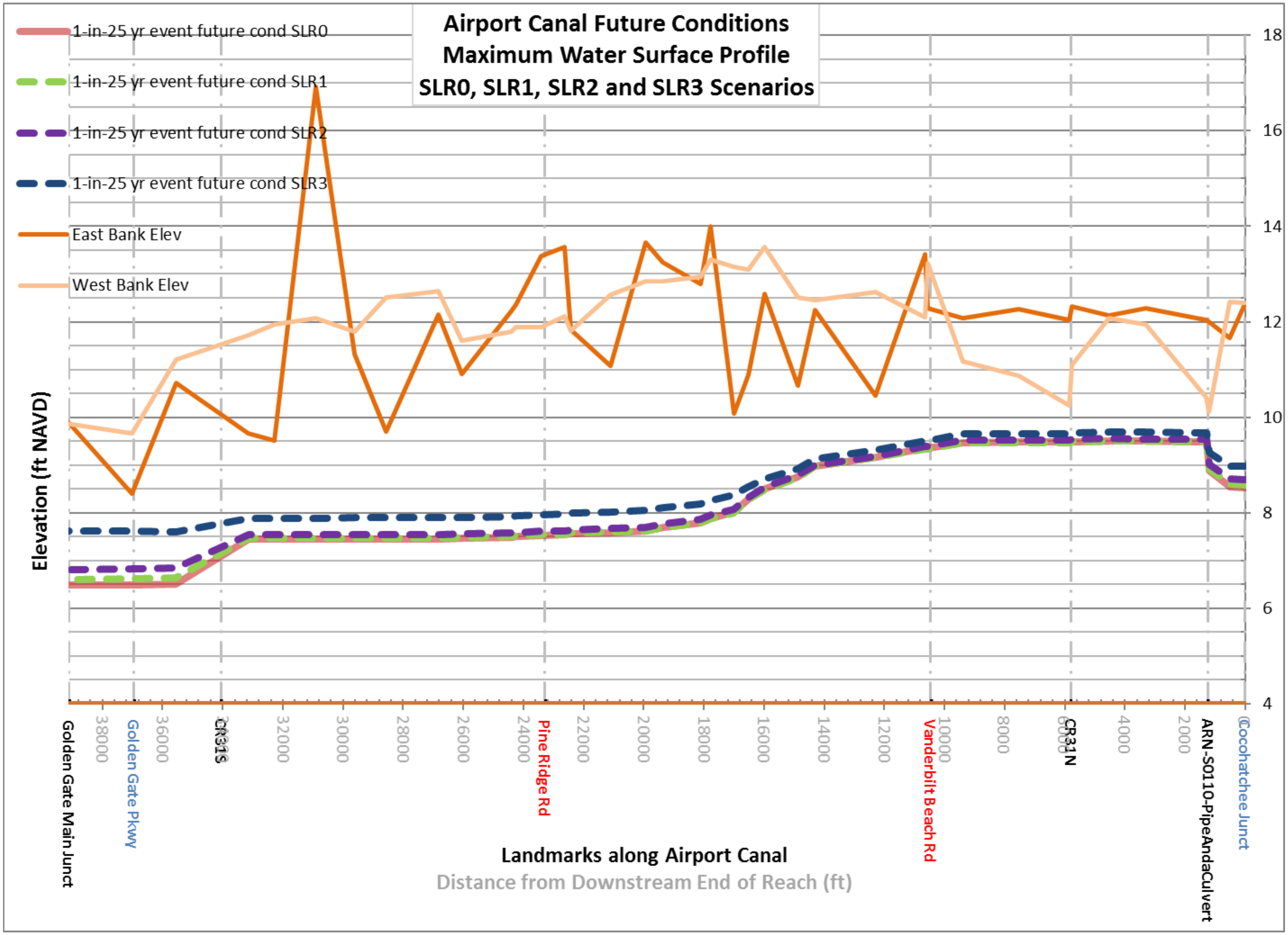


Figure 7.1.2 – Airport Road Canal Peak Stage Profiles – Current and Future Sea Level Scenarios

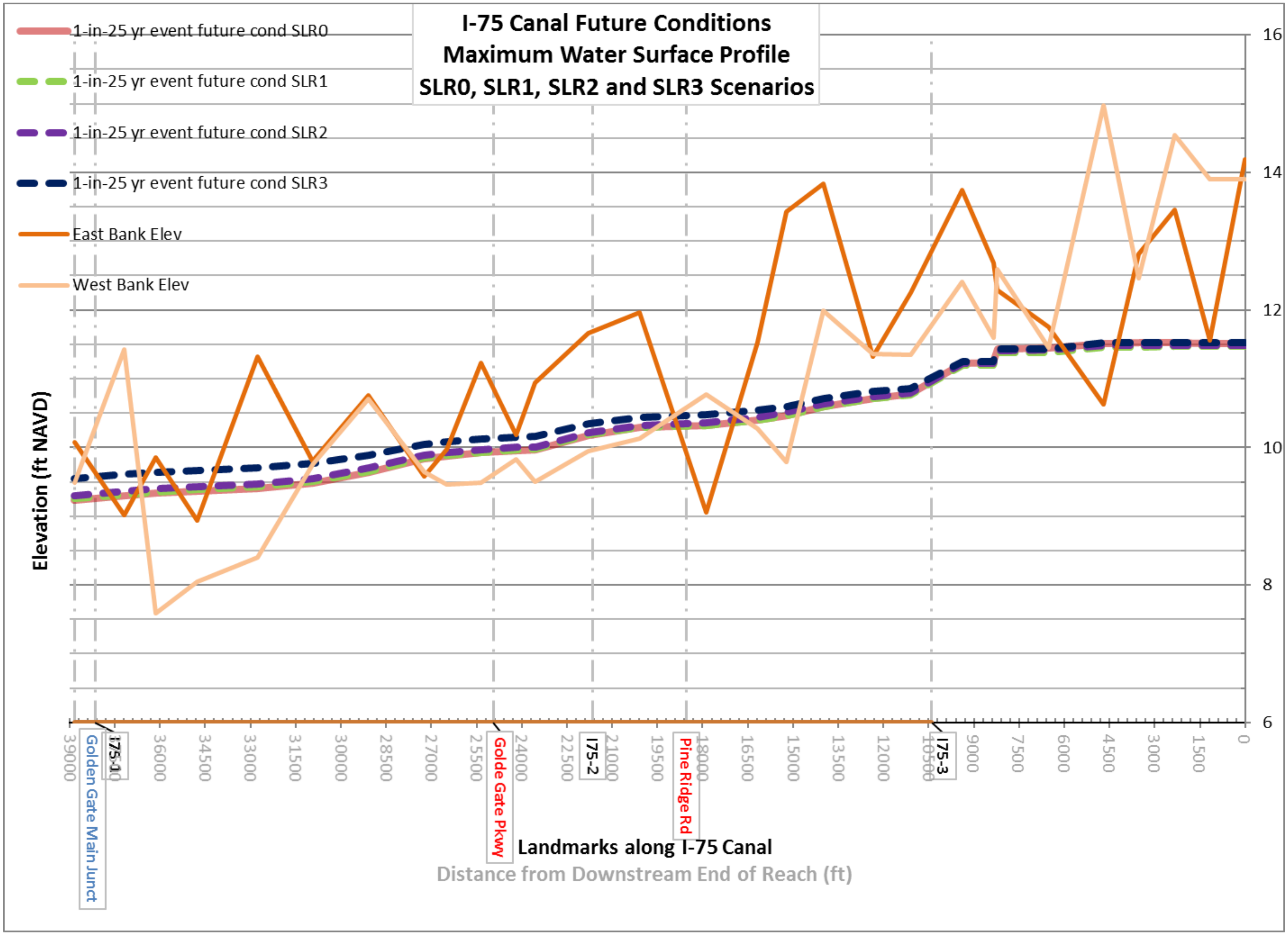


Figure 7.1.3 – I-75 Canal Peak Stage Profiles – Current and Future Sea Level Scenarios

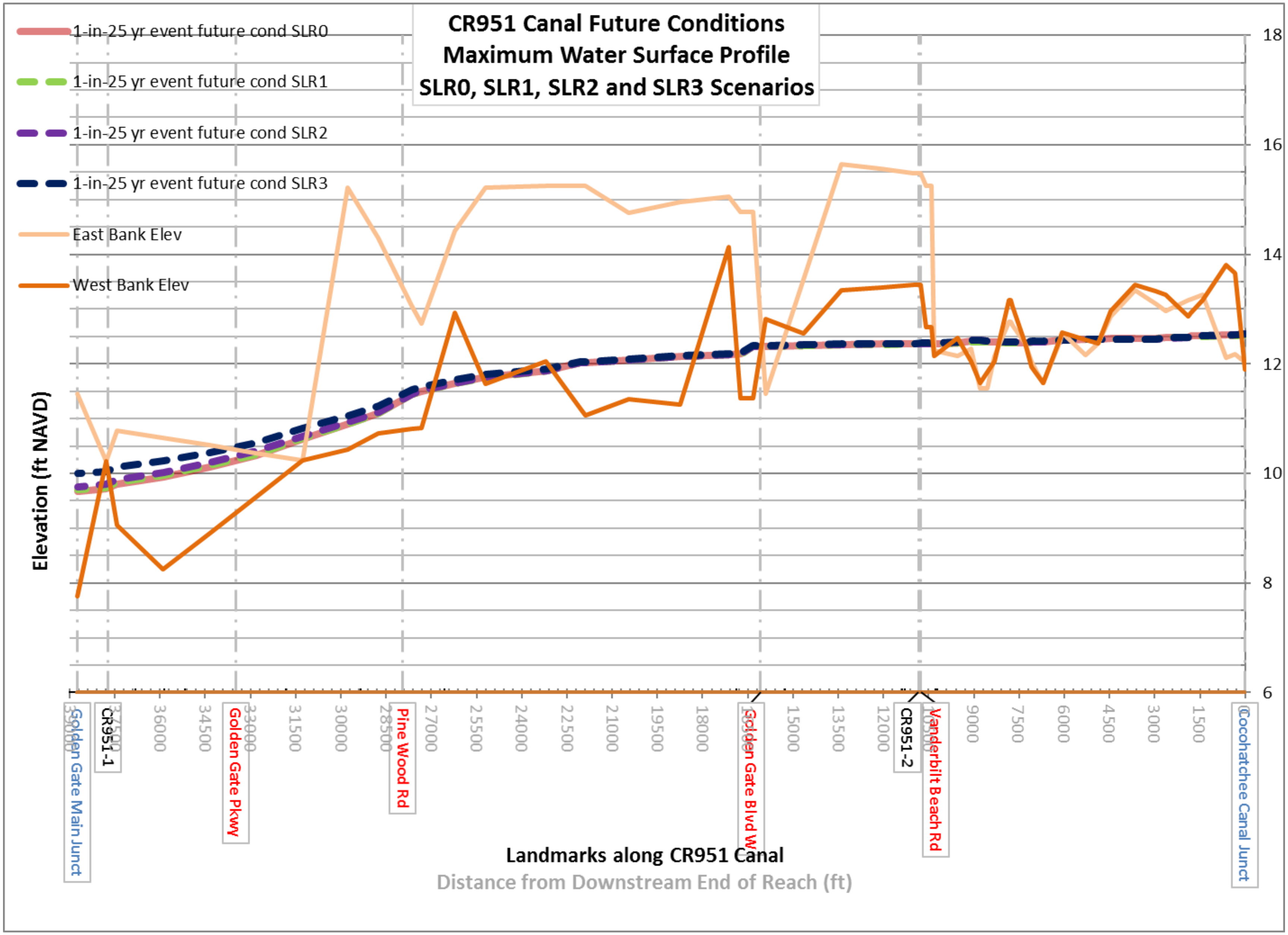


Figure 7.1.4 – CR 951 Canal Peak Stage Profiles – Current and Future Sea Level Scenarios

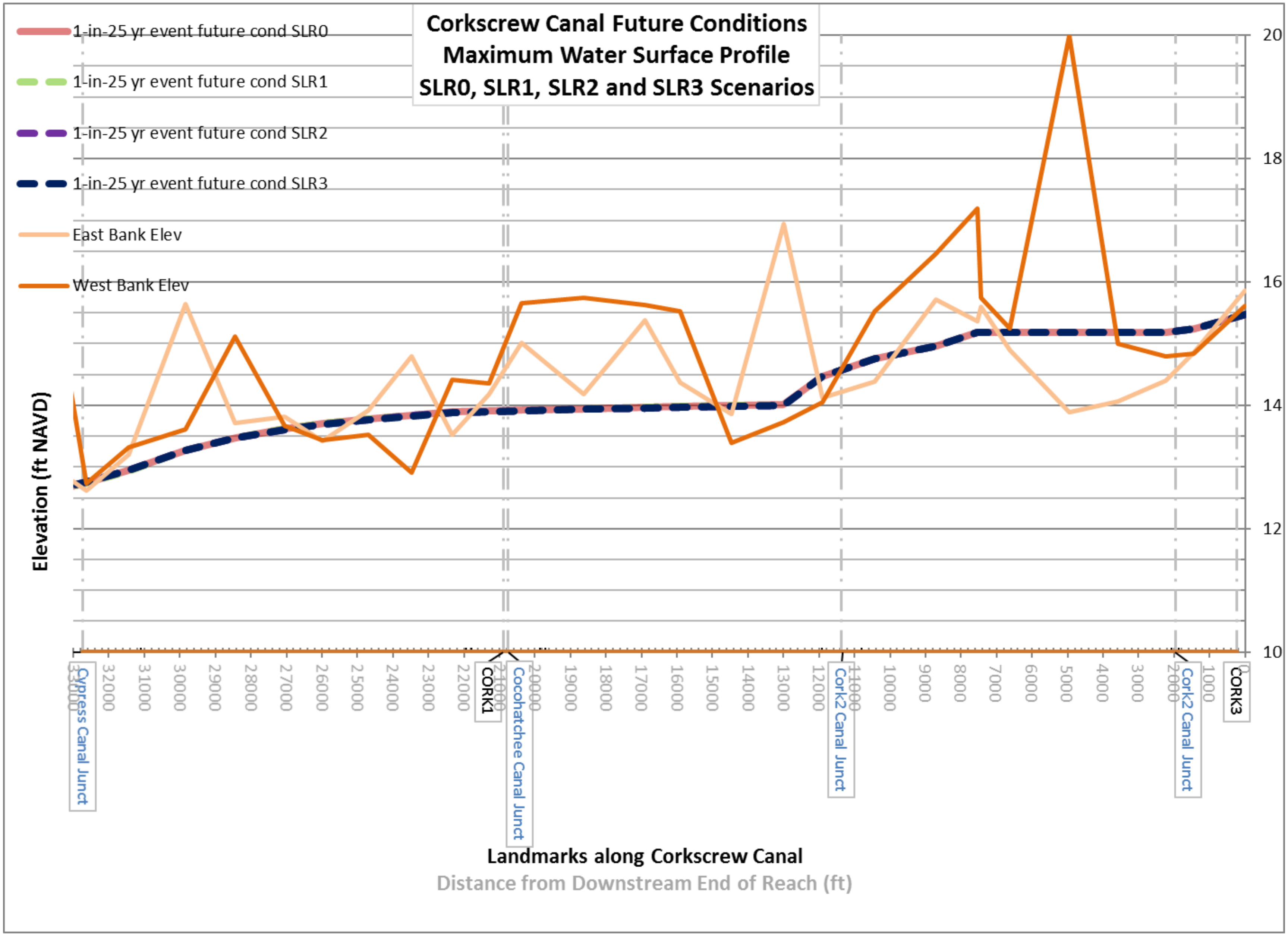


Figure 7.1.5 – Corkscrew Canal Peak Stage Profiles – Current and Future Sea Level Scenarios

7.2 PM #2 – Maximum Daily Discharge Capacity through the Primary Canals, Future Sea Level

Maximum discharge capacity for the 25-year design storm event was calculated for each of the three future sea level scenarios (SLR1, SLR2, and SLR3) and compared to the future land use conditions model results for SLR0 (refer to the Future column in **Table 6.2.1**). The discharge capacities for the canal segments evaluated were essentially unaffected by the sea level rise scenarios, except for the GG1 segment. Under SLR3, the discharge capacity was reduced from 105.4 cfs per square mile to 101.6 cfs per square mile (less than a 4% decrease).

7.3 PM #4 – Peak Storm Runoff – Effects of Sea Level Rise

PM #4 is the maximum conveyance capacity of a watershed at the tidal structure for a range of design storms. It shows the maximum conveyance (moving 12-hour average) for a specific design storm and a specific tidal boundary condition. This metric examines the behavior of the system under severe stress and can be used to check if conditions exceed design limits. In evaluating this PM, it is assumed that design rainfall and design storm surge occur simultaneously.

Figure 7.3.1 provides a graph of the flow vs. return period for current sea level conditions (SLR0). The 25-year results for SLR3 are also shown on the graph. The higher tailwater elevations associated with SLR1 and SLR2 would result in negligible decreases (less than 2%) in discharge capacity, while SLR3 would result in a reduction in discharge capacity of approximately 100 cfs, about a 4% reduction.

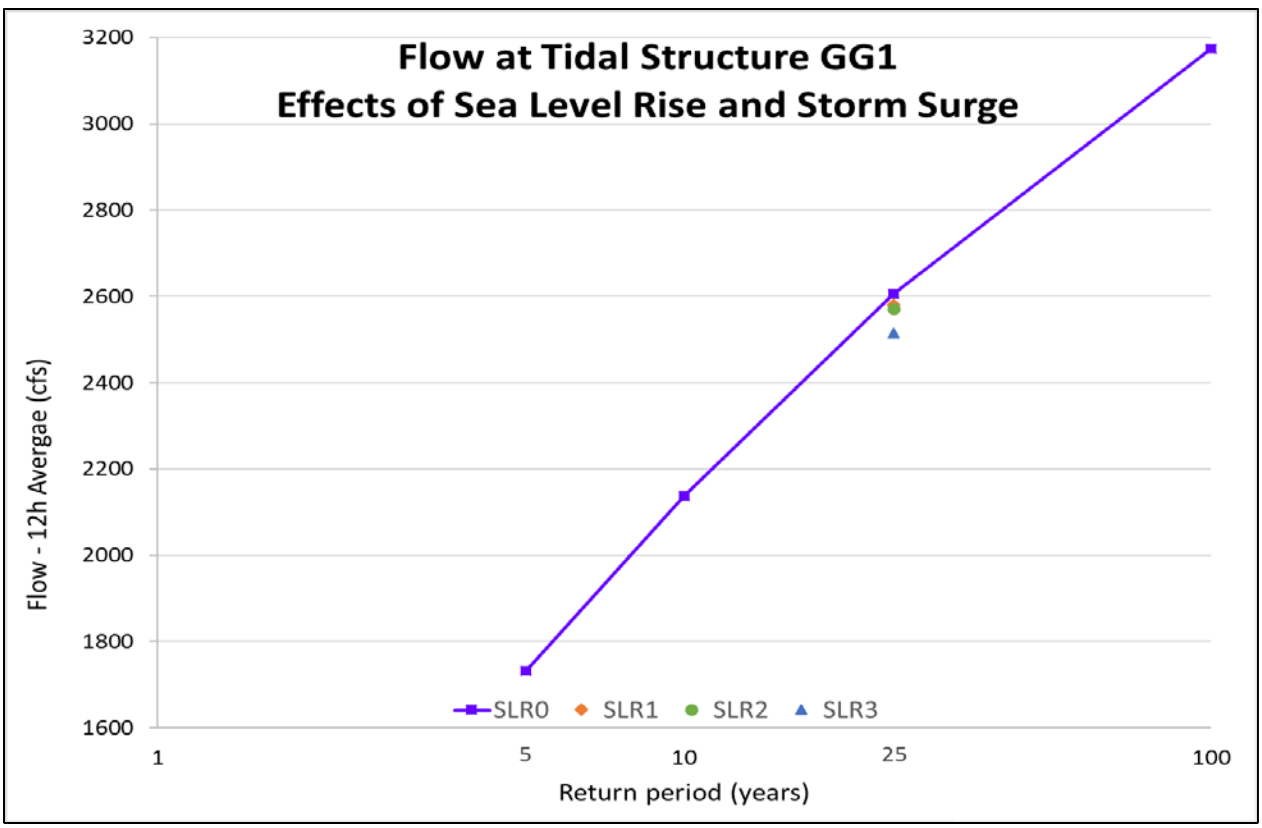


Figure 7.3.1 – Flow at Tidal Structure GG1

7.4 PM #5 – Frequency of Flooding, Future Sea Level

The three future sea level rise scenarios evaluated in this modeling effort would result in little or no change to the frequency or depths of flooding throughout the majority of the Golden Gate Watershed for the 25-year design storm event. Part of the reason for this is that the western boundary of the Golden Gate Watershed is two or more miles inland from the coast, where the largest impacts are likely to occur. The inundation maps prepared for SLR1, SRL2, and SLR3 are therefore very similar to the one prepared for SLR0 (refer to **Figure 6.3.3**, Future Land Use Conditions Inundation Map for 25-year Design Storm Event).

Under the SLR3 scenario, some localized and/or relatively small increases in the 25-year inundation depths would occur within the bounds of the Golden Gate Watershed (**Figure 7.4.1**). The largest increases (up to 0.4 feet) within the Golden Gate Watershed would occur adjacent to the Golden Gate Canal, west of I-75. East of I-75, the increases would be limited to less than 0.2 feet. However, these small increases would extend upstream past the GG3 structure to the Preferred Materials mine pits. Under SLR1 and SLR2 scenarios, the differences in inundation depths for the 25-year event would be negligible.

Another way to measure inundation impacts due to sea level rise is to consider maximum water table stages. Higher water table stages can affect infrastructure such as septic tanks, older, unlined sanitary sewers (through higher infiltration), and roadway base material. Pronounced differences in maximum water table stages are evident in coastal areas west and south of the Golden Gate Watershed. **Figures 7.4.2** through **7.4.4** show differences in peak water table elevations throughout the model domain for sea level rise scenarios SLR1, SLR2, and SLR3 respectively, relative to SLR0.

Within the Golden Gate Watershed, the largest and most widespread increases in maximum water table stage would occur under the SLR3 scenario, in an area west of Collier Blvd and south of Pine Ridge Road, with increases on the order of 0.3 feet in places. Under the SLR2 scenario, impacts would occur over a smaller area and would generally be limited to less than 0.2 feet, while the SLR1 scenario would result in no appreciable increase in maximum water table stage within the Golden Gate Watershed for the 25-year event.

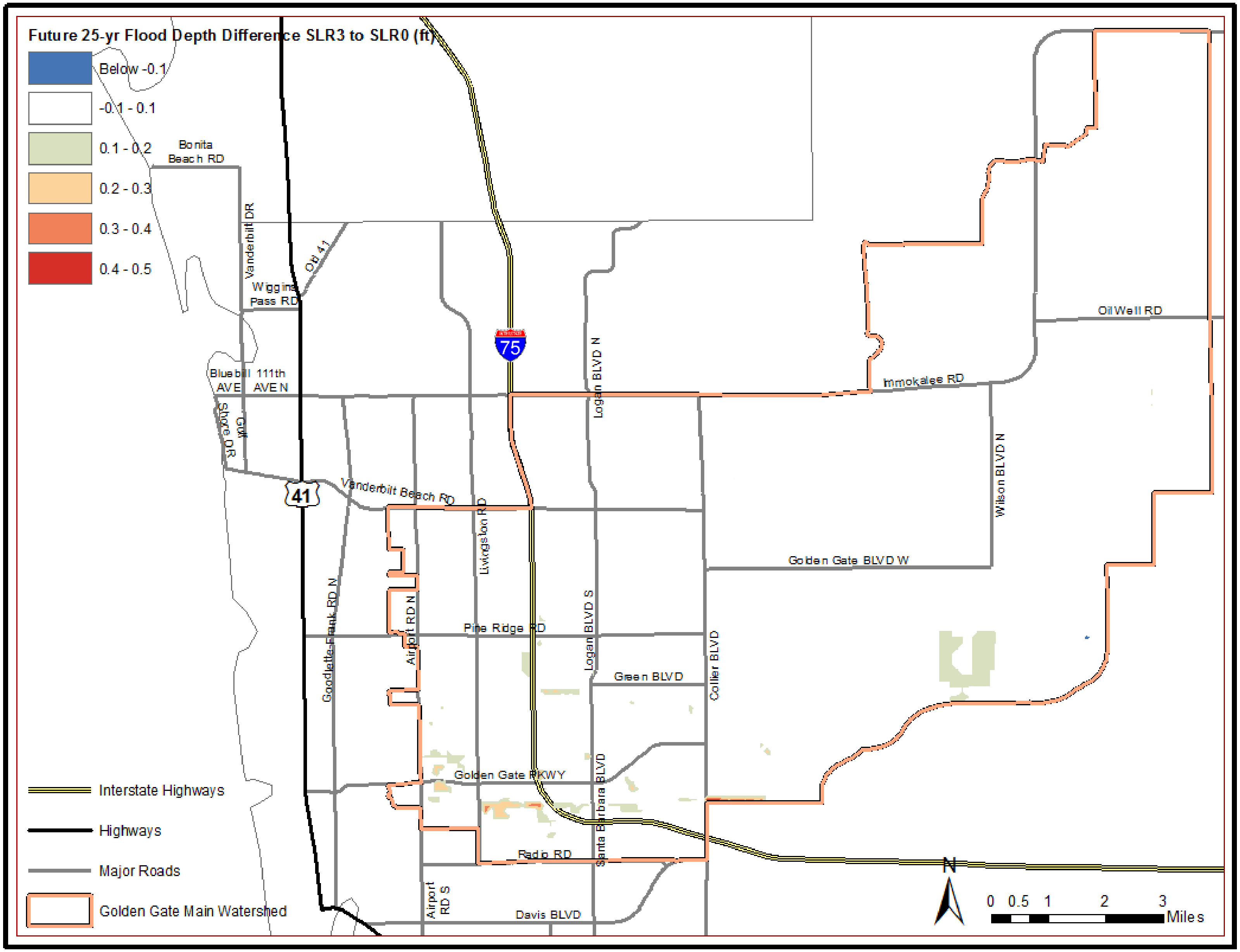


Figure 7.4.1 – Golden Gate 25-Year Inundation Difference Map – Future SLR3 minus Current Sea Level (SLR0)

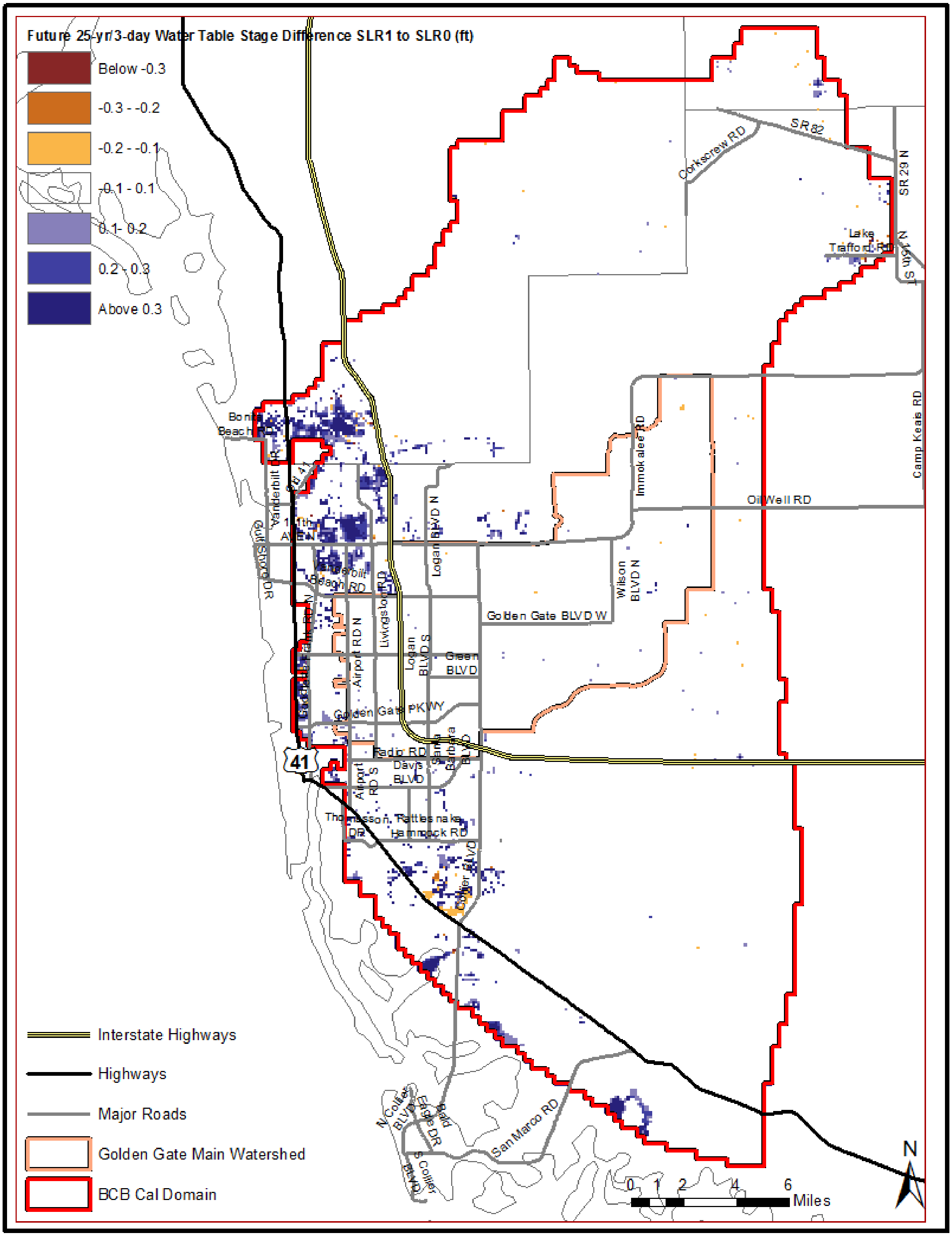


Figure 7.4.2 – Difference Map of Maximum 25-Year Water Table Elevation, SLR1 vs. SLR0

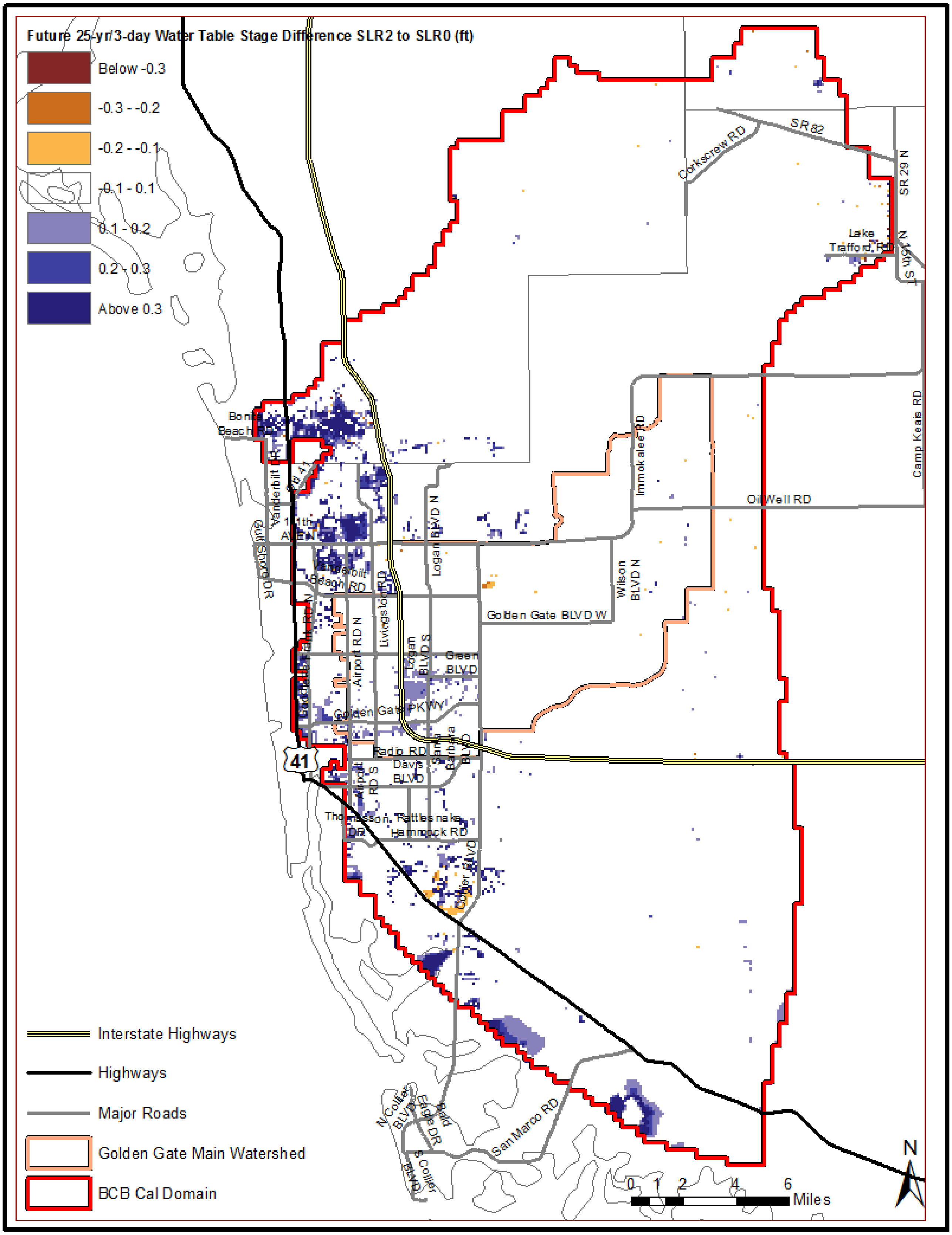


Figure 7.4.3 – Difference Map of Maximum 25-Year Water Table Elevation, SLR2 vs. SLR0

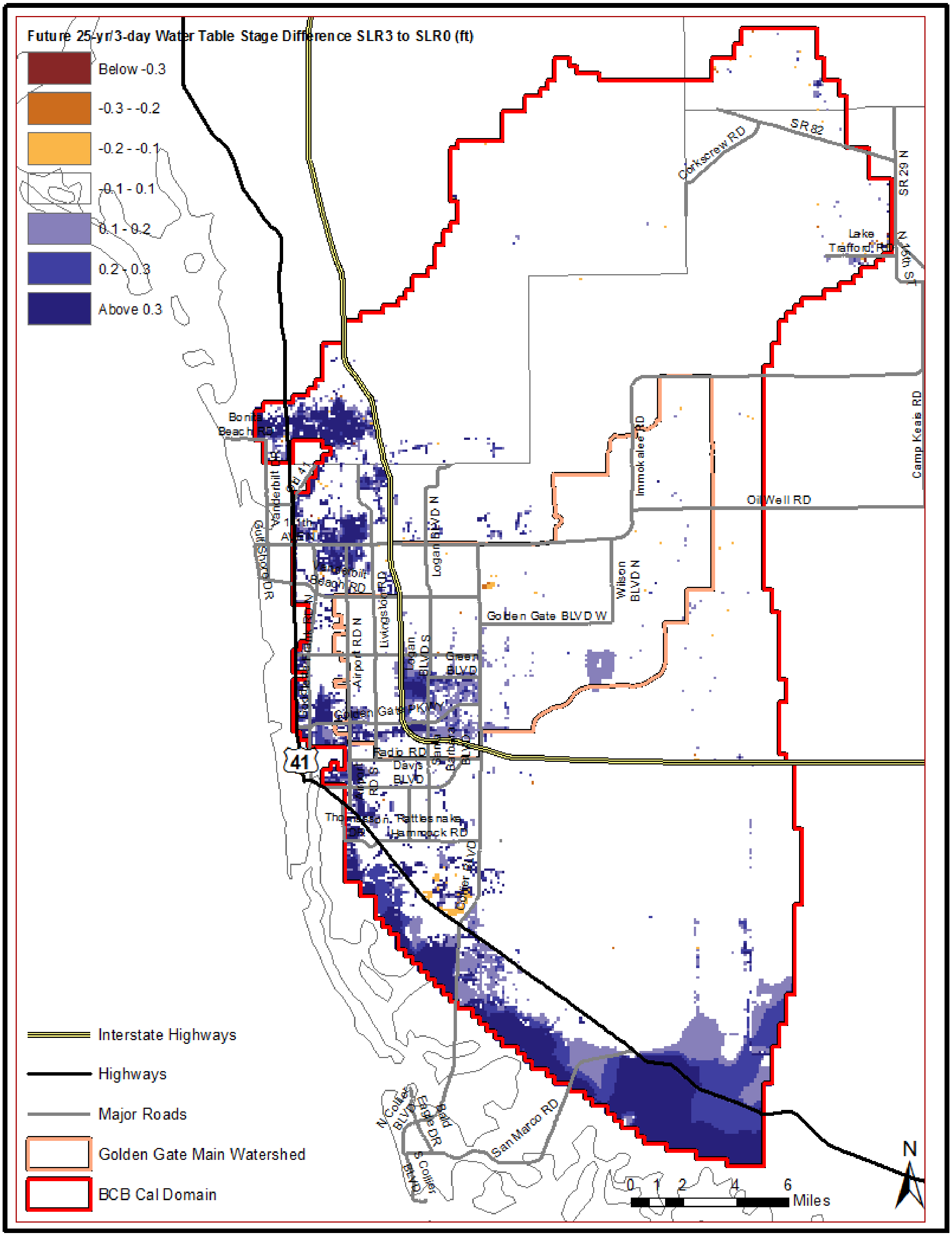


Figure 7.4.4 – Difference Map of Maximum 25-Year Water Table Elevation, SLR3 vs. SLR0

7.5 PM #6 – Duration of Flooding, Future Sea Level

Table 7.5.1 lists the durations of flooding at the six locations previously identified on **Figure 6.4.1** for the 25-year storm event under the three sea level rise scenarios. Durations of flooding at five of the six points of interest (Locations 2 through 6) were basically unchanged as a result of the three sea level rise scenarios evaluated. However, the duration of flooding at Location #1 would increase significantly (by about 6 hours) under the SLR1 and SLR2 scenarios, and by over three days under the SLR3 scenario. This is to be expected because Location #1 would flood due to the high tailwater conditions imposed at the GG1 structure under current conditions; and it stands to reason that the higher tailwater conditions under future sea level conditions would prolong the flooding at this location.

Table 7.5.1 – Flood Duration (days) for Future Sea Level Rise Scenarios

Location	Canal	Chainage (ft)	25-yr			
			SLR0	SLR1	SLR2	SLR3
1	Golden gate main	138,513	1.75	2	2	5
2	AirportS	11,696	0	0	0	0
3	I75	20,082	0	0	0	0
4	CR951	28,754	0.08	0.1	0.1	0.1
5	Cypress	34,724	0.08	0.08	0.08	0.1
6	Cypress	1,936	2.25	2.25	2.25	2.25

8.0 Conclusions

The current and future conditions design storm simulation results were evaluated with respect to five performance measures, which together provide insight into the level of flood protection provided by the current Golden Gate Watershed primary canal network and associated control structures under current and potential future stressors. The future conditions FPLOS was compared with the current conditions FPLOS to identify where significant degradations in FPLOS would likely occur as a result of potential changes in land use and sea level rise.

8.1 Current Conditions

The design storm simulation results were evaluated with respect to four performance measures, which together provide insight into the level of flood protection provided by the current Golden Gate Watershed primary canal network and associated control structures. Based on the results presented herein, it appears that the Golden Gate primary canal network generally provides a 10-year level of service, with some areas receiving a 25-year level of service or better. The few exceptions to the 10-Year FPLOS include the vicinity of GG1, where the area around the structure would be inundated by the combined effects of storm surge and high canal flows for the 5-year event. In addition, the Corkscrew Canal area would be mostly inundated for the 5-year event, and areas of Golden Gate Estates in the vicinity of the GG4 structure would be inundated for the 10-year event.

Several canal segments currently provide less than a 25-year FPLOS, including portions of the Golden Gate Main, Cypress Canal, I-75 Canal, and the CR951 Canal. And although all of the canals contained the 10-year profile along the majority of the bank lengths, the PM #1 profiles show the bank elevation to be exceeded for the 10-year event in multiple locations for all canals except the Airport Road Canal. In many of these locations, the flooding at these locations was localized (e.g., extending less than 500', or the width of one MIKE SHE grid cell from the canal bank), and thus many of the bank exceedances were not evident in PM#5 (frequency of flooding). For the 100-year event, the model results suggest that most canal segments would be overwhelmed as the majority of the watershed would be inundated to some degree during peak flood conditions.

8.2 Future Land Use

Future land use was simulated based on a number of assumptions. Among these are implementation of topographic changes based on FEMA requirements, drainage improvements based on modern design standards, and compliance with current (at the time of model development) regulatory requirements for control of peak discharge rates. Overall, the model results indicate that there would be no widespread degradation of flood protection level of service as a result of the projected land use changes in the Golden Gate Watershed. In fact, the assumed connection of the Preferred Materials mine pits to the Golden Gate Canal as part of development of the surrounding uplands was identified in the modeling as having the potential to provide widespread improvements in flood protection level of service, particularly with respect to PM #1 (maximum stage profiles), PM #2 (maximum discharge capacity, and PM #5 (frequency of flooding).

However, one area was identified adjacent to the Golden Gate Canal, upstream of GG4 where the flooding is projected to worsen for the 10-year and 25-year flood events under future land use conditions. This is an existing low area within the Golden Gate Estates that was identified in the analyses of both PM #1 and PM #5 as an area where current out-of-bank flooding could be exacerbated by future land use changes in the upper reaches of the watershed. As noted in the discussion of the PM #1 and PM #5 results, actual development will differ from the configurations assumed for modeling purposes. For example, Collier County recently reduced the allowable post-development discharge for the Main Golden Gate Canal Basin and the Cypress Canal Basins, from 0.15 cfs/acre to 0.04 and 0.06 cfs per acre, respectively. This regulatory action alone might be sufficient to mitigate the potential increases in flooding predicted by the land use changes simulated in this study.

8.3 Future Sea Level Rise

Changes in flood protection LOS were evaluated in response to three hypothetical sea level rise scenarios. As expected, the largest increases in maximum canal stages would occur under the most severe of the three sea level rise (SLR3) conditions. The location most affected would be the downstream end of the Golden Gate Canal (with a maximum rise of 1.7 feet) and adjacent upland areas west of I-75. Smaller increases would extend several miles upstream to beyond the GG3 structure. Under SLR2, the maximum increase is projected to be approximately 0.5 feet upstream of GG1; this increase rapidly diminishes to approximately 0.1 feet at the GG2 structure. Stage increases associated with the SLR1 scenario are projected to be minimal within the Golden Gate Watershed.

Discharge capacity (PM #2) at the primary control structures within the Golden Gate system would be largely unaffected by the three sea level rise scenarios, with the exception of the tidal structure GG1 under the SLR3 scenario. Model results indicate that the peak discharge for the 25-year design storm event would be reduced by about 100 cfs, or 4% of the current conditions peak. The computed discharge capacity of the canal segment between GG1 and GG2 would be reduced by a similar percentage.

Although not a formal performance metric, maximum water table stages were also evaluated under the future sea level rise scenarios. Higher water table stages can affect infrastructure such as septic tanks, older sanitary sewers (through higher infiltration), and roadway base material. Pronounced differences in maximum water table stages are evident in coastal areas west and south of the Golden Gate Watershed. Within the Golden Gate Watershed, the largest and most widespread increases in maximum water table stage would occur under the SLR3 scenario, in an area west of Collier Blvd and south of Pine Ridge Road where maximum increases would exceed 0.3 feet.

It is important to understand that the three sea level scenarios evaluated in this modeling effort are only projections. Future changes in sea level are difficult to predict, and actual changes by year 2065 could be outside the range of the three scenarios chosen for this study. Future development in the watershed will also likely differ from the projections and assumptions relied on for this study. And finally, it should be noted that the model results are subject to certain limitations associated with the scale of the 2-dimensional model grid. Because the grid cells are 500 feet x 500 feet, the model results are suitable for the subregional-scale flood protection LOS evaluation presented herein, but should not be extended to local-scale evaluations or regulatory determinations of flooding extents, where considerable variations in topography can occur within the area of each grid cell.

A – Peak Stage and Discharge Rate Summary Tables

(Current and Future Conditions)

Table 1 Peak Stage Summary

Structure	Mike 11 Chainage	Peak Stage (ft, NAVD)										
		5yr		10yr		25yr					100yr	
		current	Future	current	Future	Current	Future-SLR0	Future-SLR1	Future-SLR2	Future-SLR3	current	Future
GG1	139702	4.93	4.92	5.39	5.37	5.95	5.95	6.10	6.42	7.49	6.90	6.90
GG2	127265	7.72	7.48	8.63	8.36	9.15	8.99	9.03	9.08	9.33	9.70	9.52
GG3	109808	8.63	8.12	9.64	9.04	10.22	9.69	9.72	9.78	10.02	10.79	10.33
GG4	51730	10.99	11.44	11.94	12.10	12.33	12.39	12.40	12.41	12.42	12.66	12.69
GG5	28357	11.81	12.82	12.95	13.65	13.59	14.07	14.08	14.19	14.19	14.12	14.43
GG6	13320	15.46	16.88	16.12	17.32	16.40	17.57	17.57	17.58	17.57	16.55	17.67
GG7	6790	13.68	14.35	14.09	15.21	14.68	15.66	15.69	15.75	15.76	15.31	16.01
CORK1	20873	11.68	12.49	12.65	13.41	13.34	13.91	13.90	13.90	13.89	13.83	14.13
CORK2	4498	15.45	15.45	15.45	15.45	15.74	15.73	15.73	15.73	15.73	16.06	16.05
CORK3	-226	15.18	15.18	15.27	15.27	15.48	15.47	15.48	15.48	15.48	15.86	15.86
AR1	22198	6.85	6.85	7.10	7.08	7.44	7.45	7.46	7.54	7.89	8.60	8.55
I75-1	37182	7.98	7.79	8.93	8.66	9.48	9.30	9.32	9.37	9.61	9.99	9.86
I75-2	20082	9.08	9.03	9.85	9.75	10.33	10.29	10.29	10.33	10.44	10.77	10.72
I75-3	9383	10.06	9.98	10.86	10.80	11.24	11.23	11.20	11.22	11.24	11.54	11.51
CR951-1	37435	8.62	8.20	9.63	9.13	10.22	9.80	9.83	9.89	10.12	10.79	10.47
CR951-2	10760	11.22	11.19	12.10	12.03	12.36	12.37	12.36	12.37	12.38	12.55	12.55
TWINEAG	2135	11.58	12.21	11.70	13.04	13.12	13.52	13.51	13.50	13.50	13.62	13.76
Cypress1	21814	10.7	11.04	11.49	11.90	12.20	12.29	12.30	12.32	12.34	12.56	12.62

Table 2 Peak Discharge Rate Summary

Structure	Mike11 Chainage	Peak Discharge (cfs)										
		5 yr		10yr		25yr					100yr	
		Current	Future	Current	Future	Current	Future-SLR0	Future-SLR1	Future-SLR2	Future-SLR3	Current	Future
GG1	140682	2,214	2,063	2,716	2,515	3,053	2,887	2,893	2,892	2,810	3,503	3,343
GG2	127772	2,077	1,918	2,525	2,354	2,795	2,643	2,640	2,629	2,546	3,003	2,837
GG3	109908	1,069	1,169	1,267	1,169	1,380	1,252	1,245	1,235	1,229	1,760	1,566
GG4	52218	860	1,160	998	1,244	1,082	1,252	1,254	1,257	1,268	1,204	1,326
GG5	30643	769	1,239	1,009	1,411	1,224	1,520	1,541	1,614	1,643	1,401	1,616
GG6	14344	301	550	416	637	463	685	686	687	686	489	706
GG7	7664	186	144	200	202	208	238	244	244	257	277	246
CORK1	20997	293	293	302	299	321	322	320	335	319	399	418
CORK2	5249	143	144	160	160	195	194	194	194	194	256	254
CORK3	16	205	205	246	245	296	294	294	294	294	345	344
AR1	23212	50	50	61	64	80	85	81	81	84	197	199
I75-1	38153	1,044	1,032	1,180	1,206	1,244	1,320	1,321	1,313	1,299	1,228	1,405
I75-2	21654	432	434	514	506	533	529	517	516	500	533	531
I75-3	10400	290	285	373	361	410	410	405	401	383	389	382
CR951-1	37730	639	659	865	864	911	929	922	925	907	1,010	1,007
CR951-2	10827	113	120	153	153	196	194	190	194	170	198	183
TWINEAGLE	2182	27	33	47	49	51	50	50	50	49	54	50
Cypress1	21995	319	376	340	403	348	398	398	397	388	394	416

B – MIKE 11 Detailed Time Series for 5-year Storm, Current Conditions

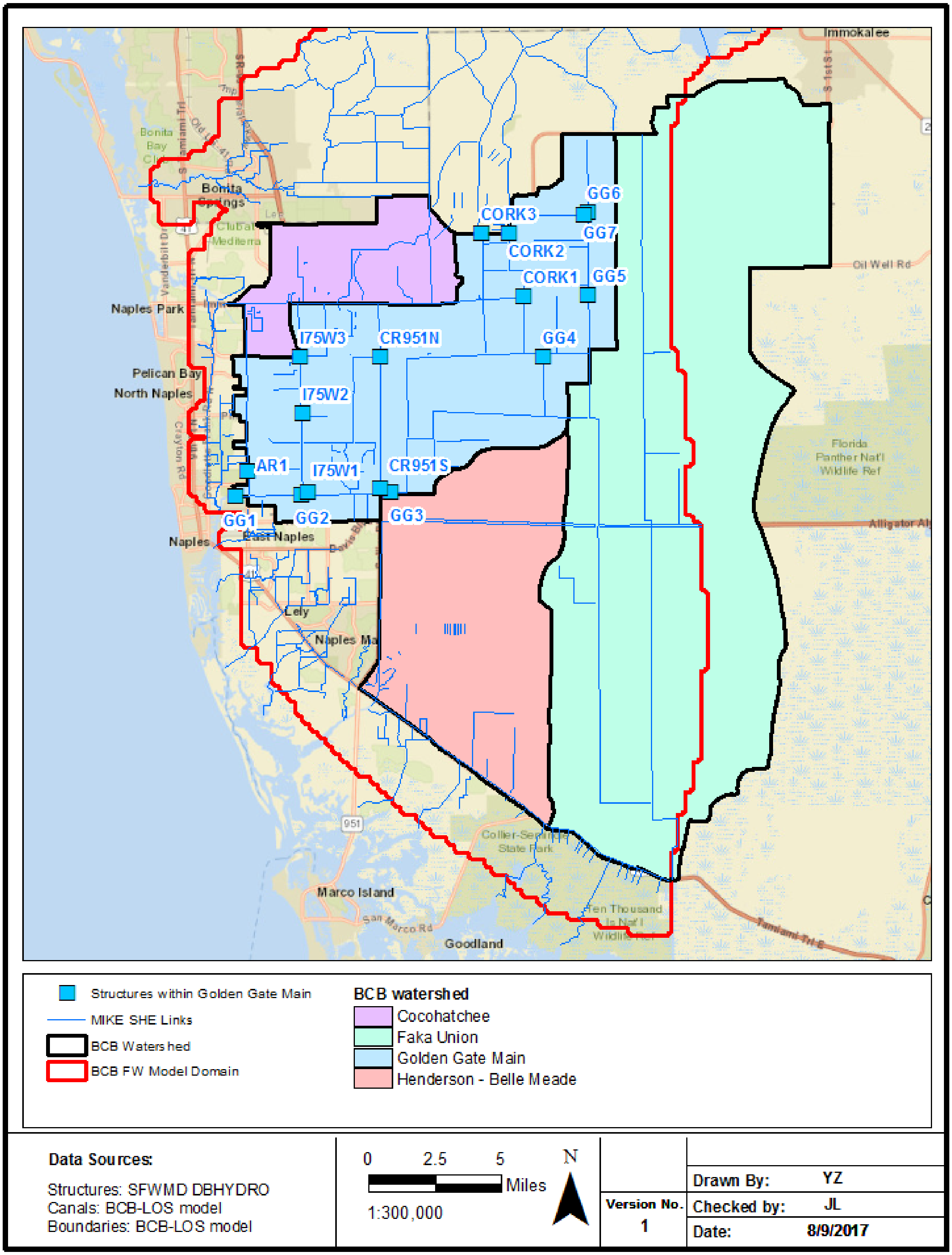


Figure 1 Mike SHE Stations within Golden Gate Watershed

List of Figures

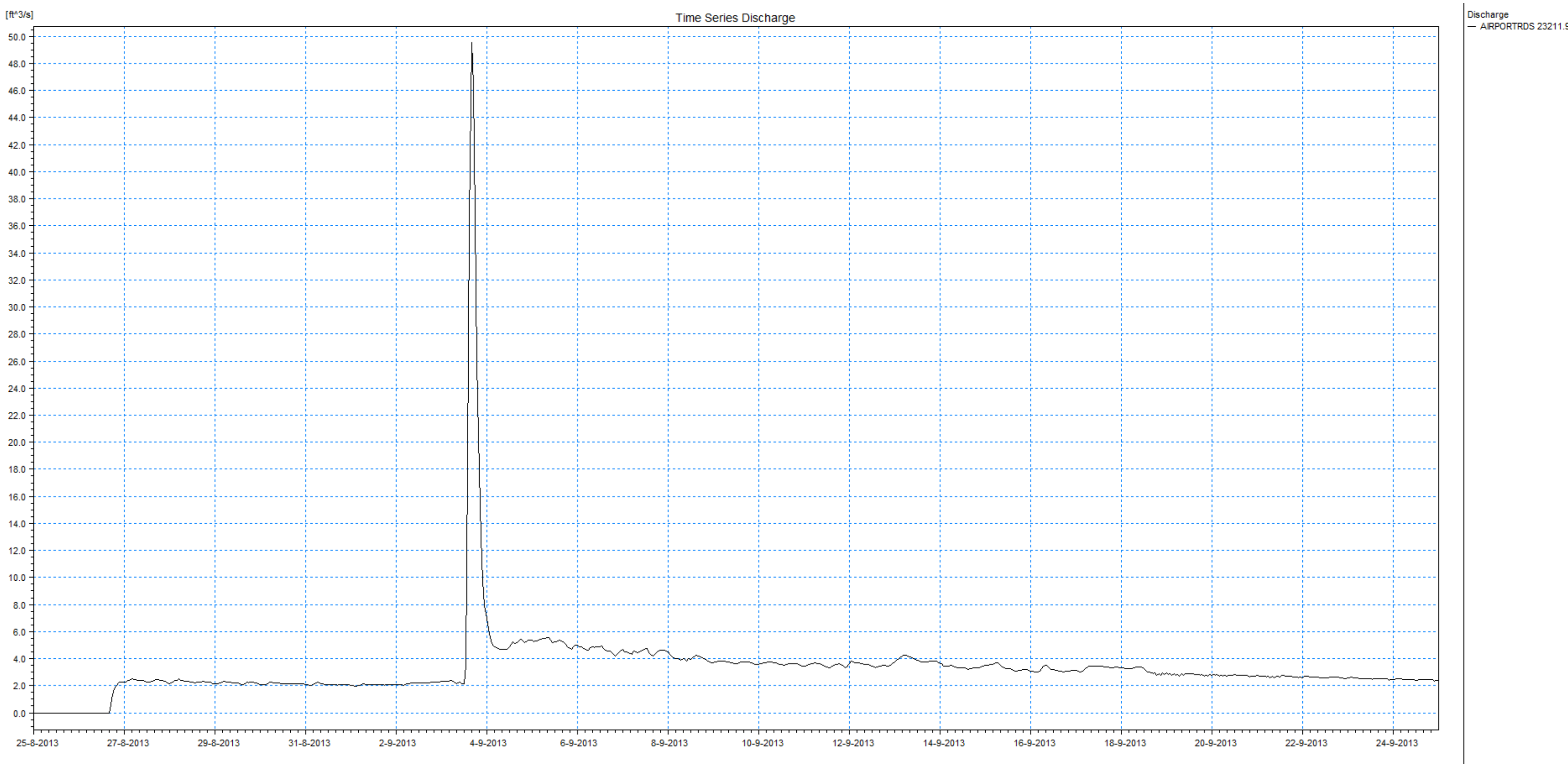


Figure 2. AR1 Discharge (cfs)

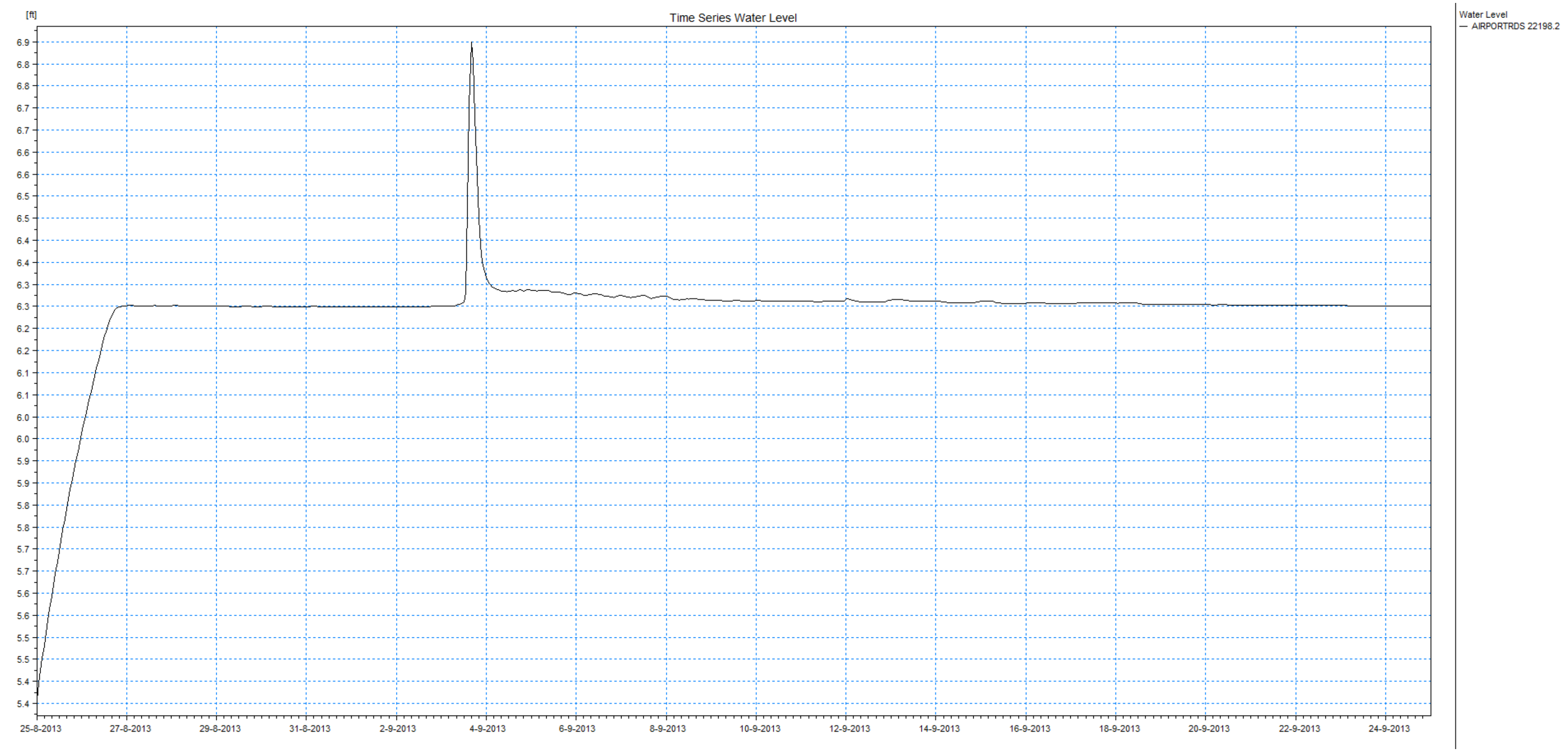


Figure 3. AR1 Headwater Stage (ft)

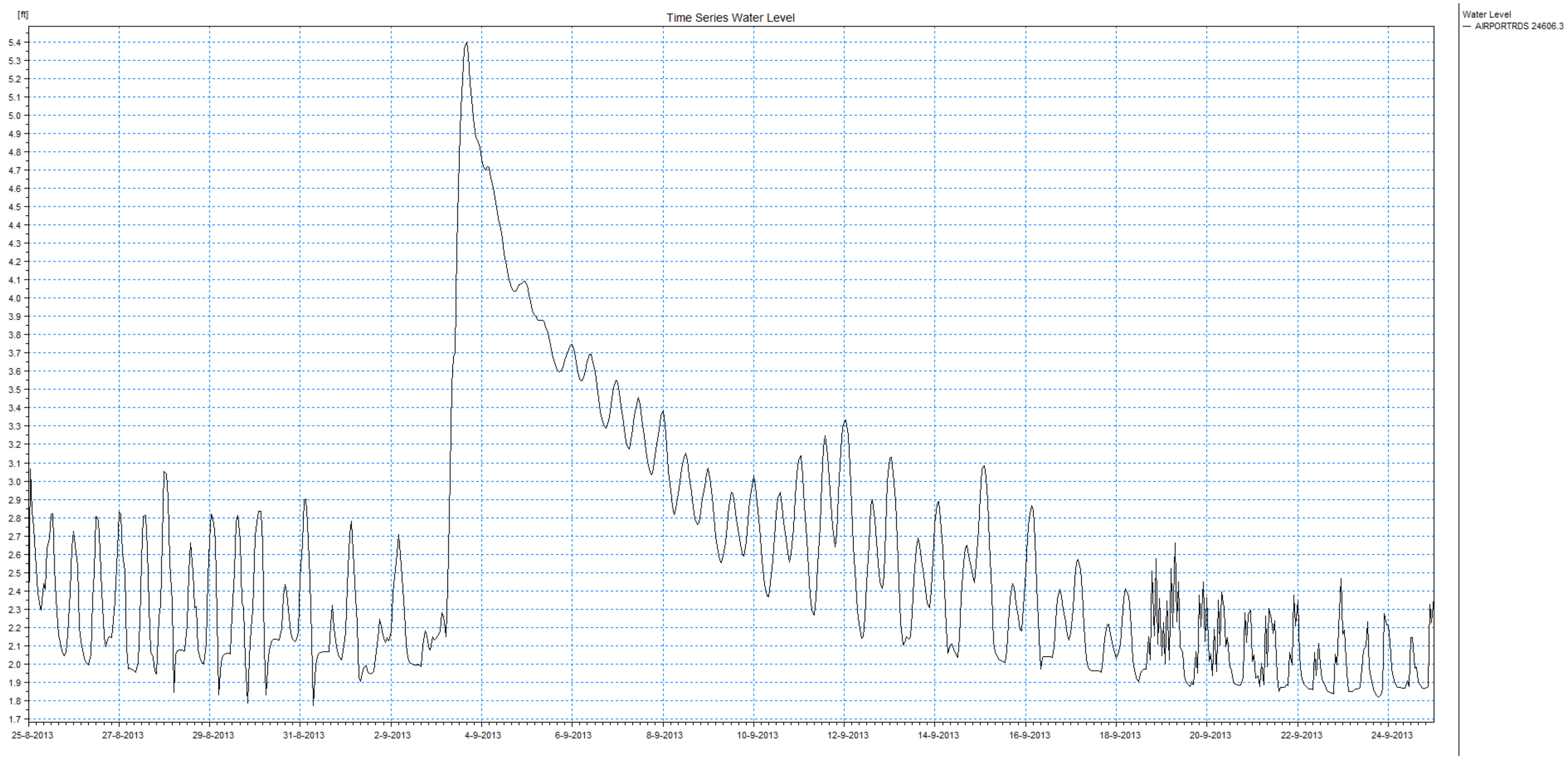


Figure 4. AR1 Tailwater Stage (ft)

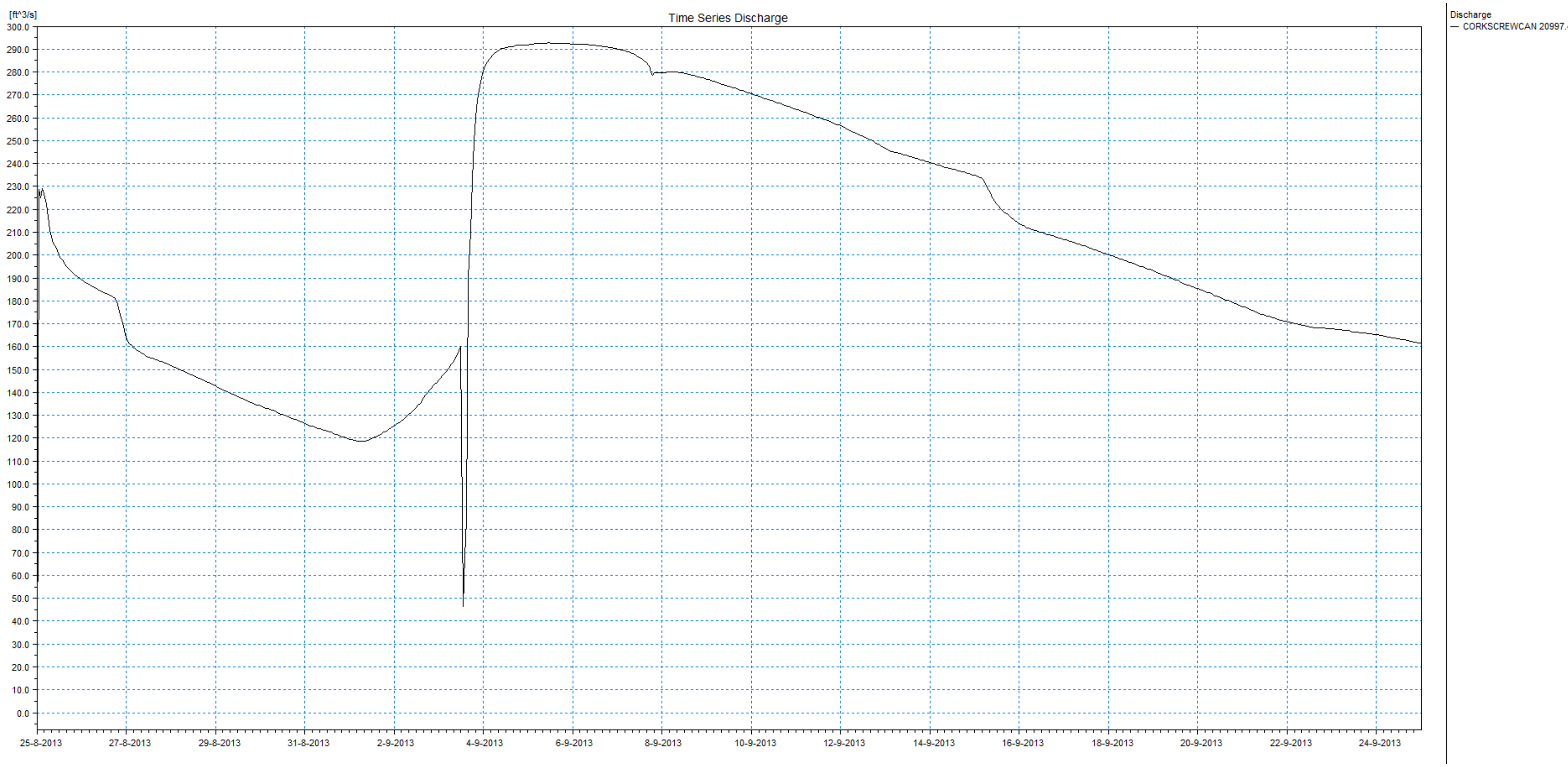


Figure 5. CORK 1 Discharge (cfs)

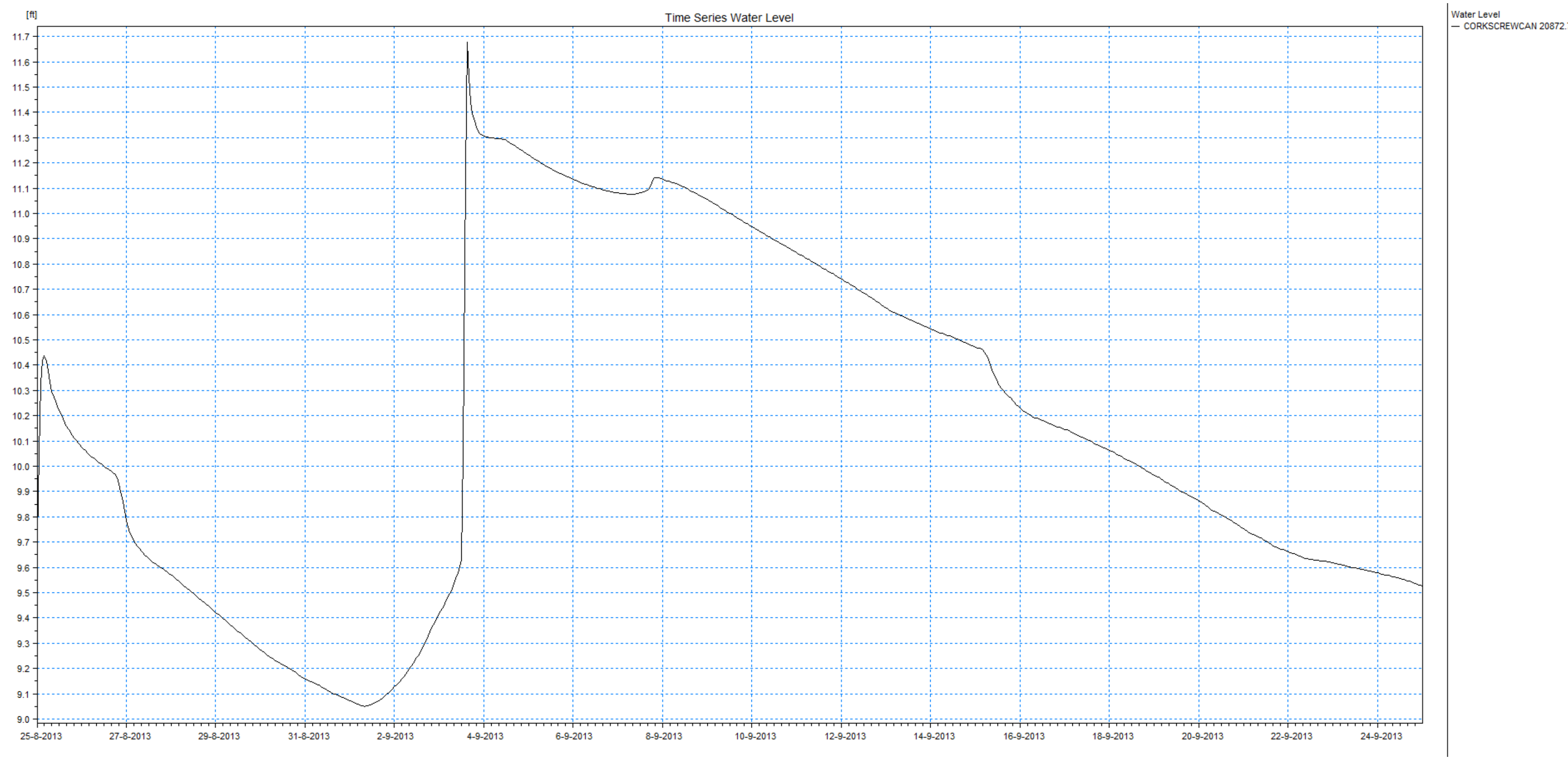


Figure 6. CORK 1 Headwater Stage (ft)

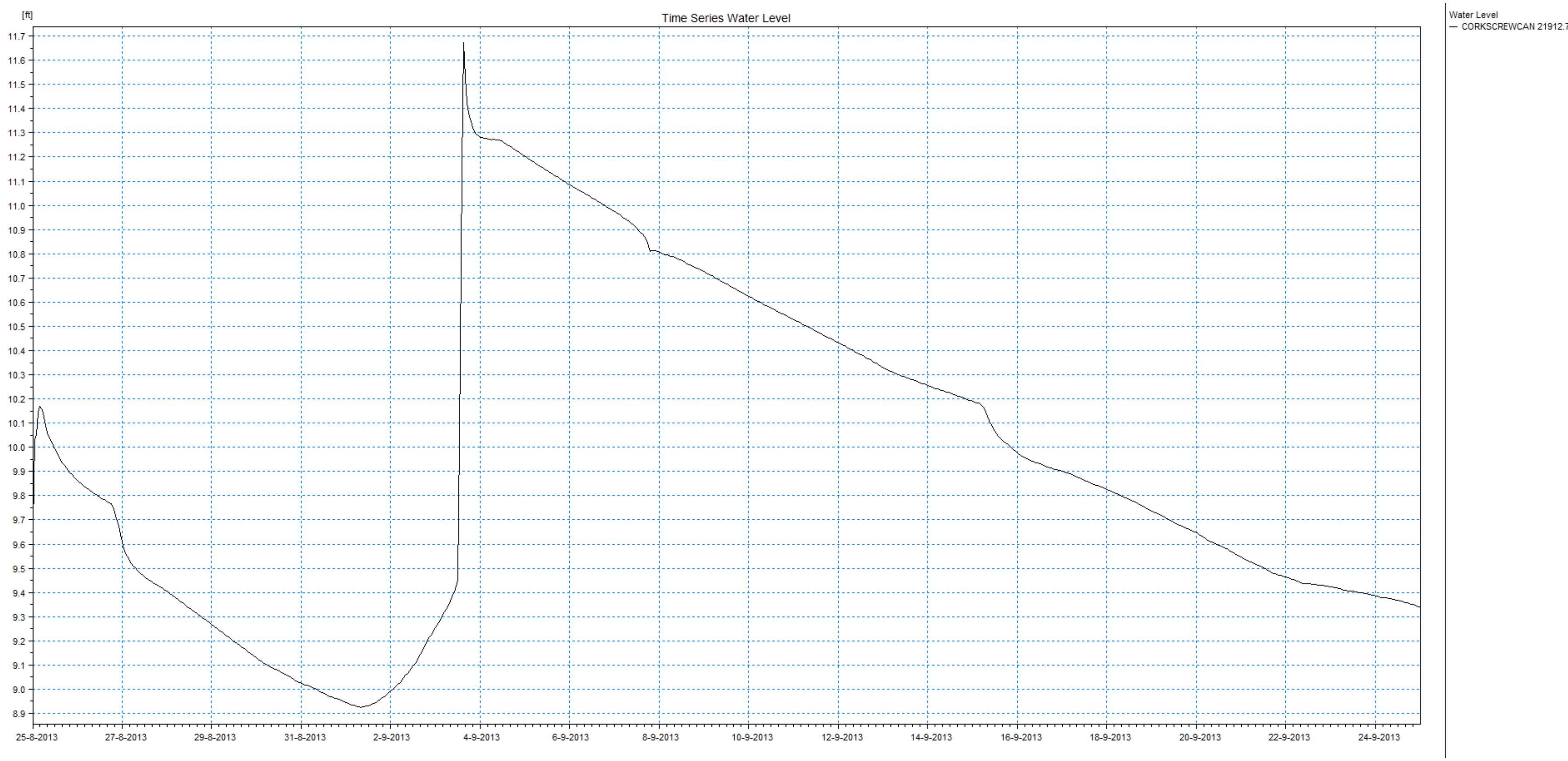


Figure 7. CORK 1 Tailwater Stage (ft)

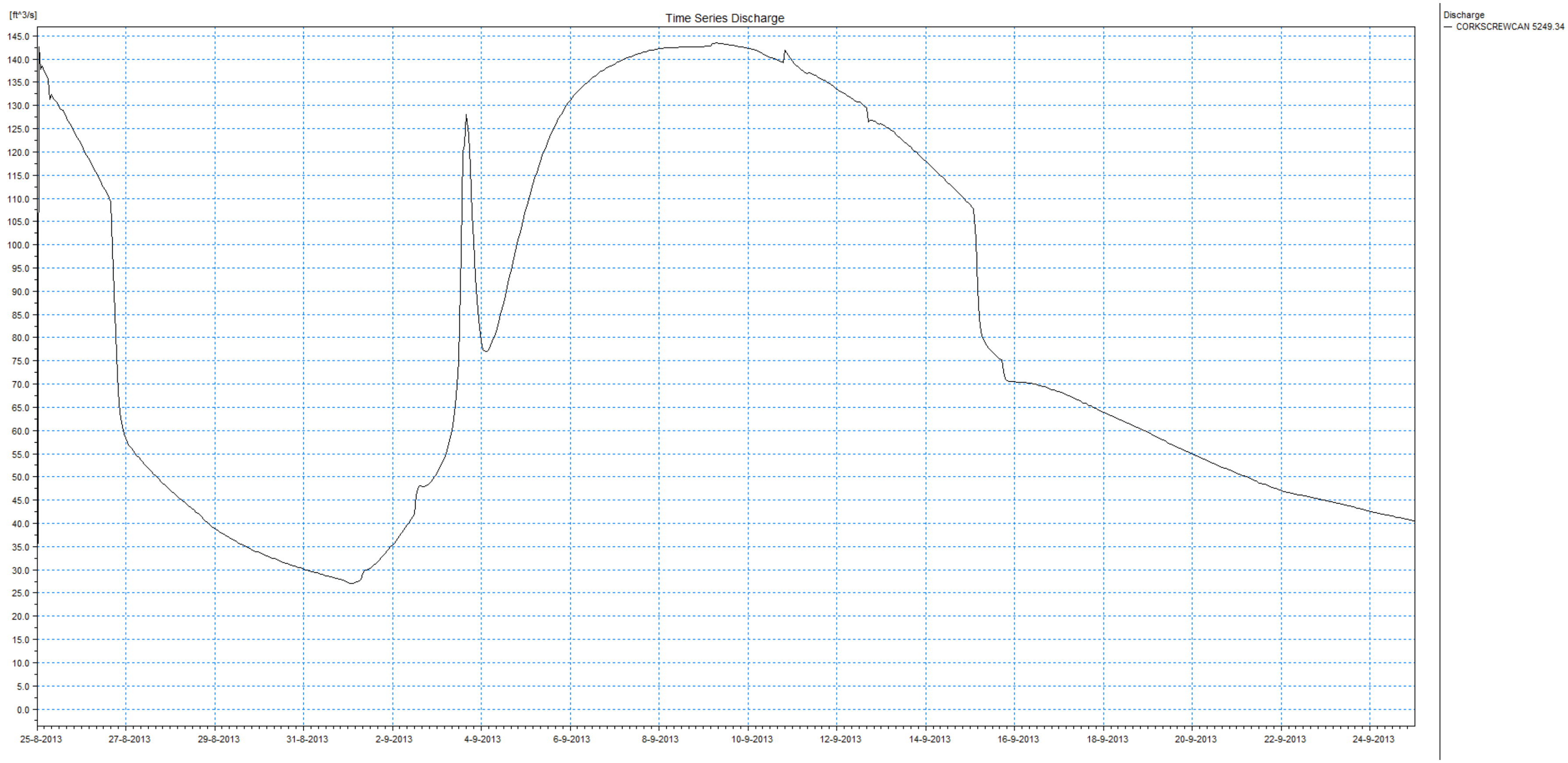


Figure 8. CORK 2 Discharge (cfs)

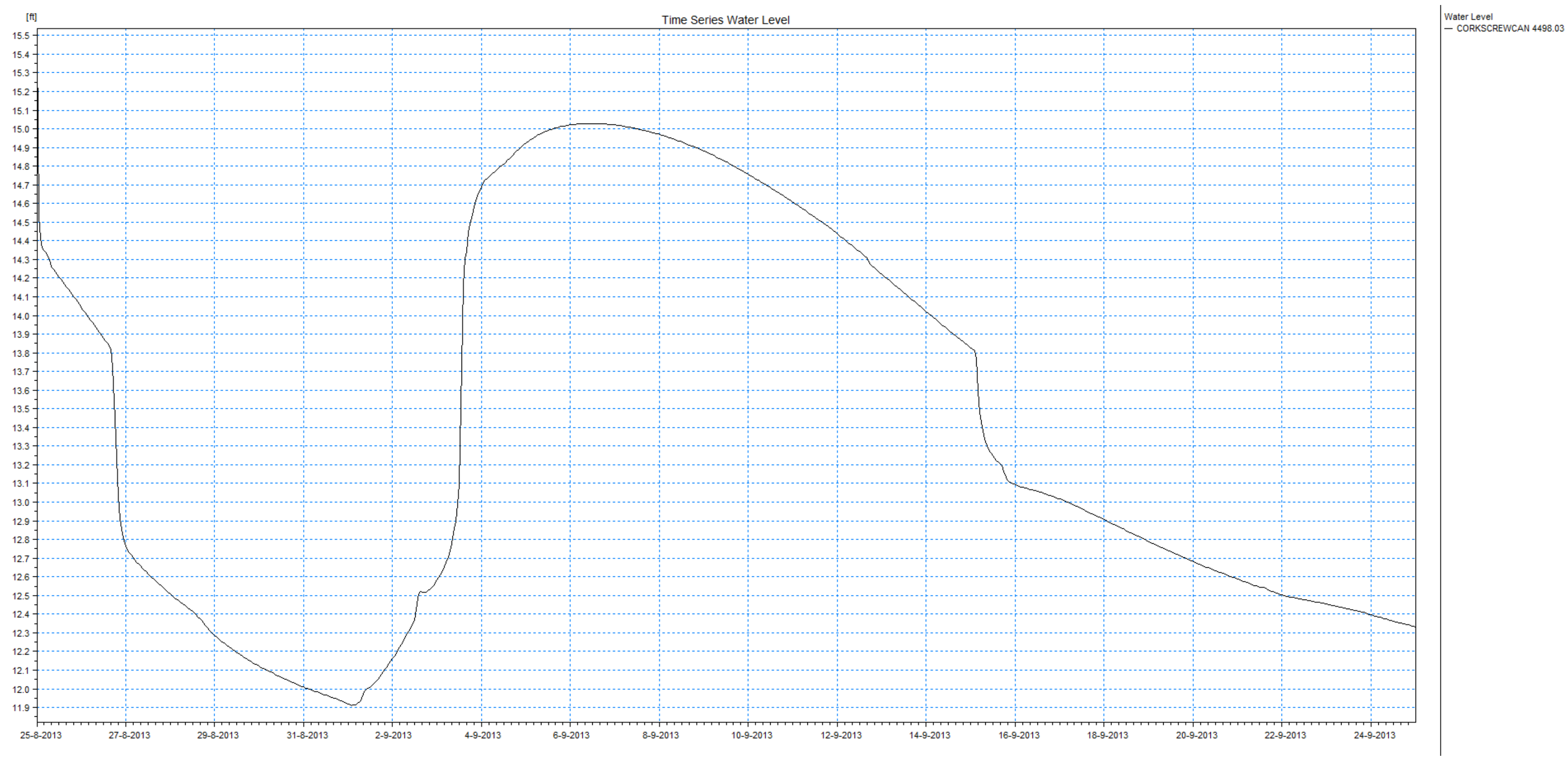


Figure 9. CORK 2 Headwater Stage (ft)

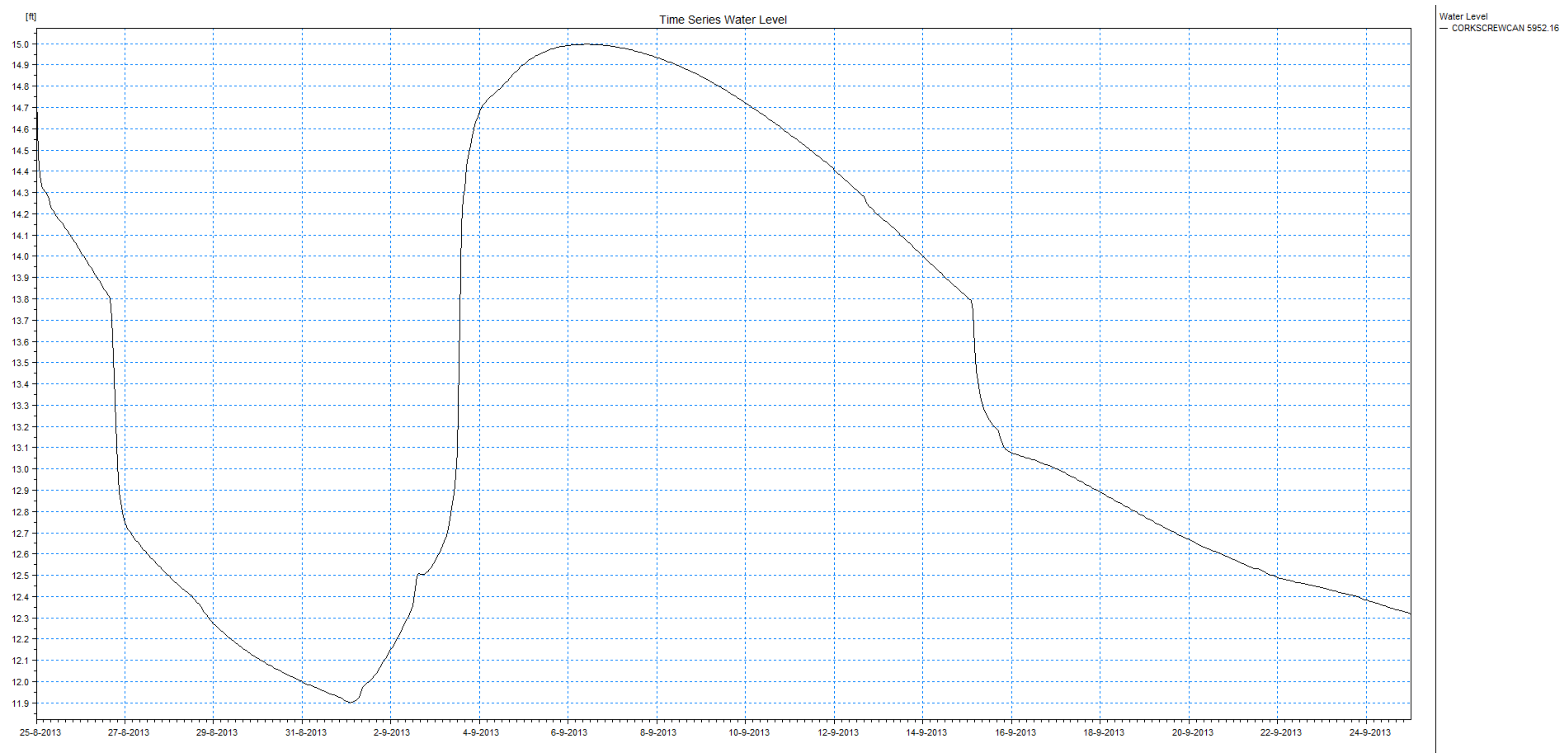


Figure 10. CORK 2 Tailwater Stage (ft)

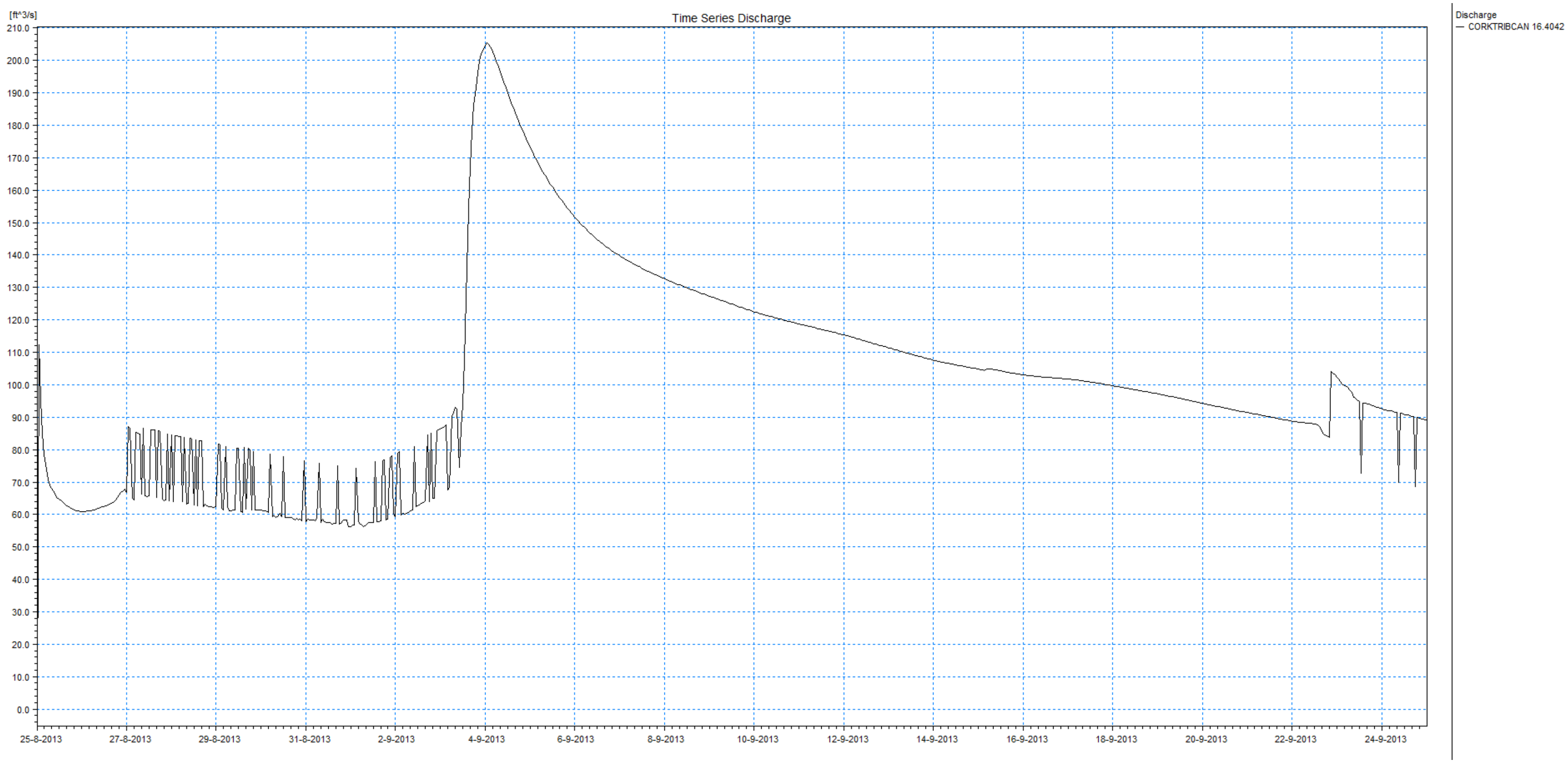


Figure 11. CORK 3 Discharge (cfs)

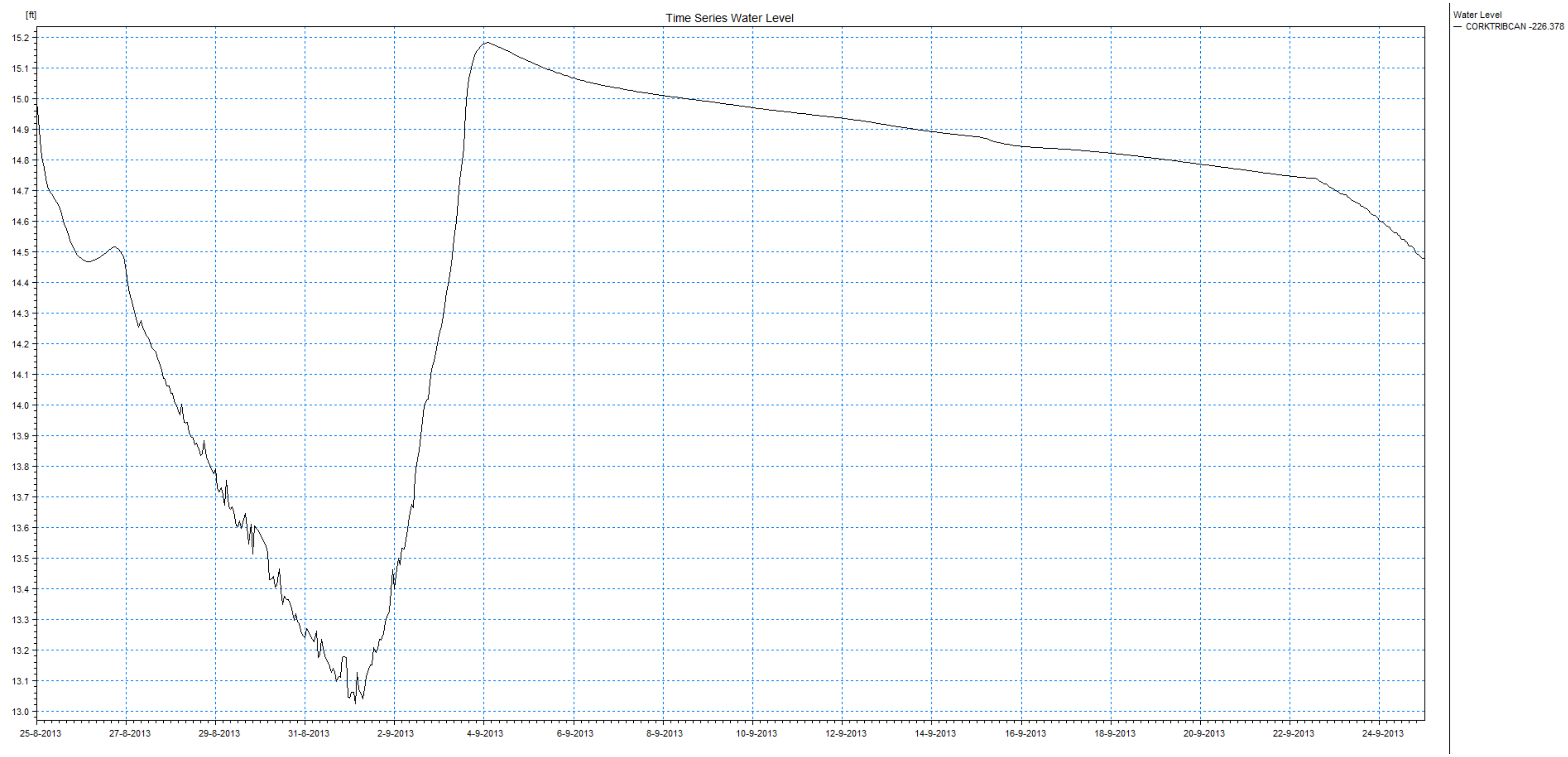


Figure 12. CORK 3 Headwater Stage (ft)

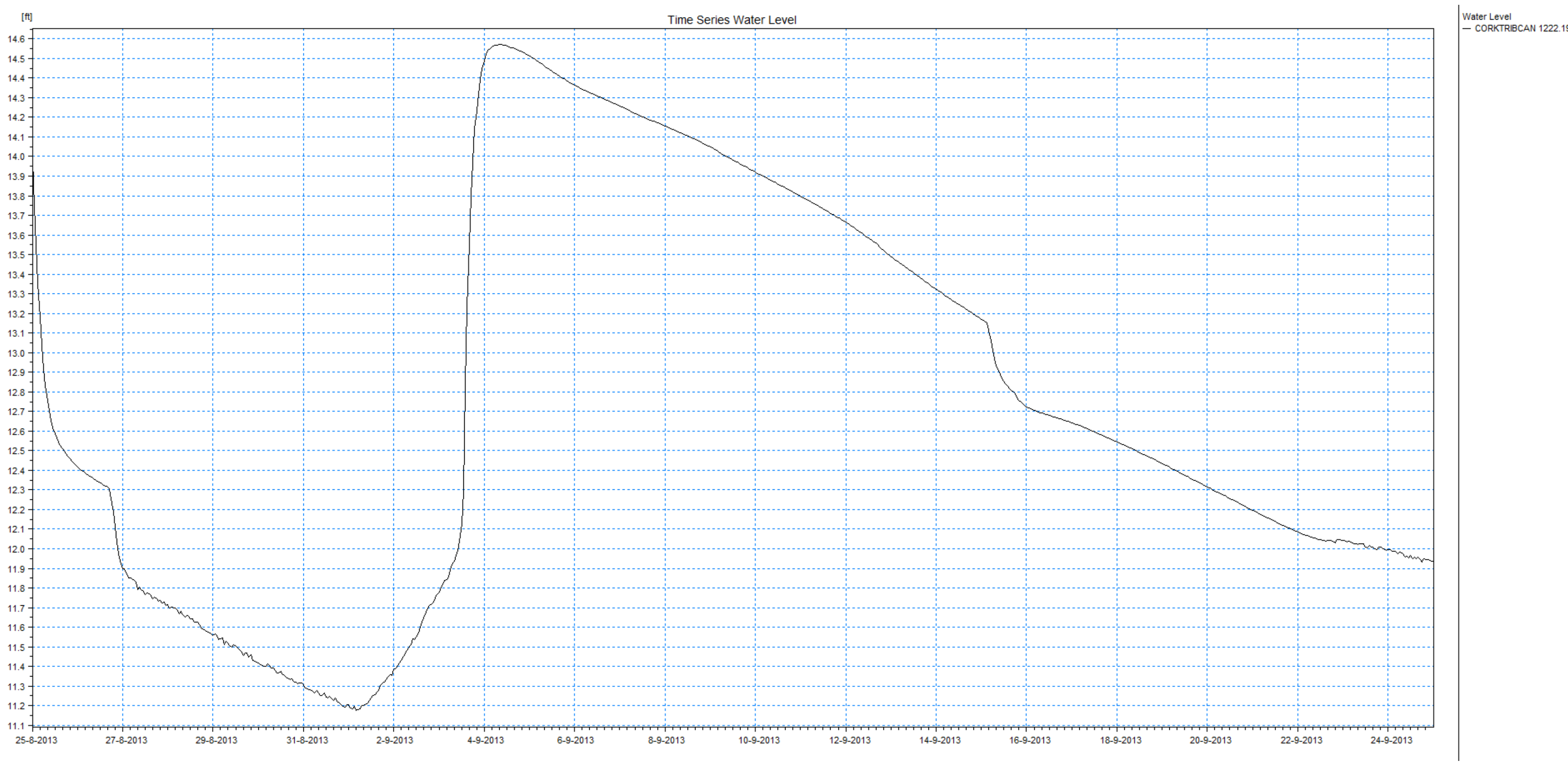


Figure 13. CORK 3 Tailwater Stage (ft)

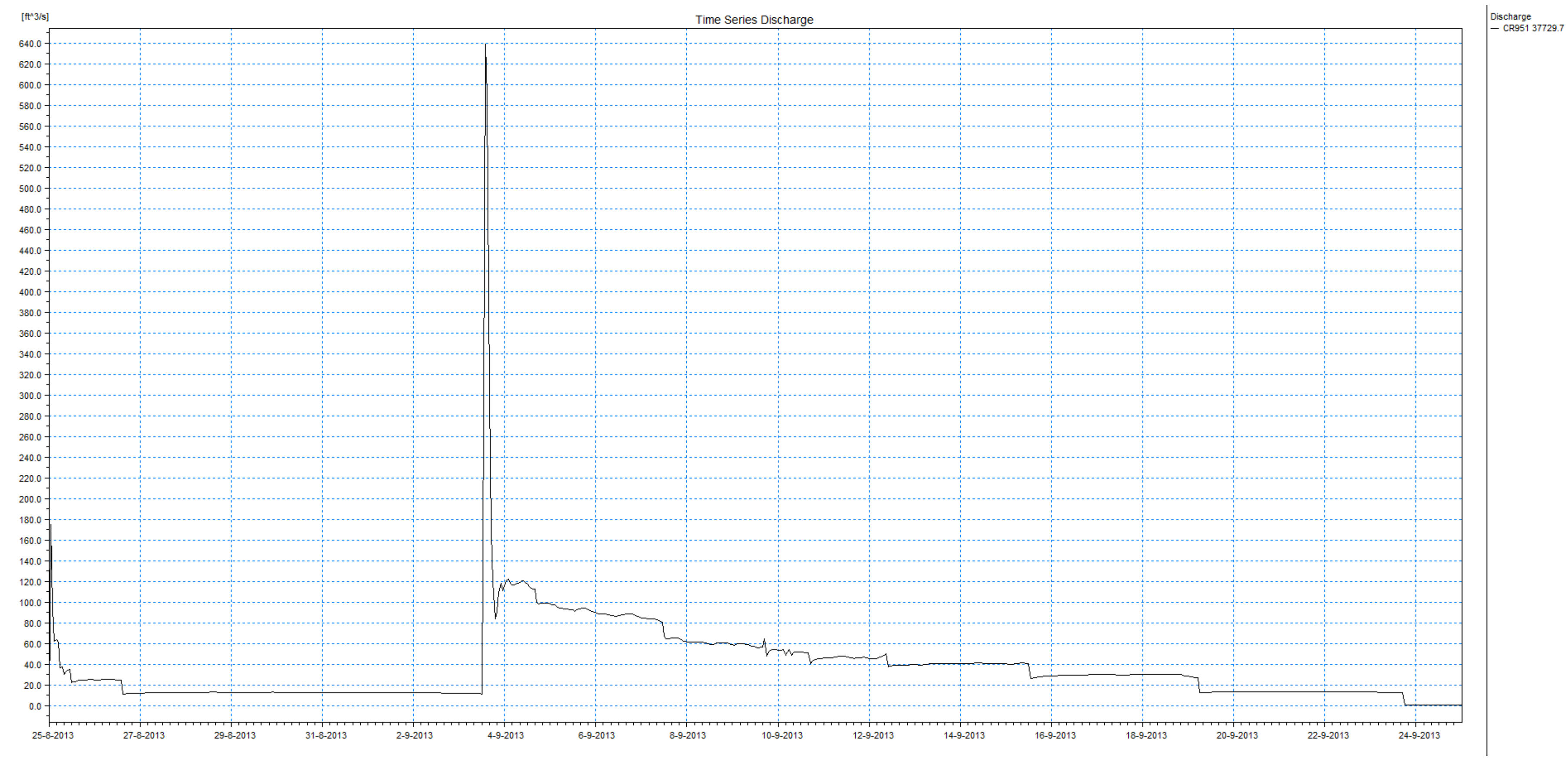


Figure 14. CR9511 Discharge (cfs)

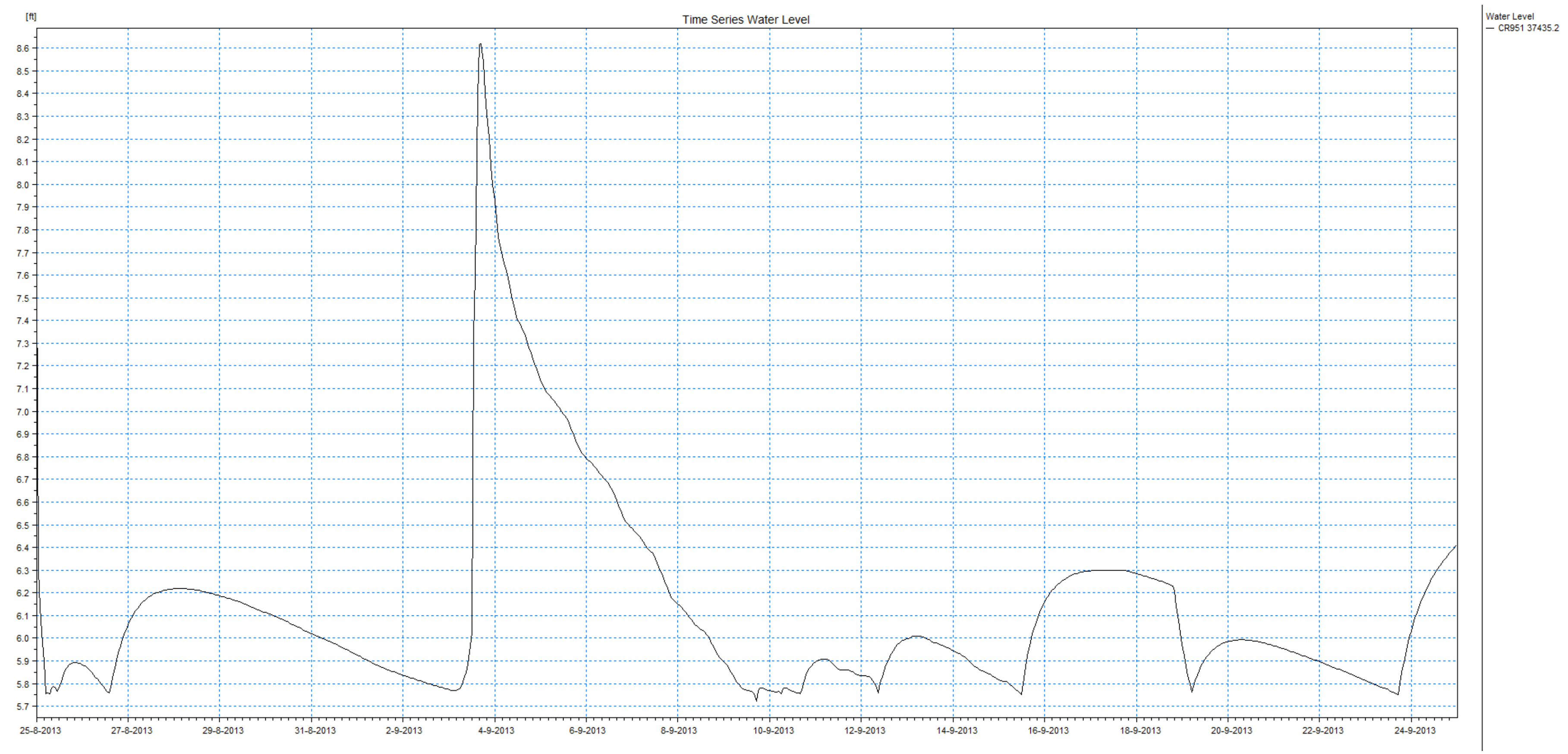


Figure 15. CR9511 Headwater Stage (ft)

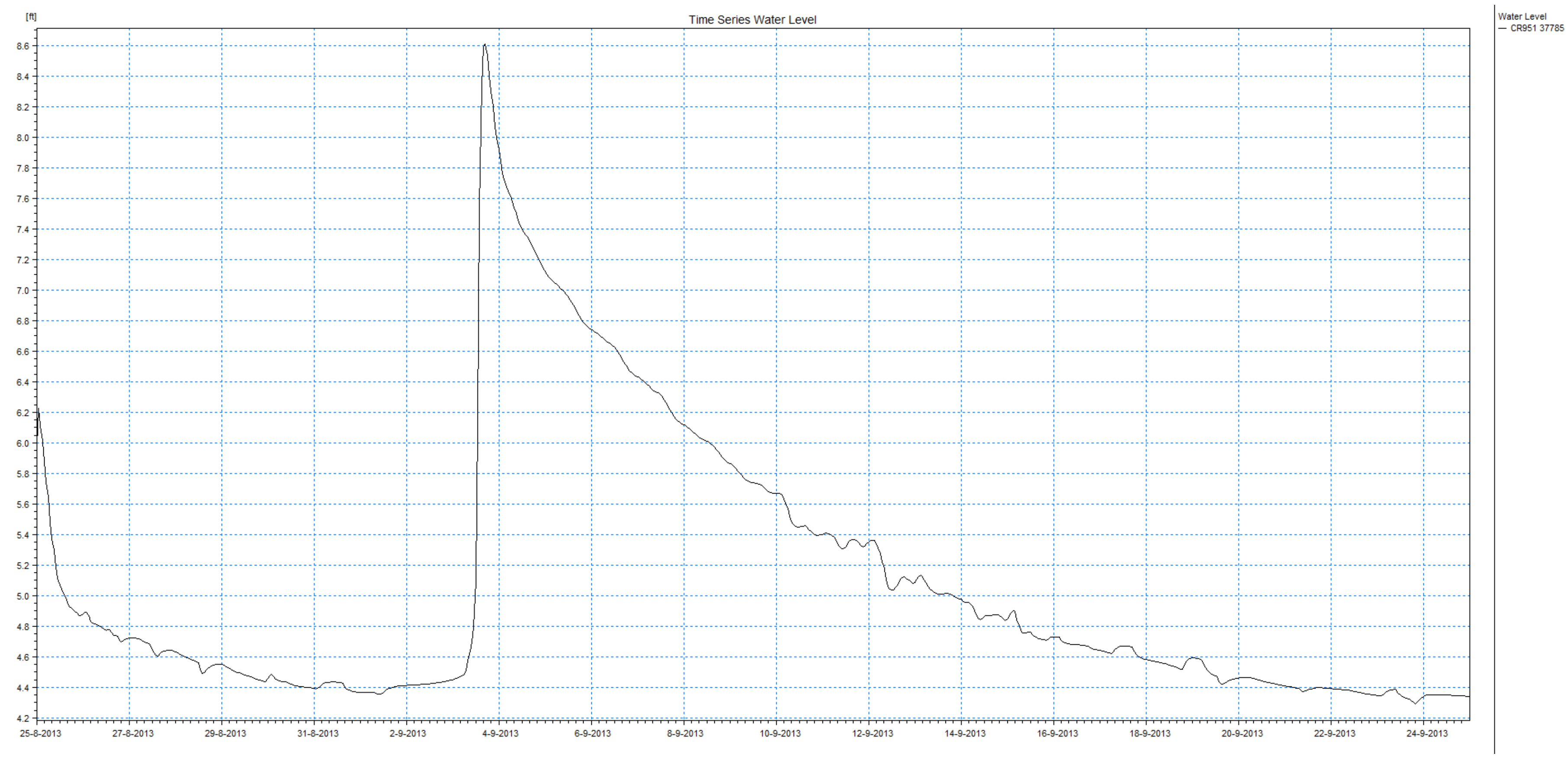


Figure 16. CR9511 Tailwater Stage (ft)

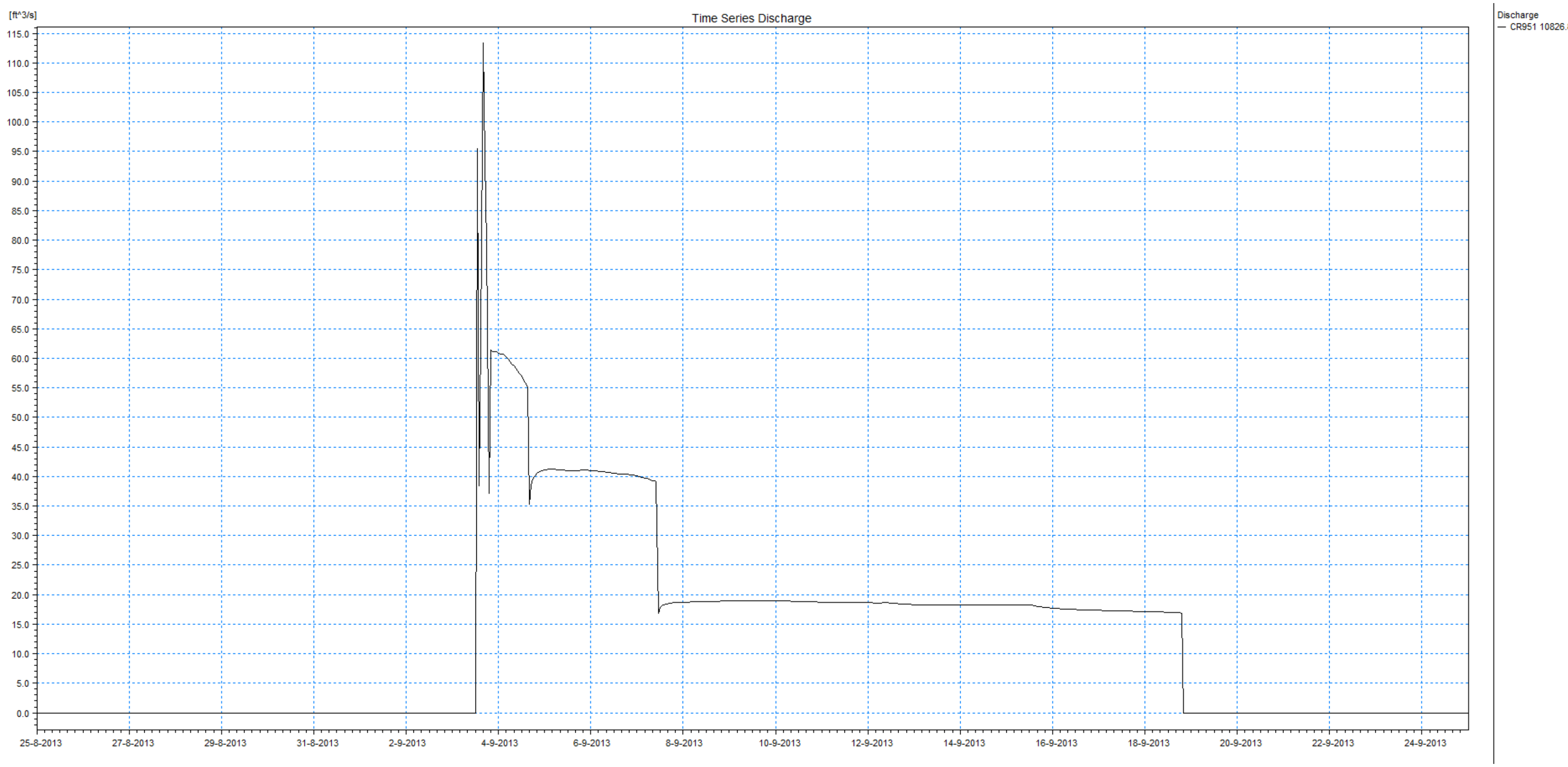


Figure 17. CR9512 Discharge (cfs)

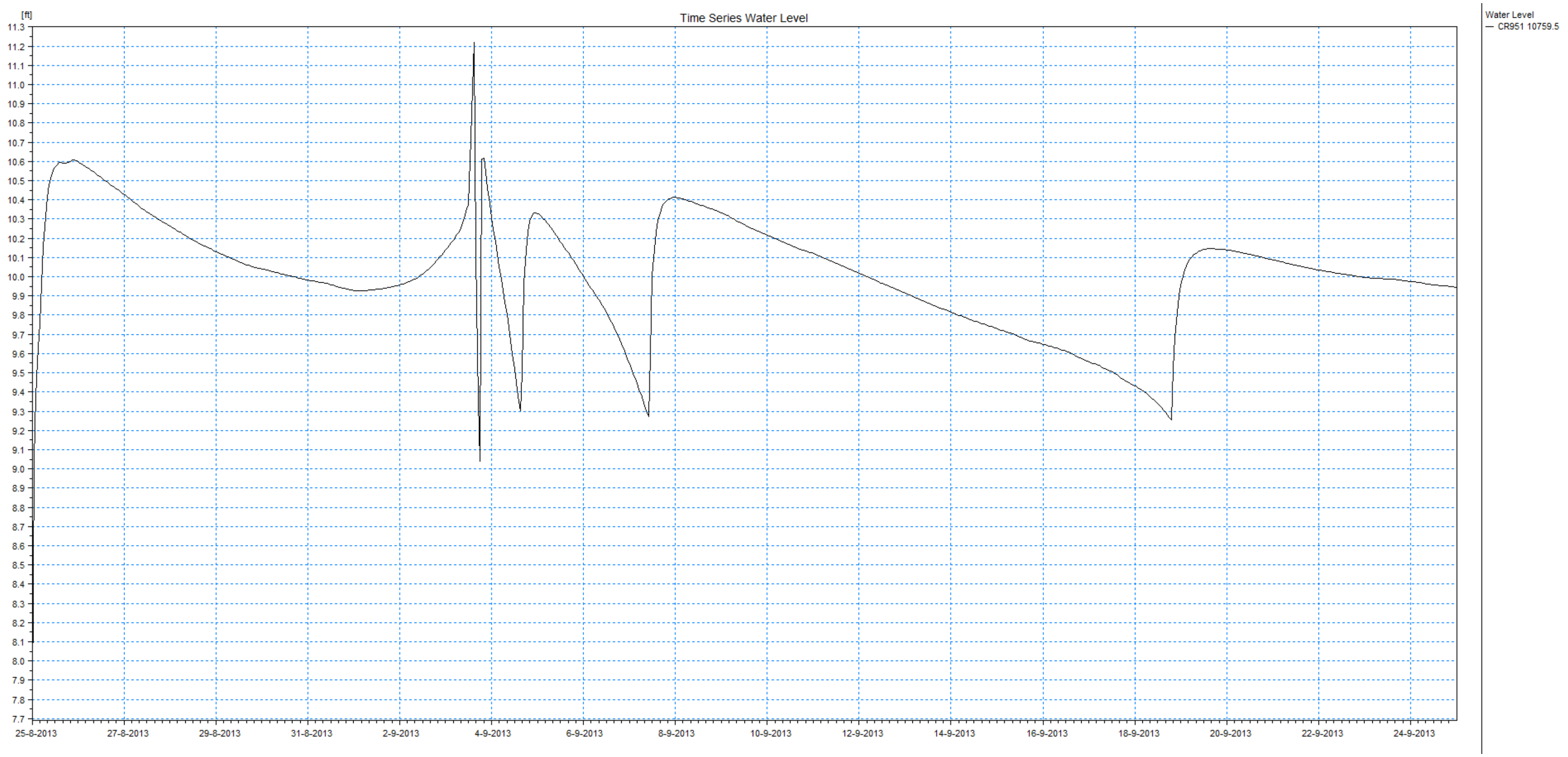


Figure 18. CR9512 Headwater Stage (ft)

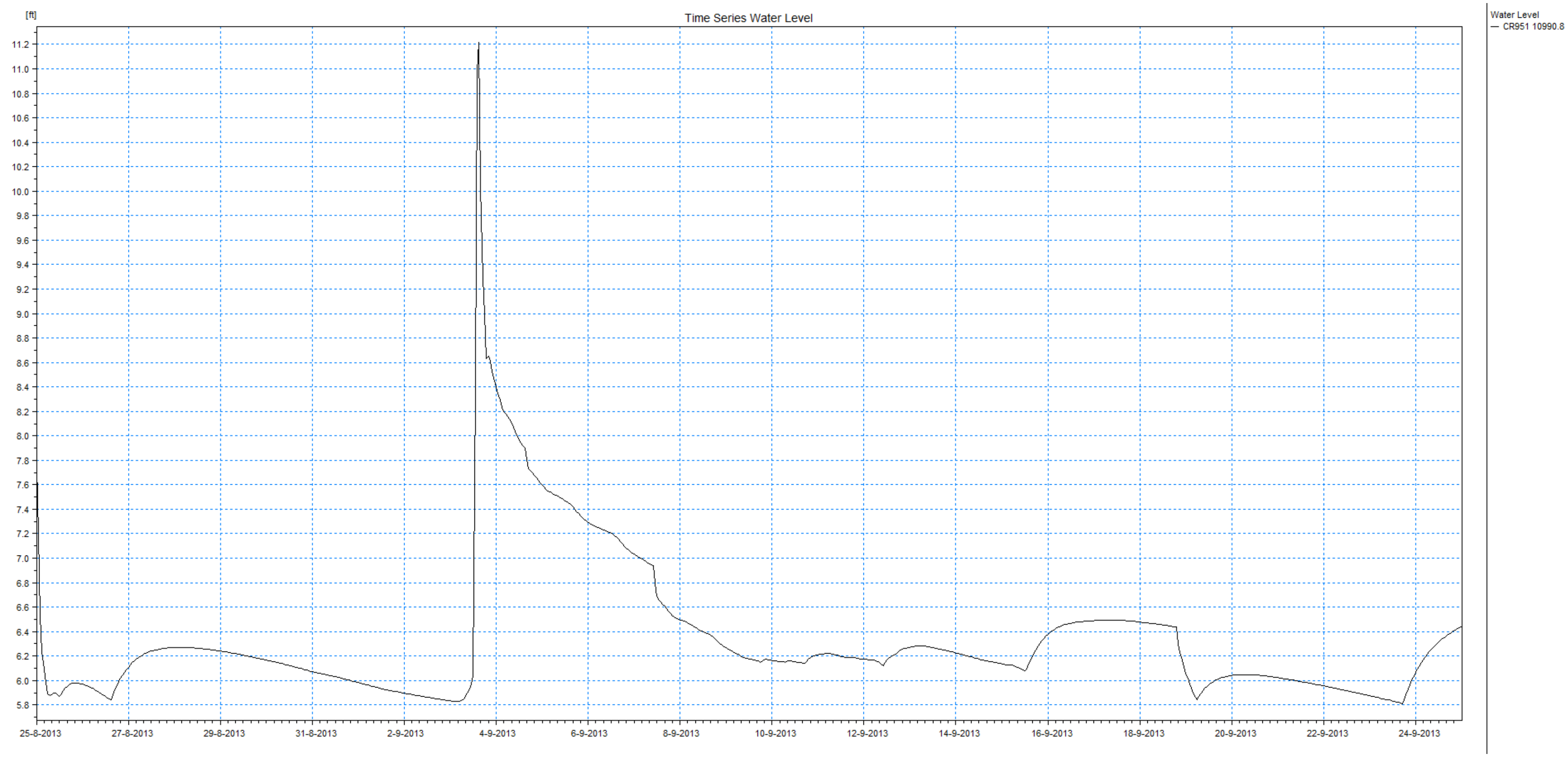


Figure 19. CR9512 Tailwater Stage (ft)

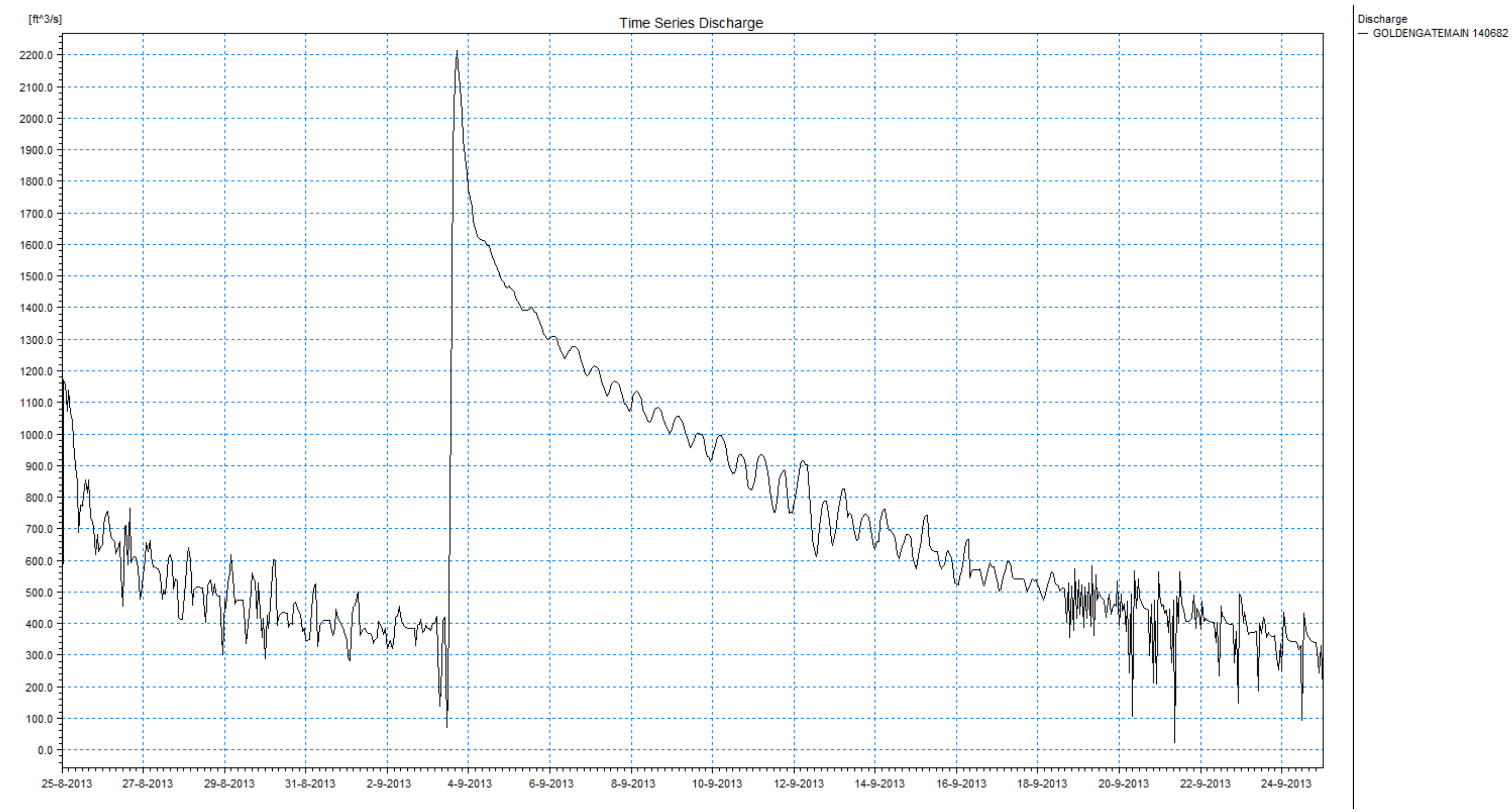


Figure 20. GG1 Discharge (cfs)

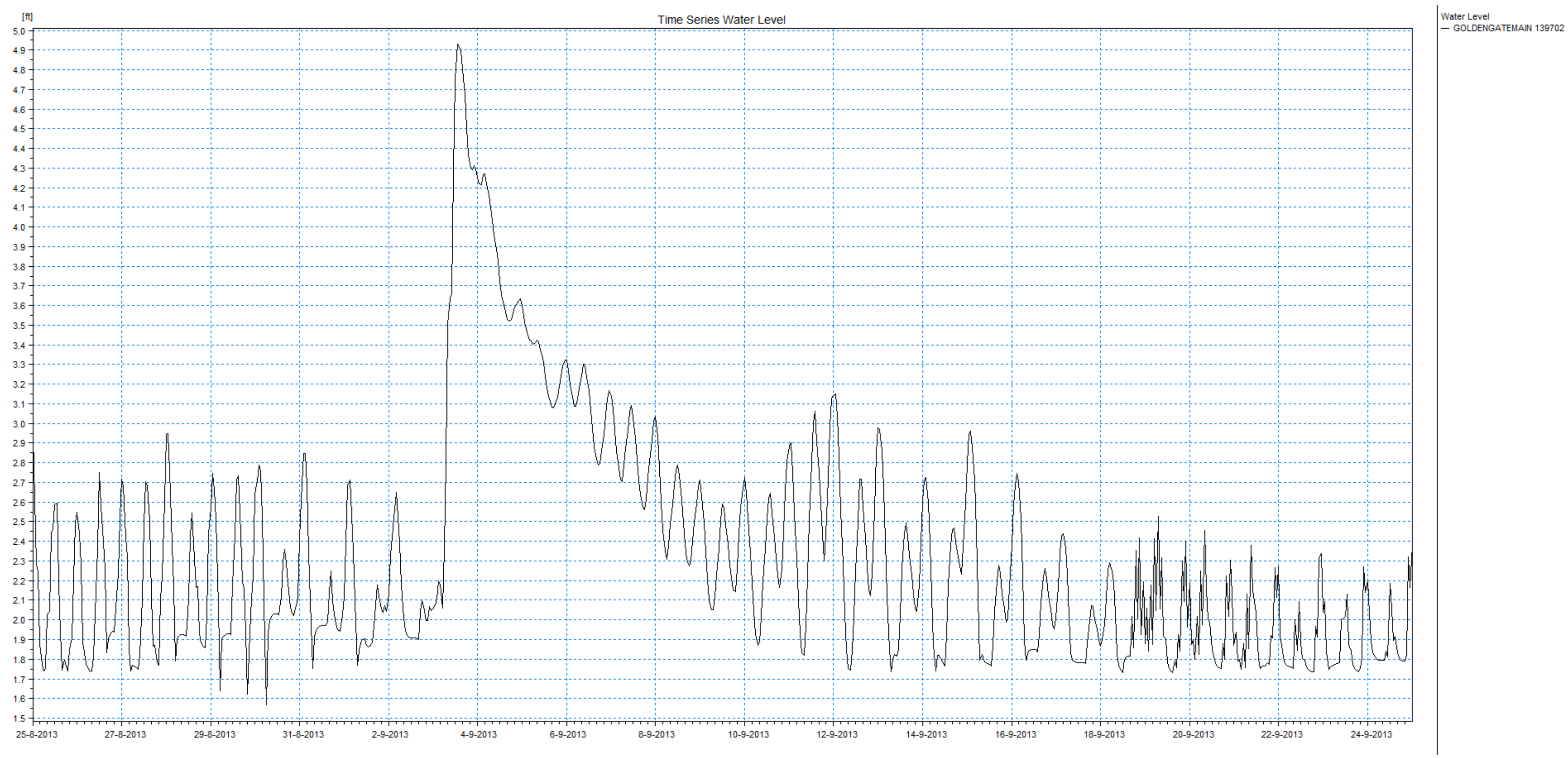


Figure 21. GG1 Headwater Stage (ft)

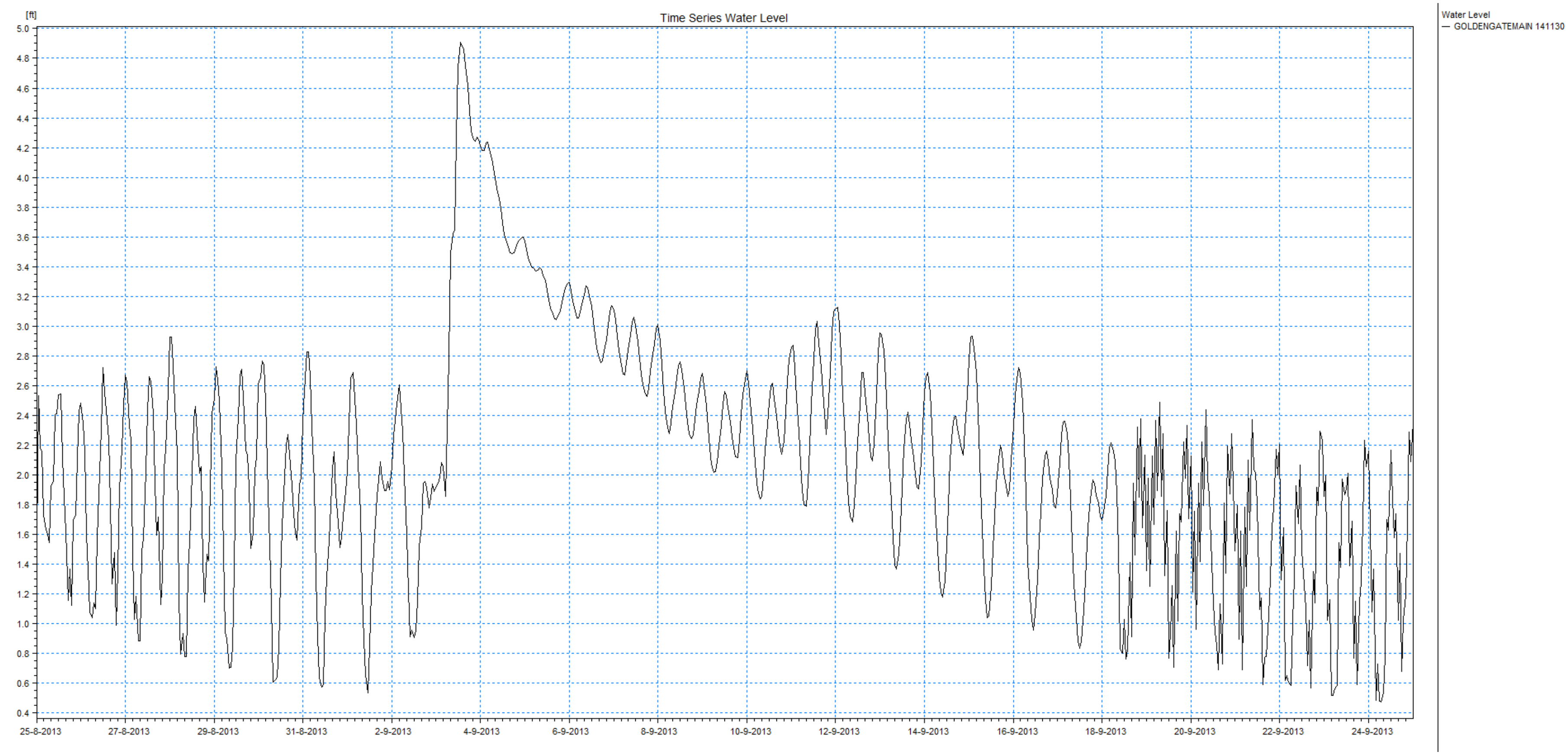


Figure 22. GG1 Tailwater Stage (ft)

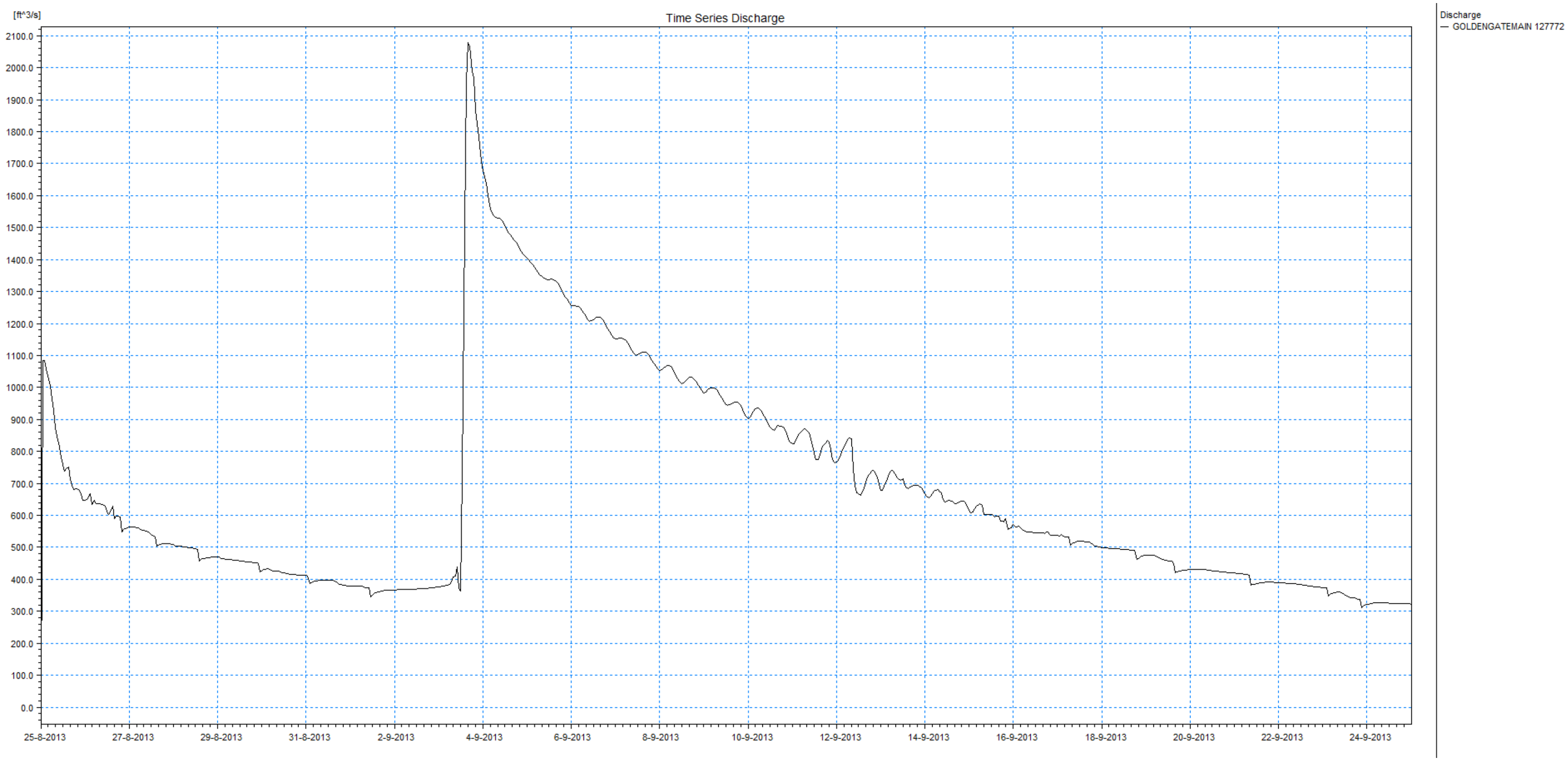
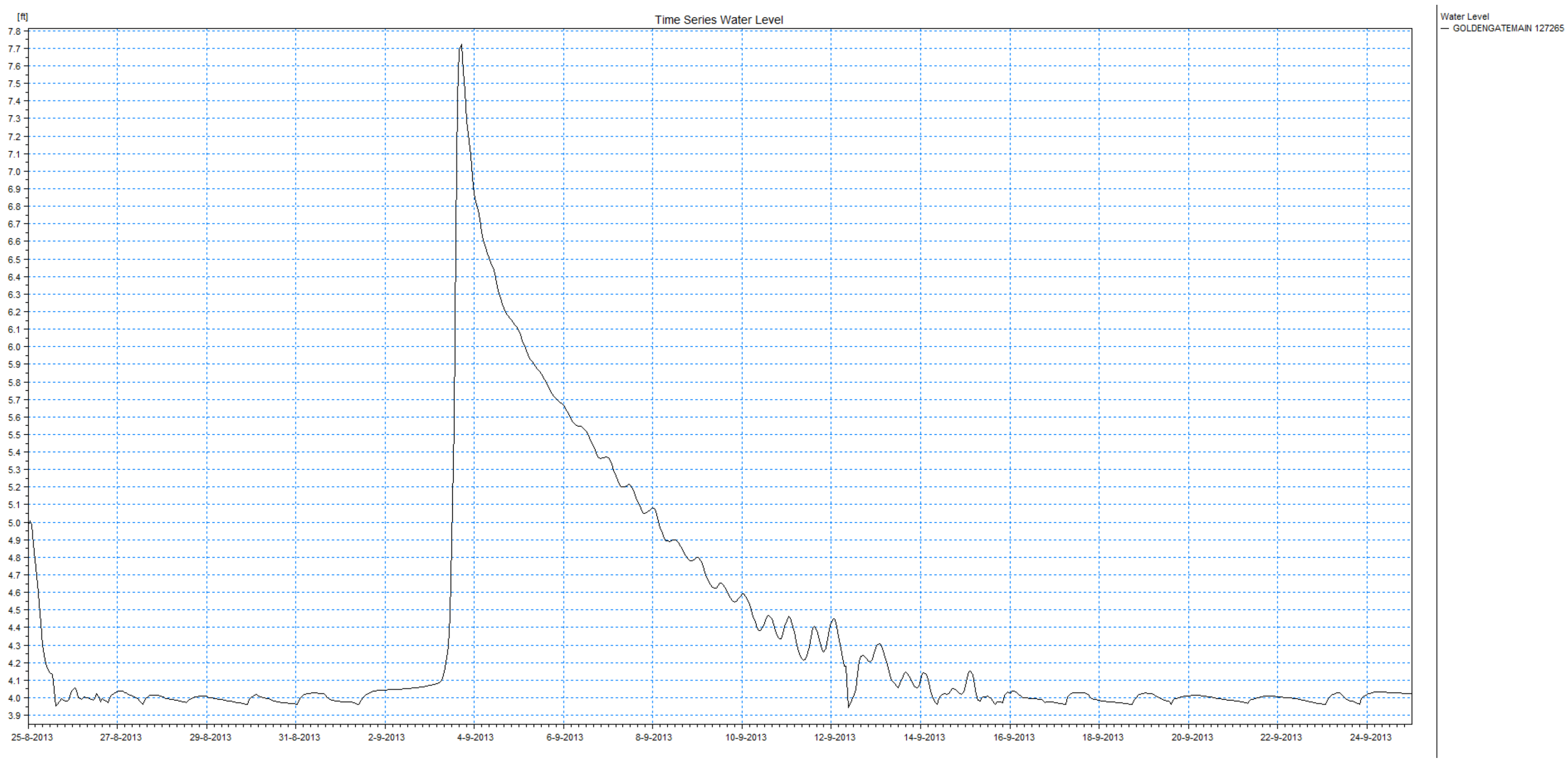


Figure 23. GG2 Discharge (cfs)



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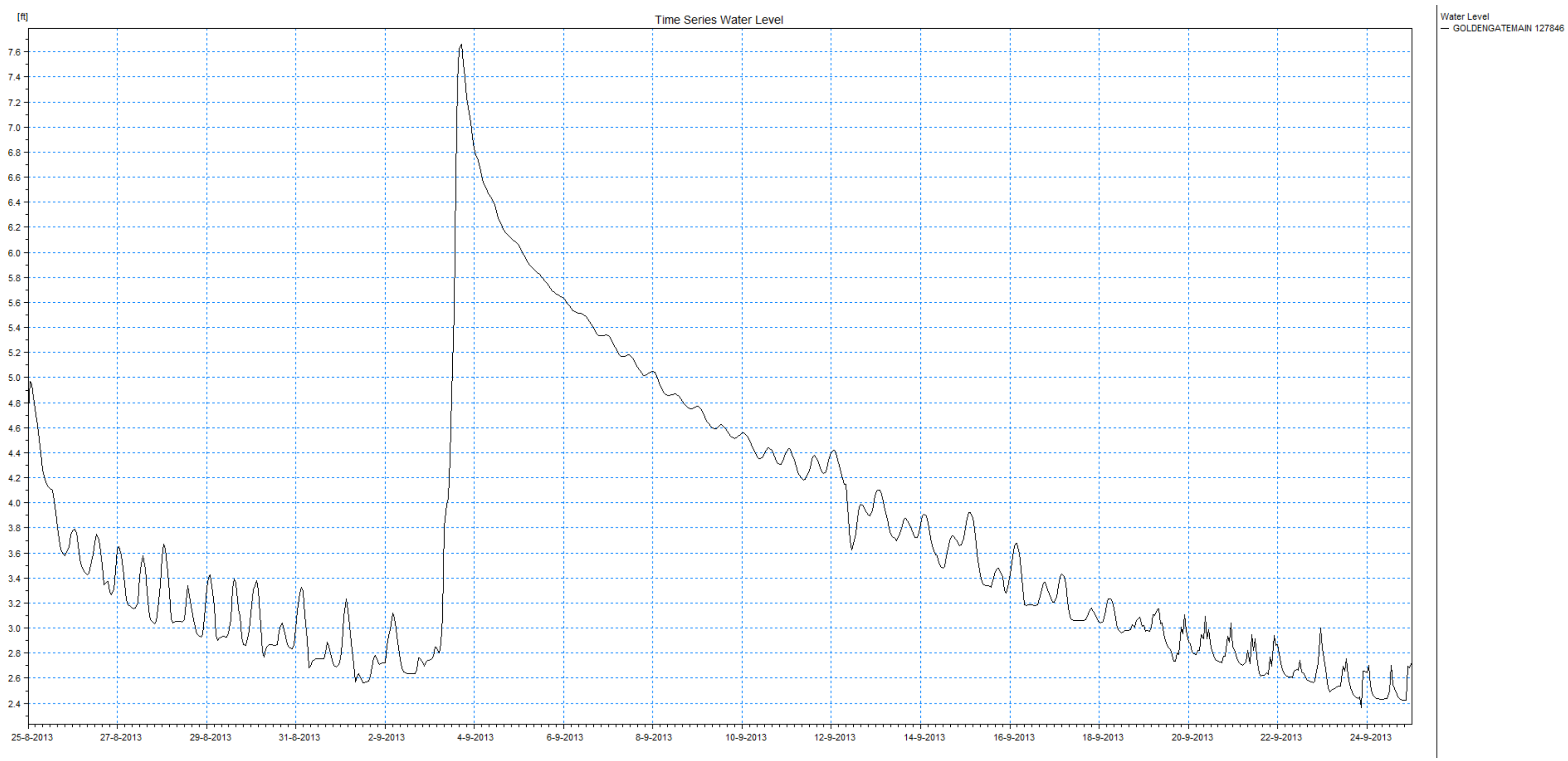

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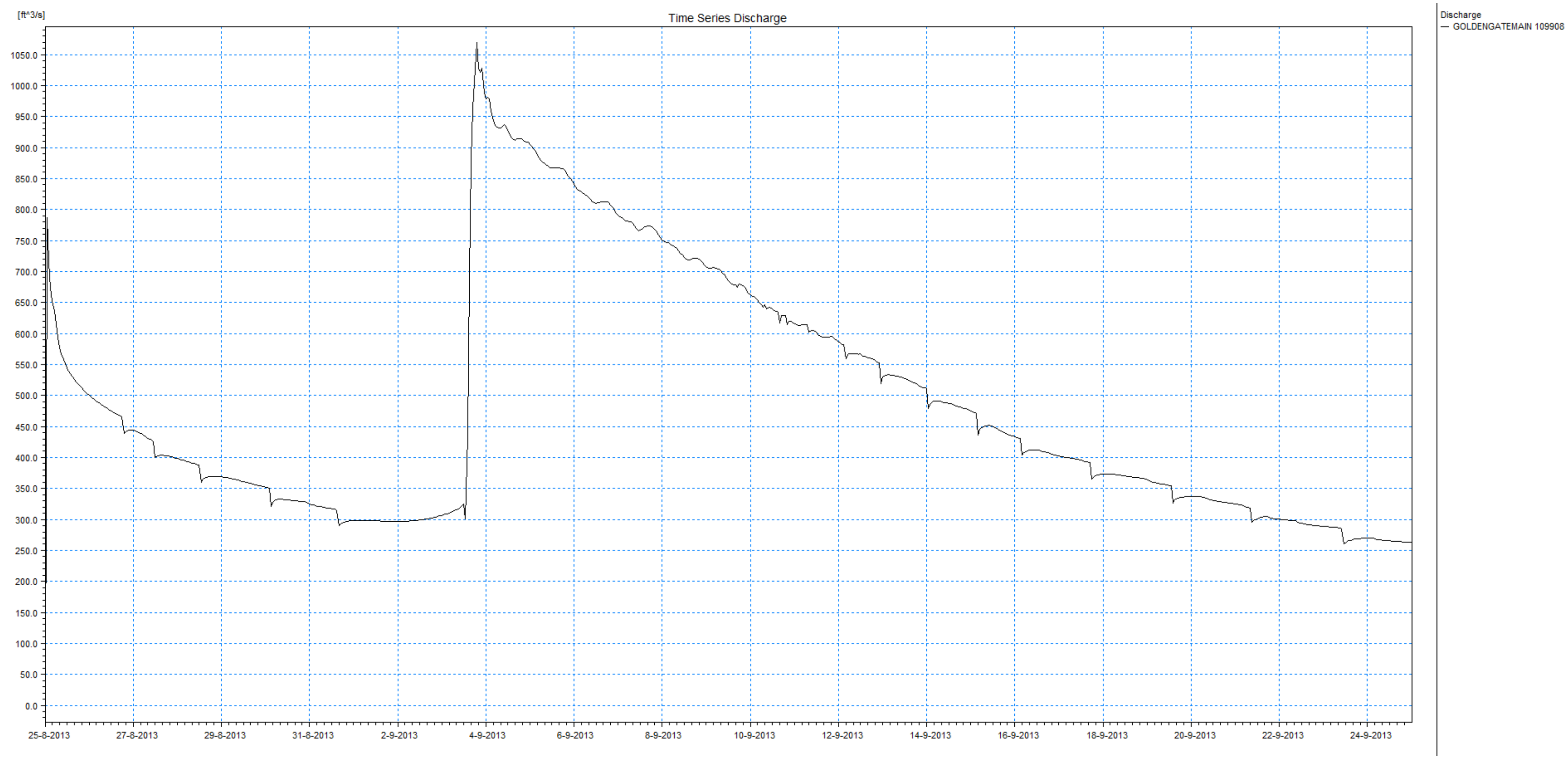

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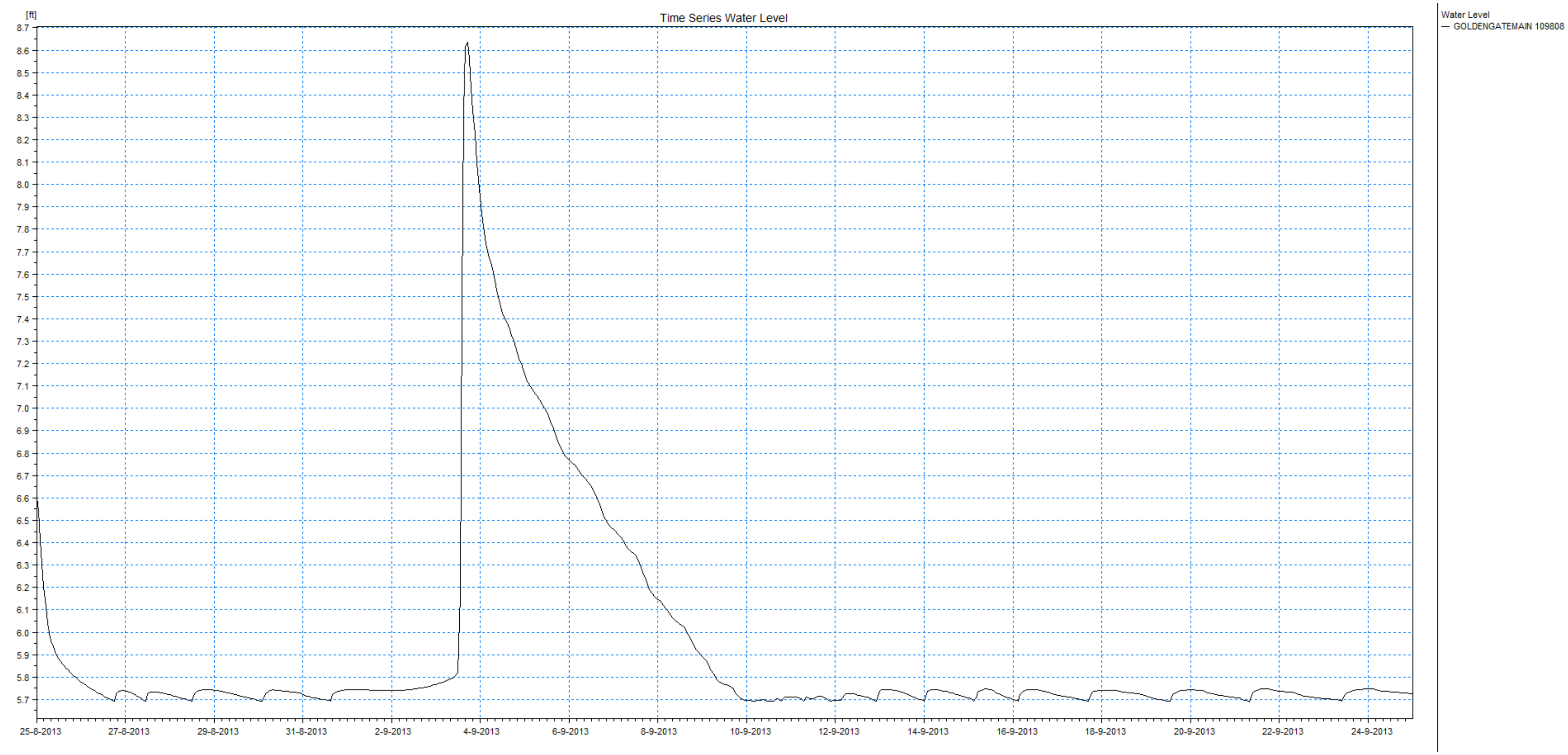
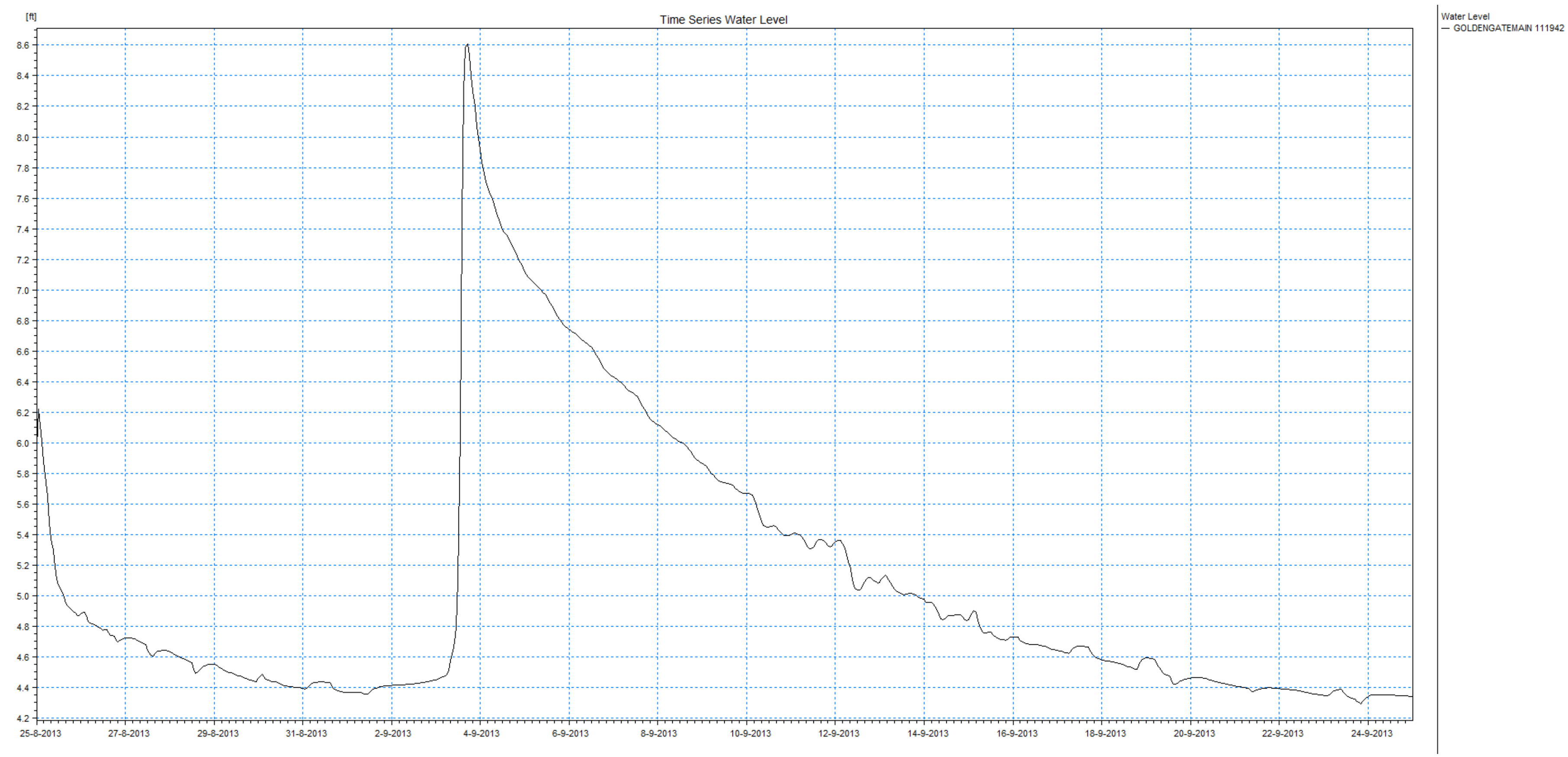


Figure 27. GG3 Headwater Stage (ft)



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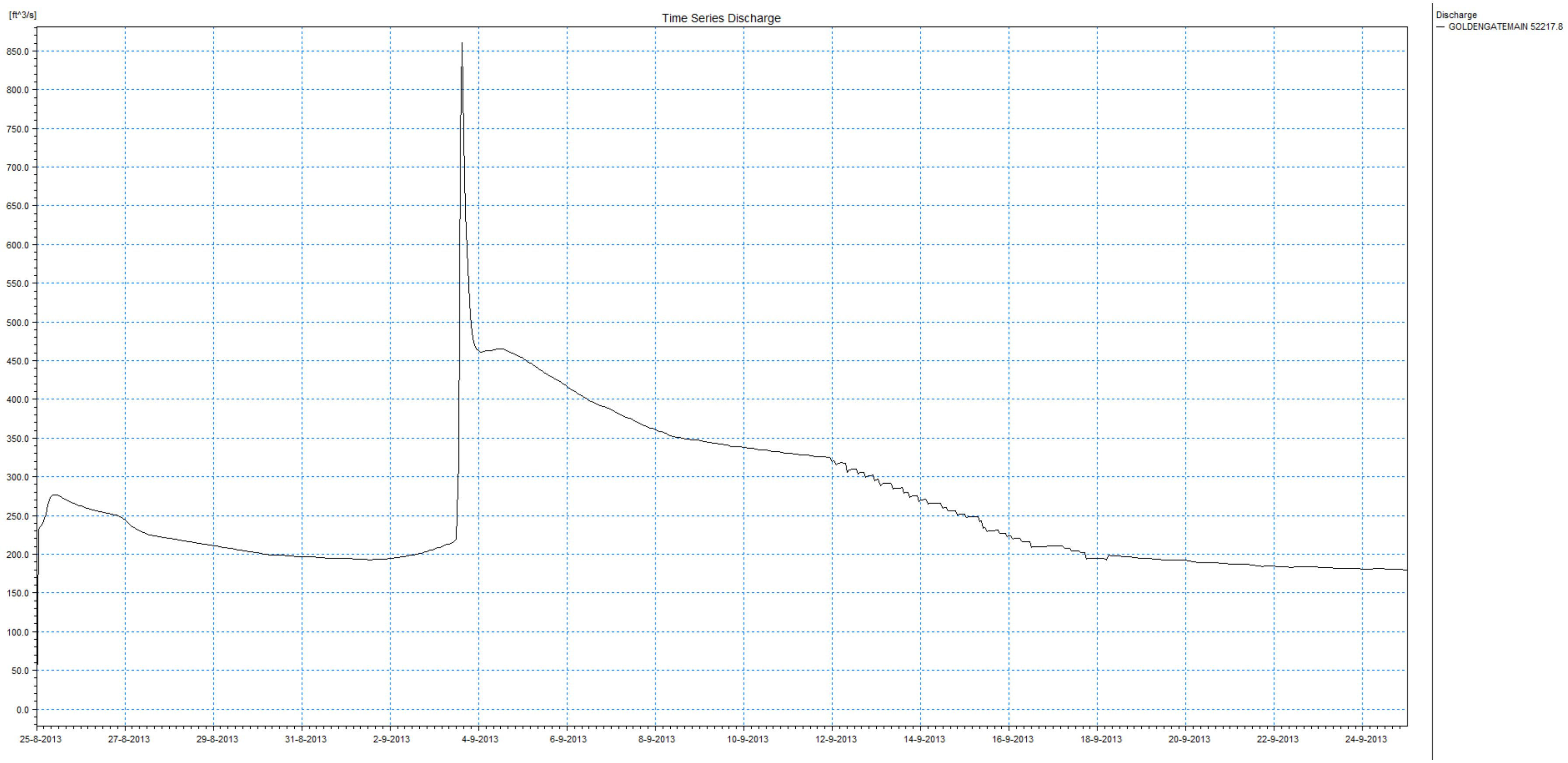
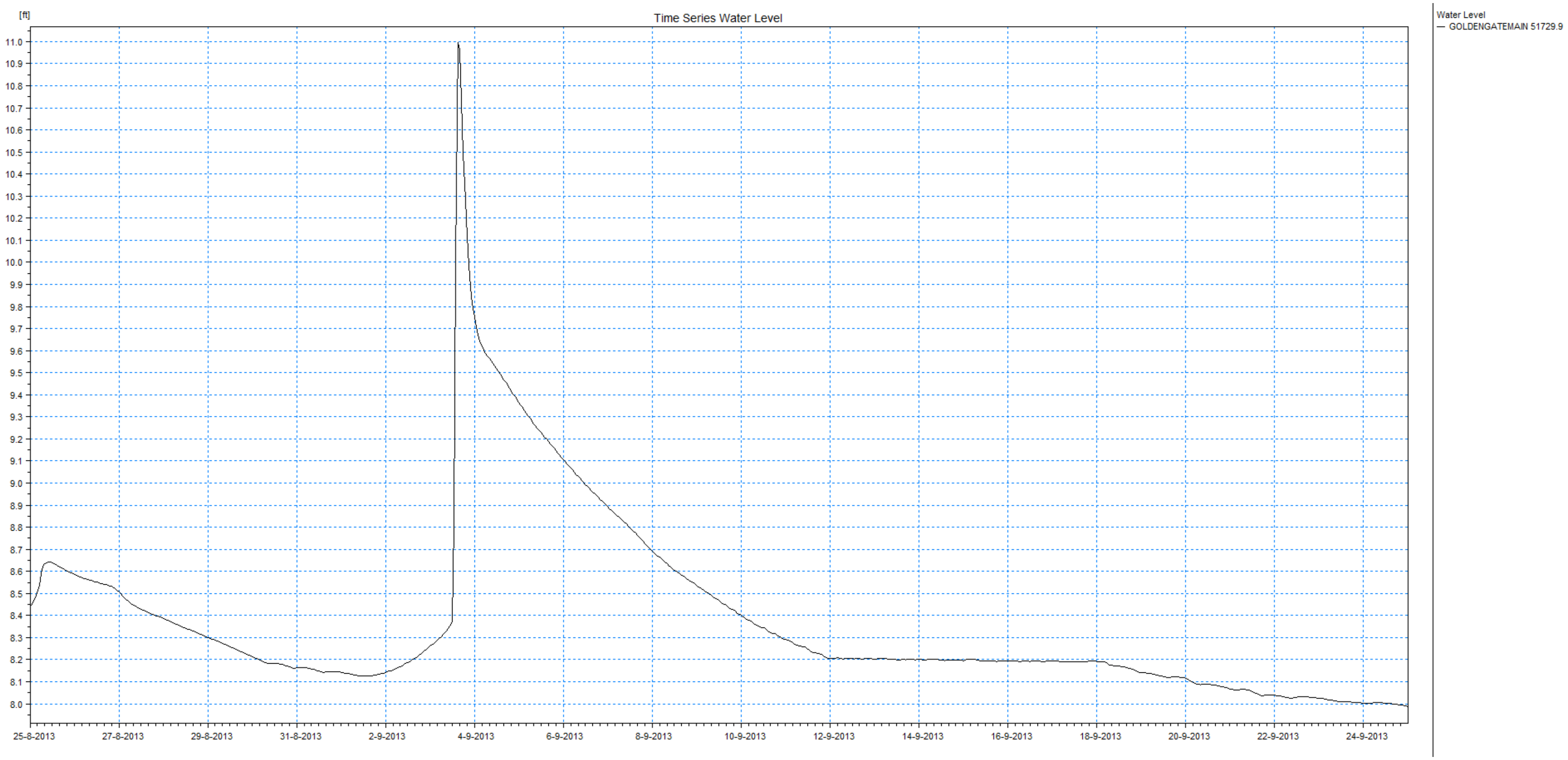


Figure 29. GG4 Discharge (cfs)



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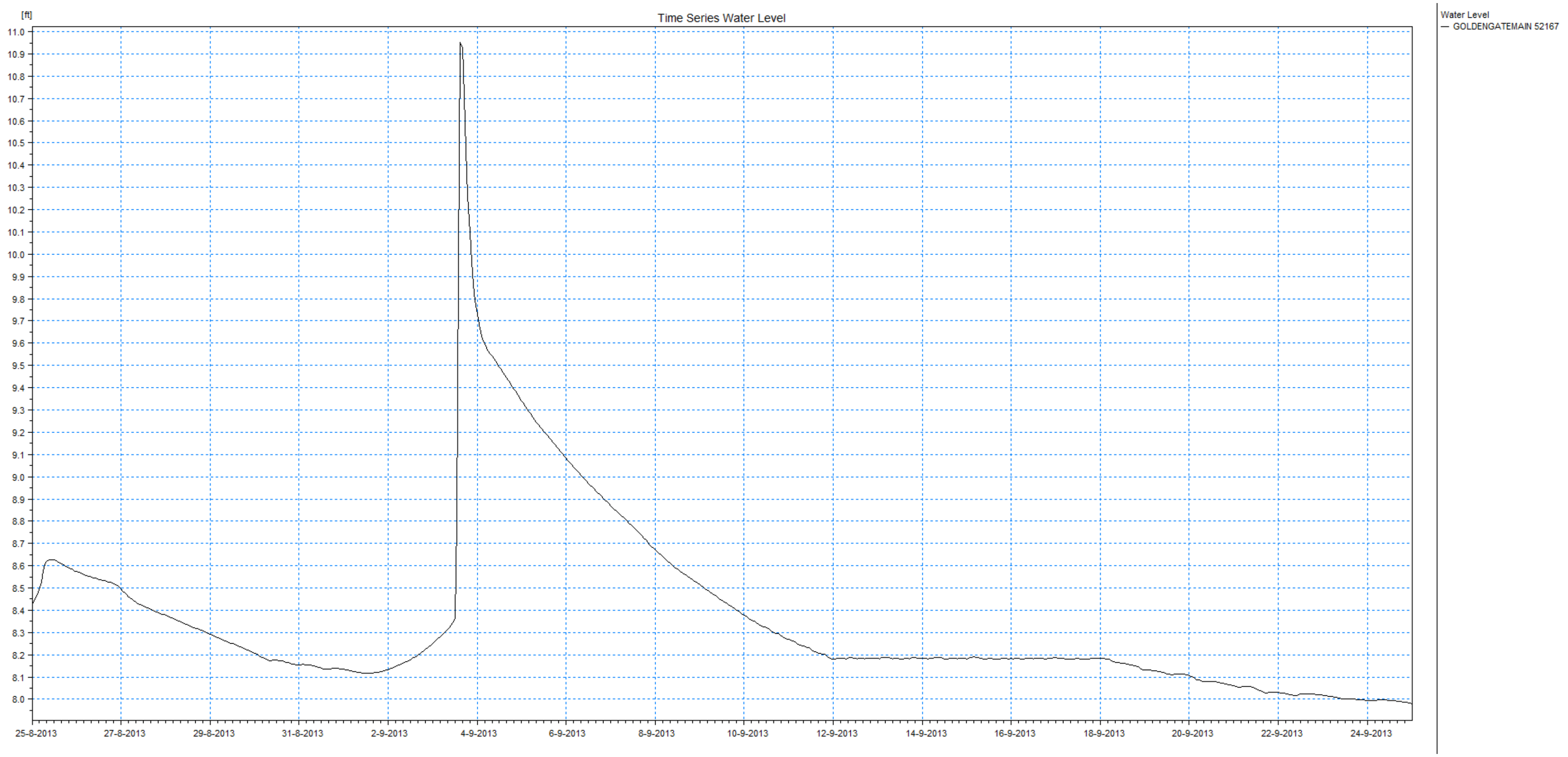


Figure 31. GG4 Tailwater Stage (ft)

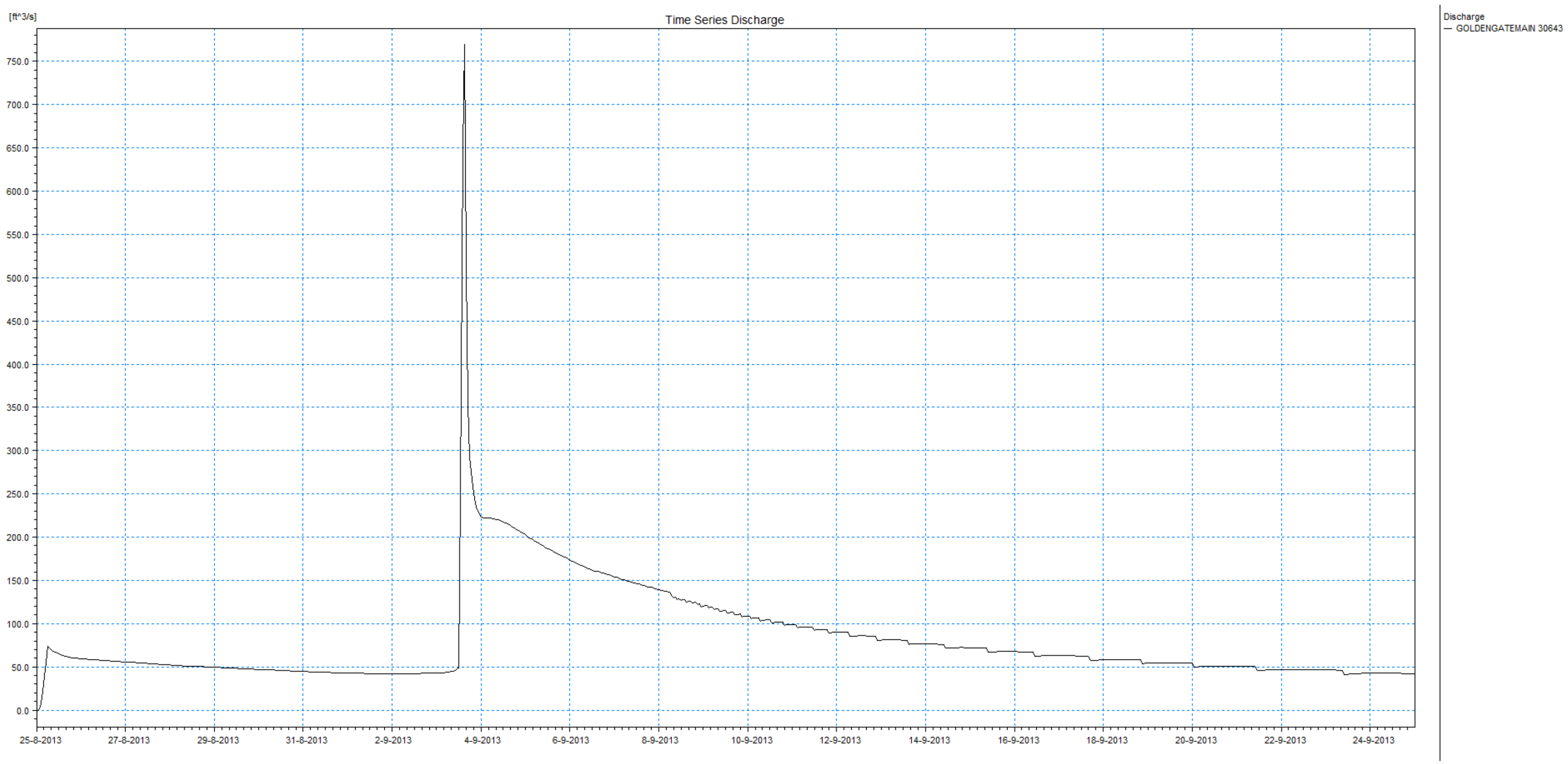
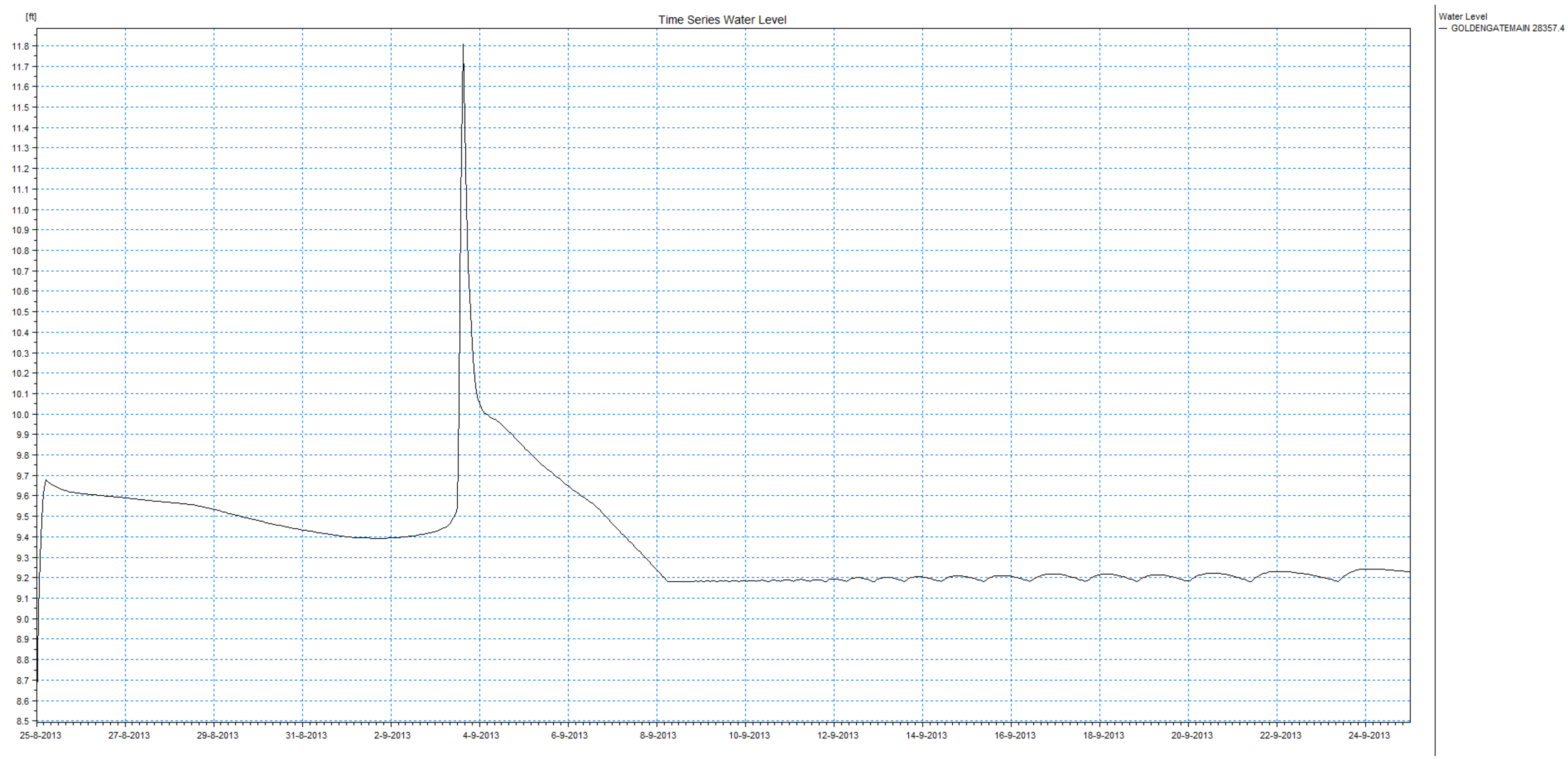


Figure 32. GG5 Discharge (cfs)



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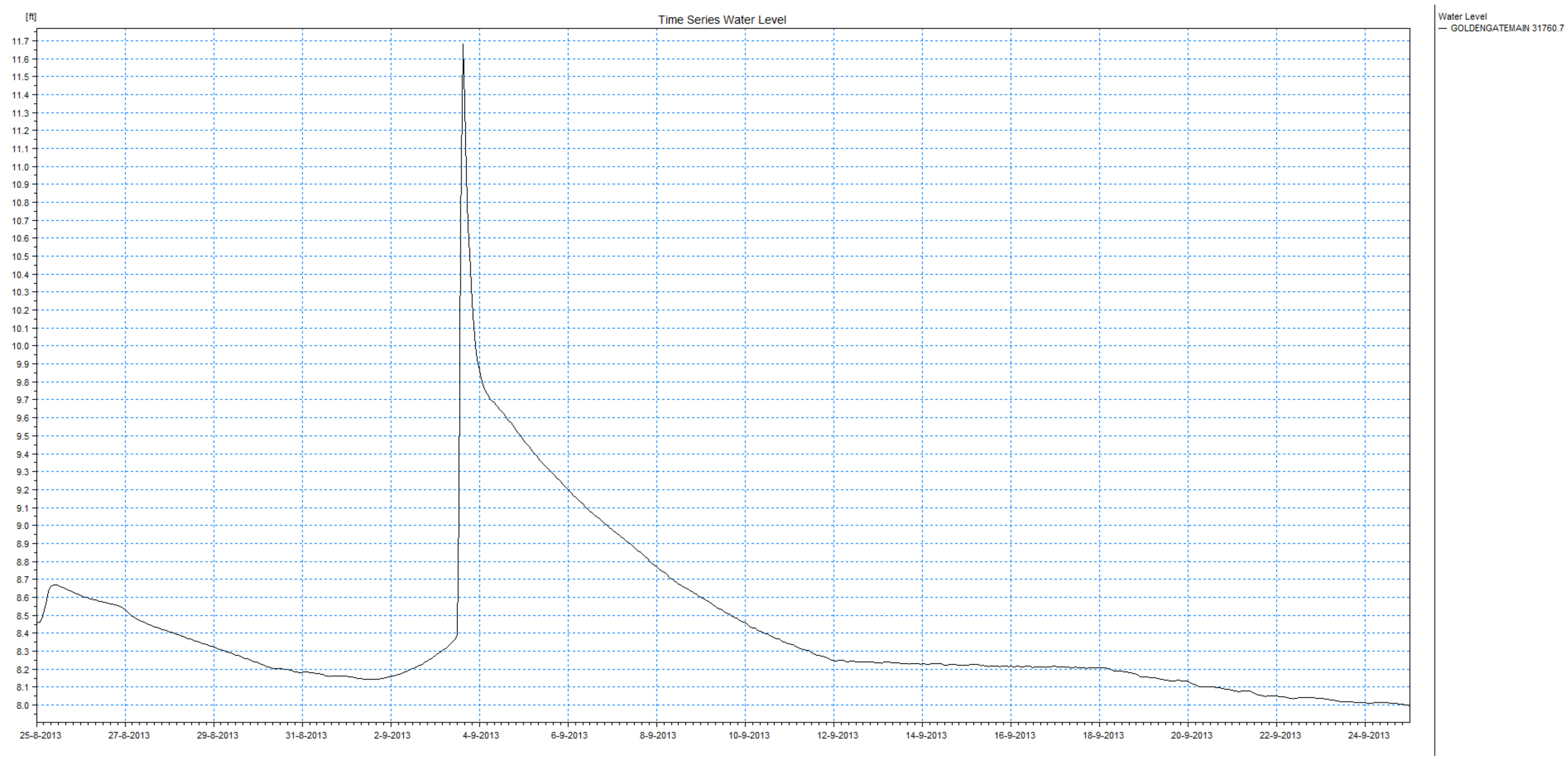

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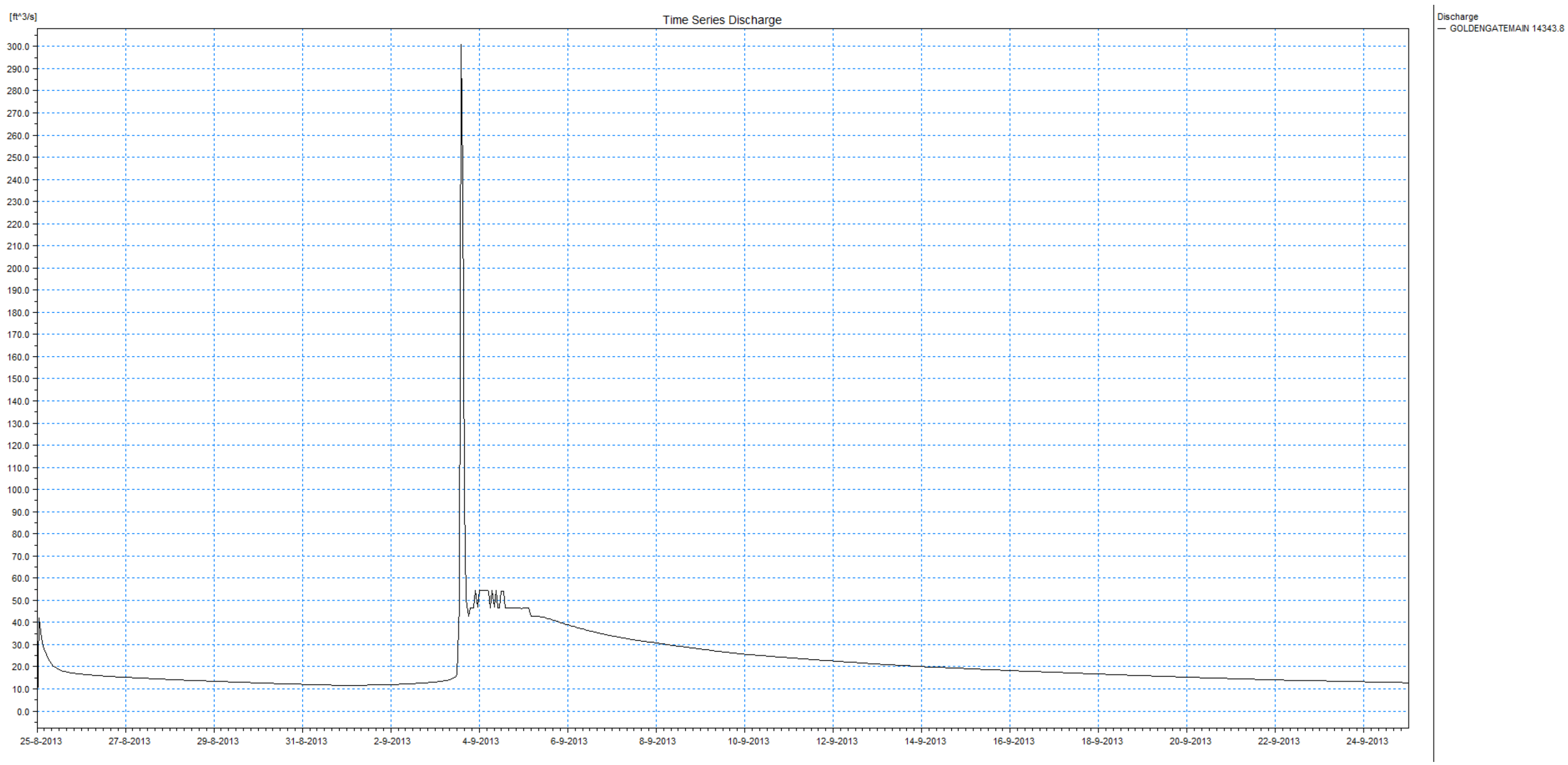

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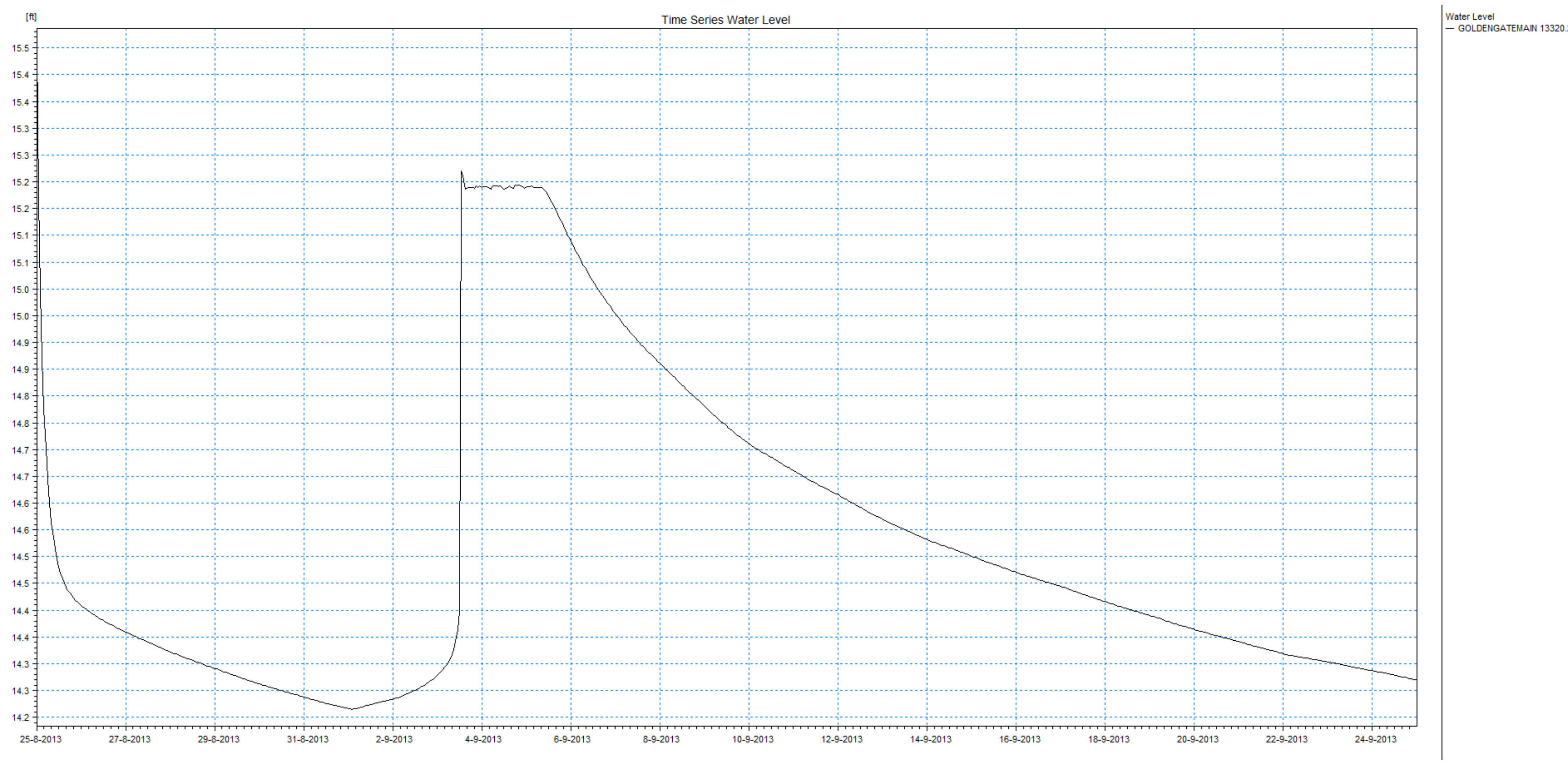

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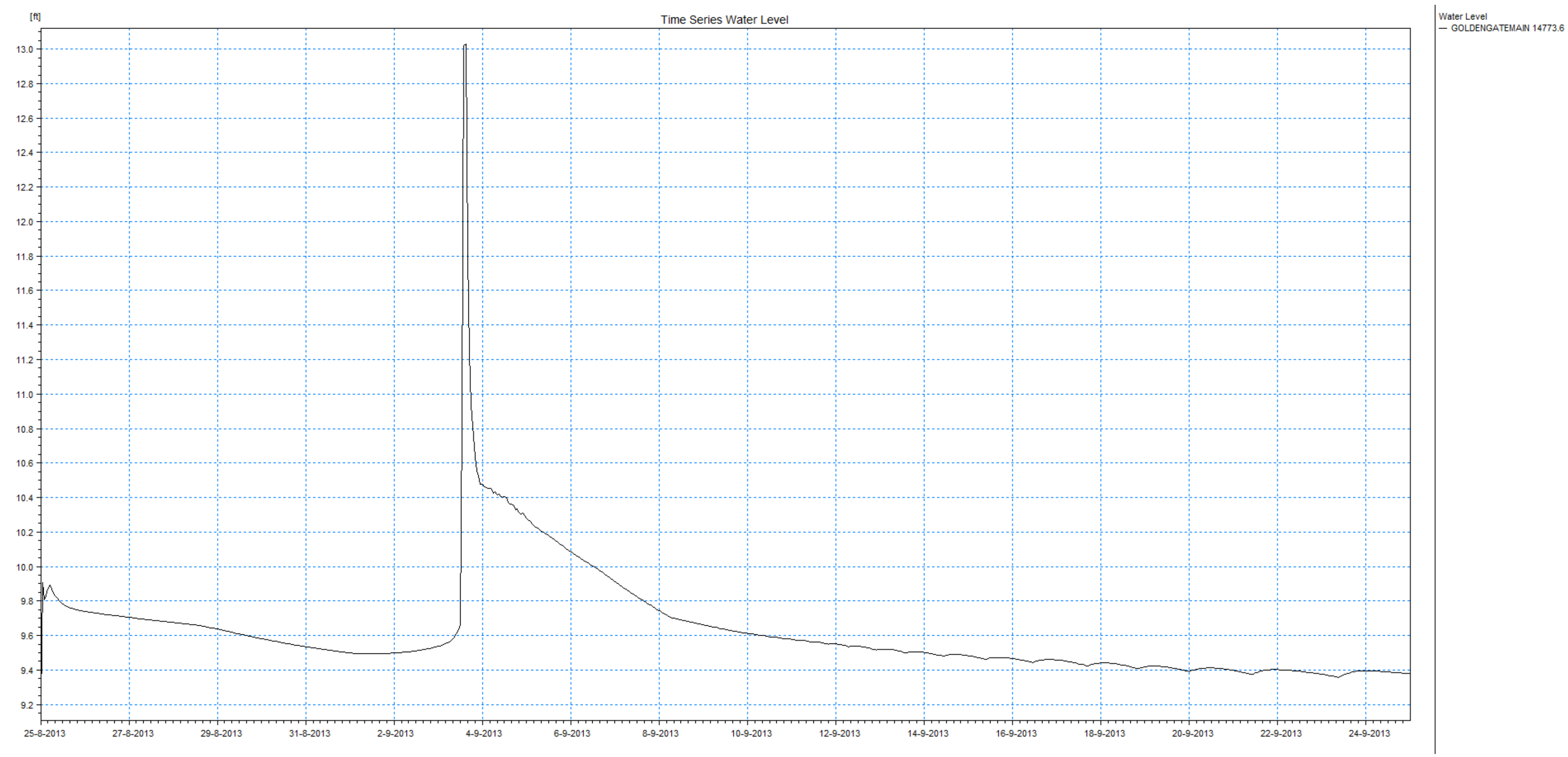

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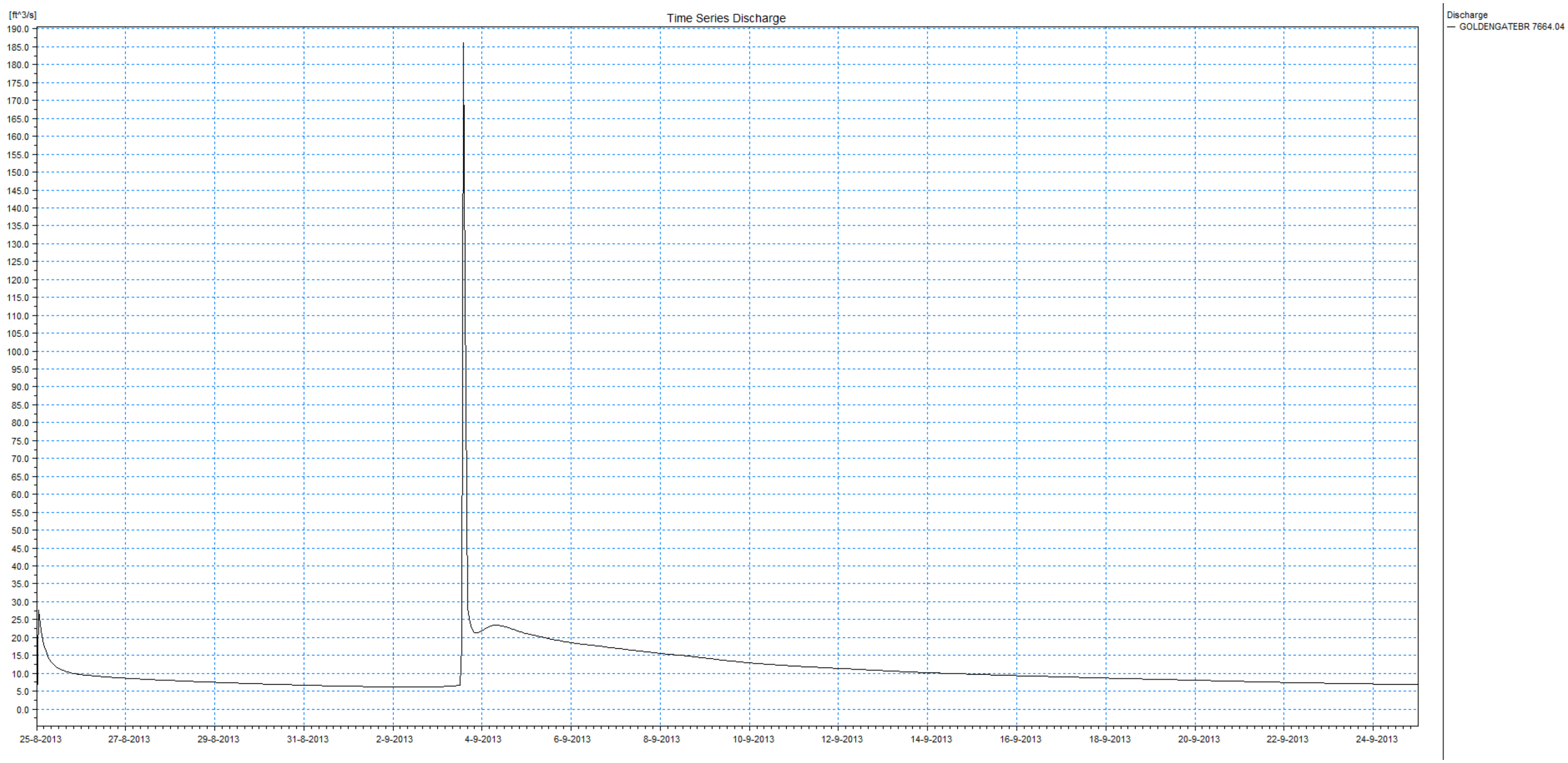

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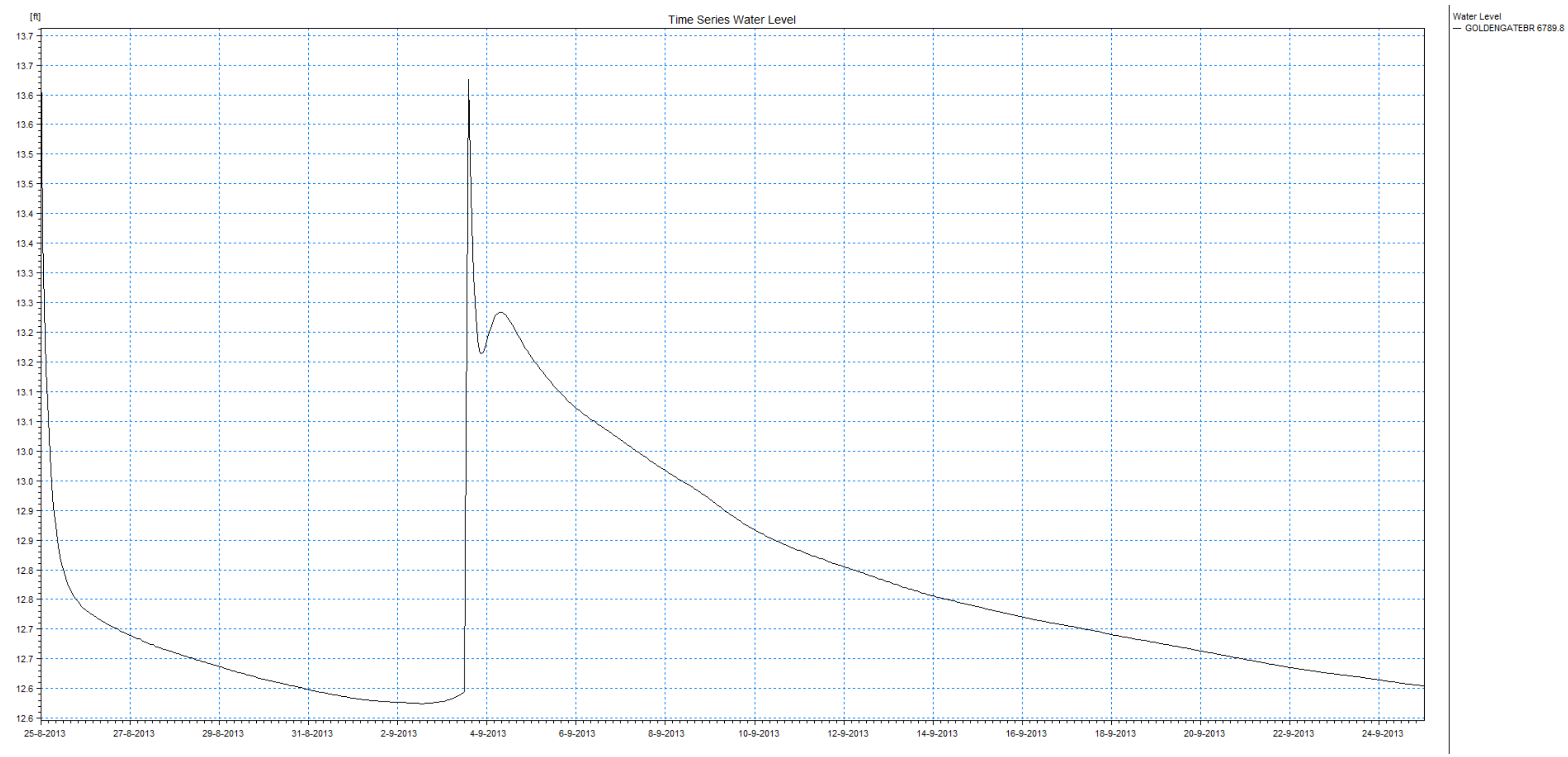

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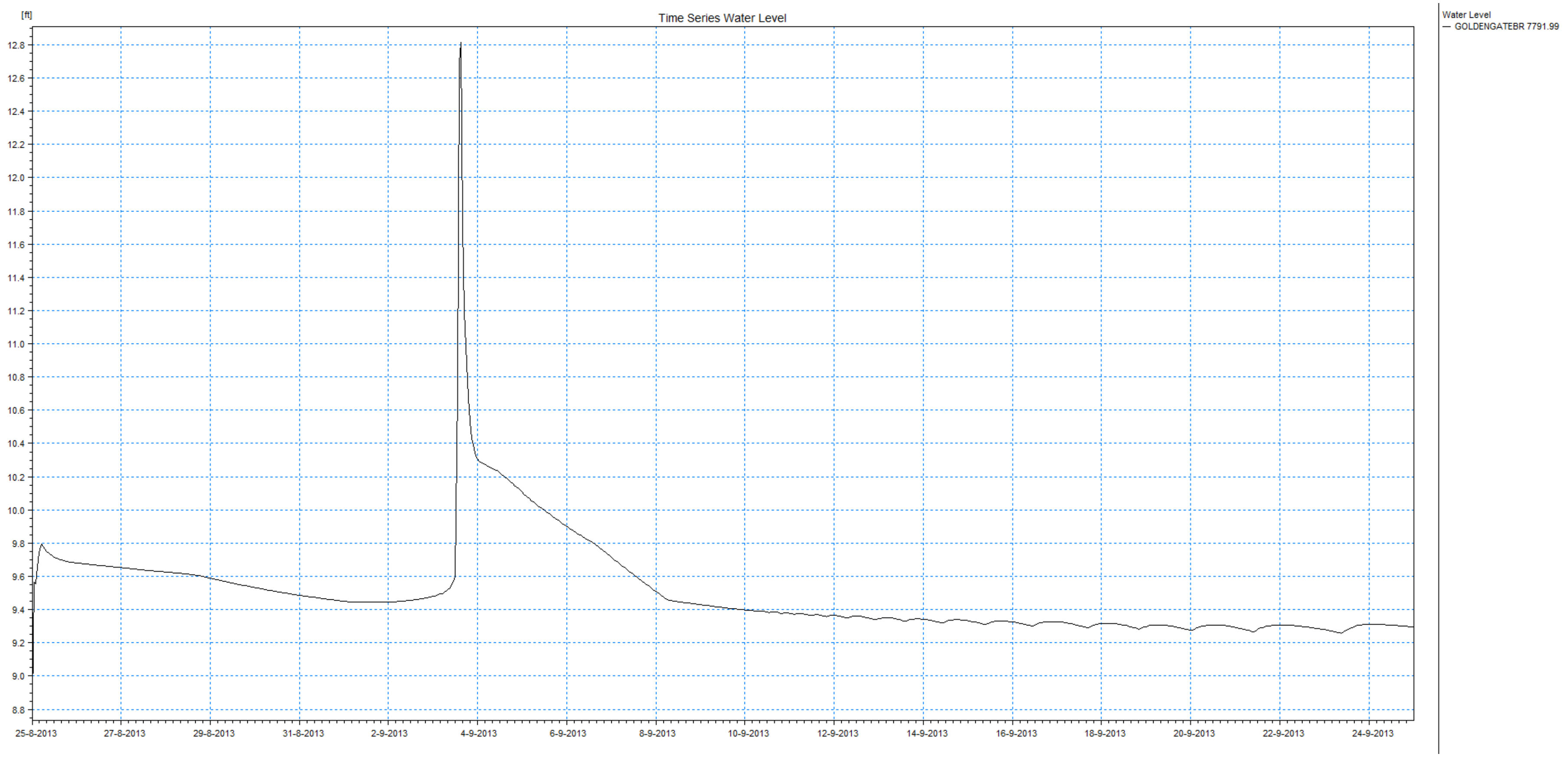

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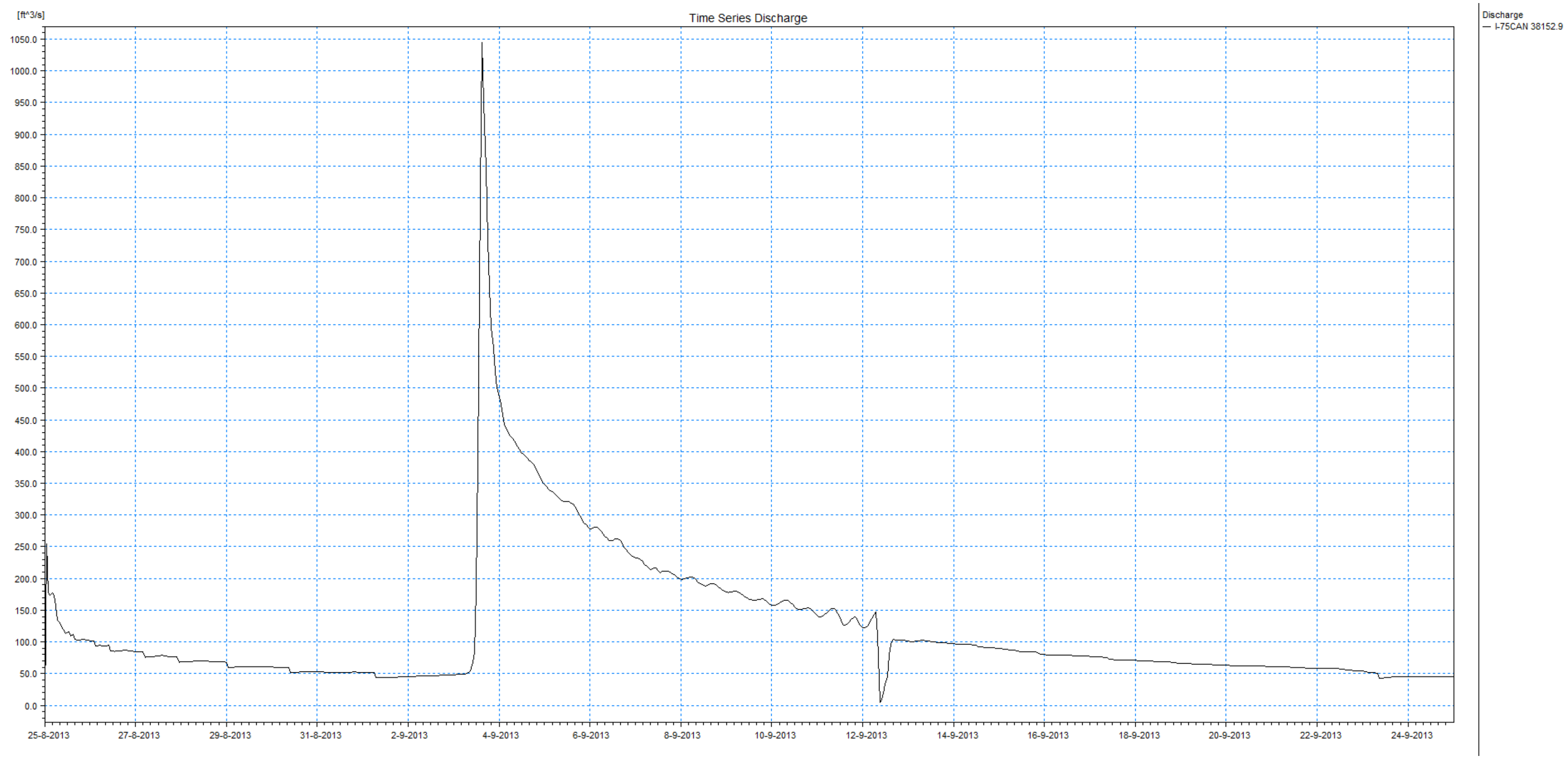

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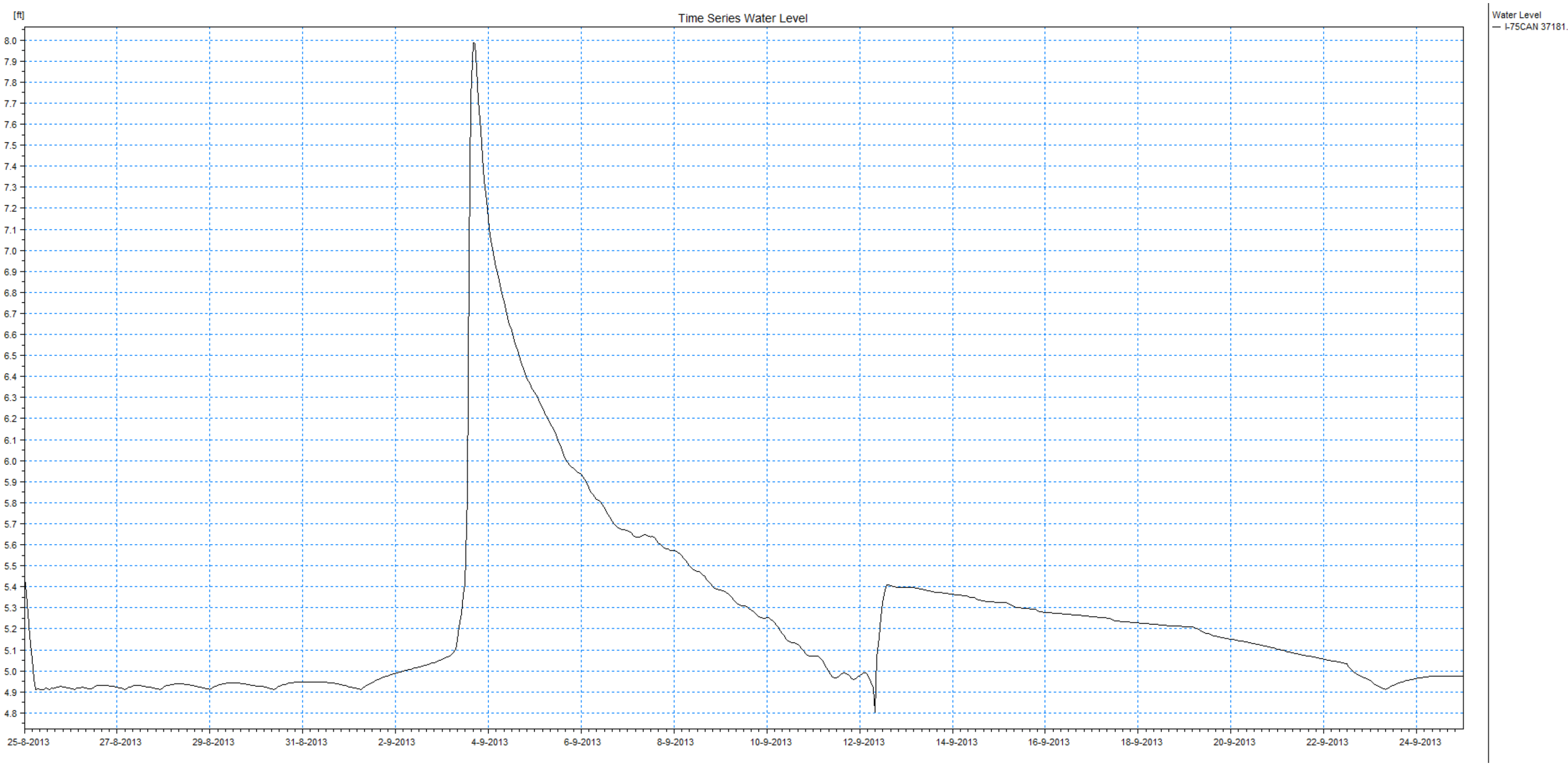

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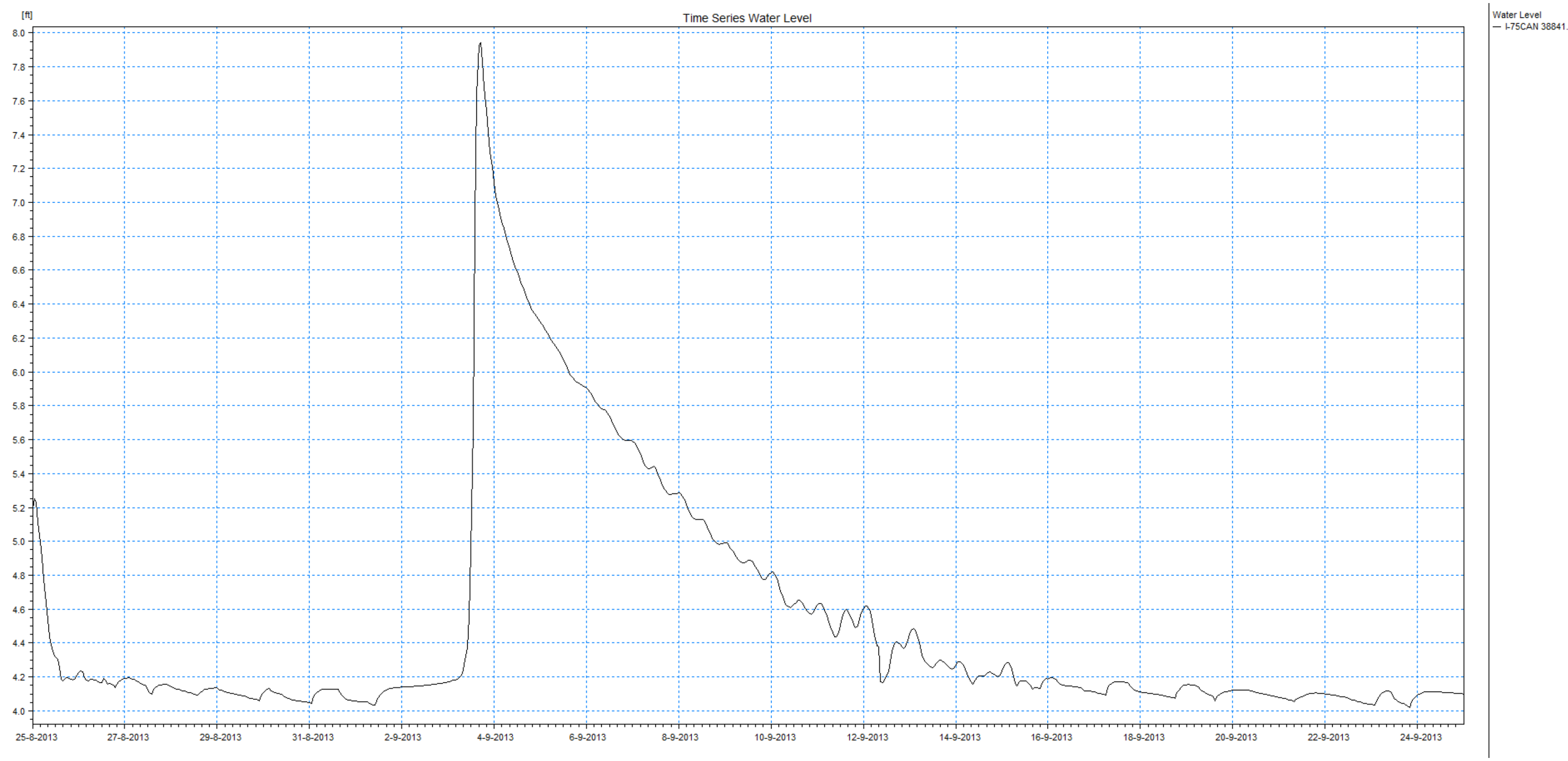

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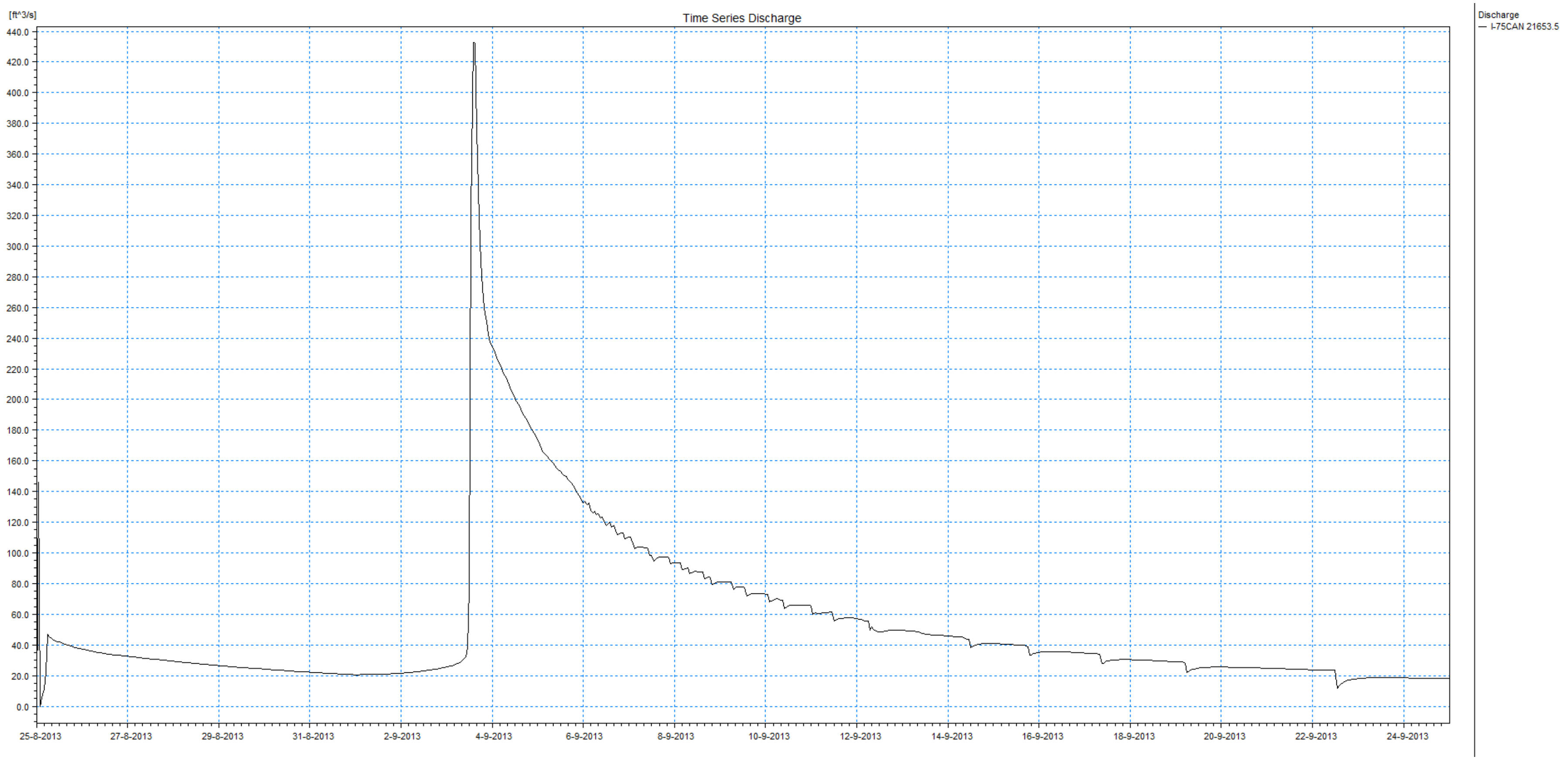

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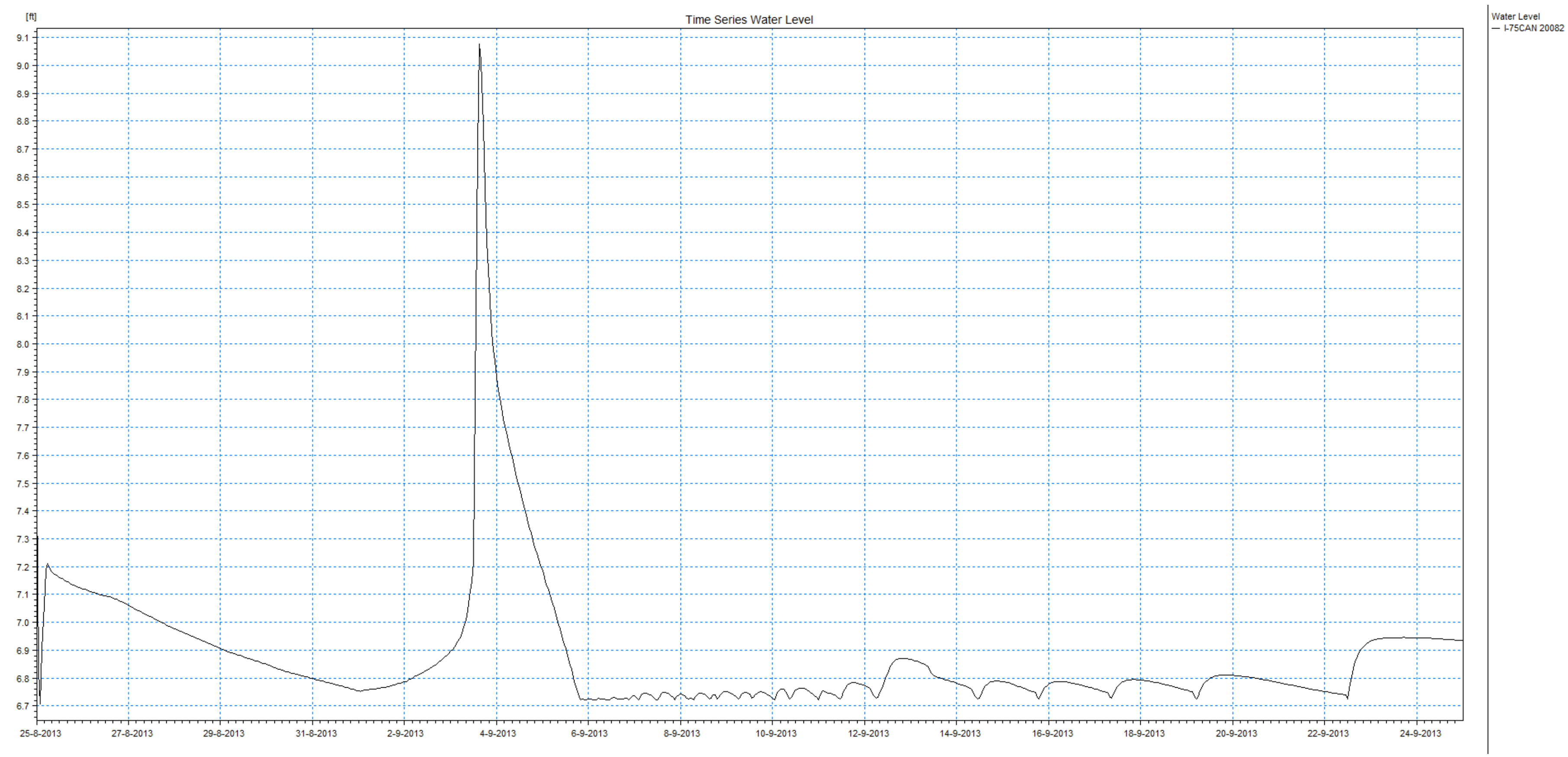

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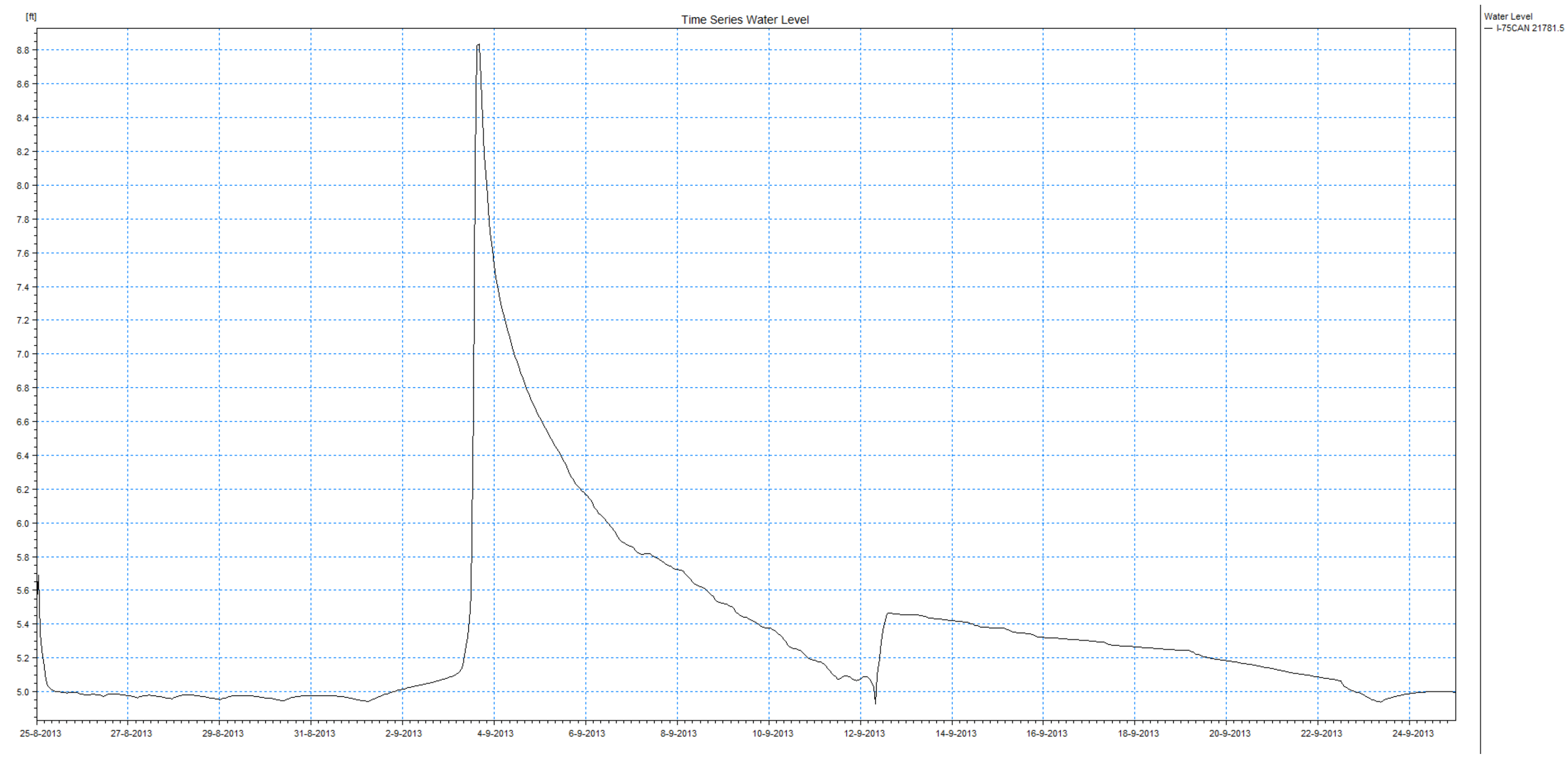

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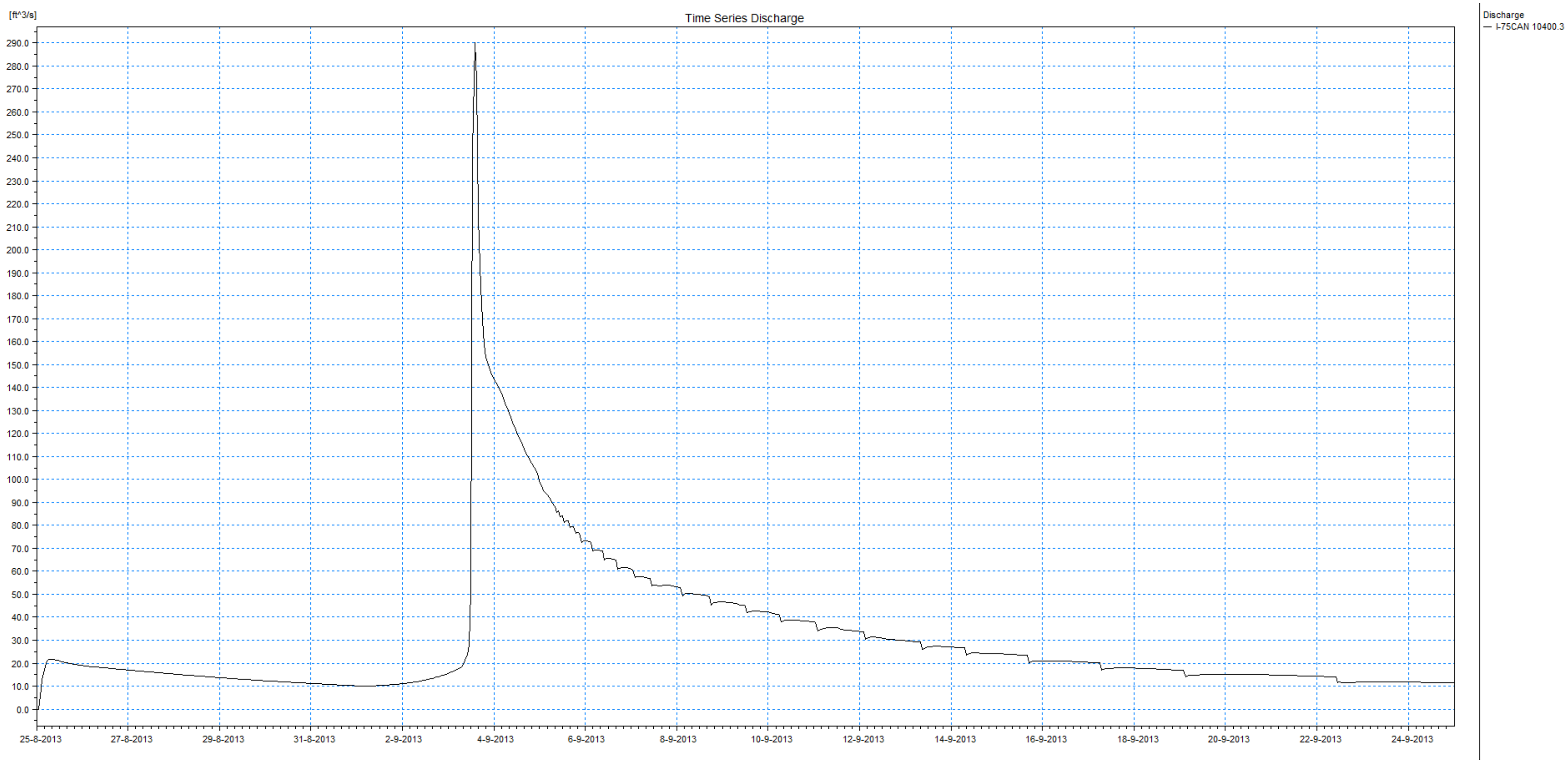

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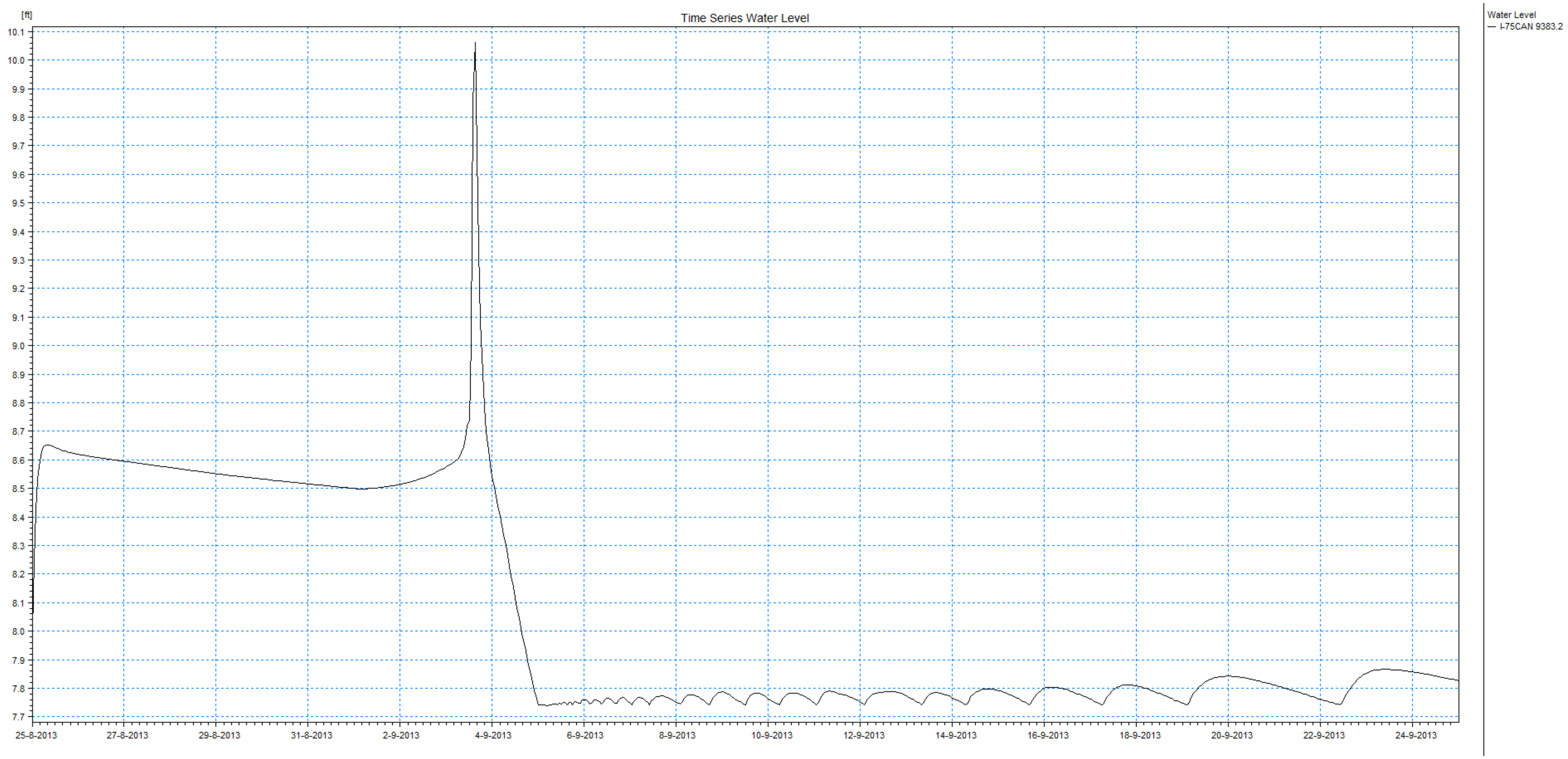

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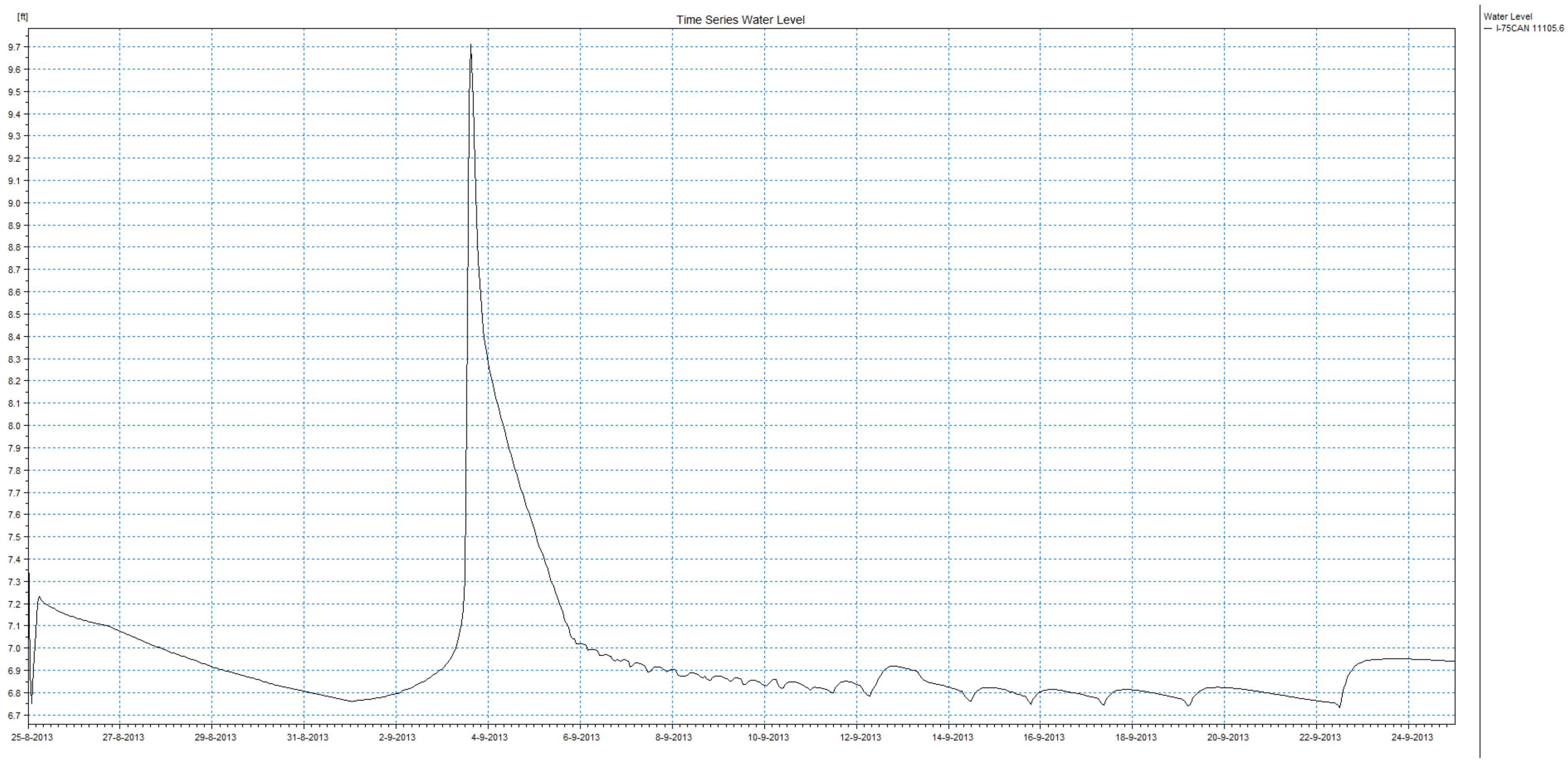

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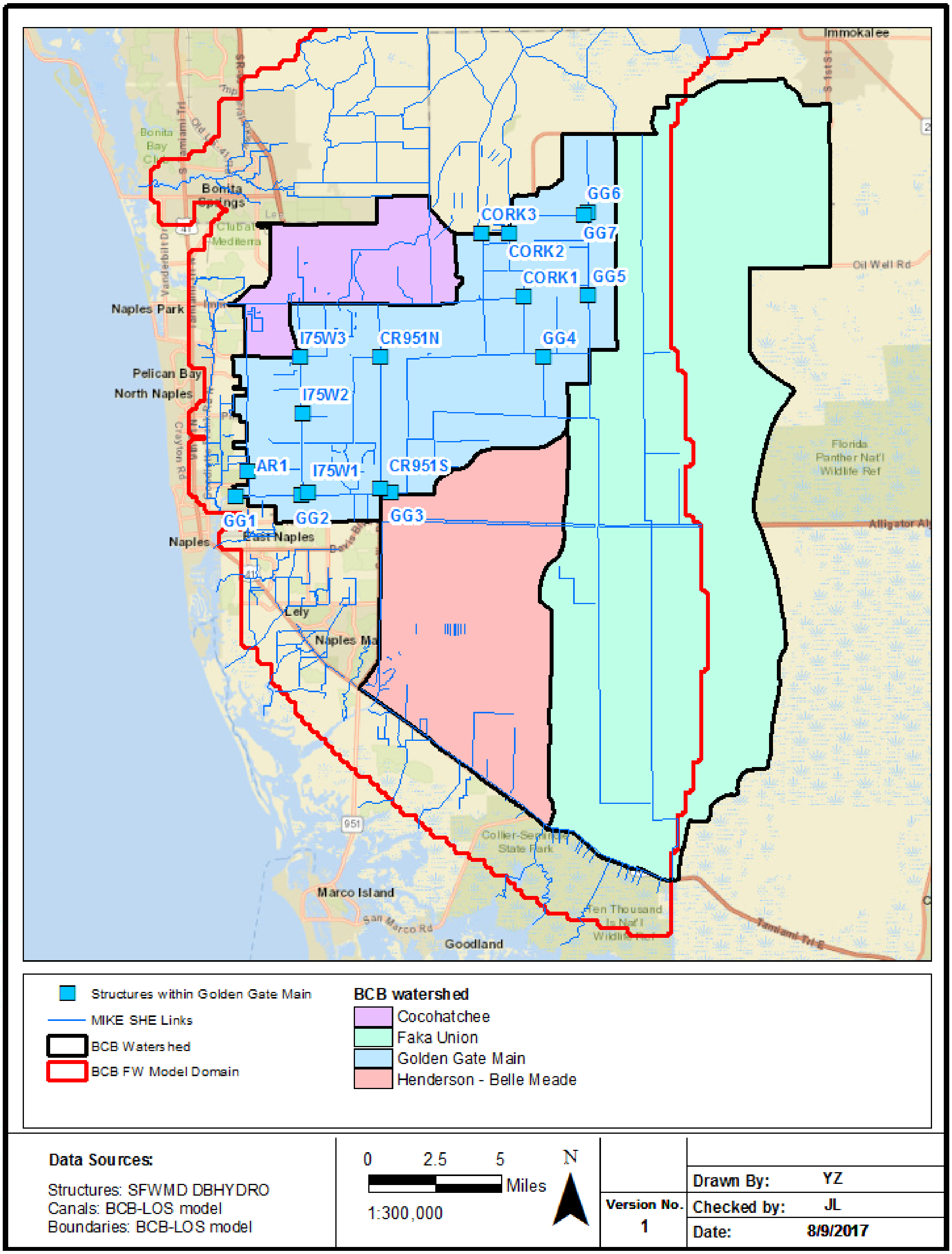

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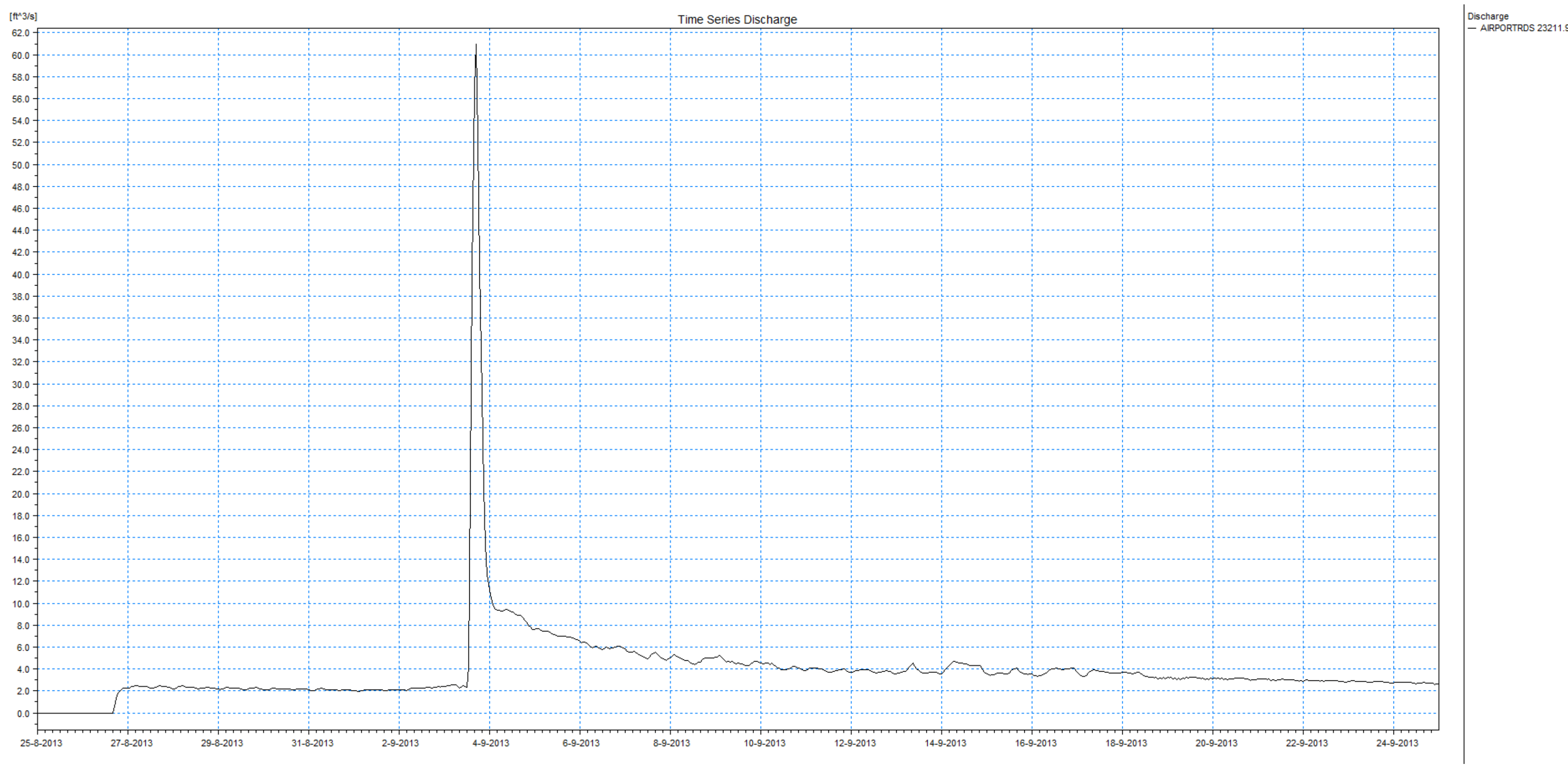

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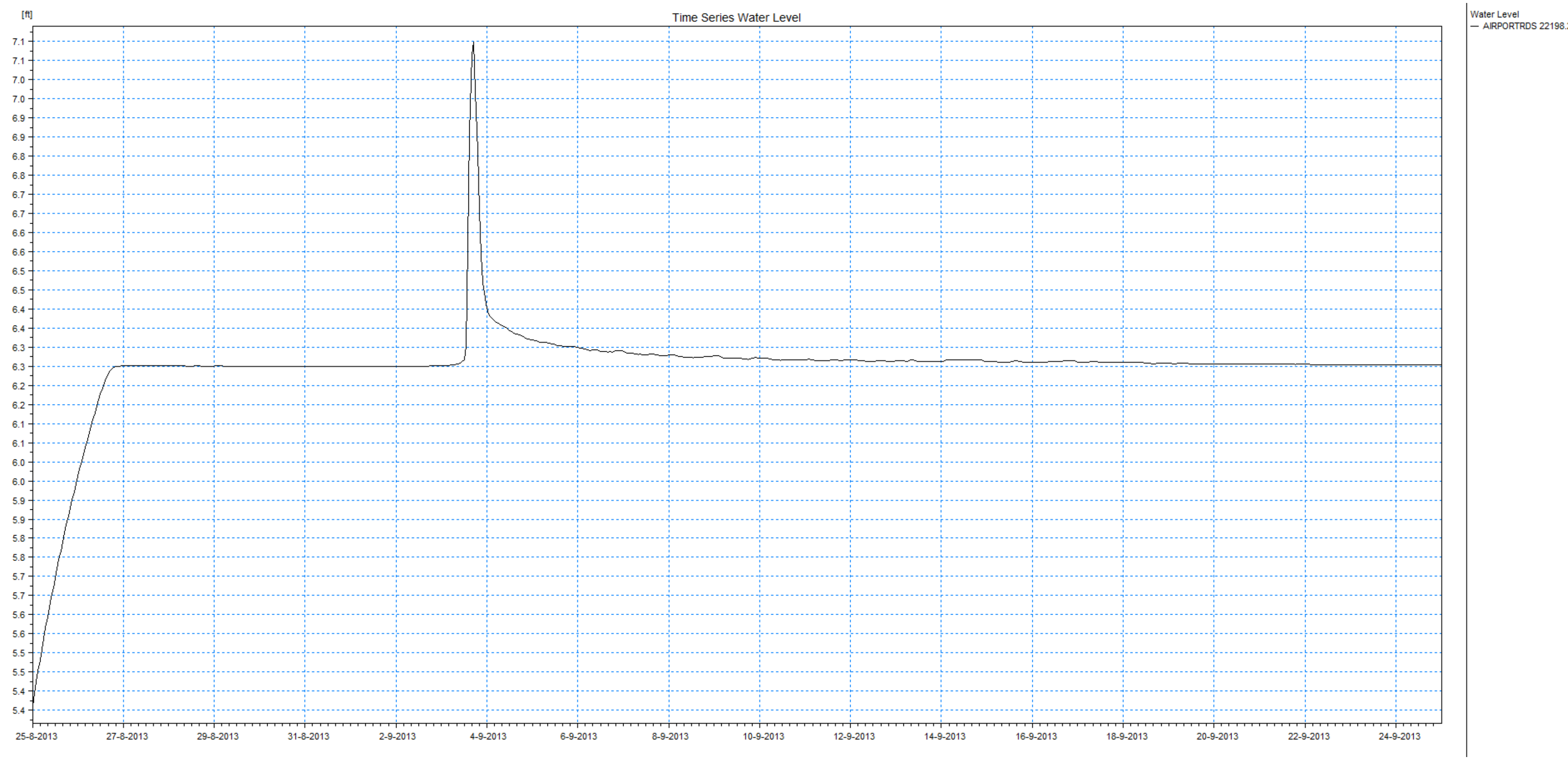

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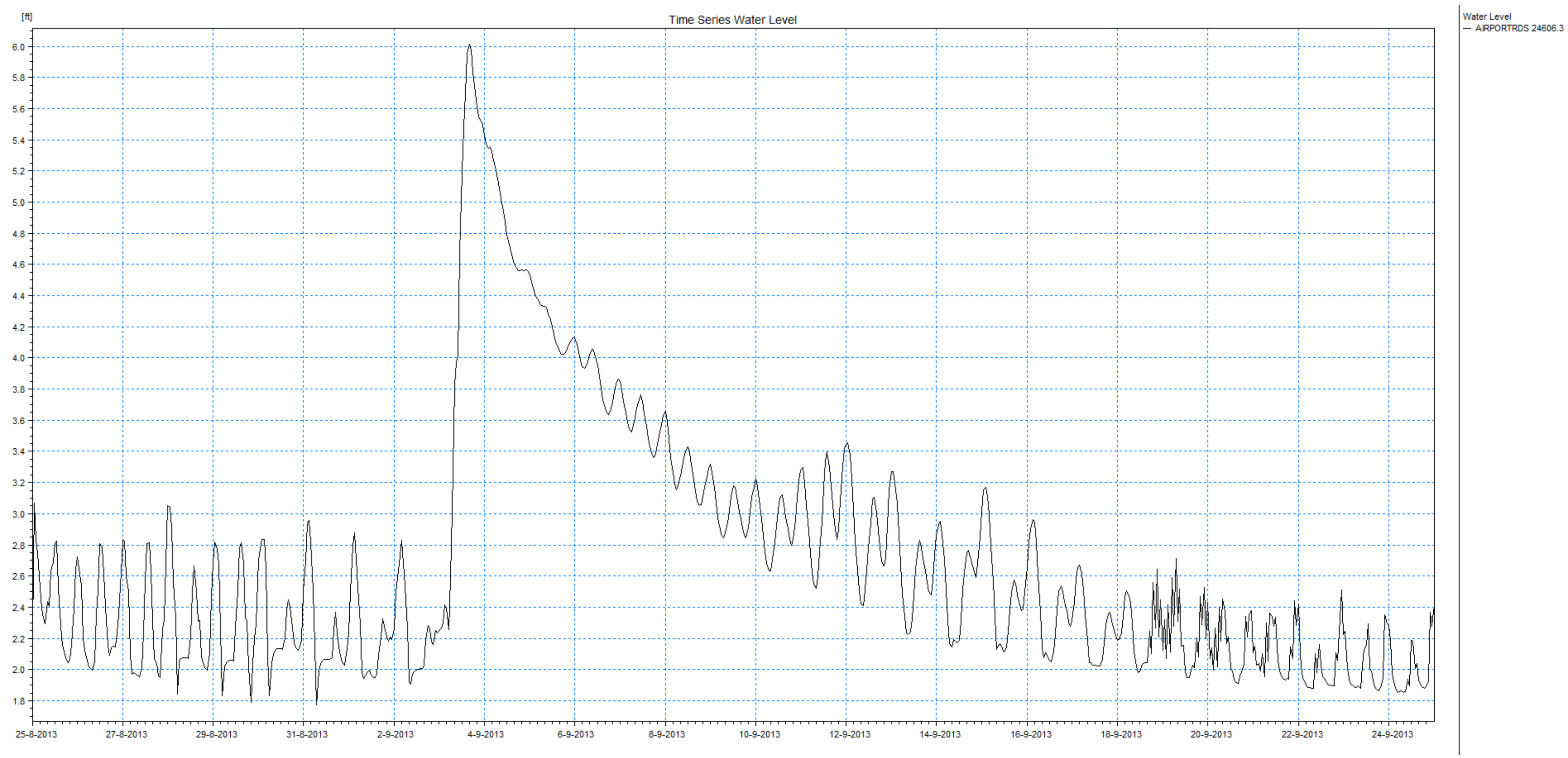

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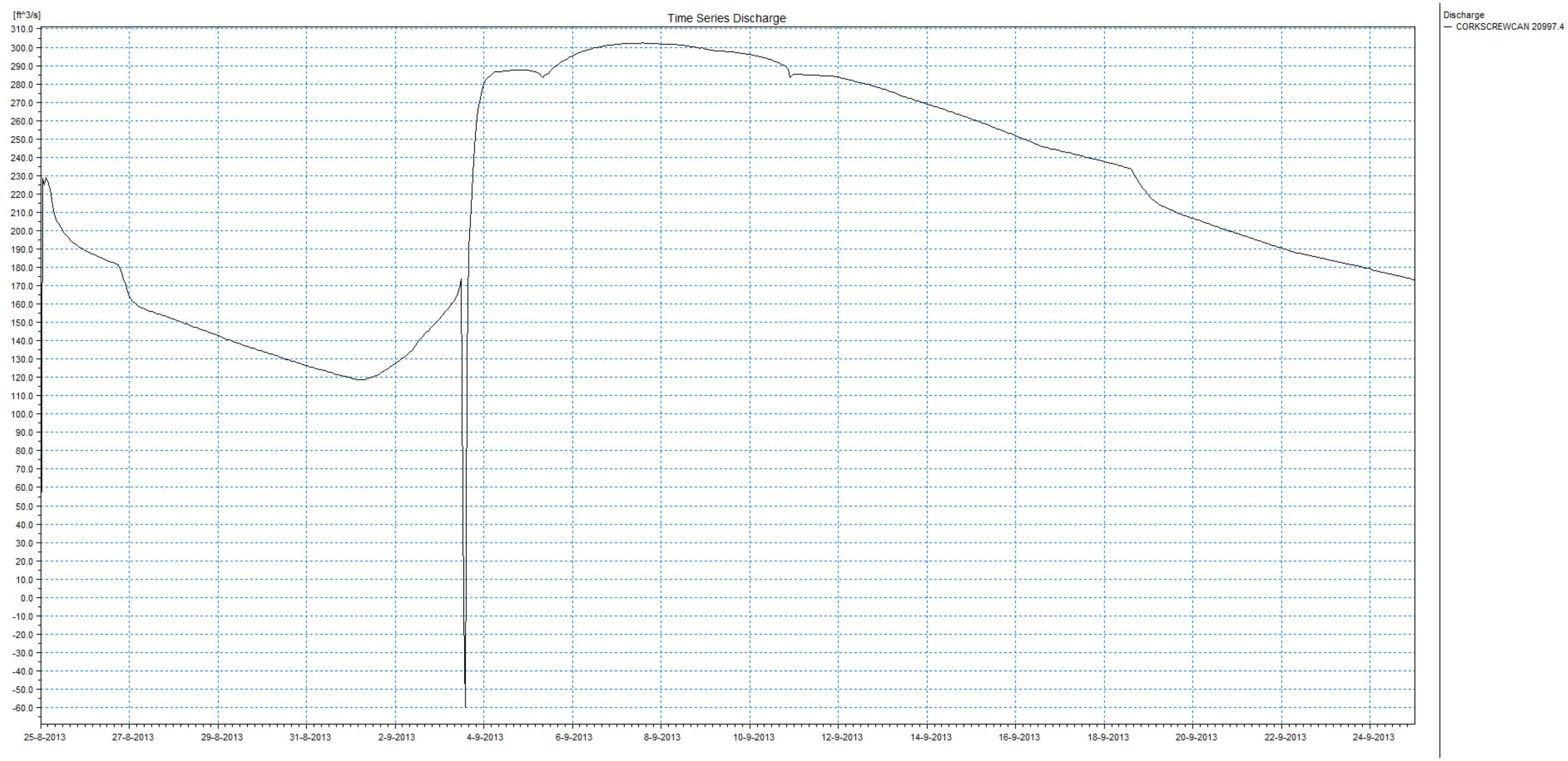

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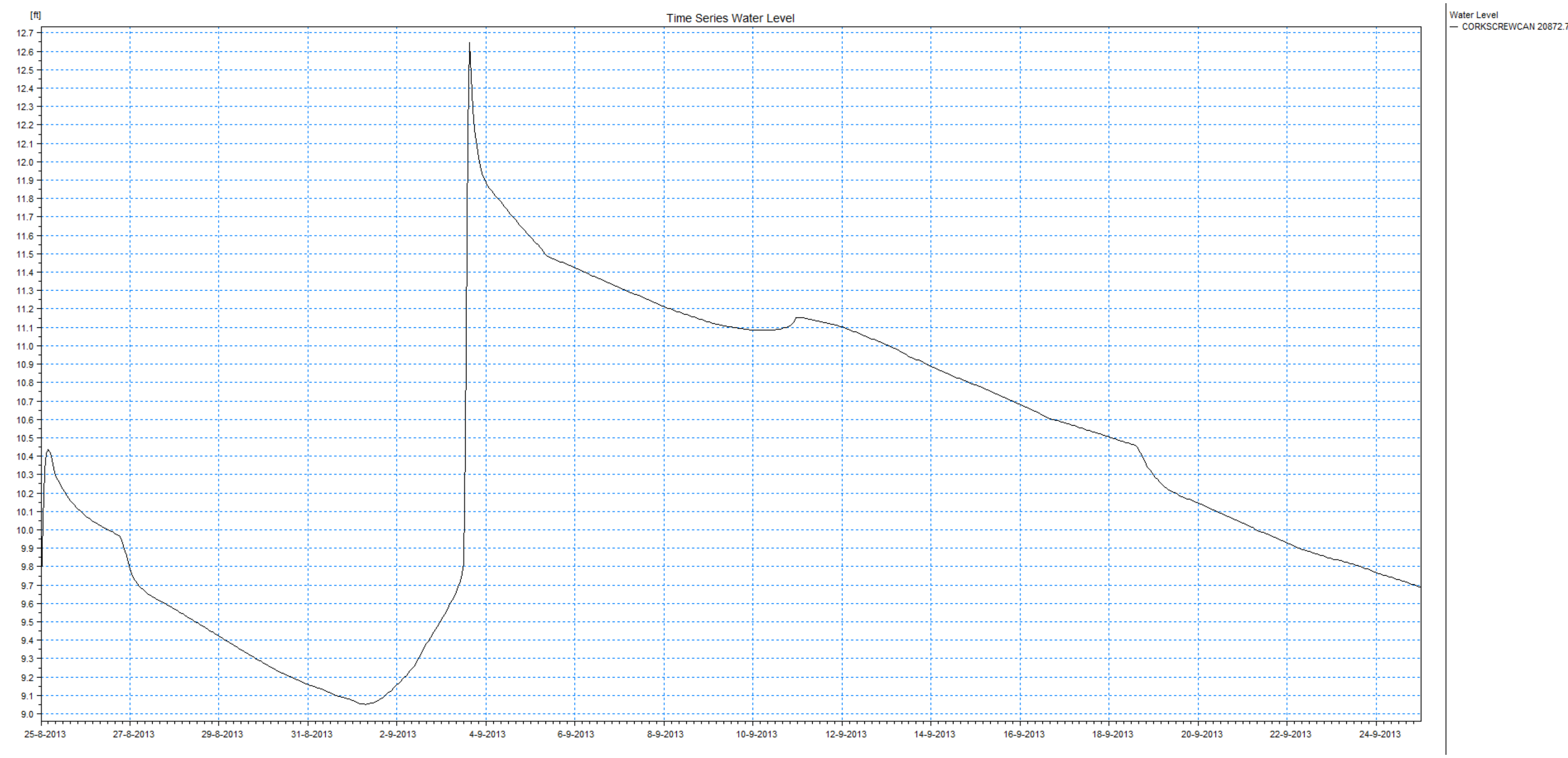

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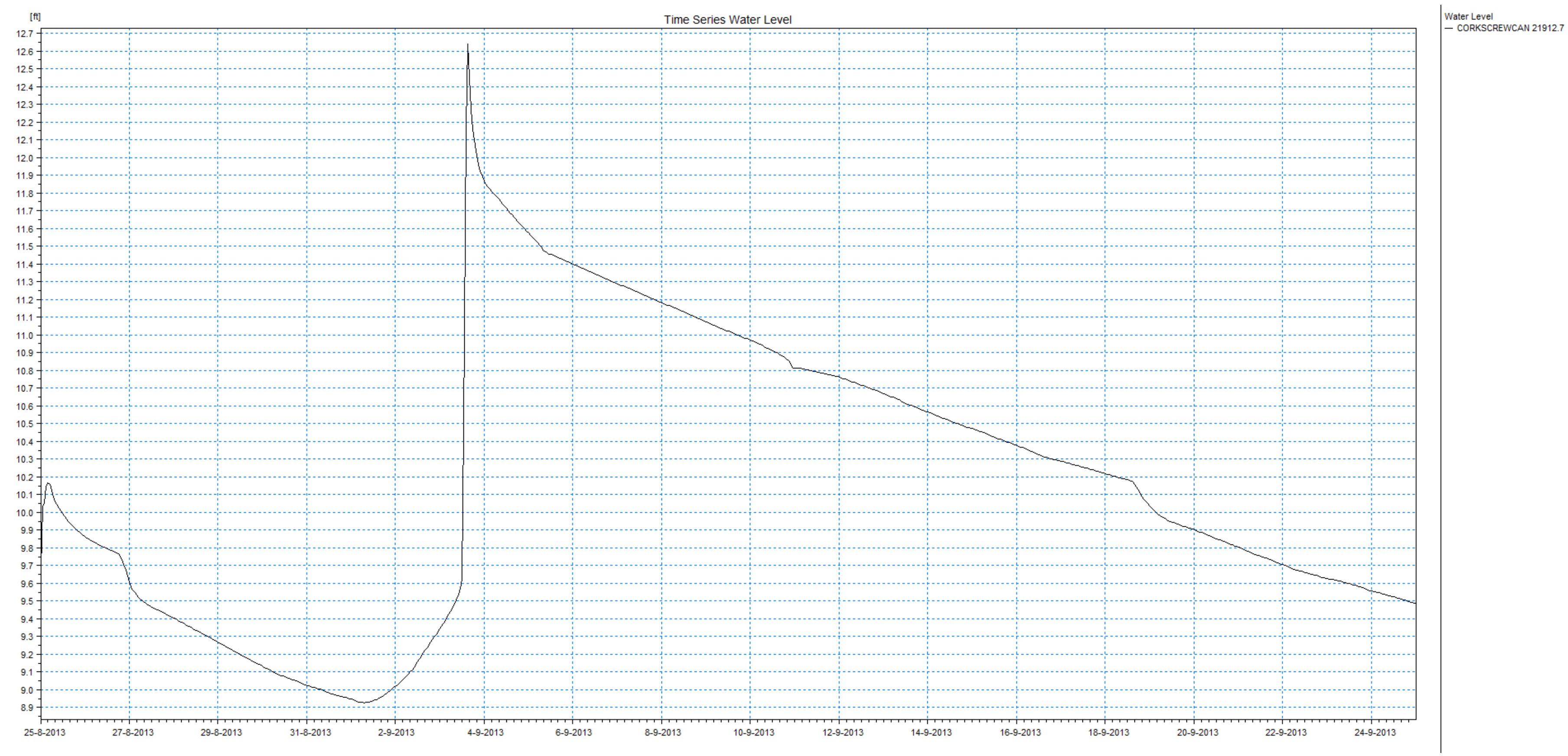

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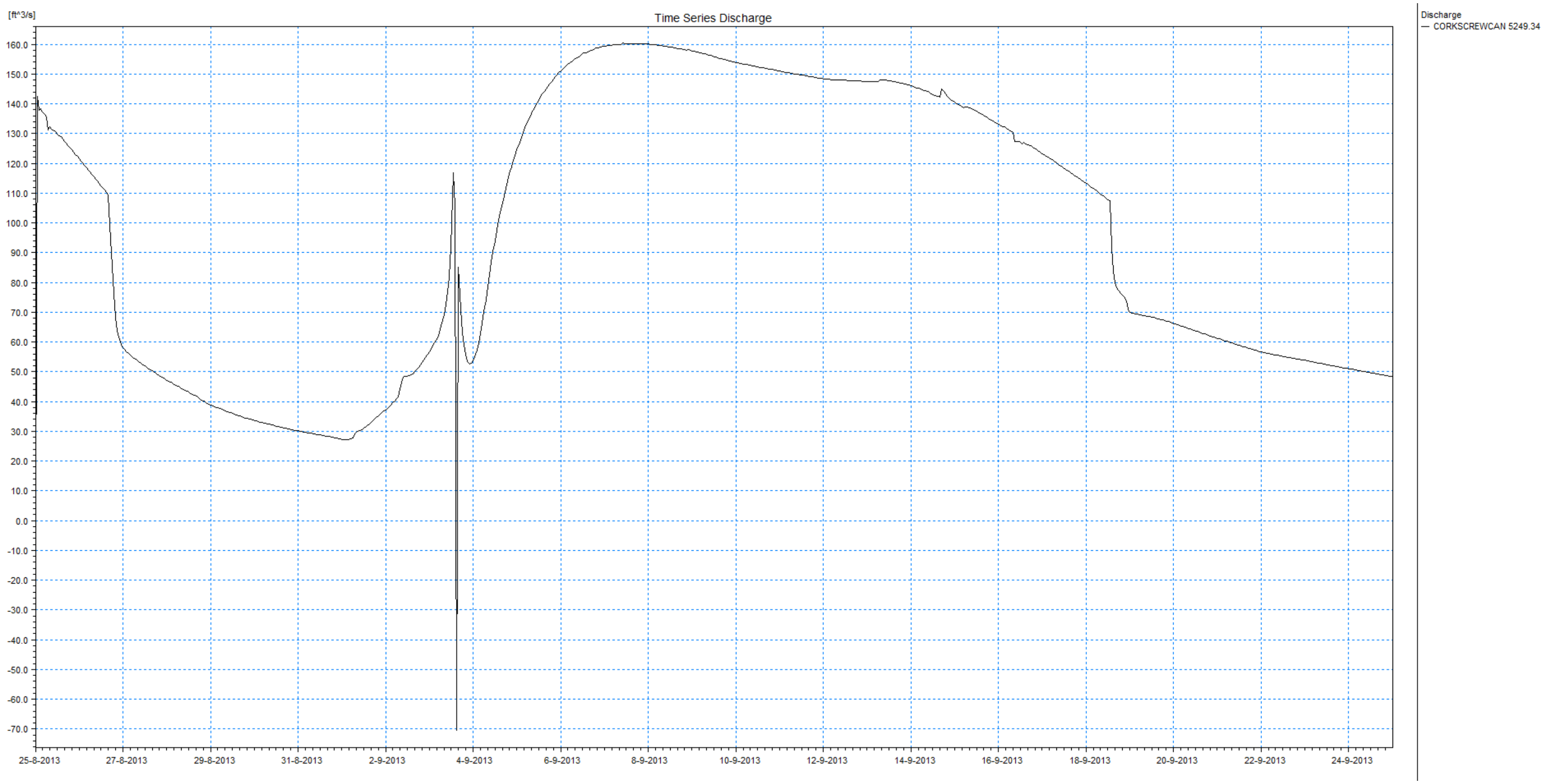

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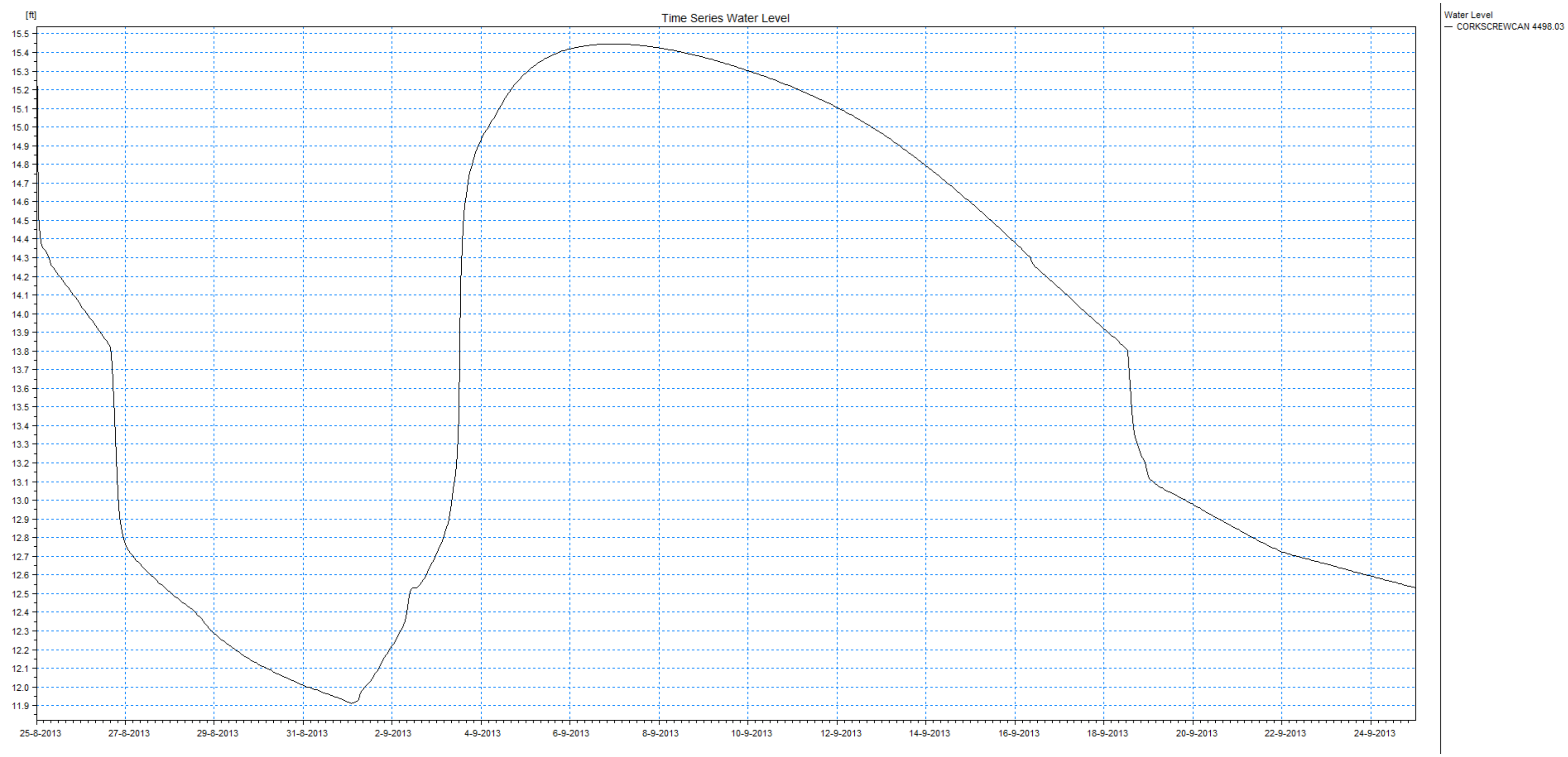

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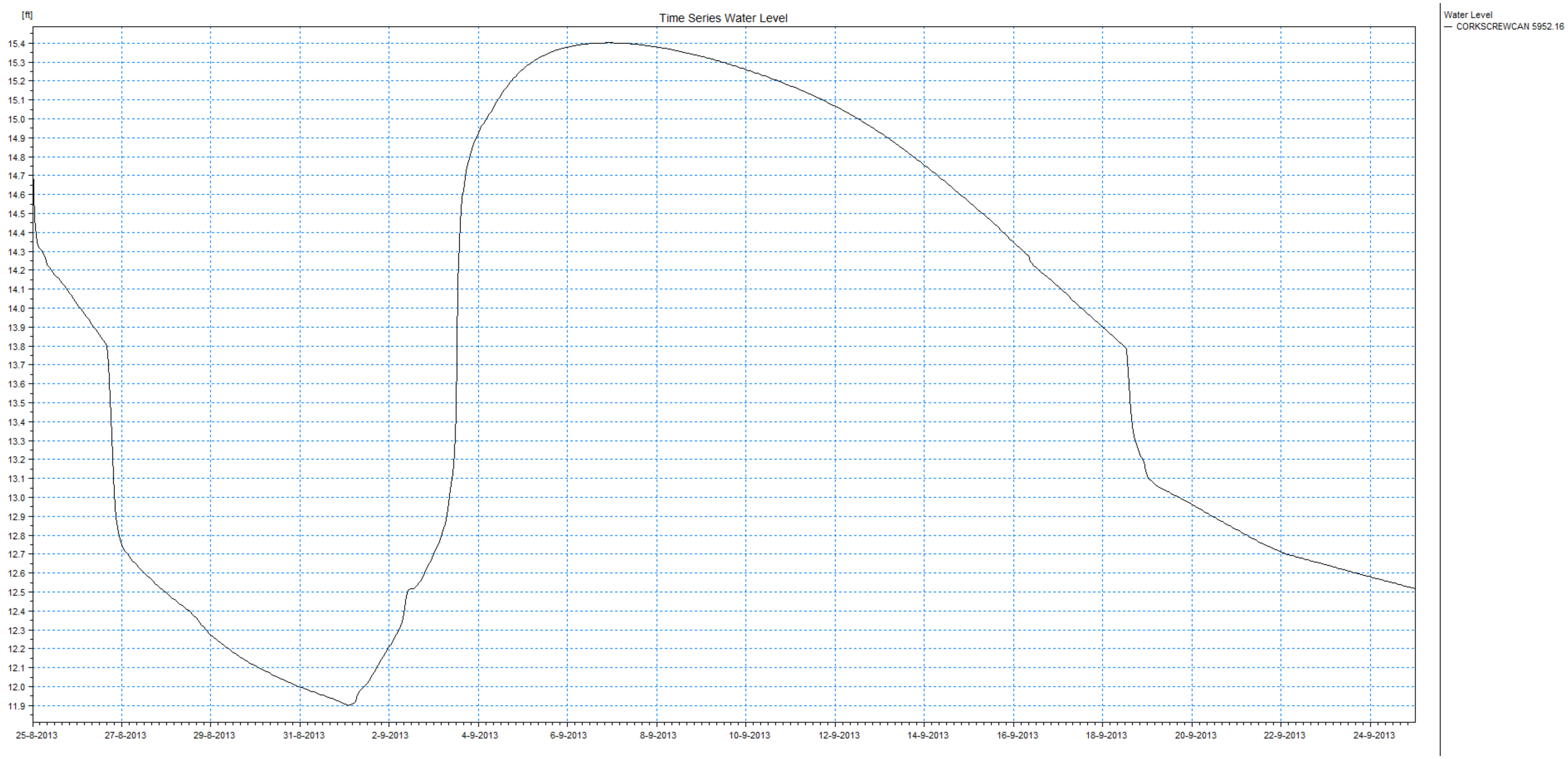

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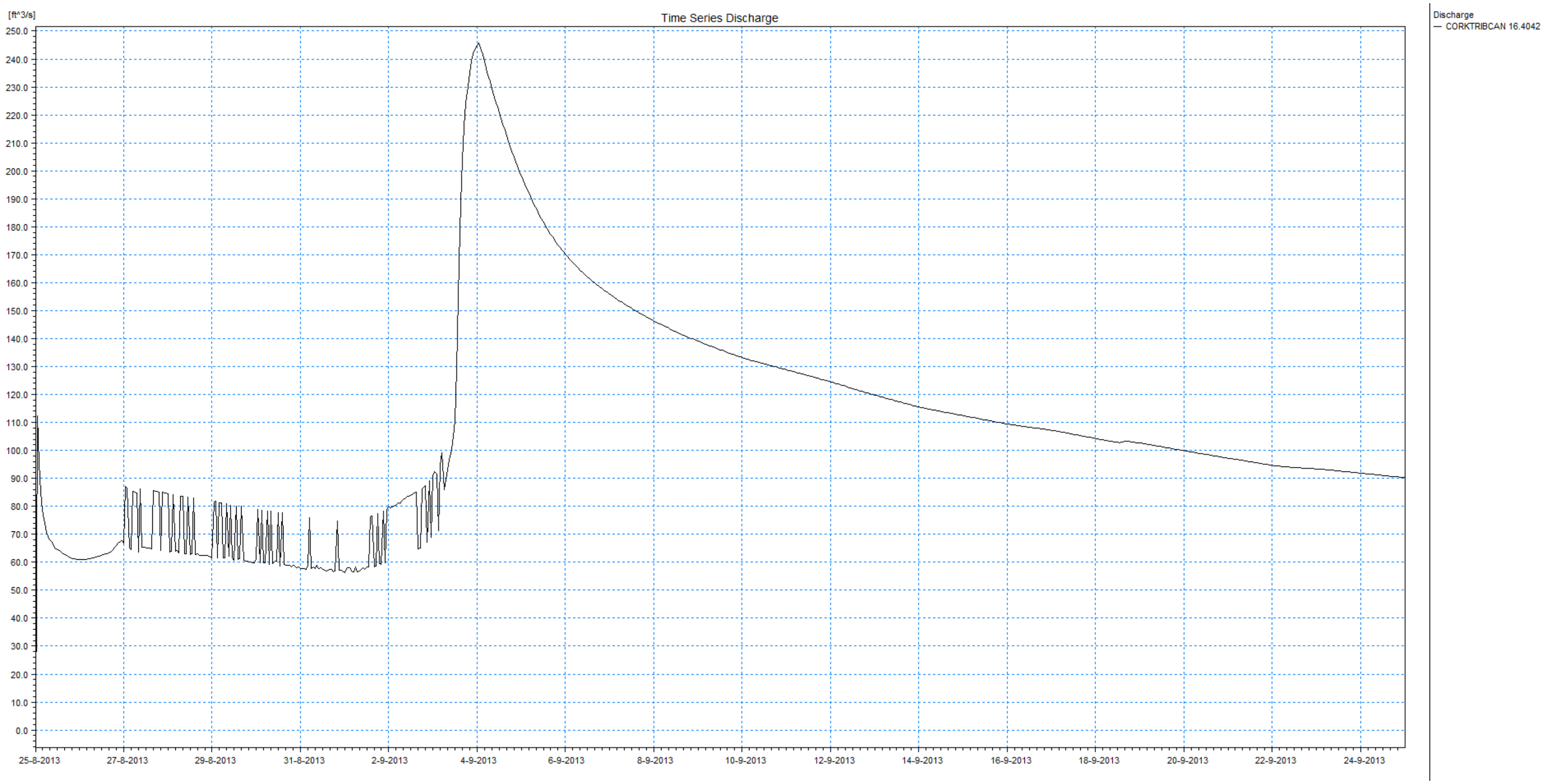

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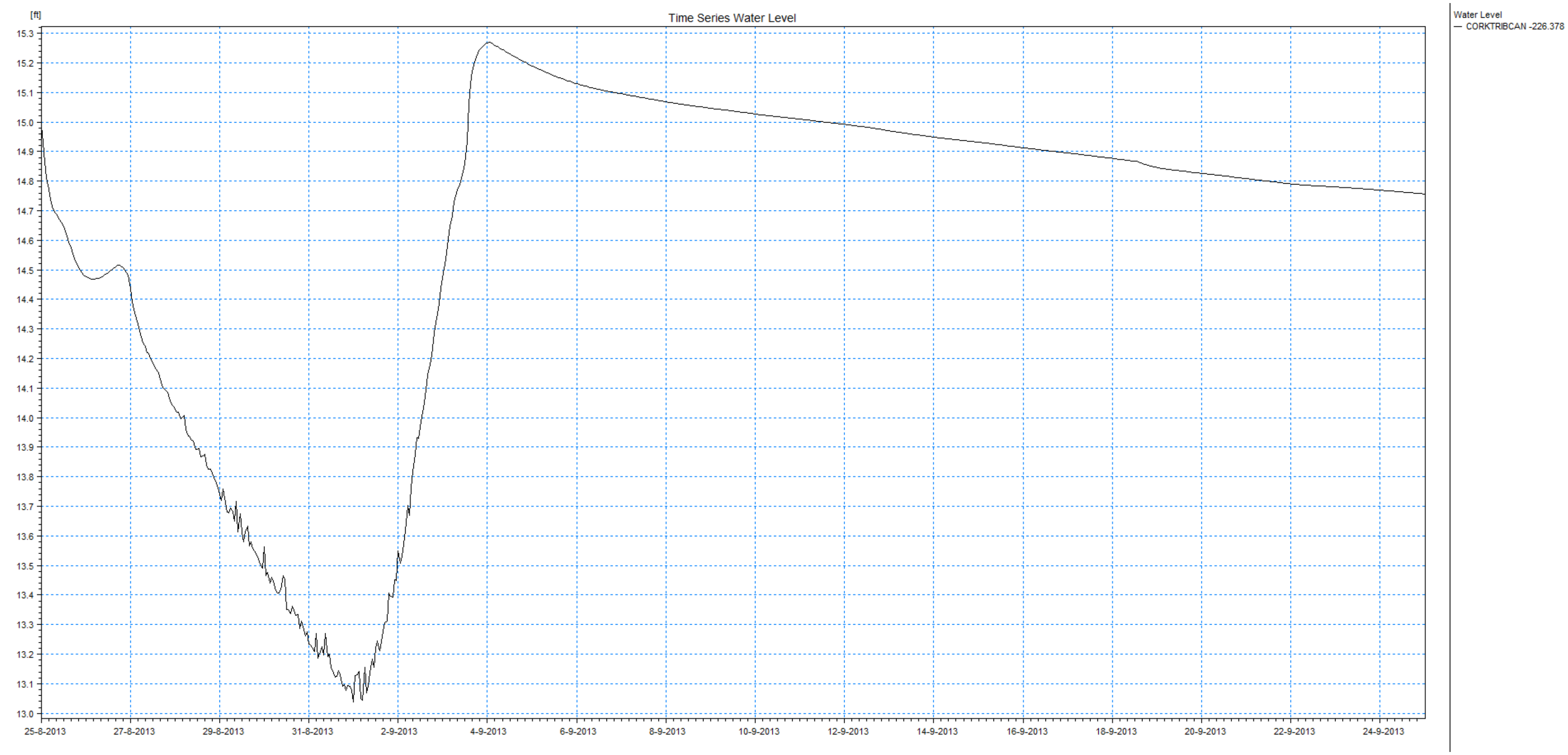

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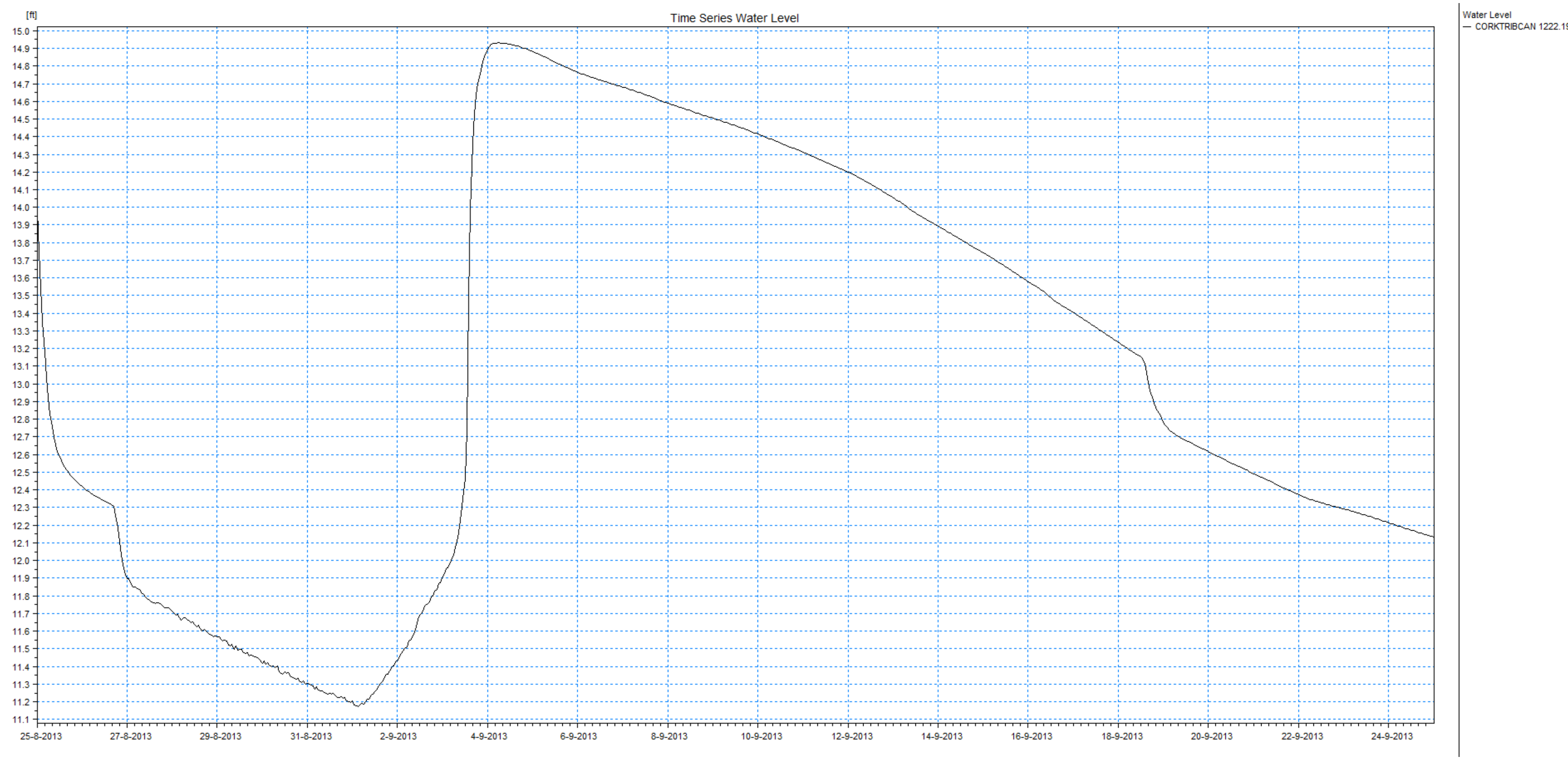

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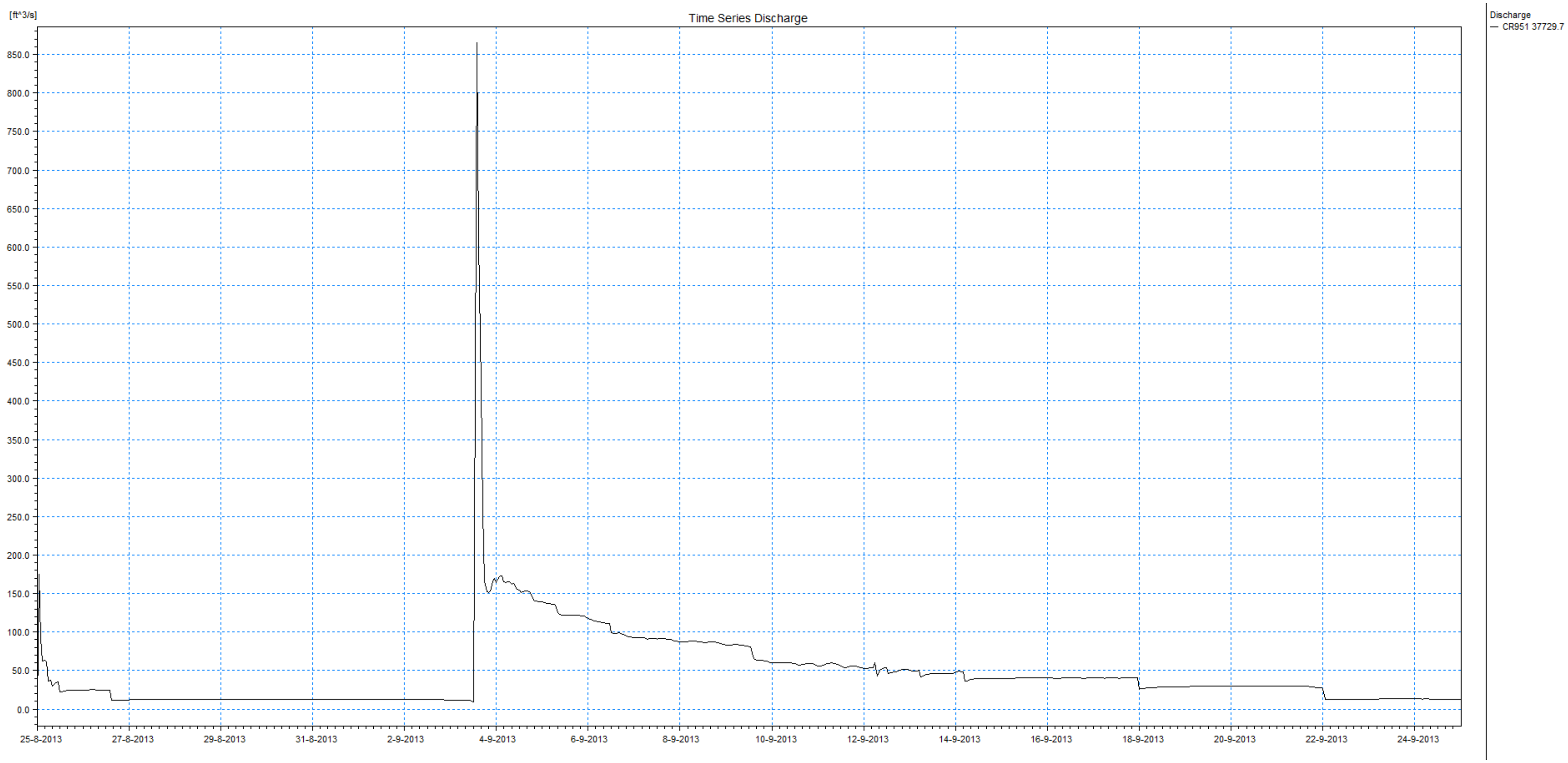

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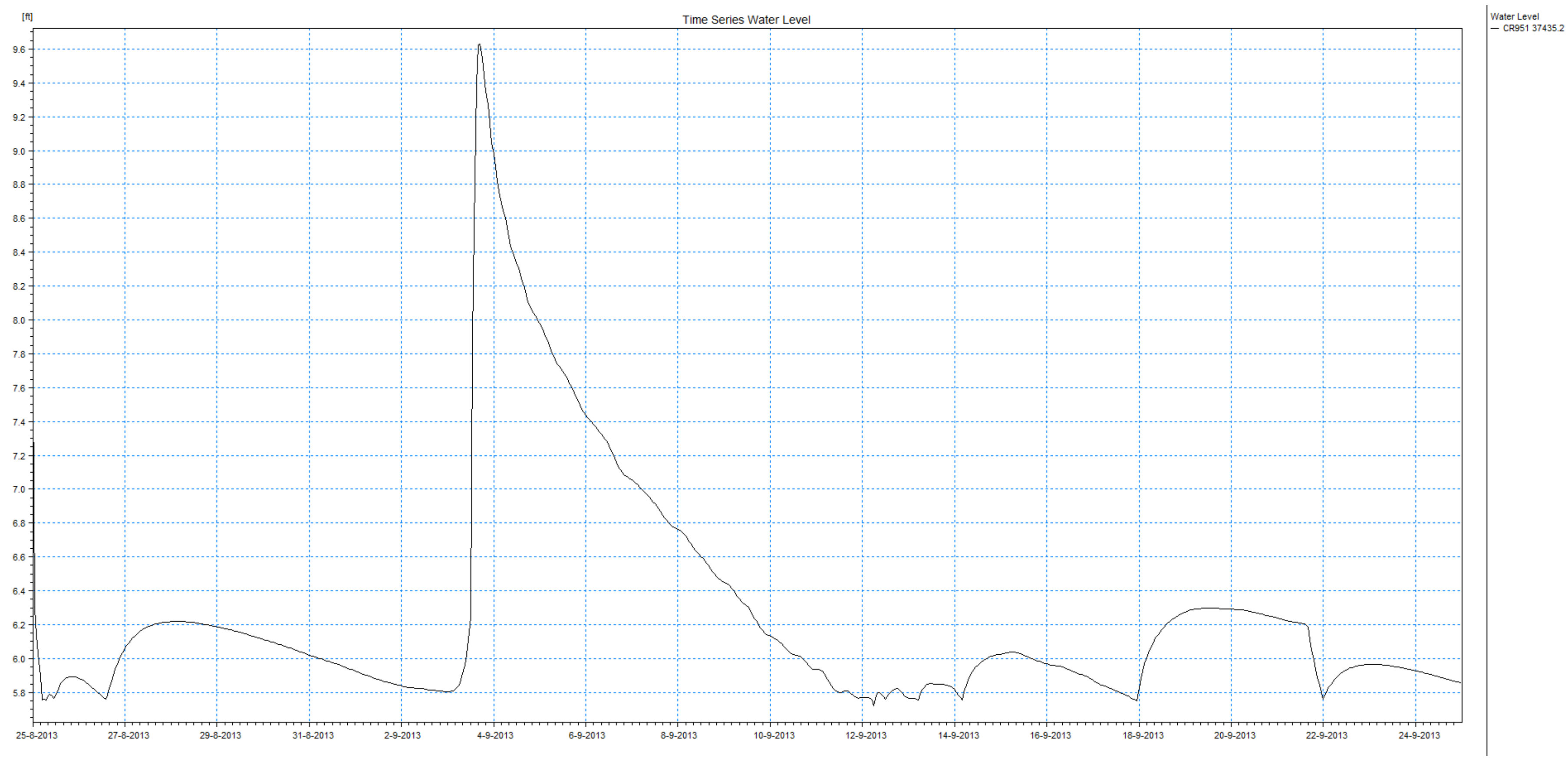

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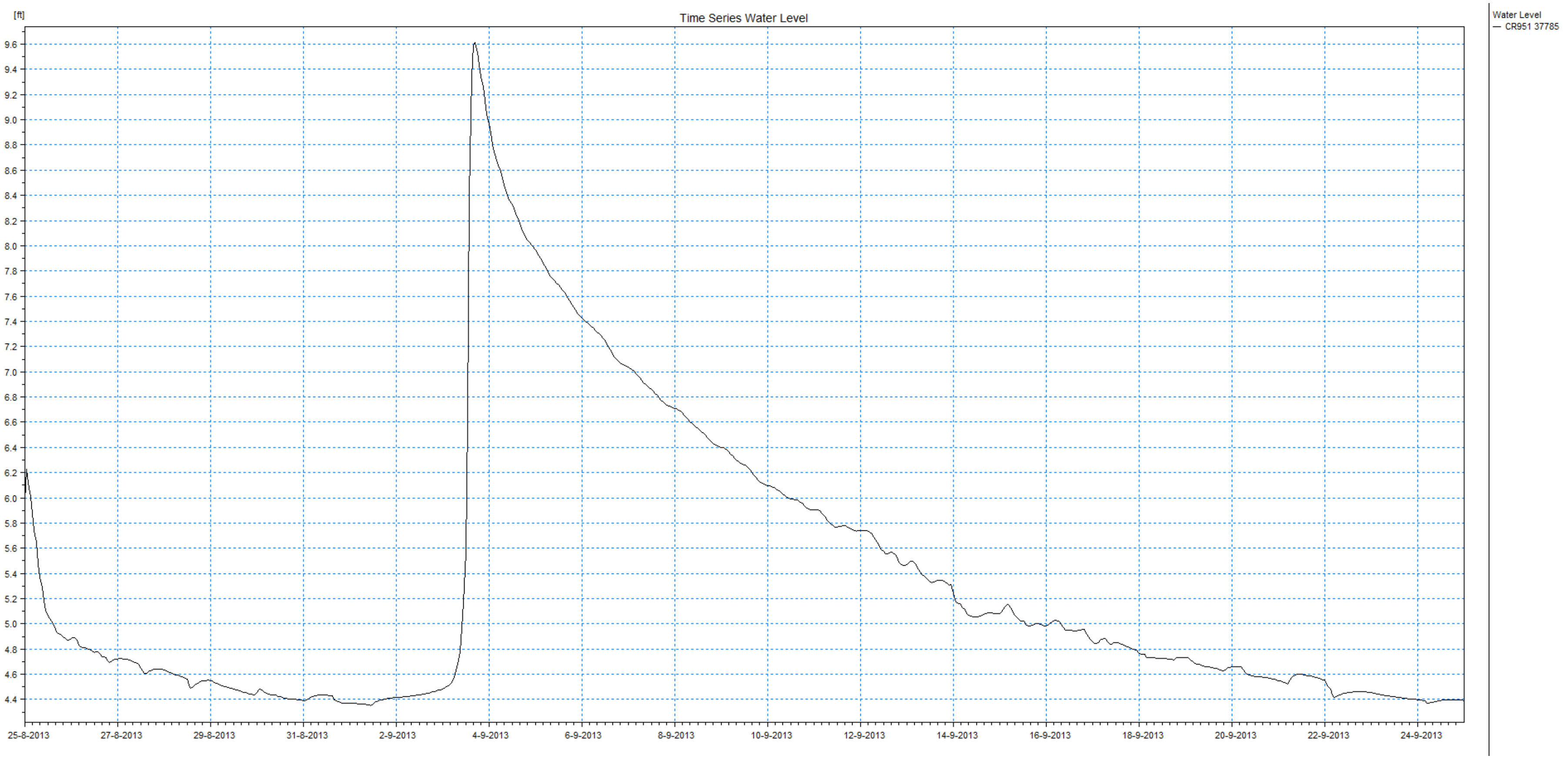

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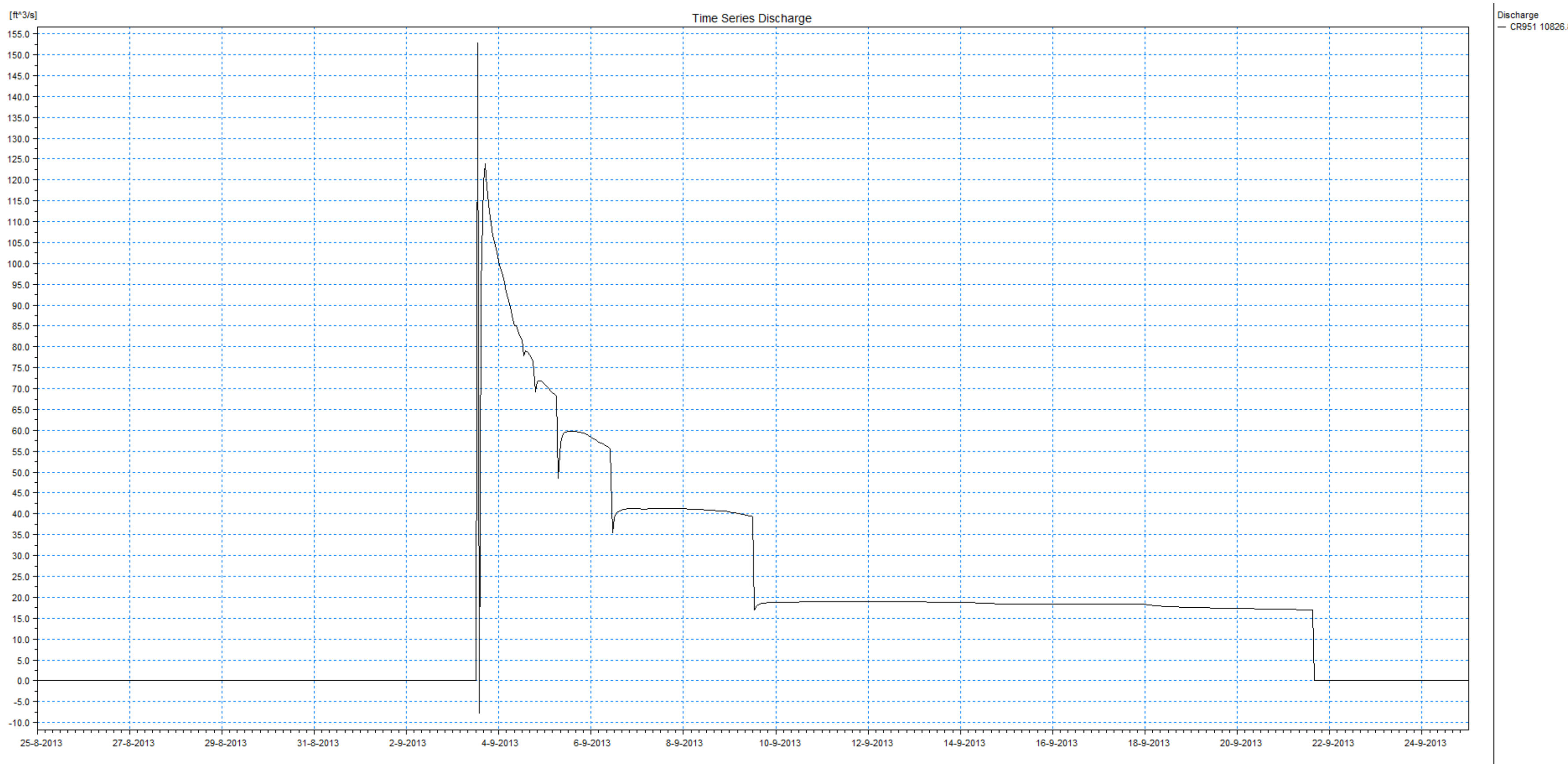

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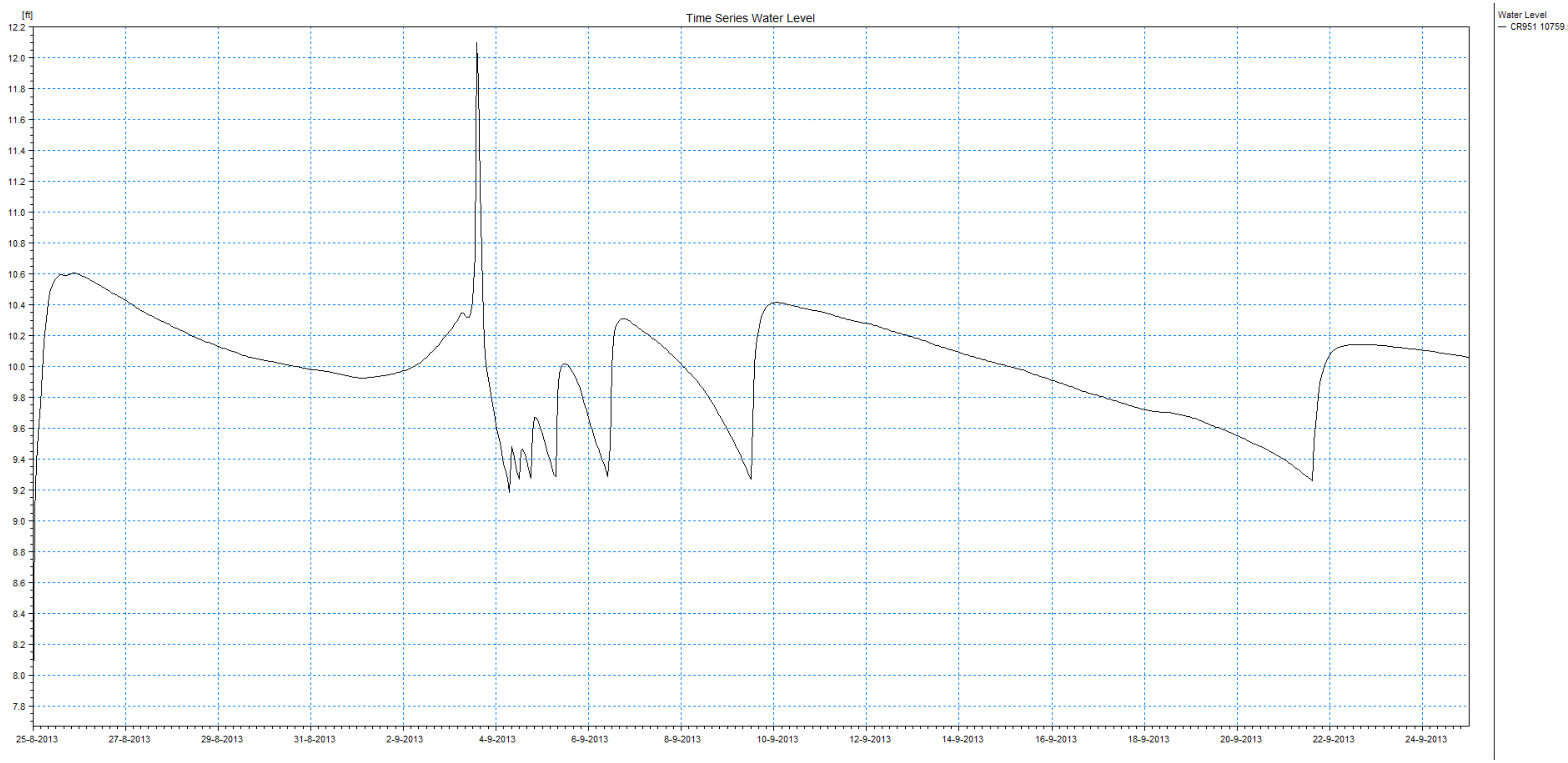

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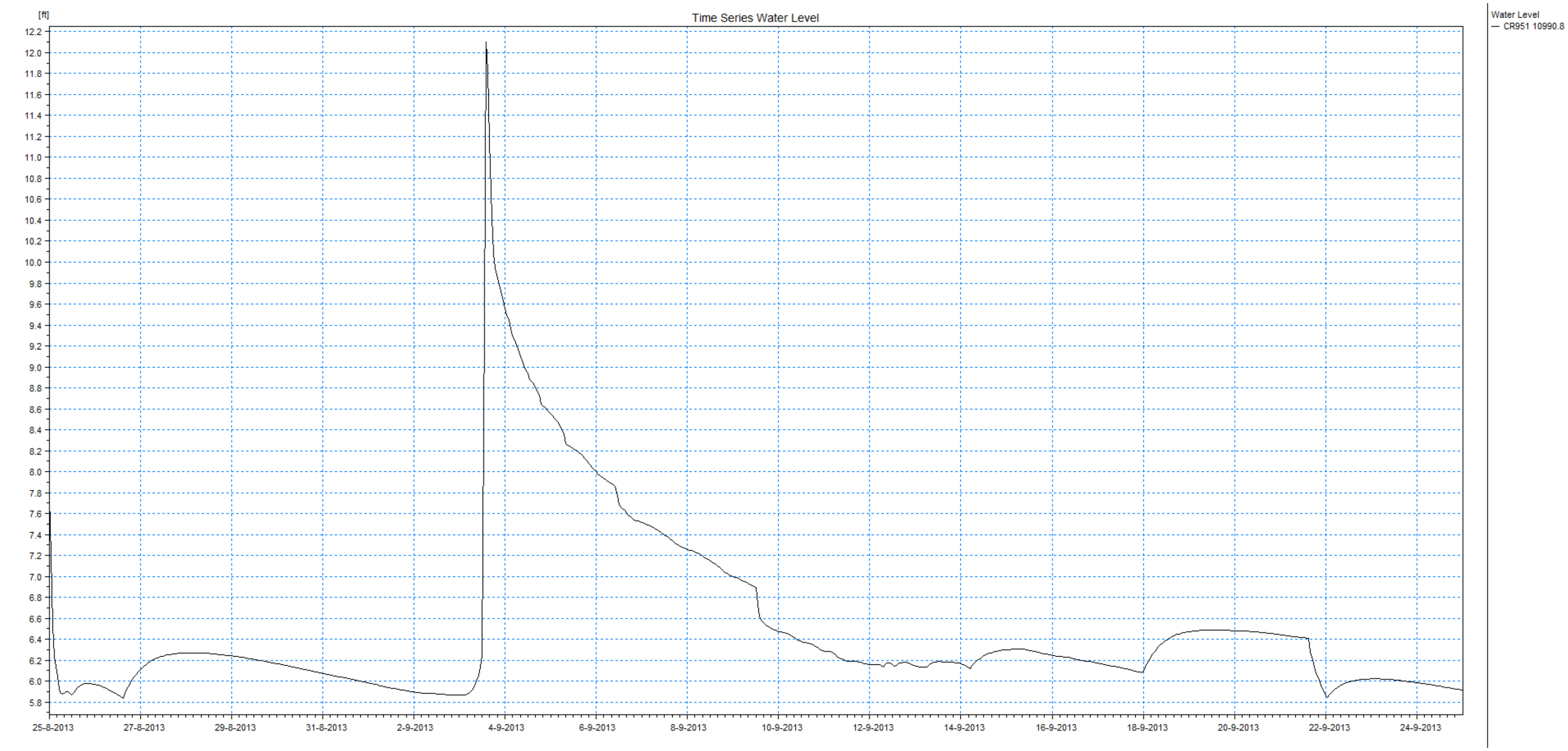

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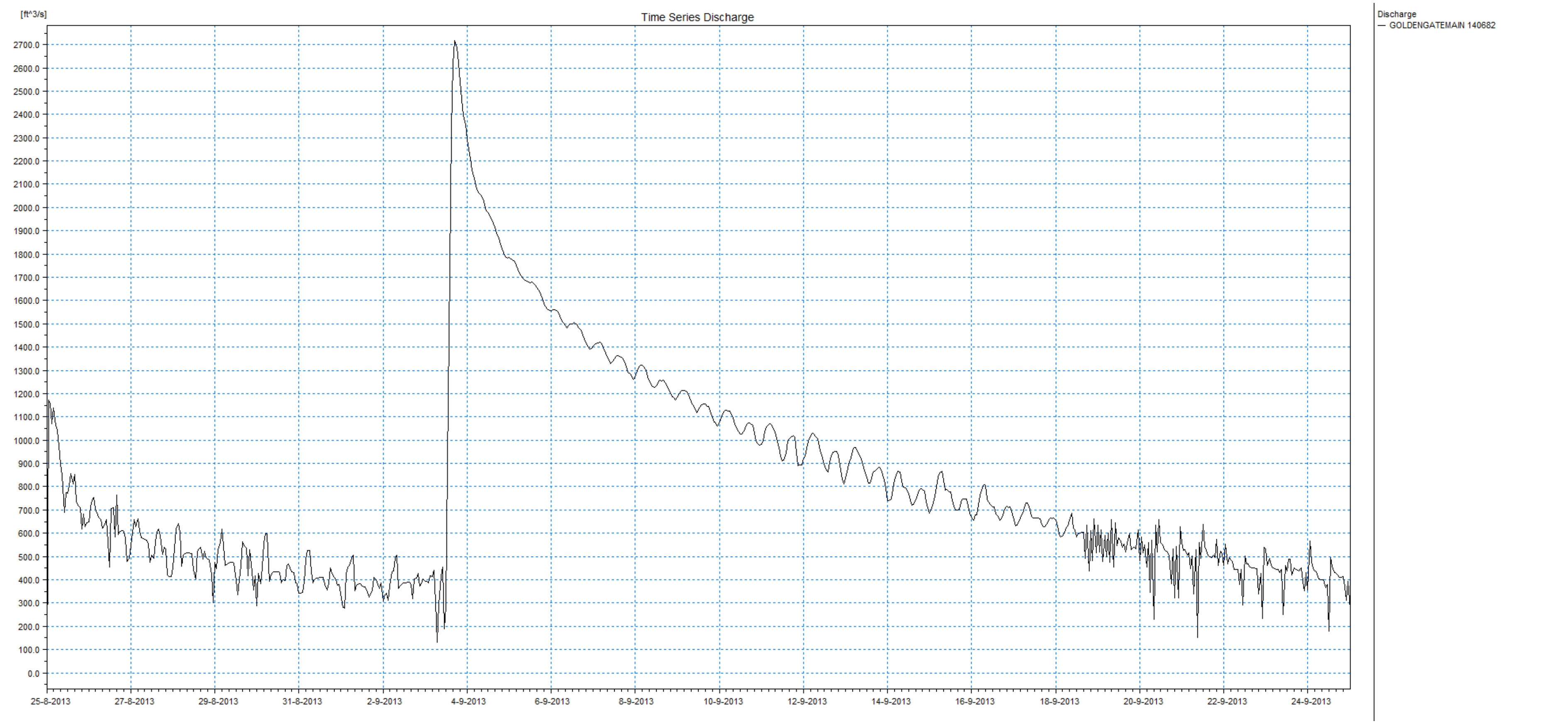

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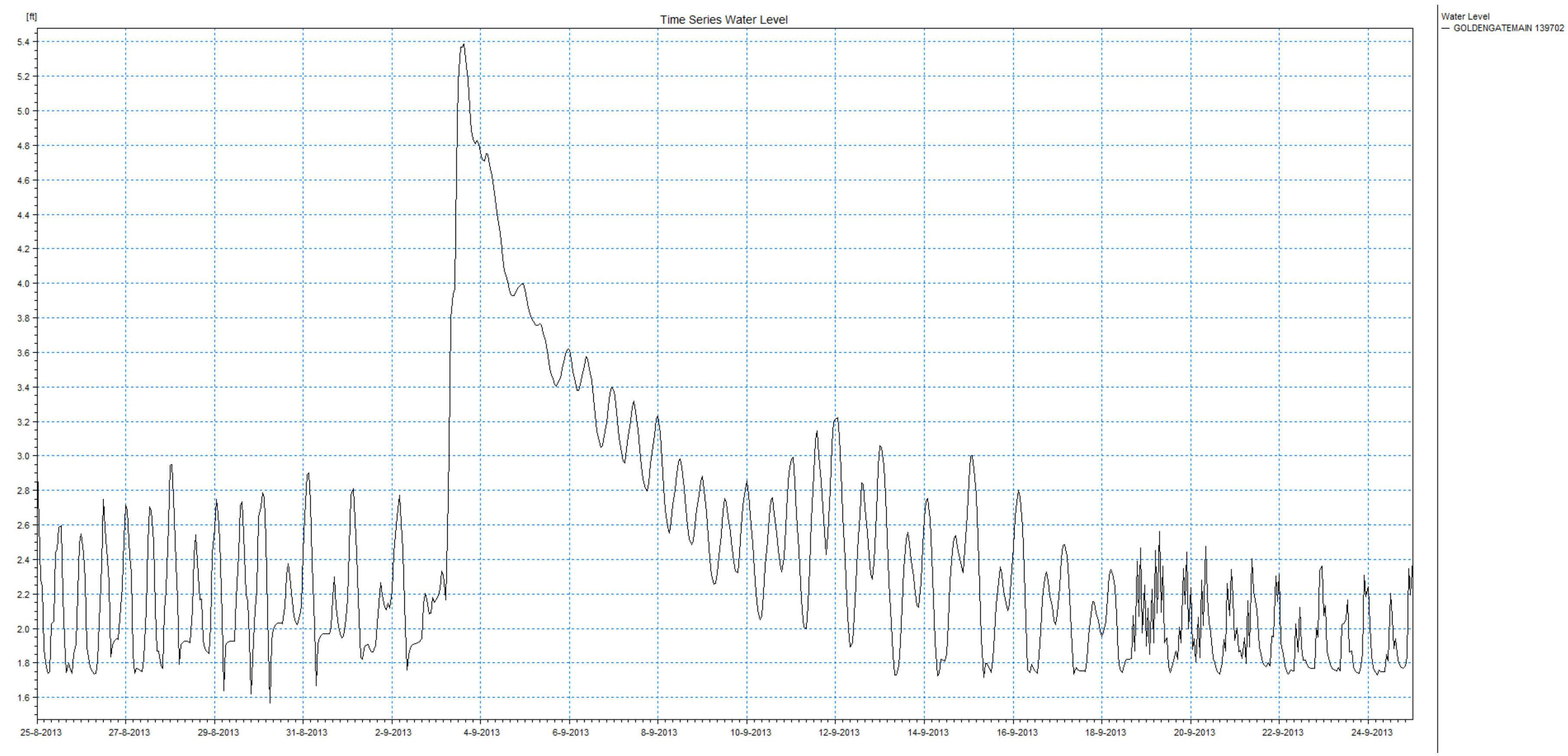

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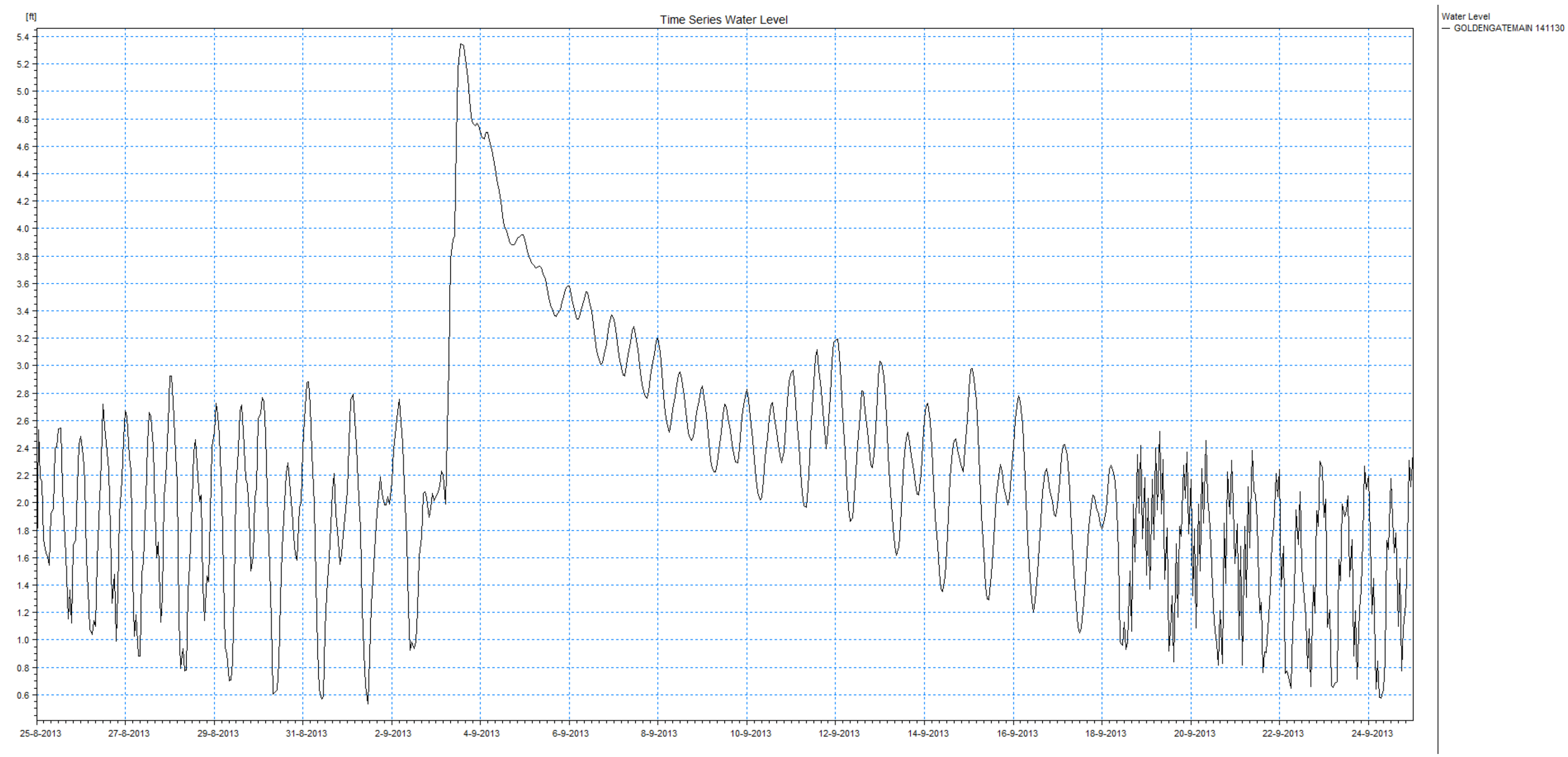

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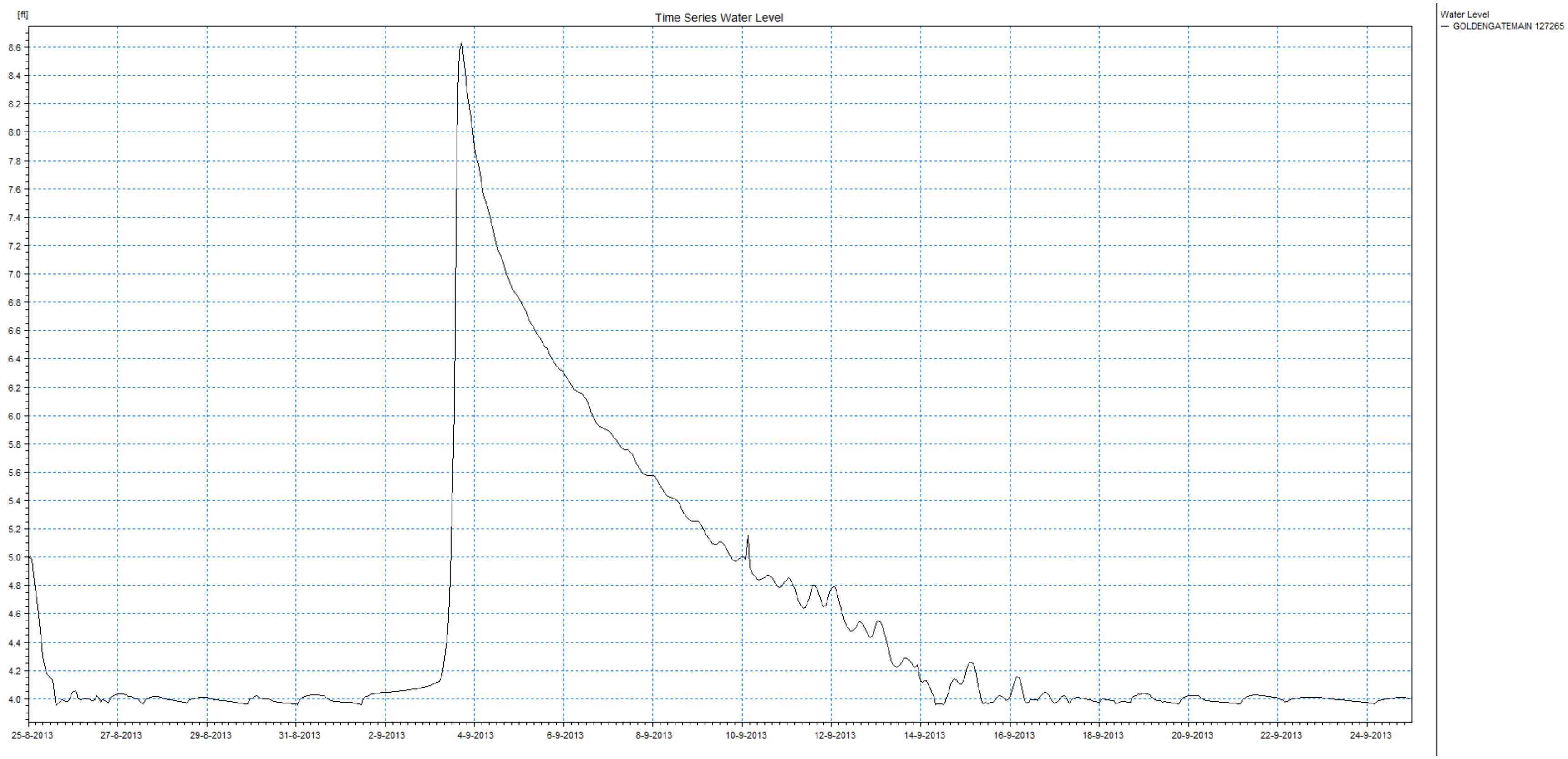

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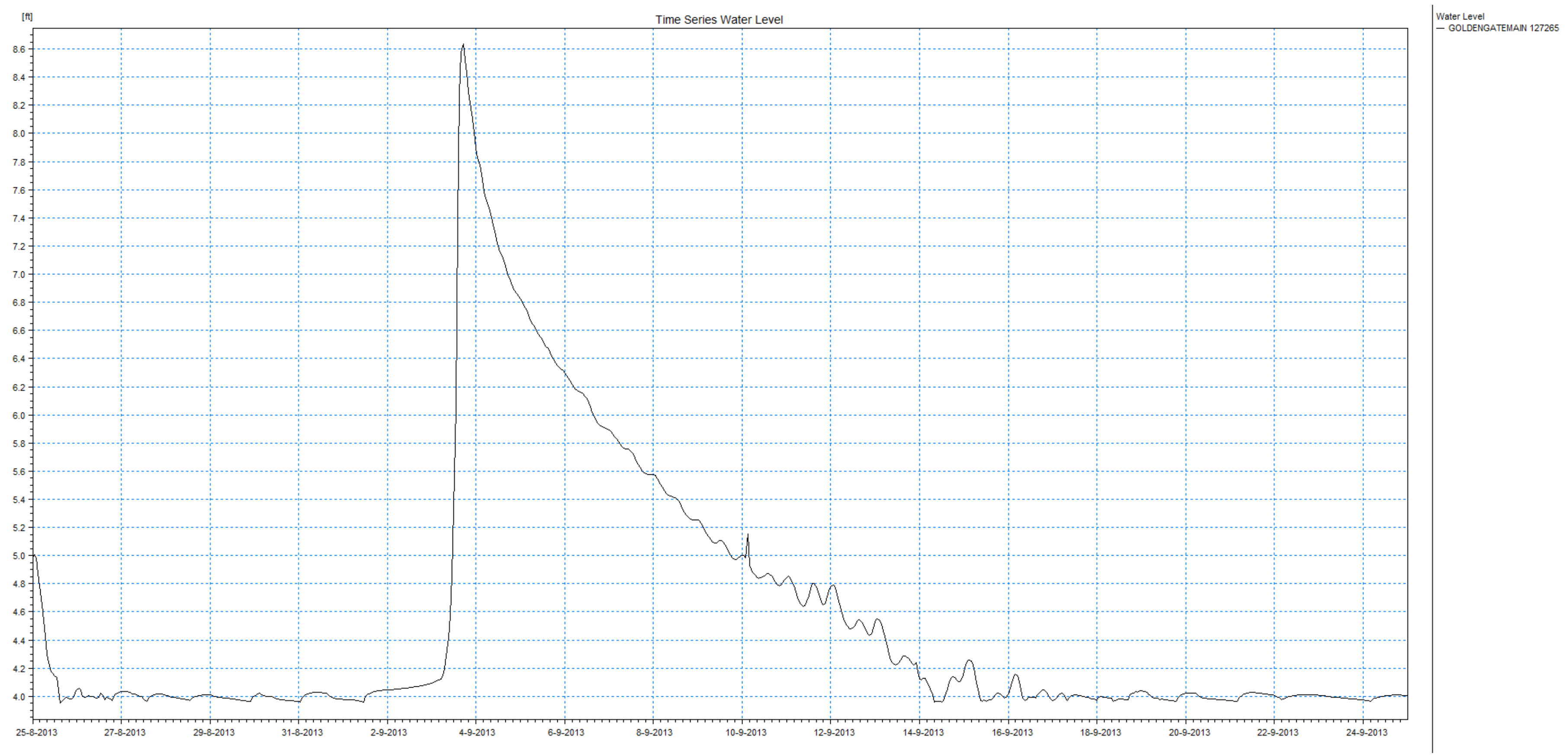

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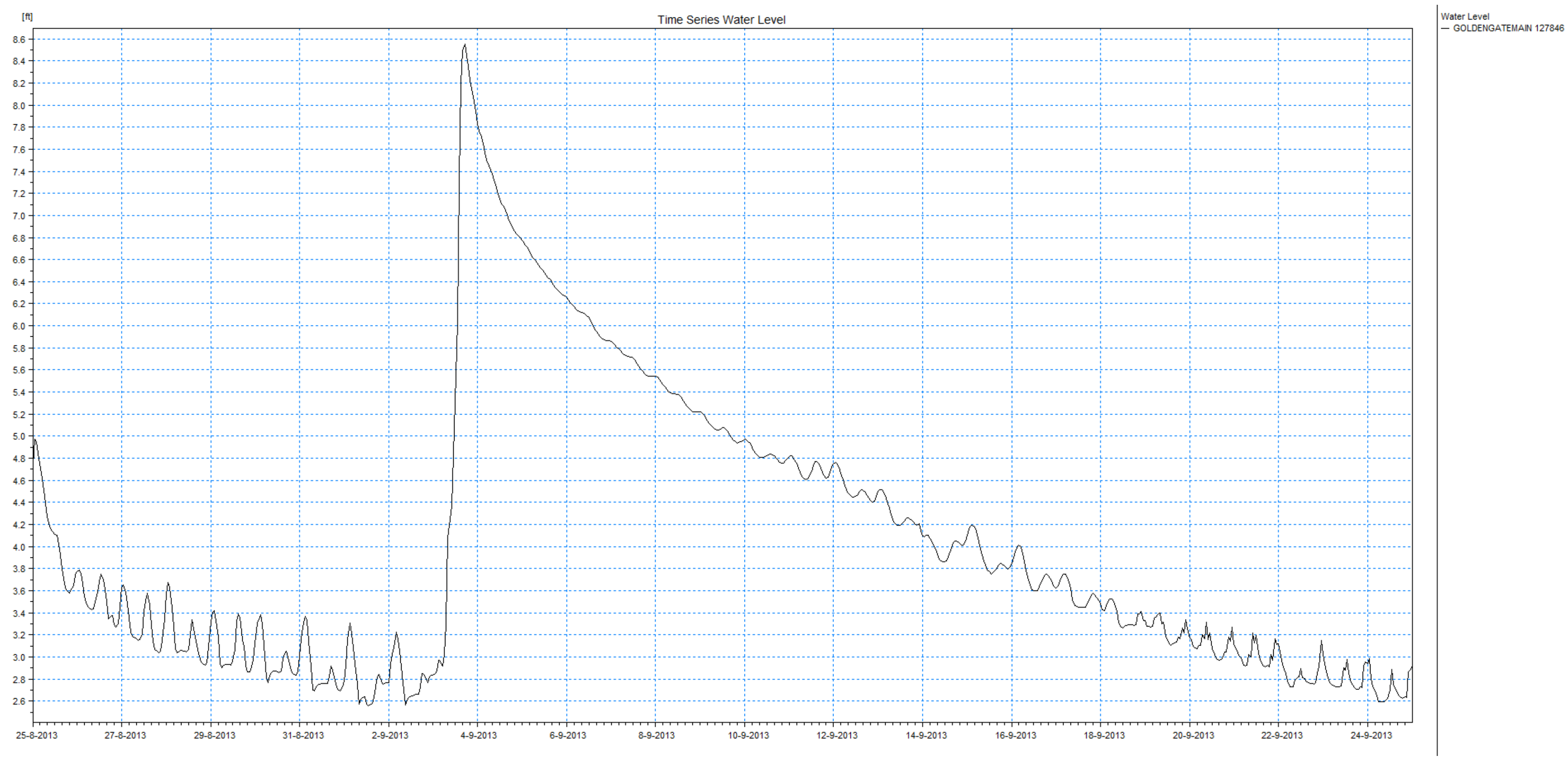

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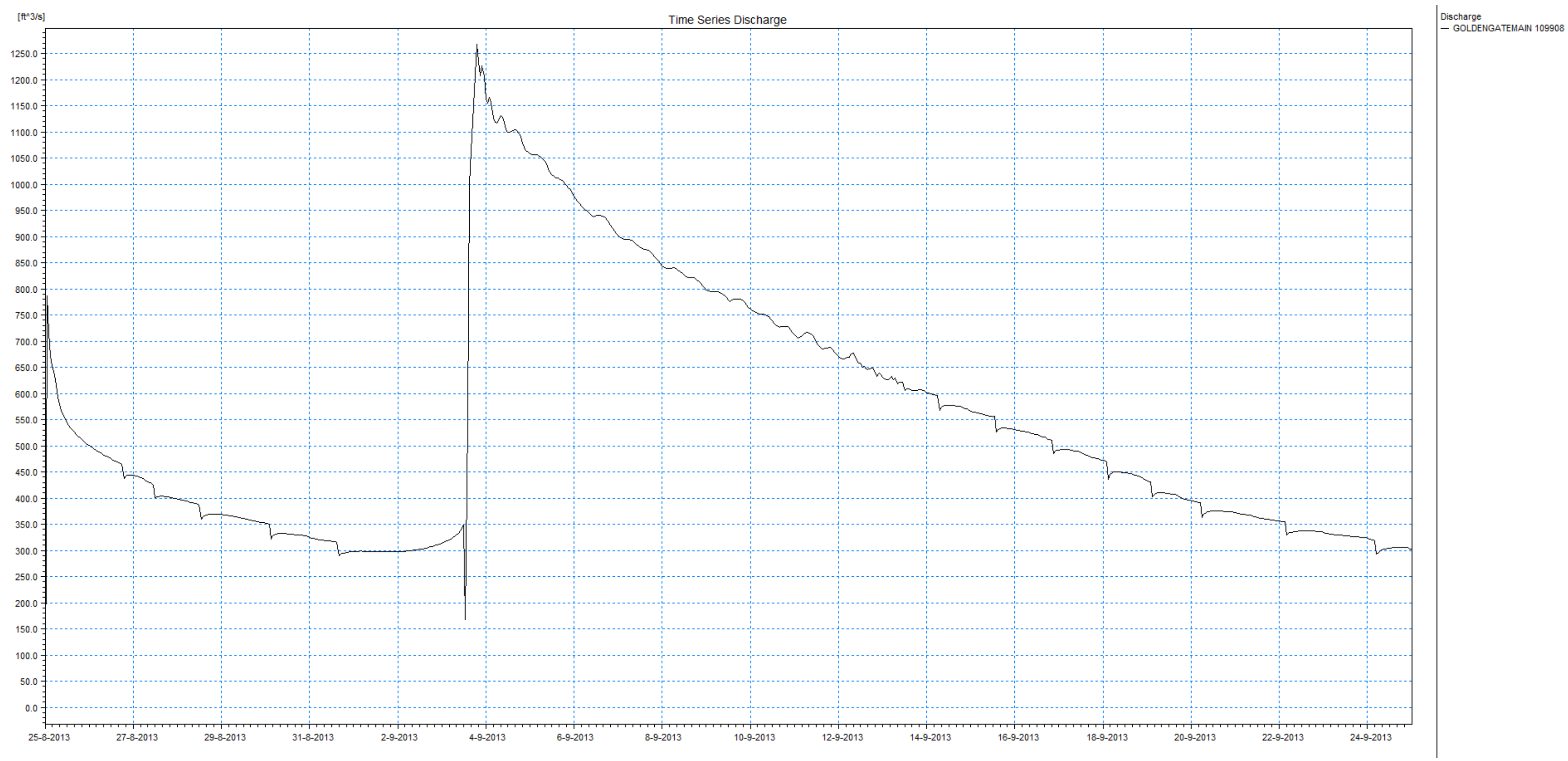

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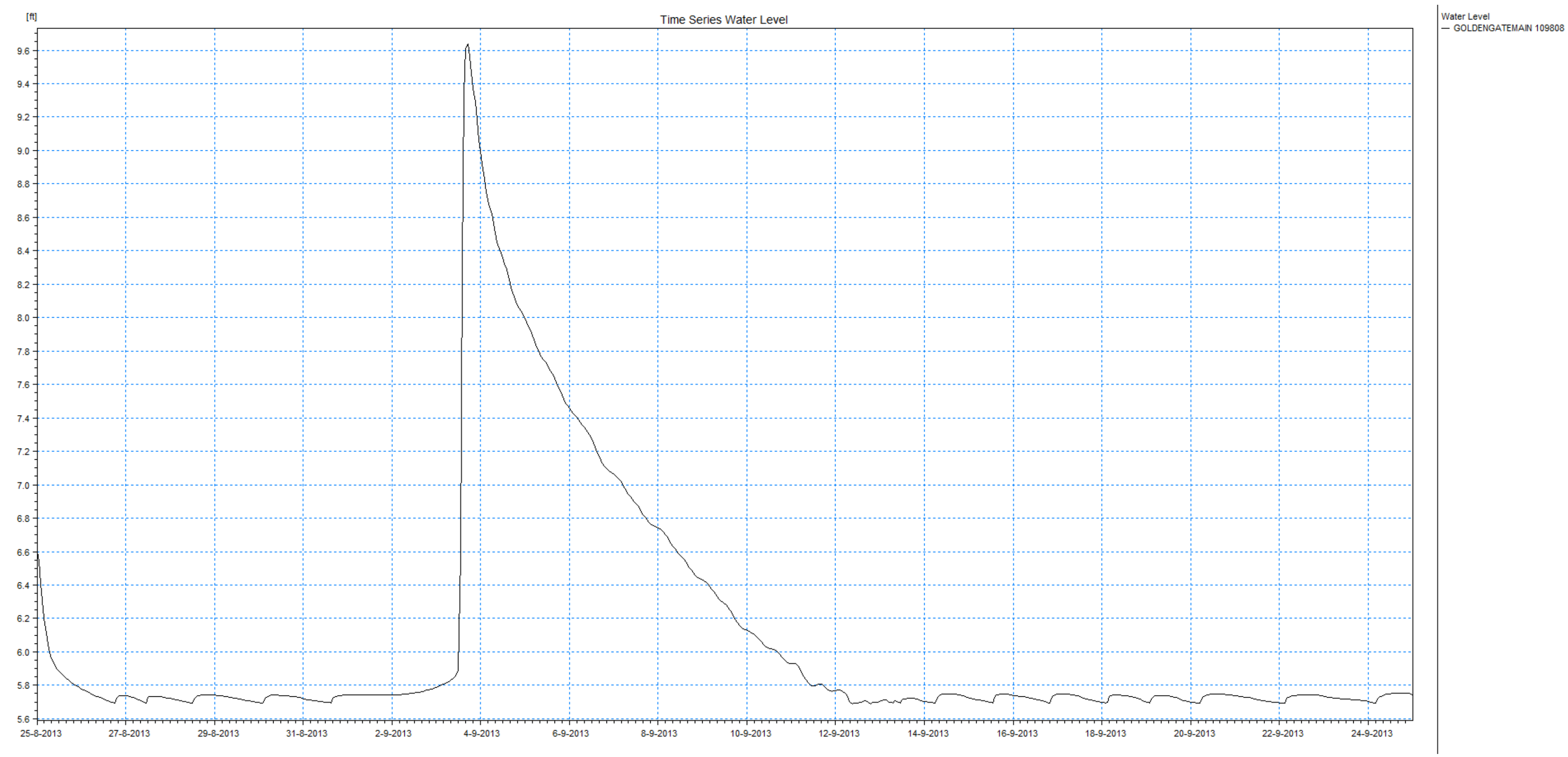

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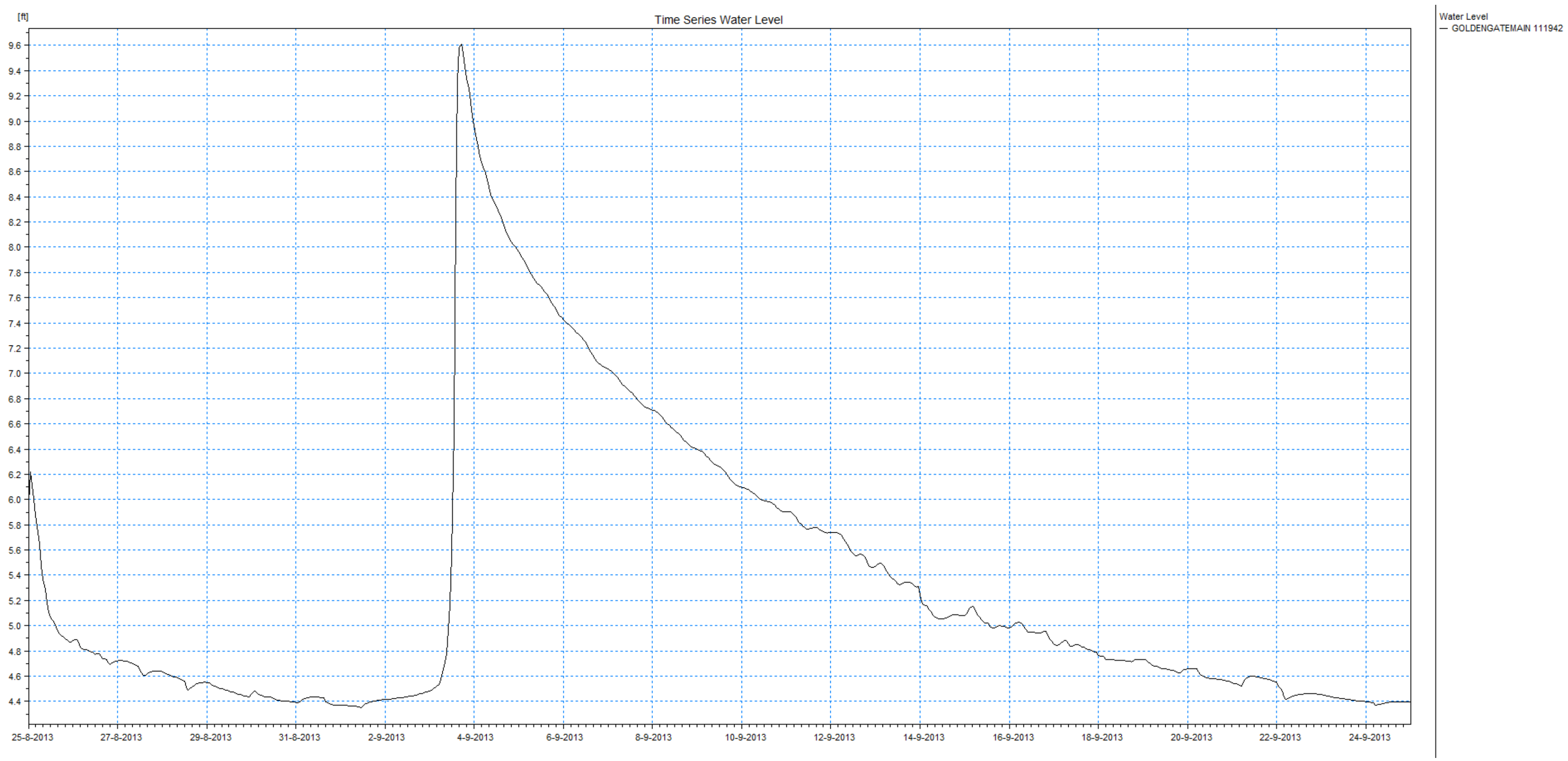

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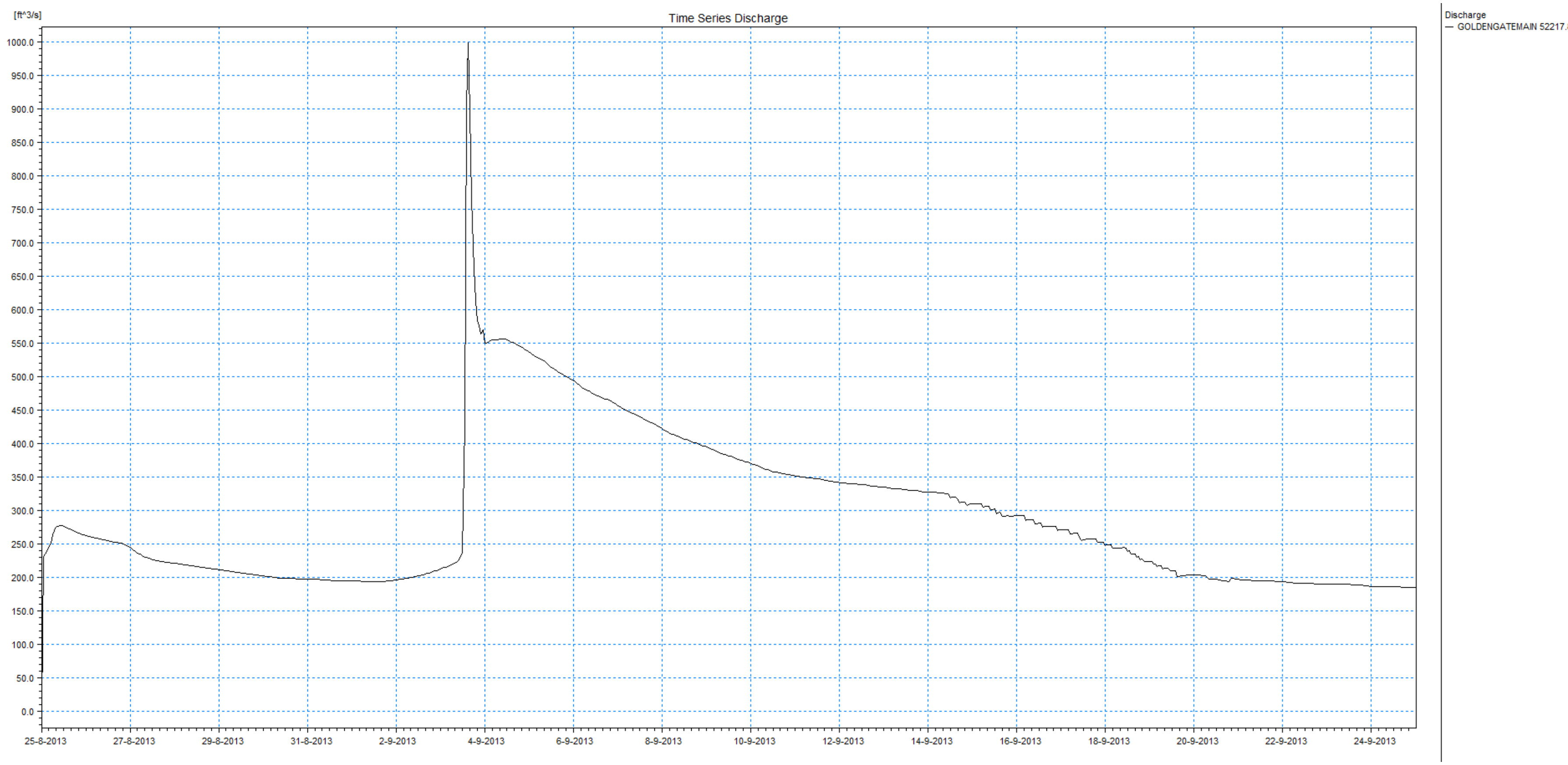

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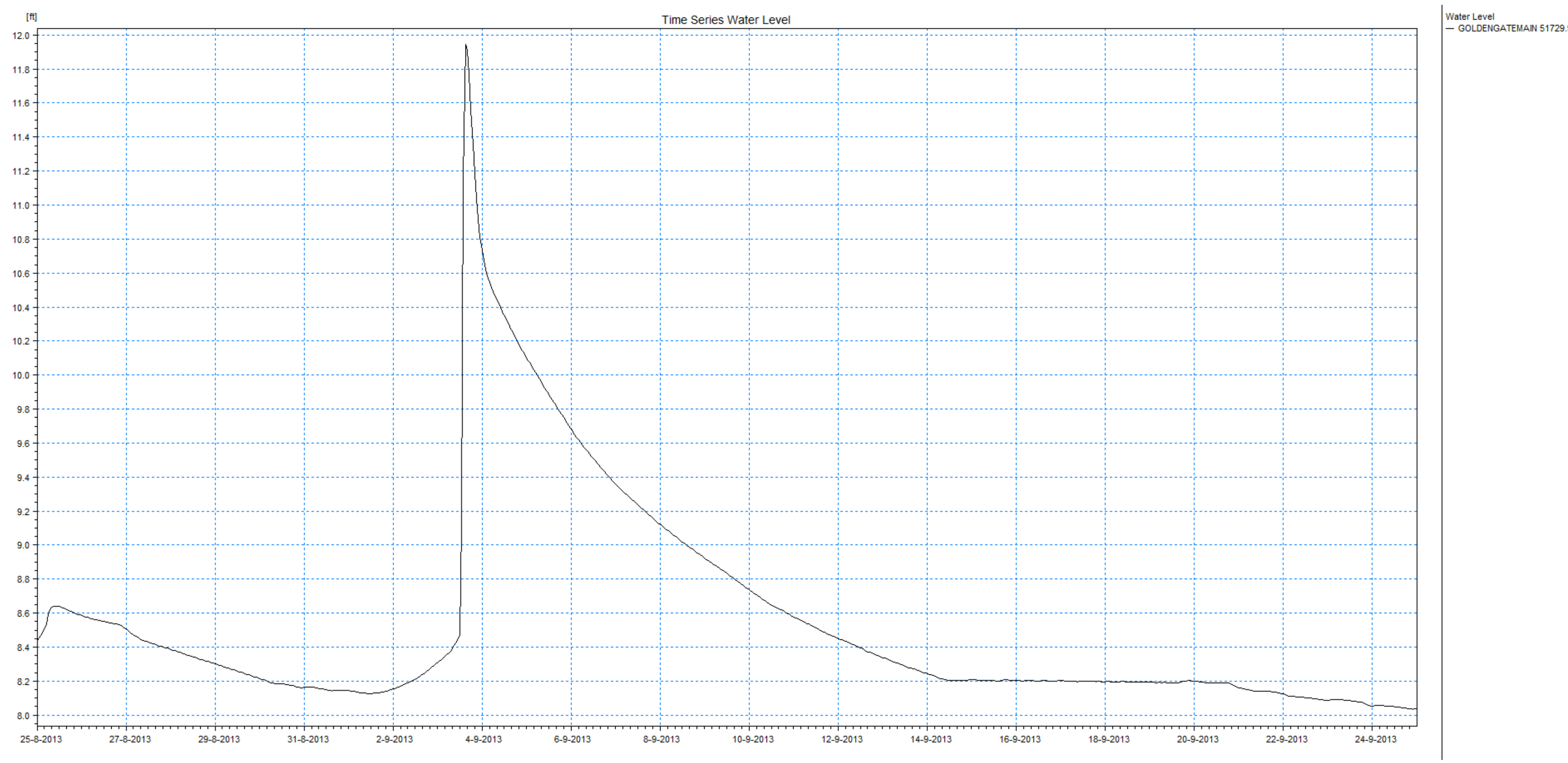

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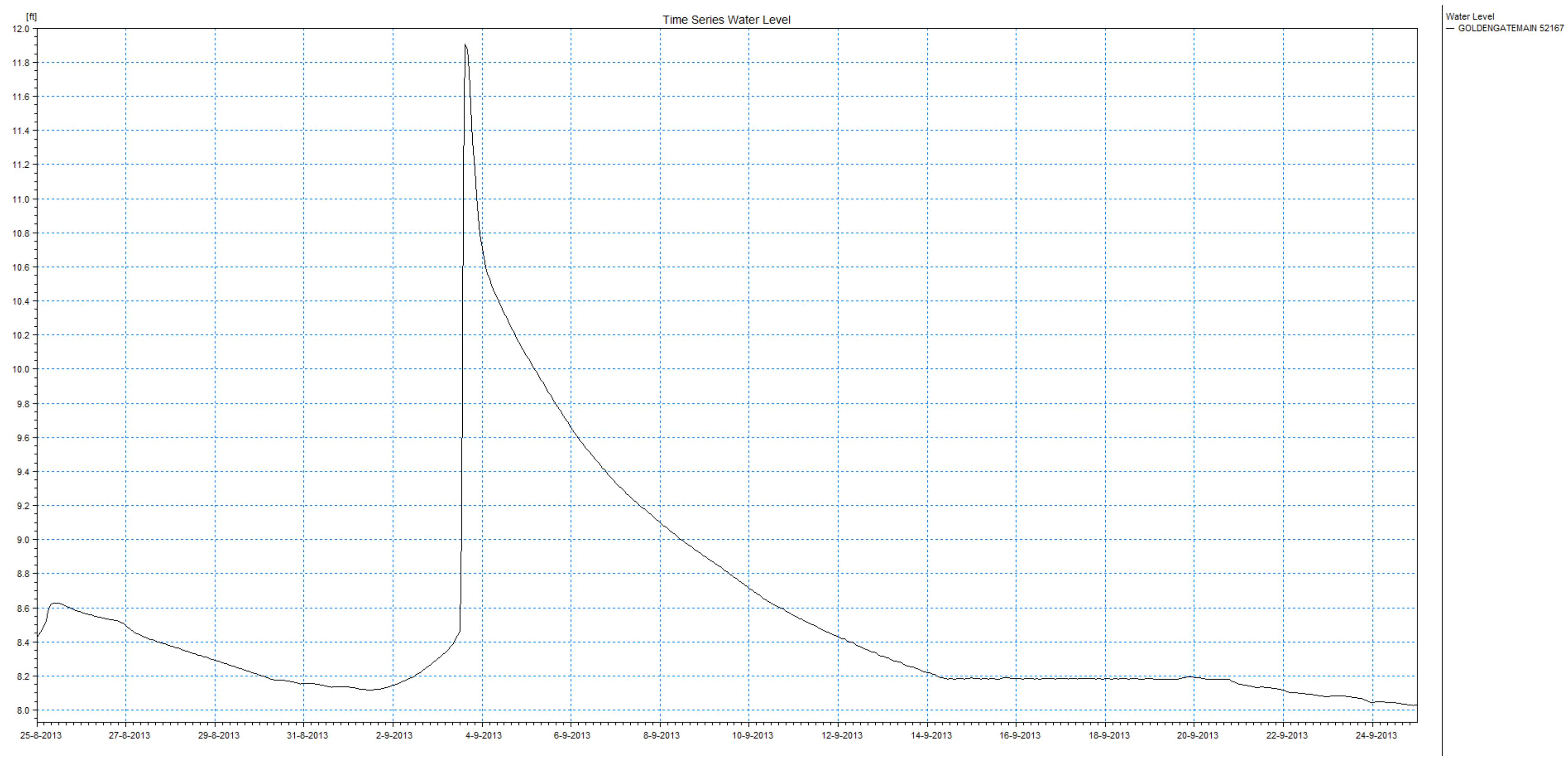

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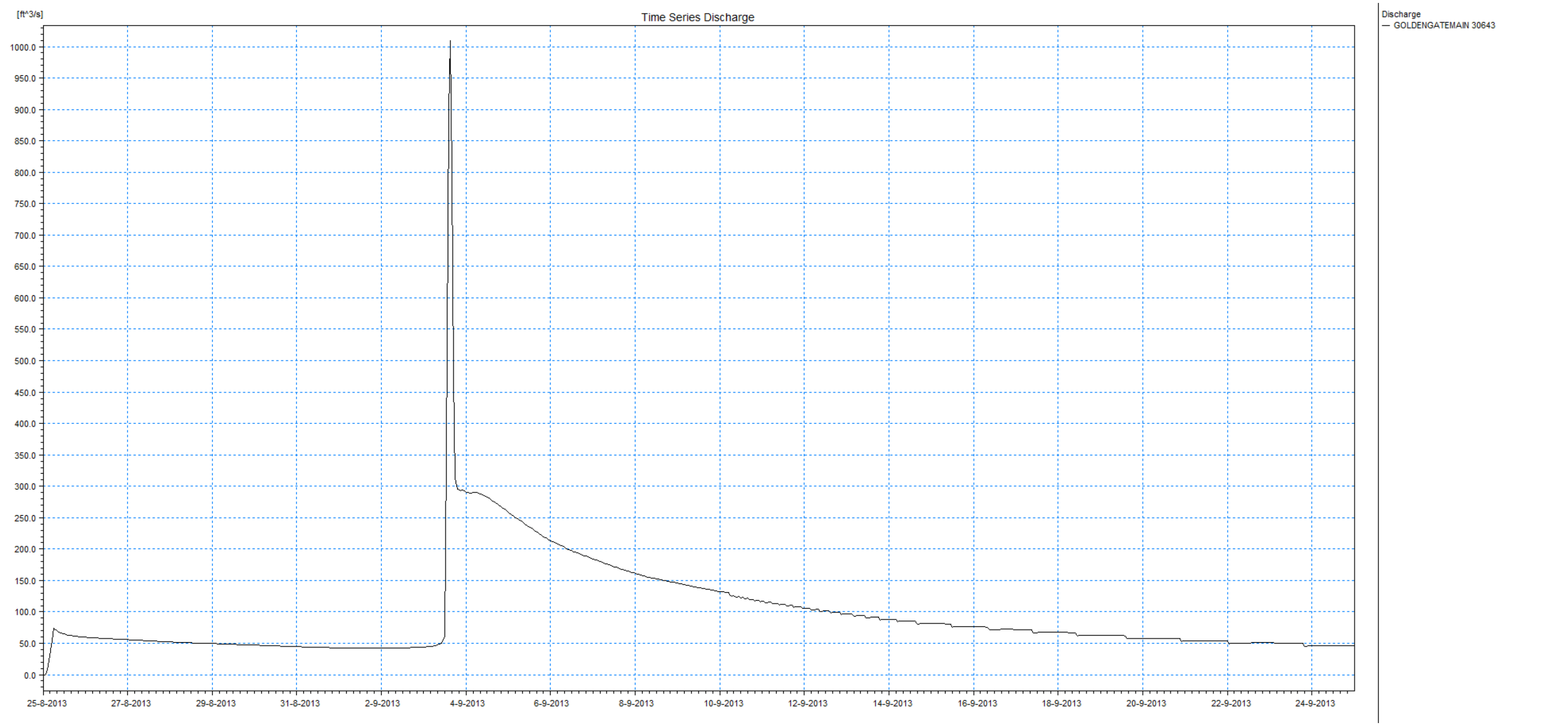

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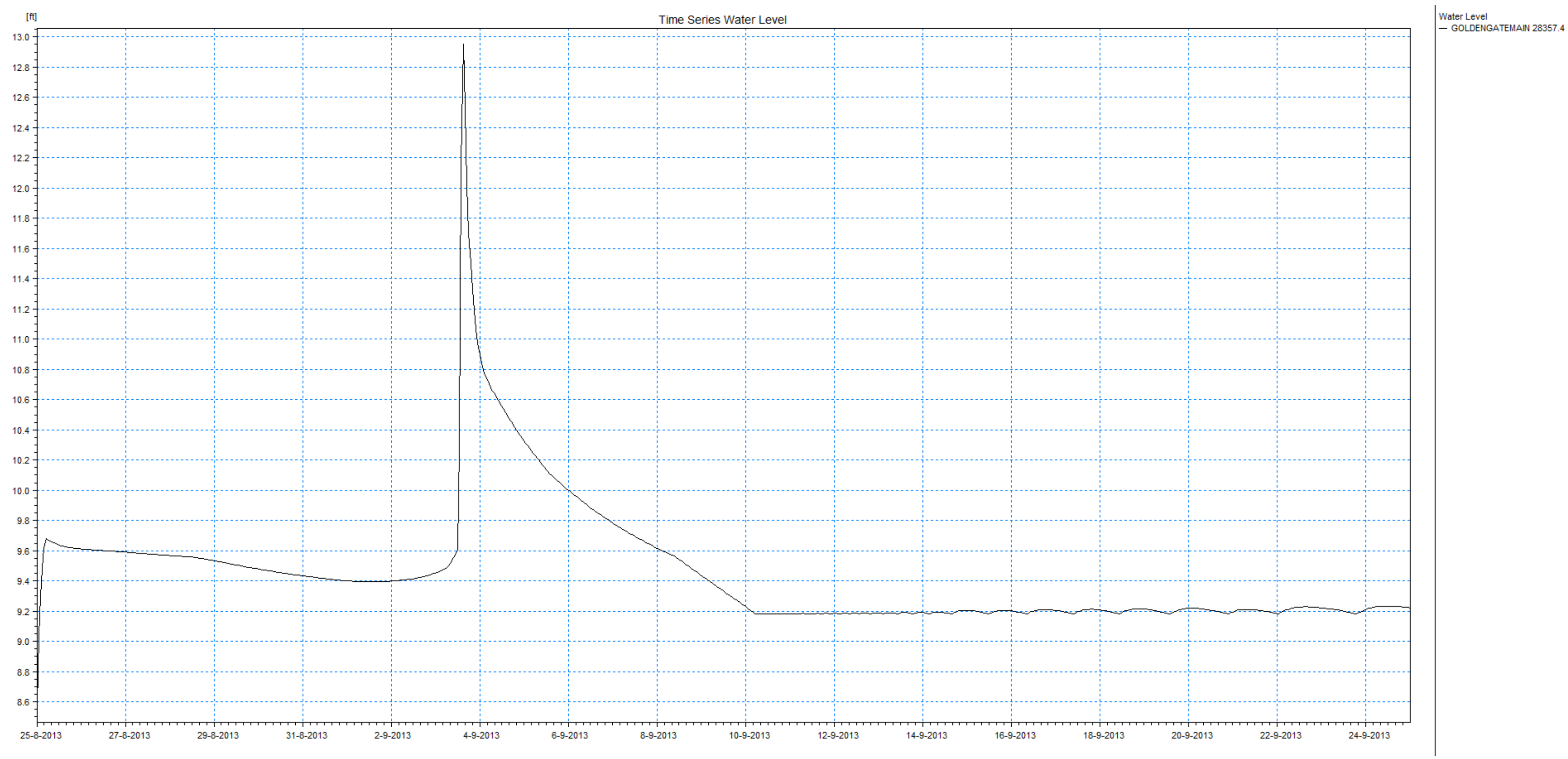

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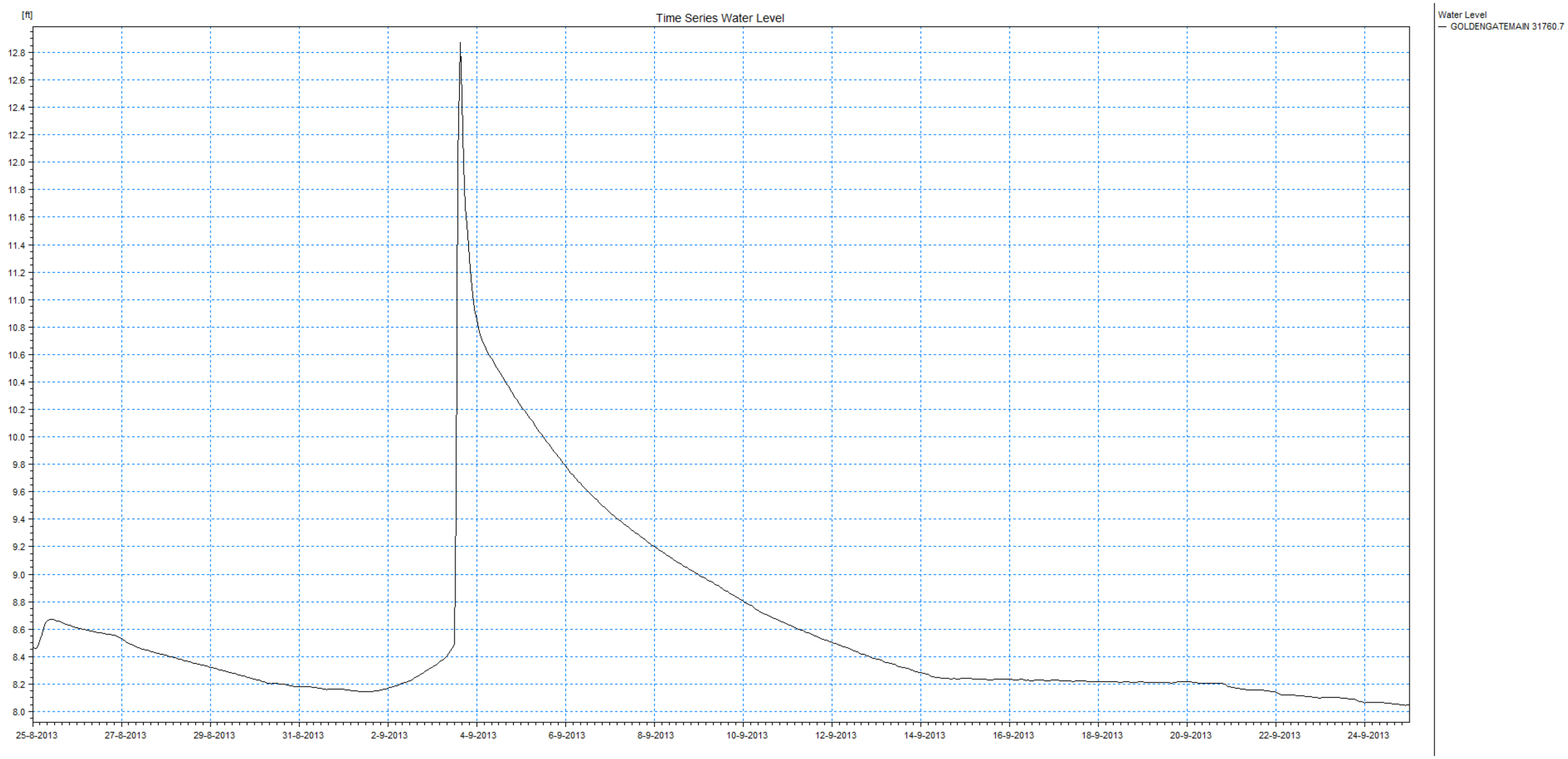

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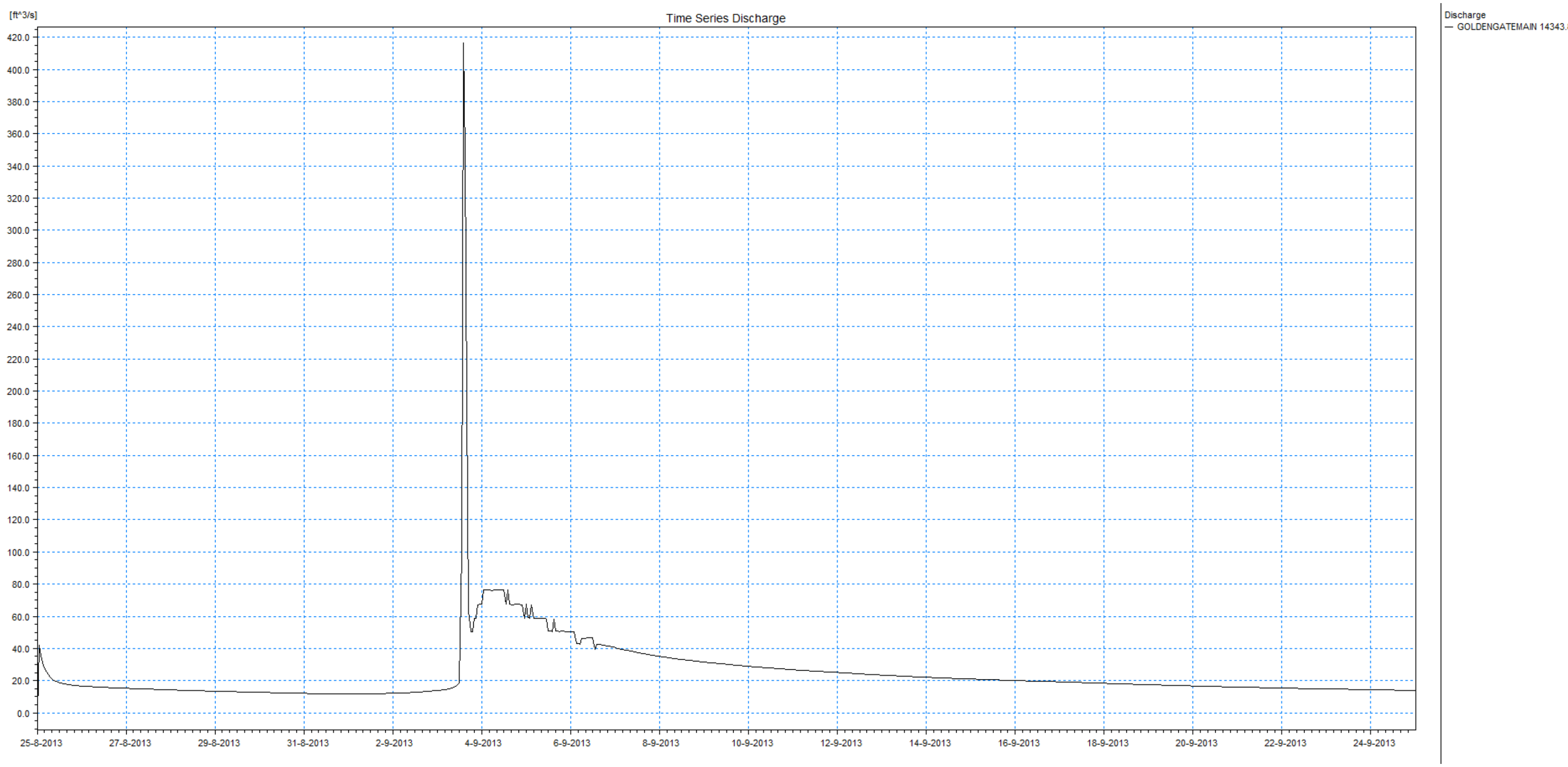

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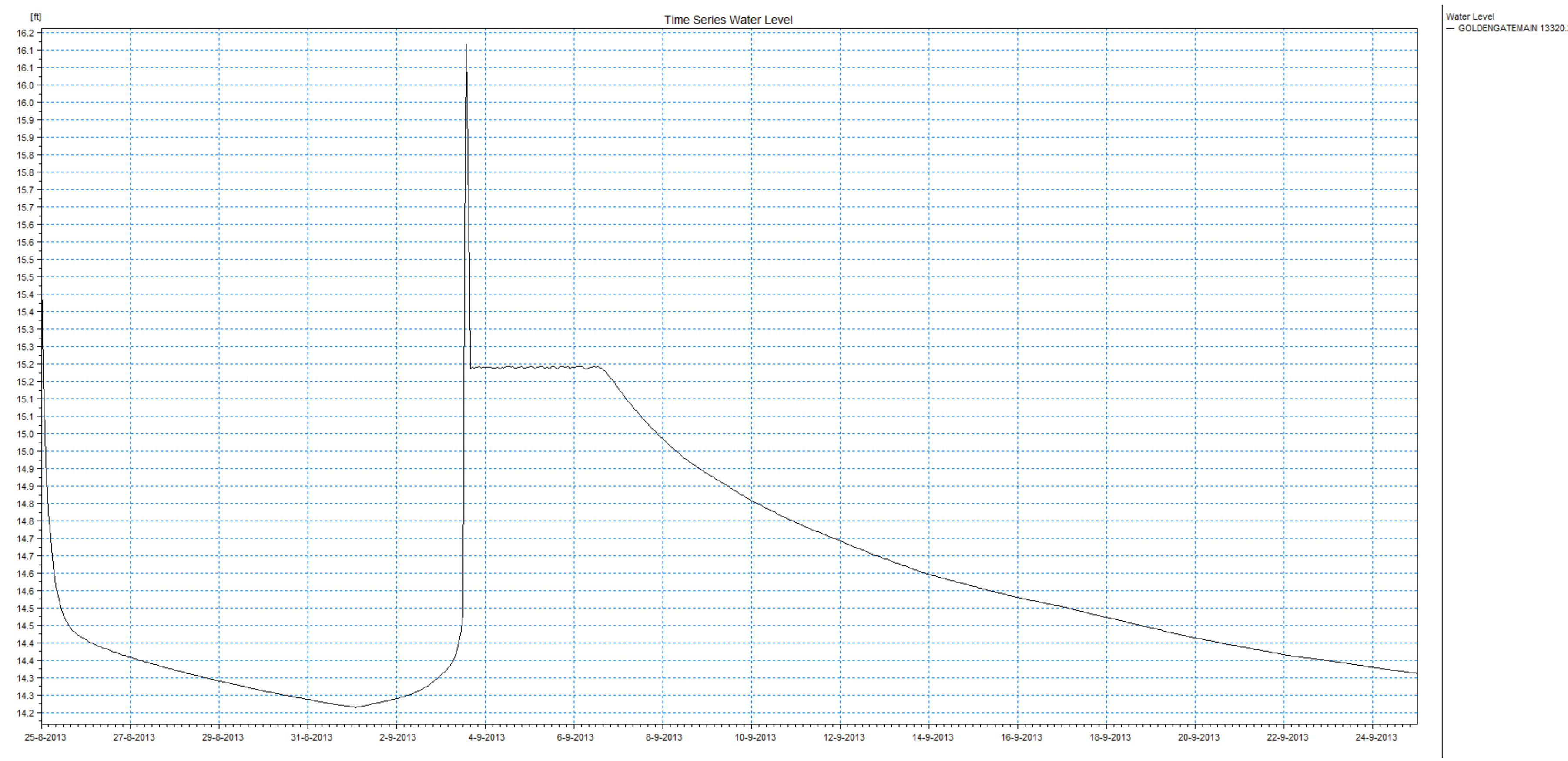

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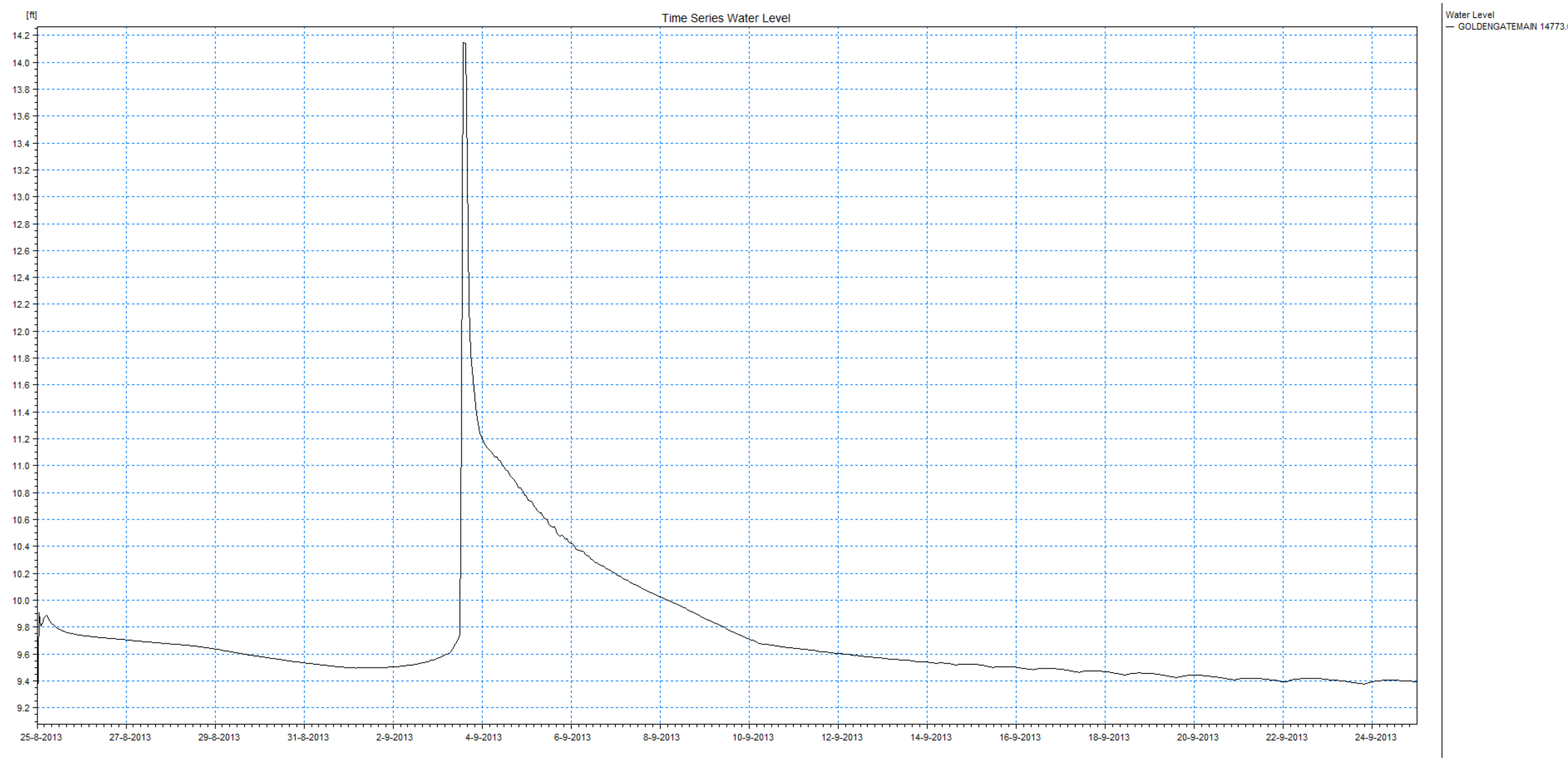

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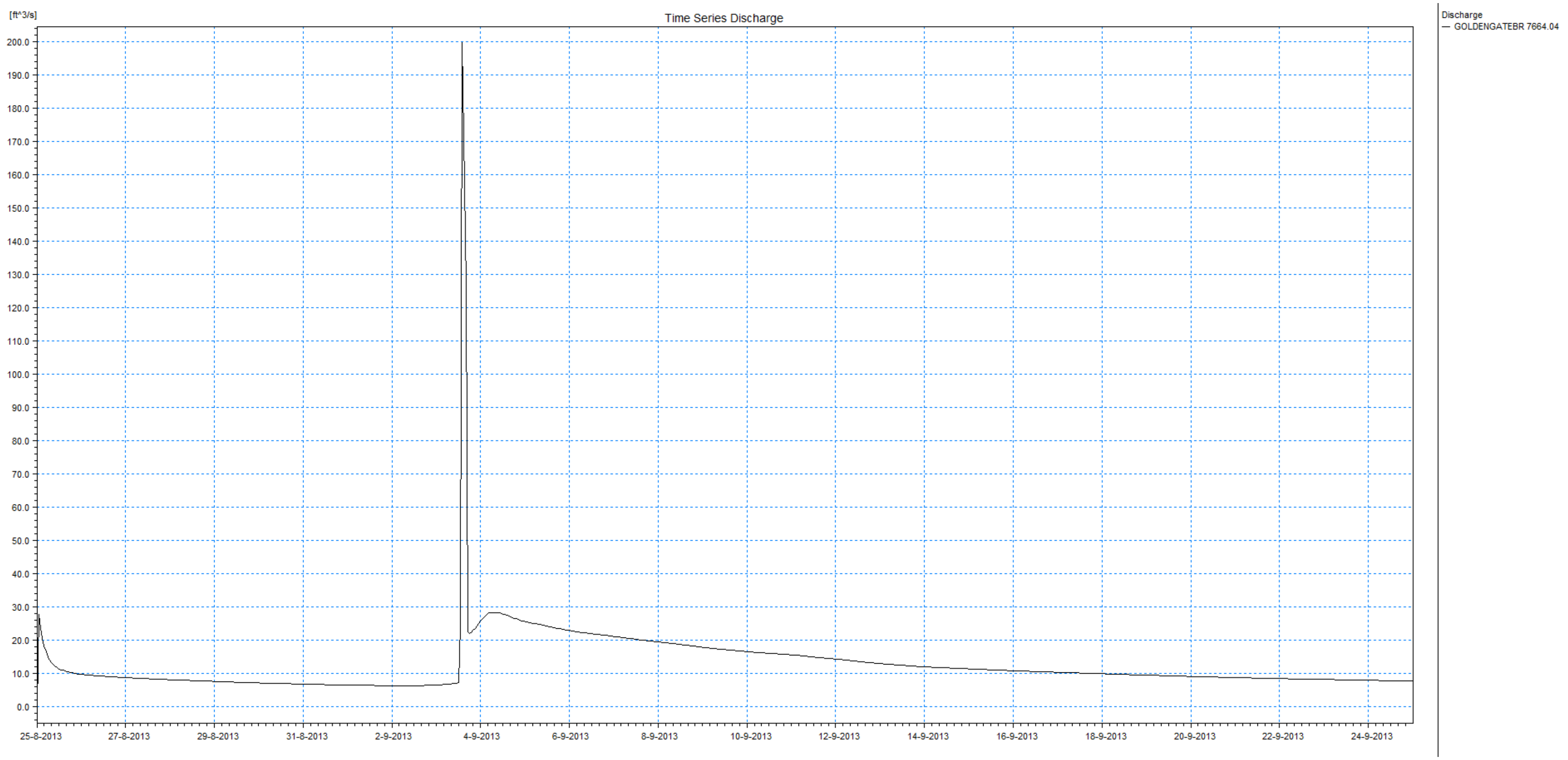

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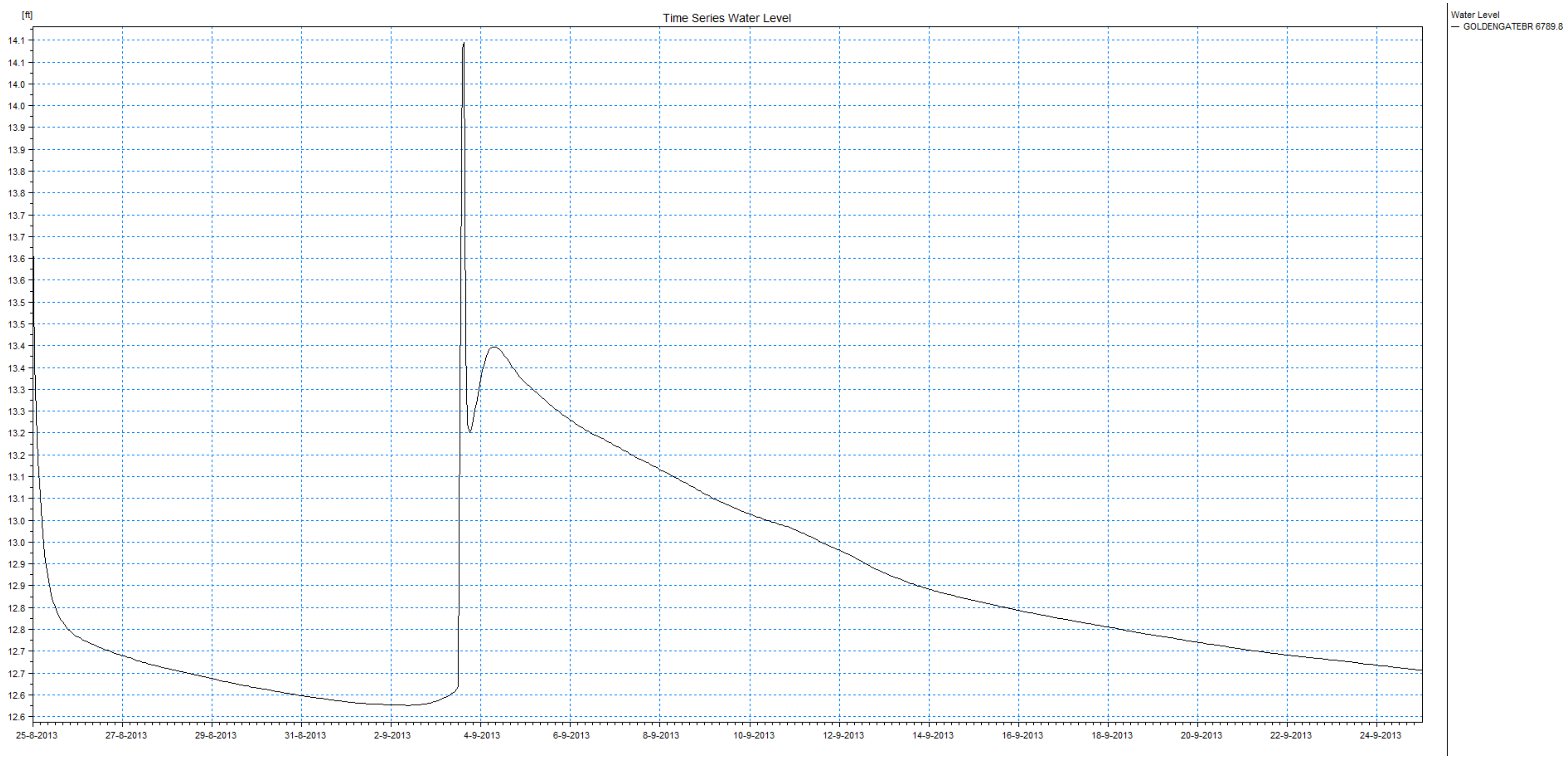

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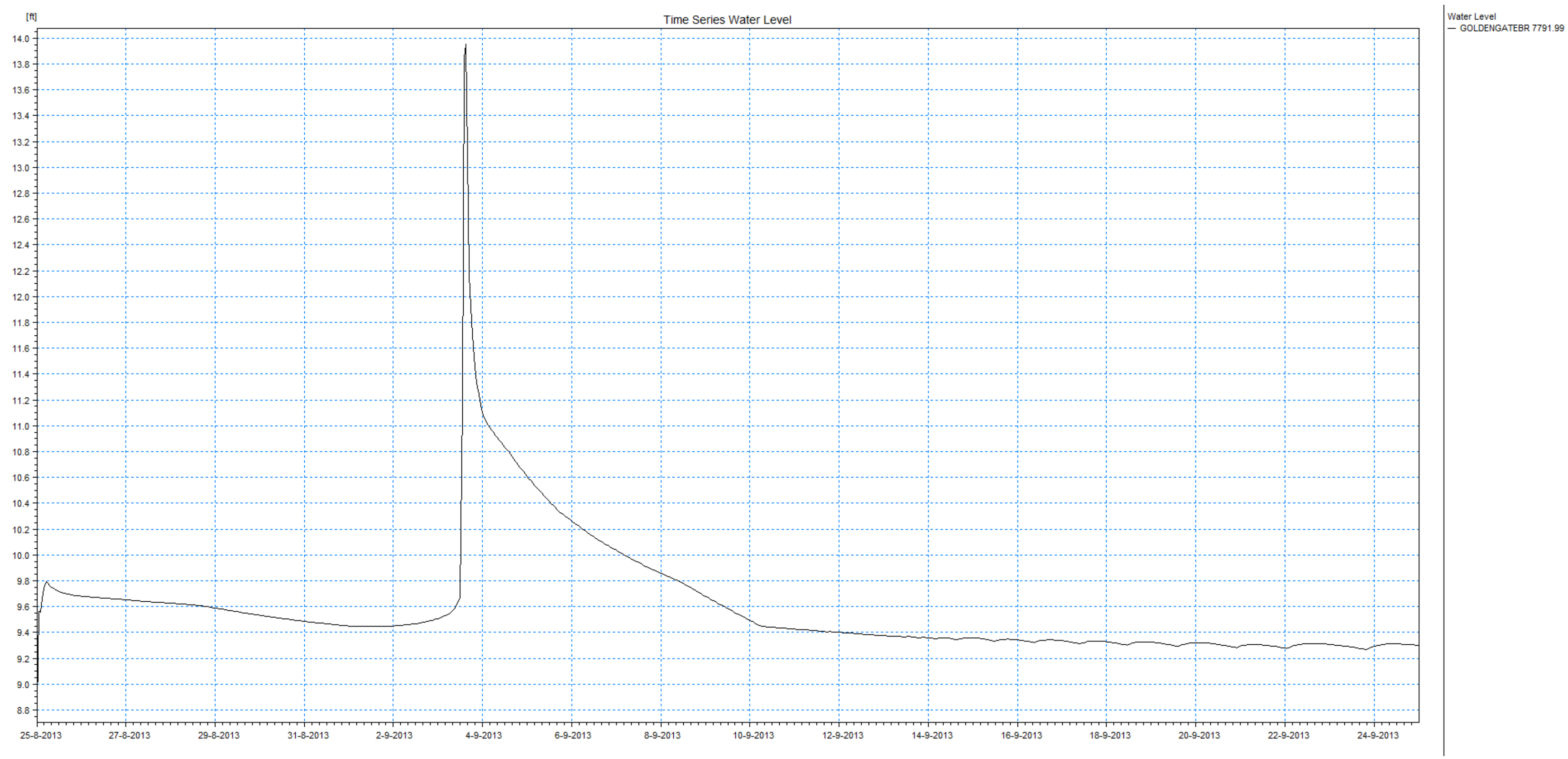

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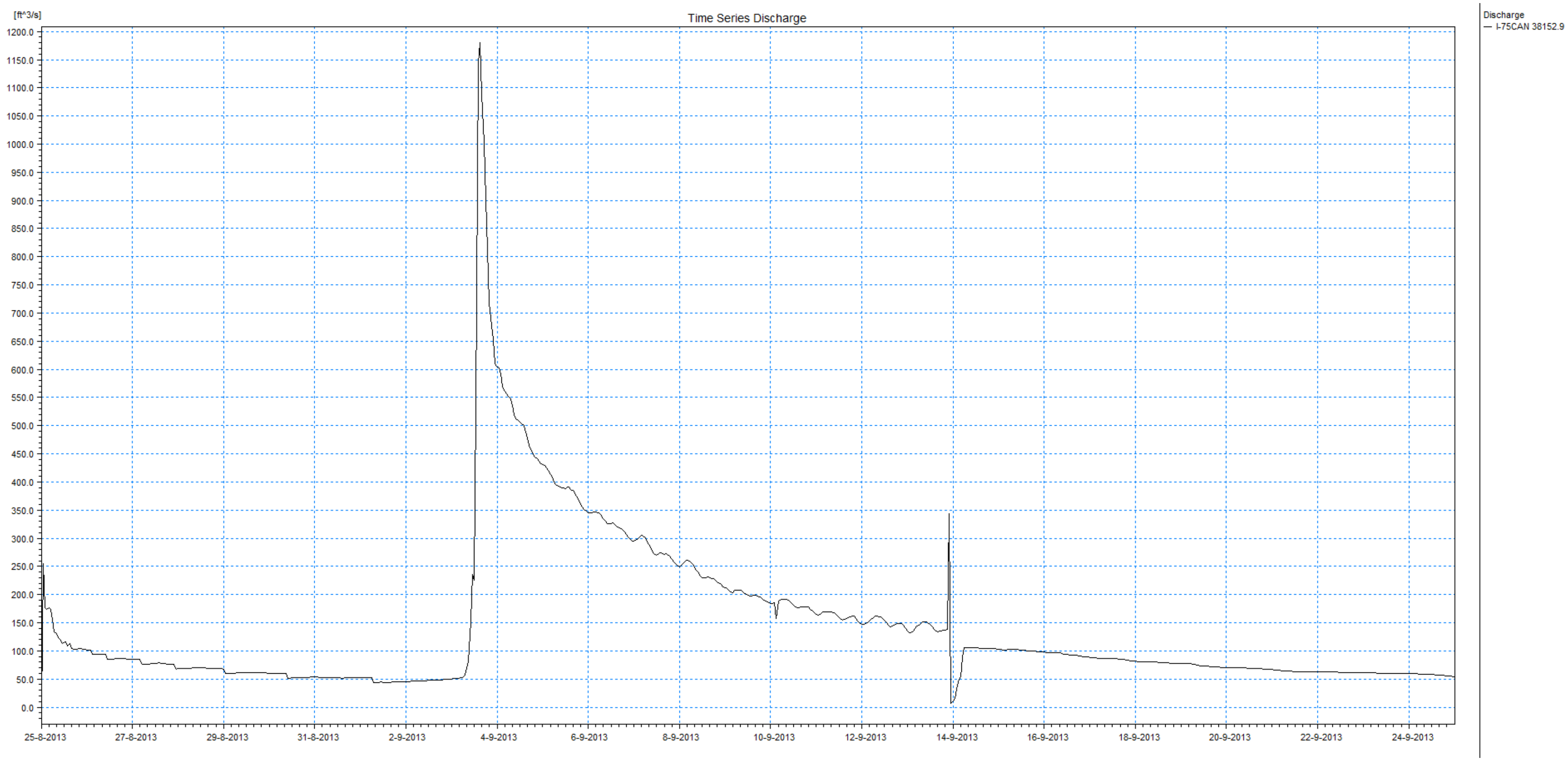

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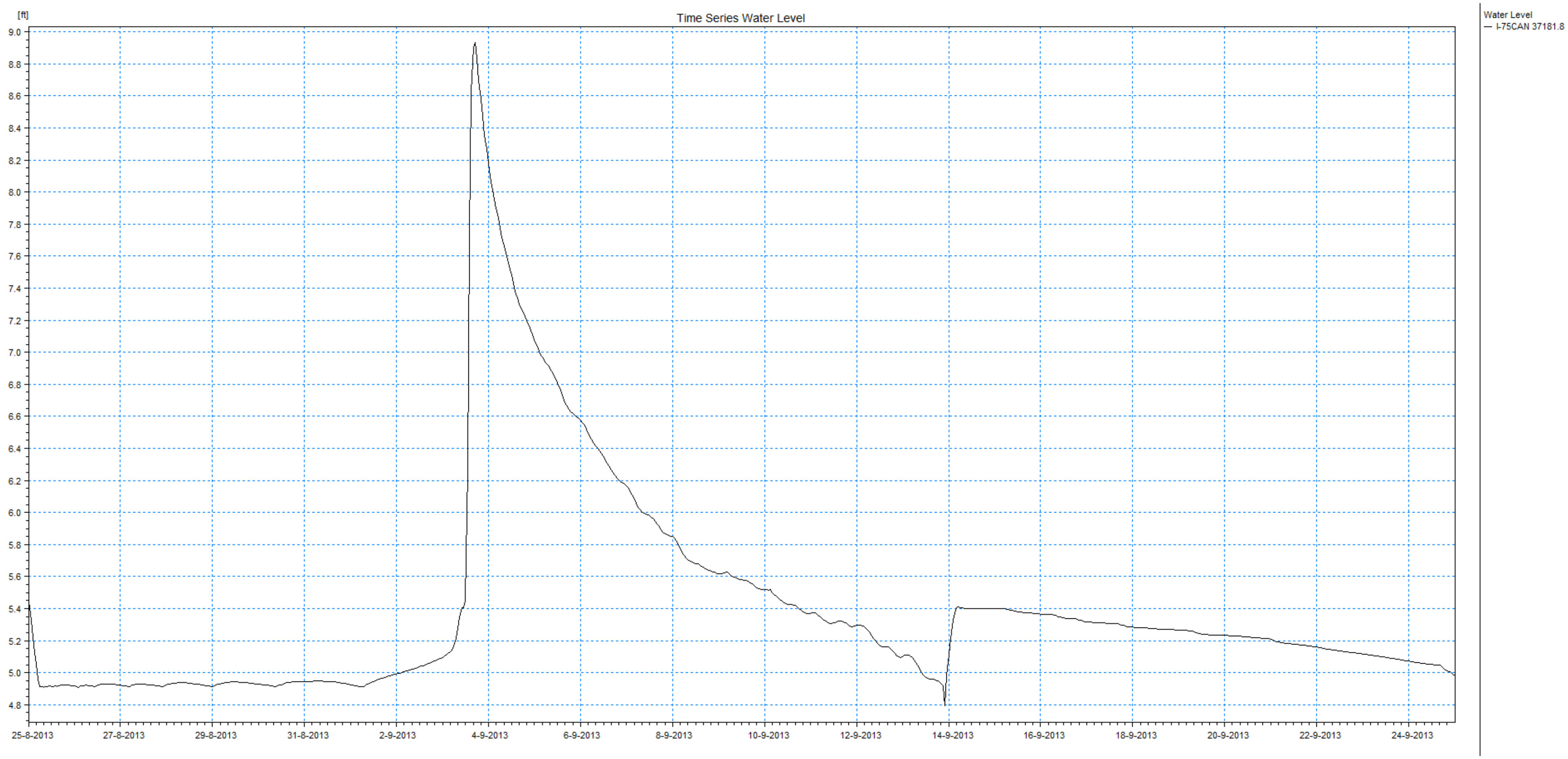

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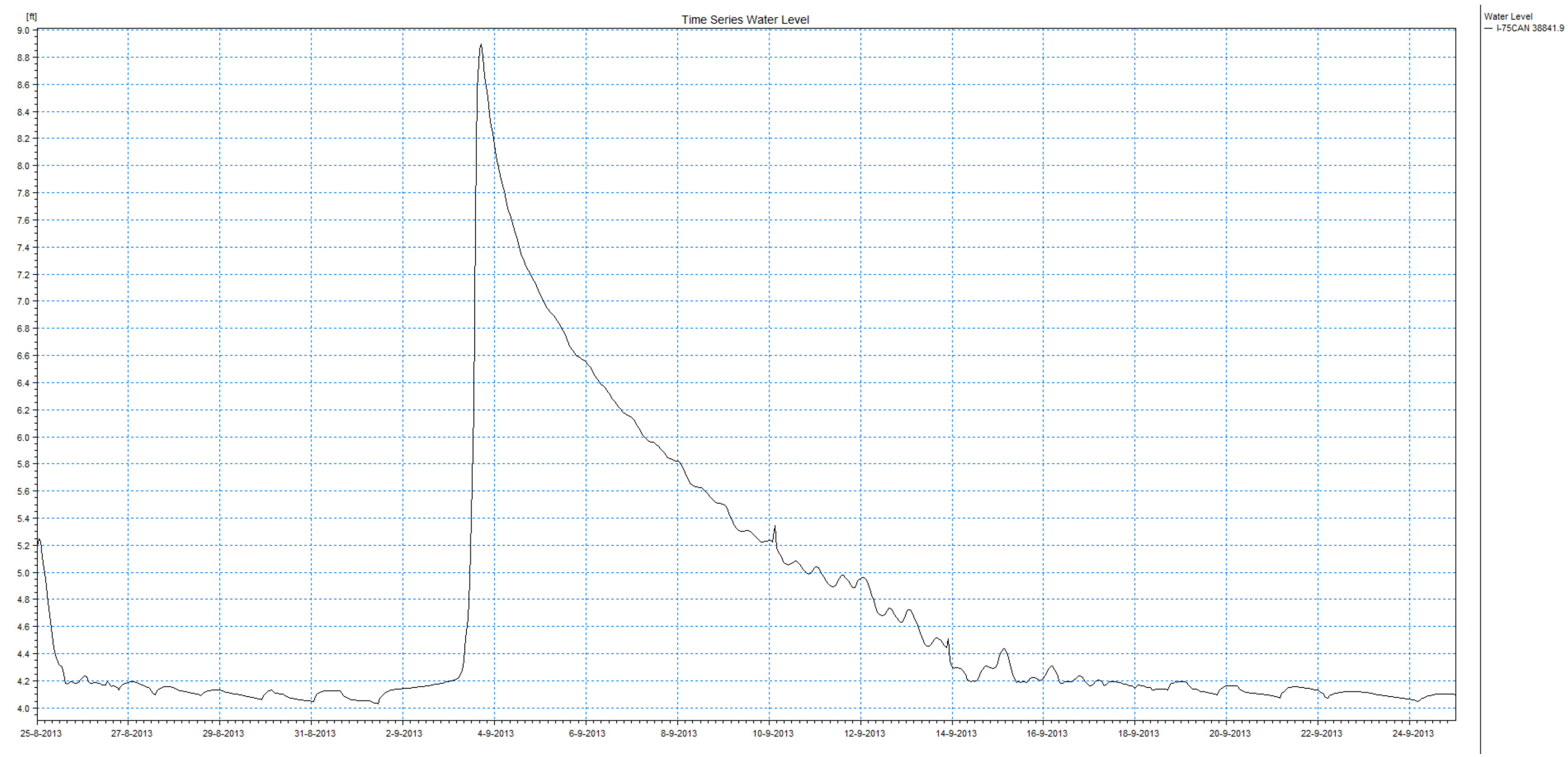
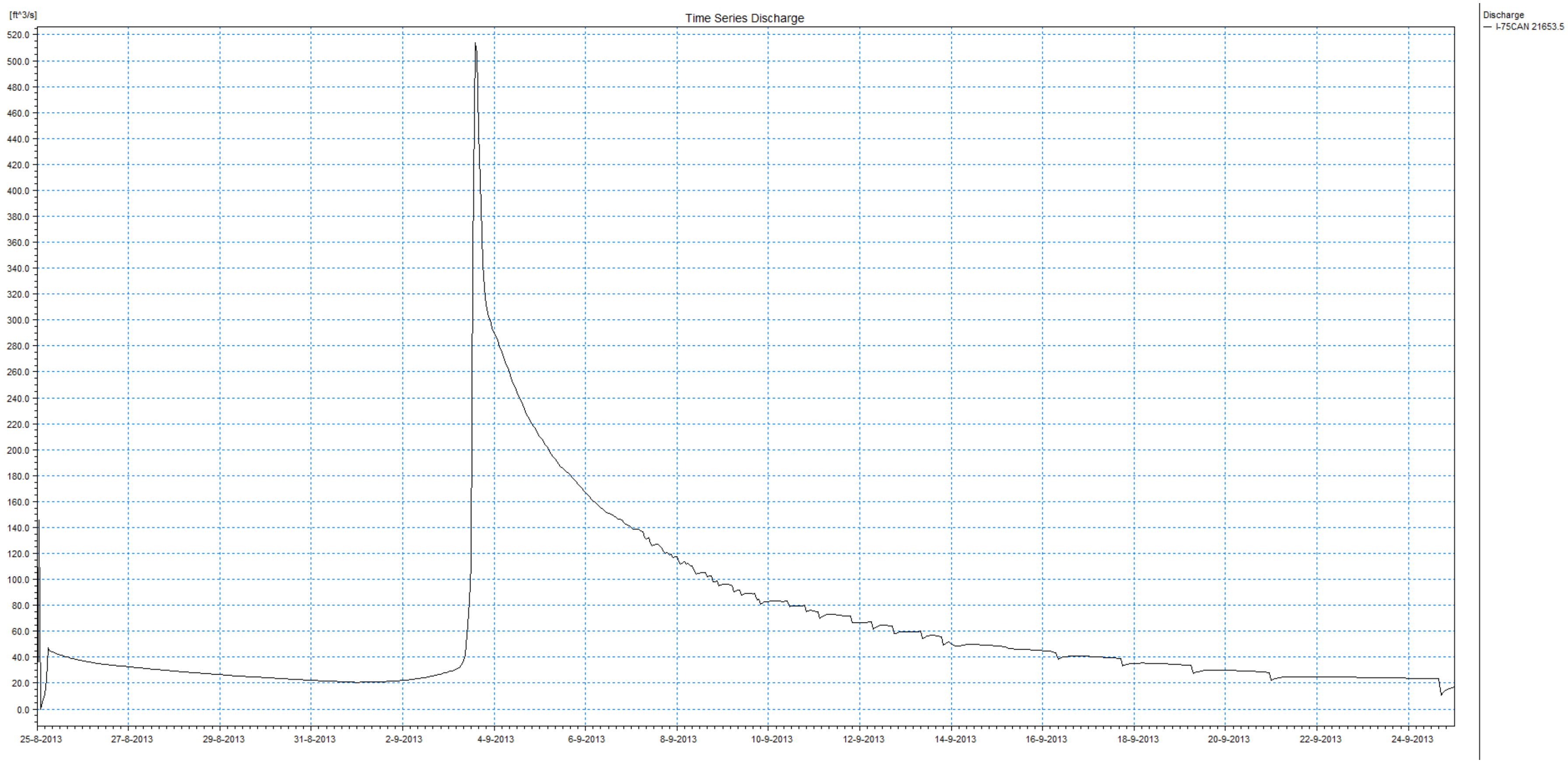


Figure 43. I75W1 Tailwater Stage (ft)



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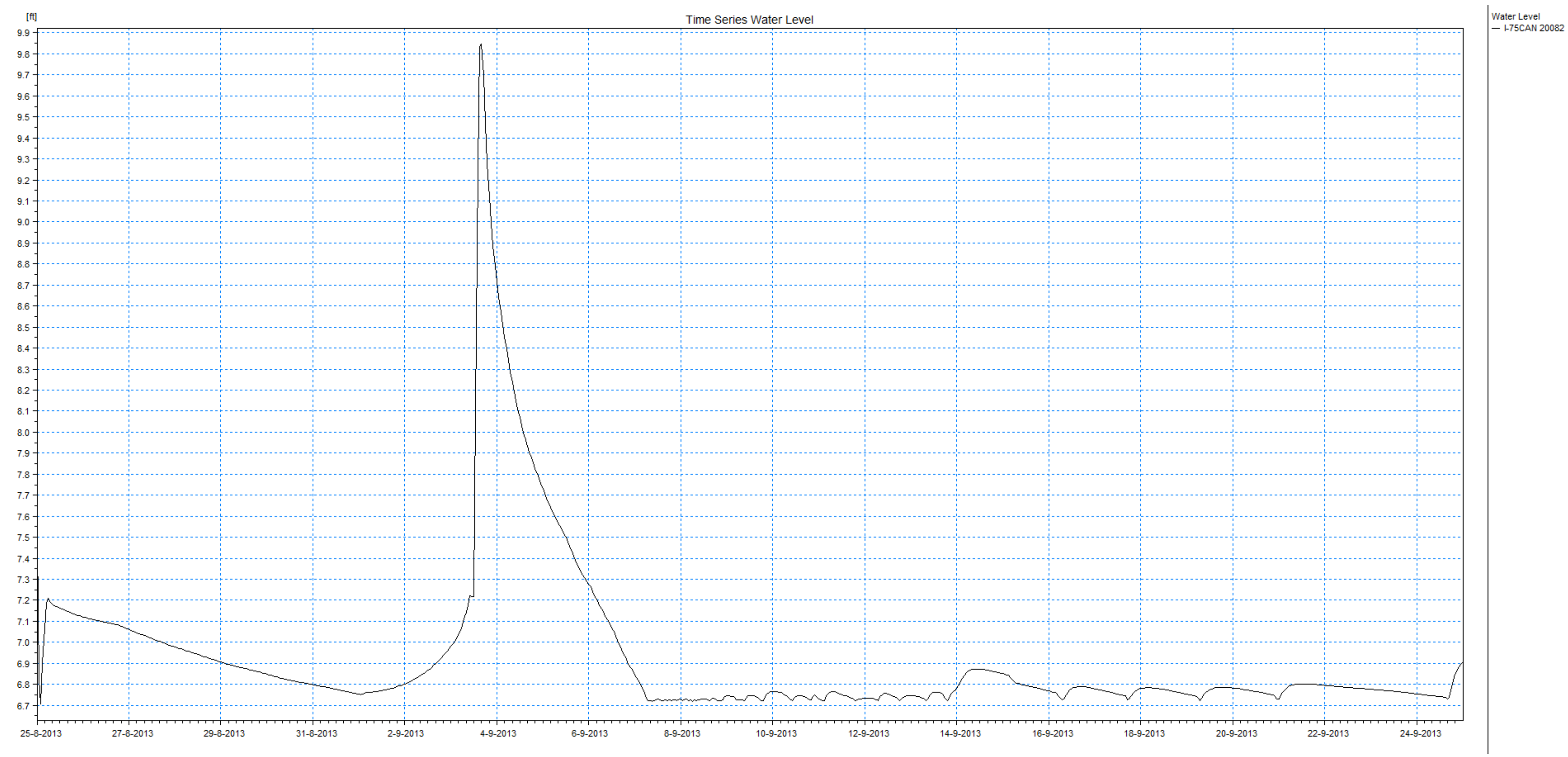

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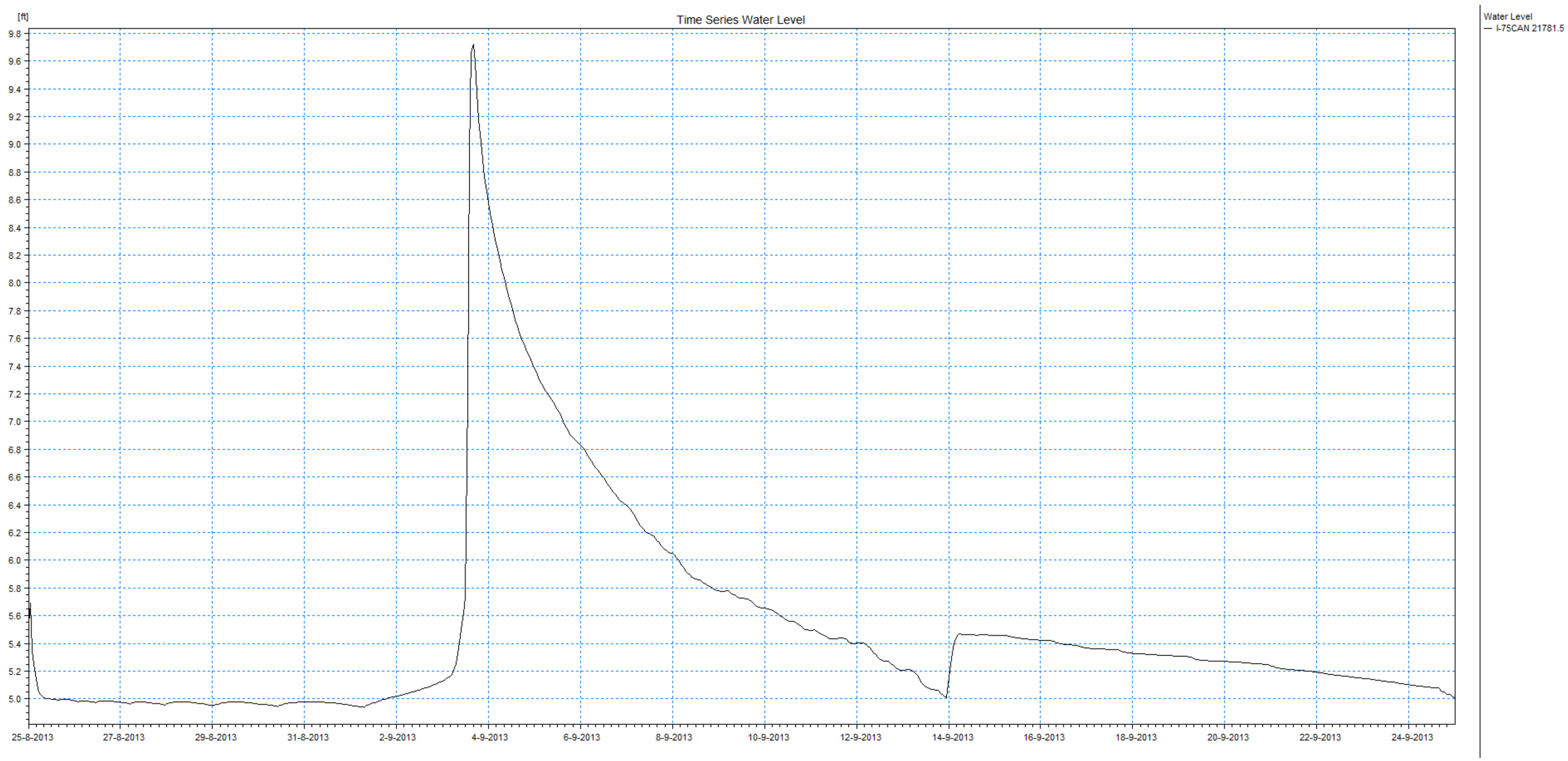

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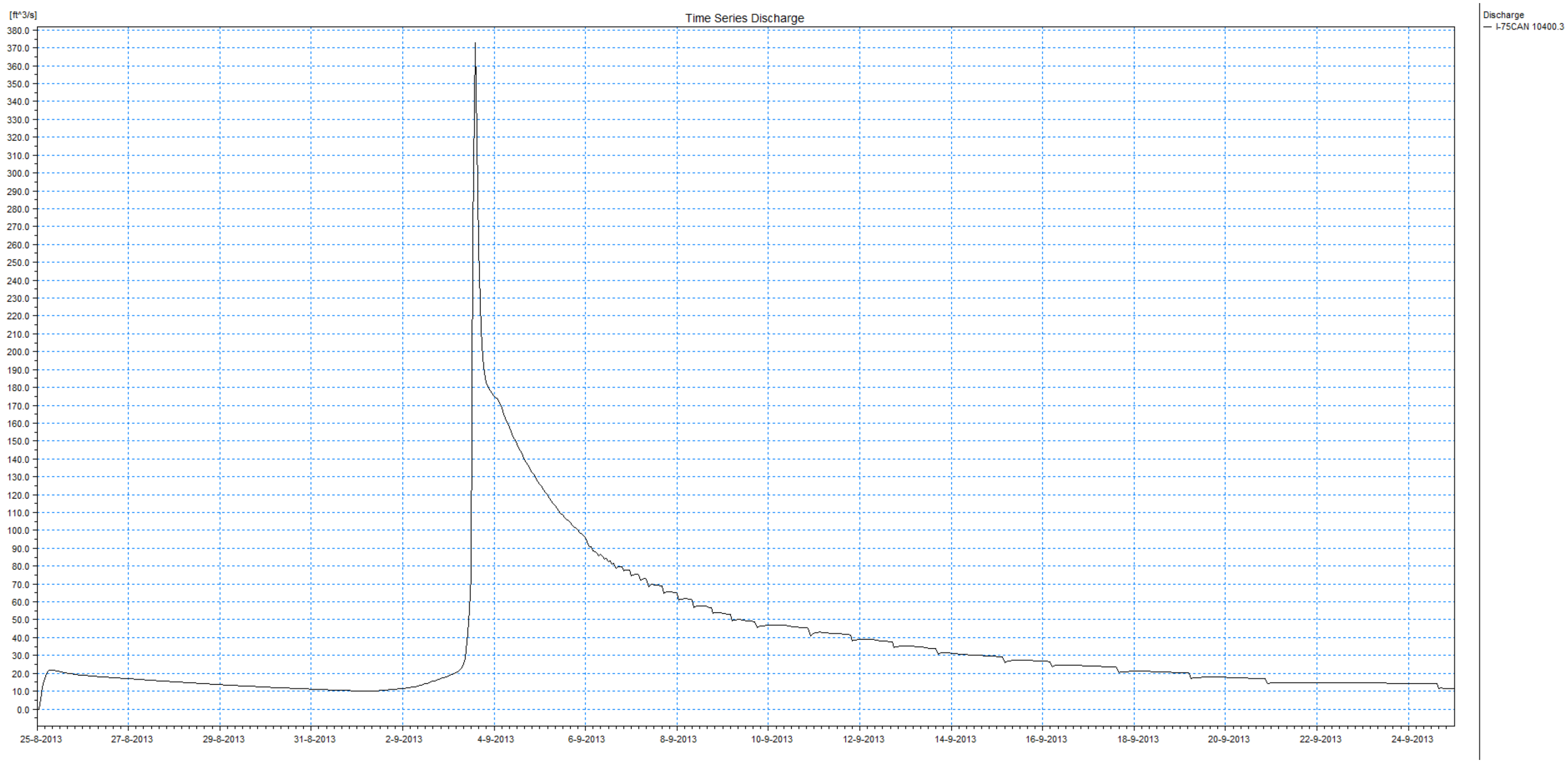

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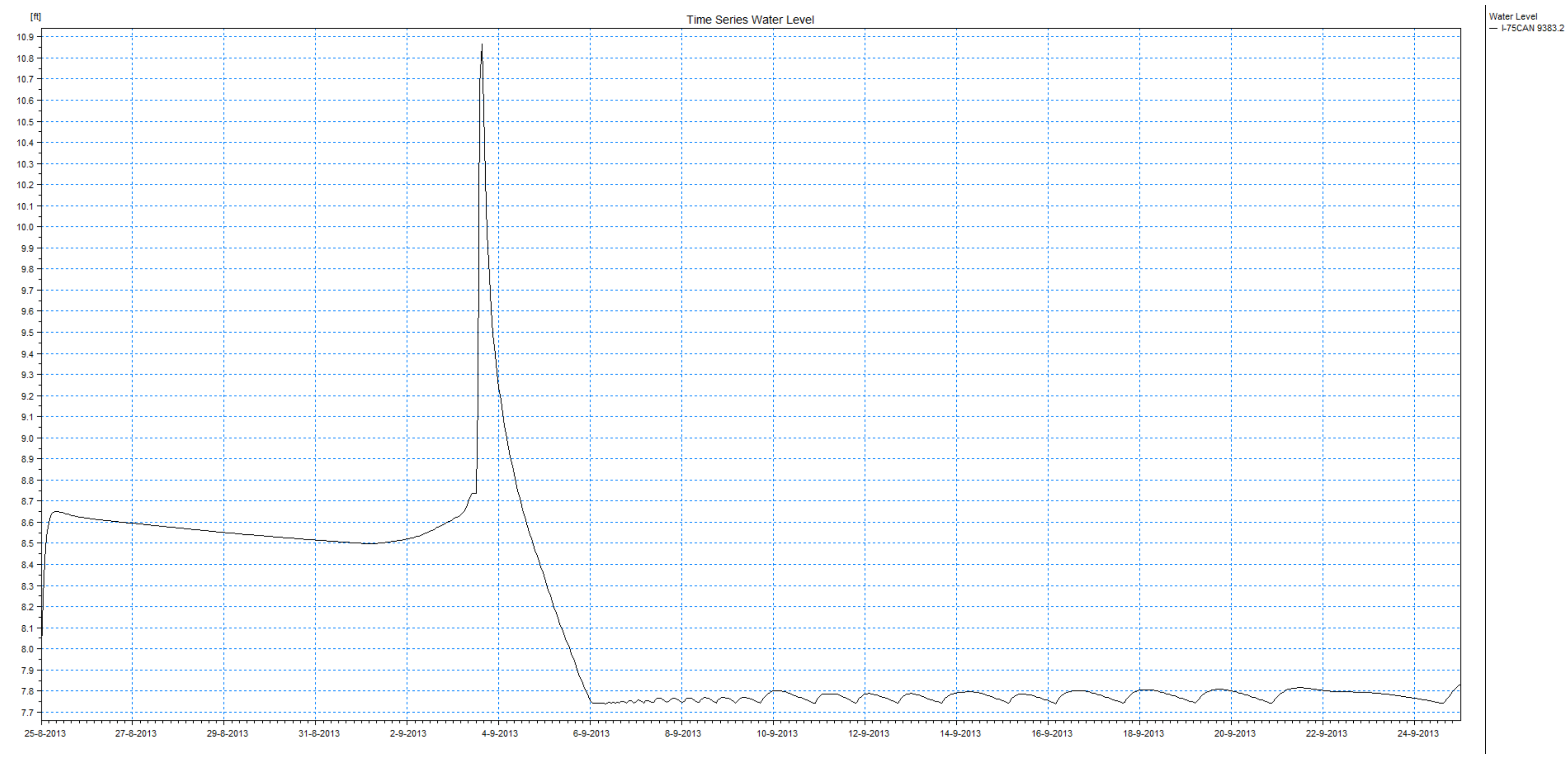

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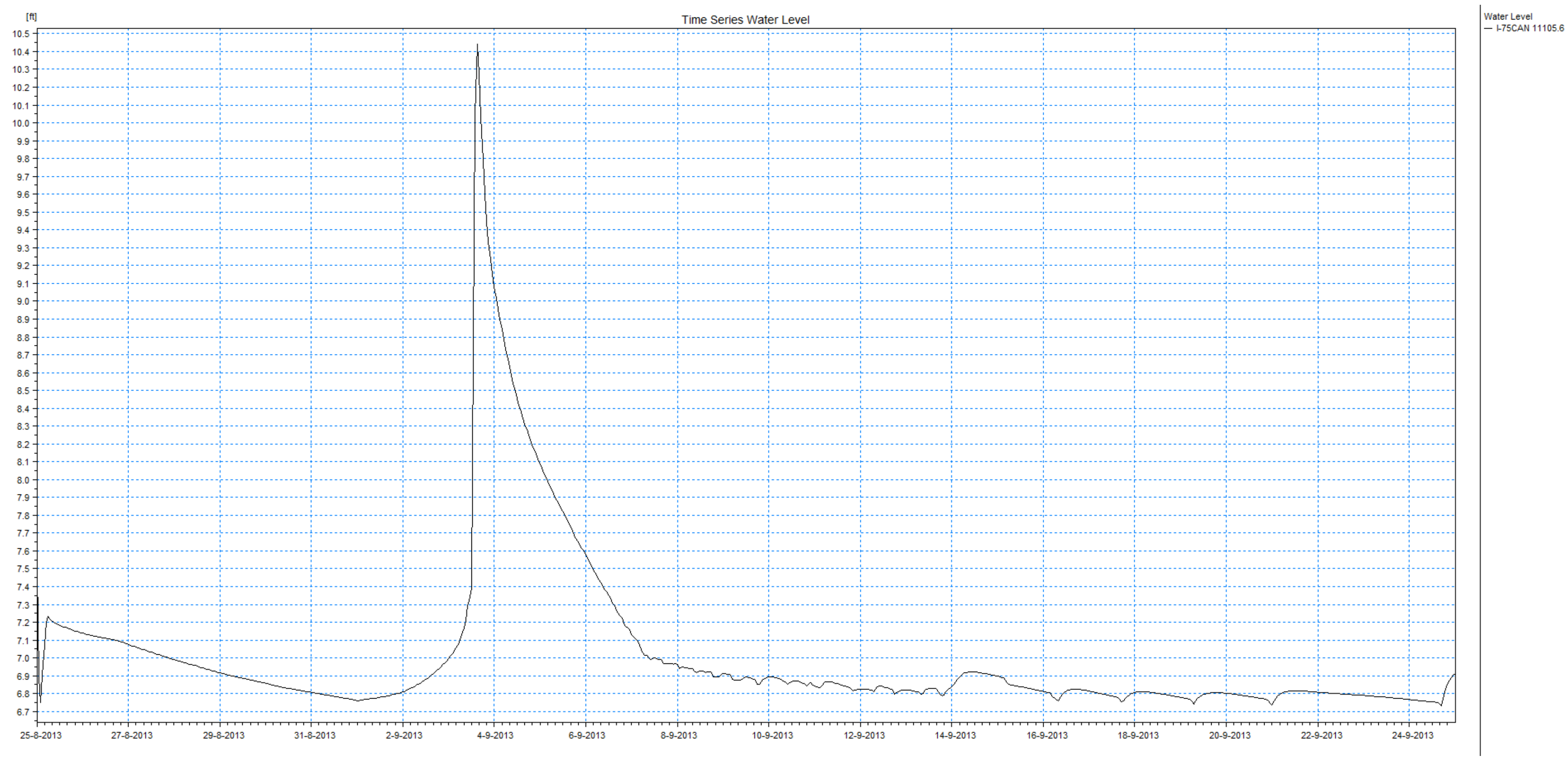

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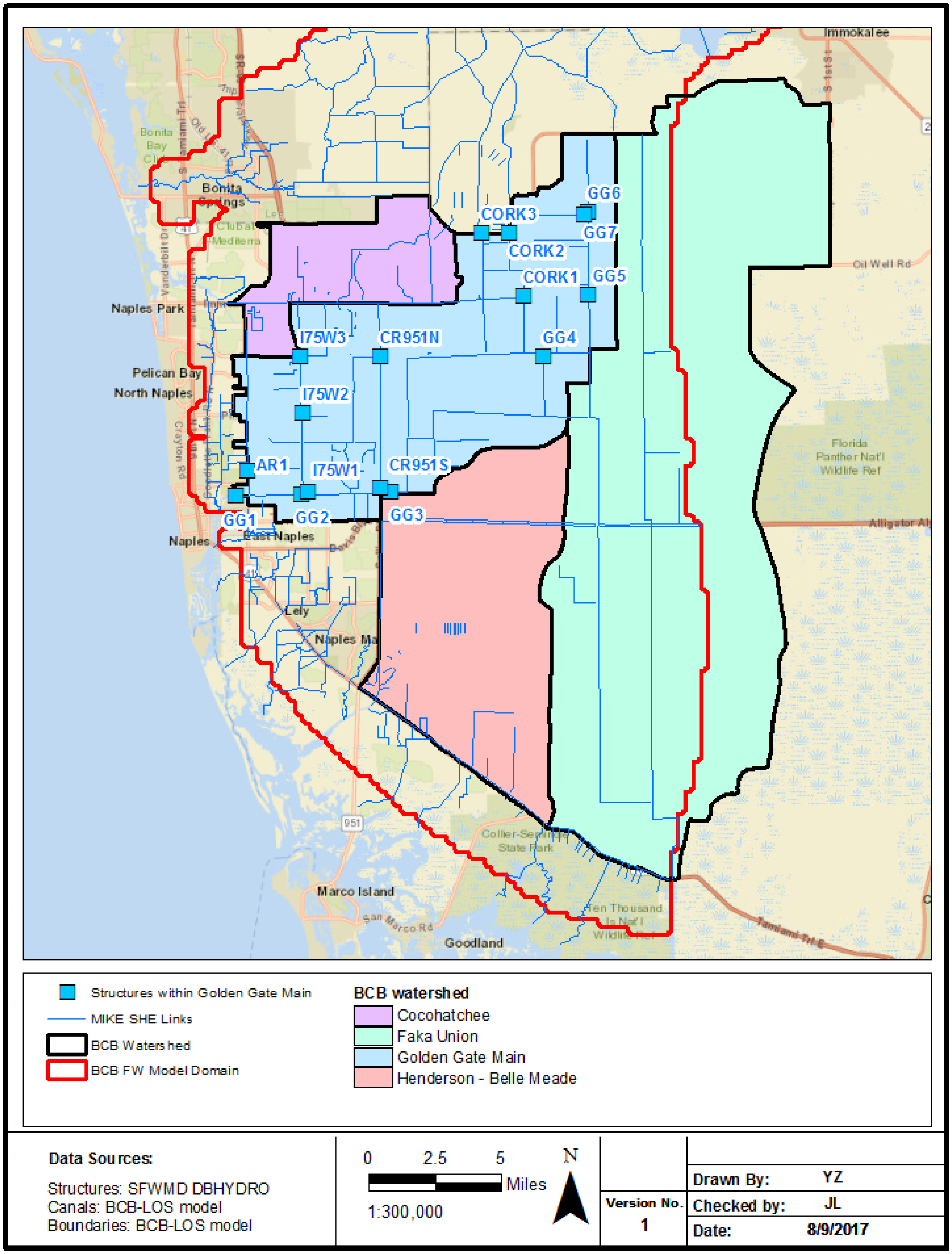

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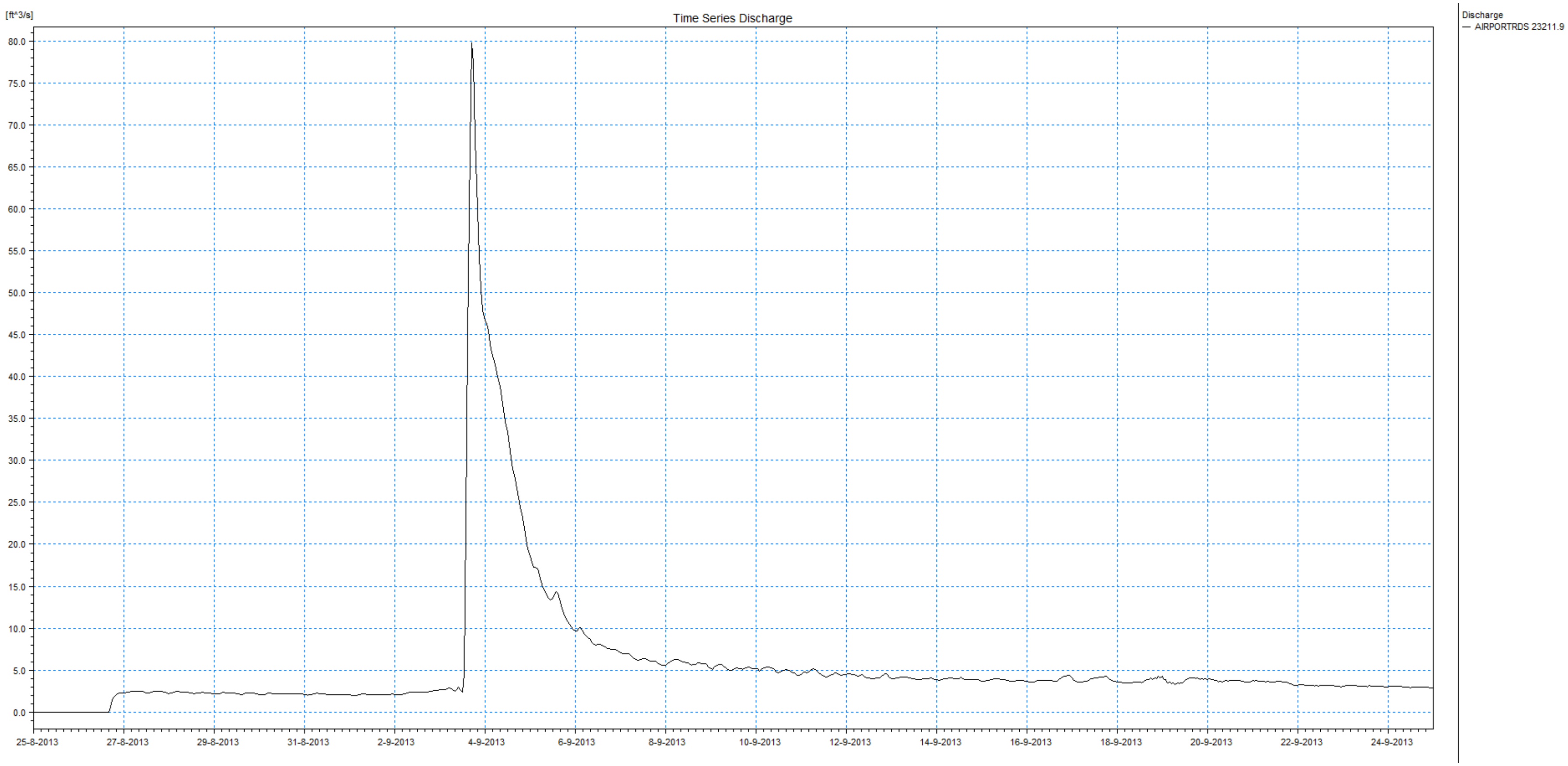

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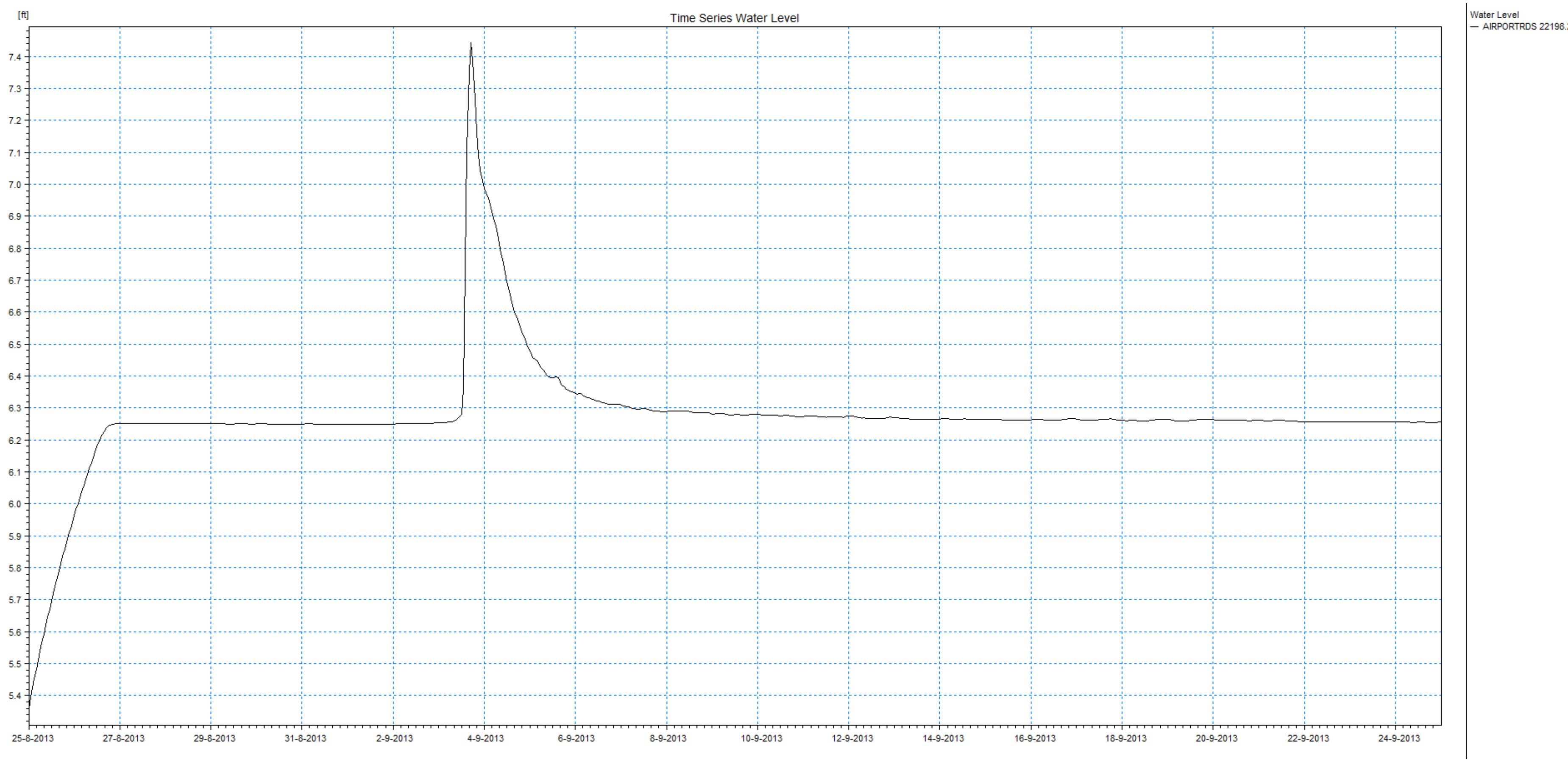

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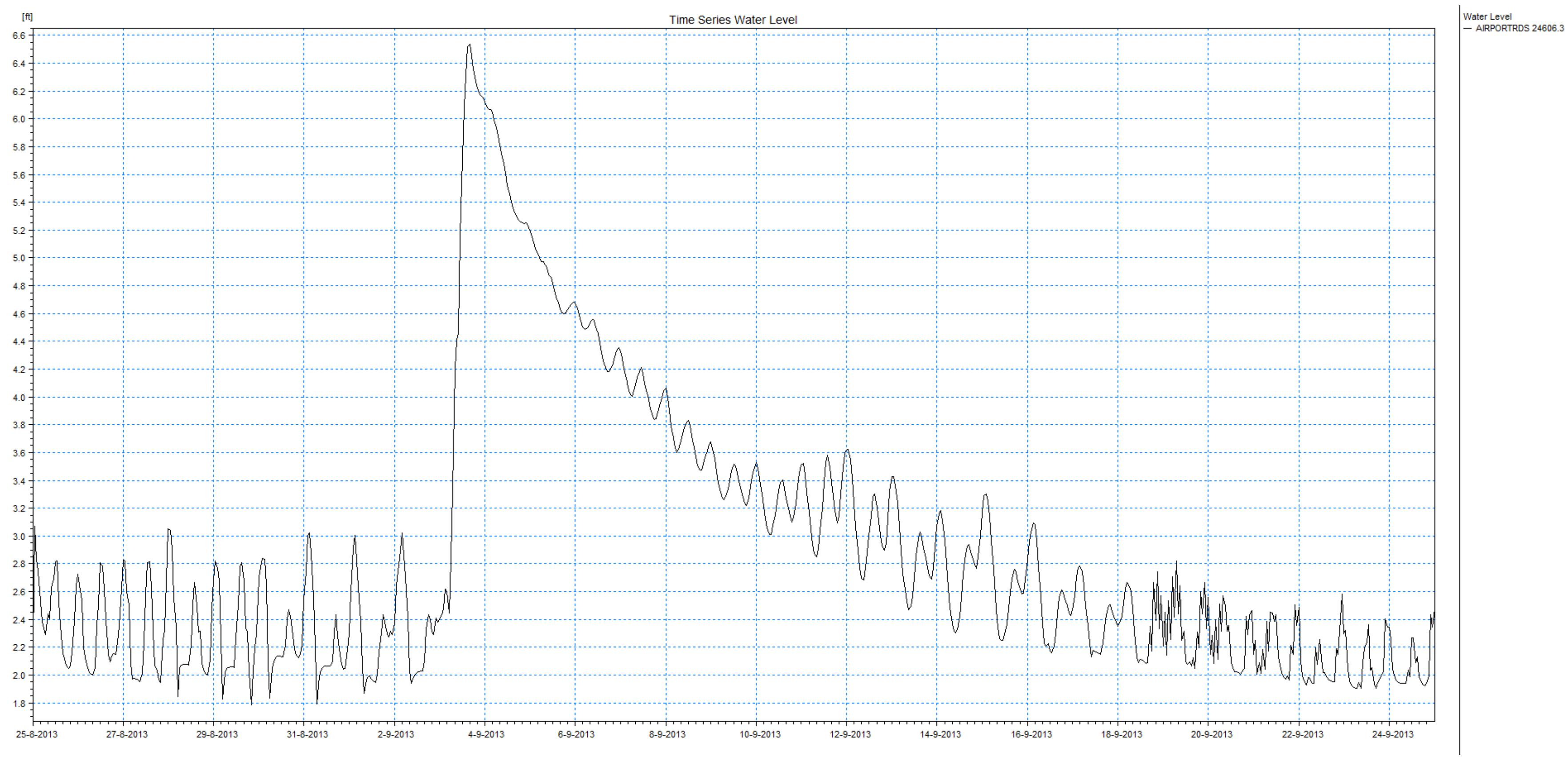

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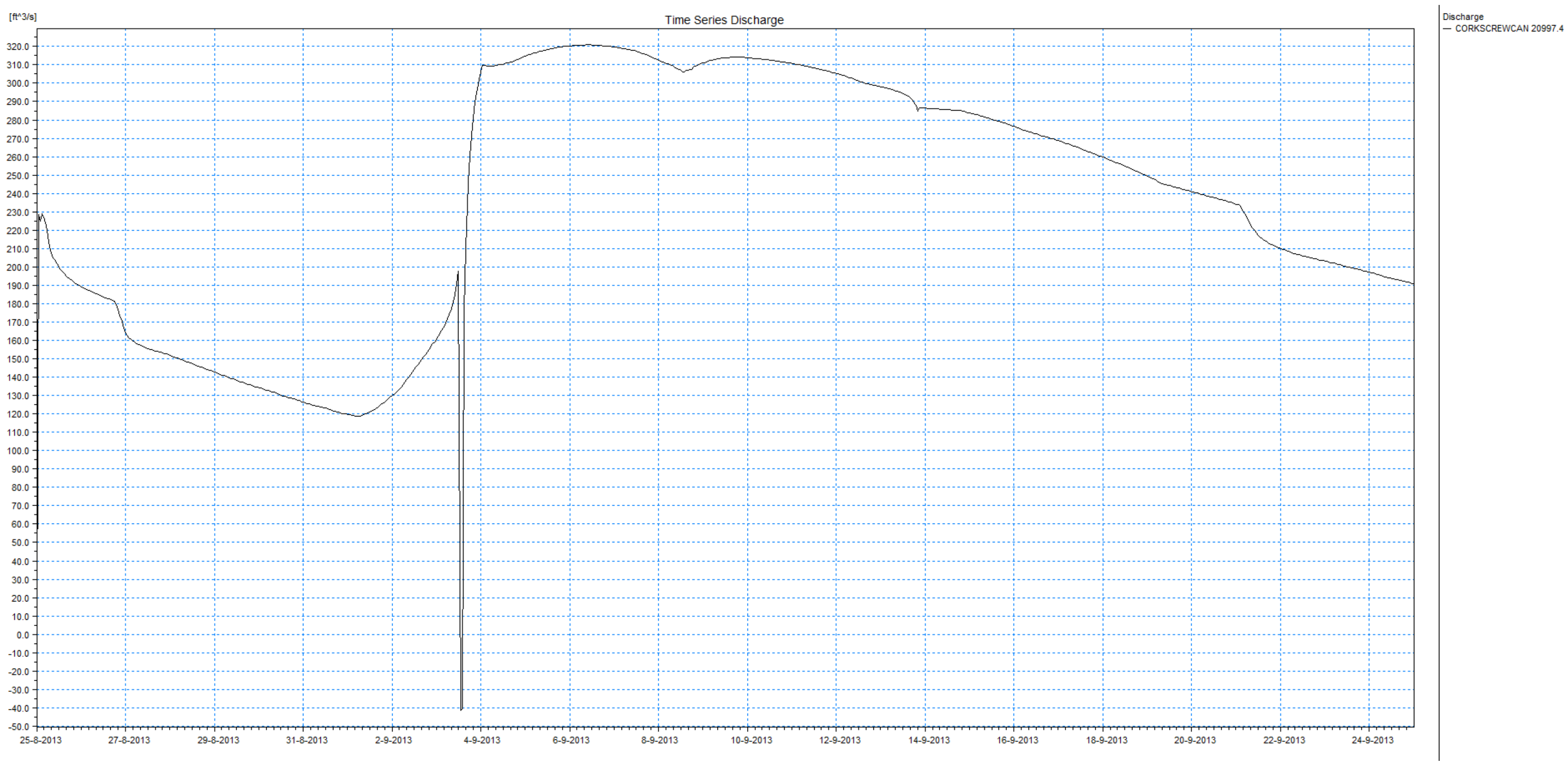

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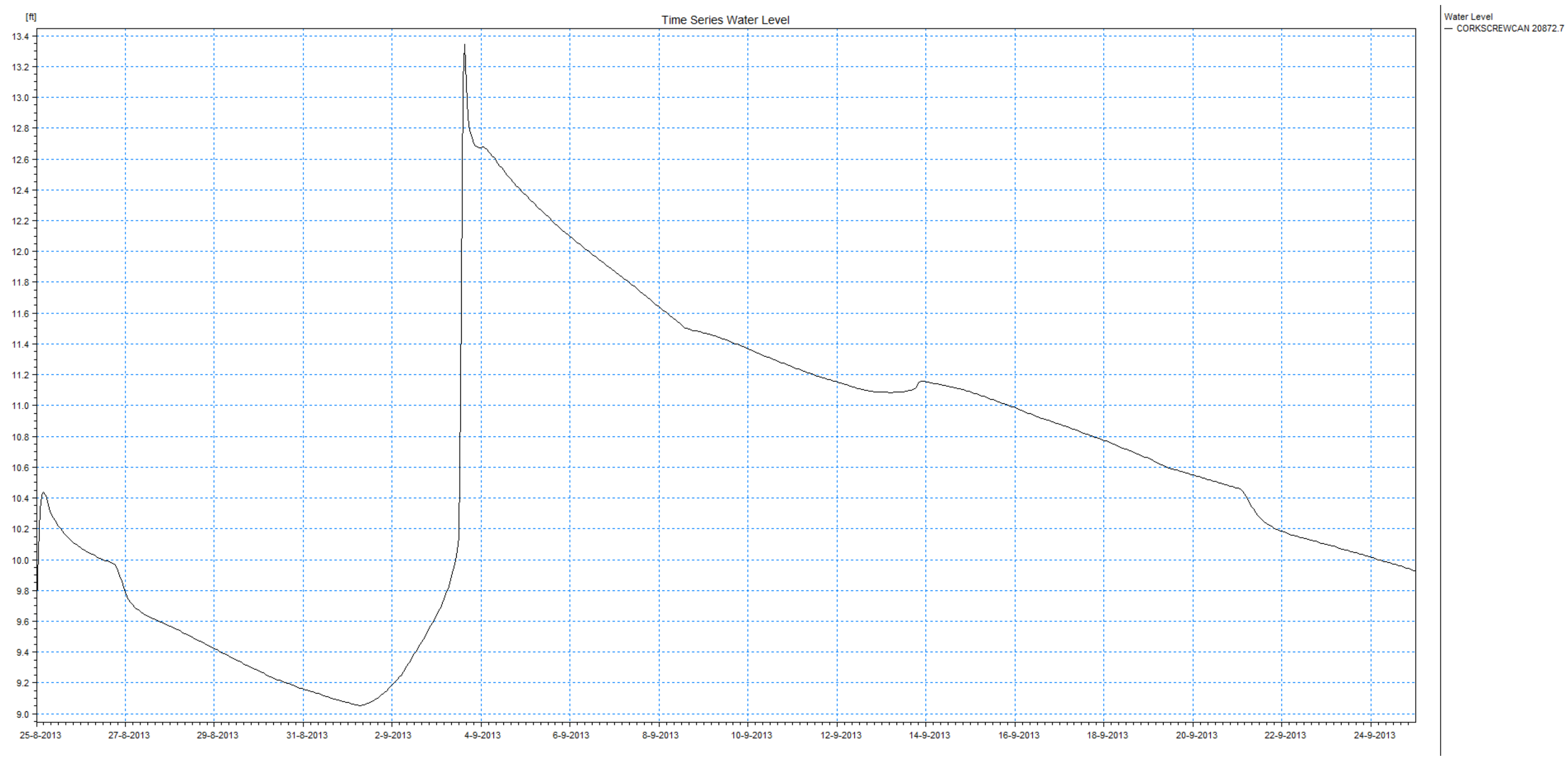

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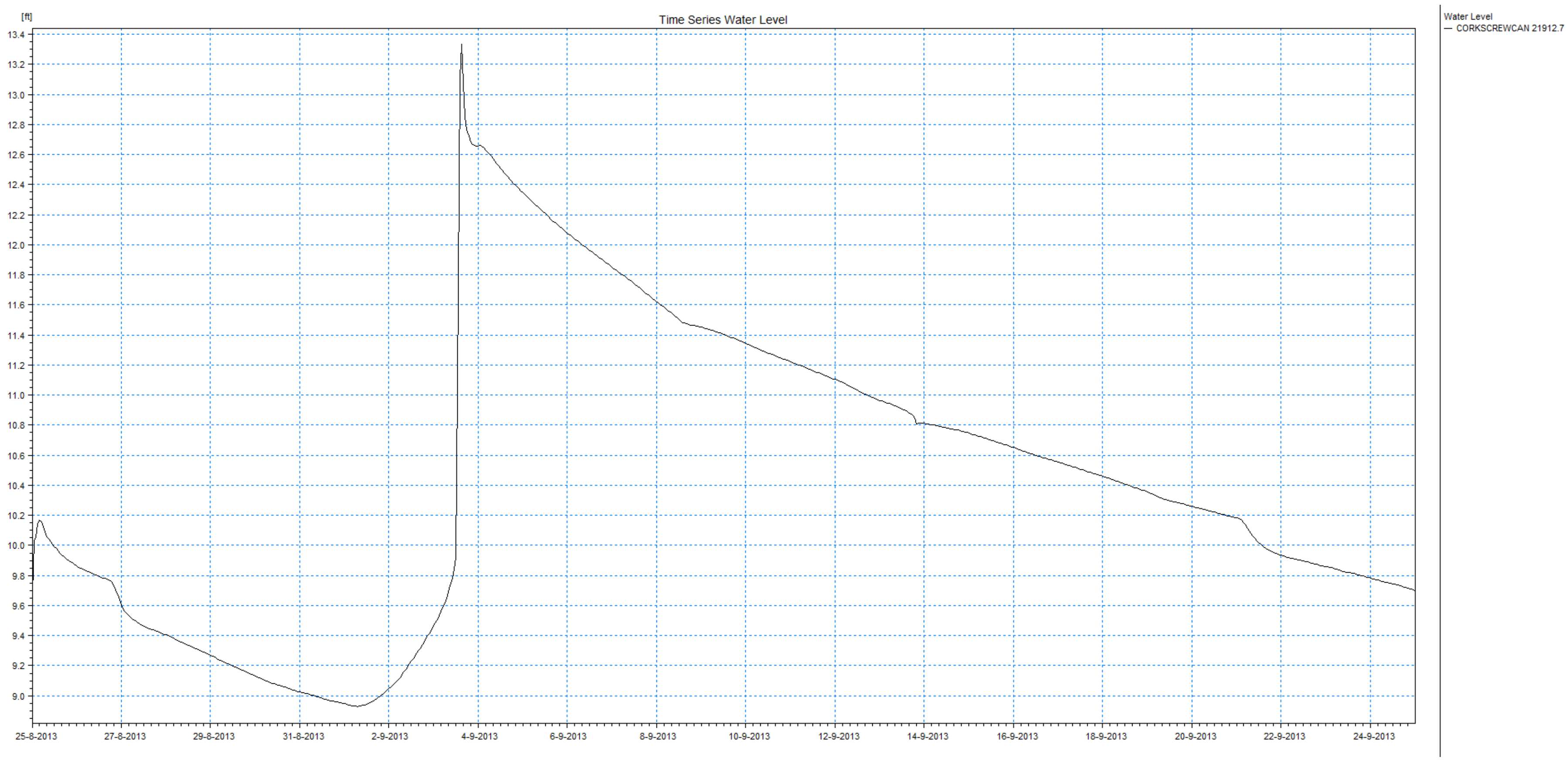

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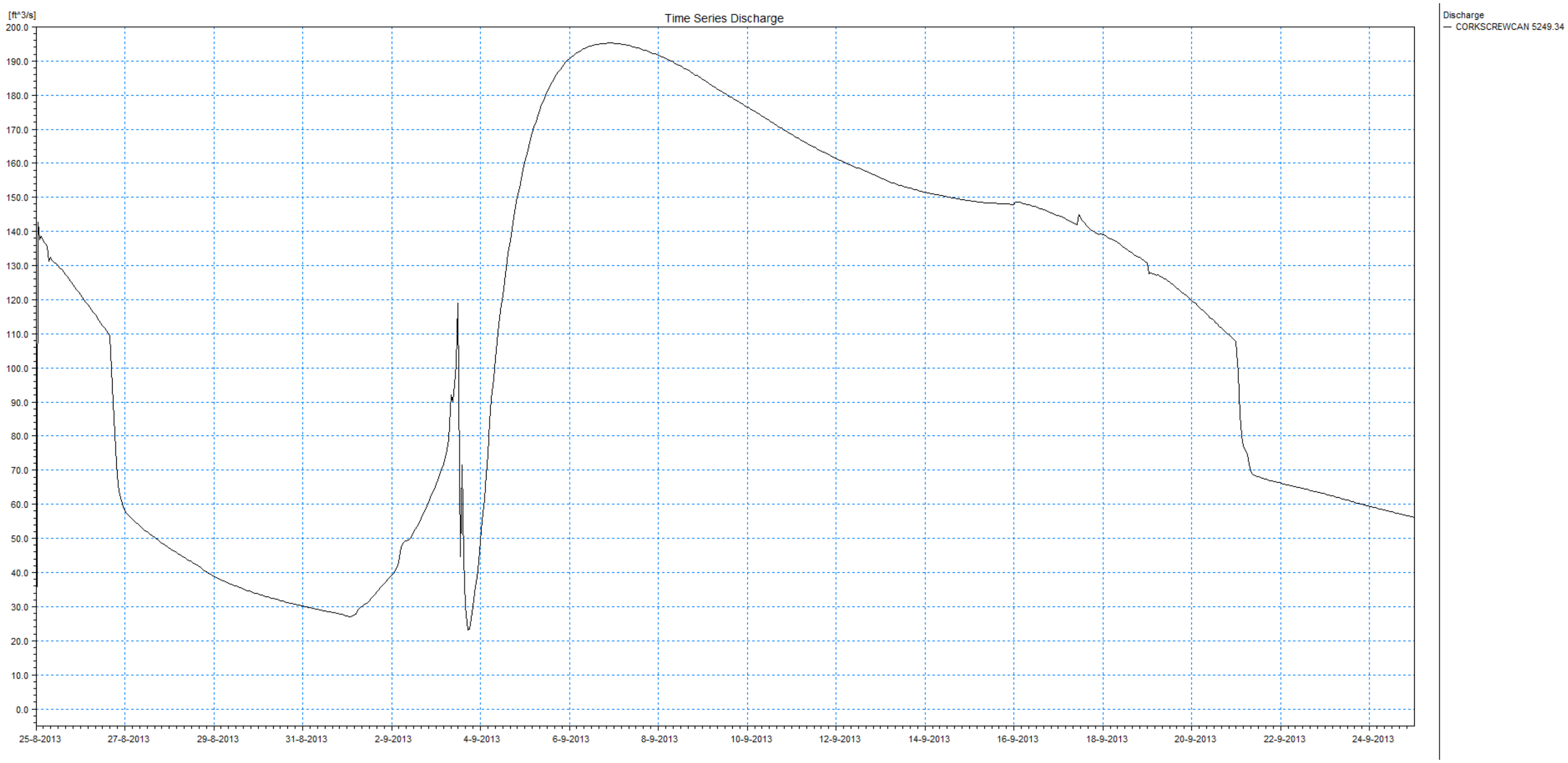

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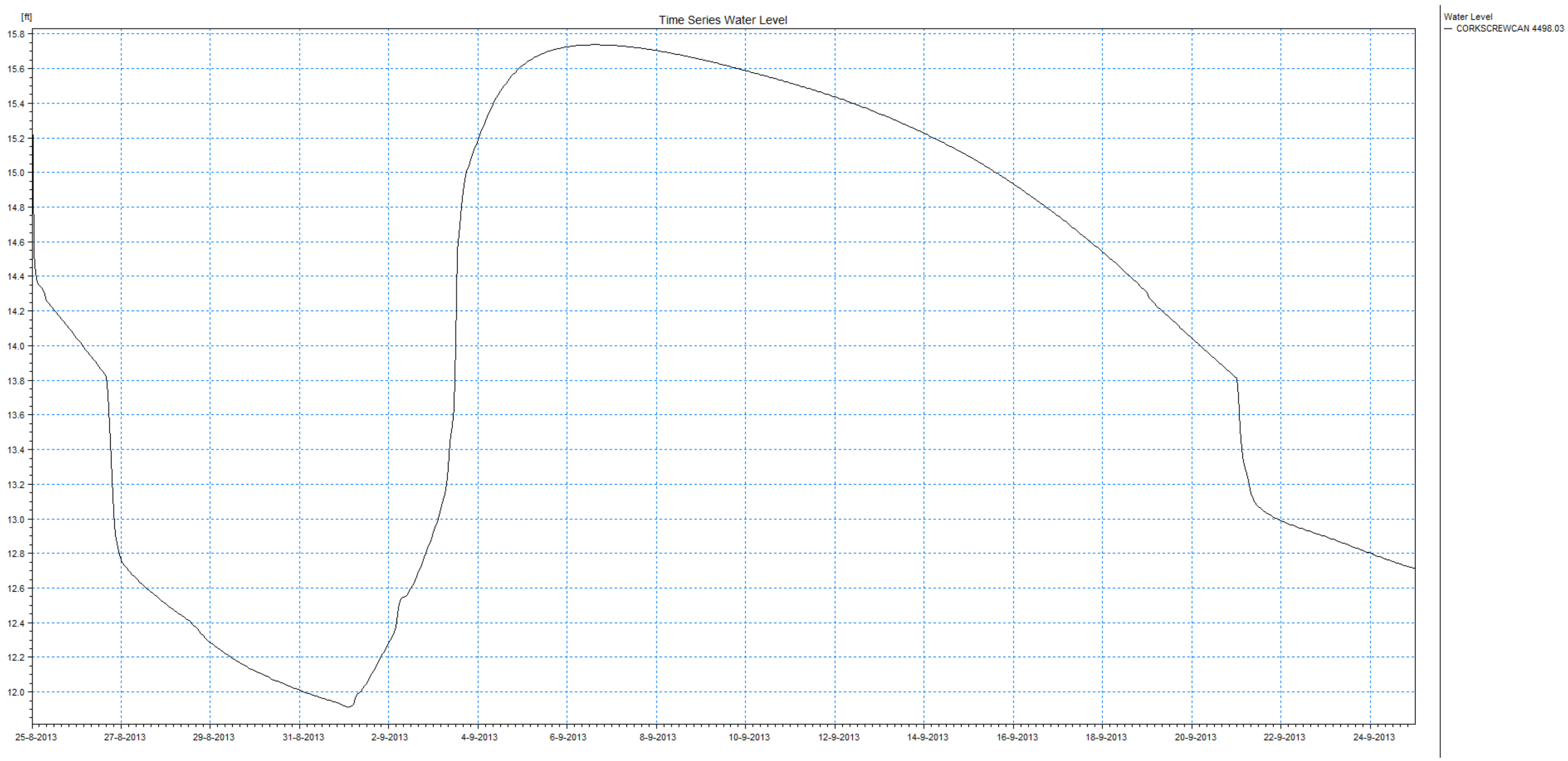

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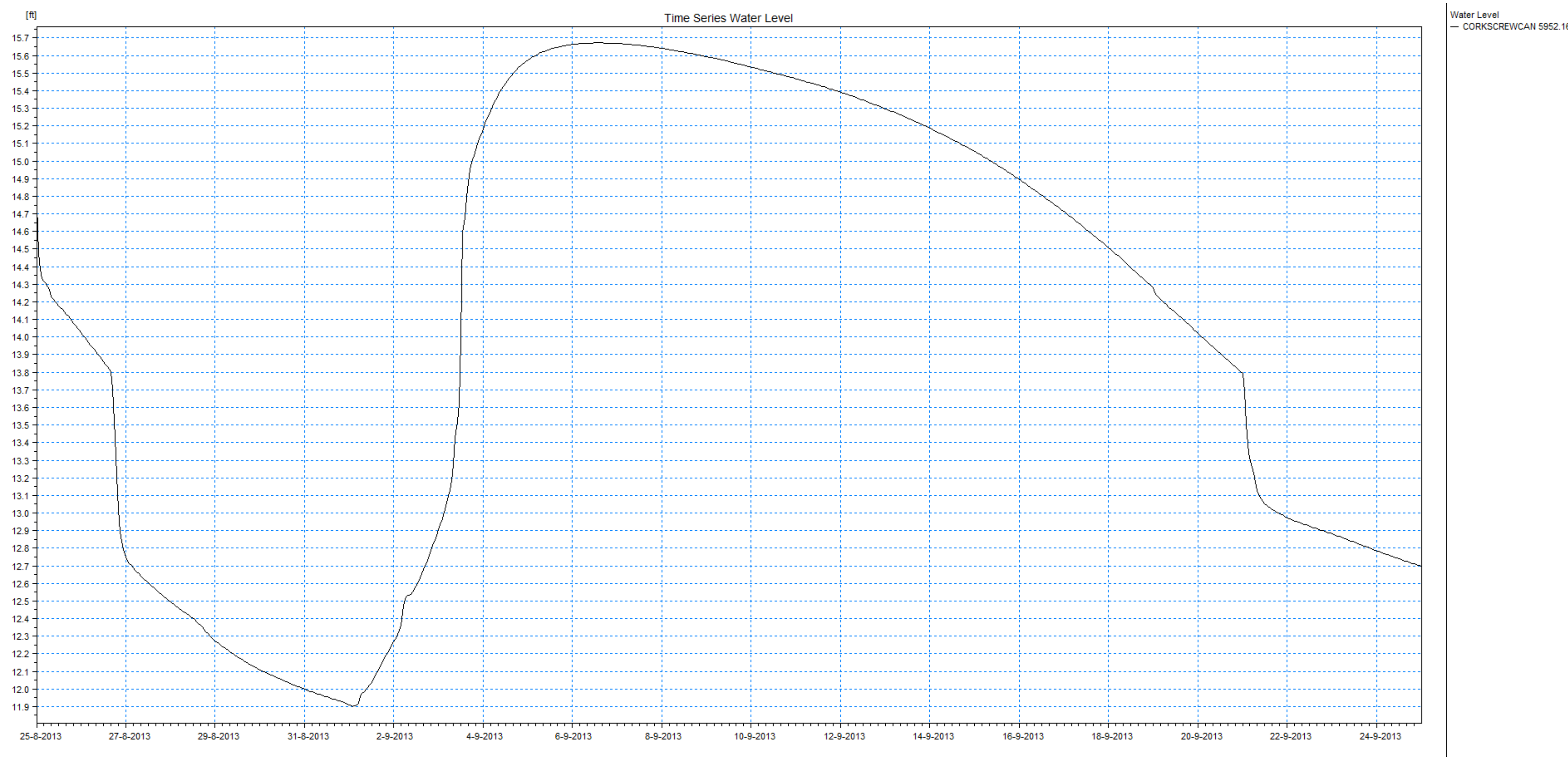

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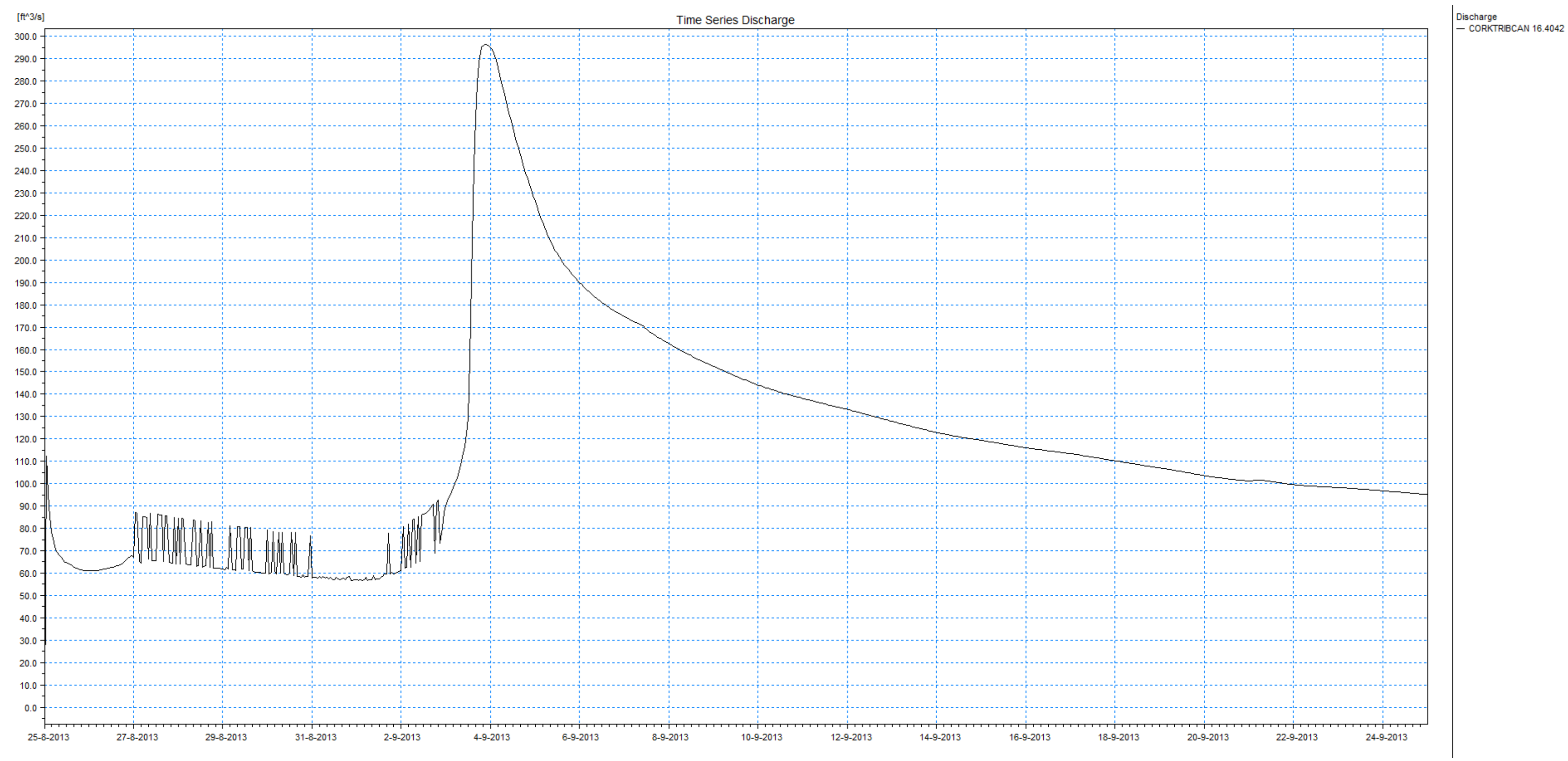

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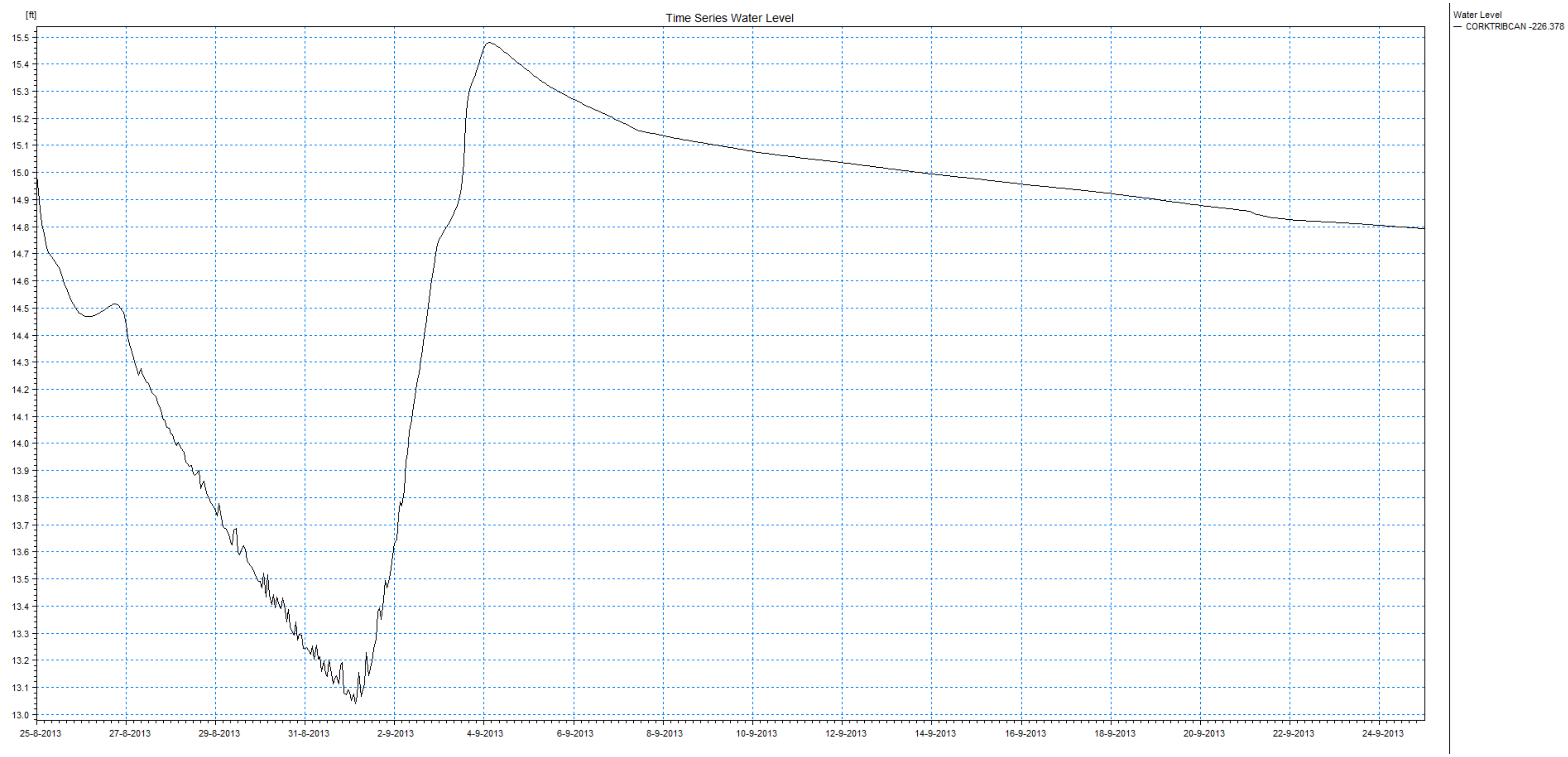

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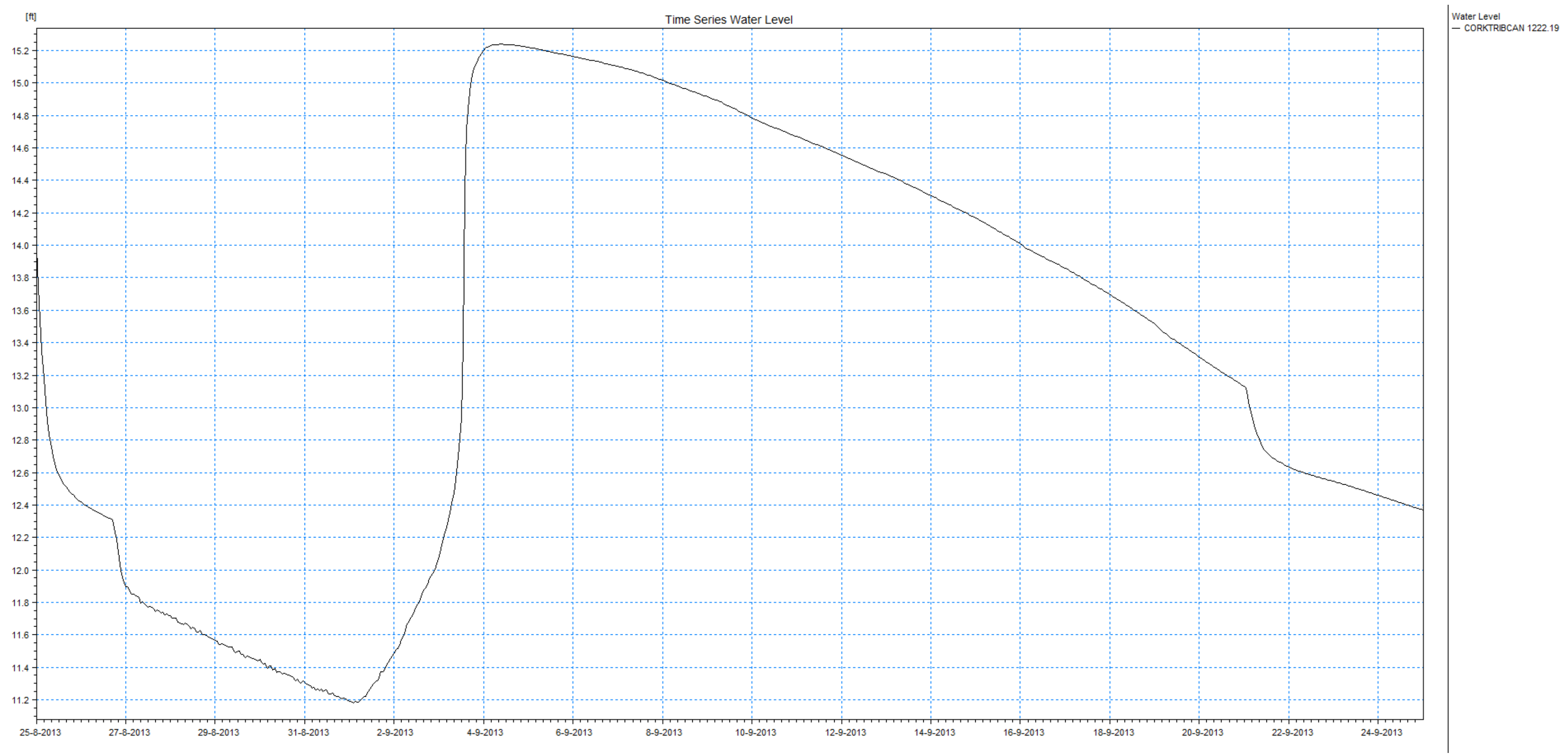

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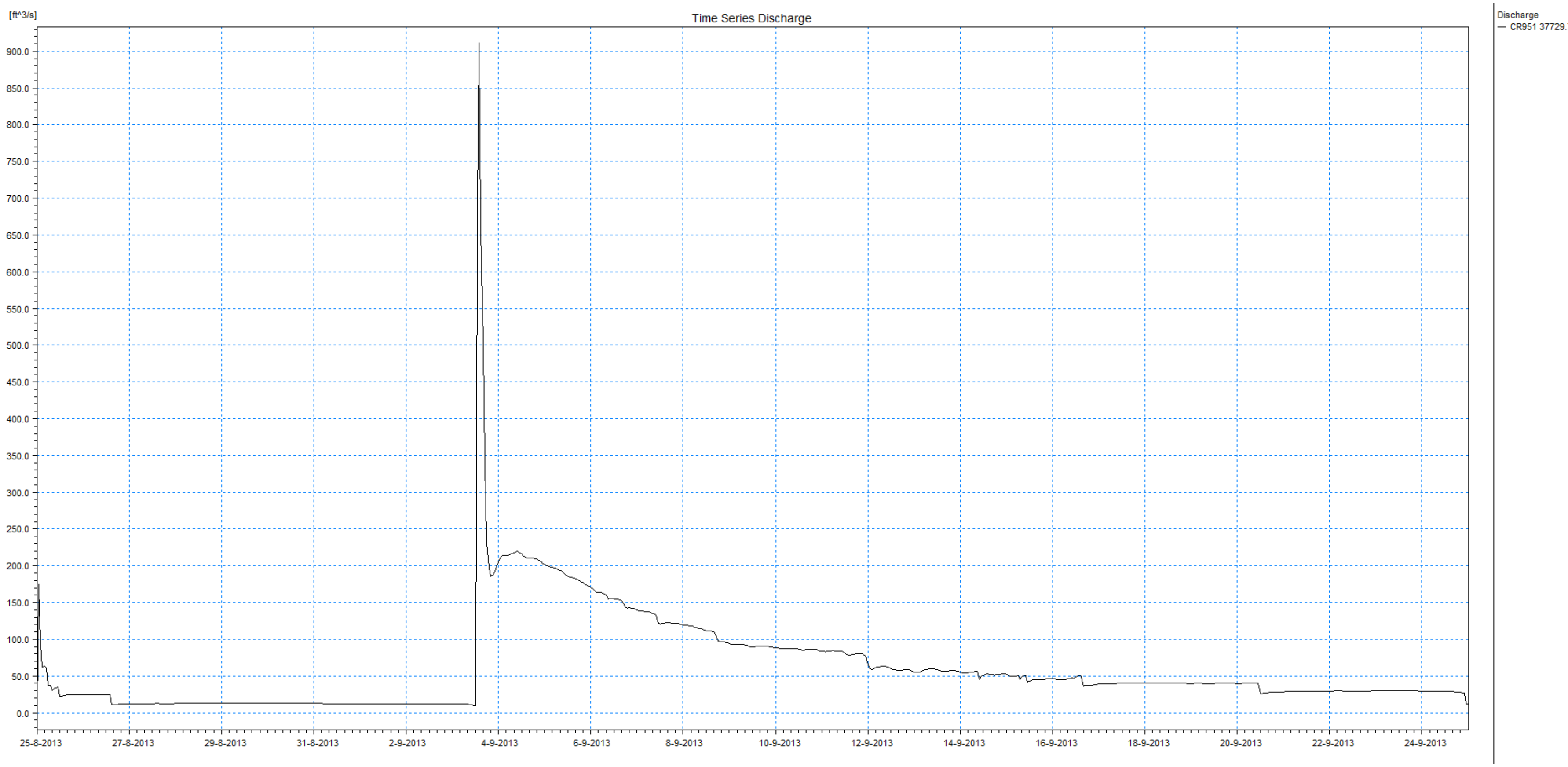

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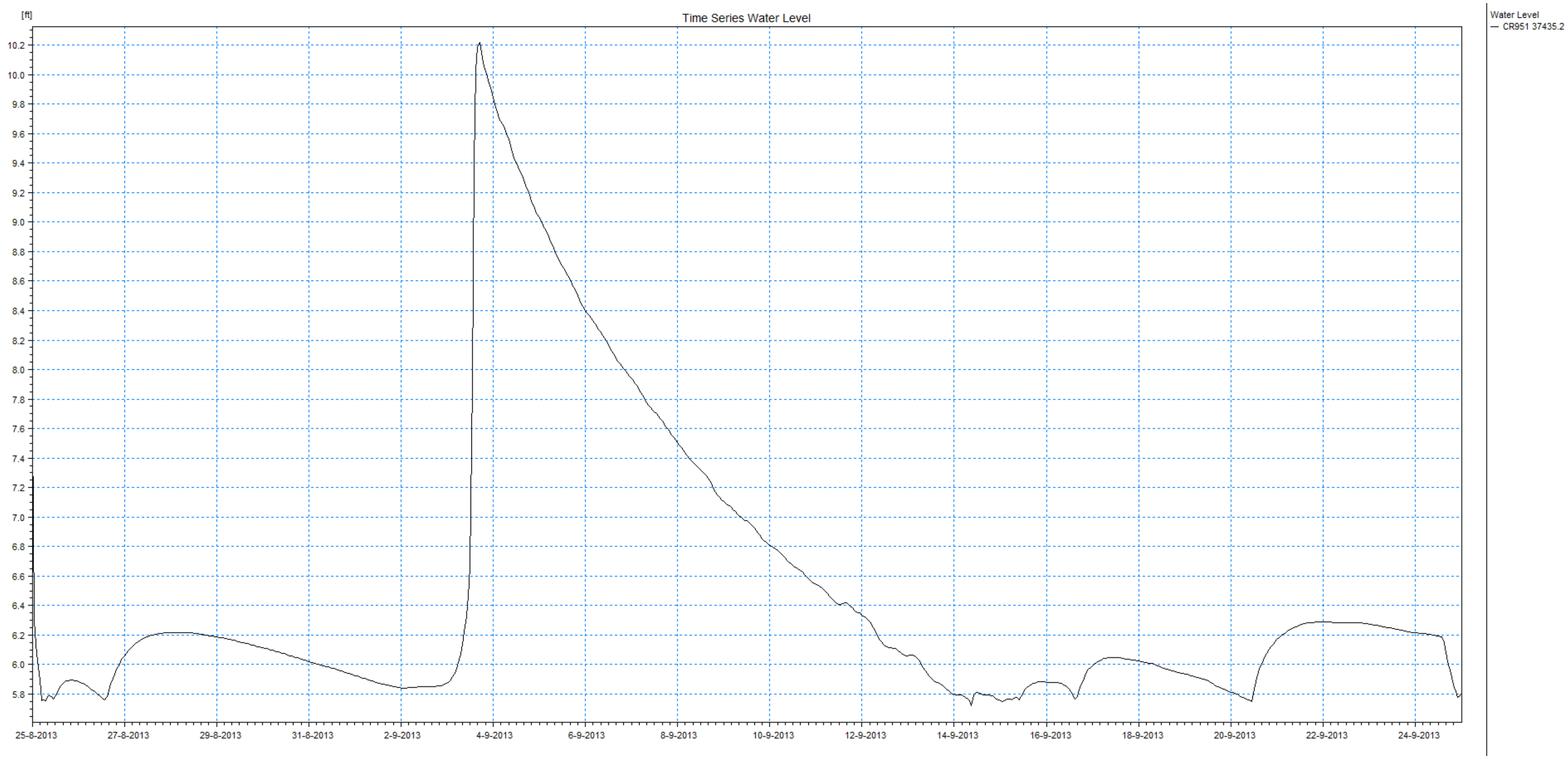

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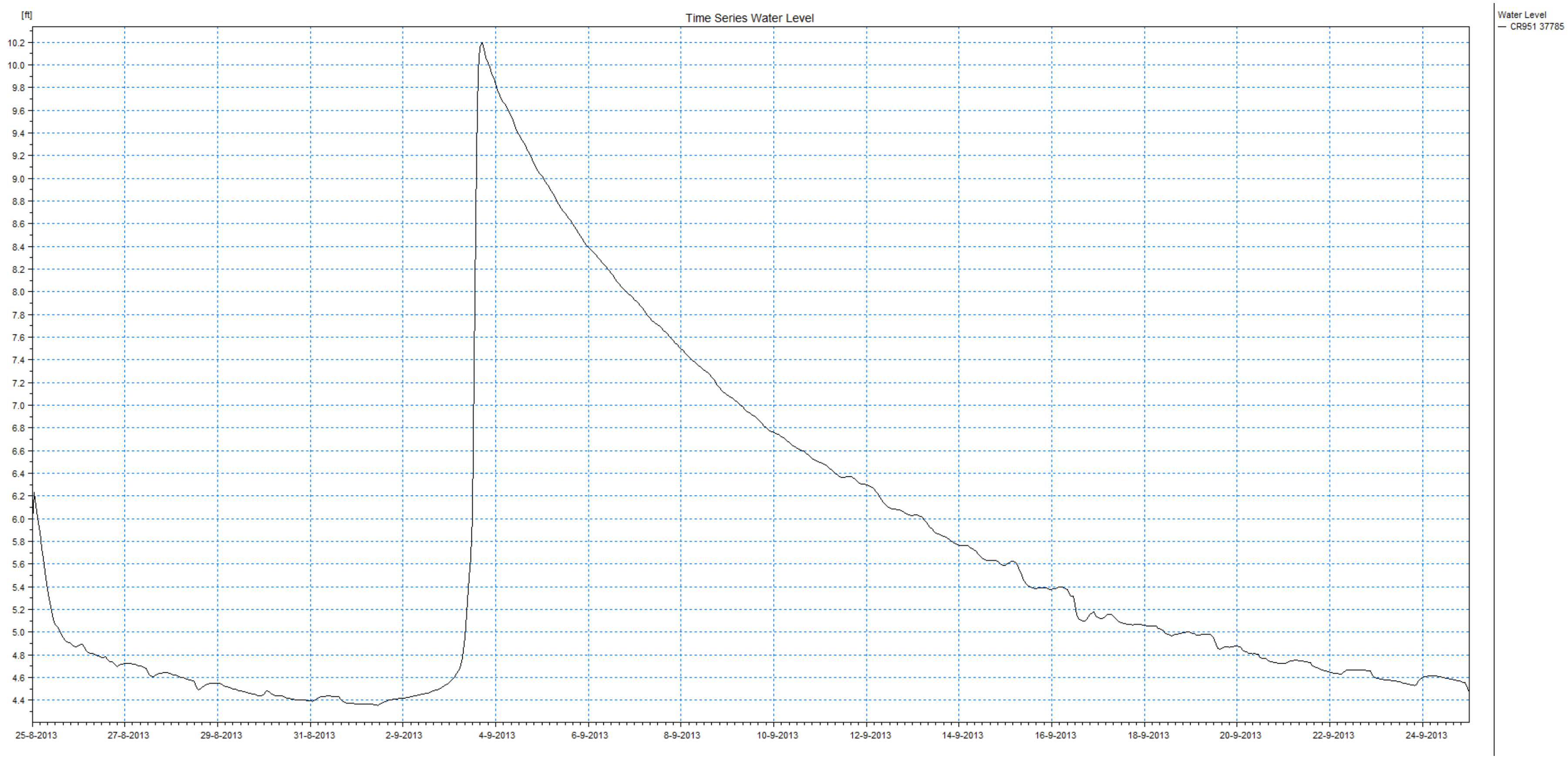

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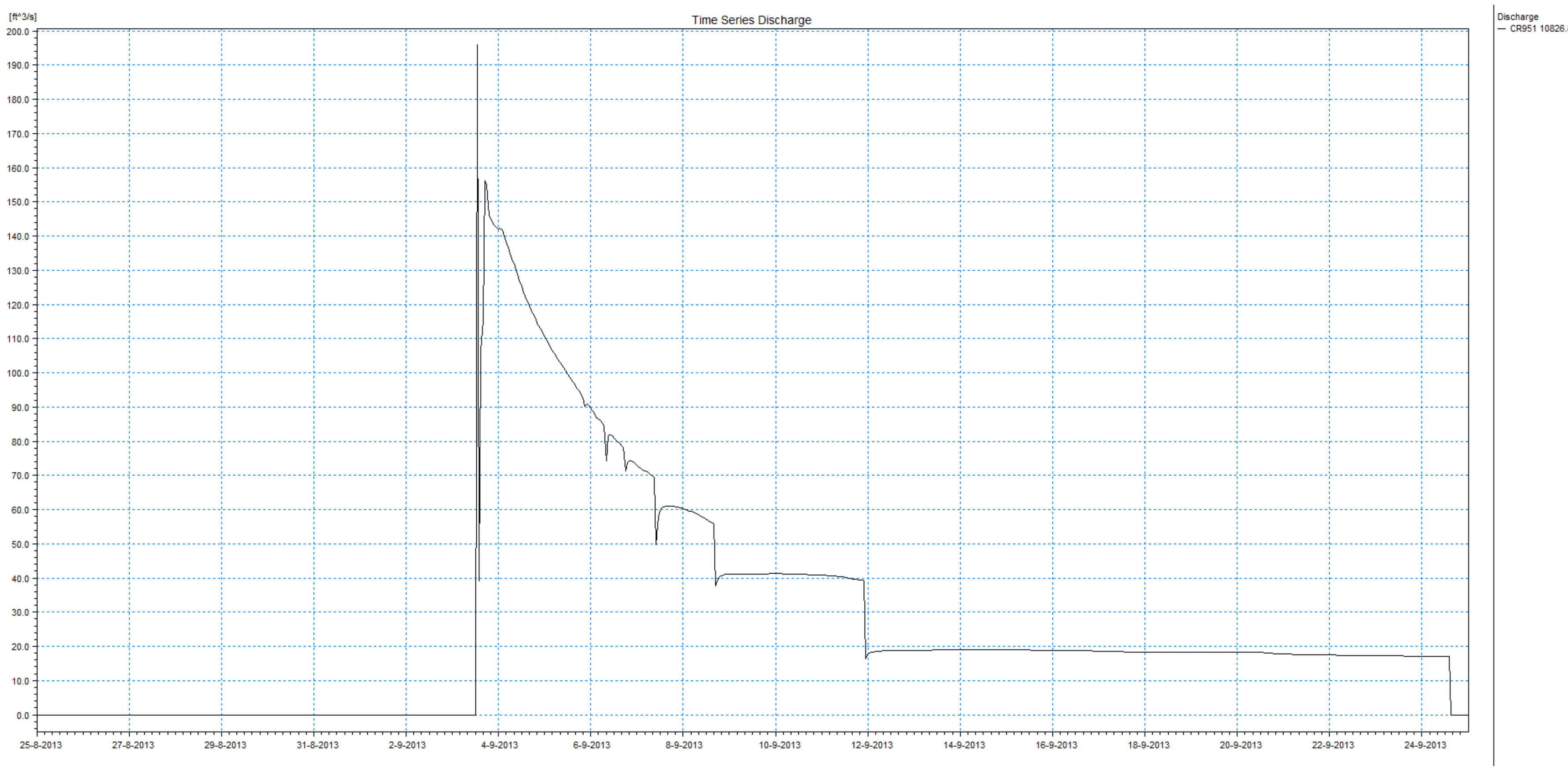

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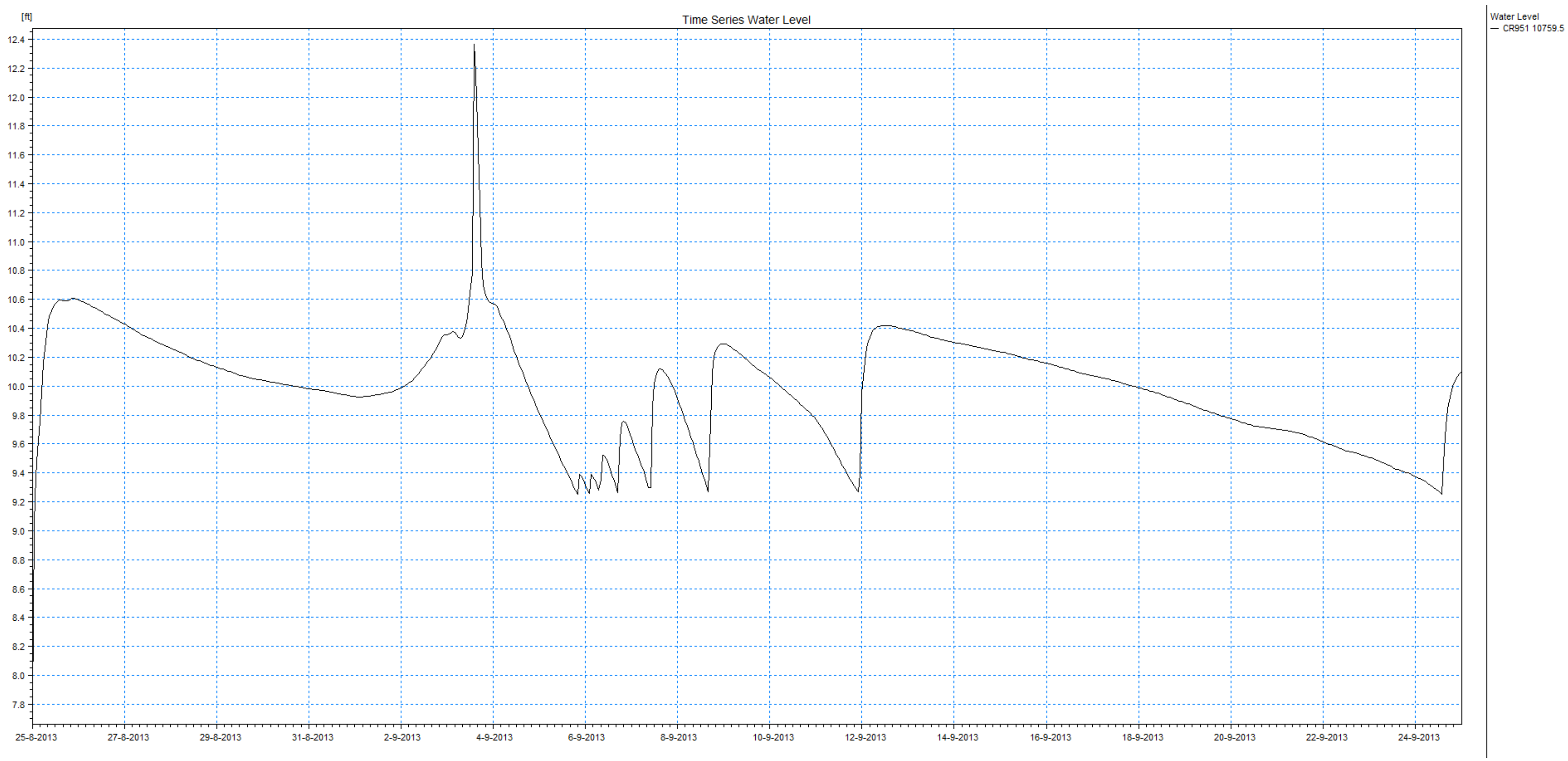

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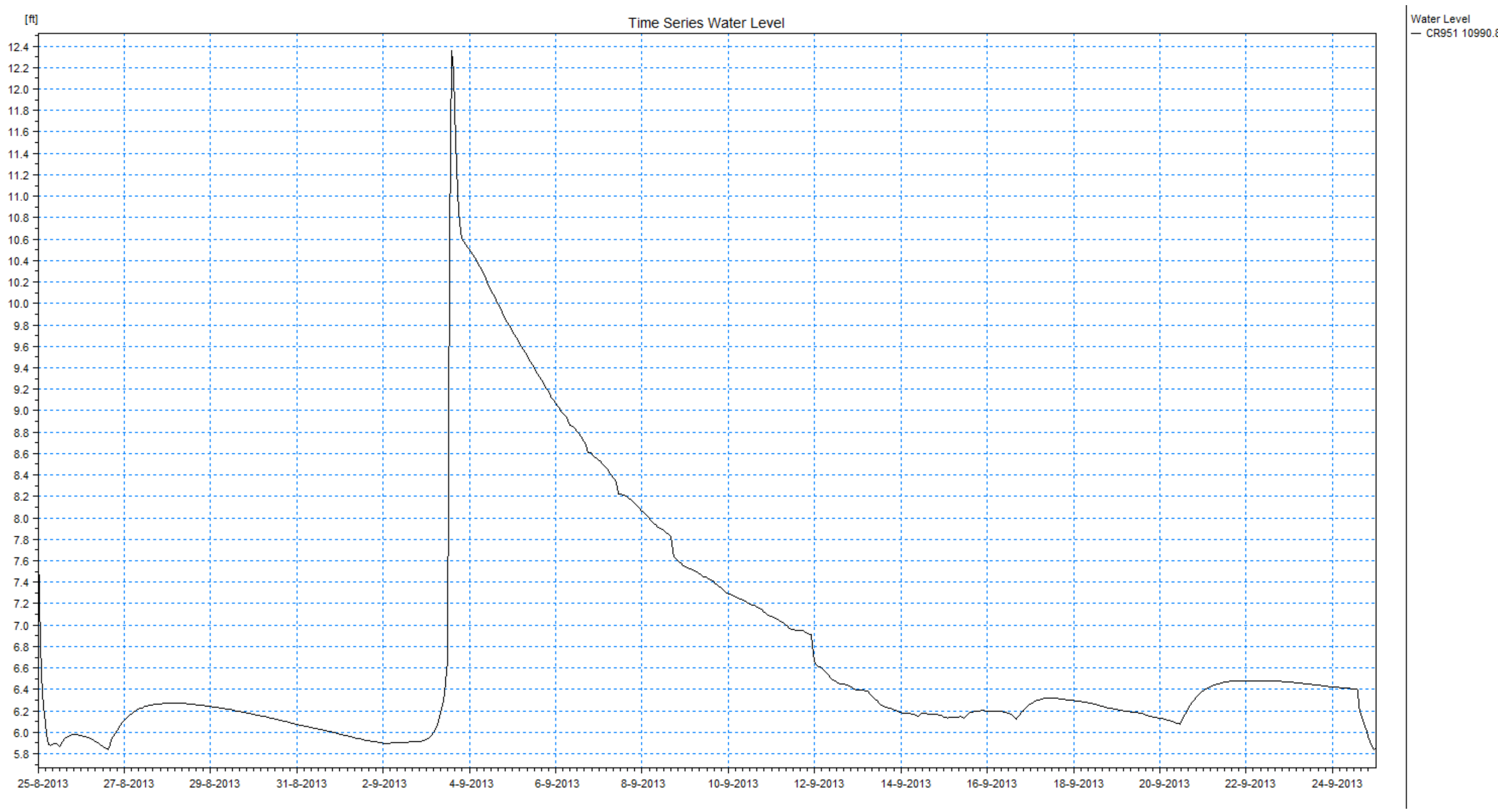

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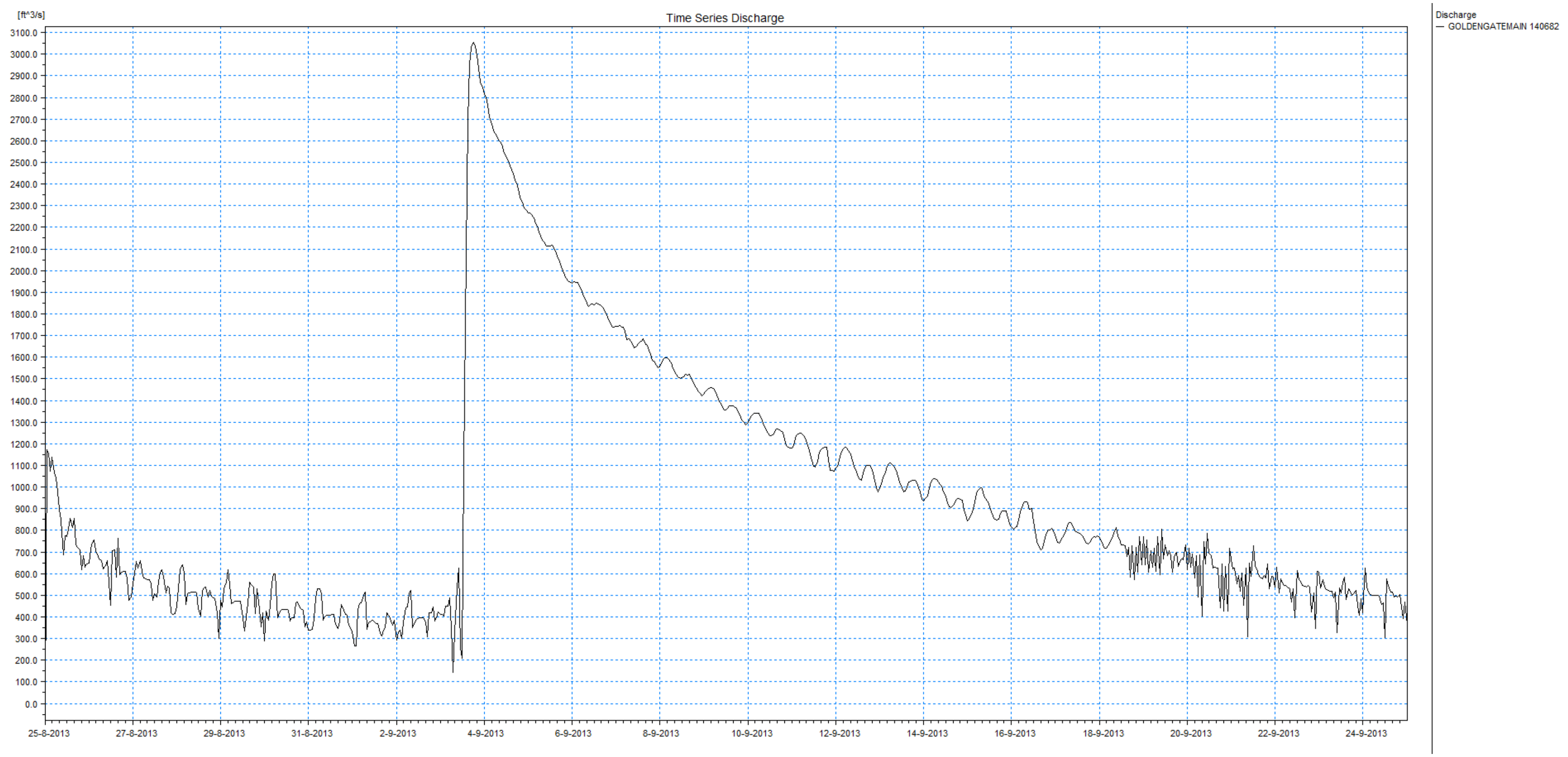

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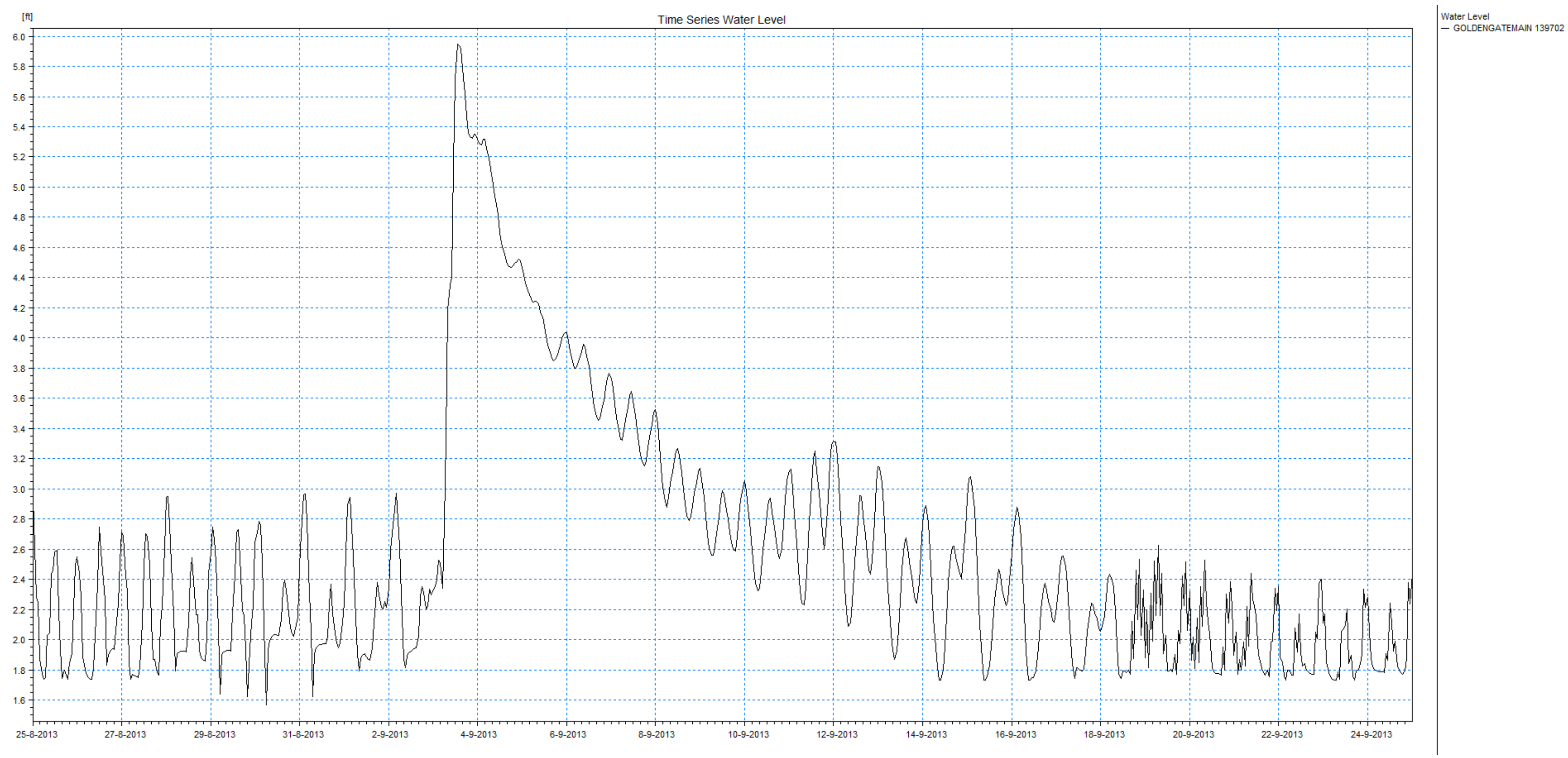

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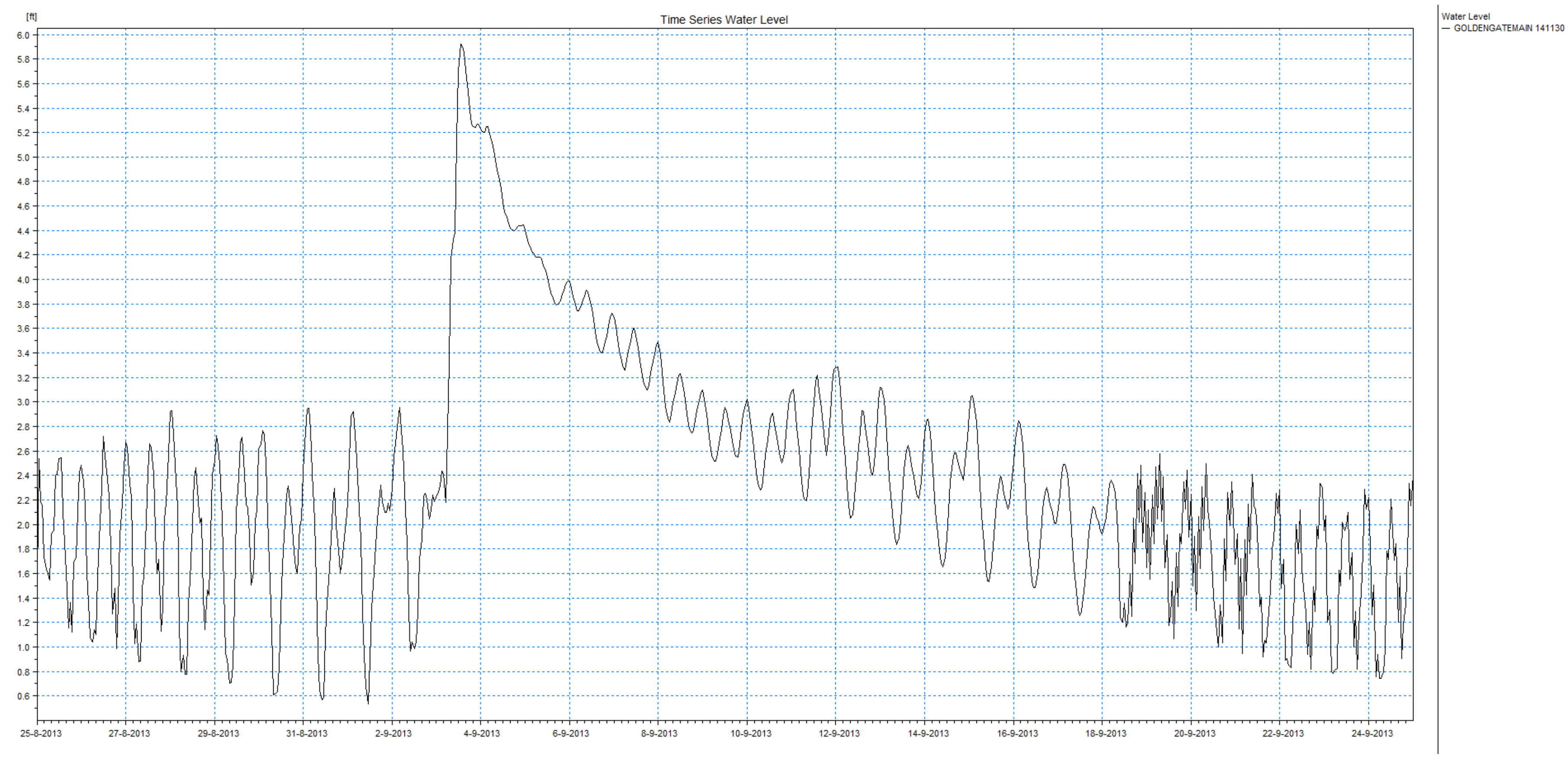

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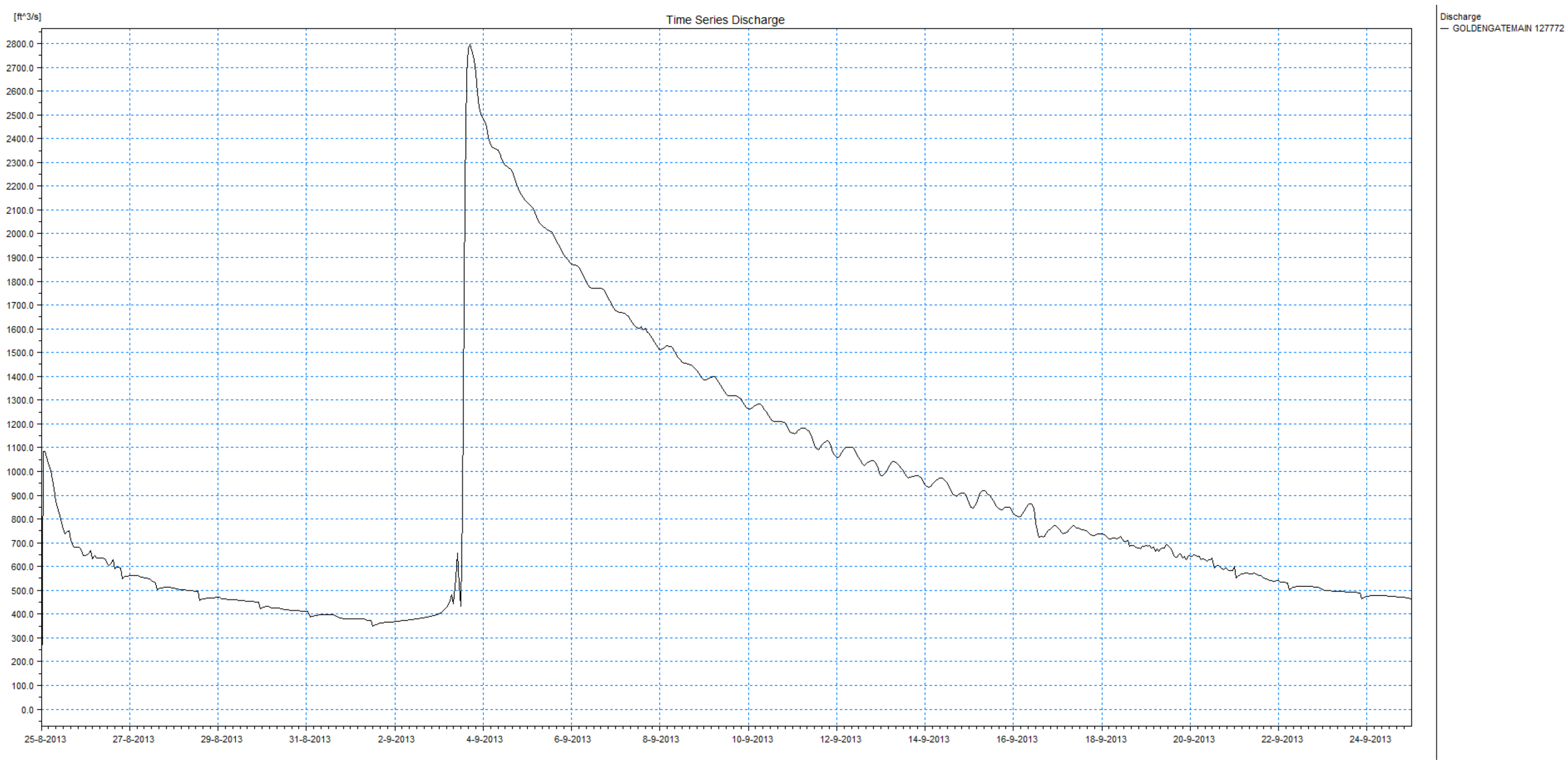

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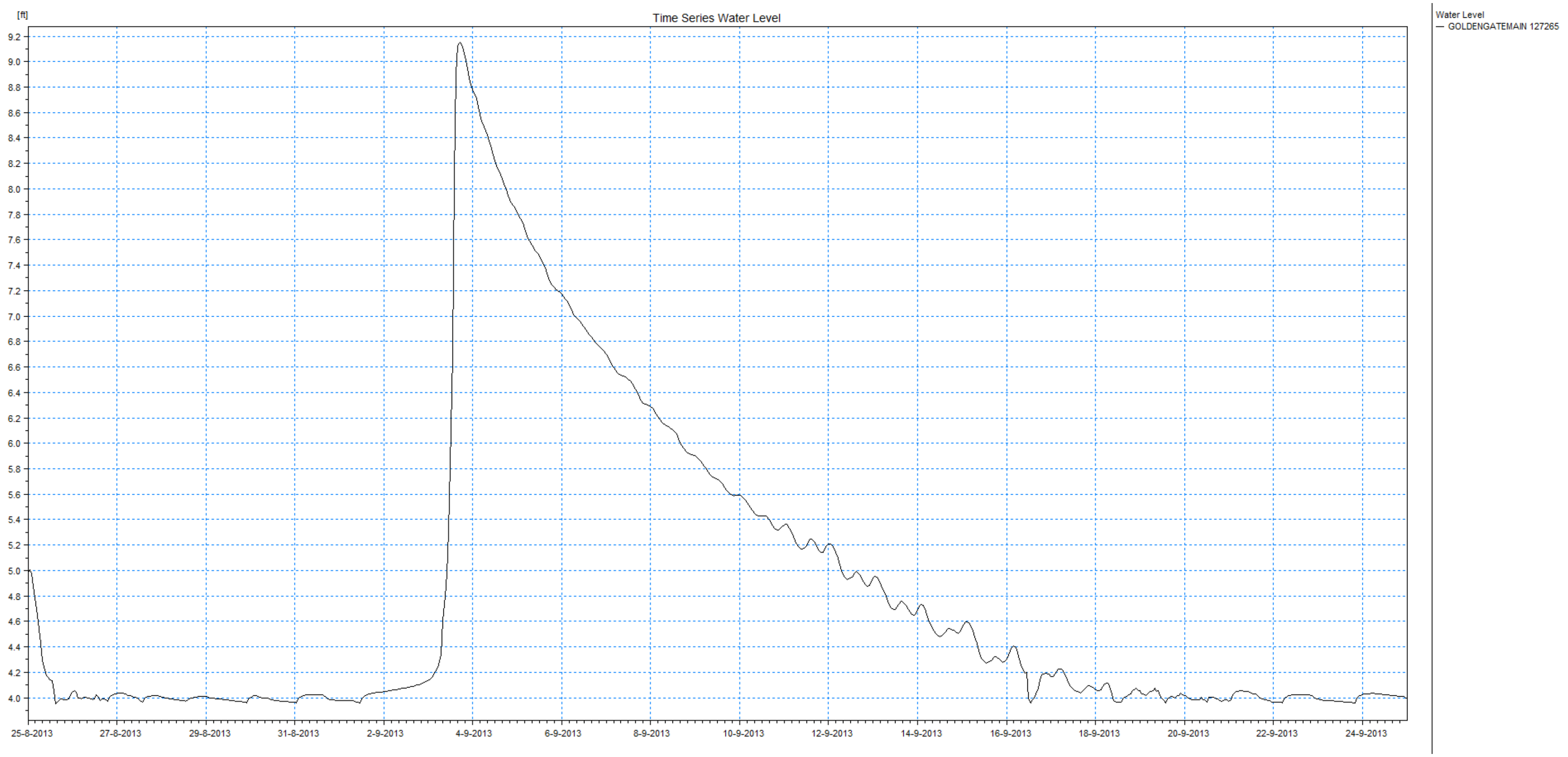

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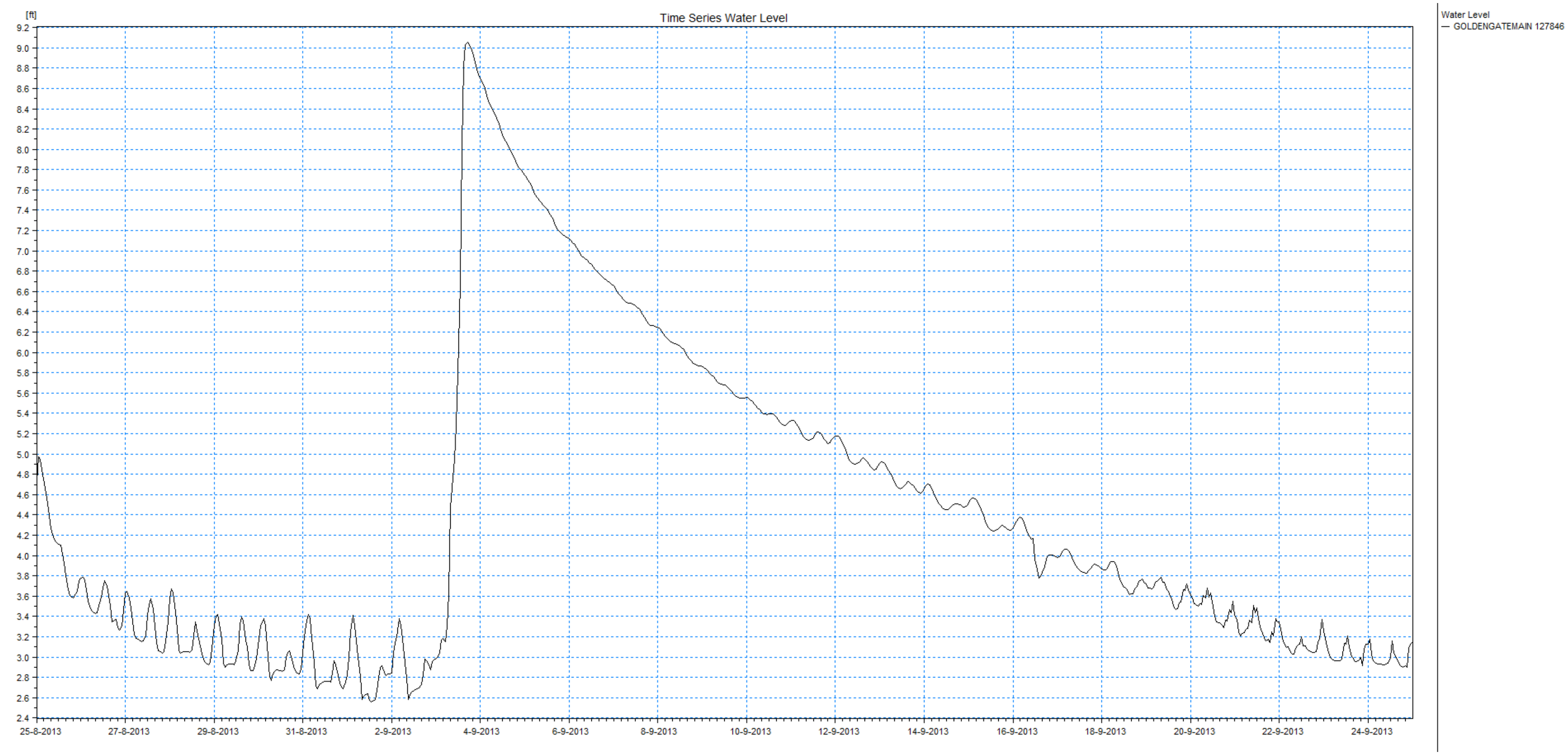

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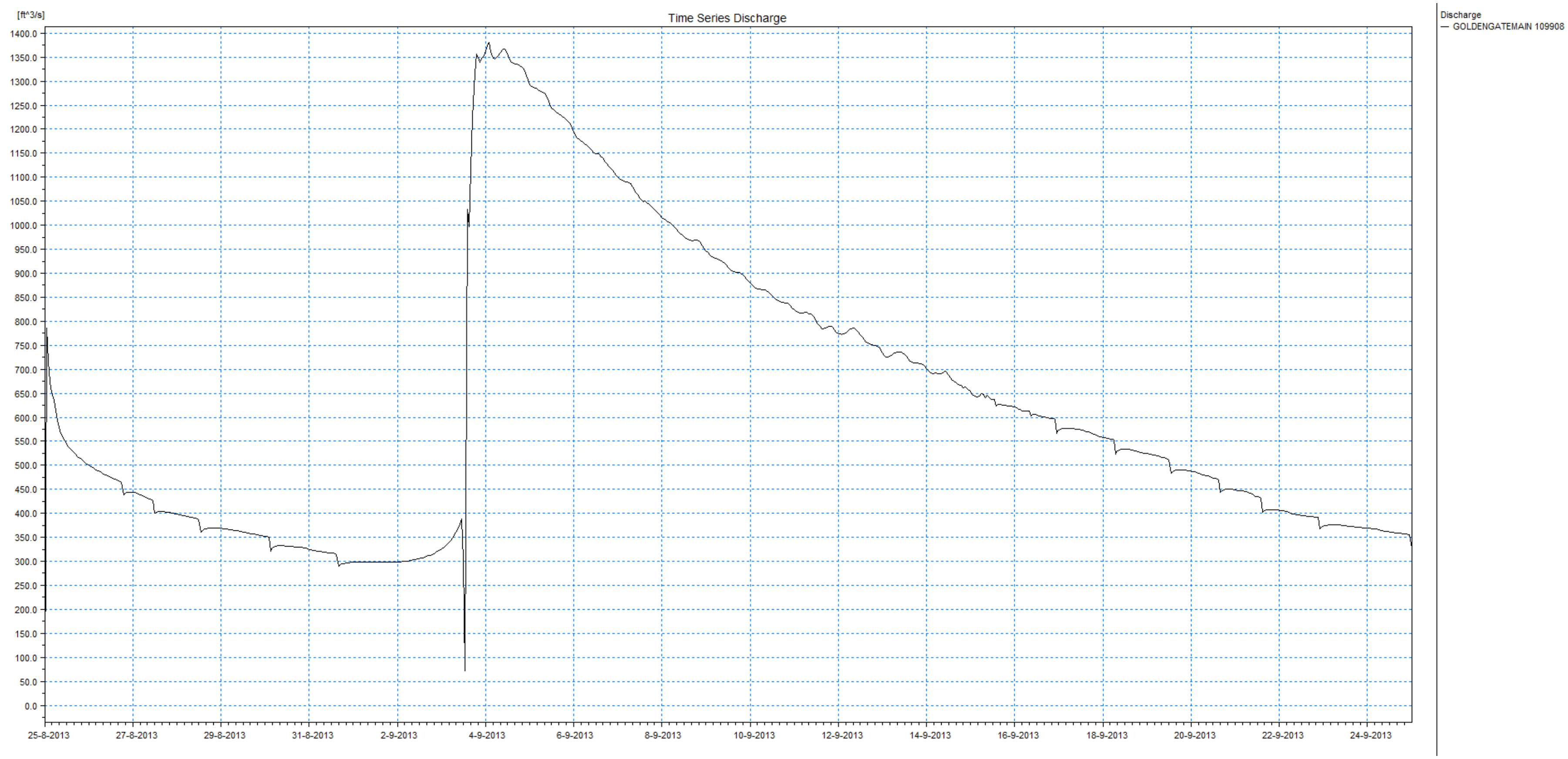

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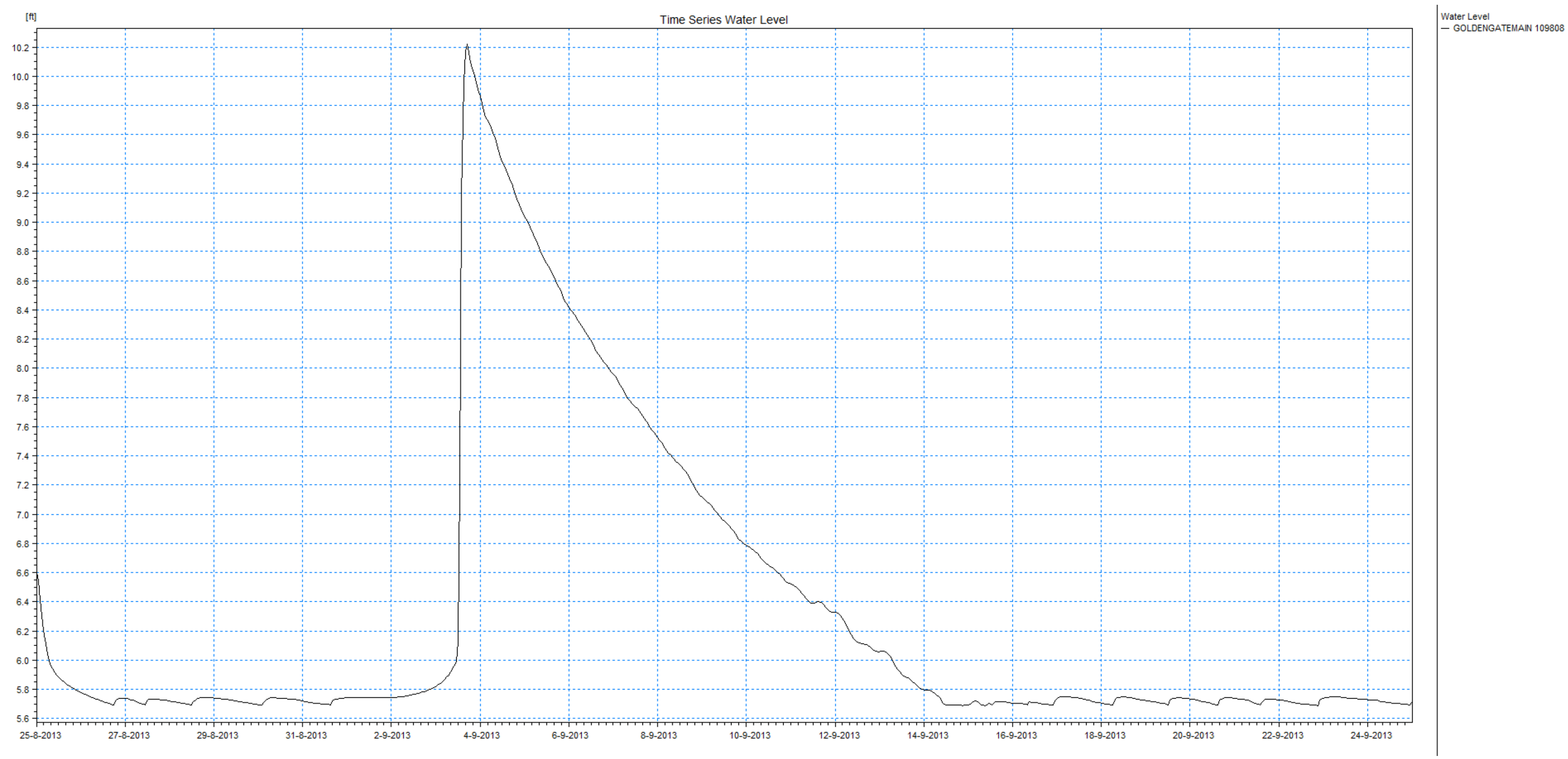

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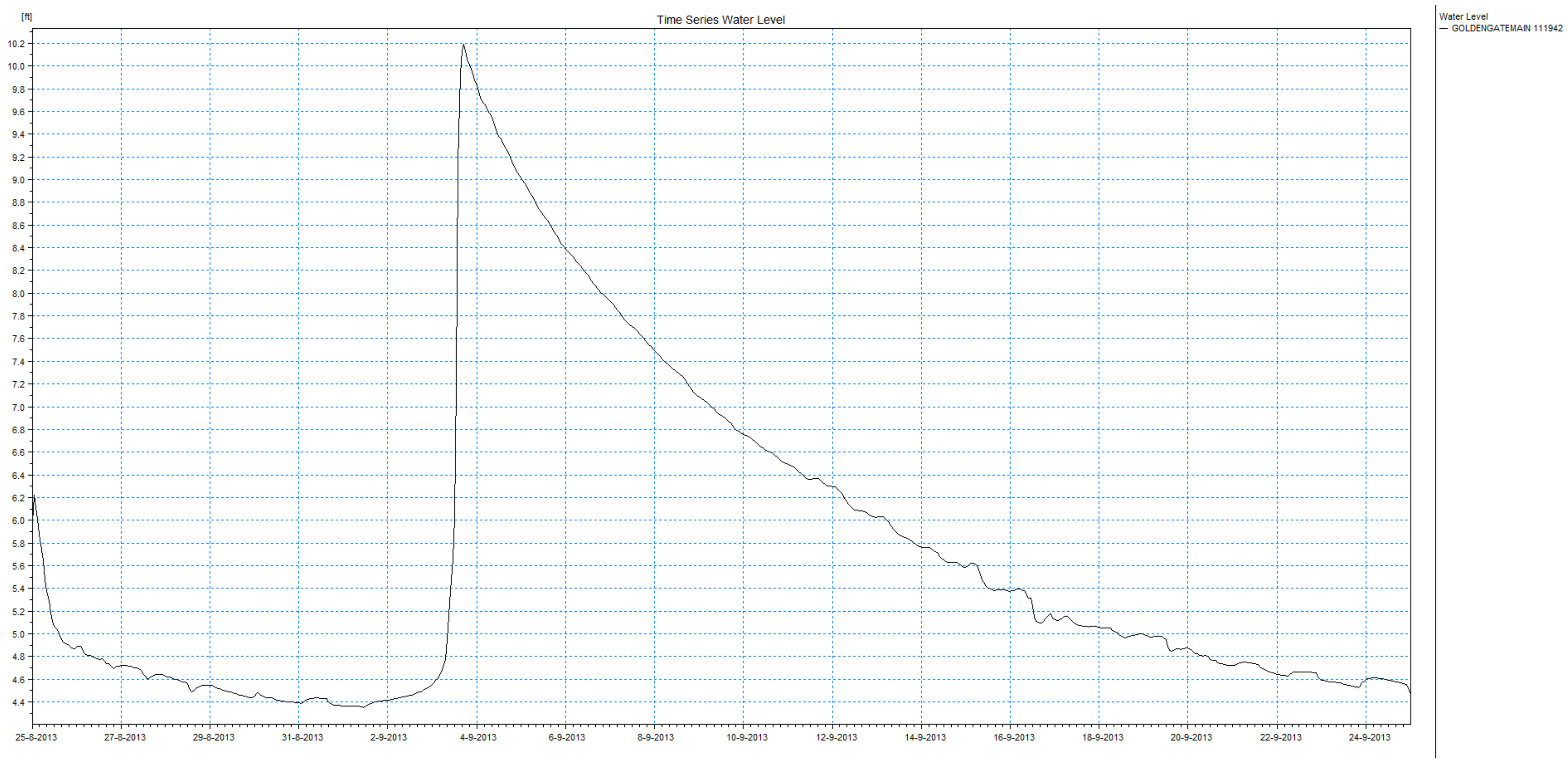

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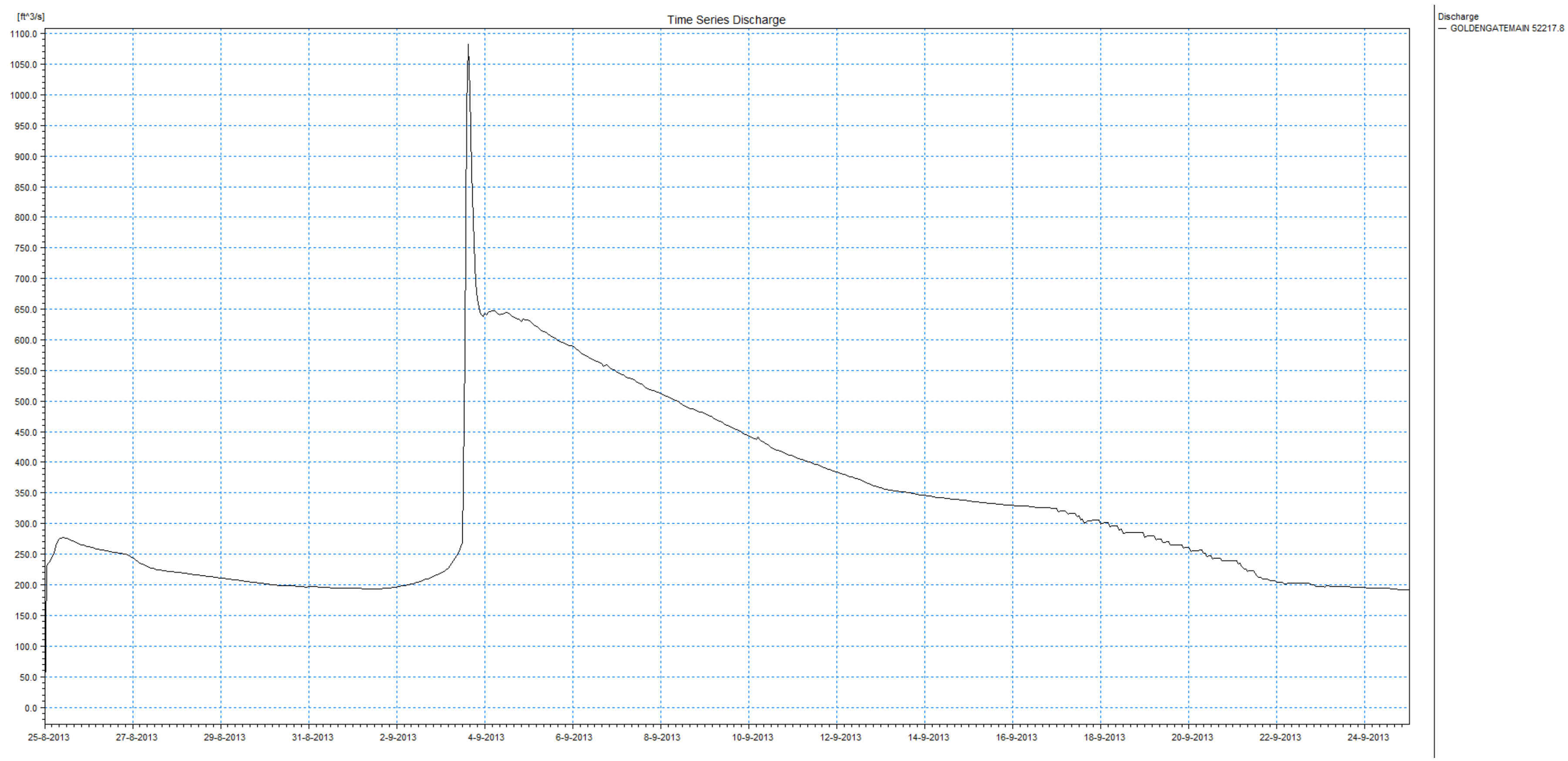

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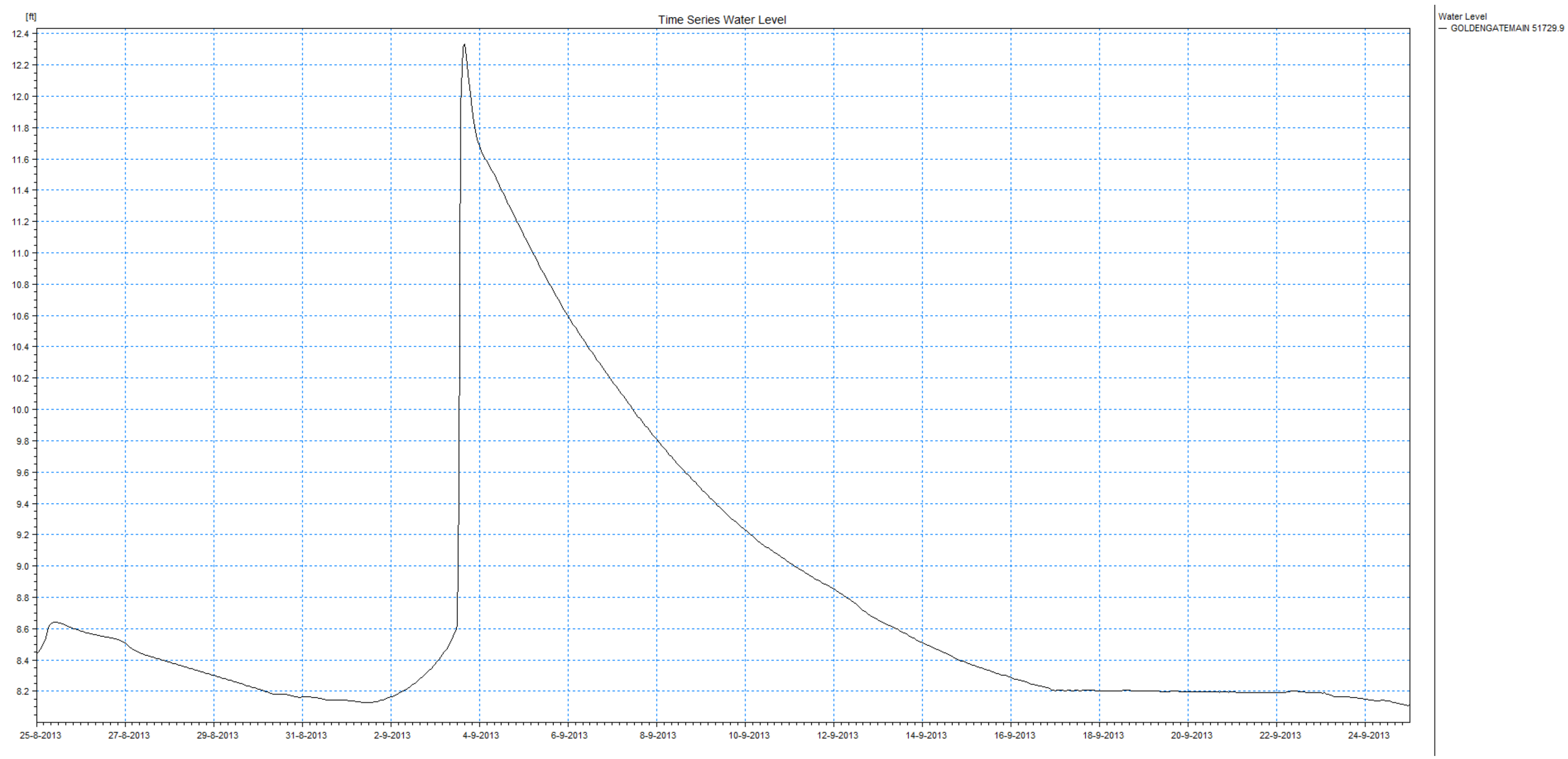

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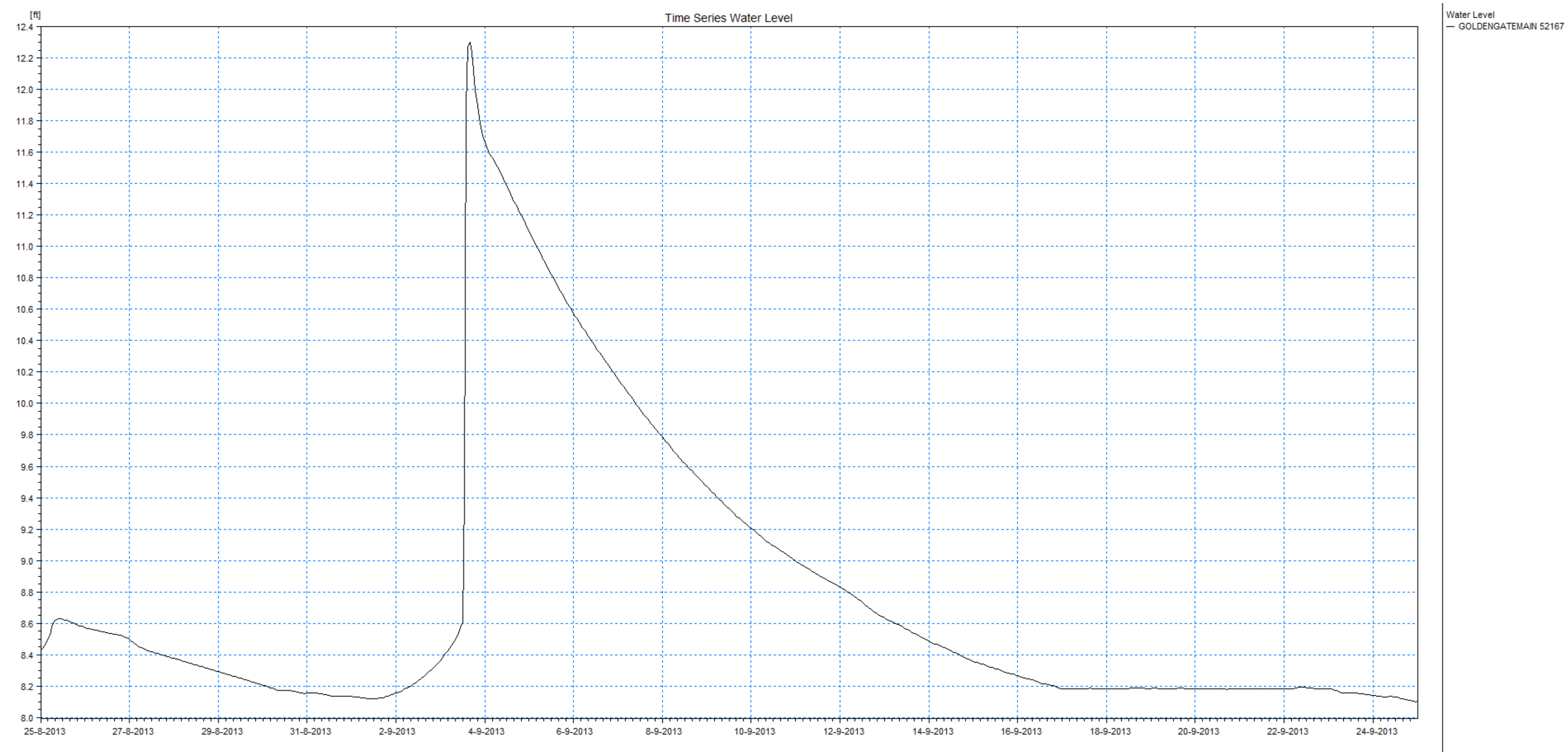

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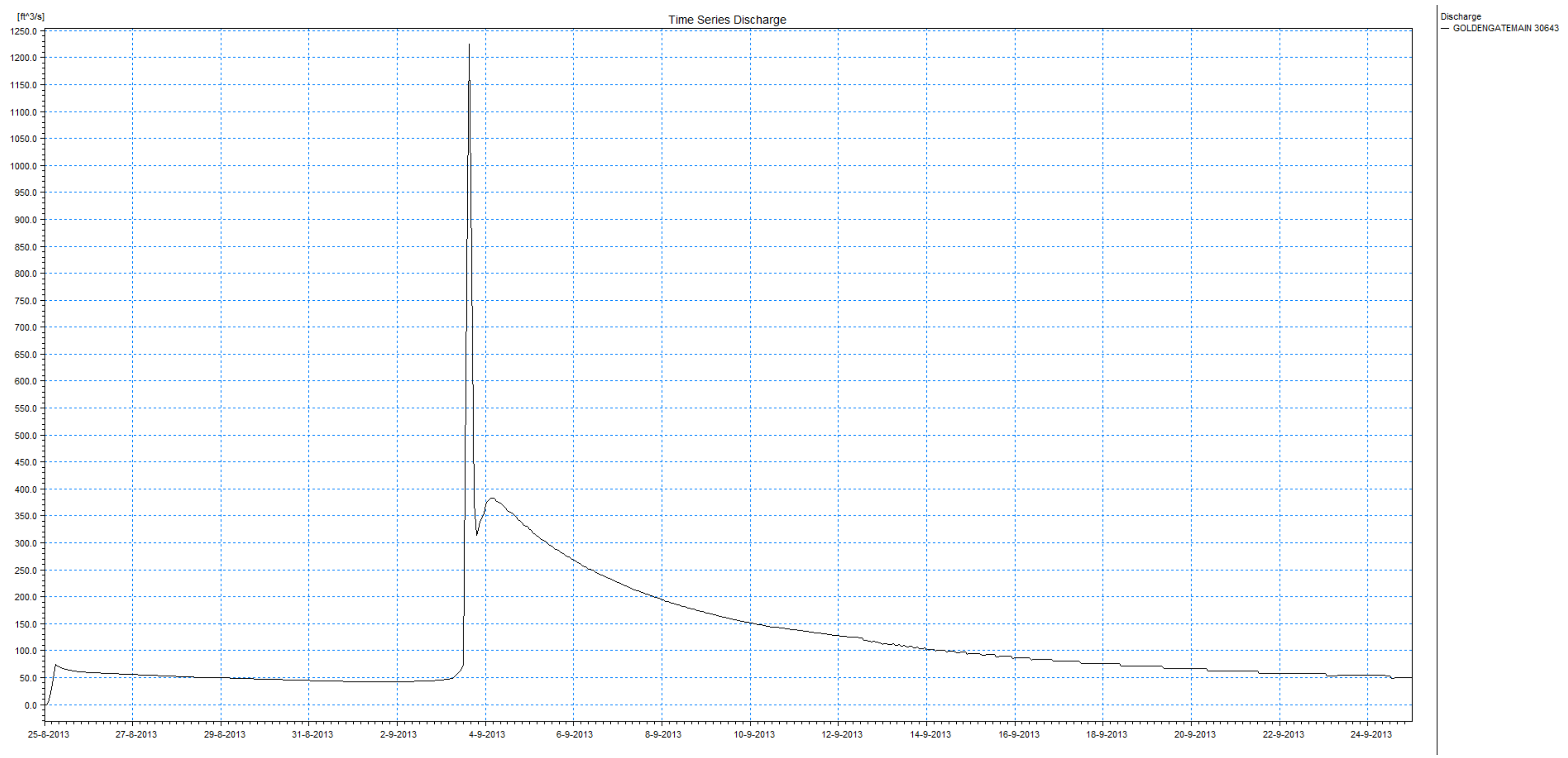

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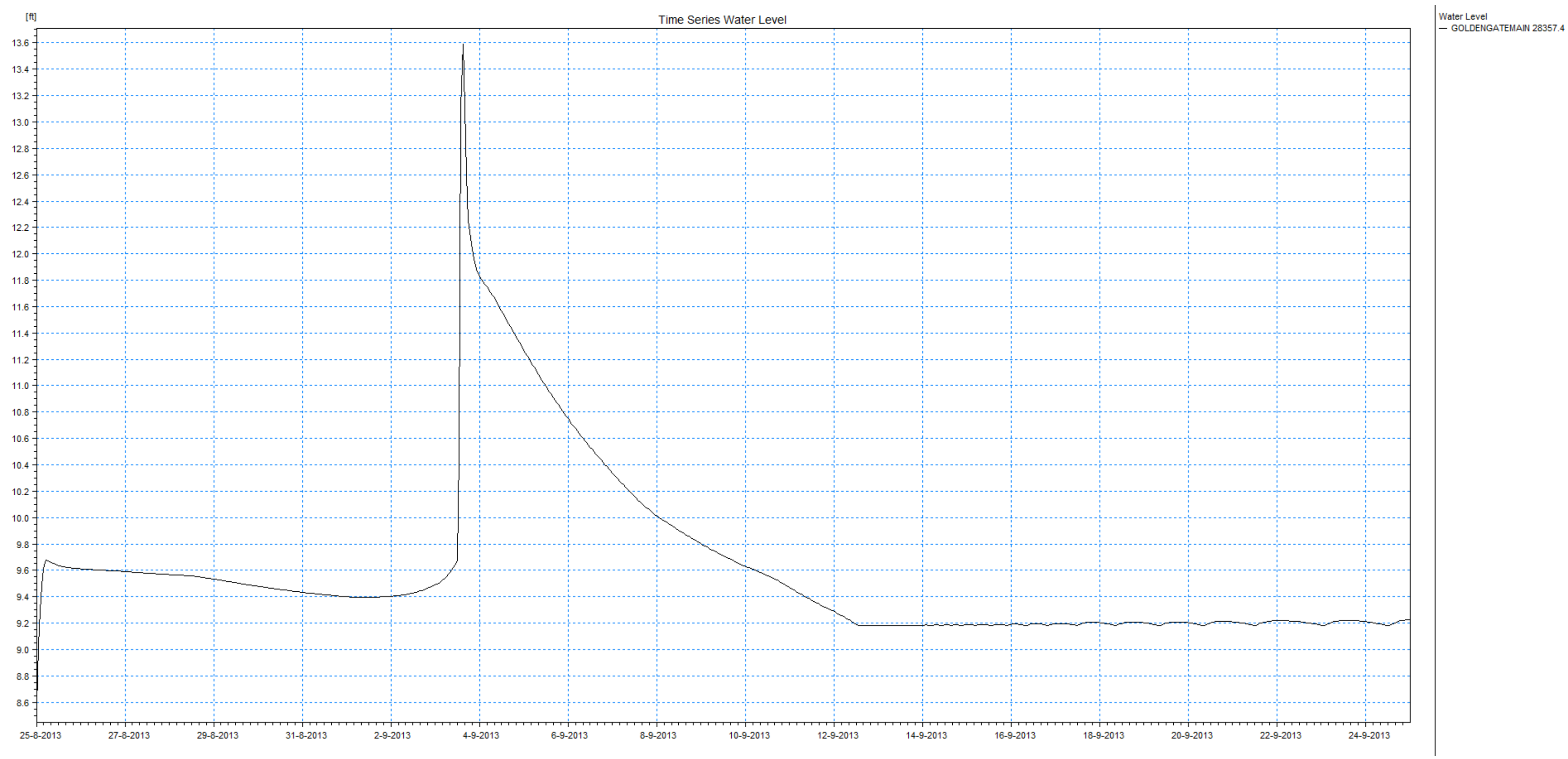

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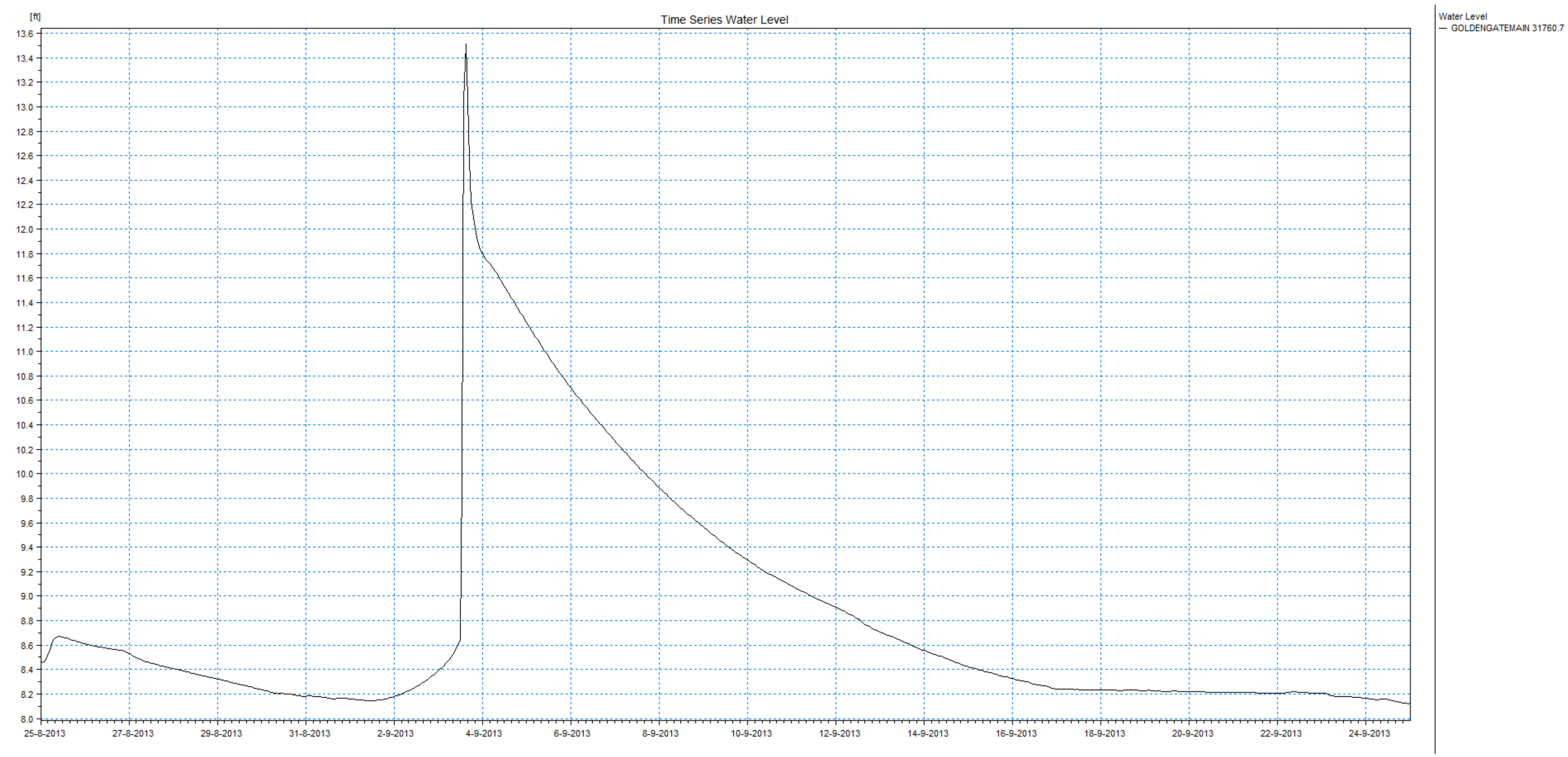

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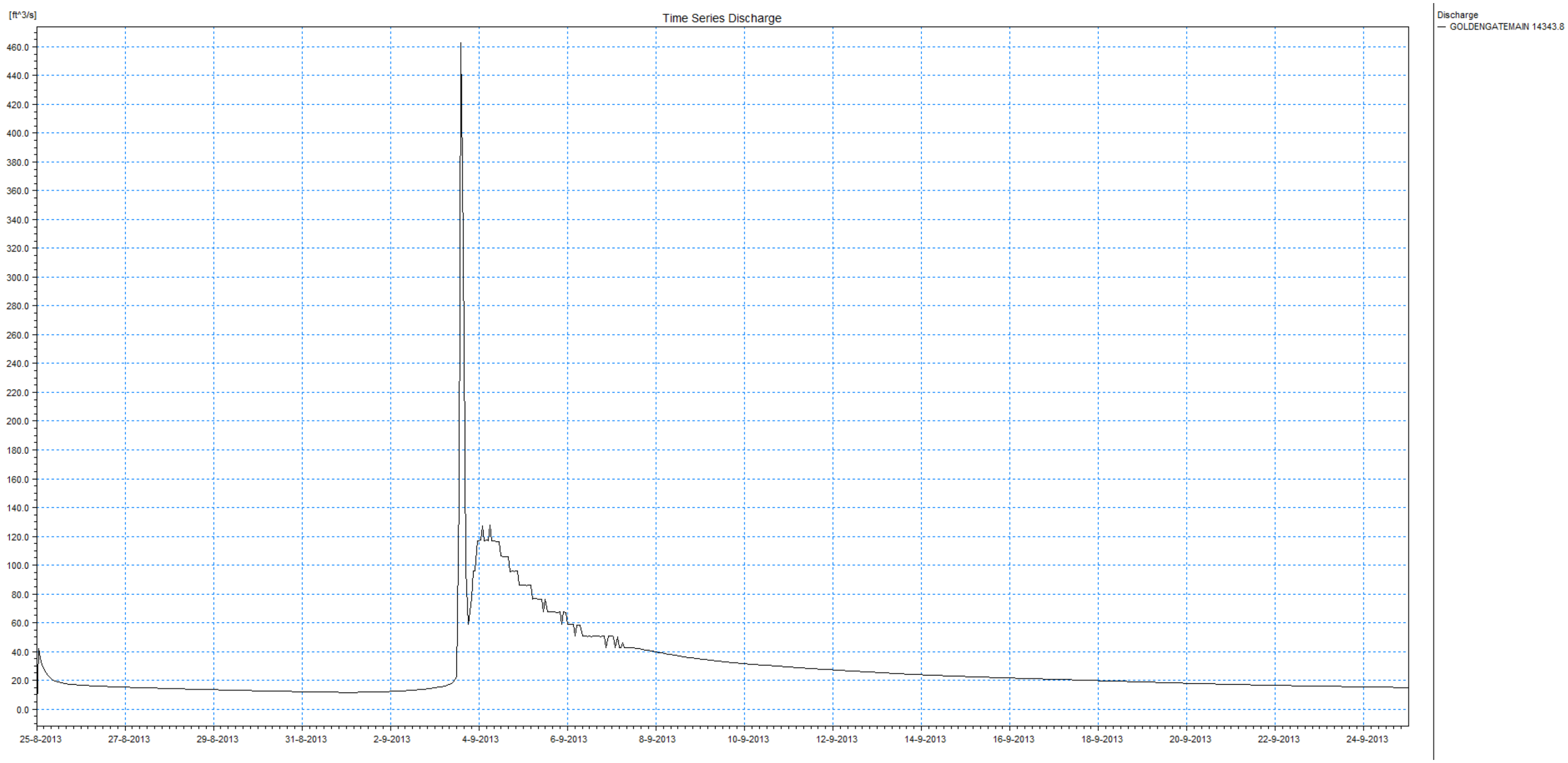

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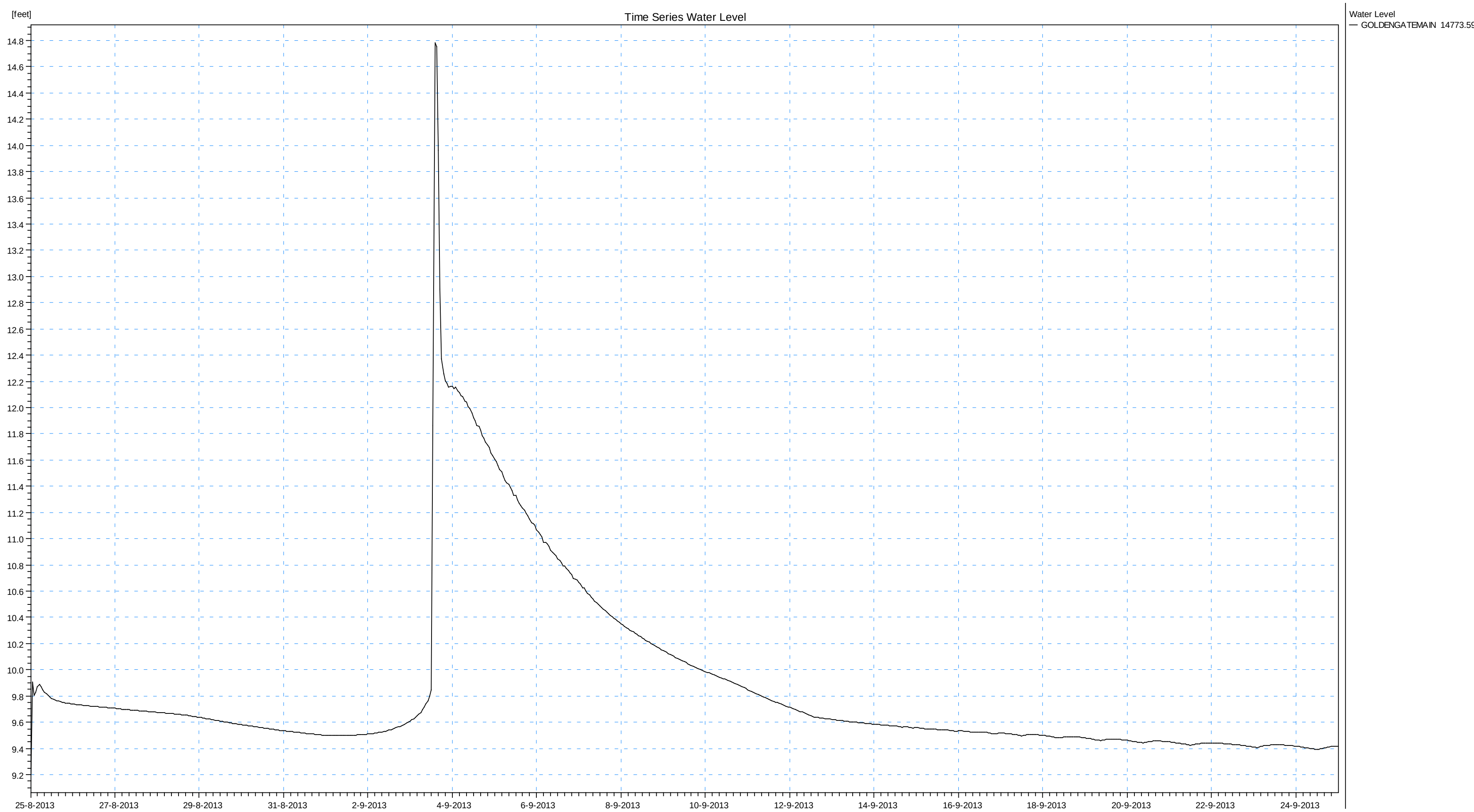

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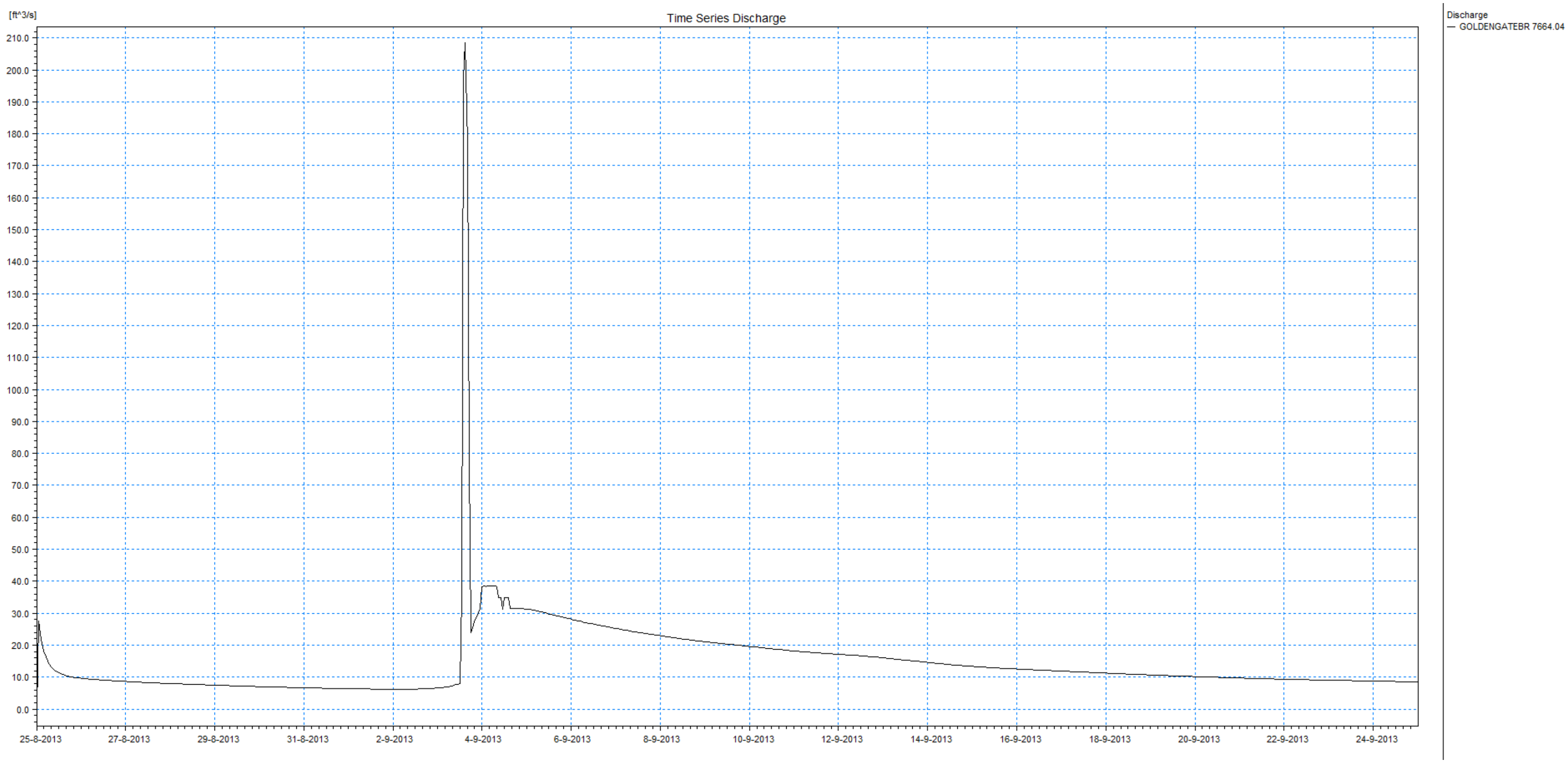

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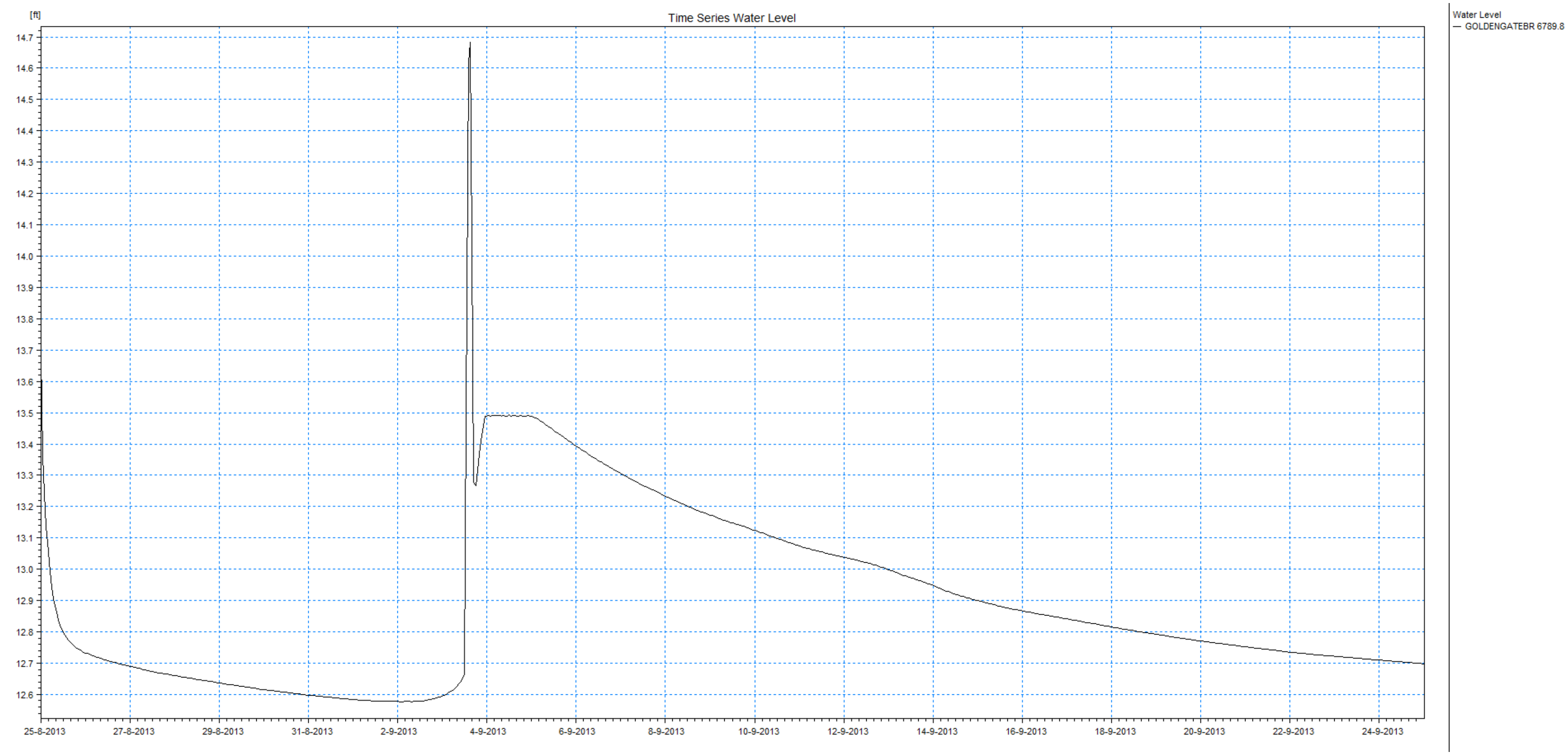

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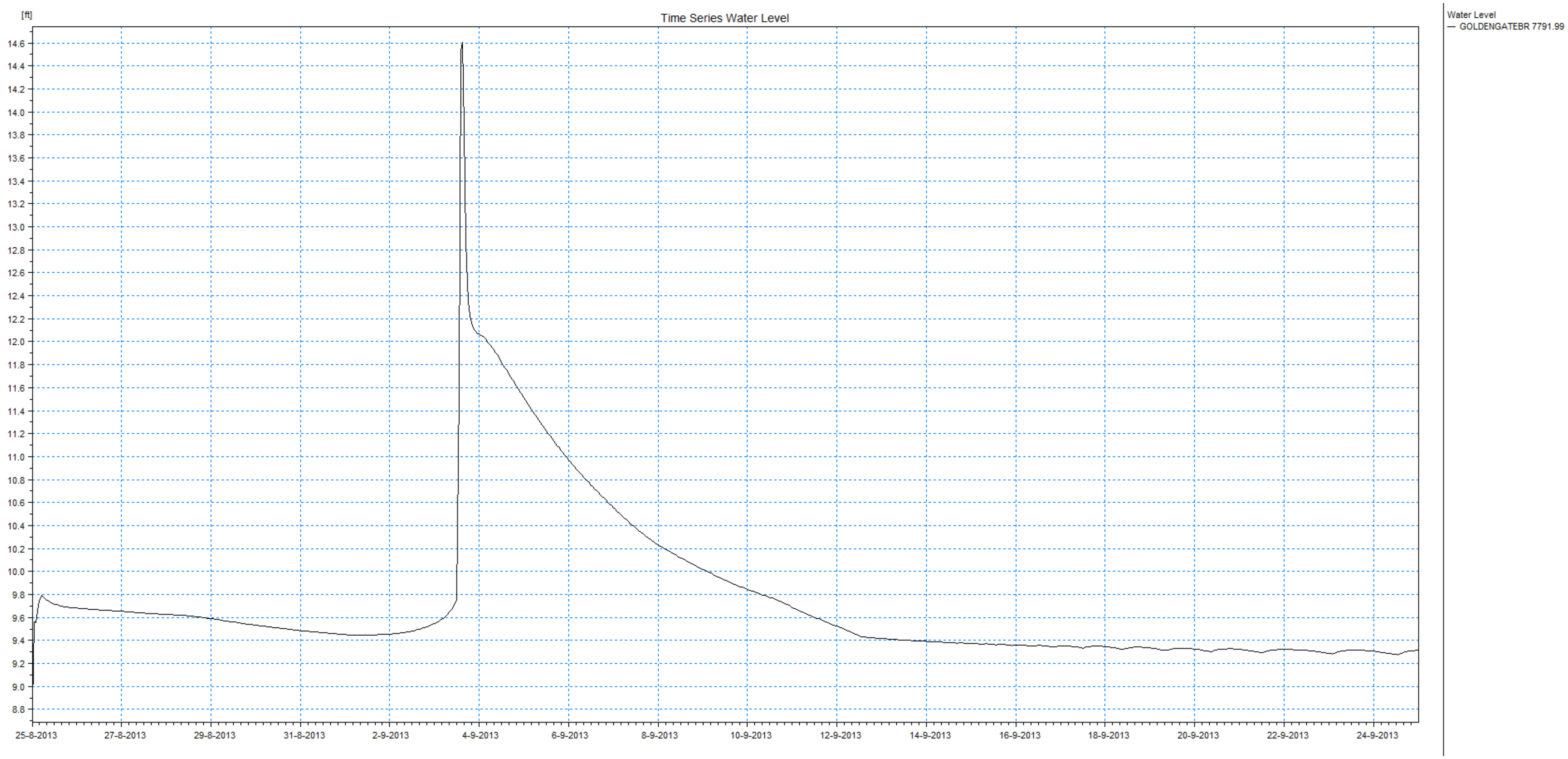

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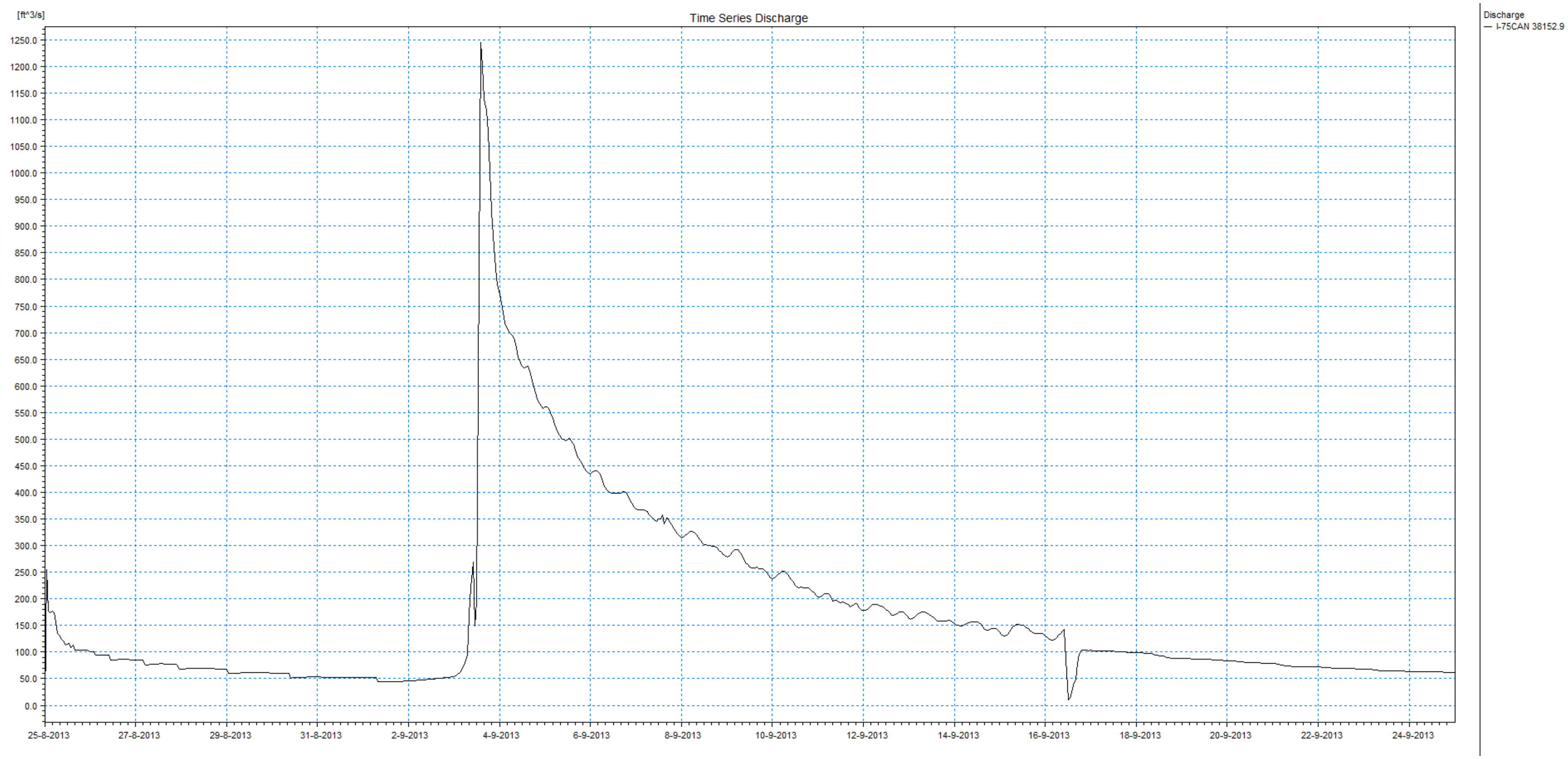

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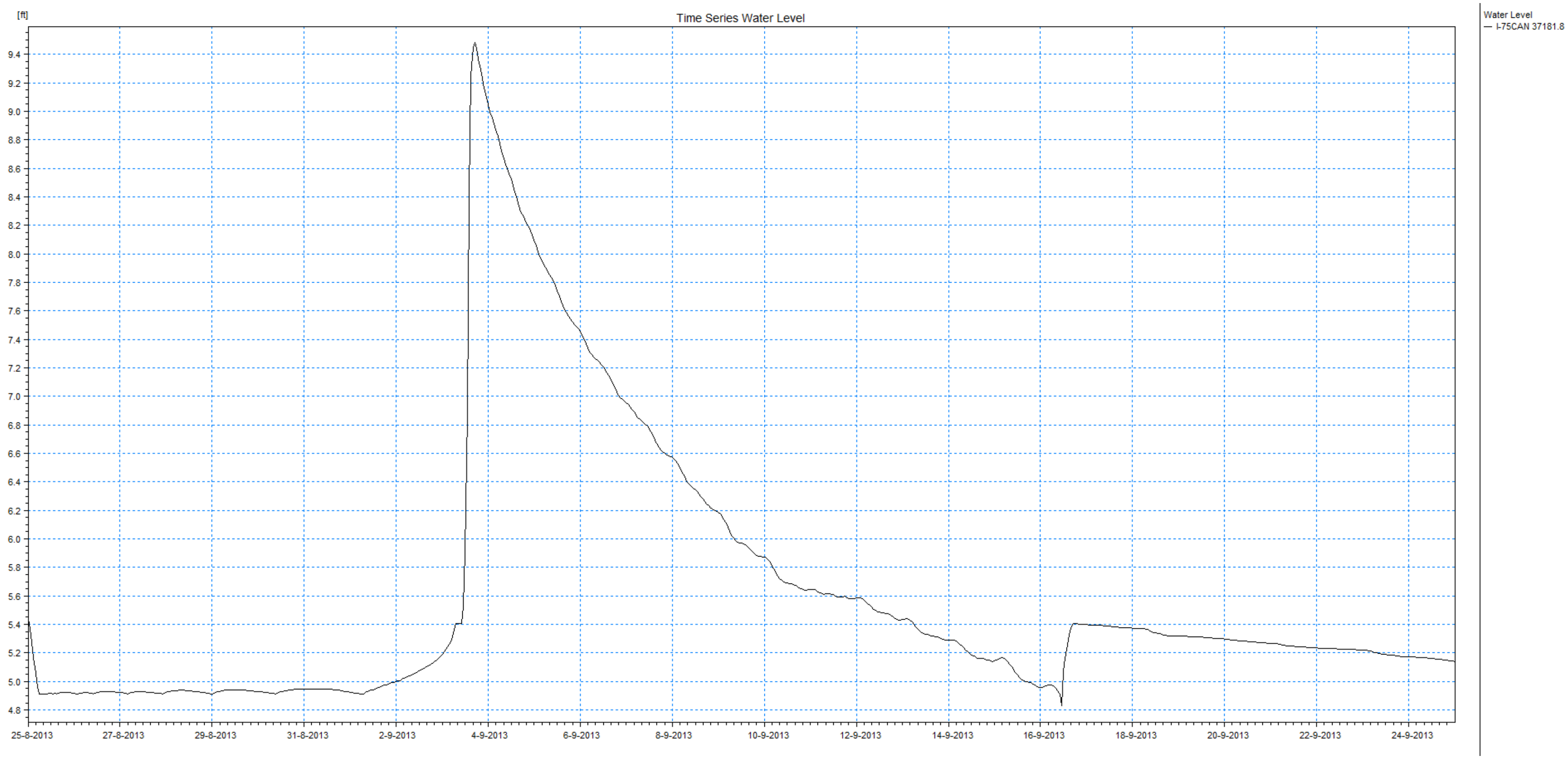

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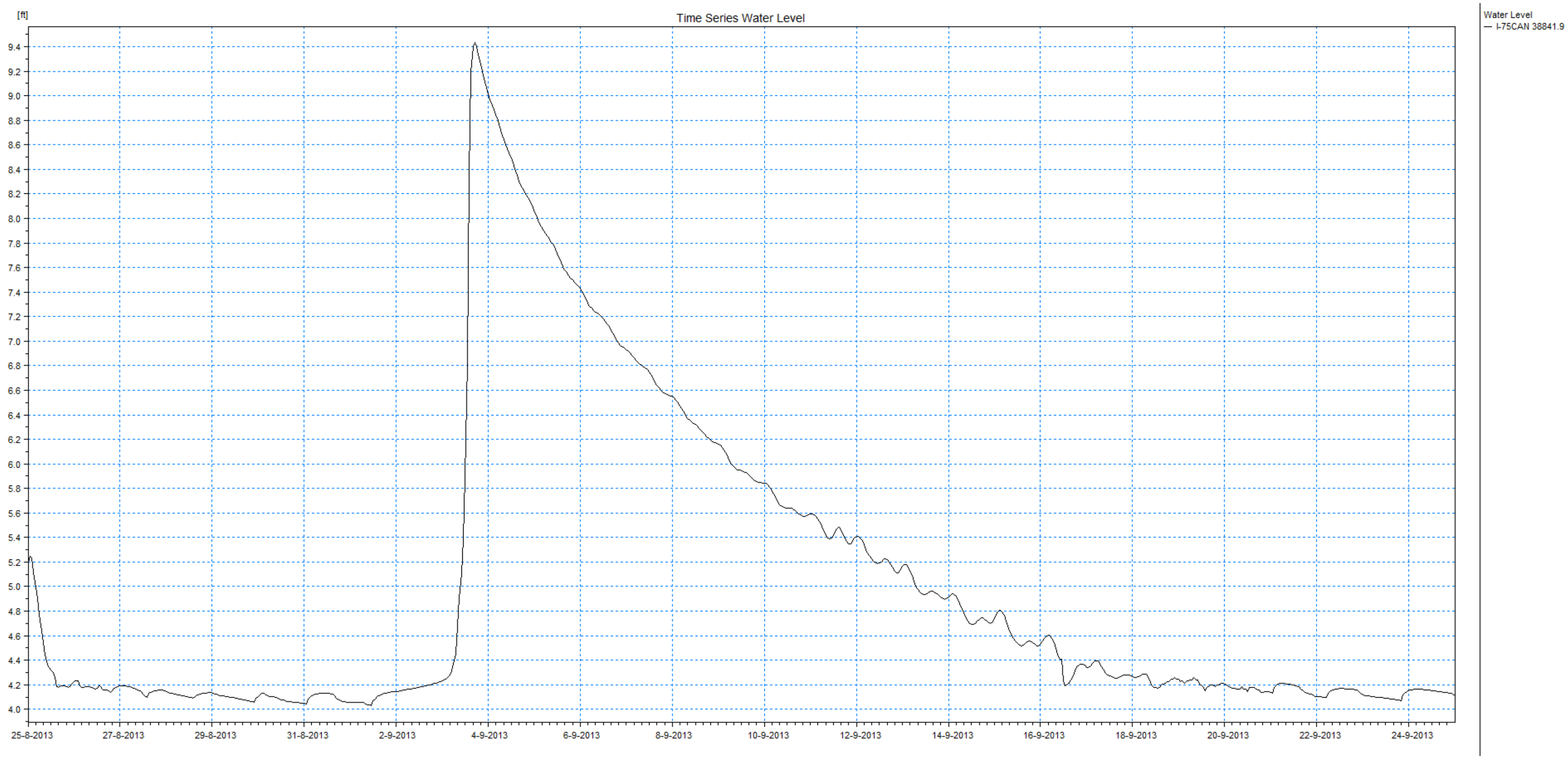

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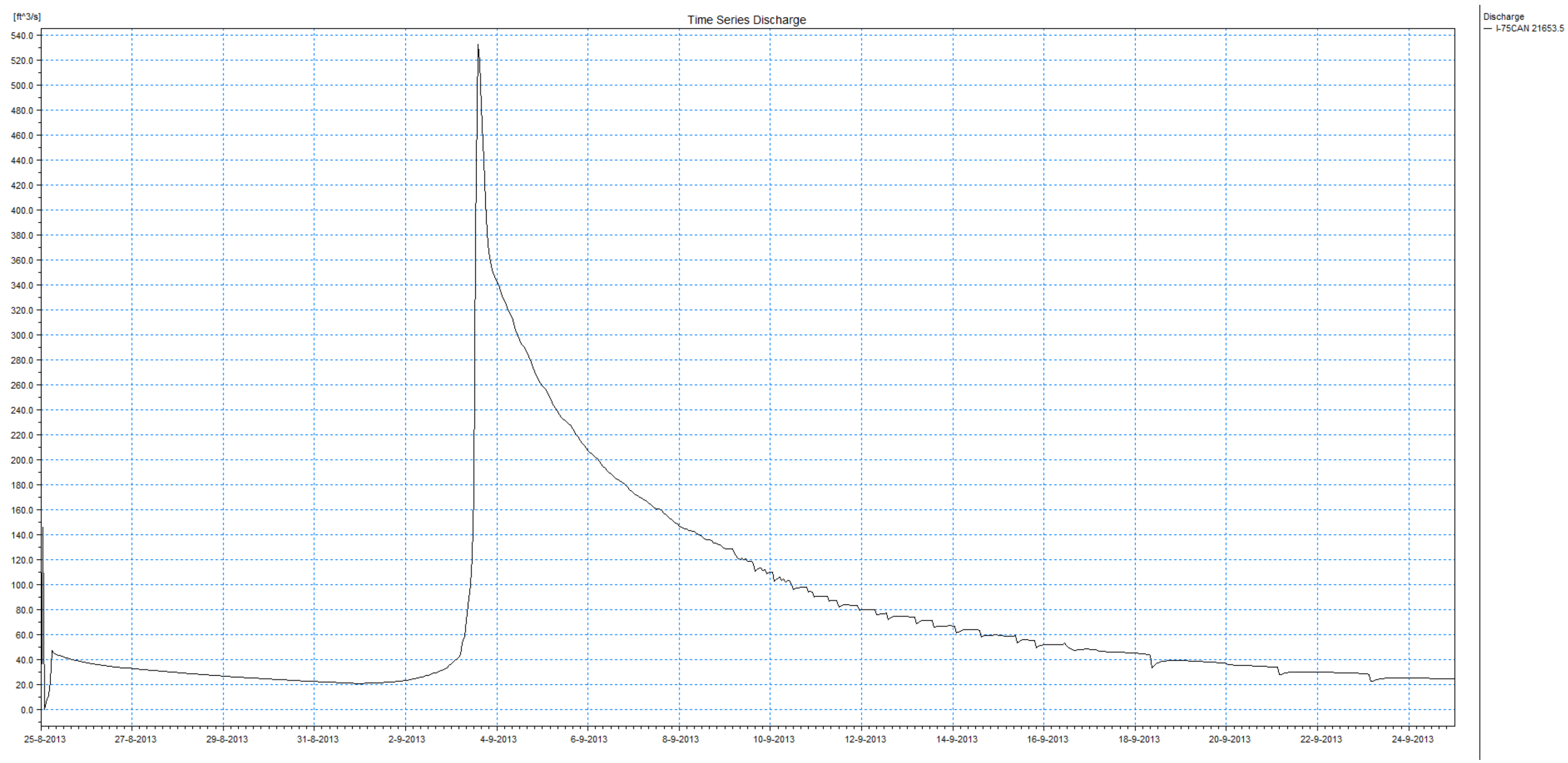

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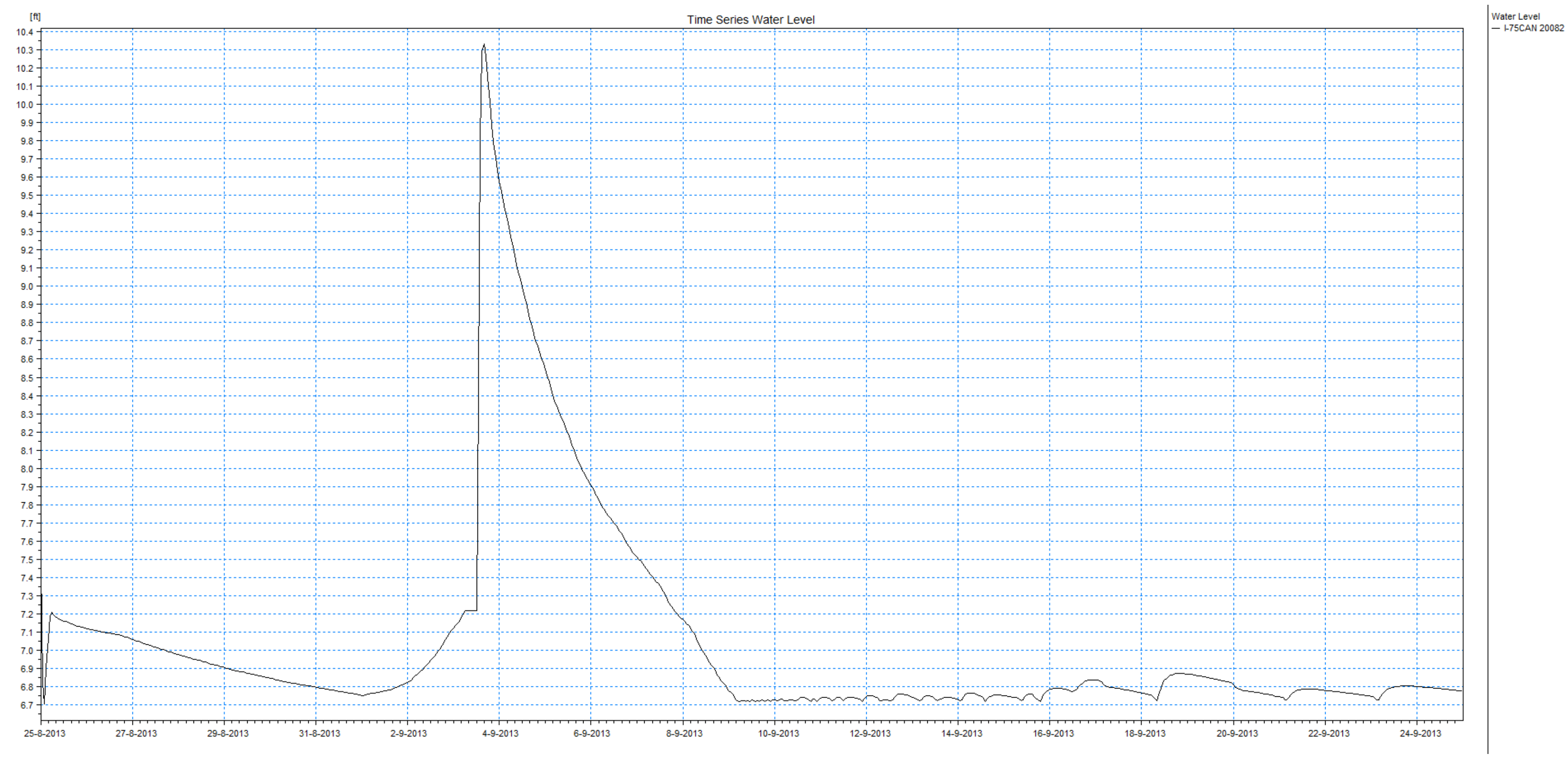

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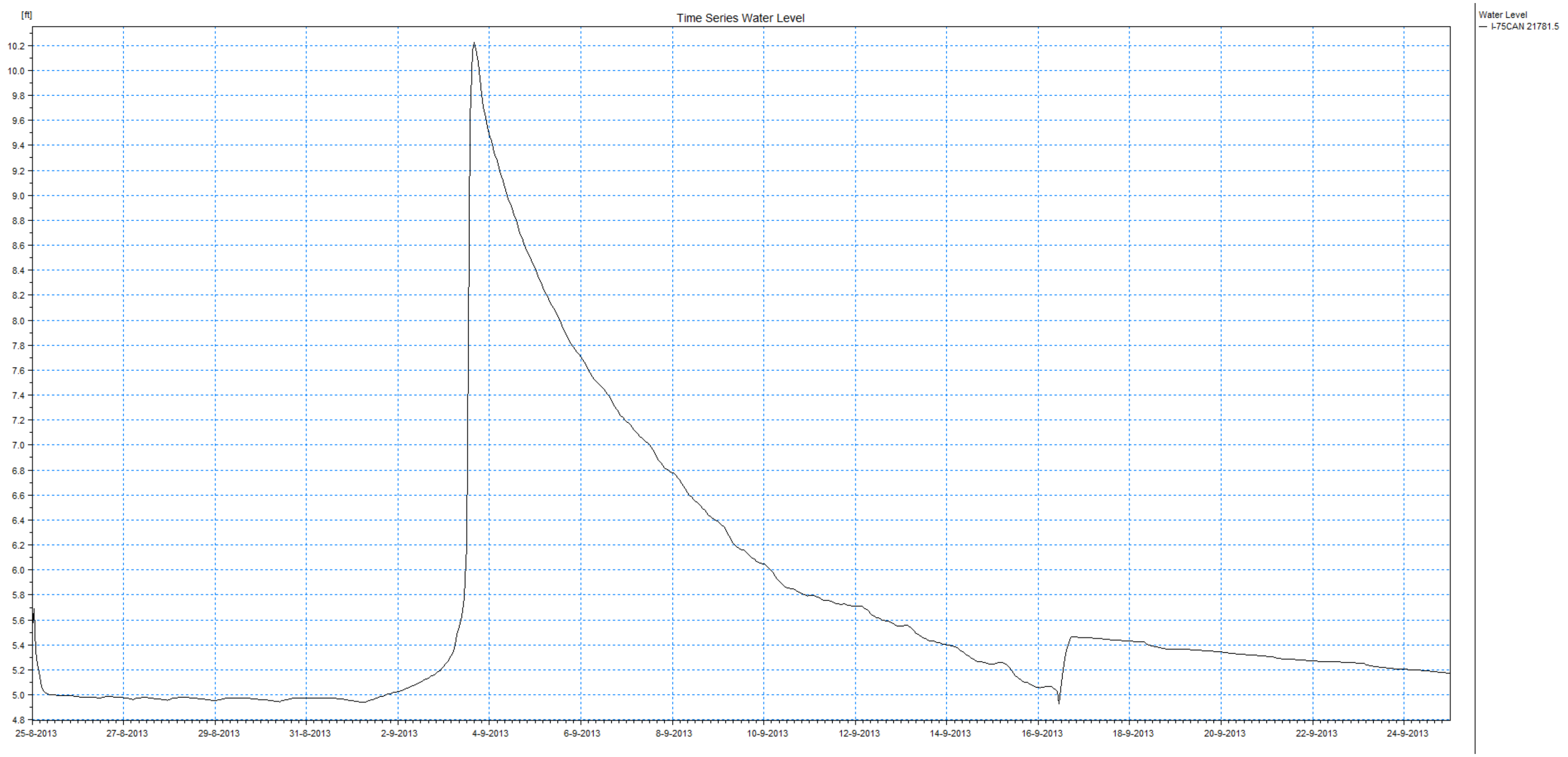

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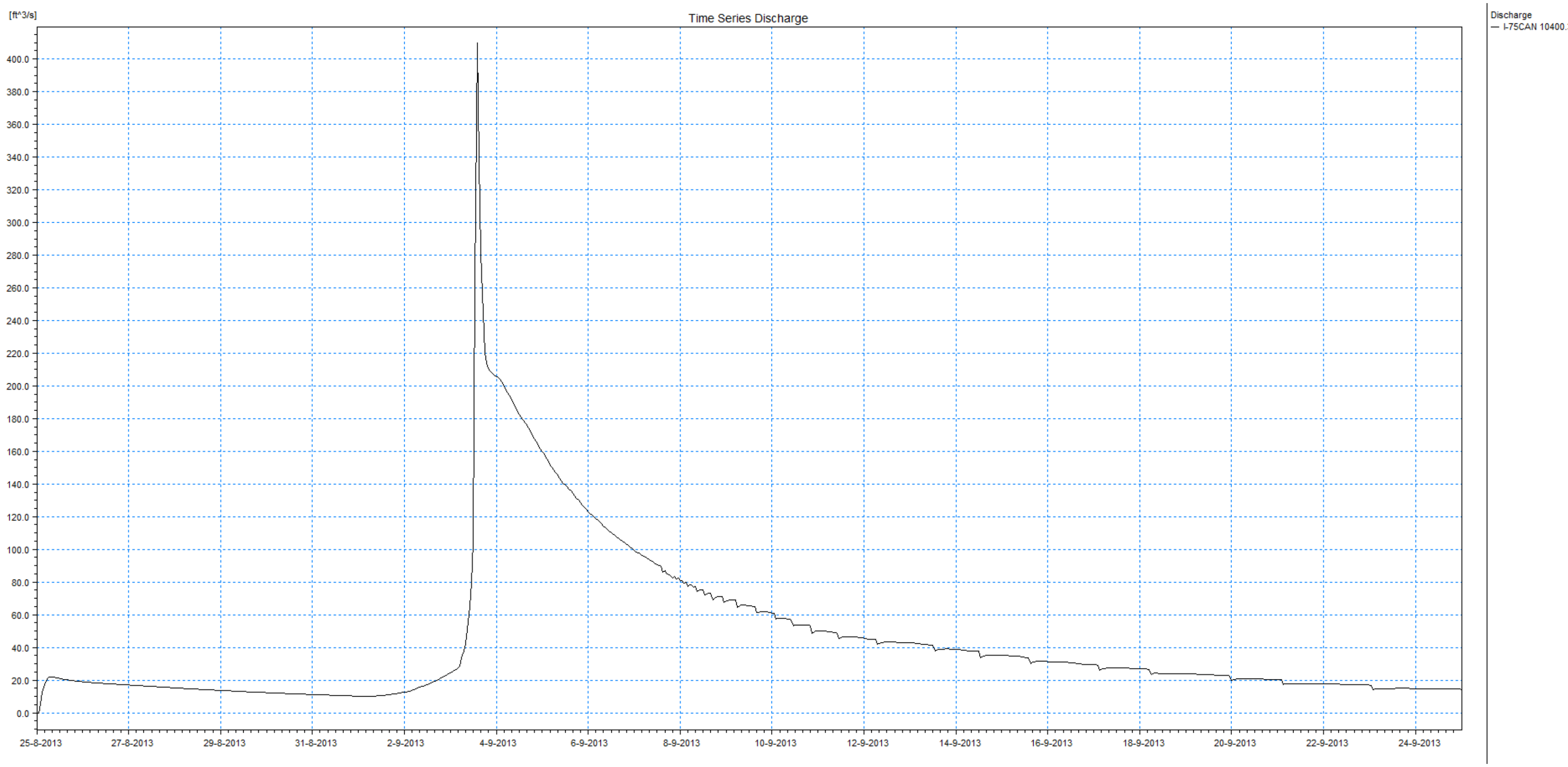

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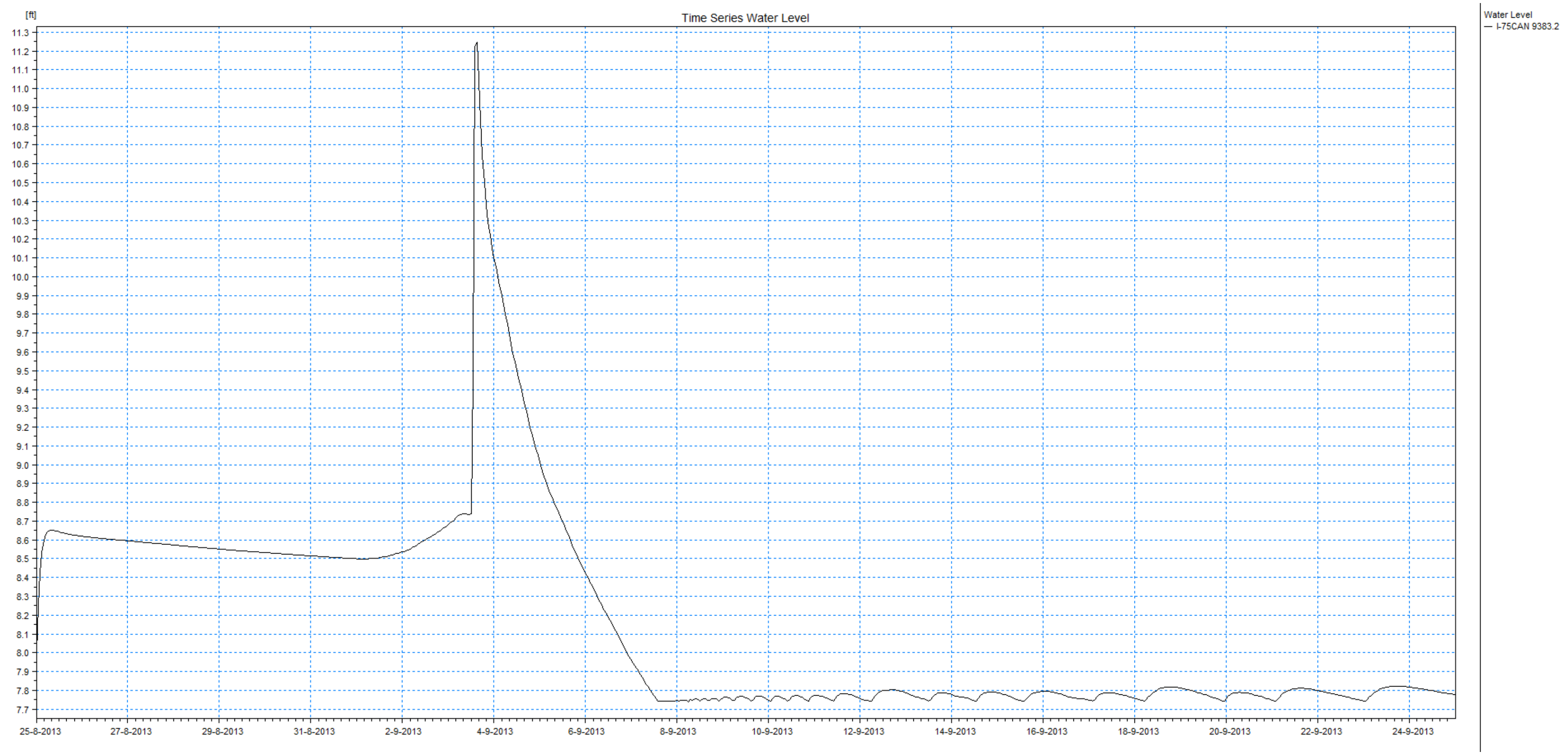

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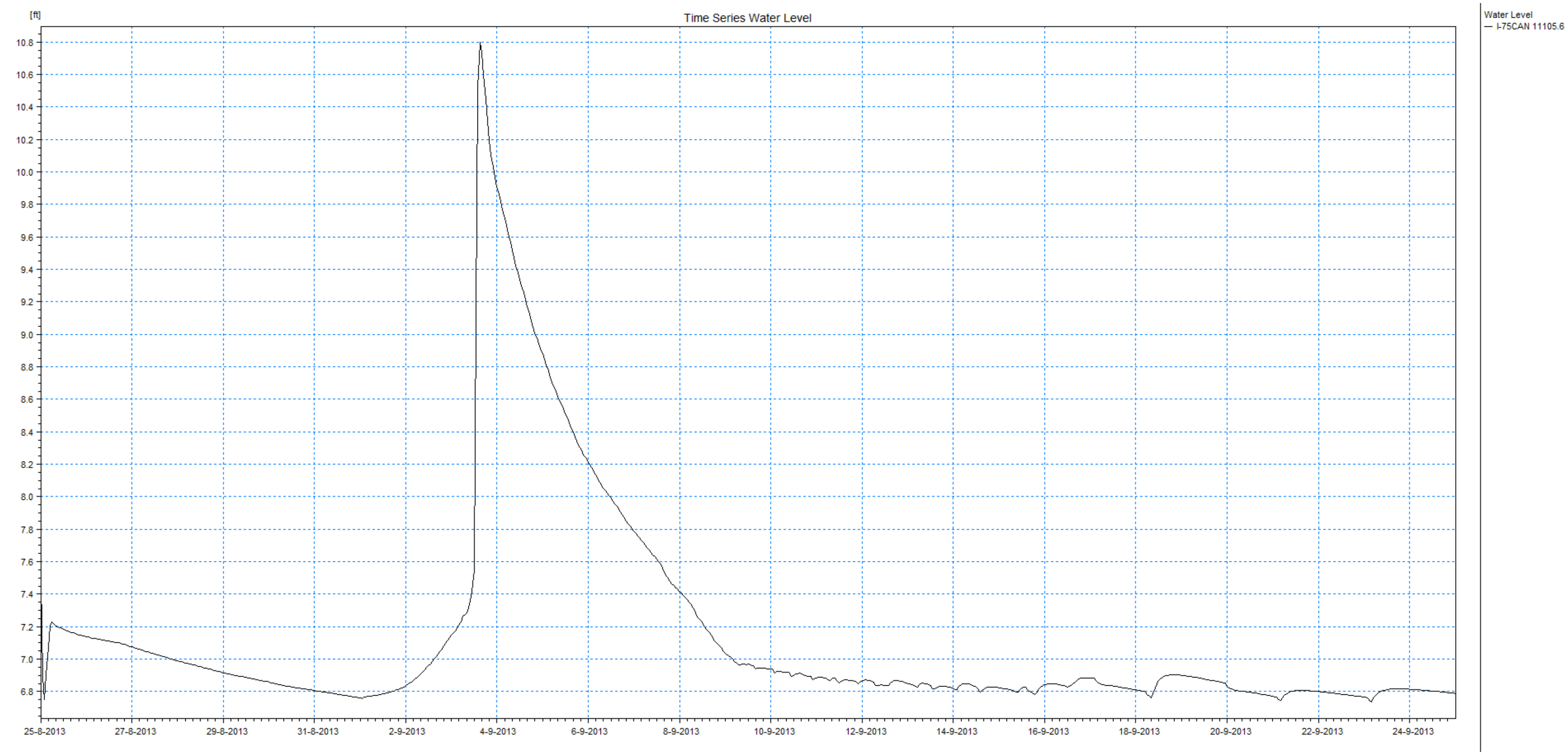

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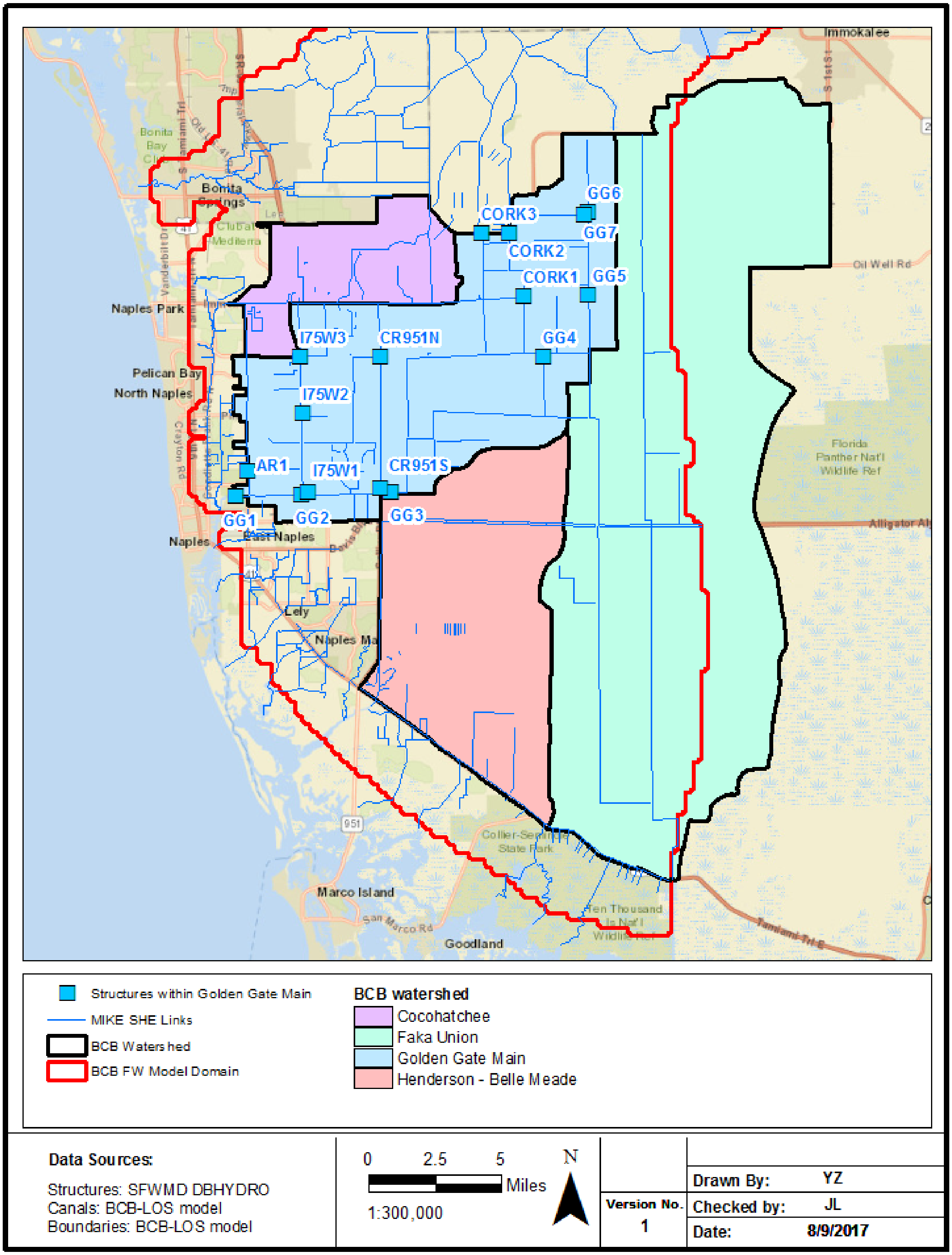

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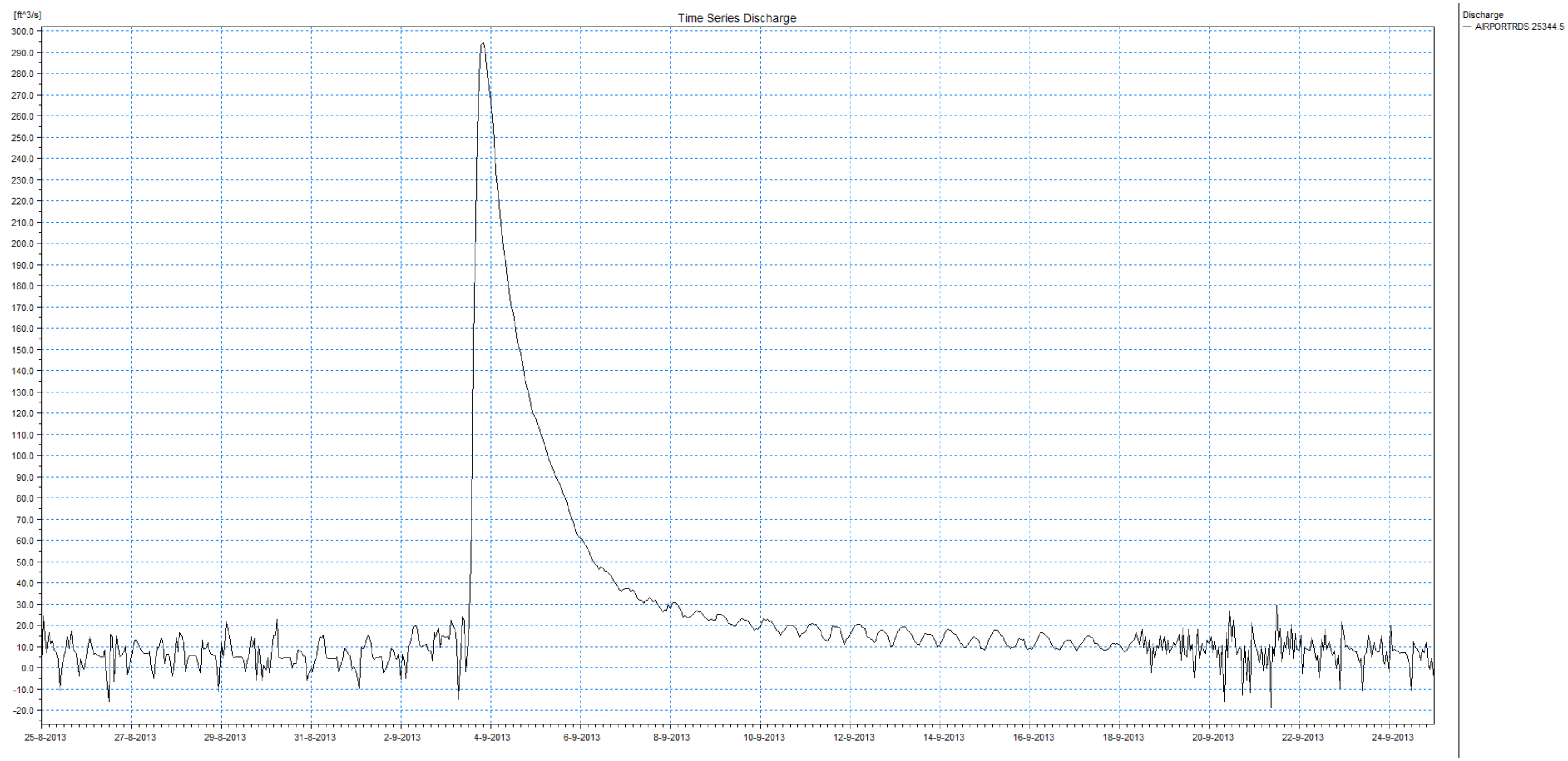

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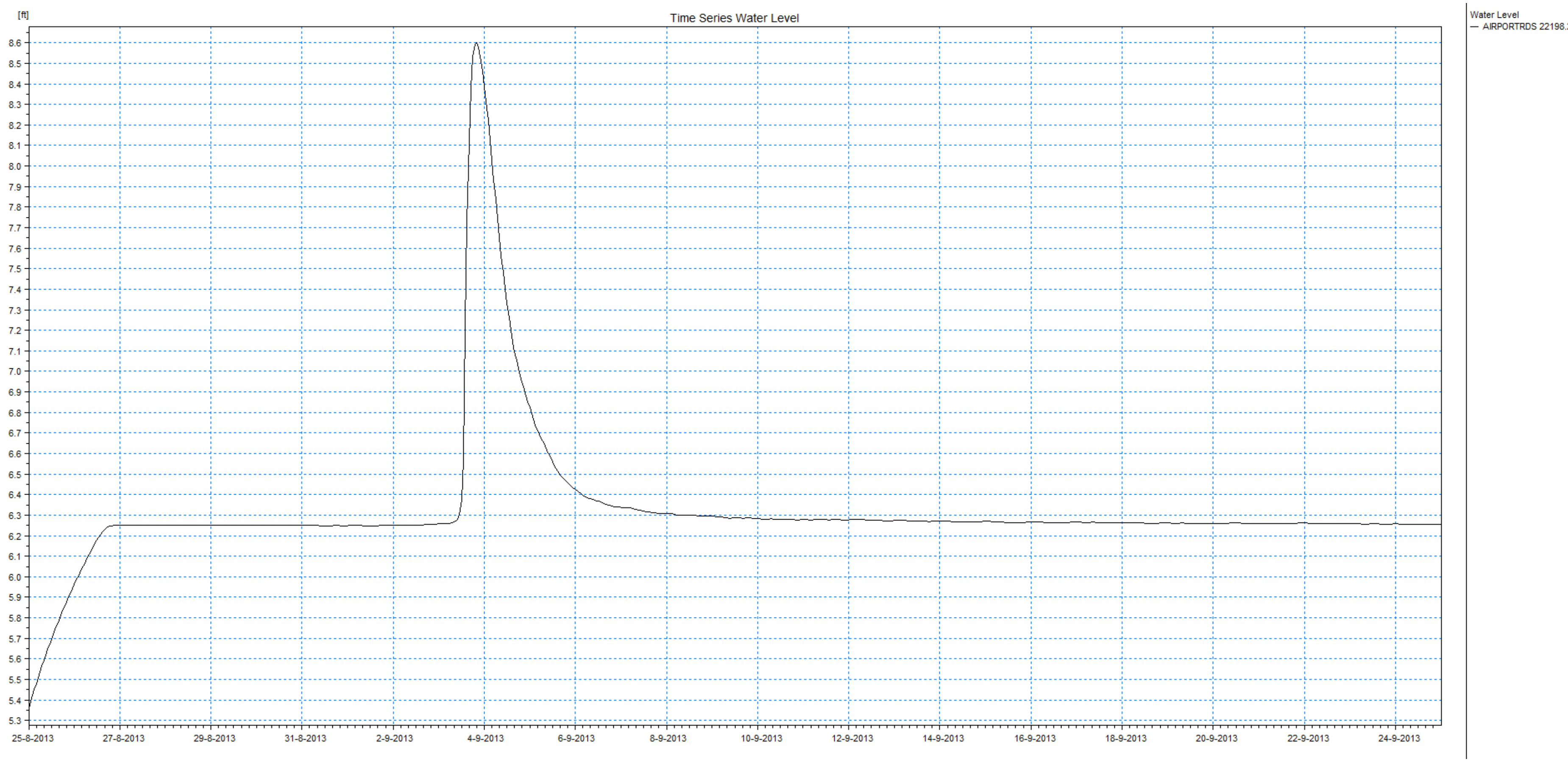

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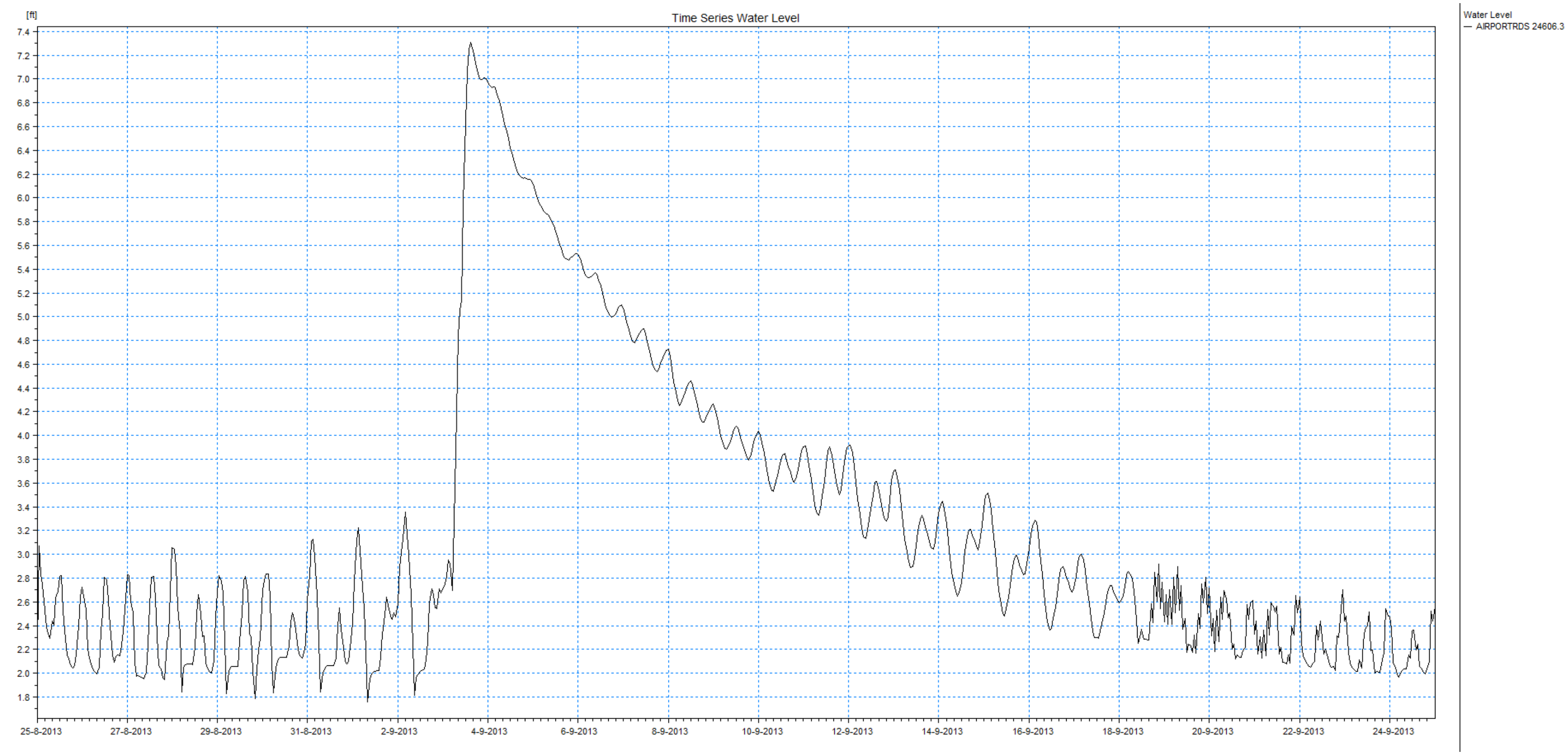

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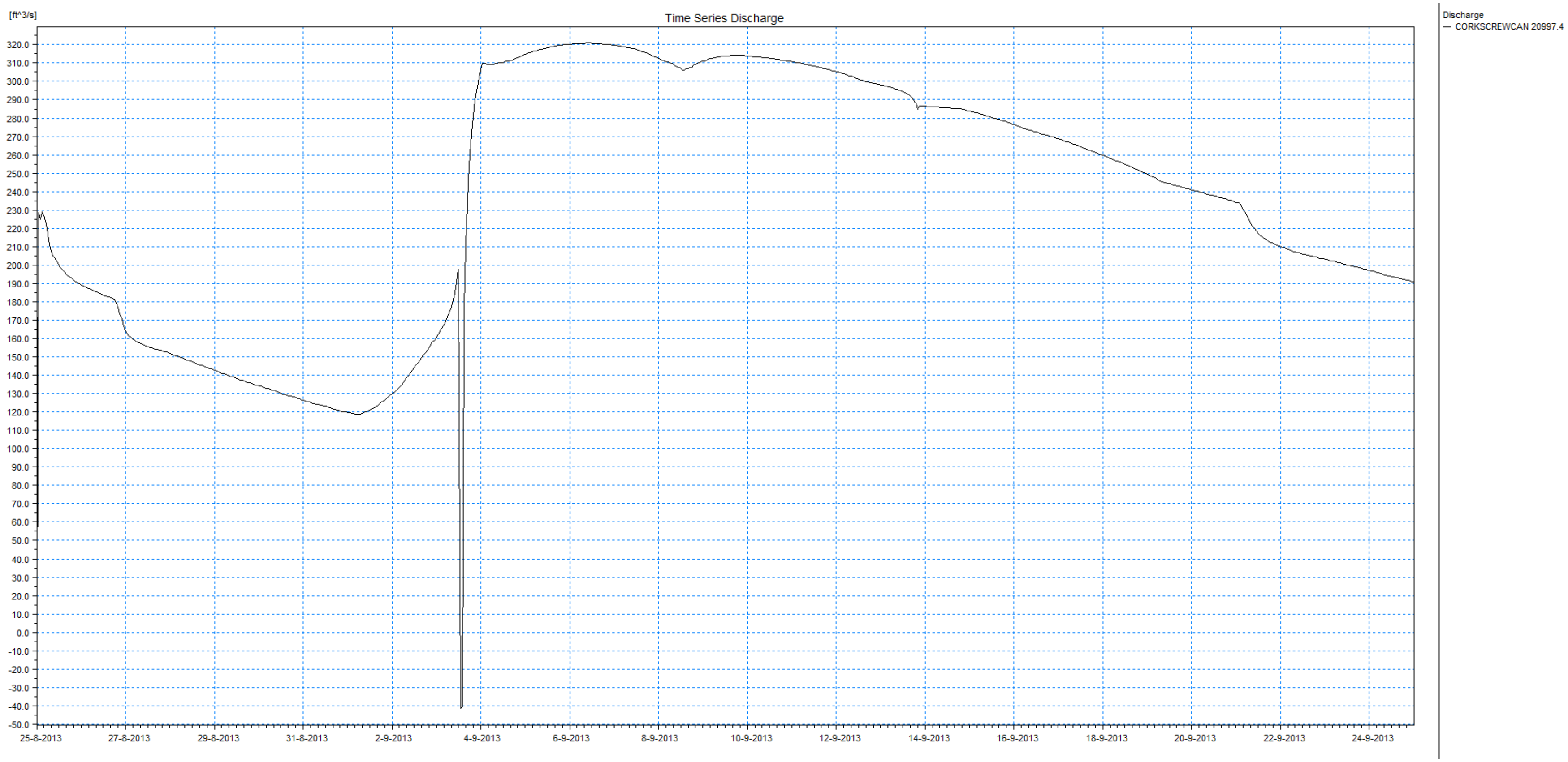

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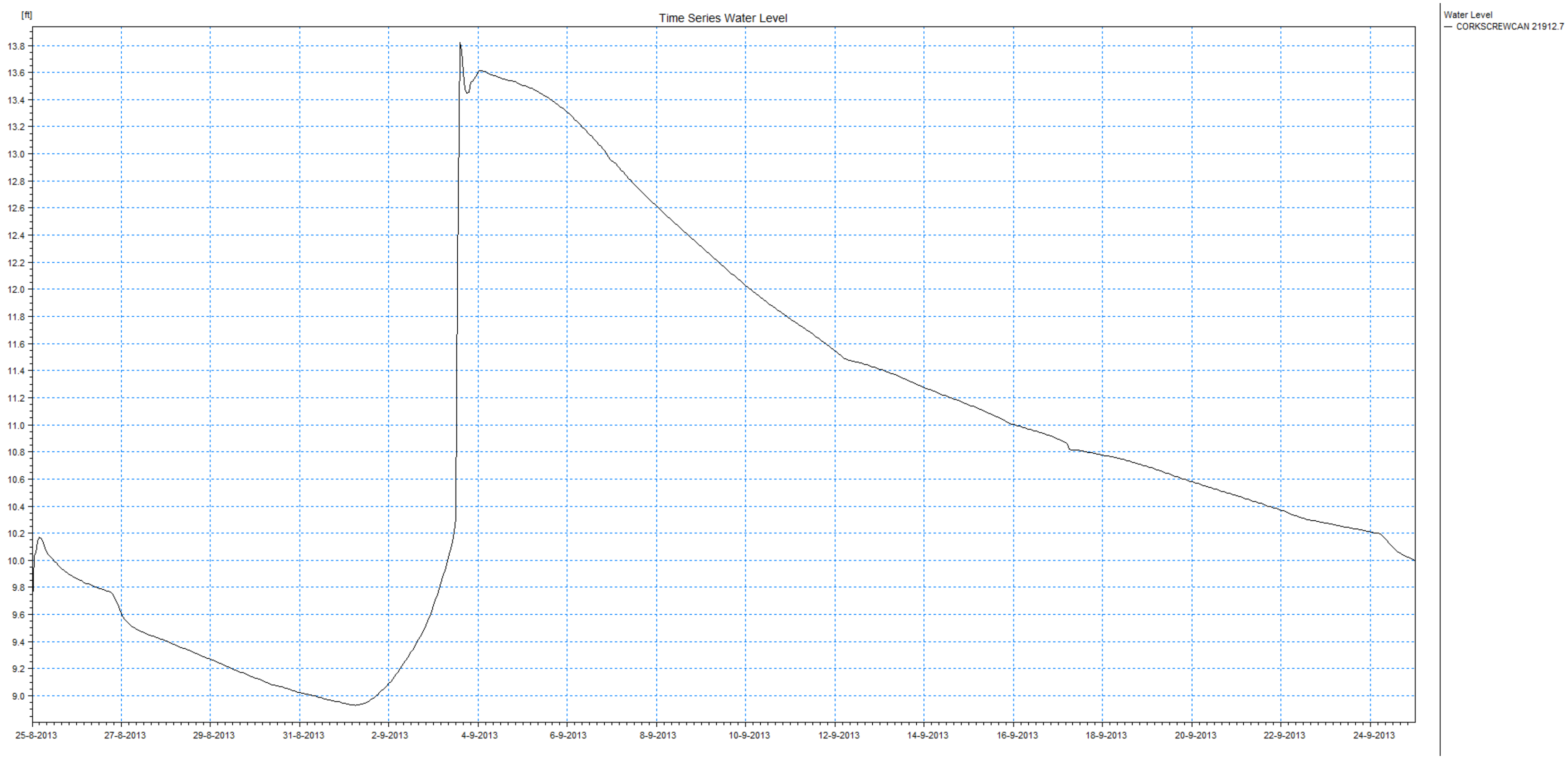

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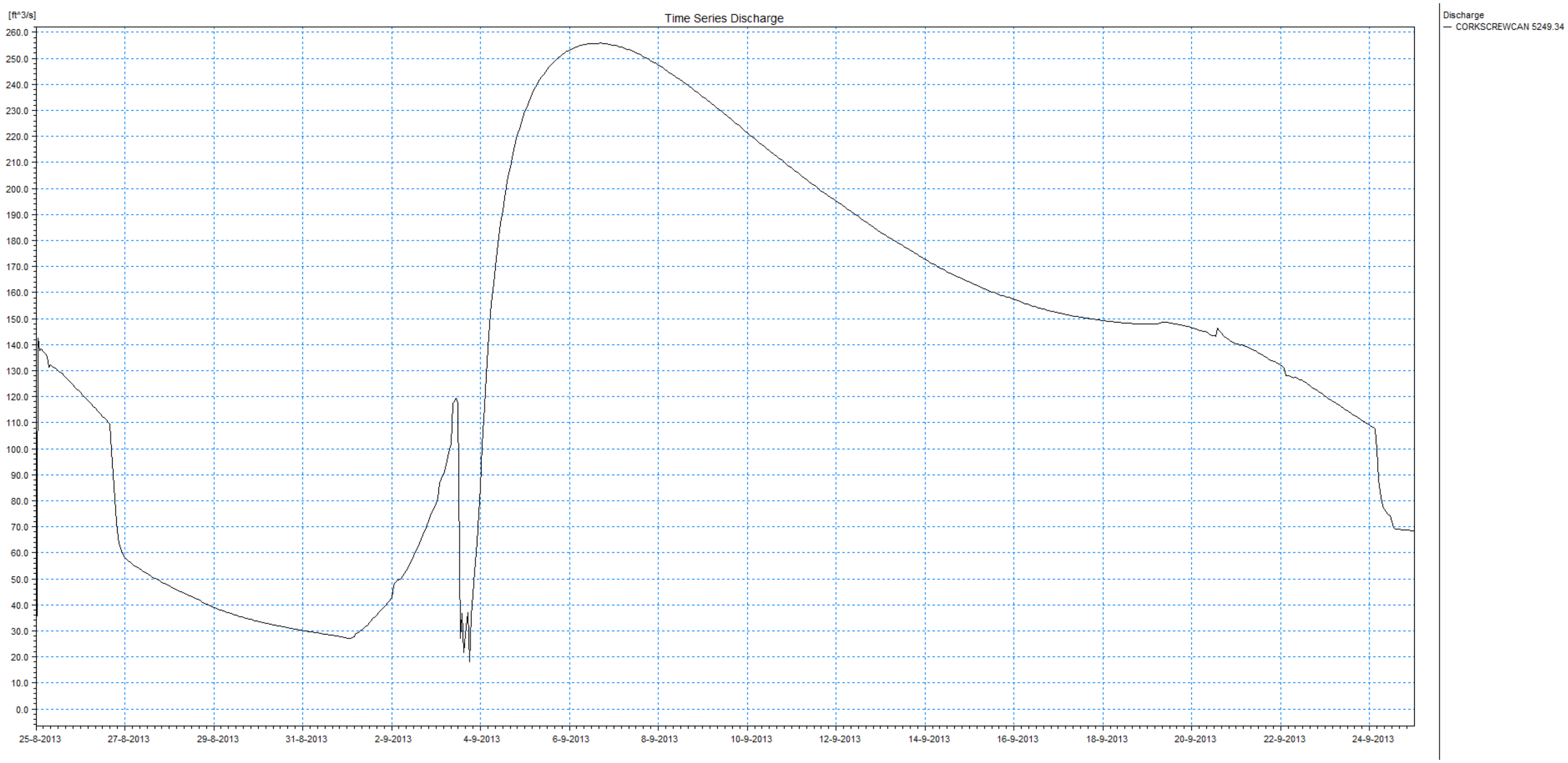

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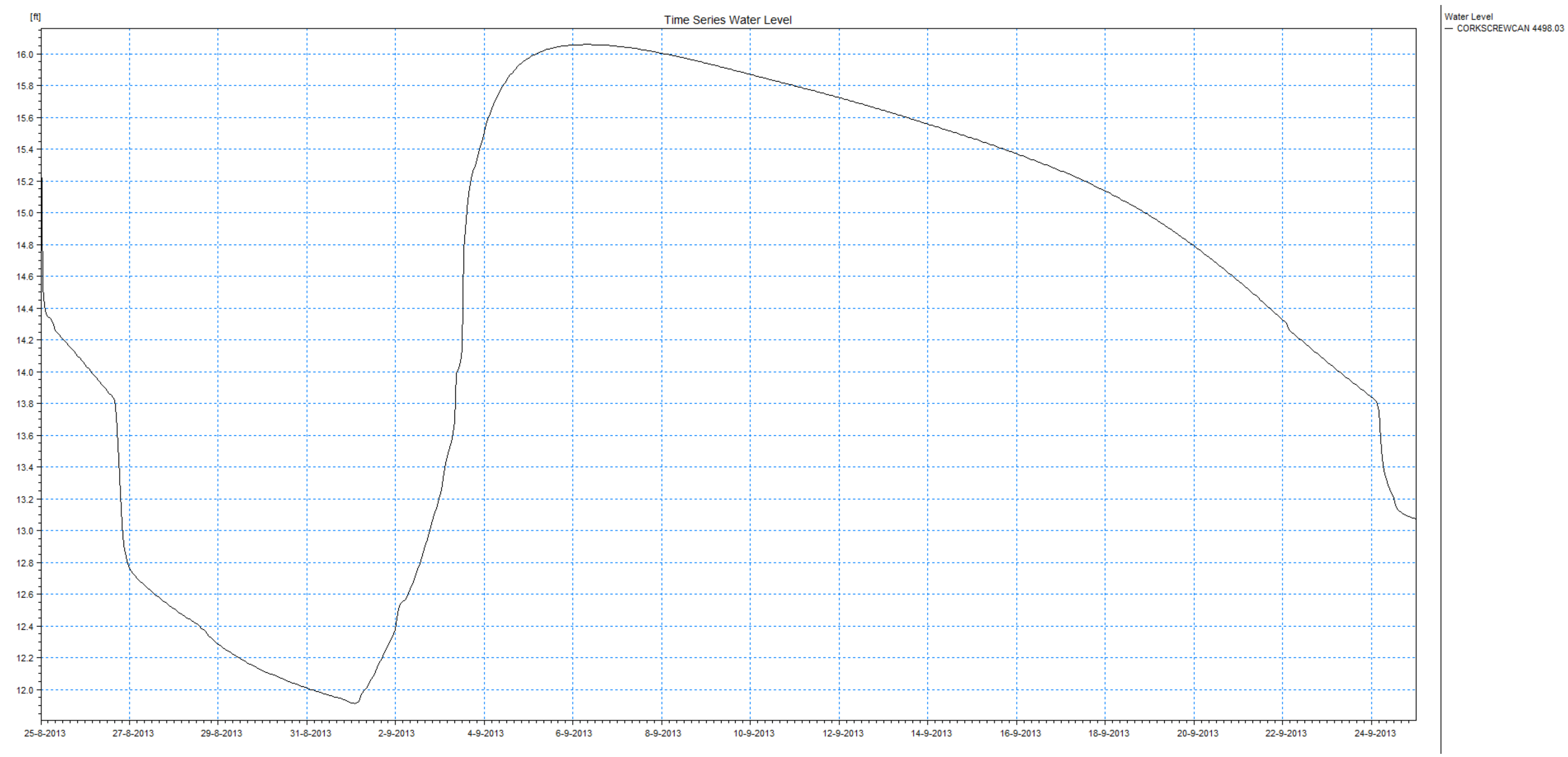

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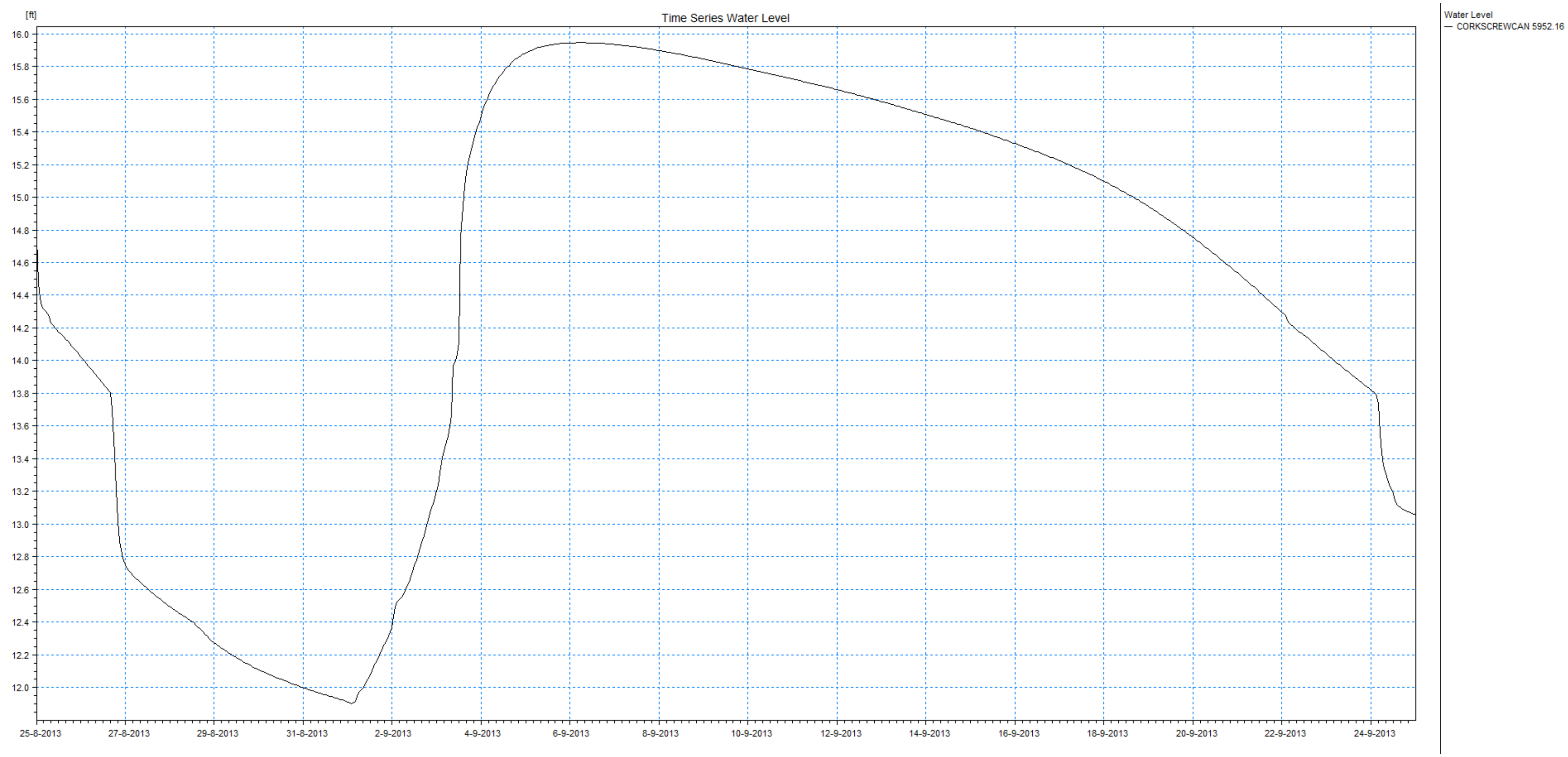

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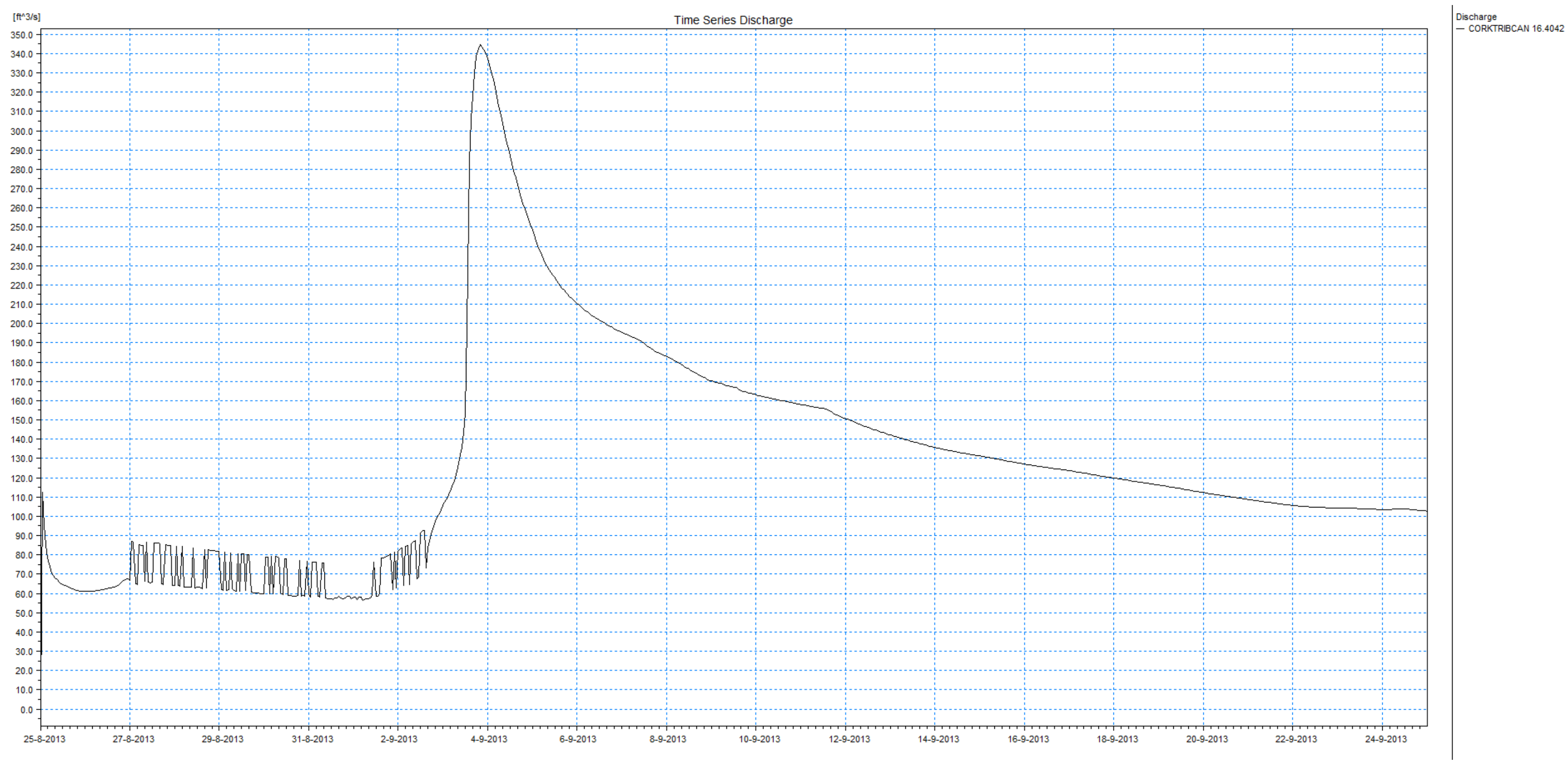

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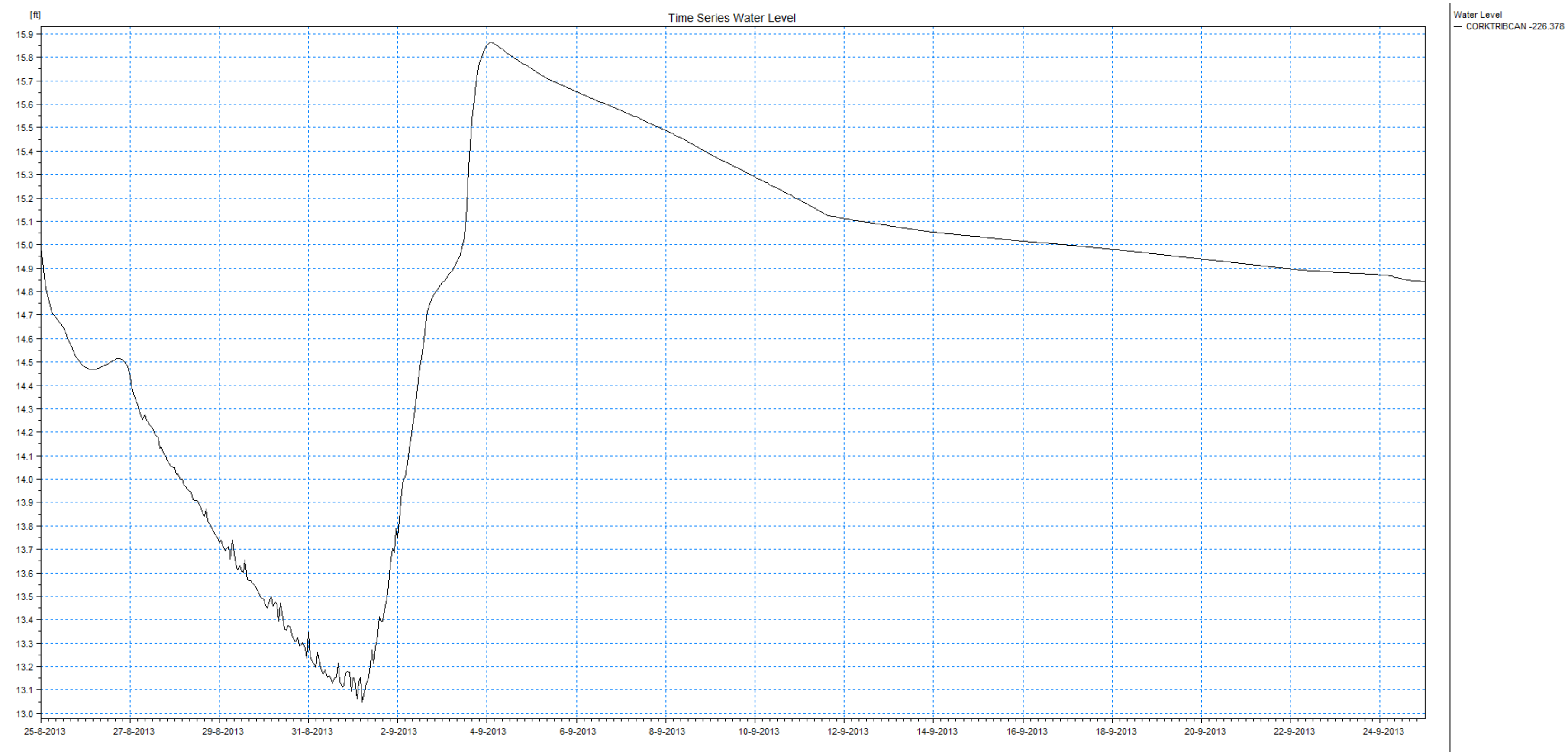

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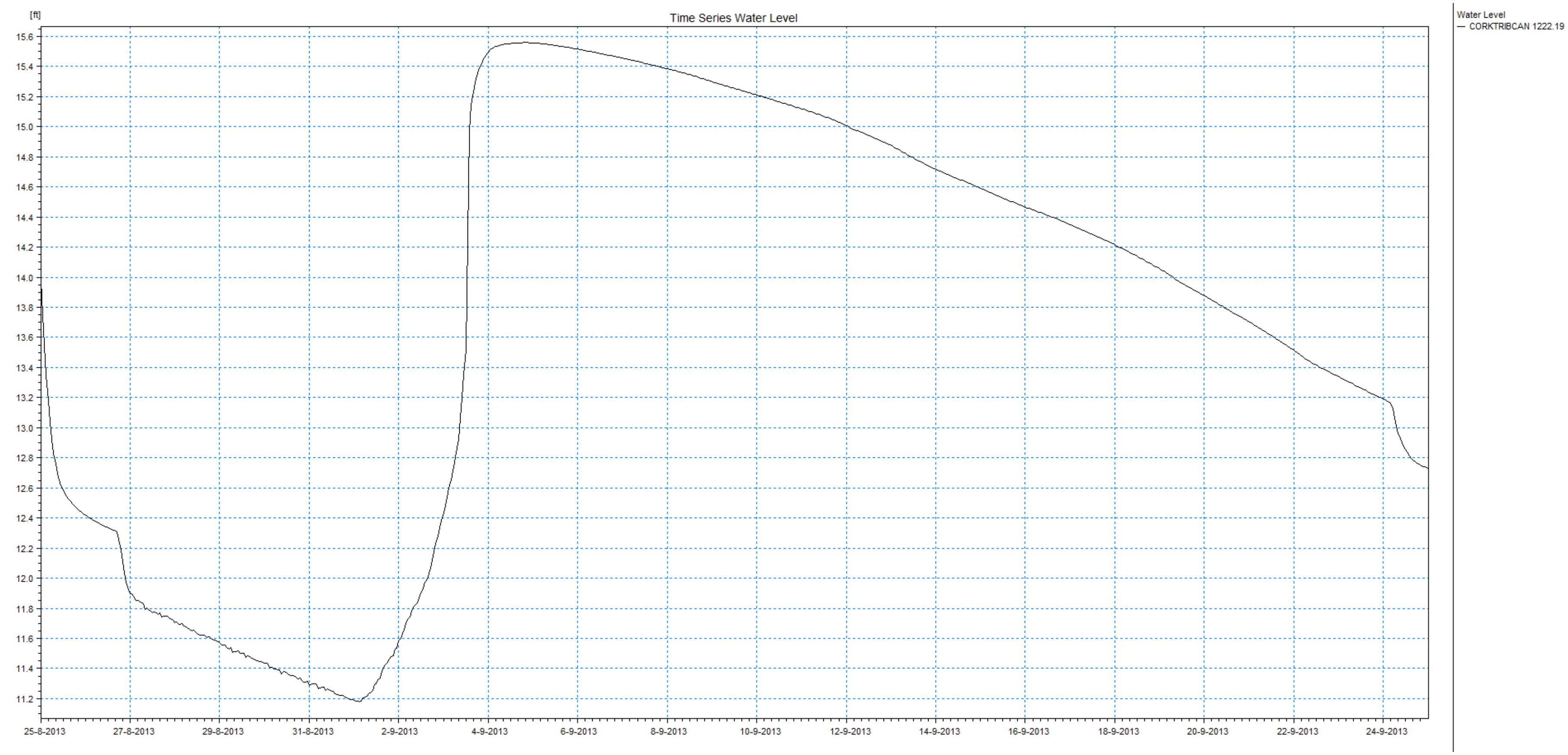

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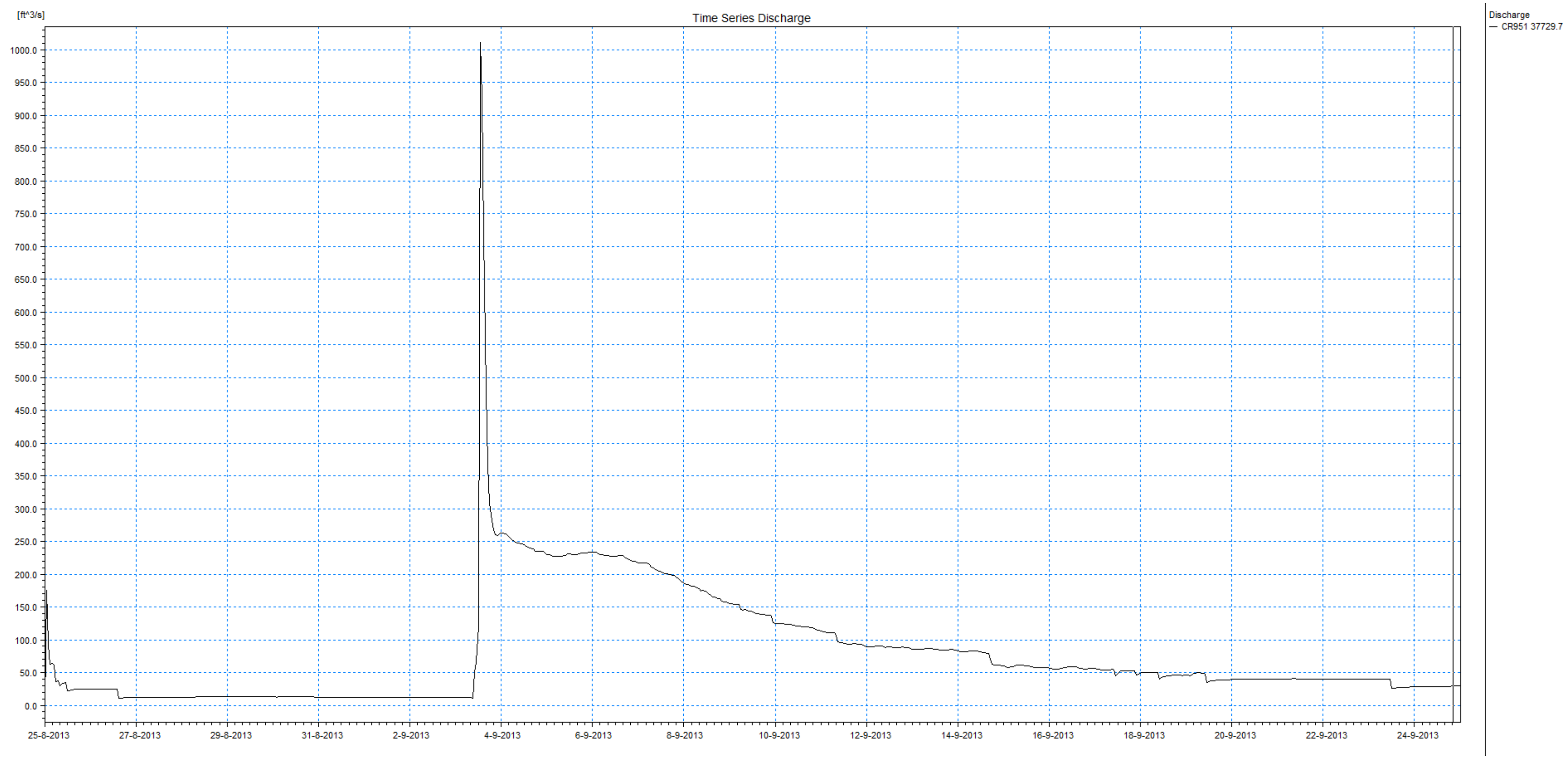

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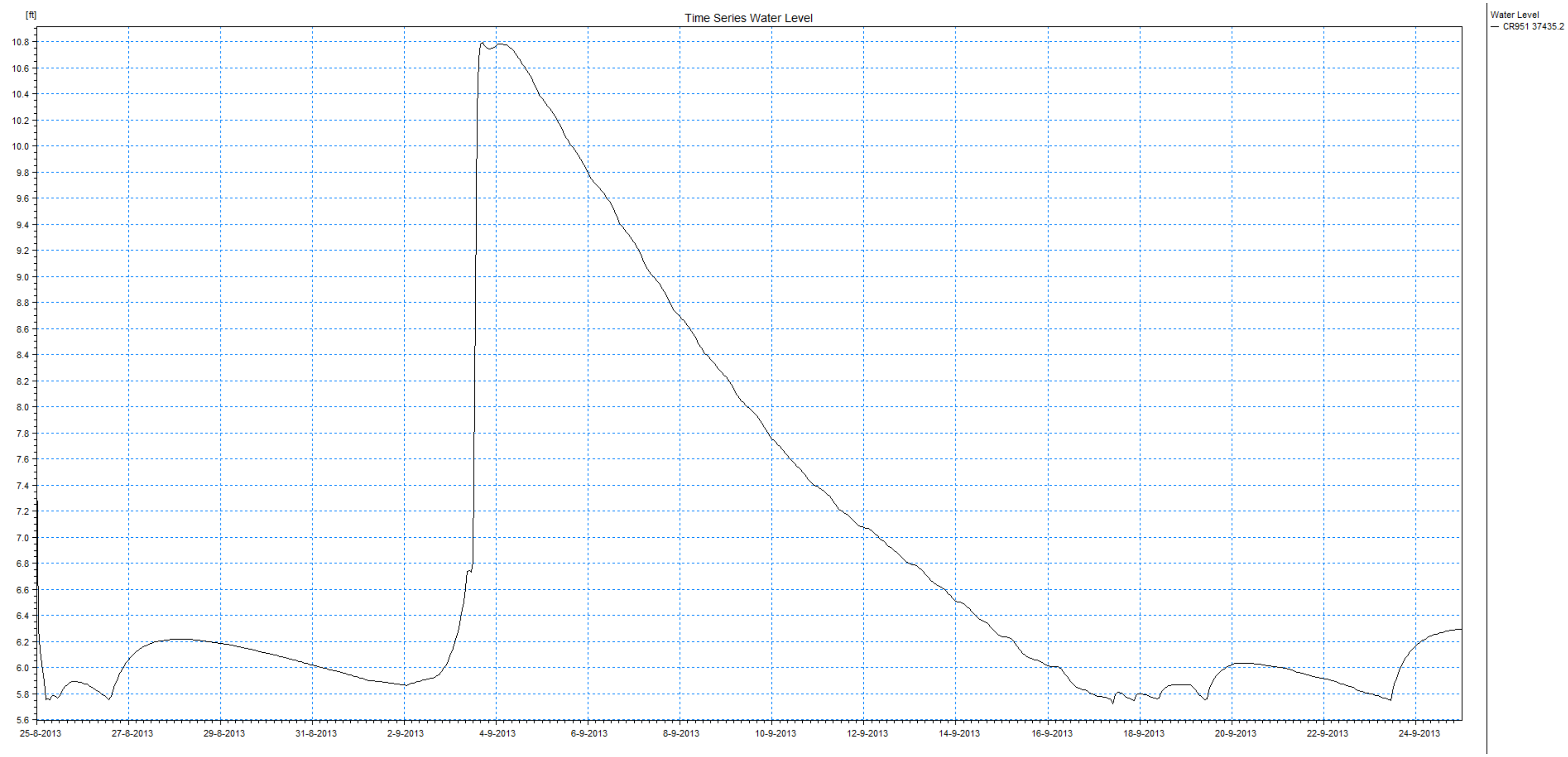

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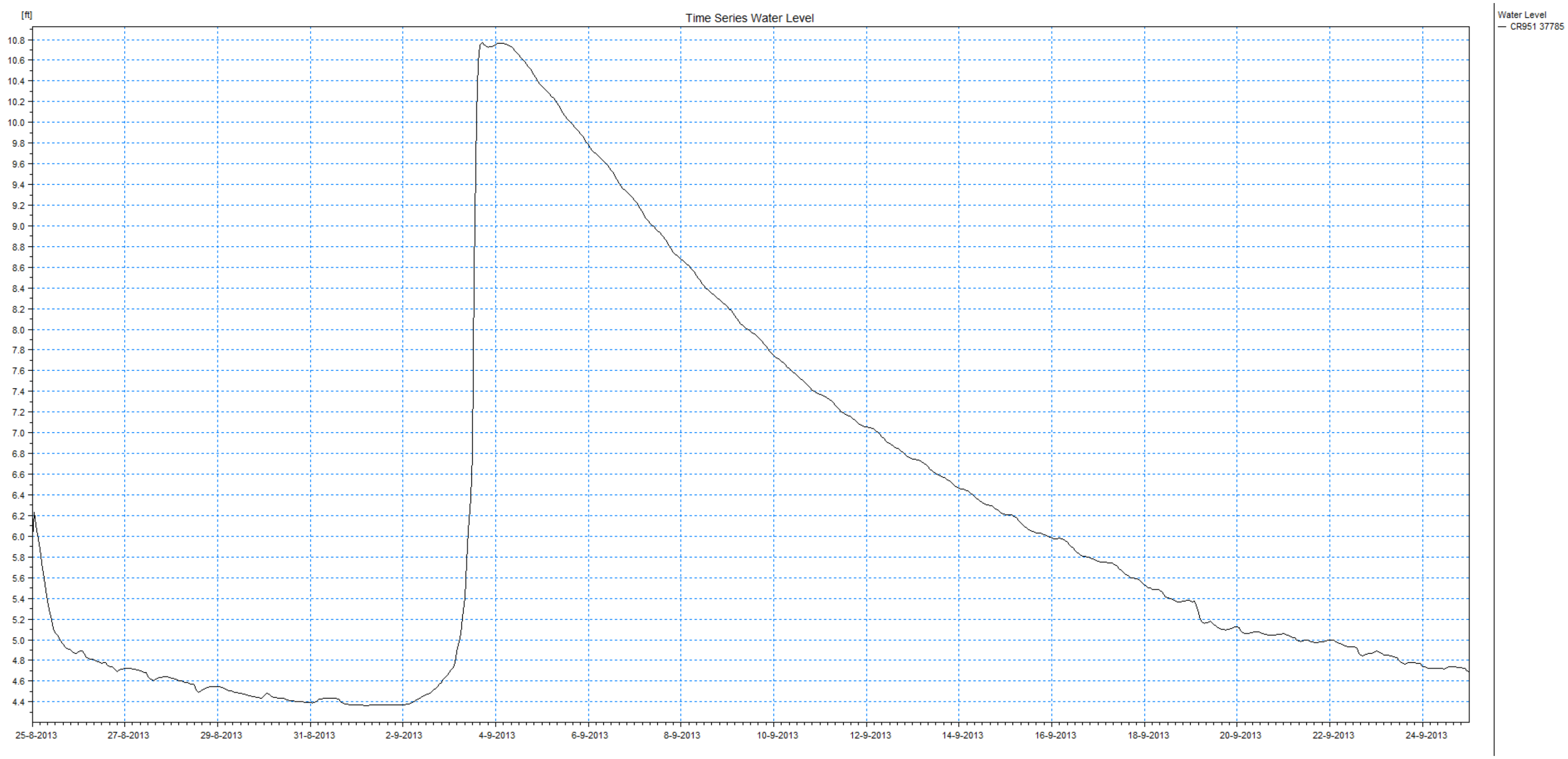


Figure 16. CR9511 Tailwater Stage (ft)

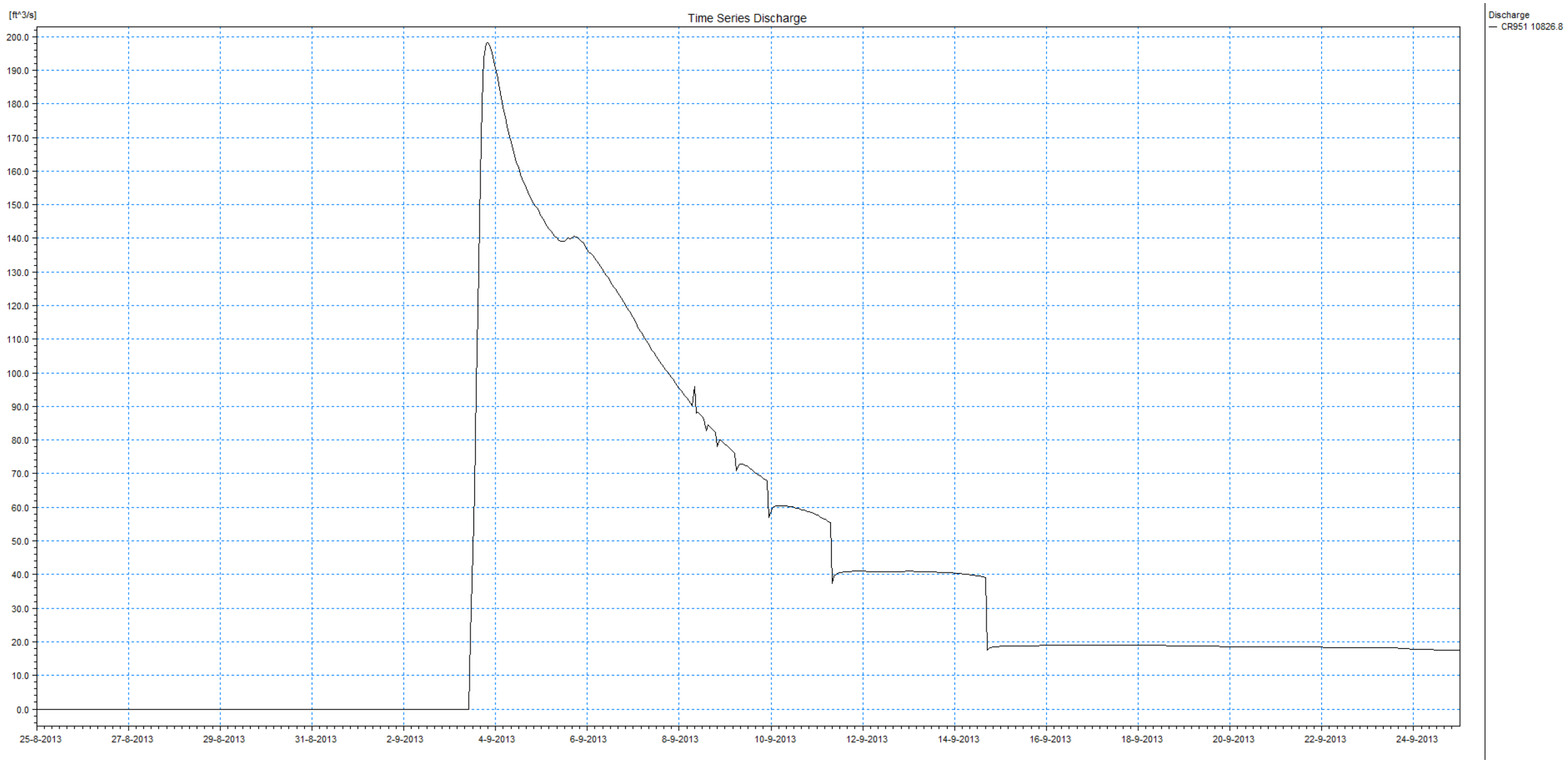


Figure 17. CR9512 Discharge (cfs)

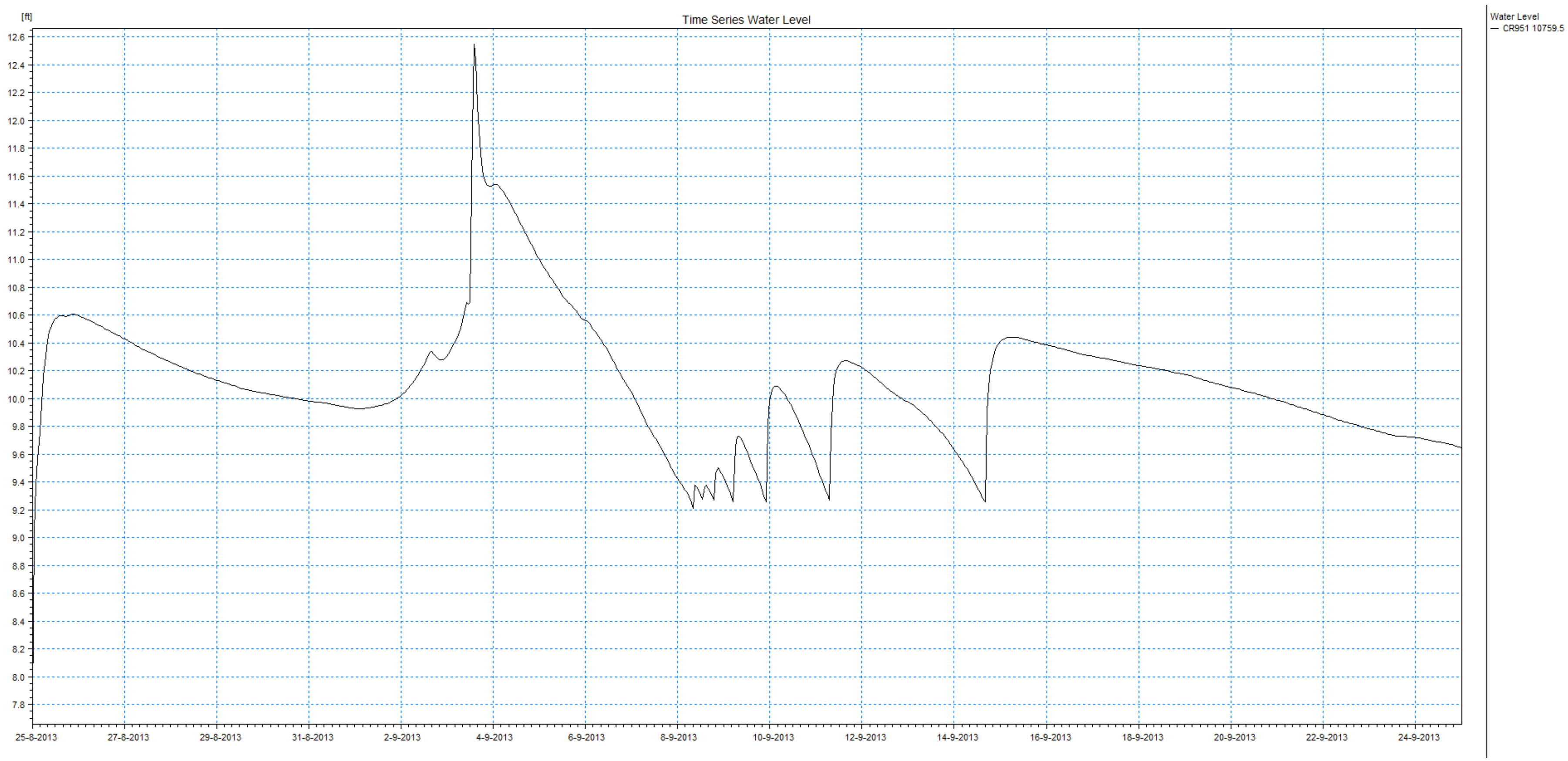
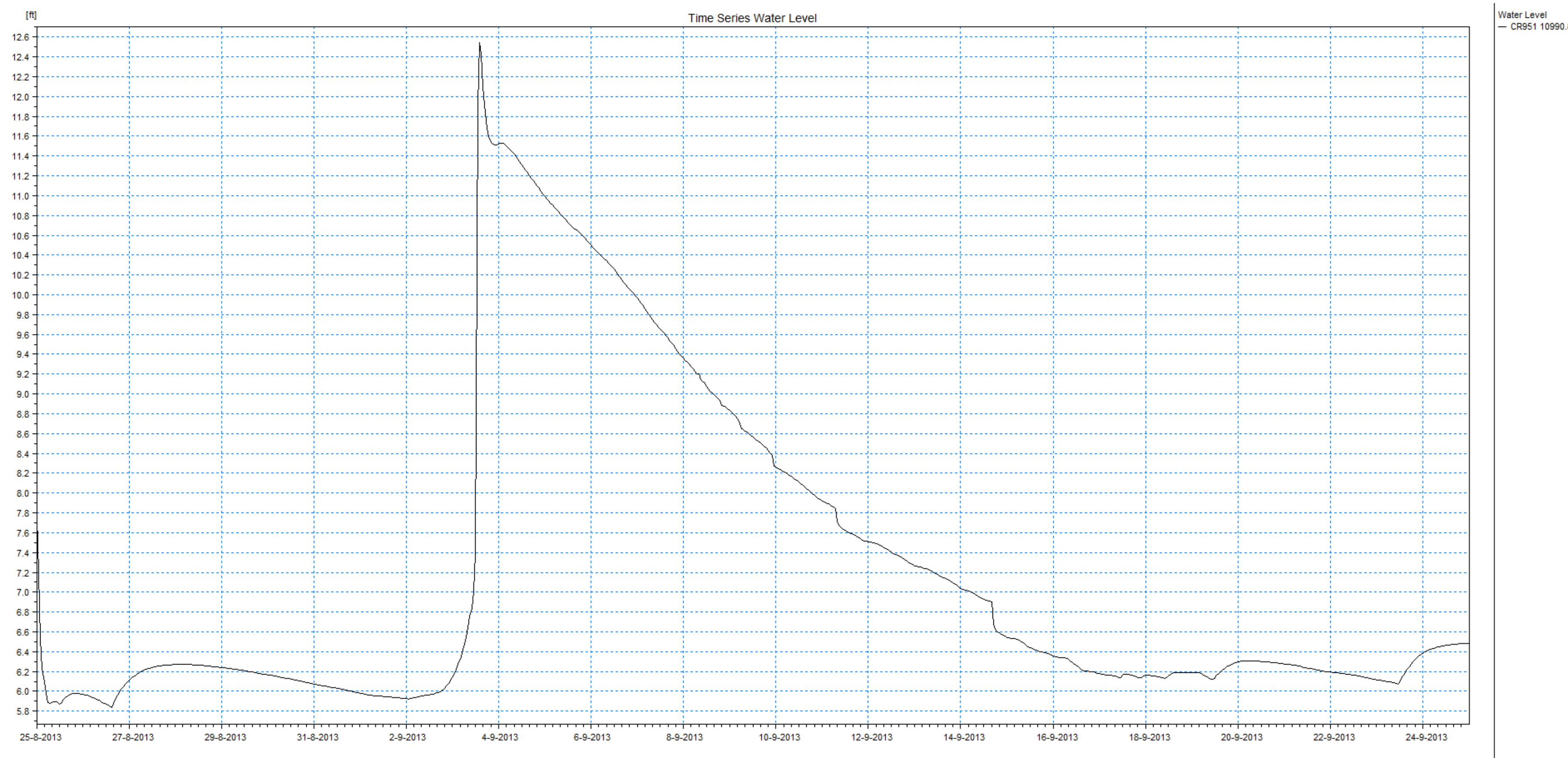


Figure 18. CR9512 Headwater Stage (ft)



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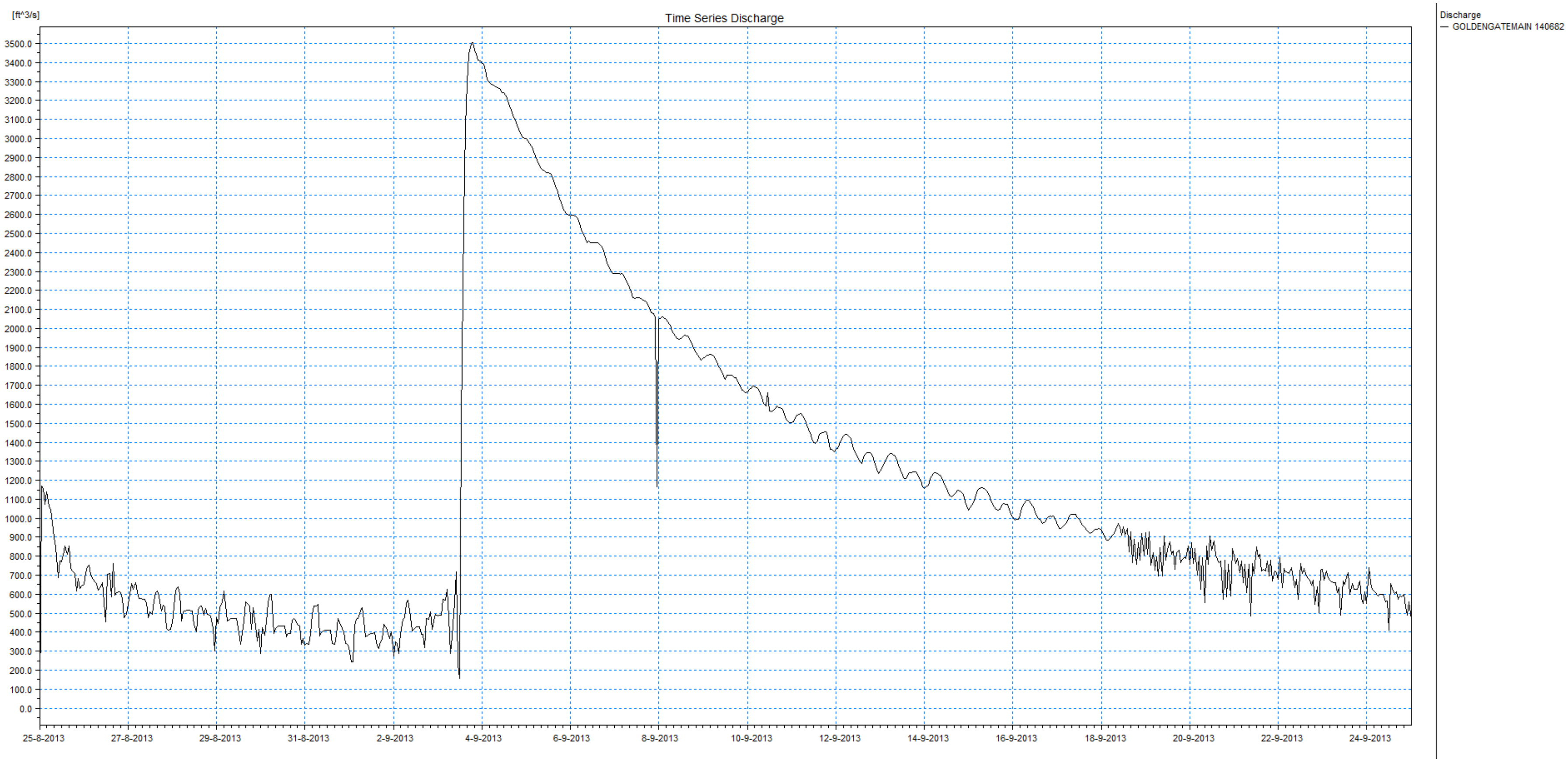

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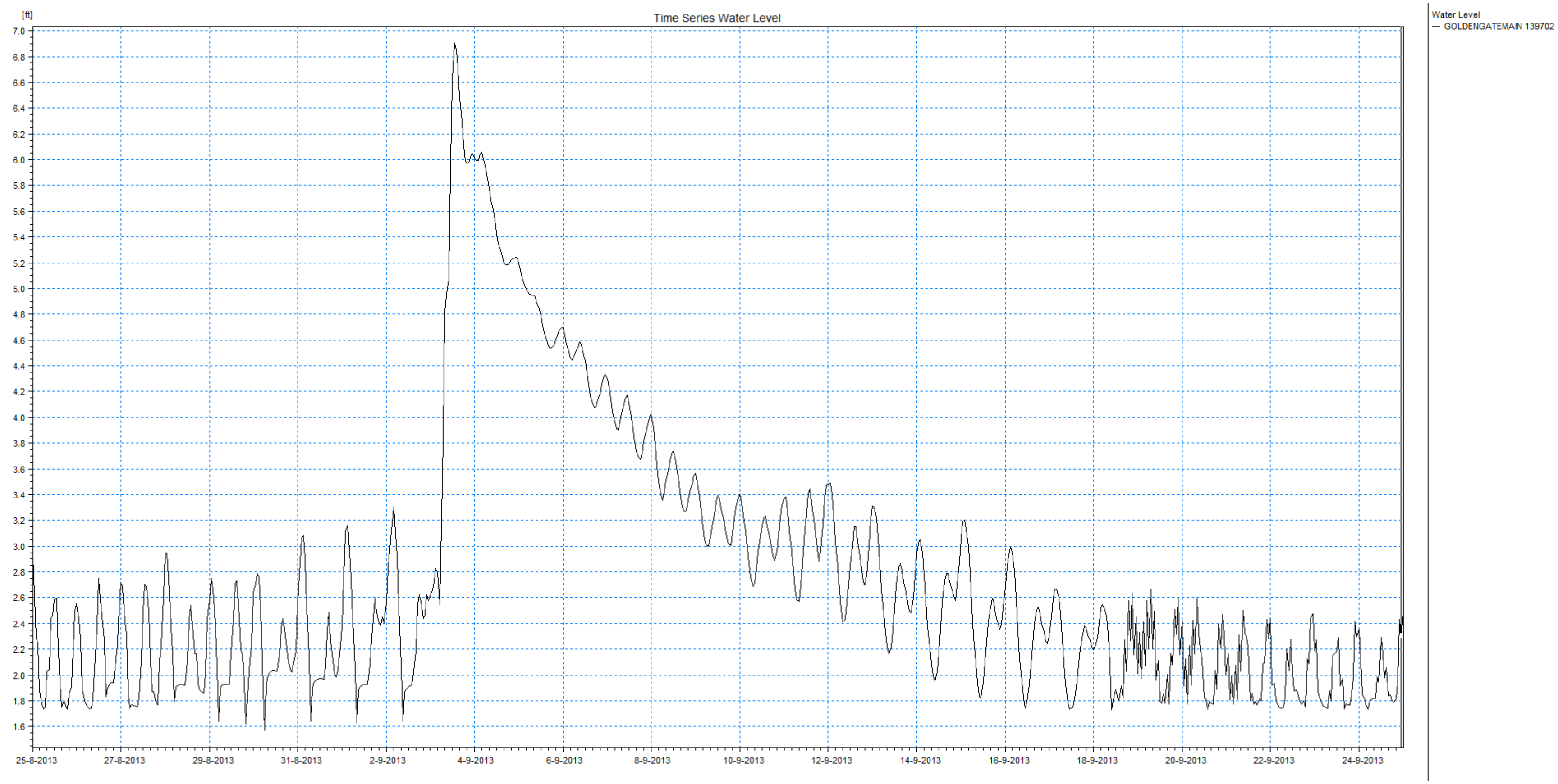

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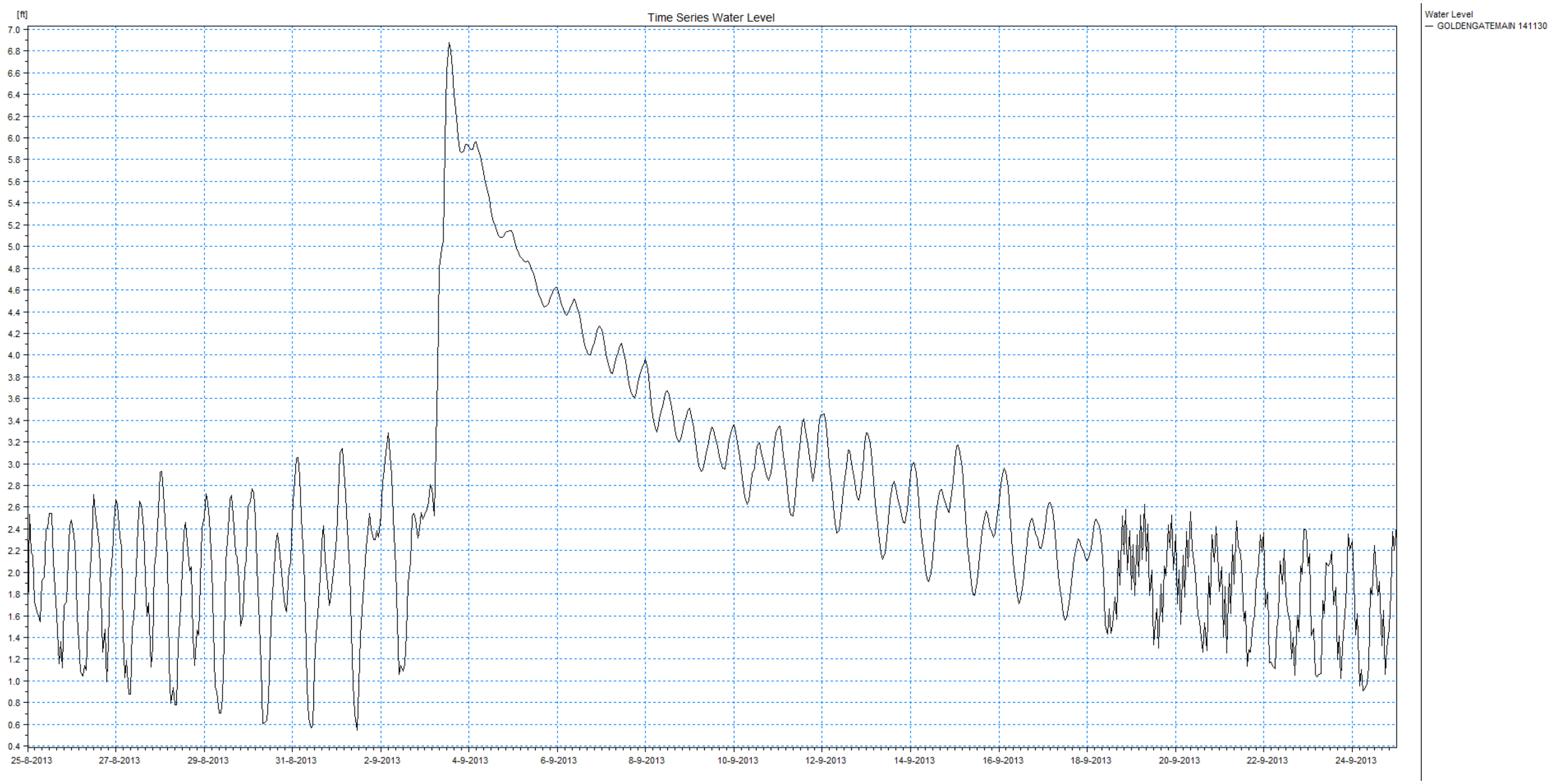

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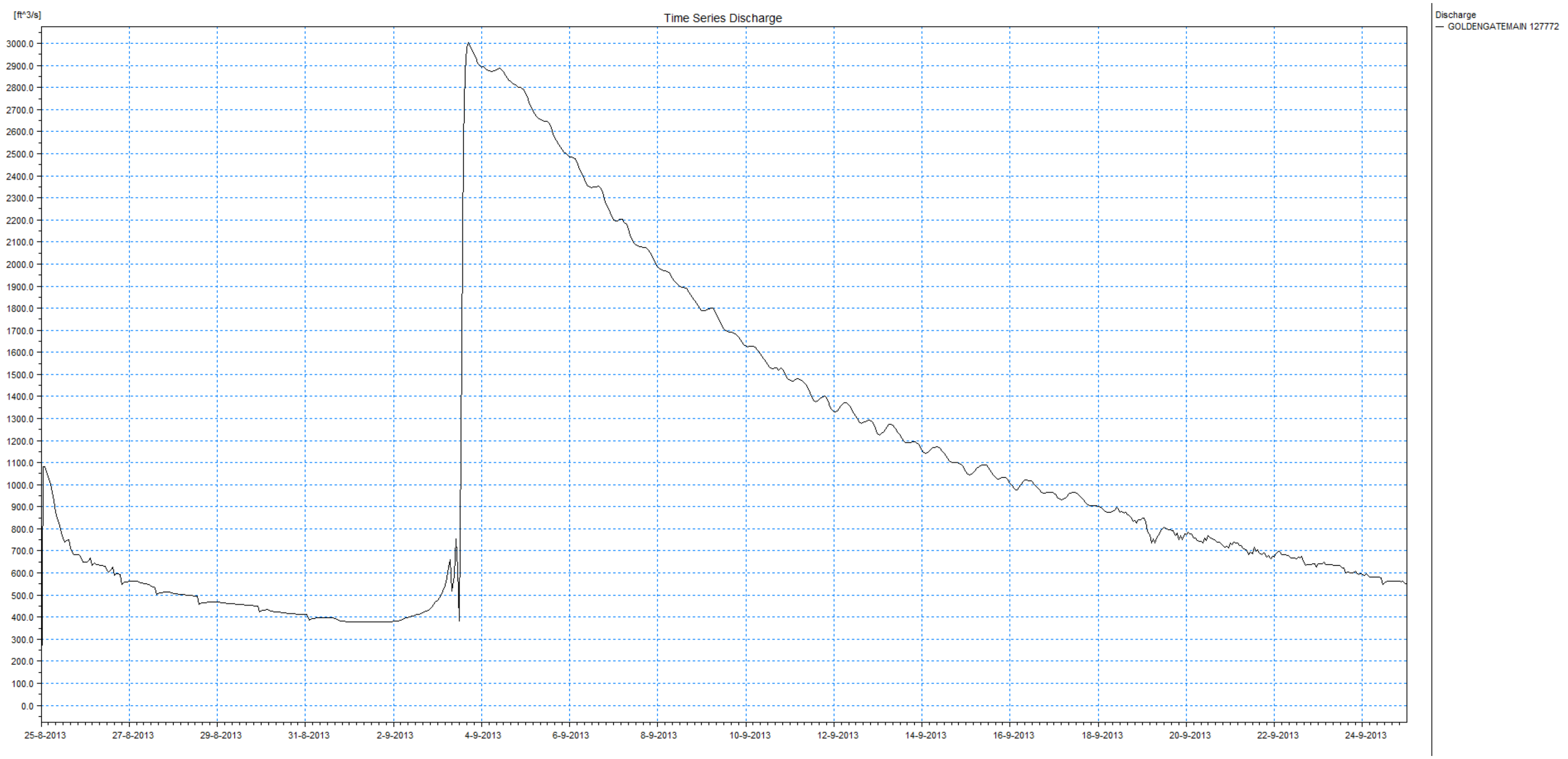

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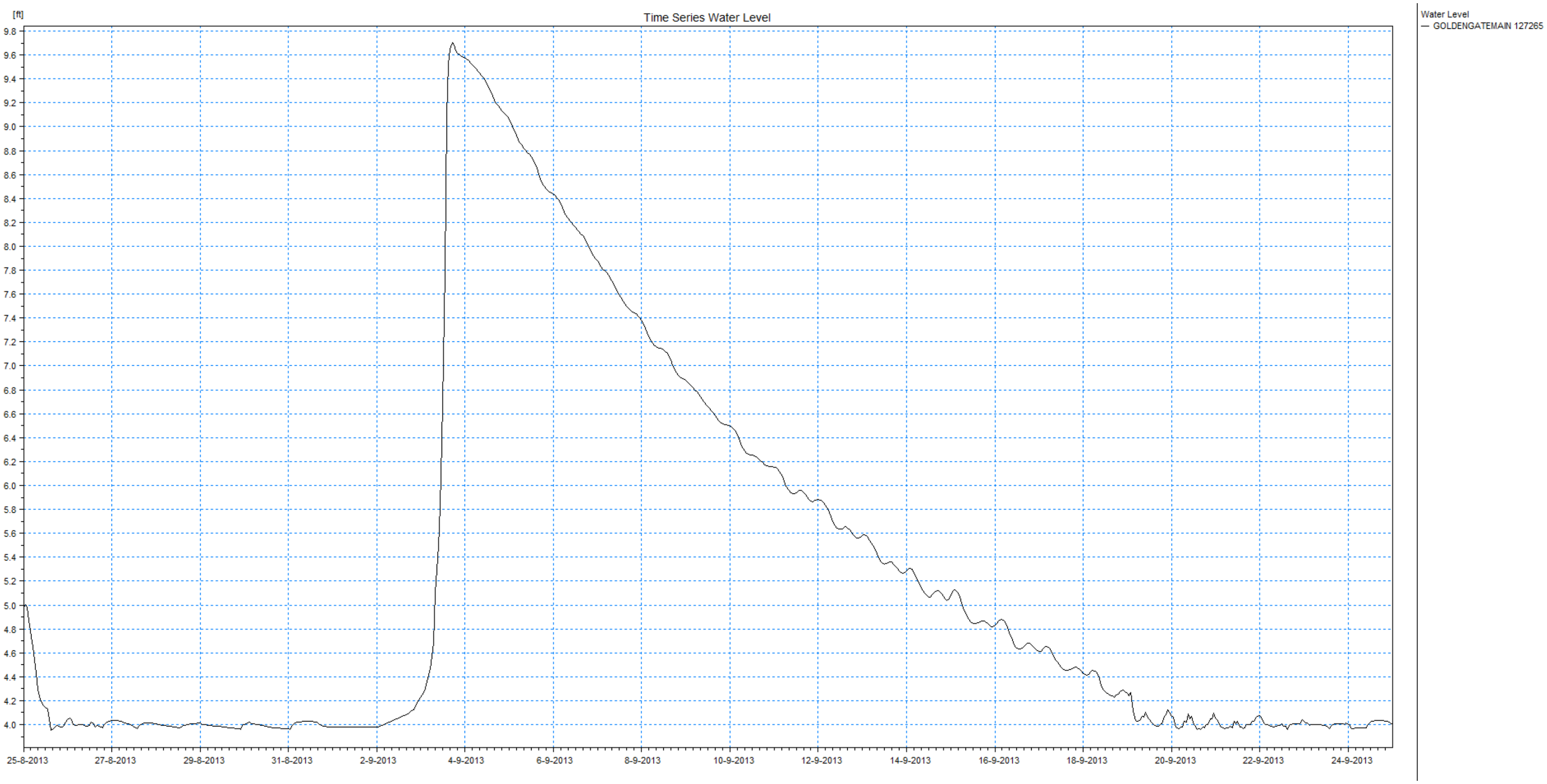

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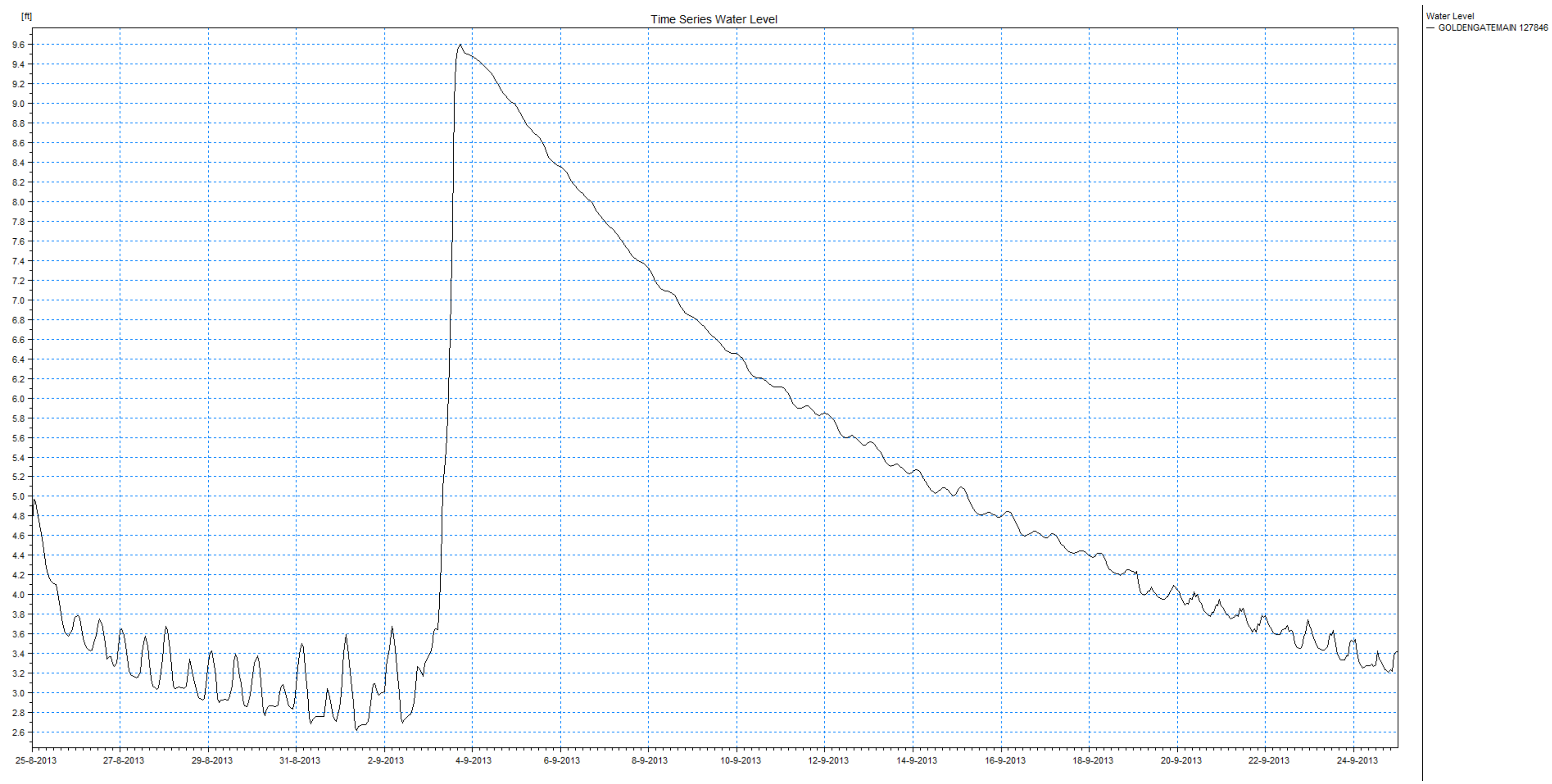

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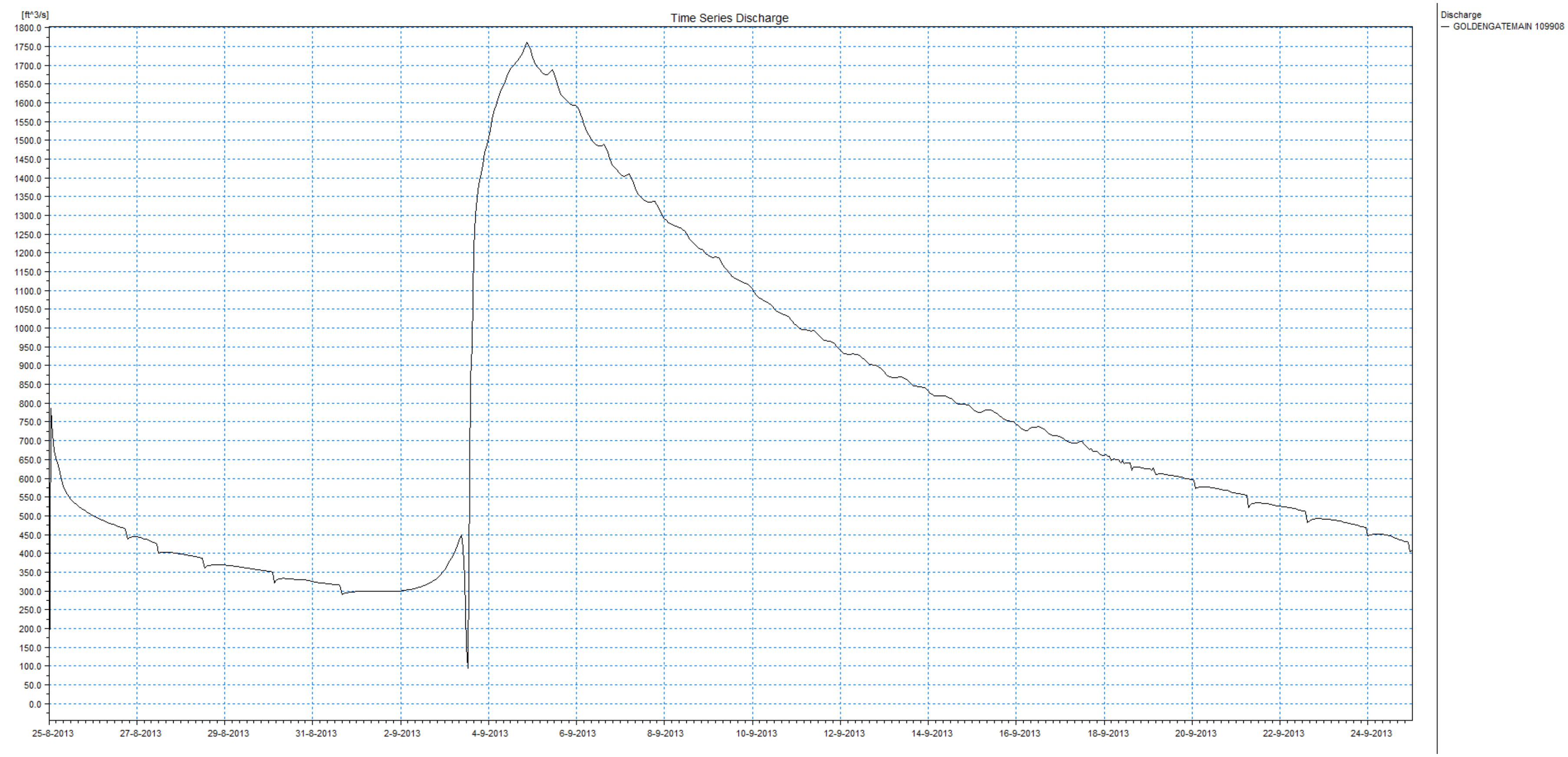

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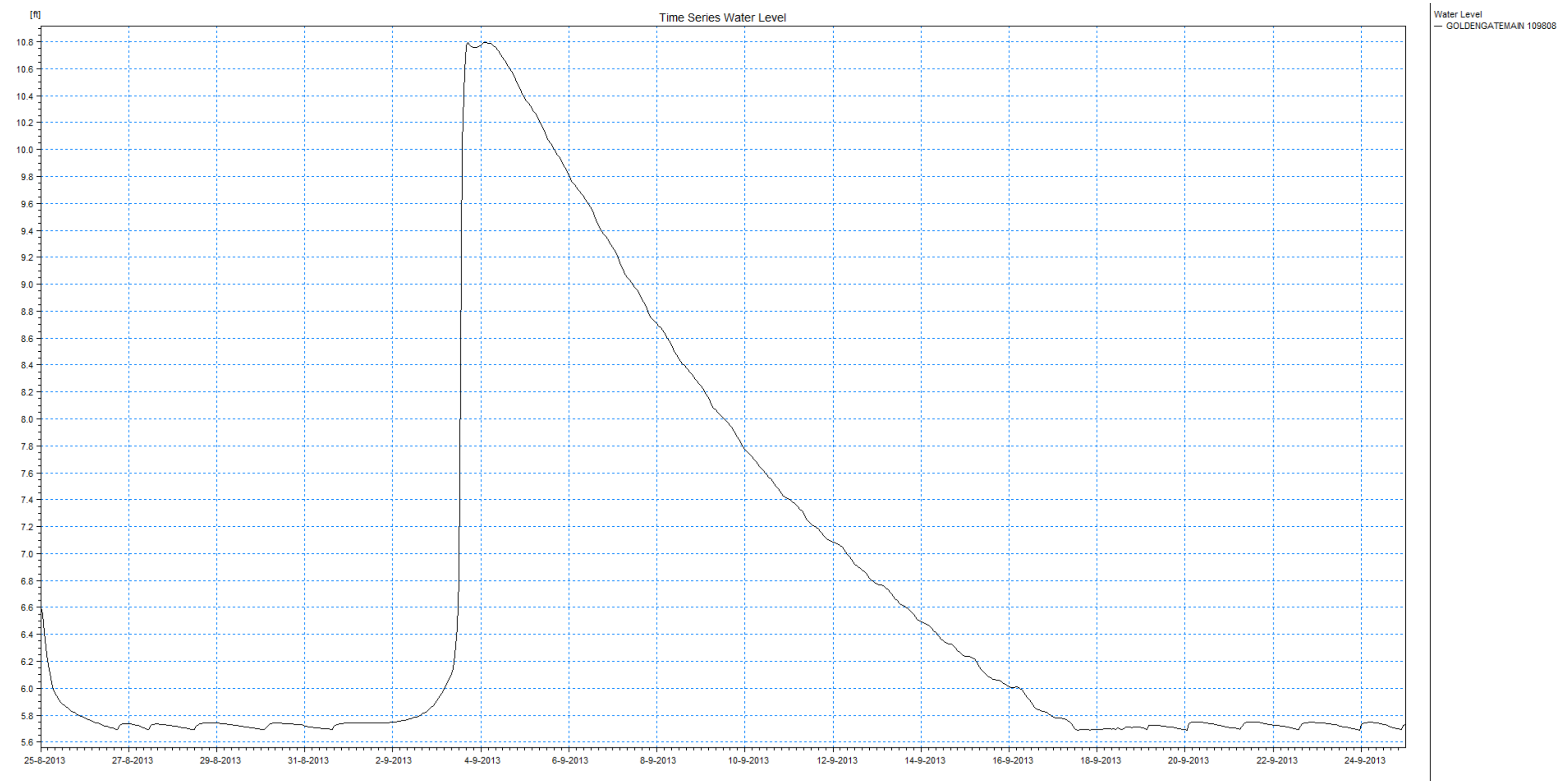

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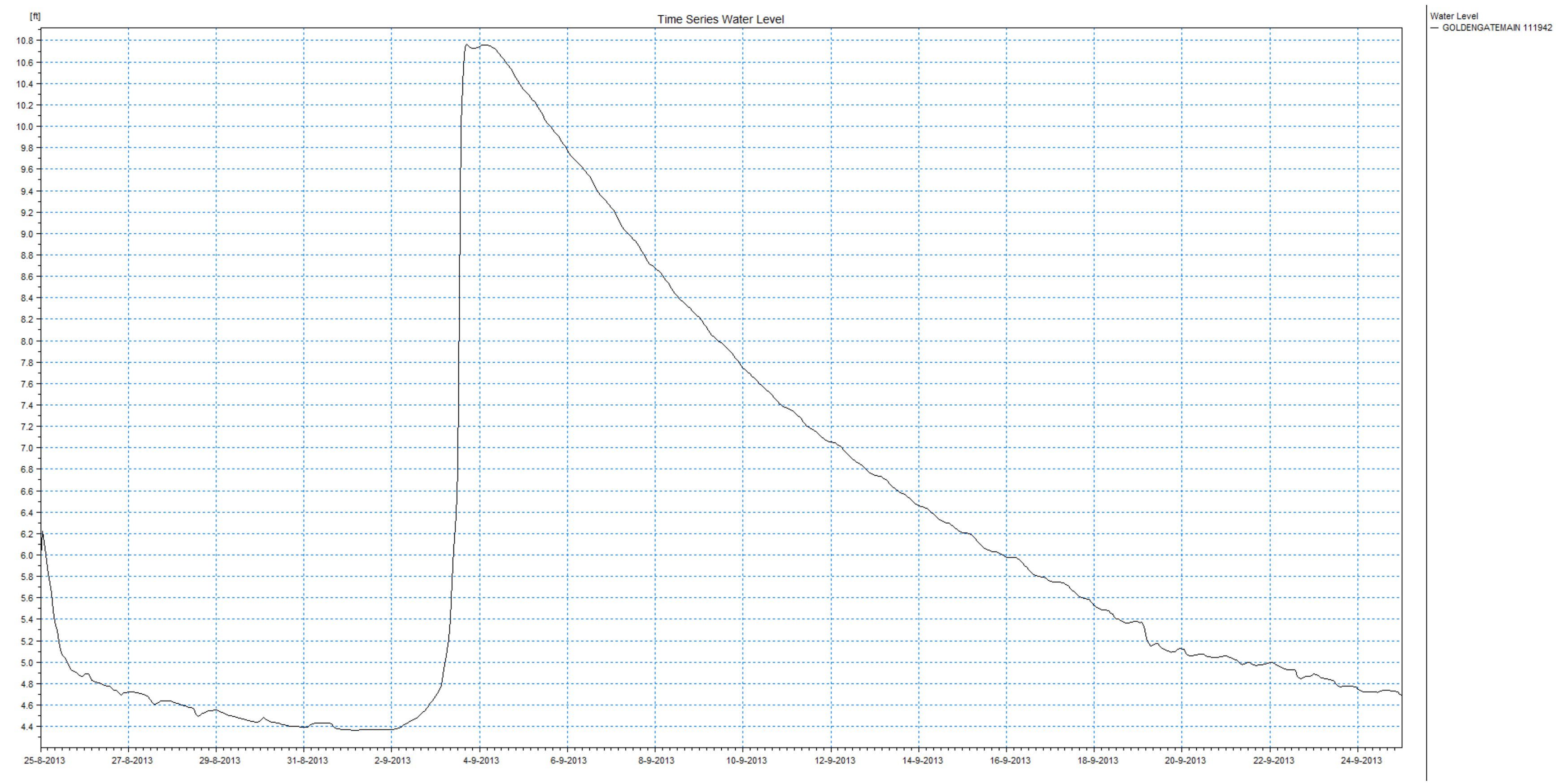

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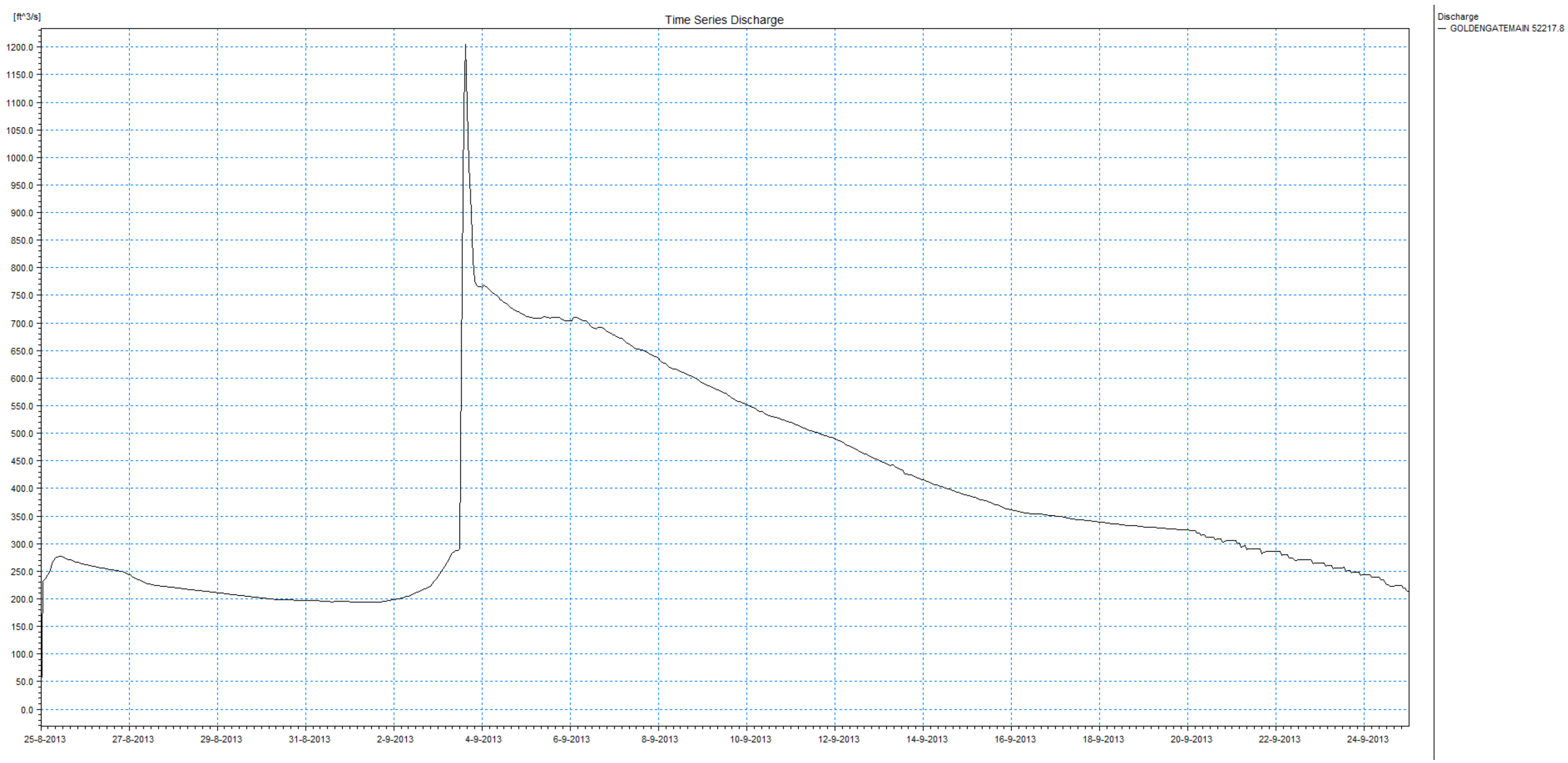
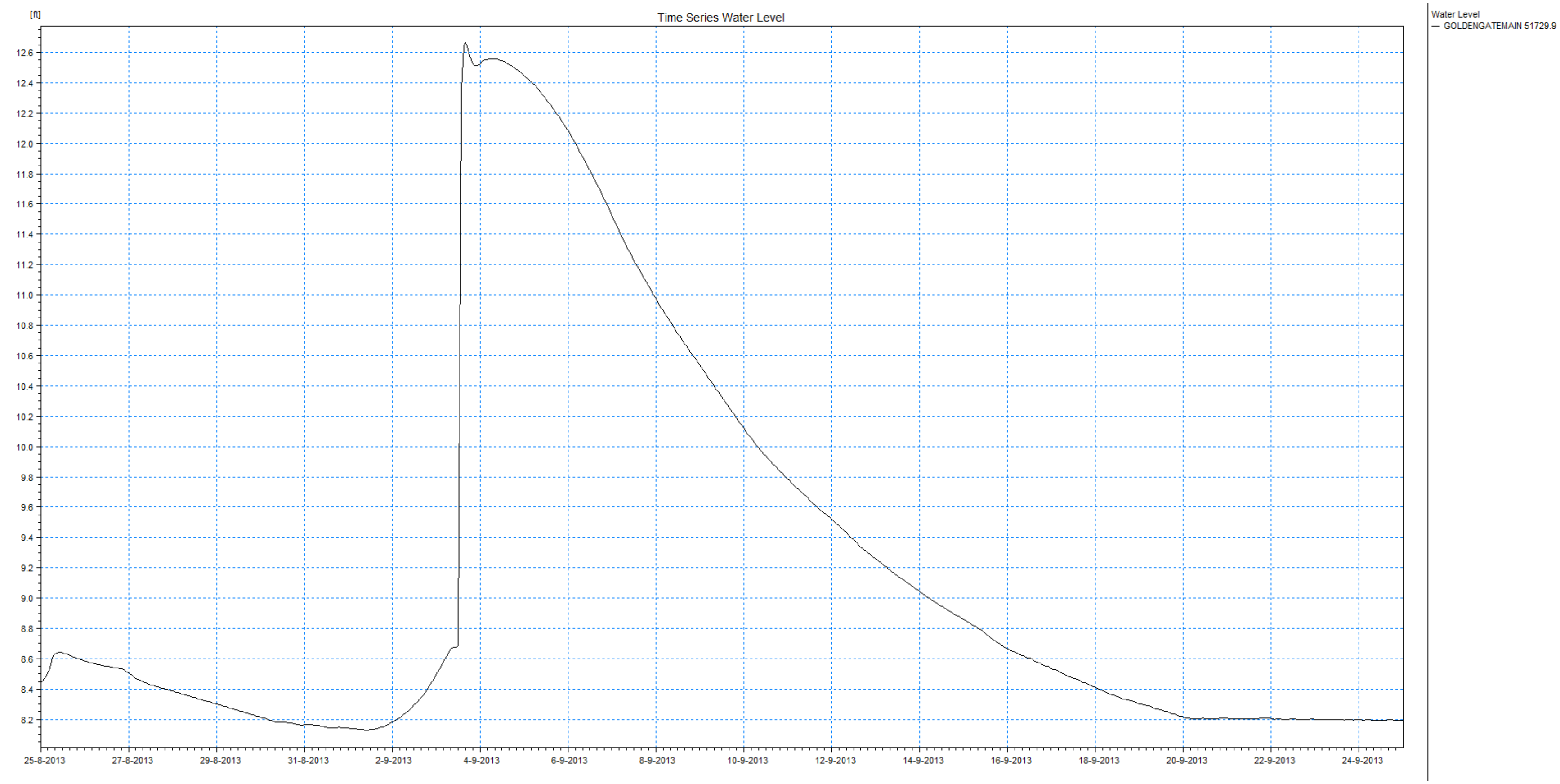


Figure 29. GG 4 Discharge (cfs)



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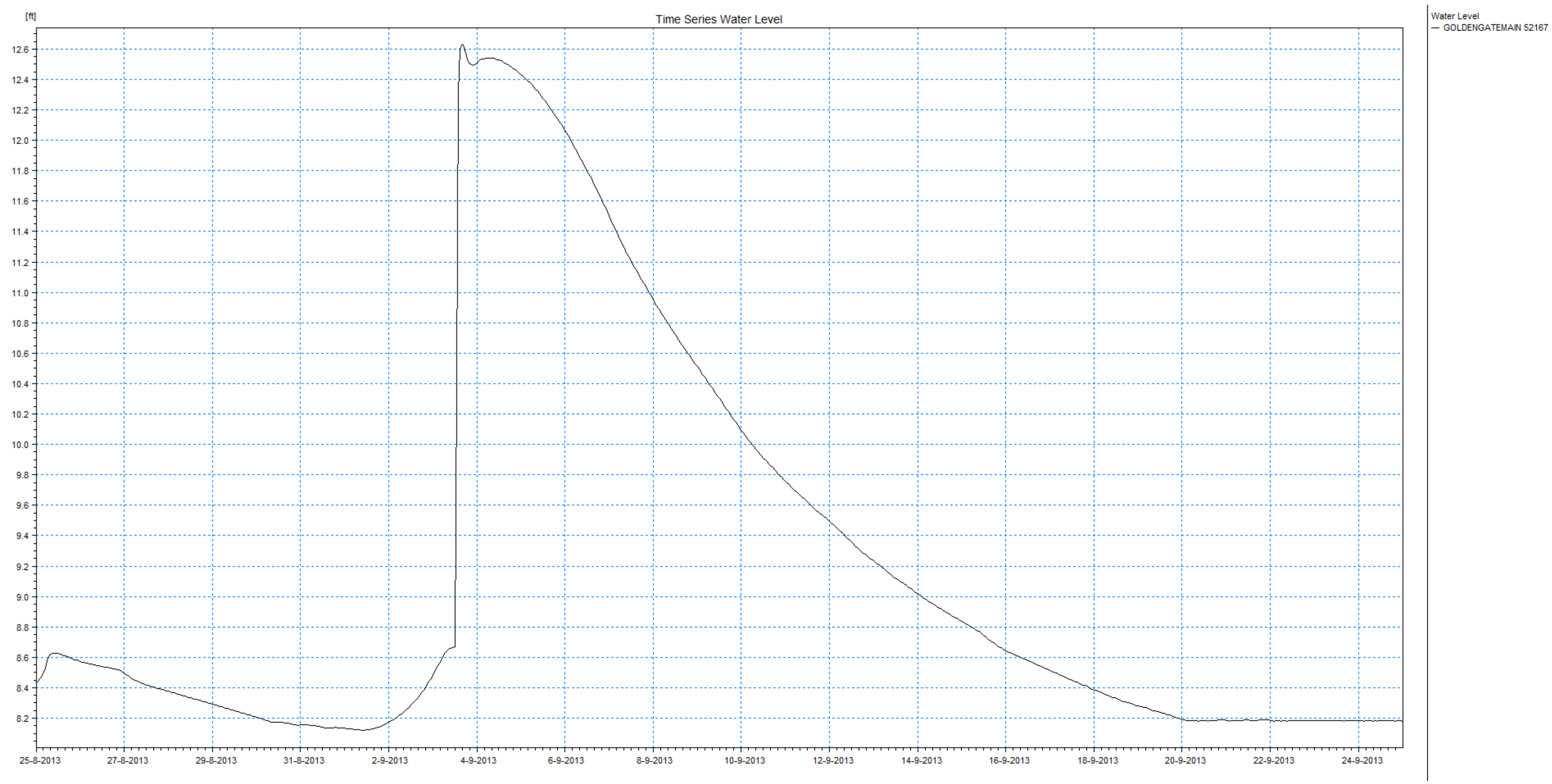
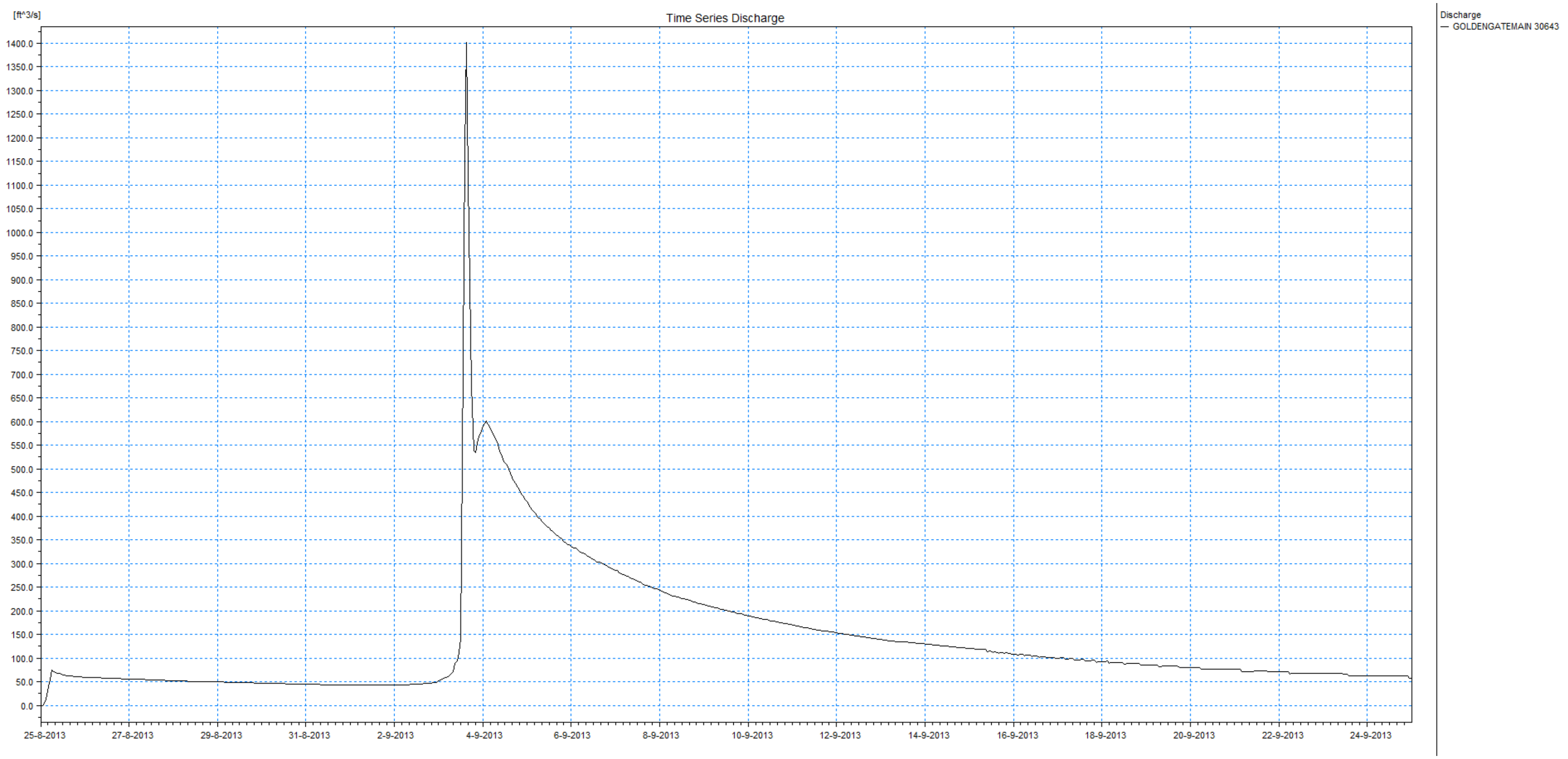


Figure 31. GG 4 Tailwater Stage (ft)



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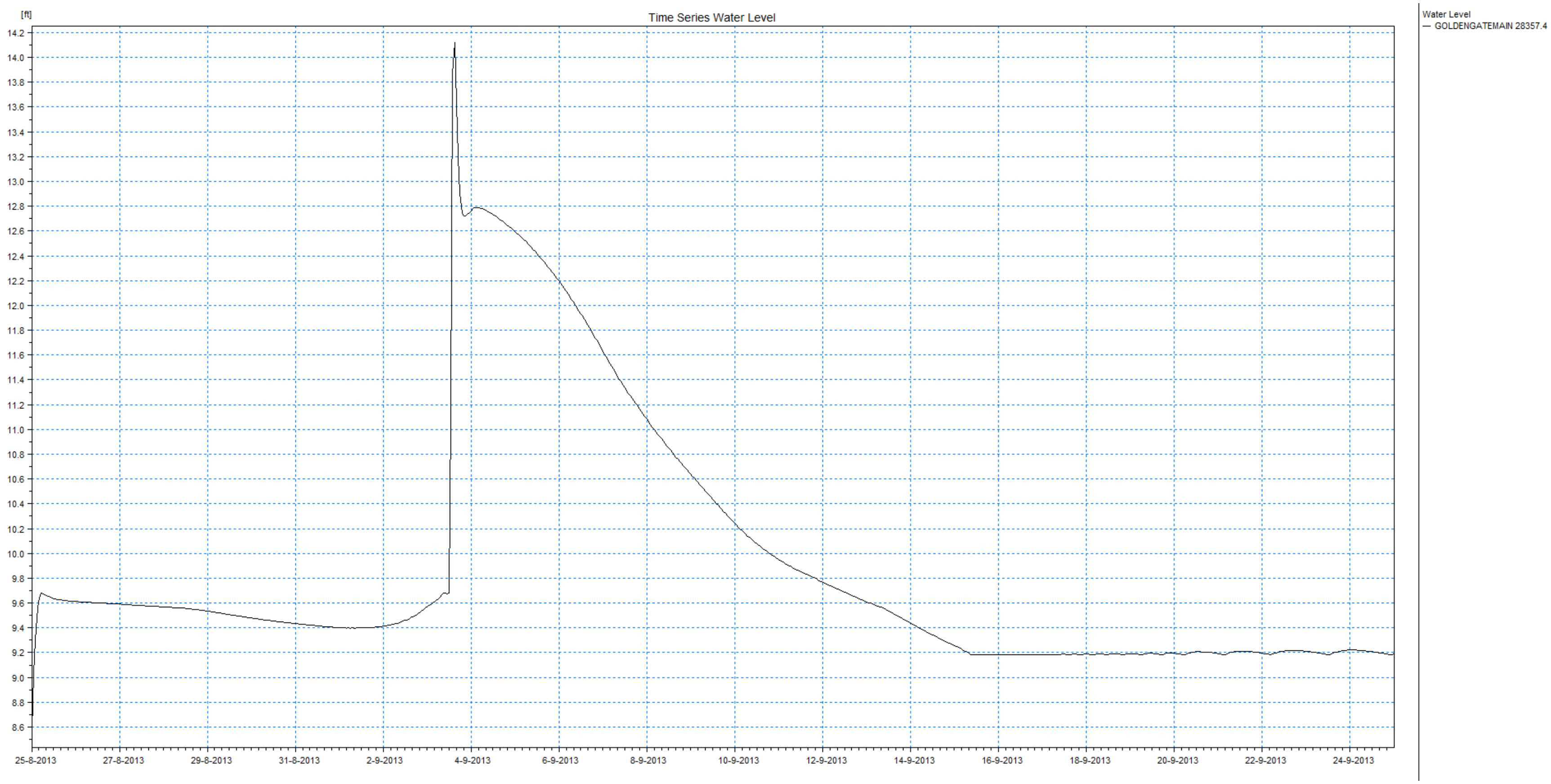

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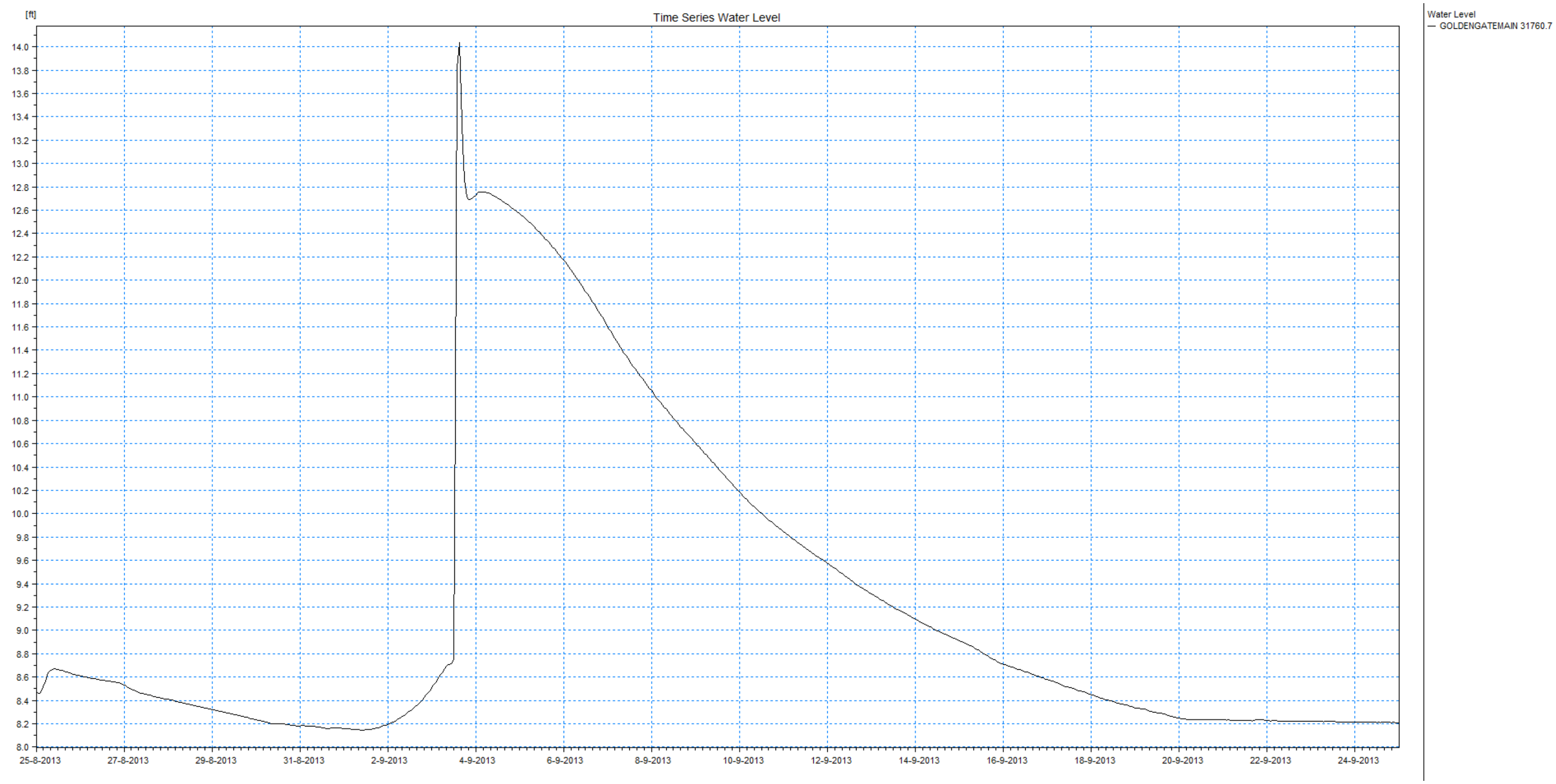

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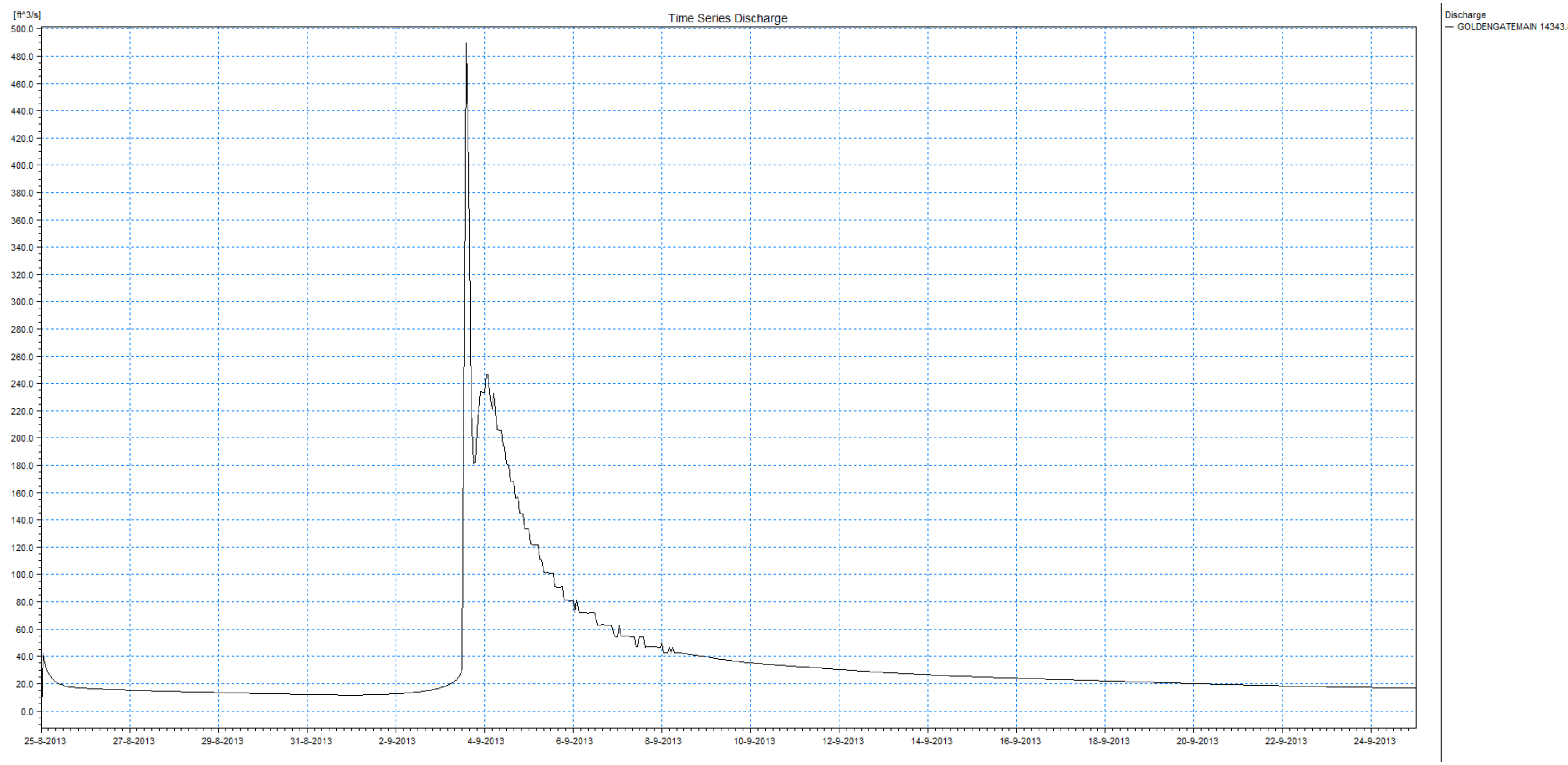

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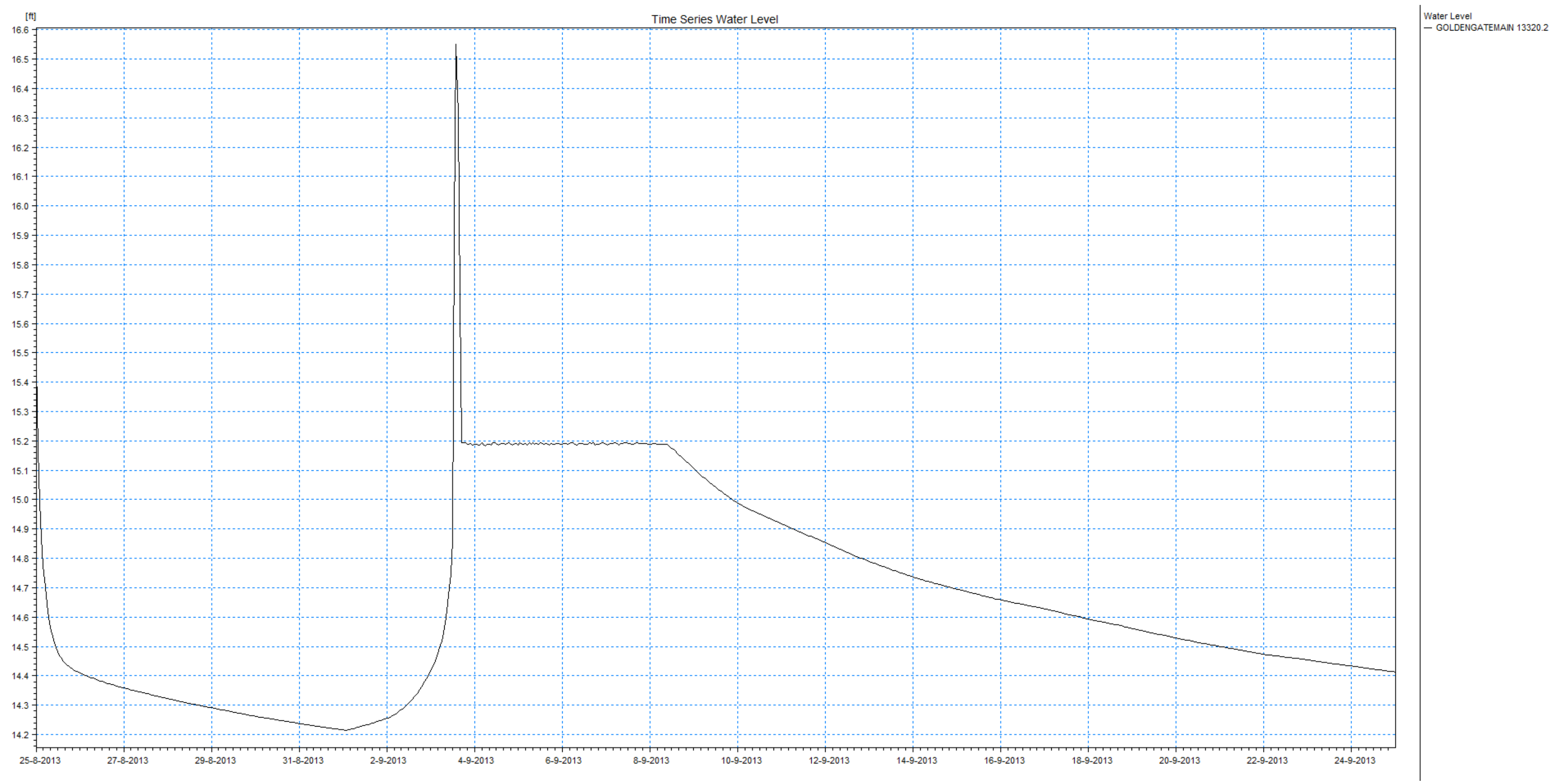

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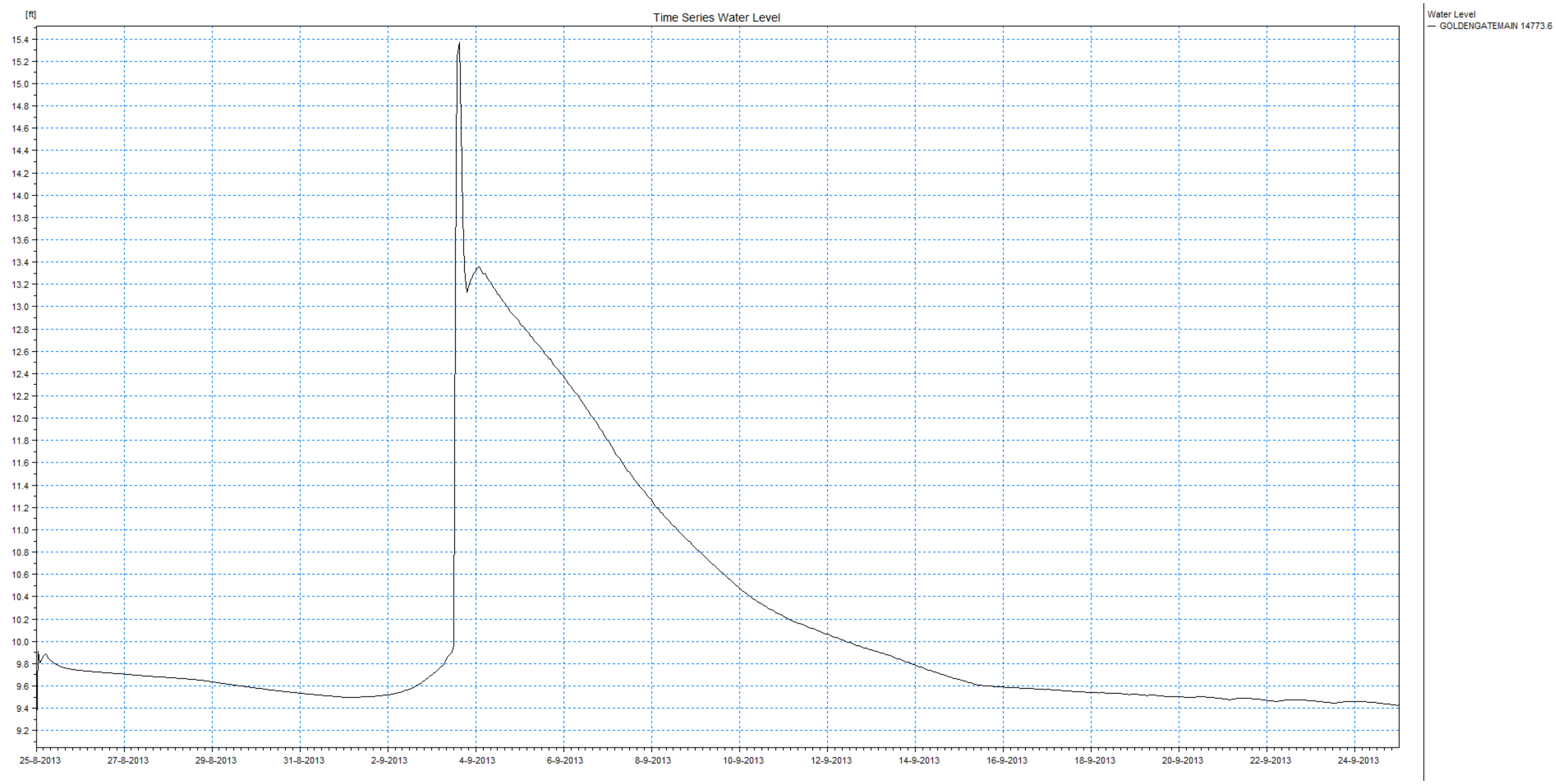

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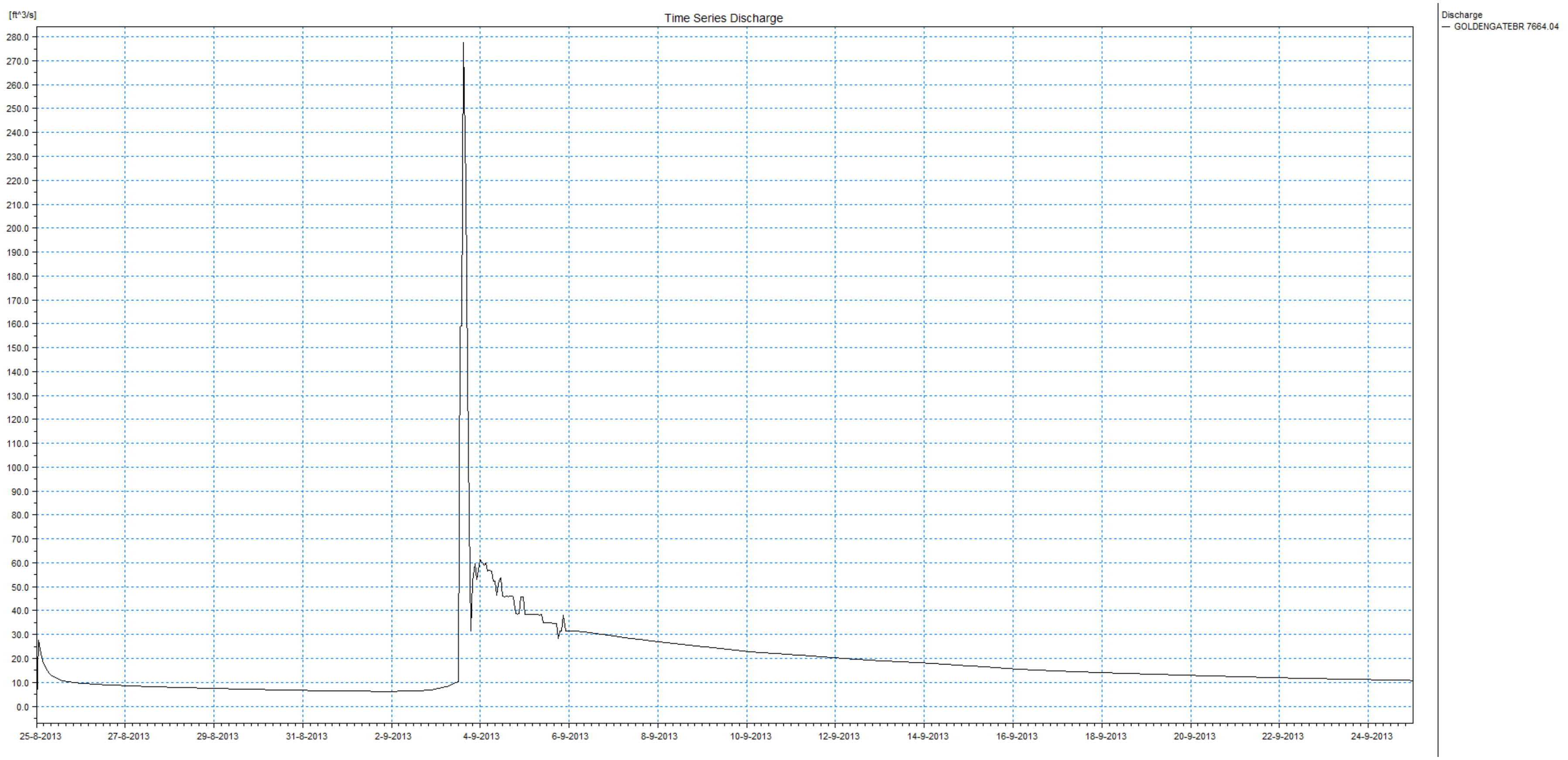

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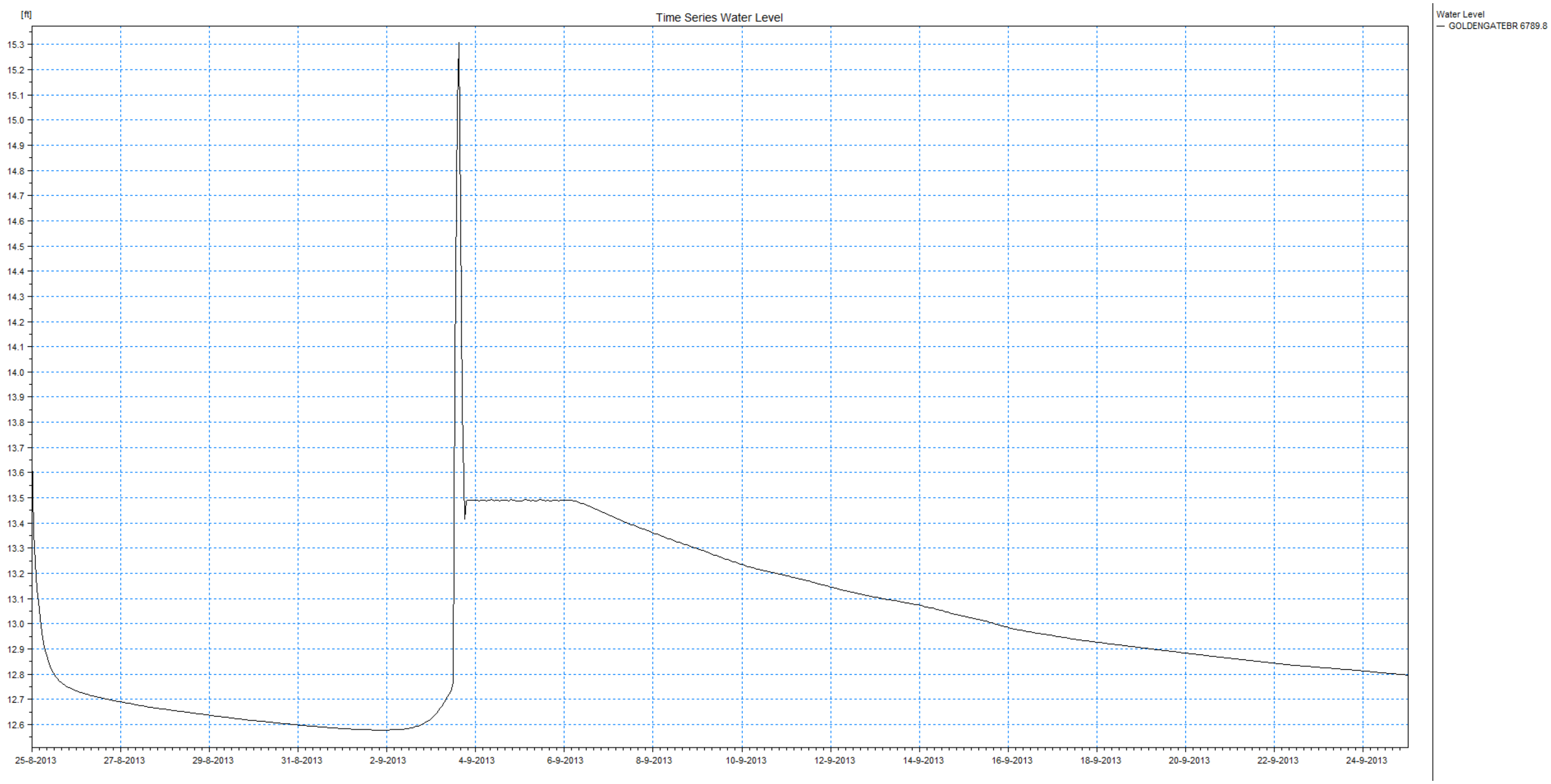

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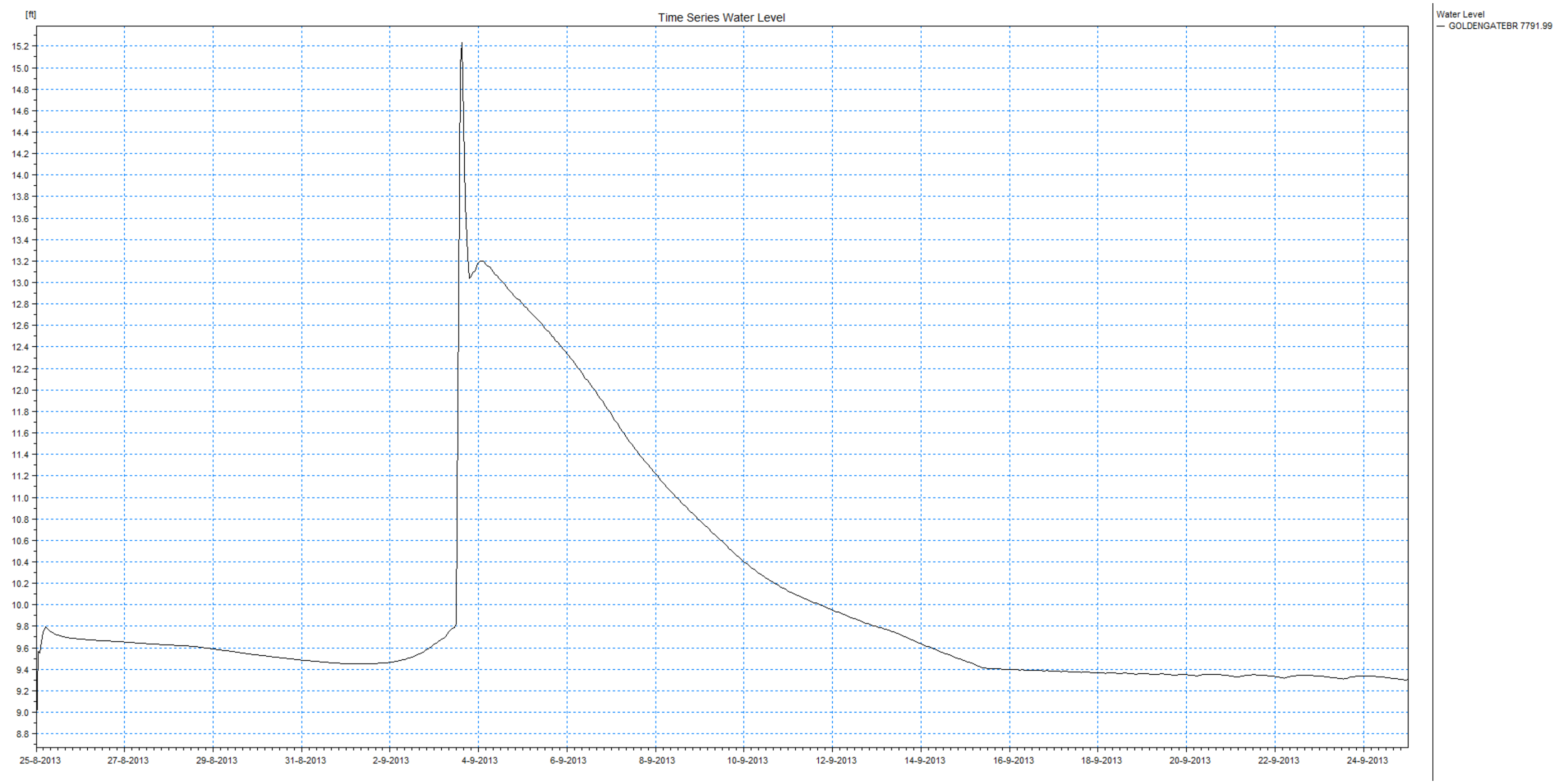

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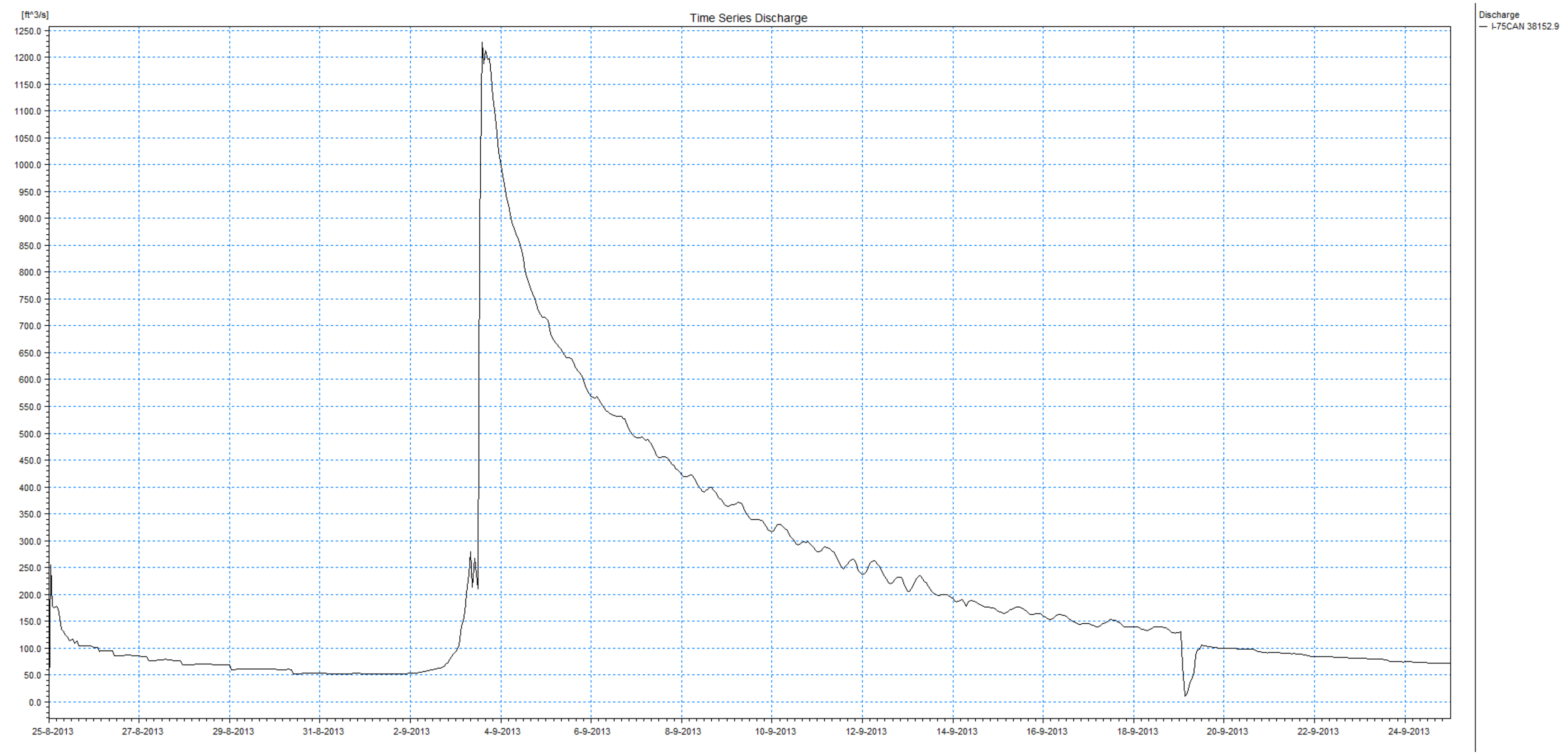
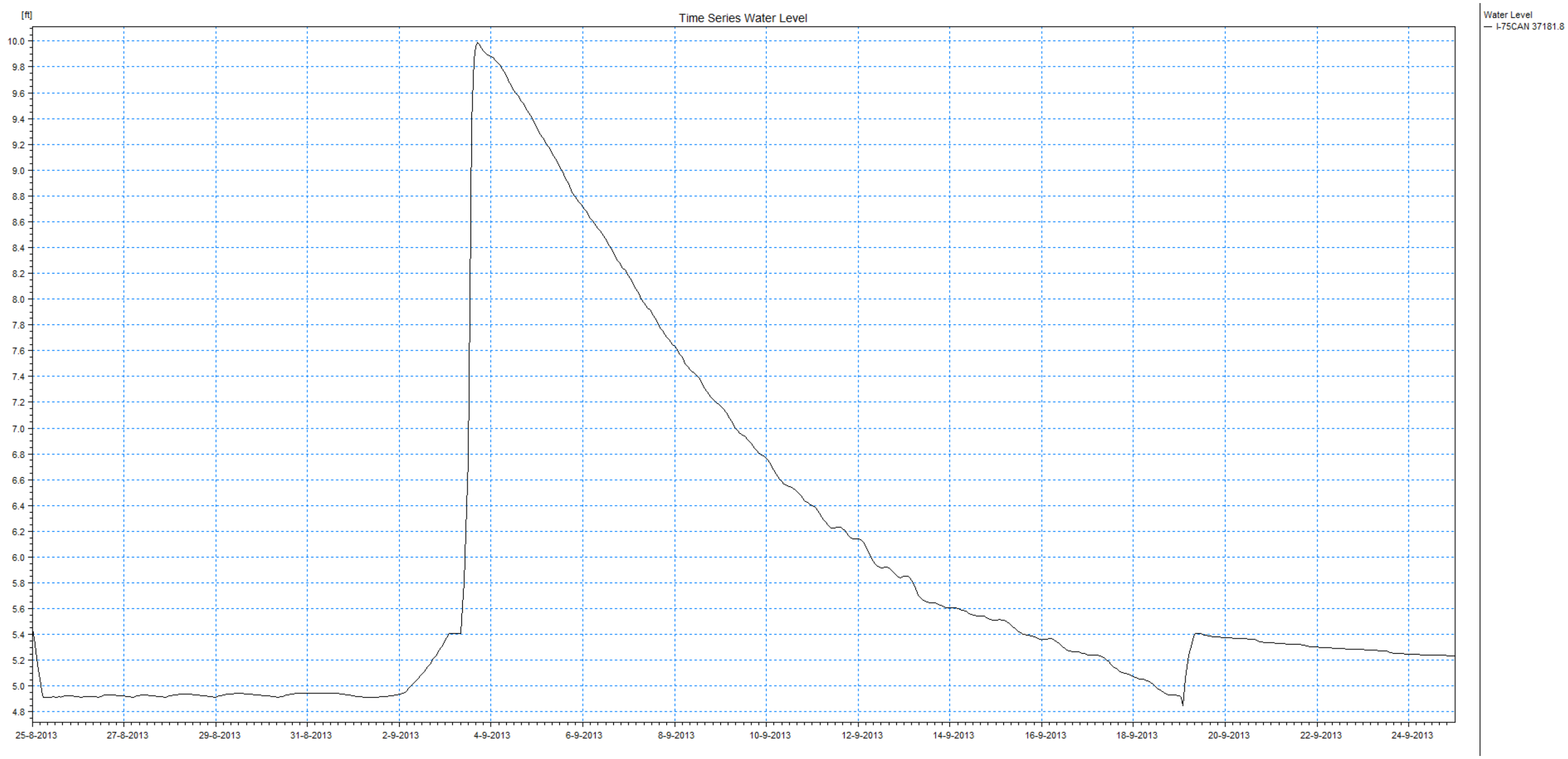


Figure 41. I75W1 Discharge (cfs)



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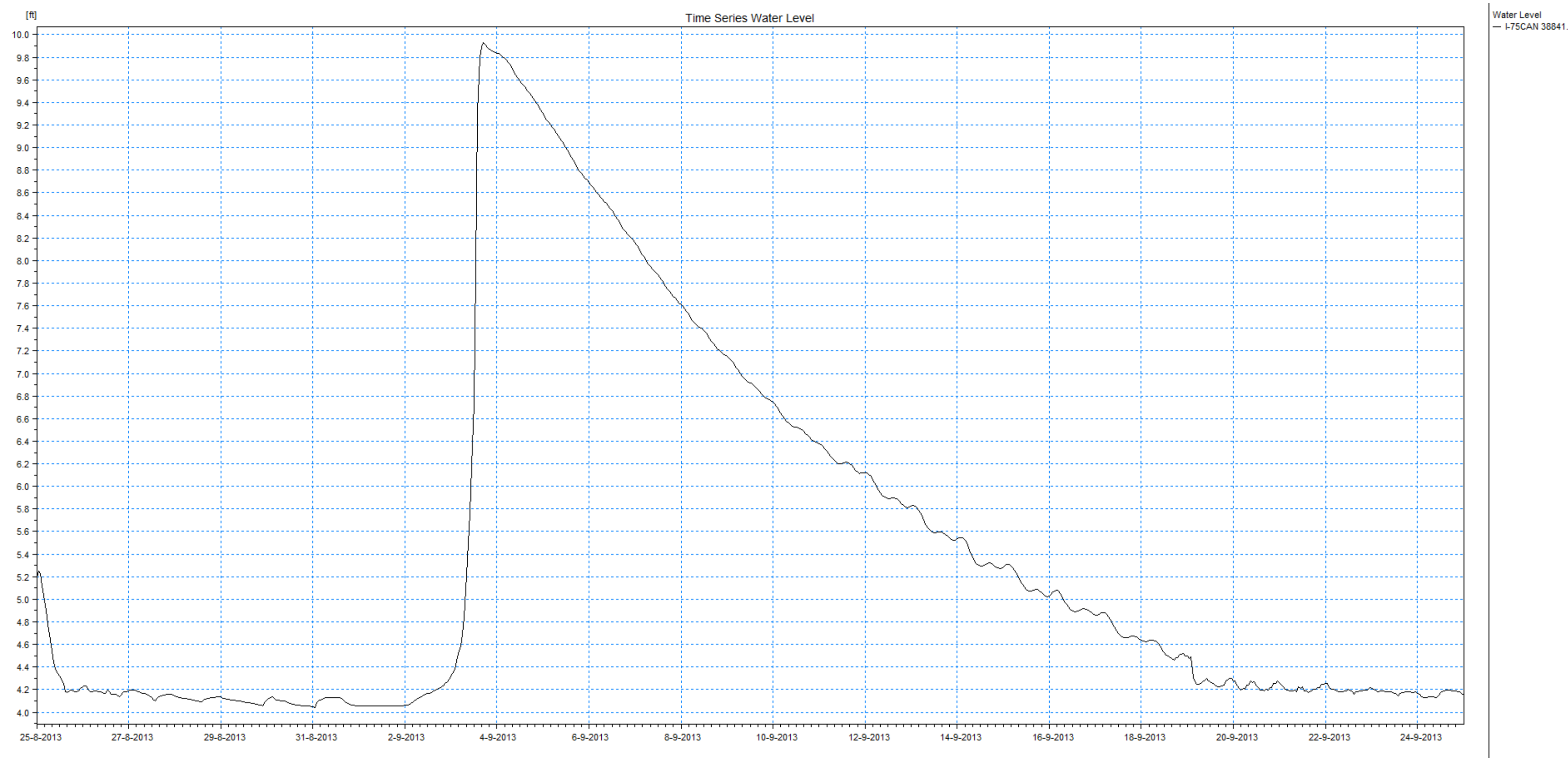

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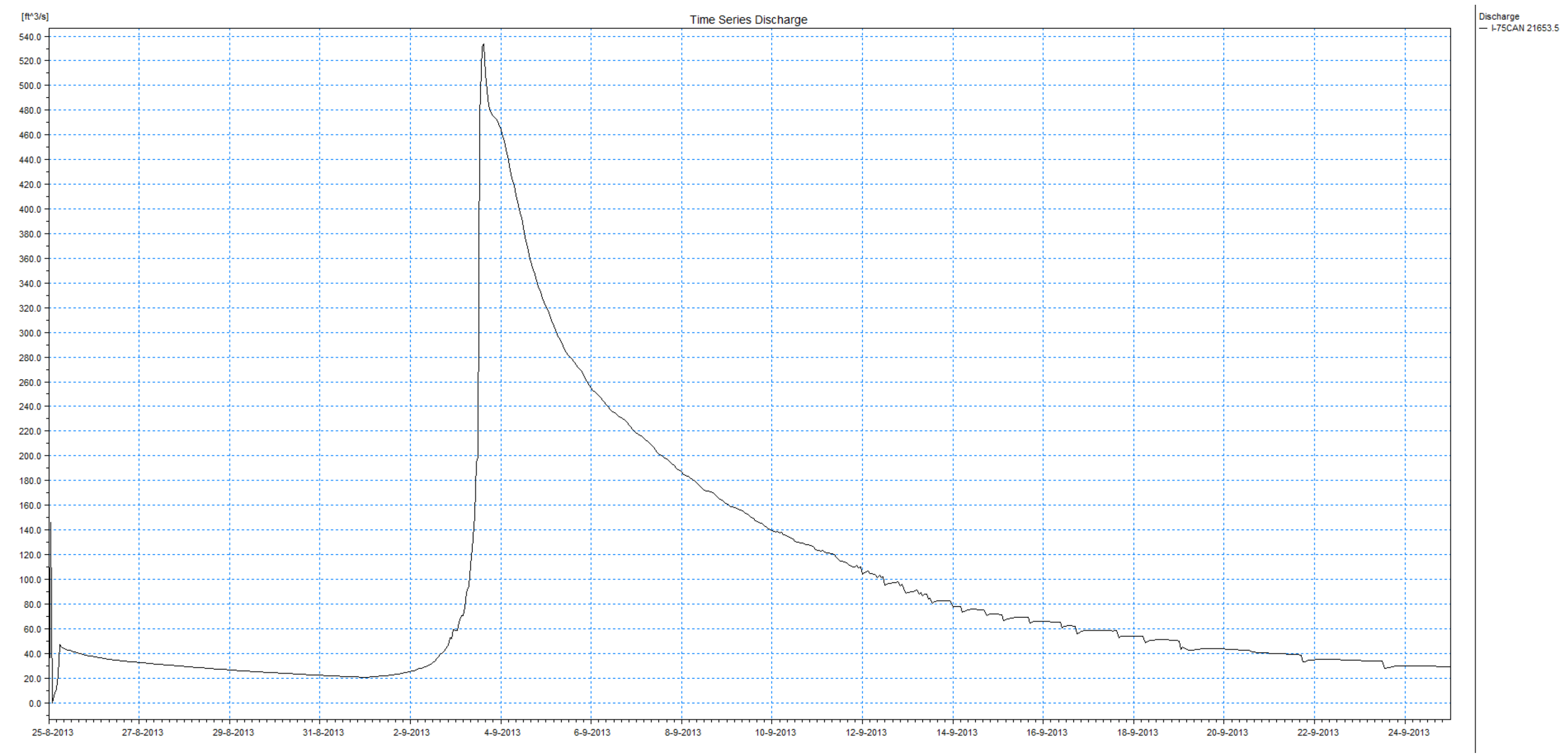

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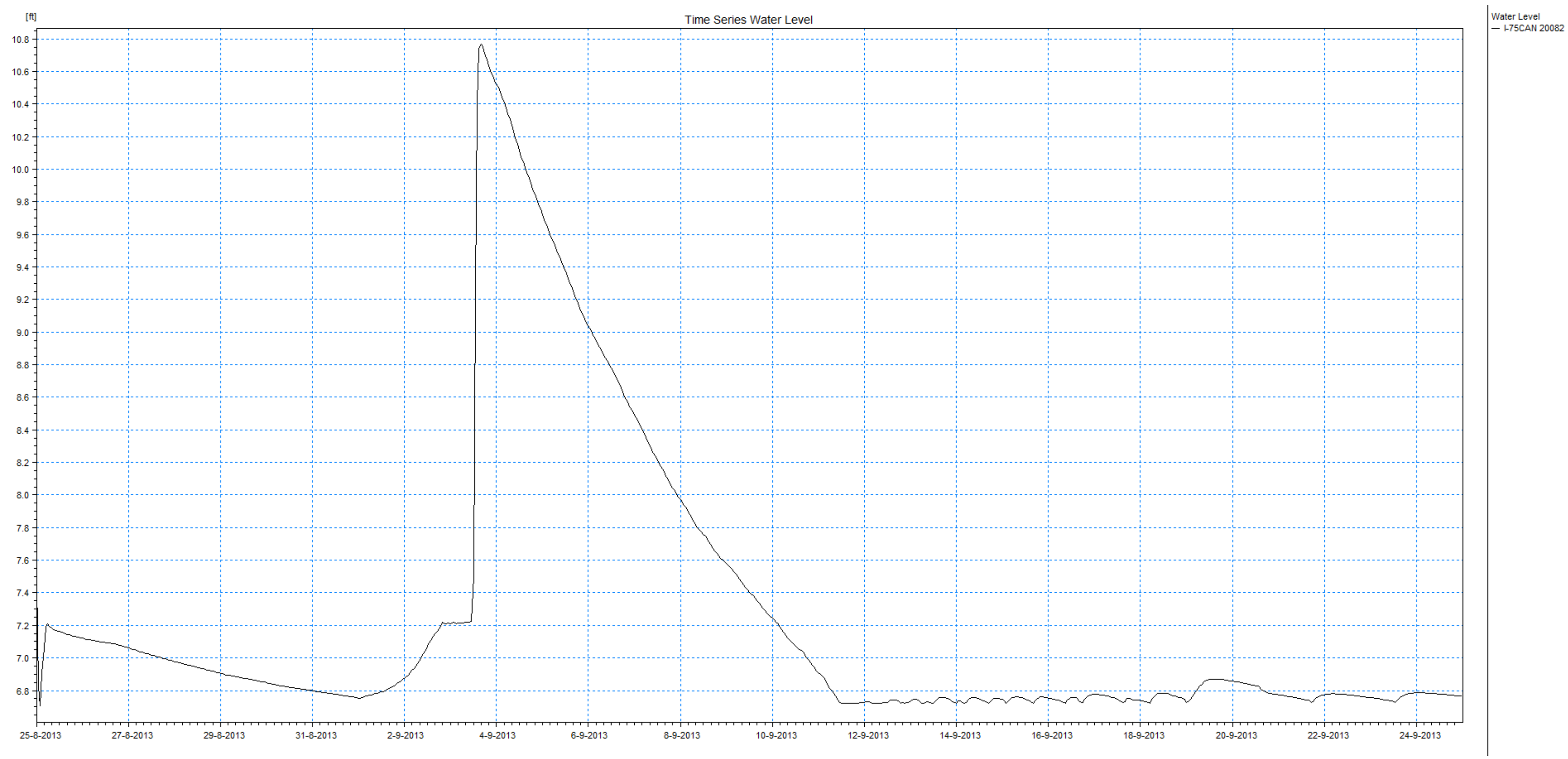

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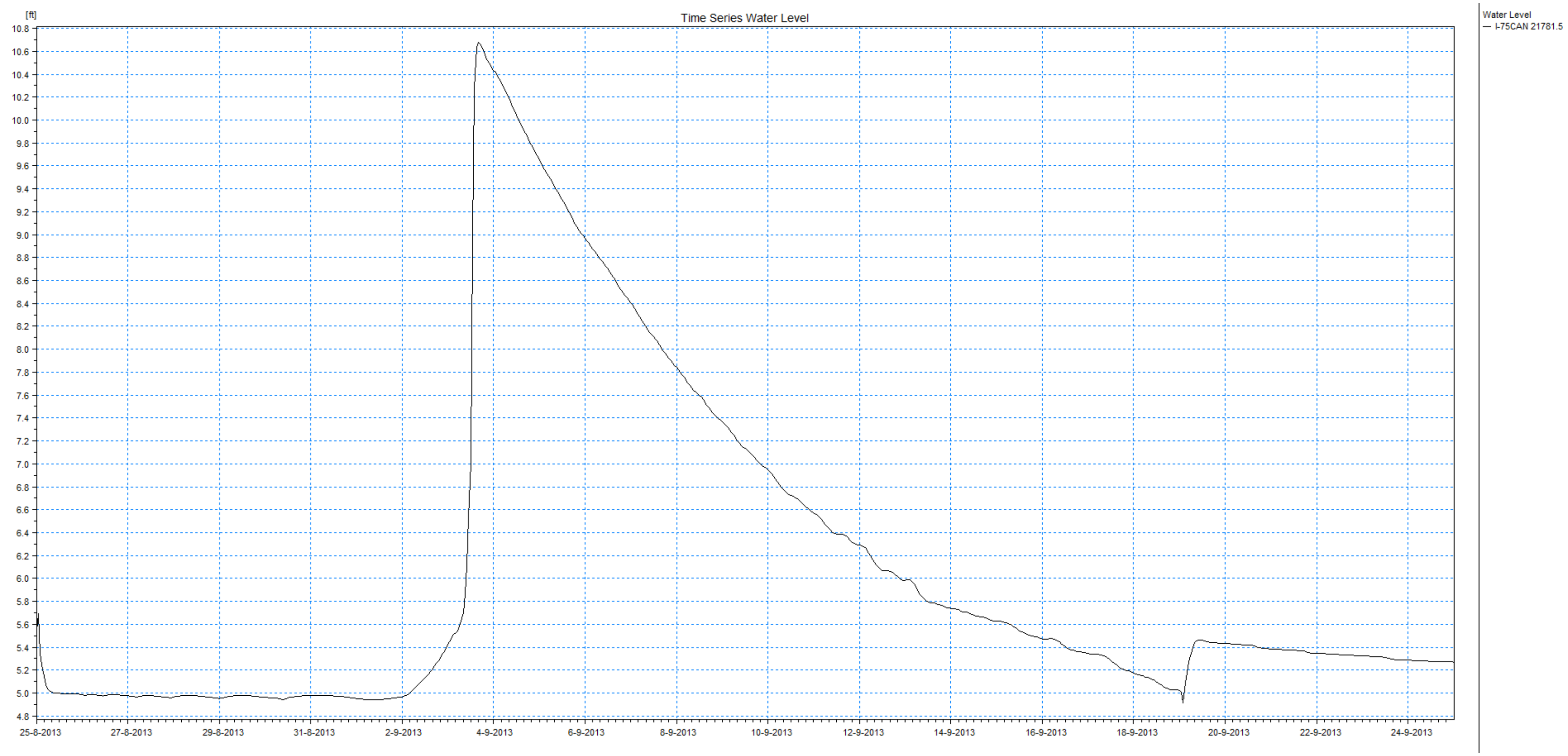

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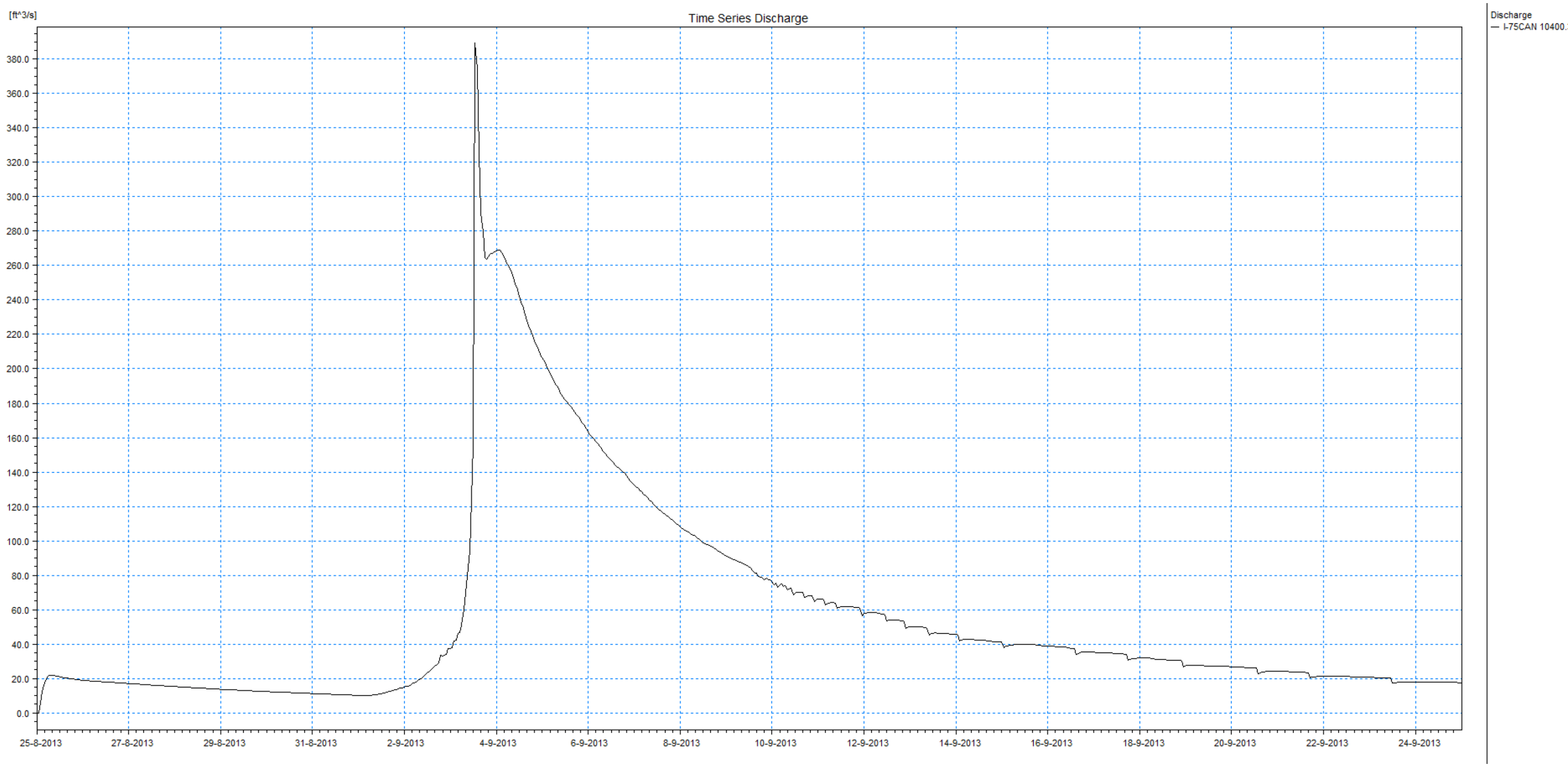

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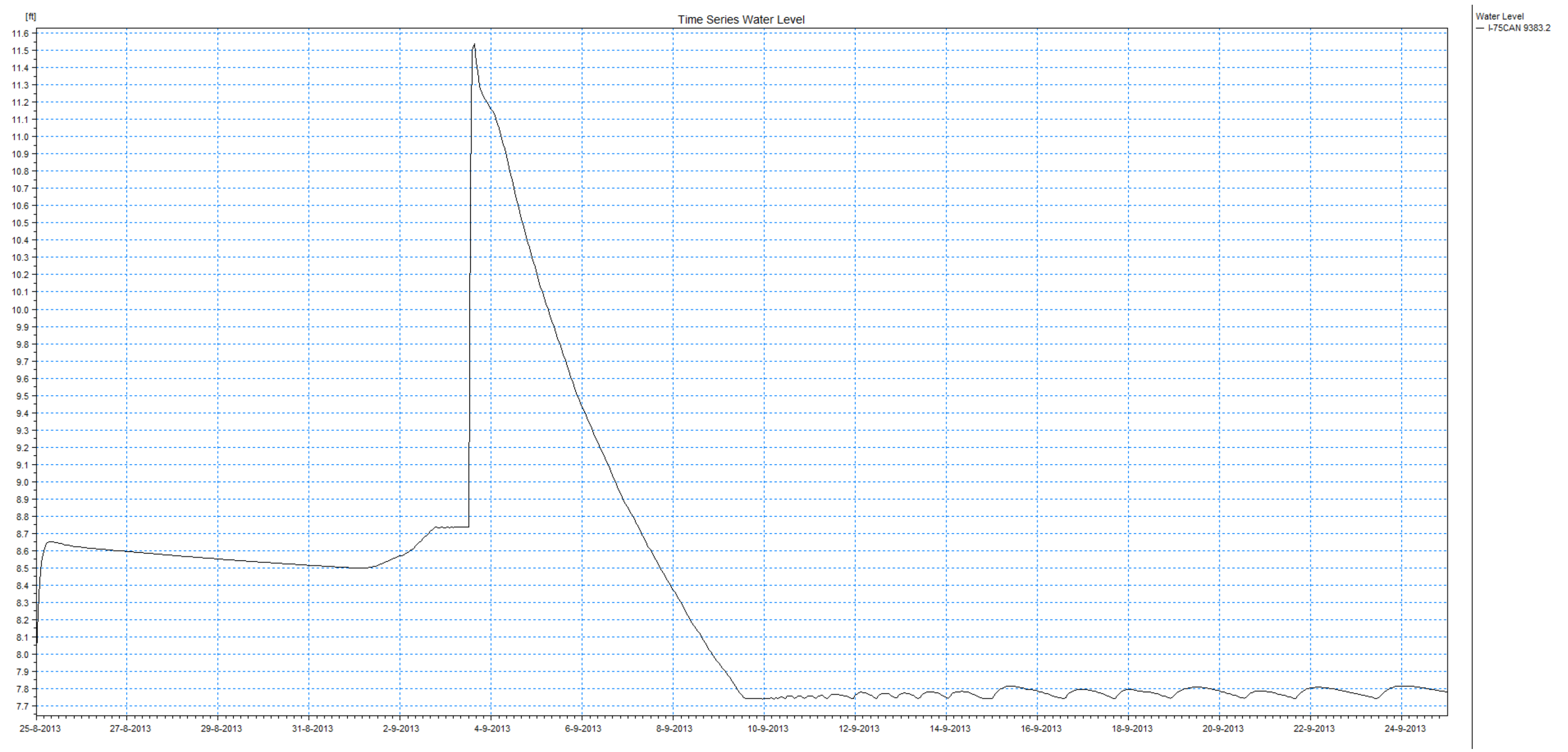

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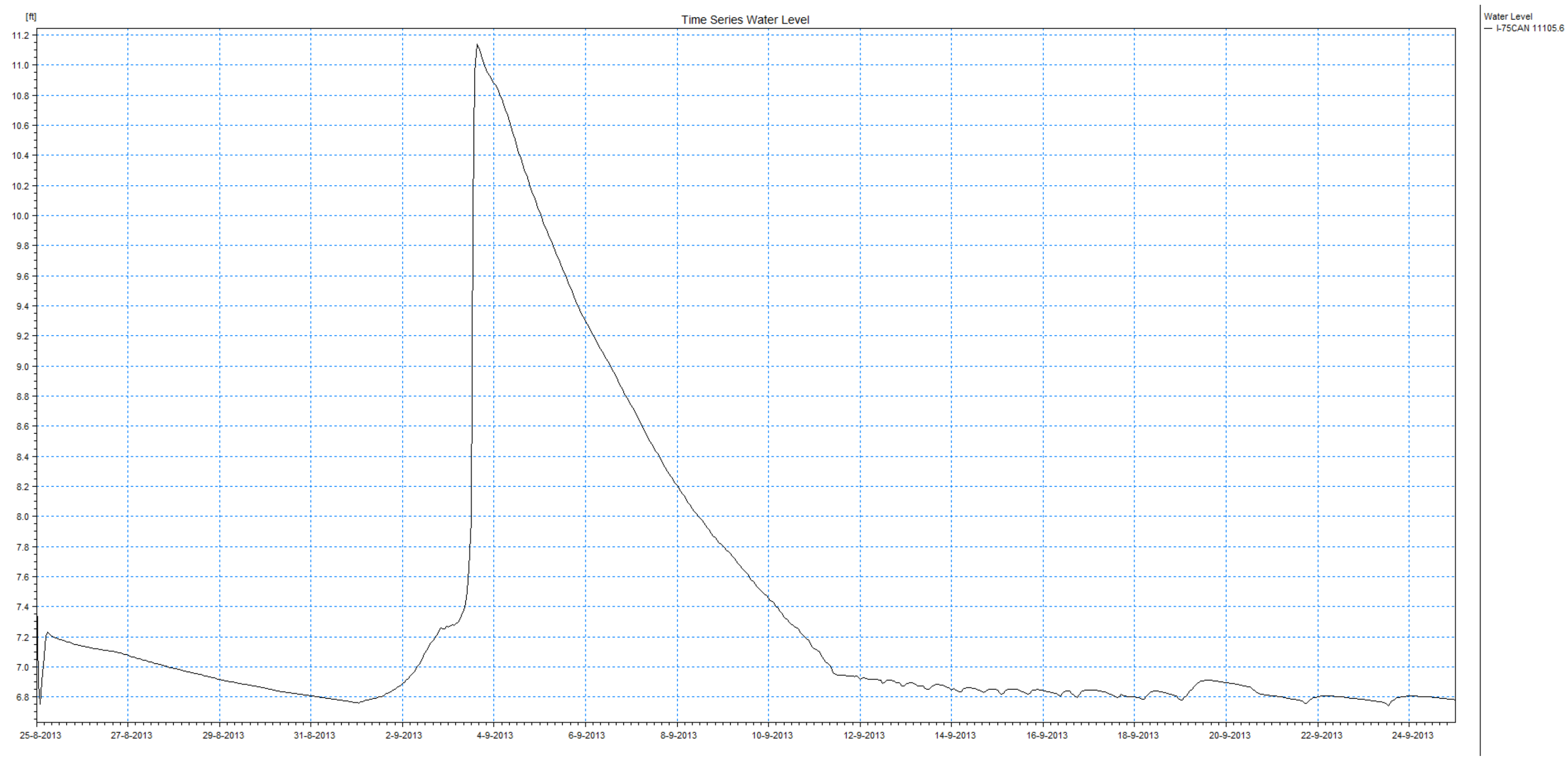

Figure 49. I75W3 Tailwater Stage (ft)

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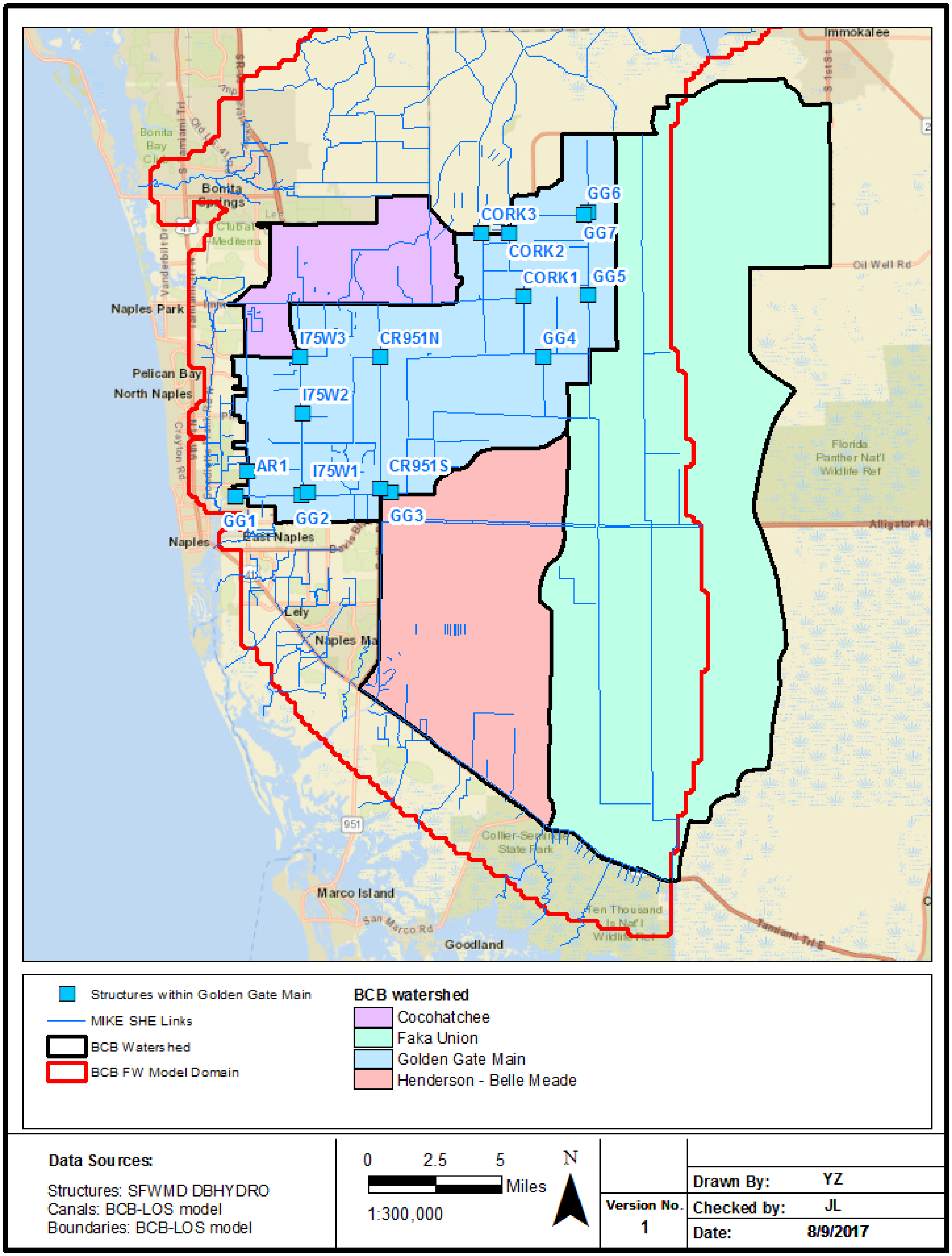

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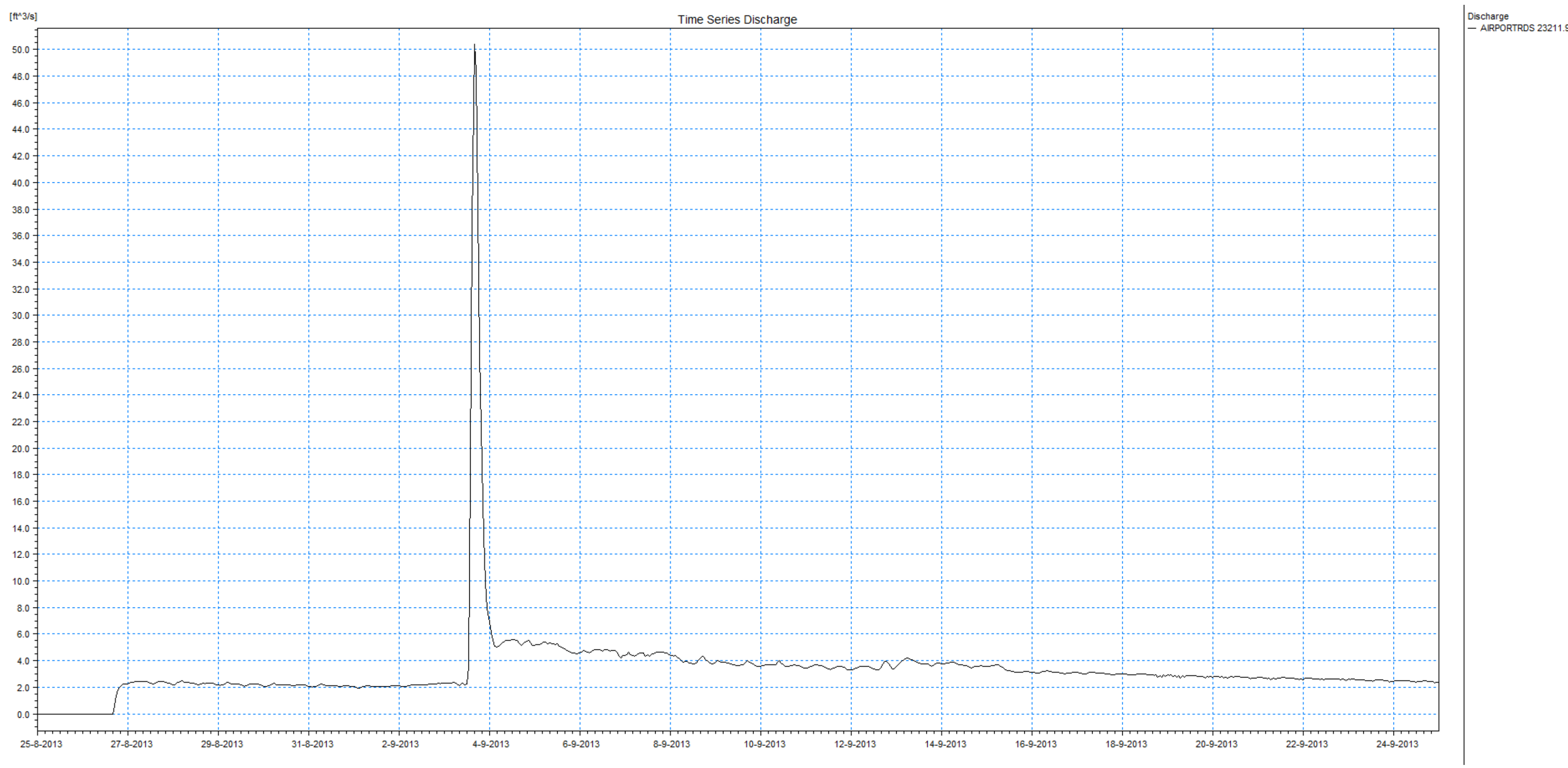

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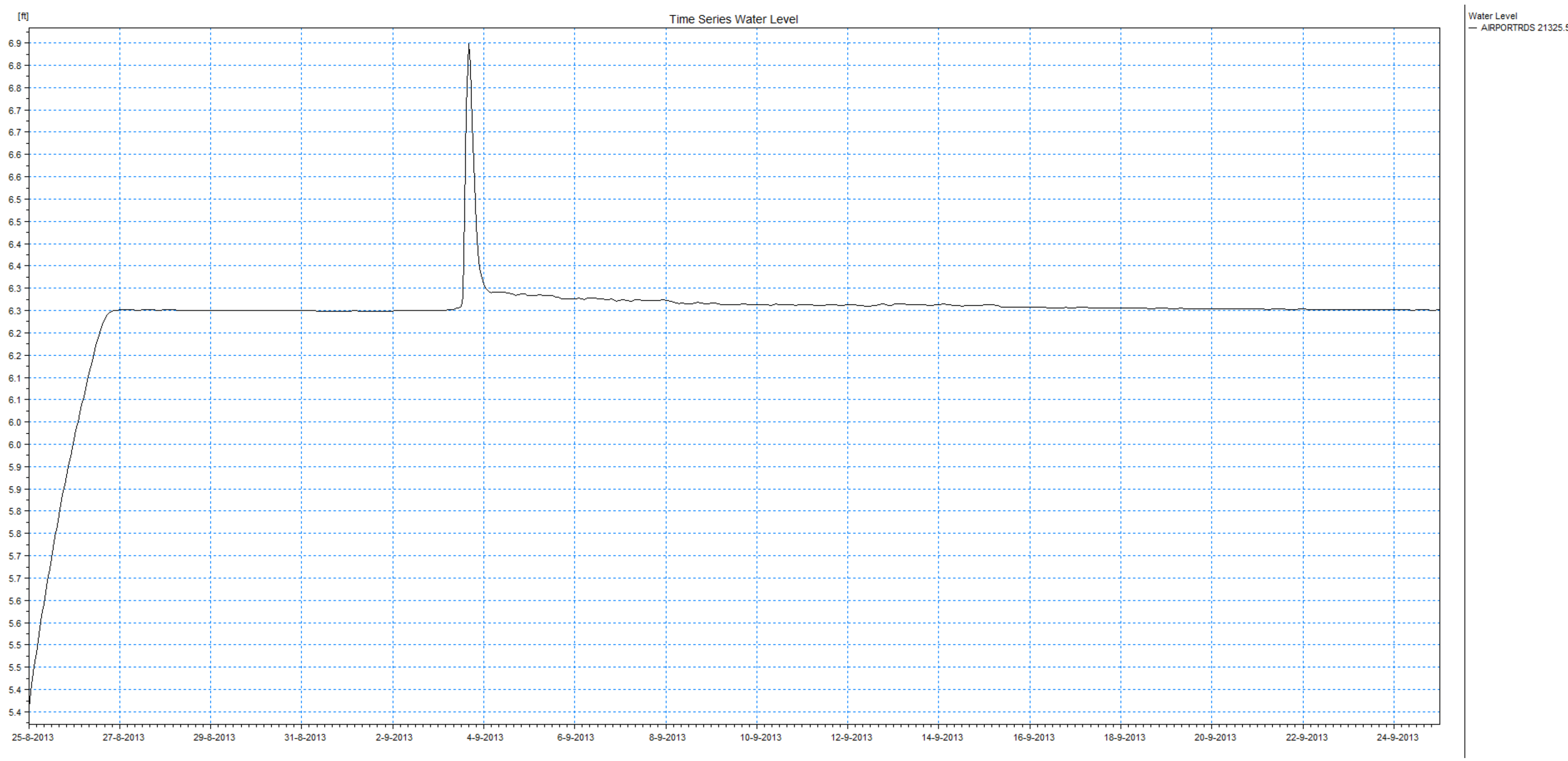

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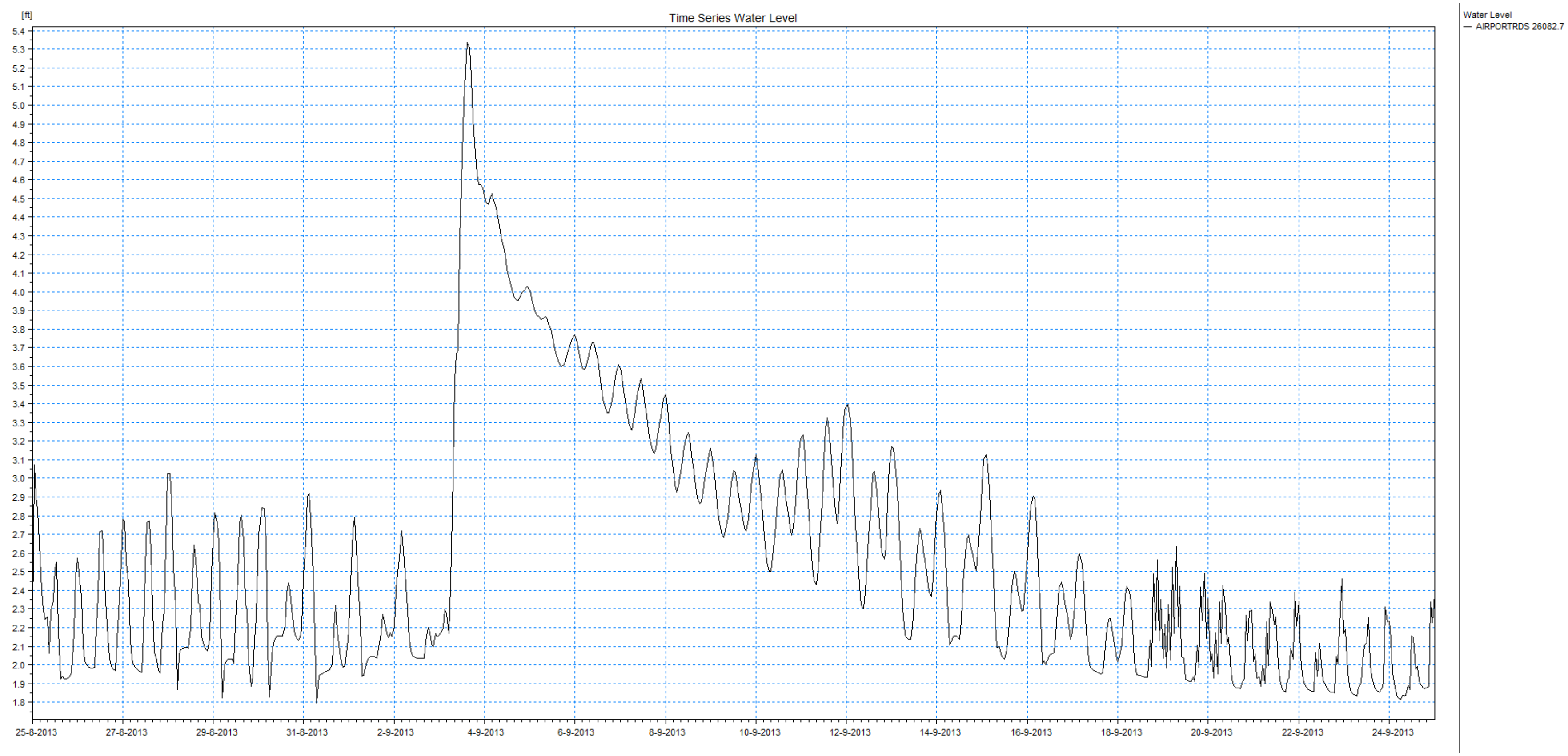

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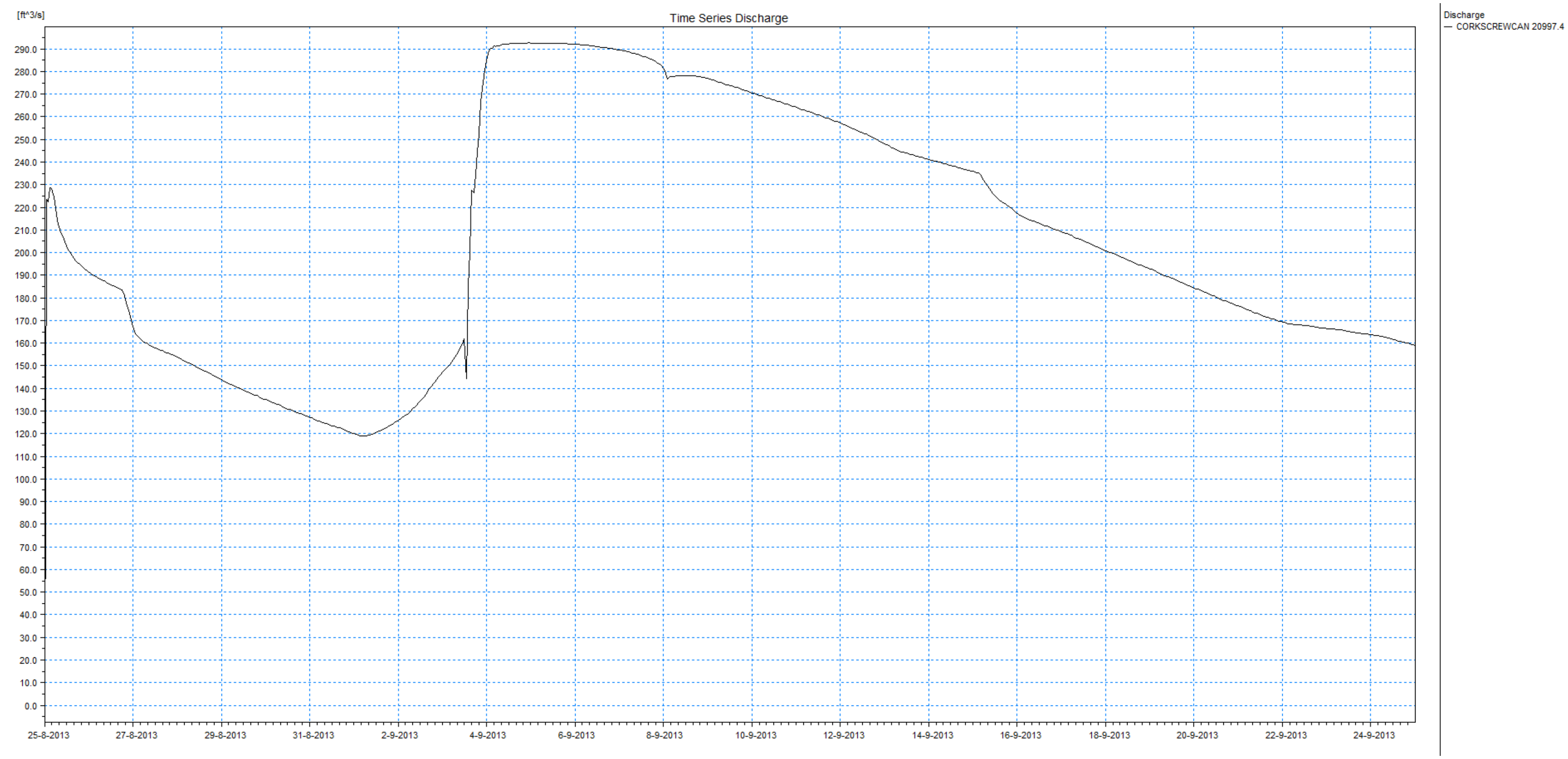

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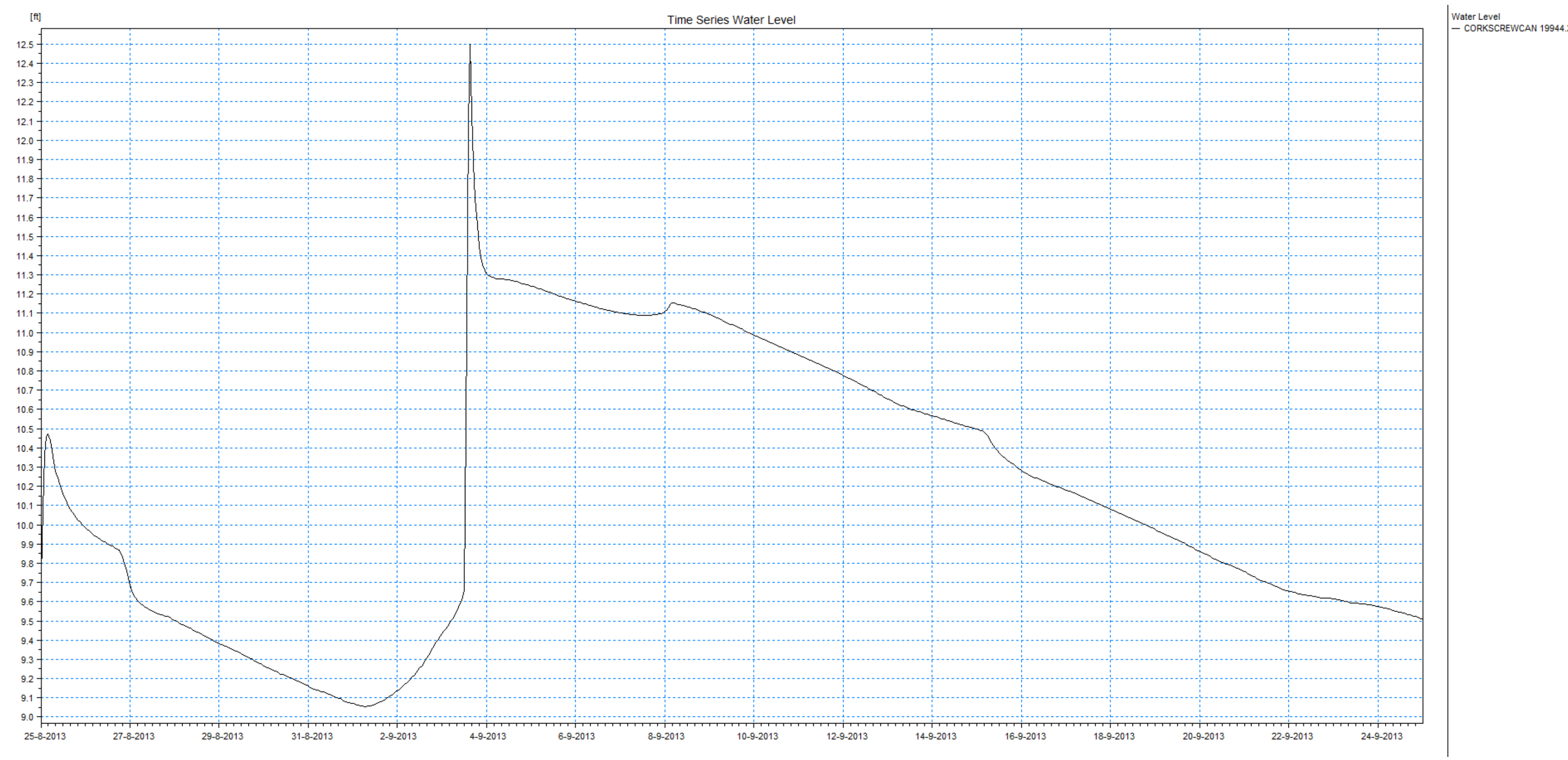

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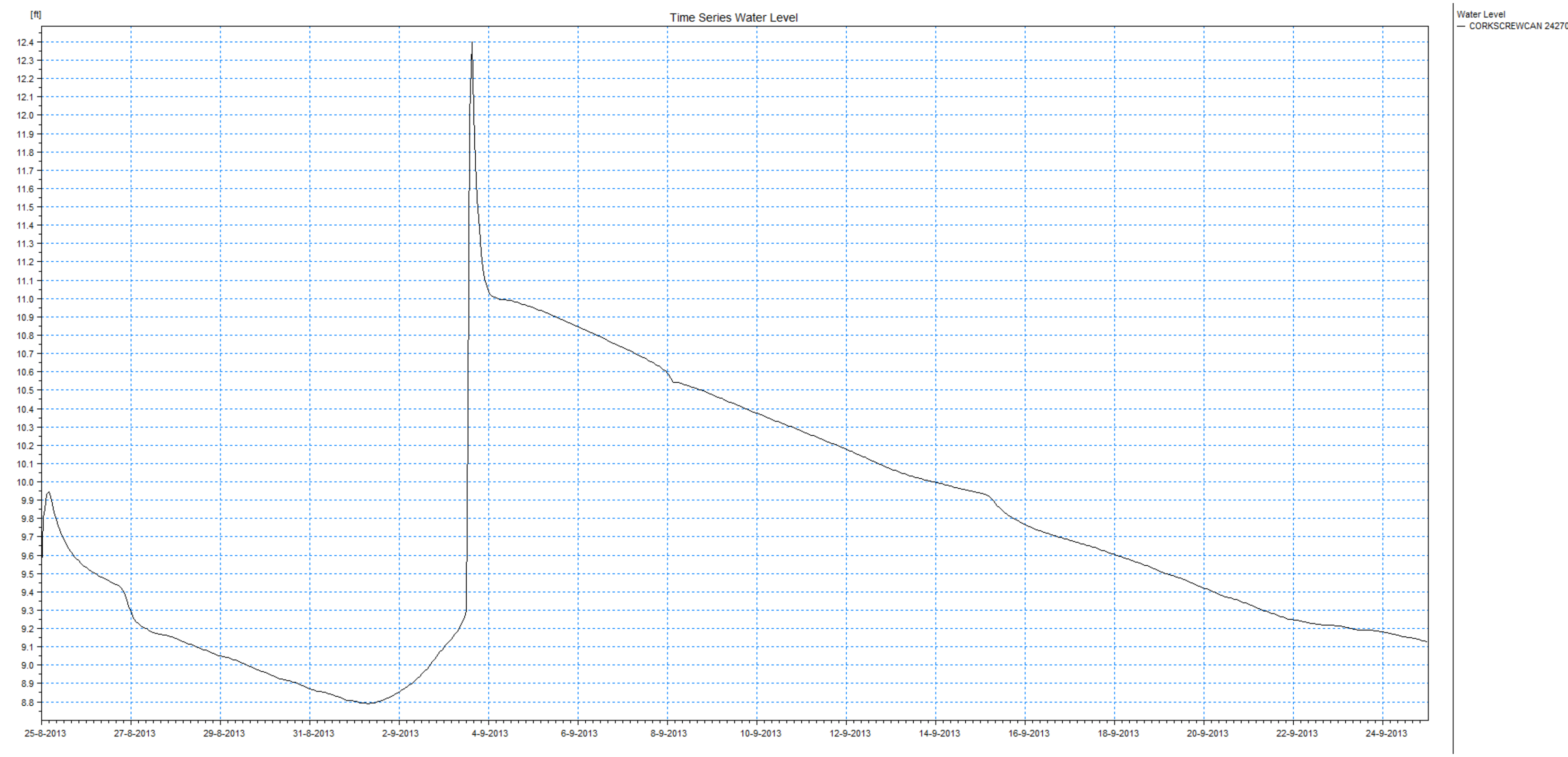

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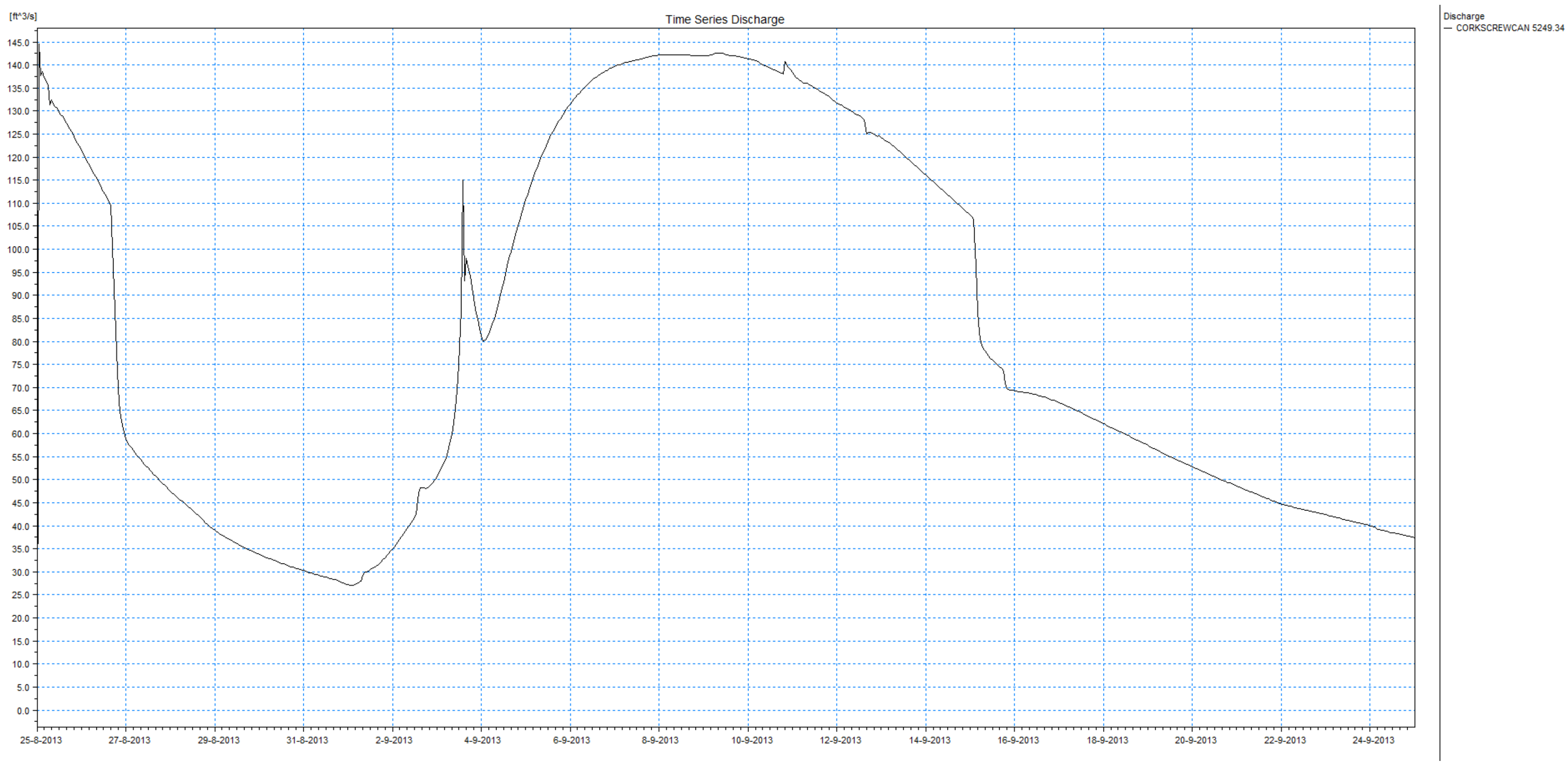

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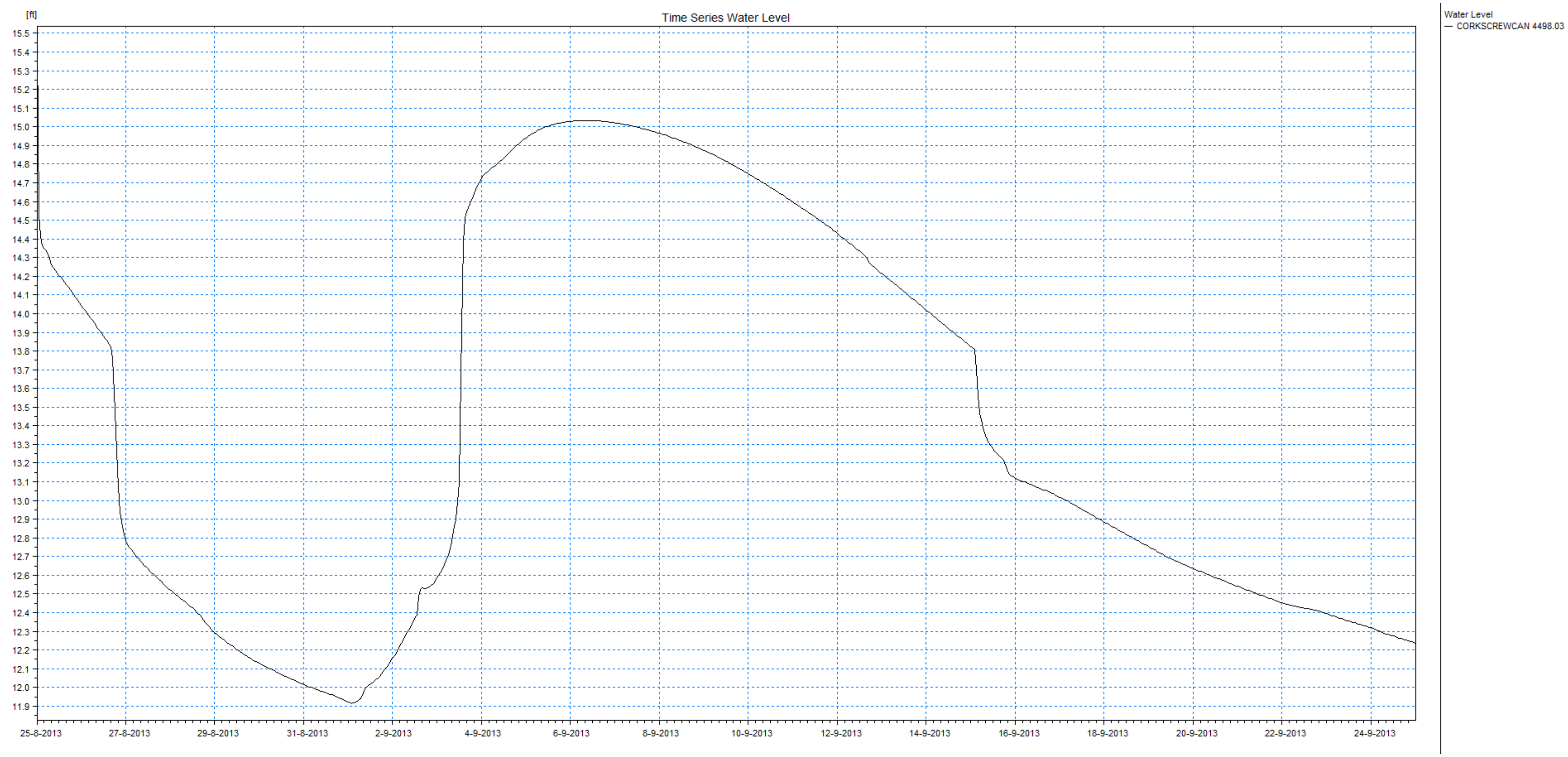

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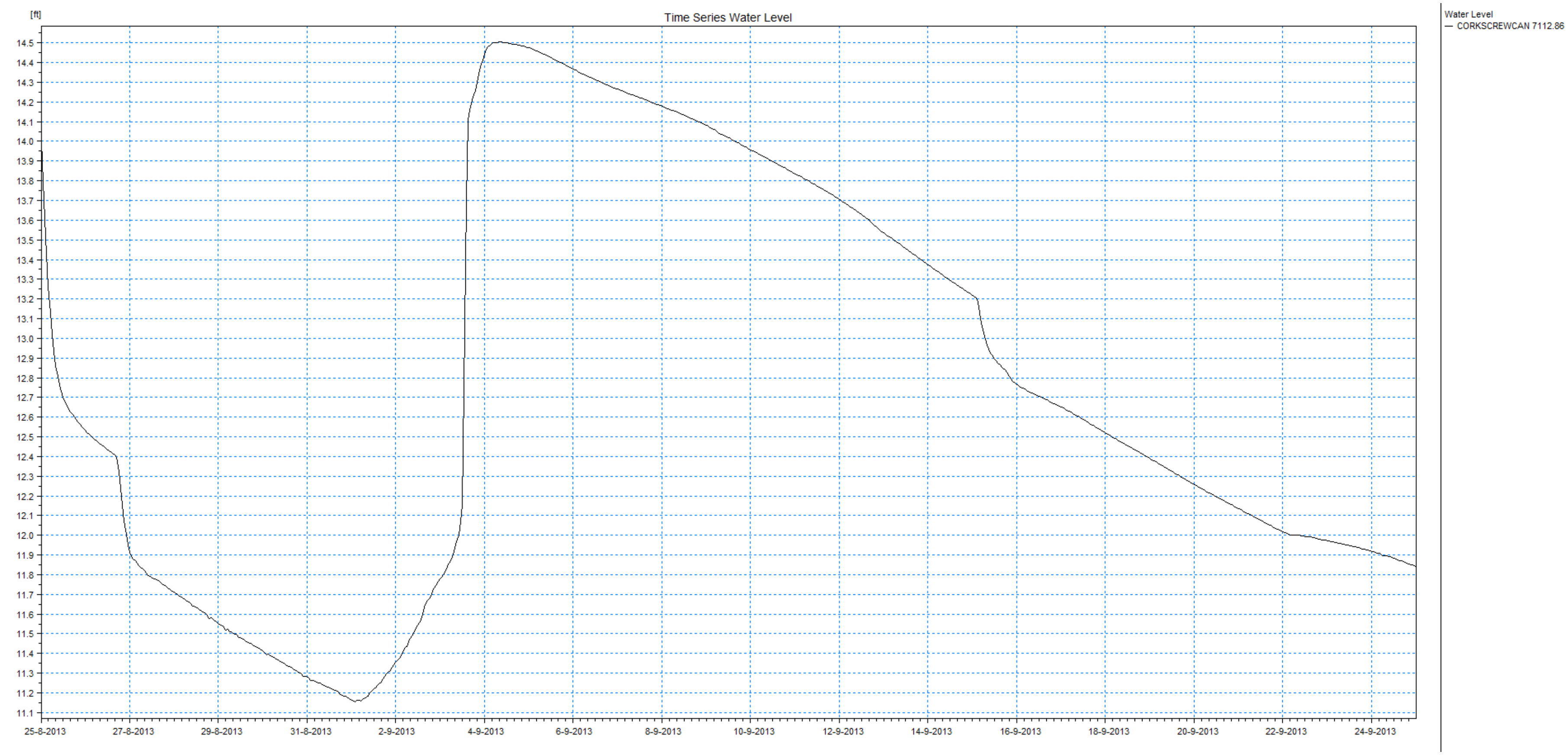

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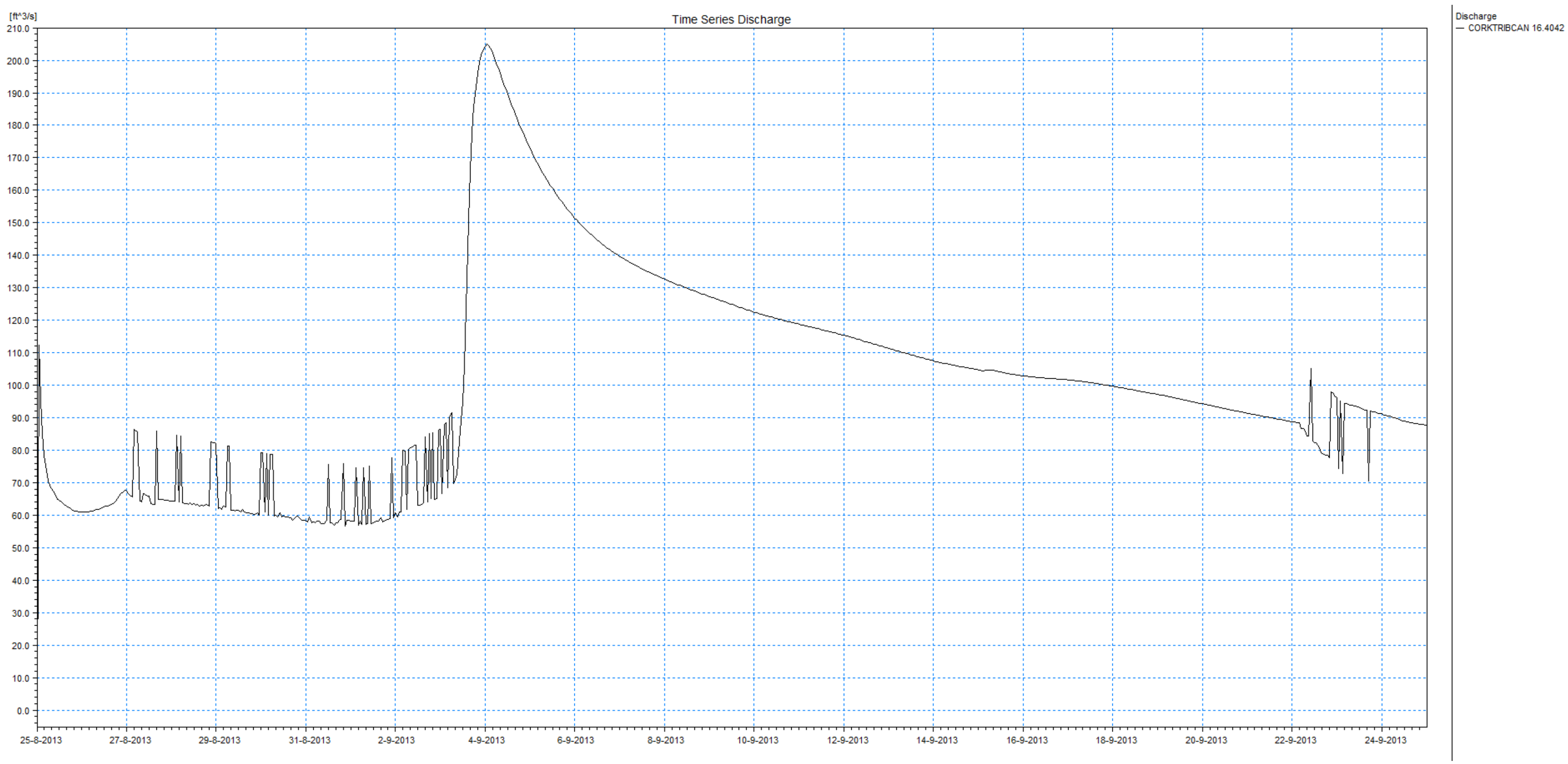

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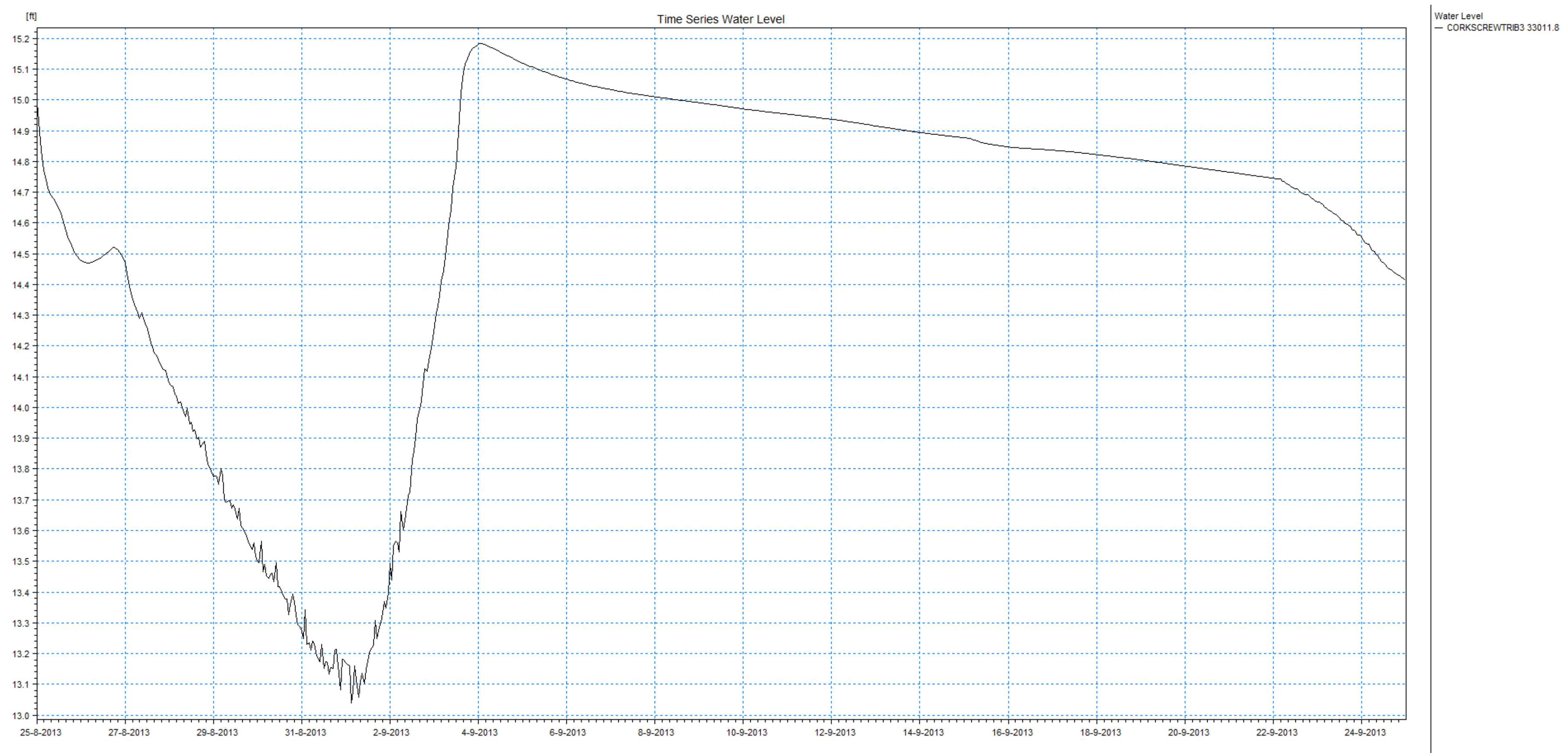

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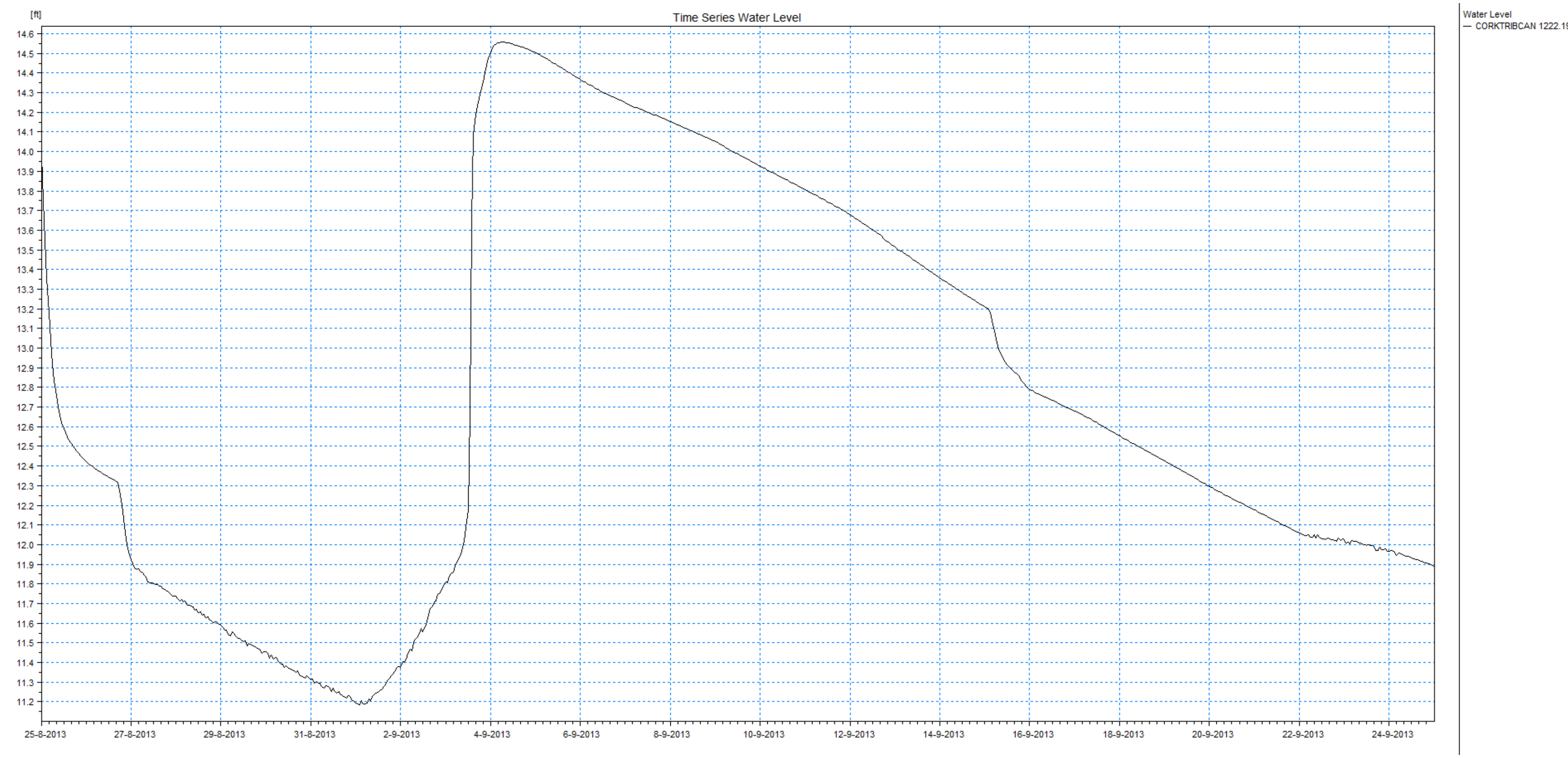

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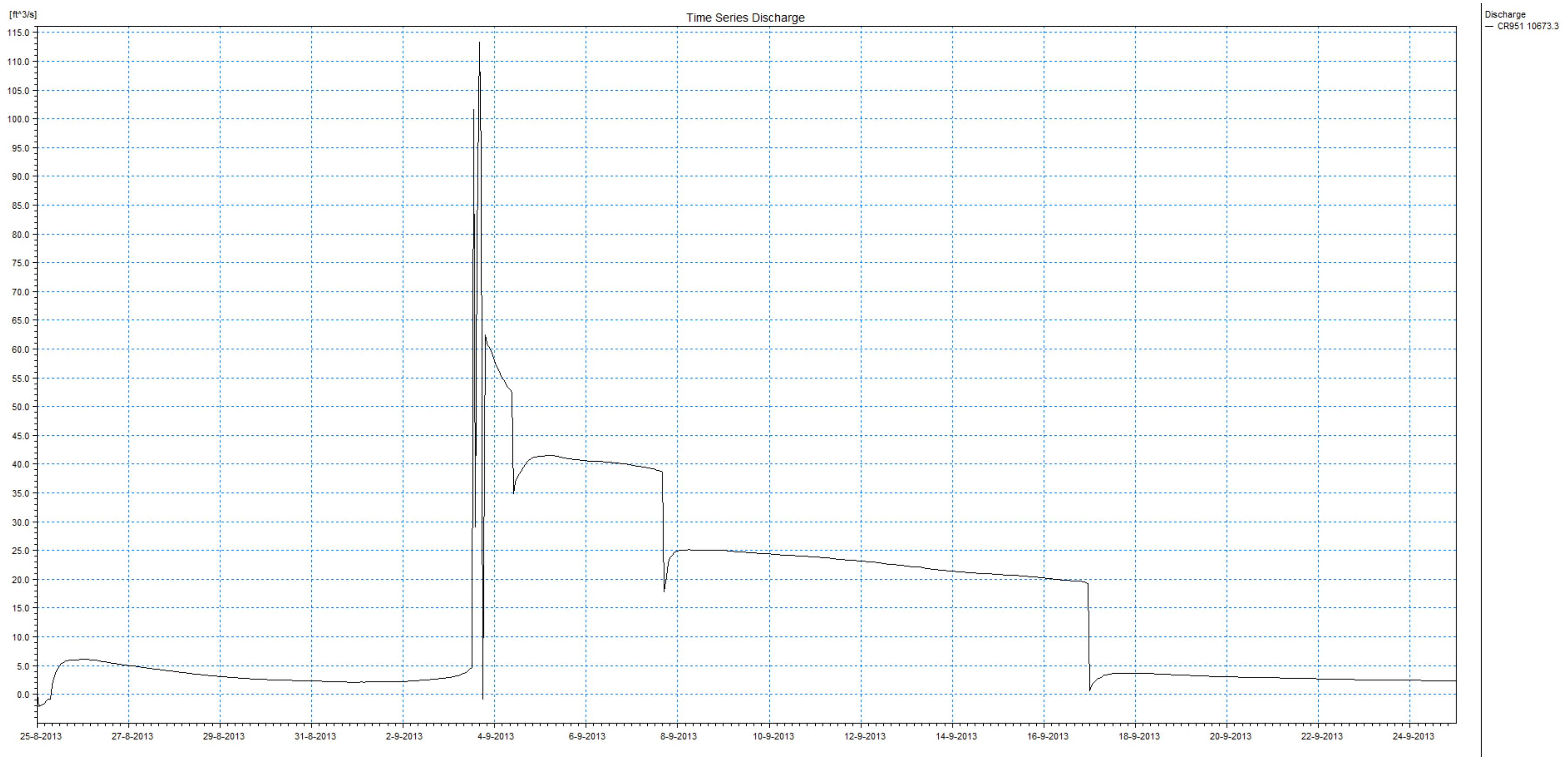

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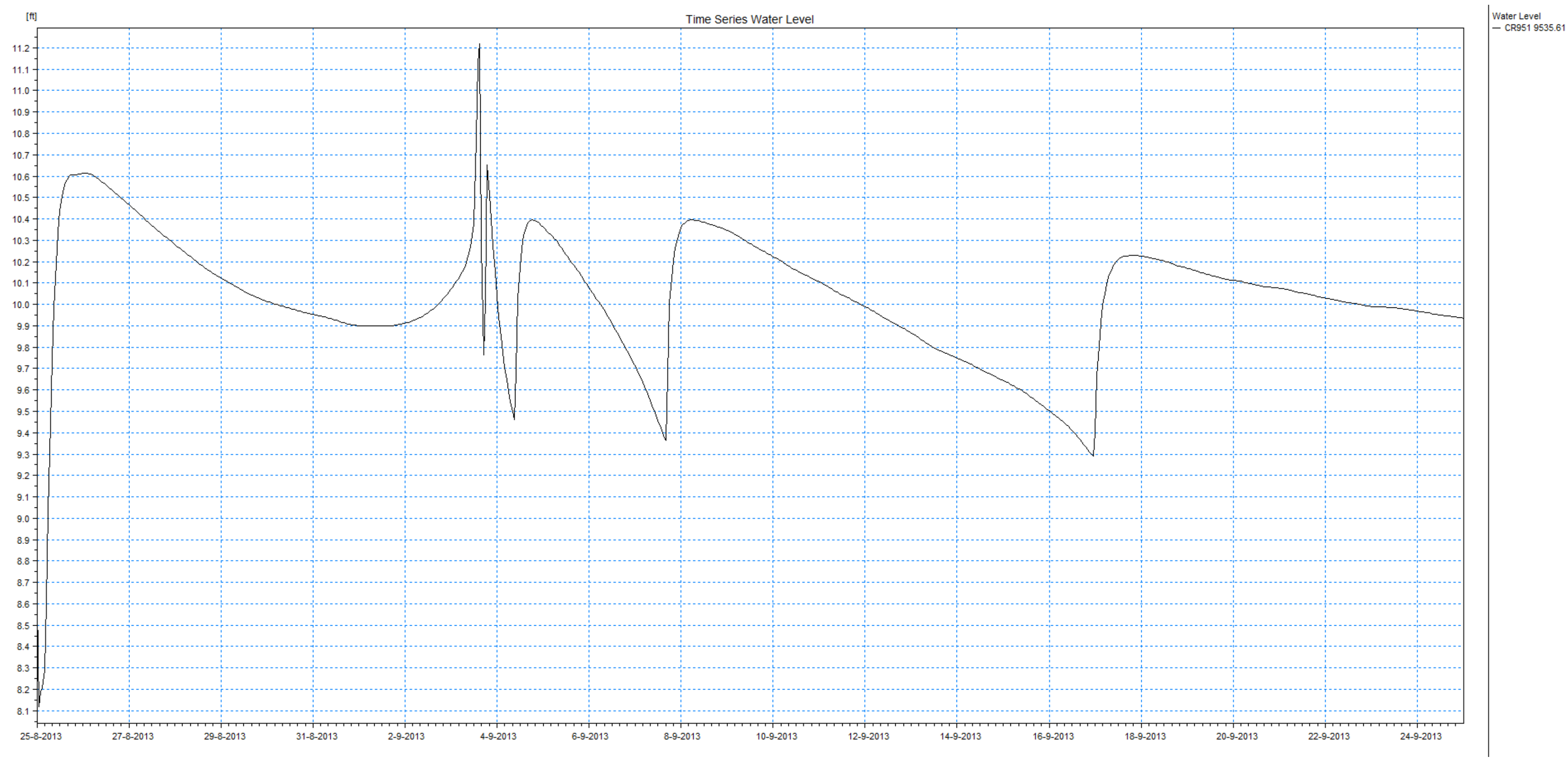

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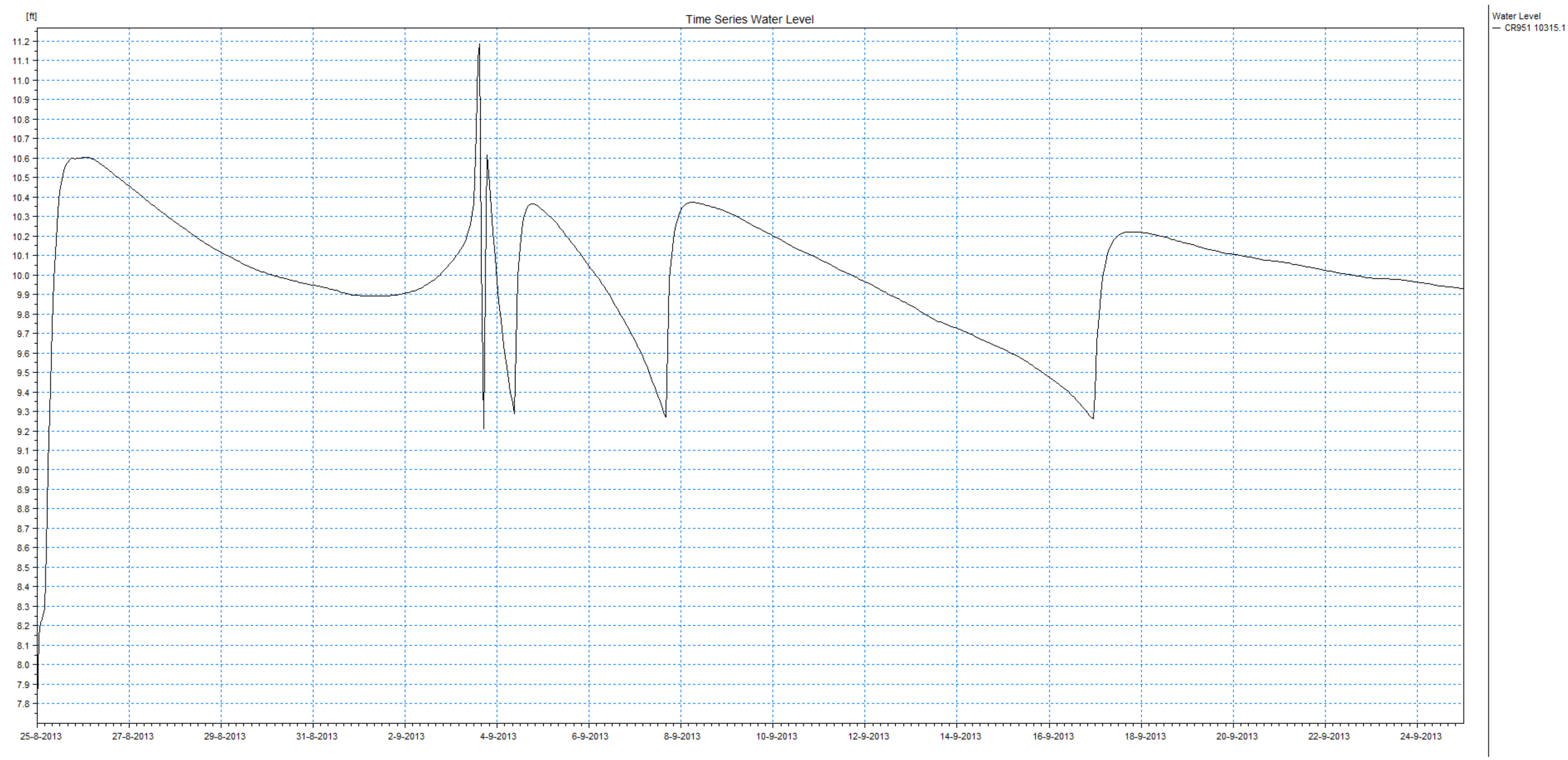

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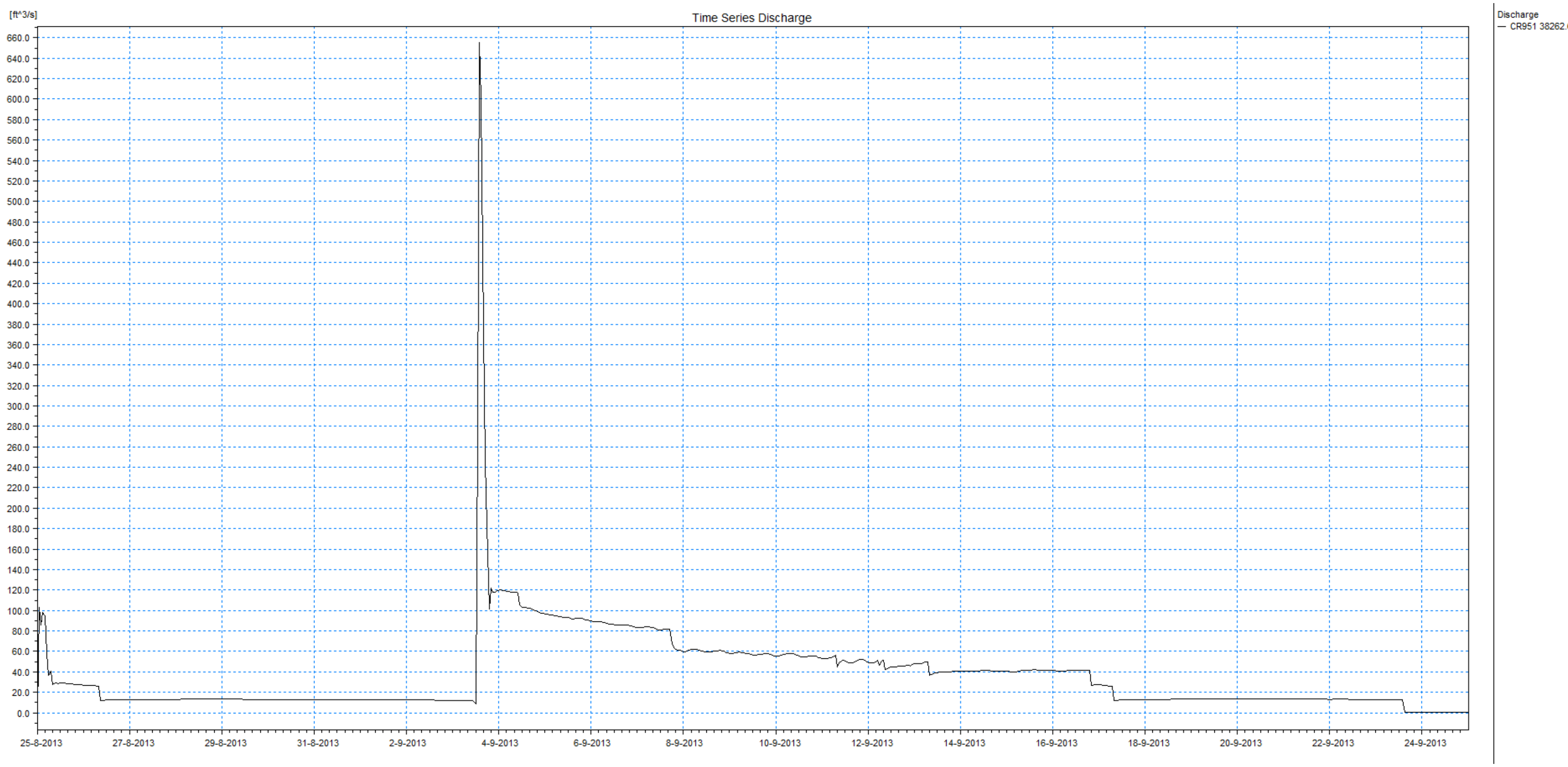

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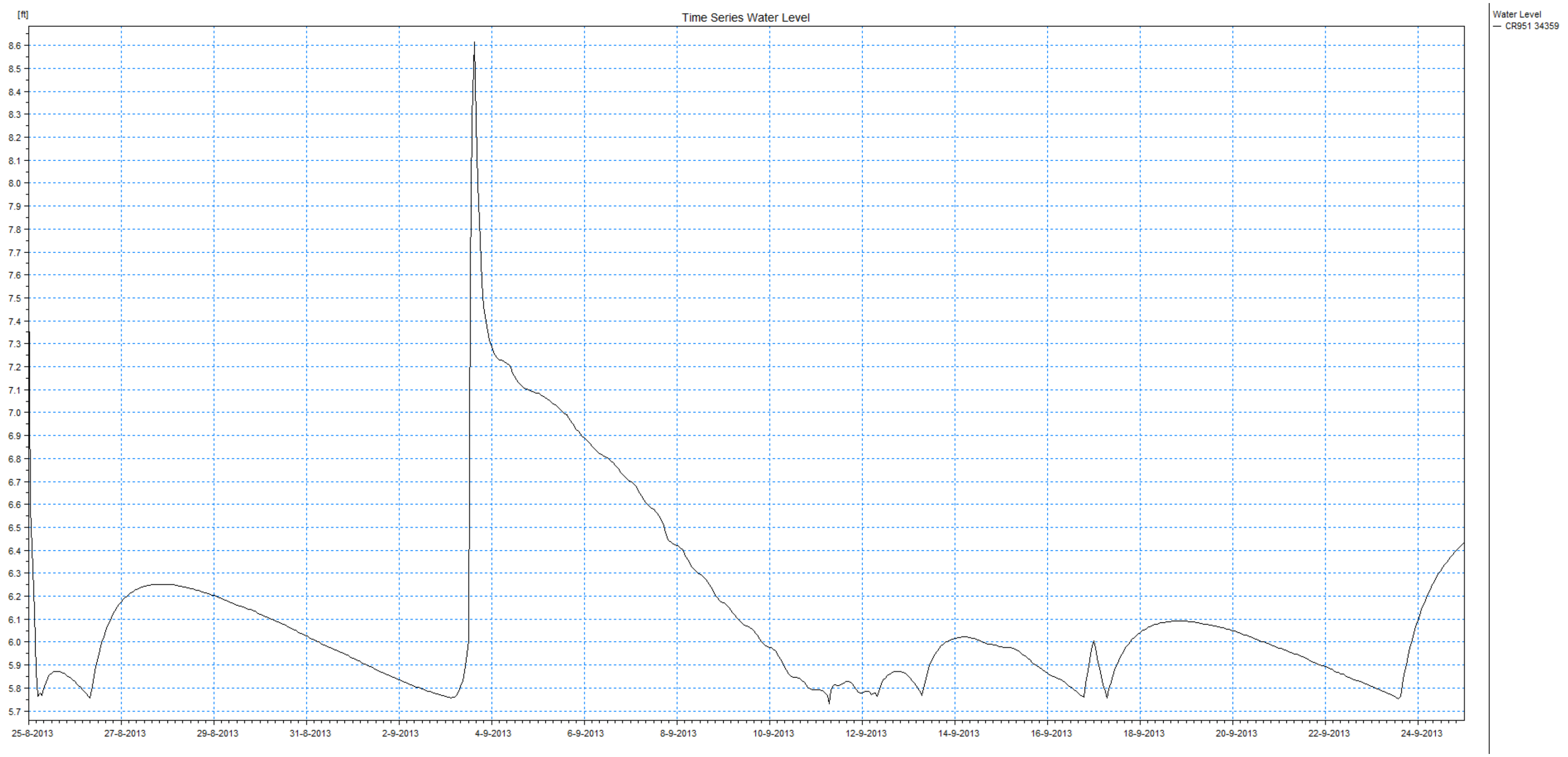

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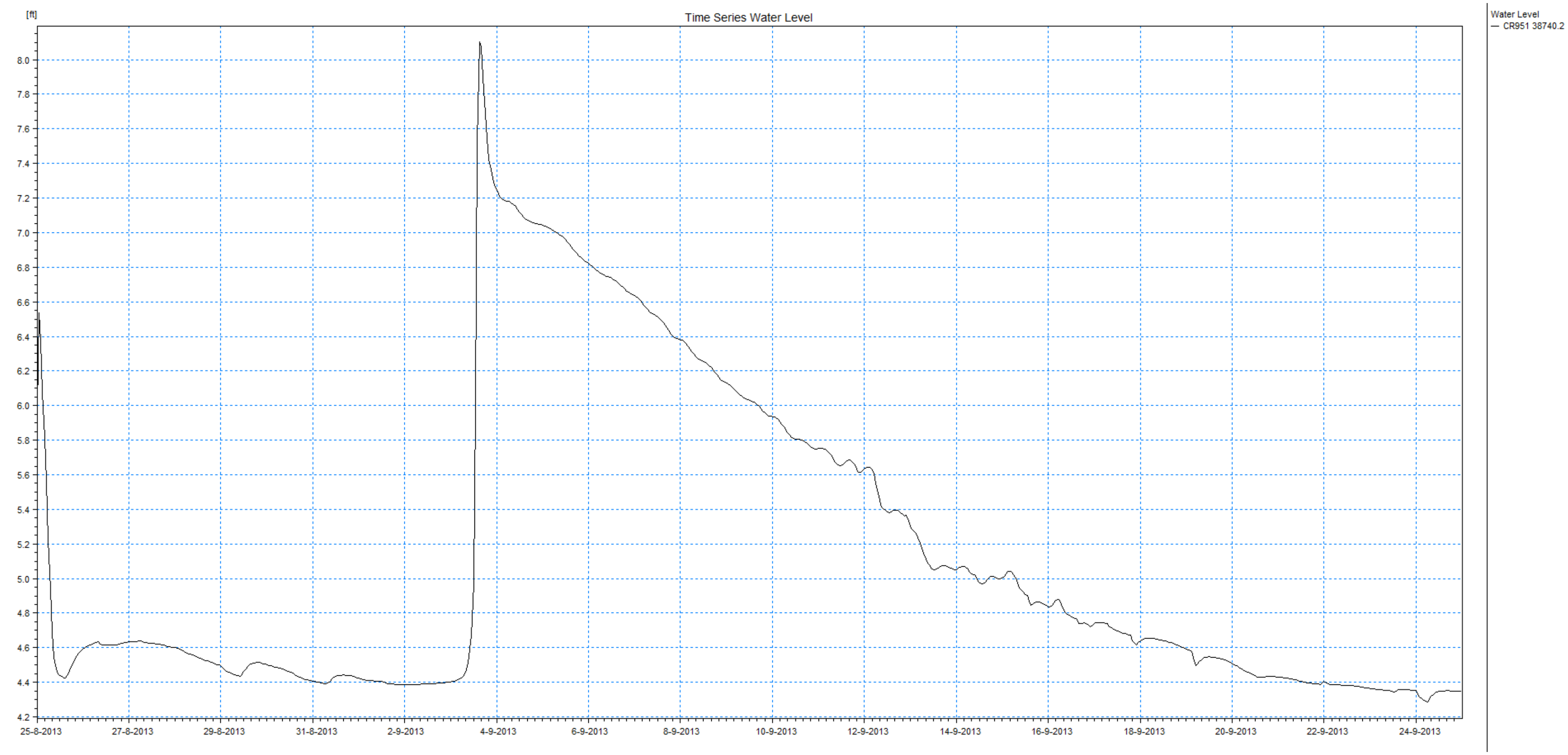

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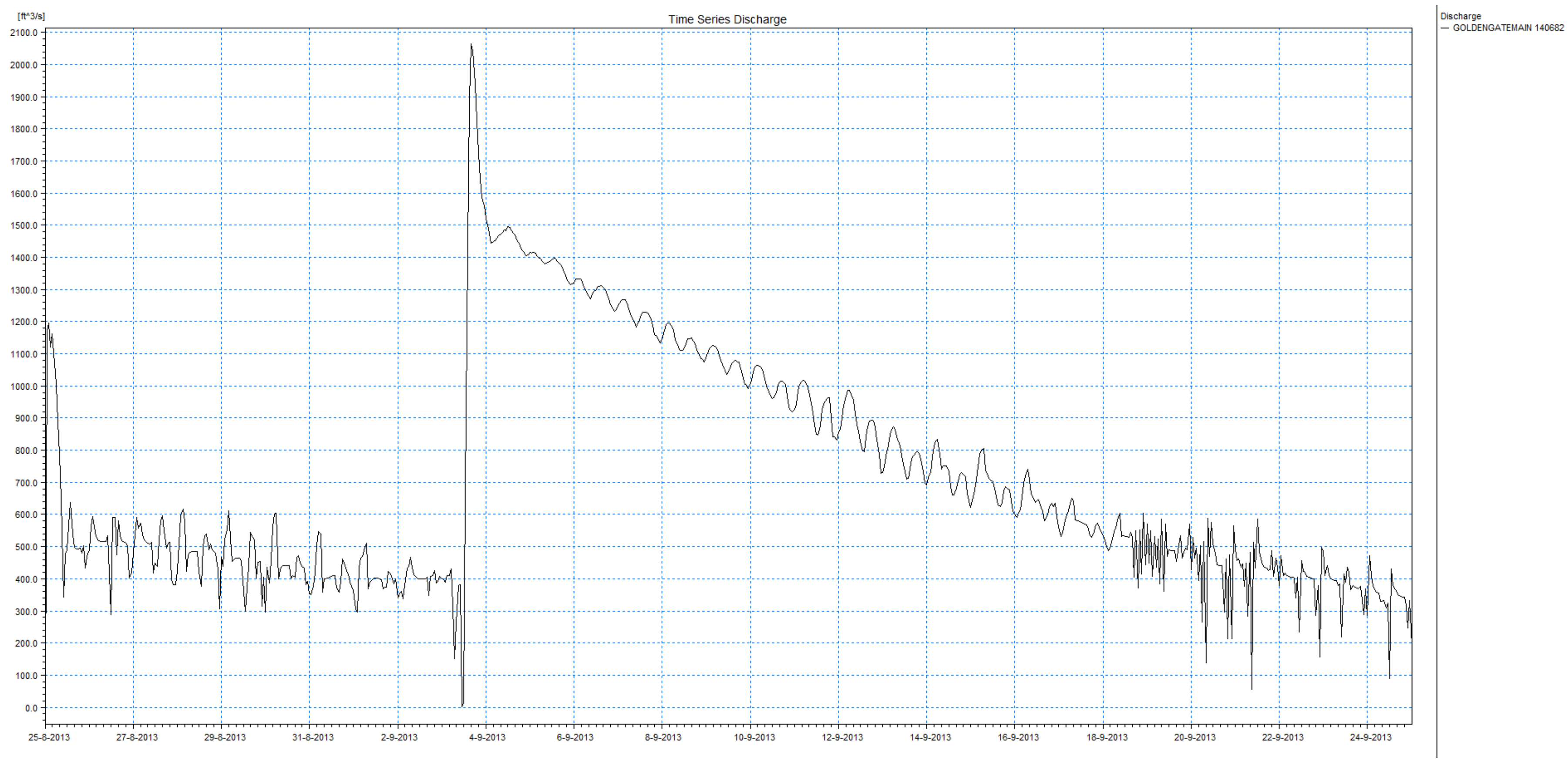

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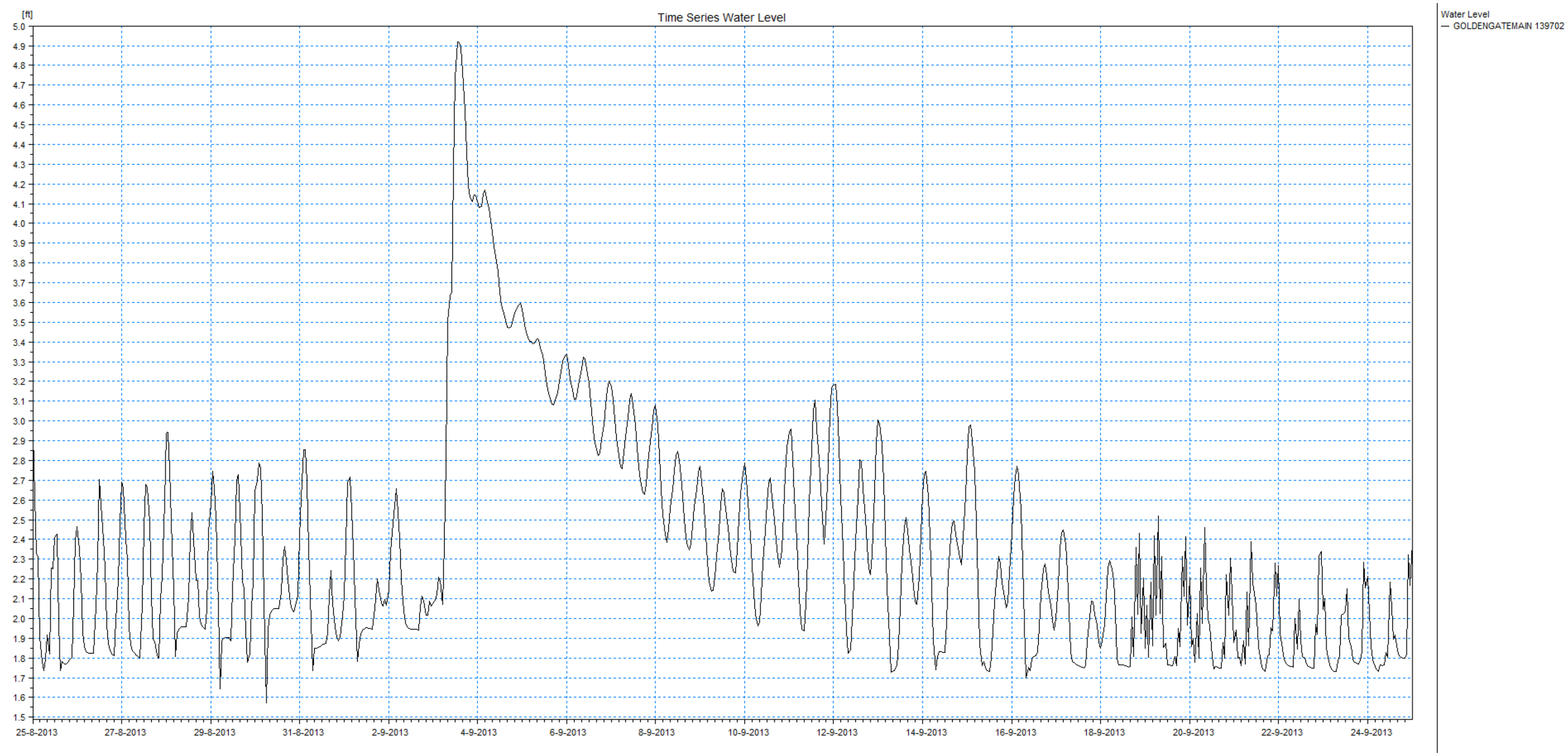

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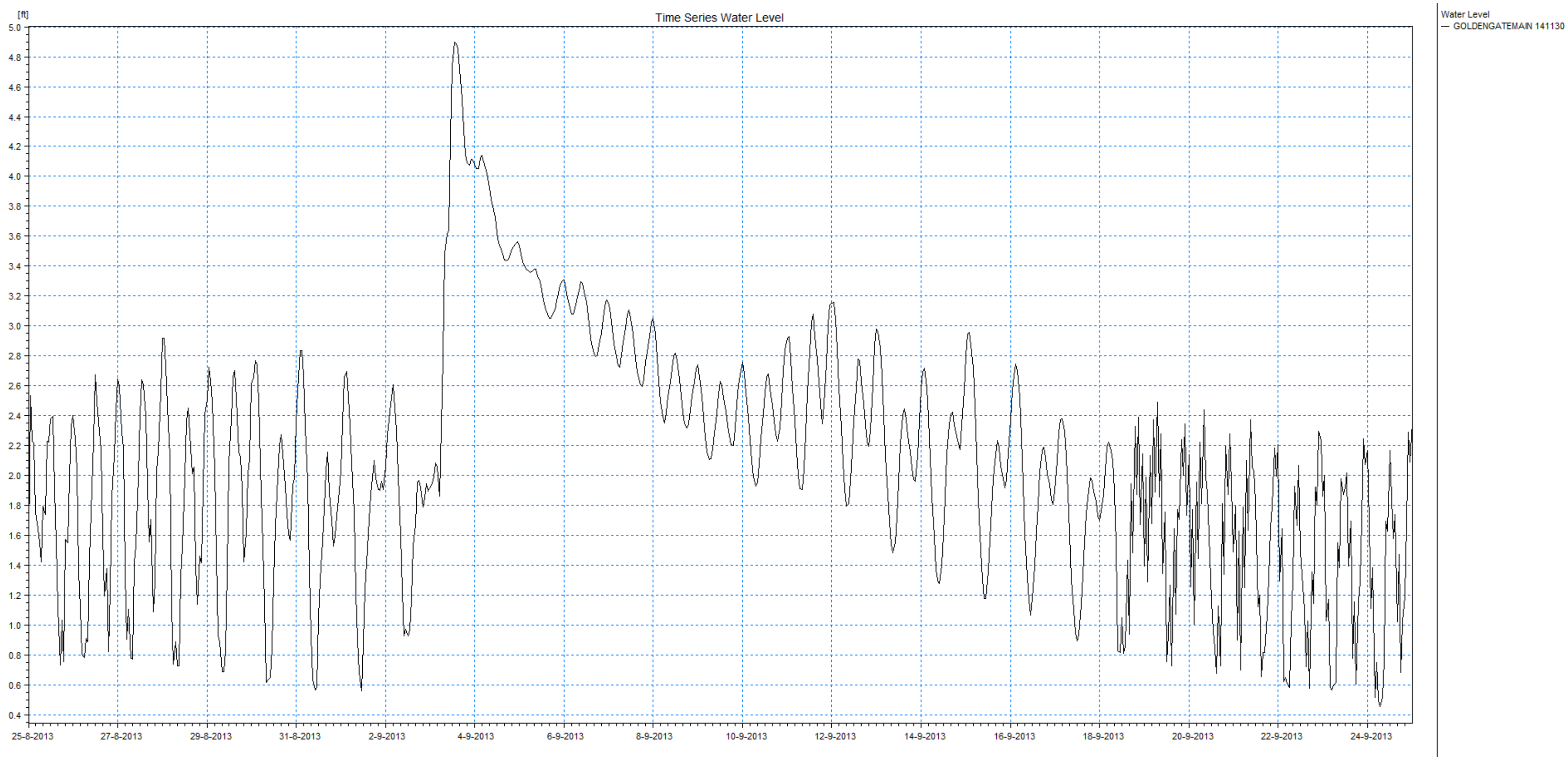

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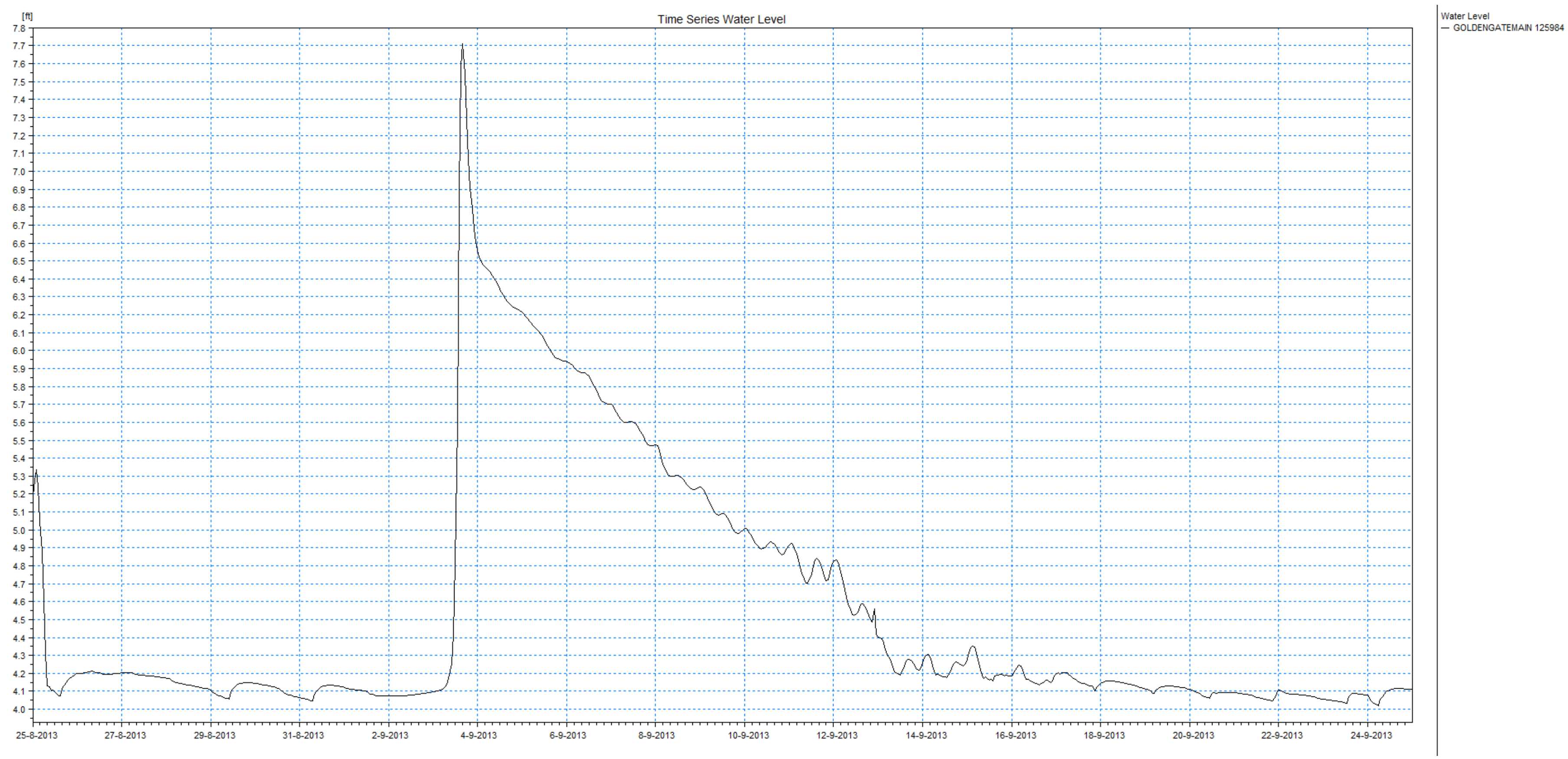

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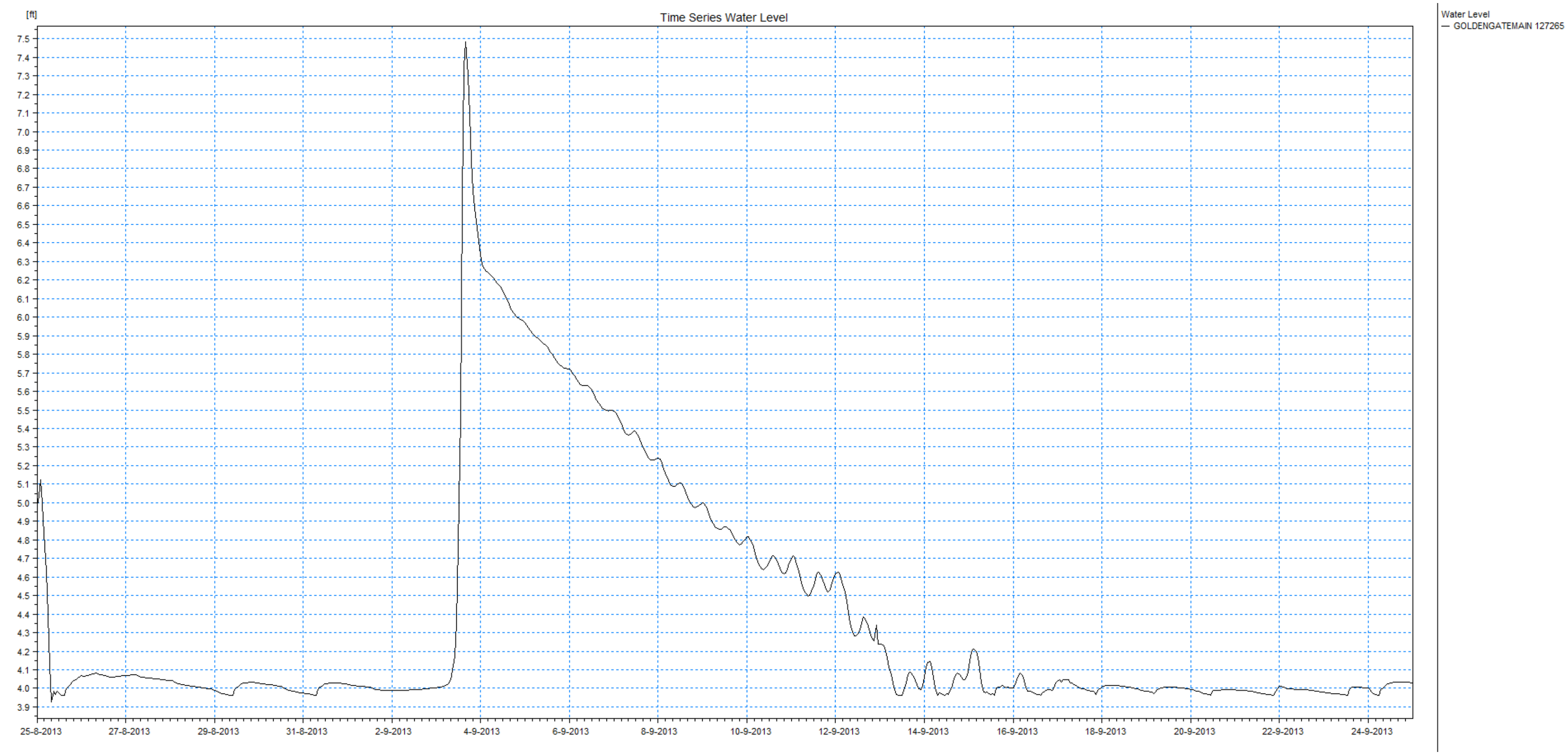

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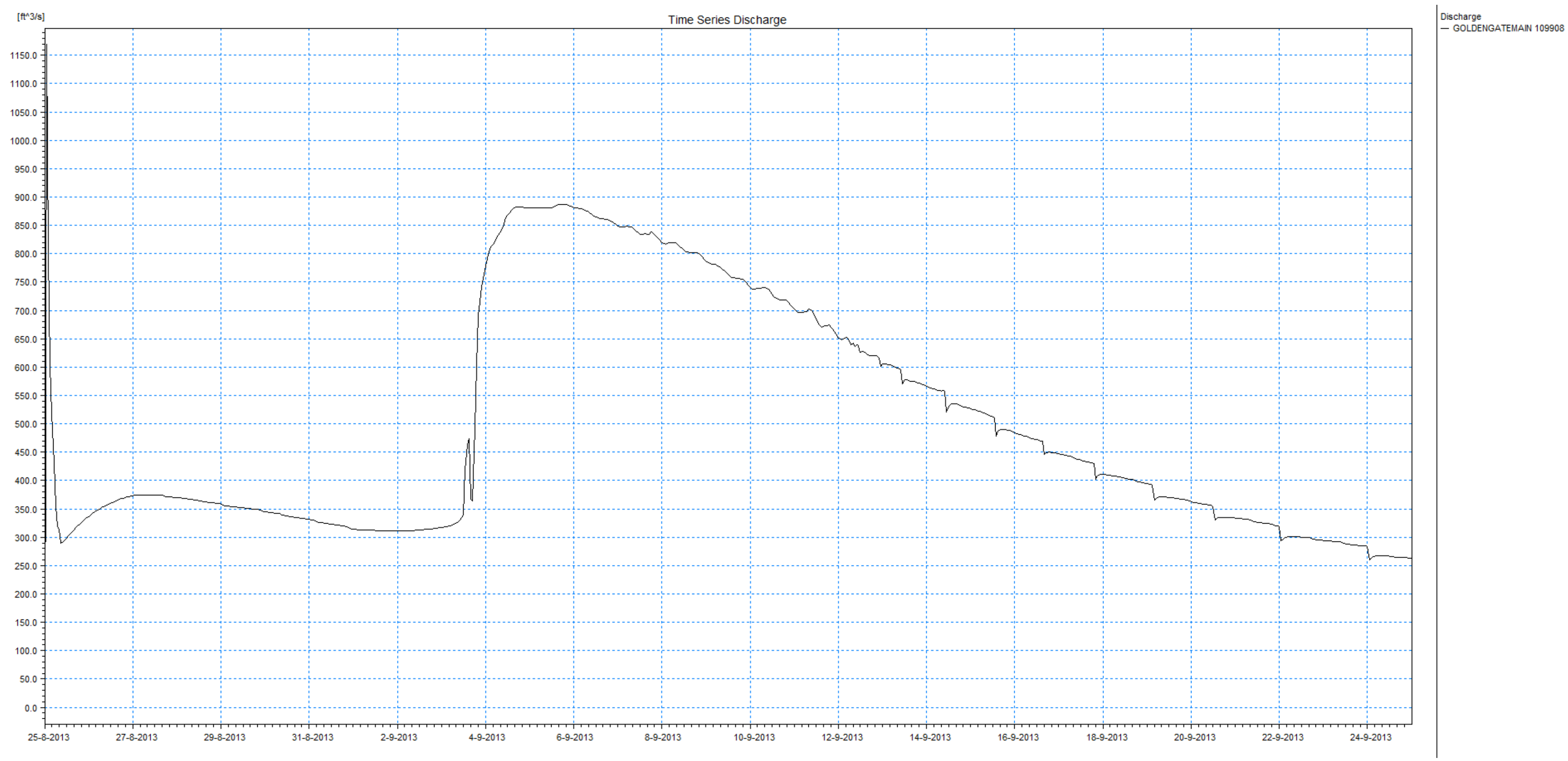

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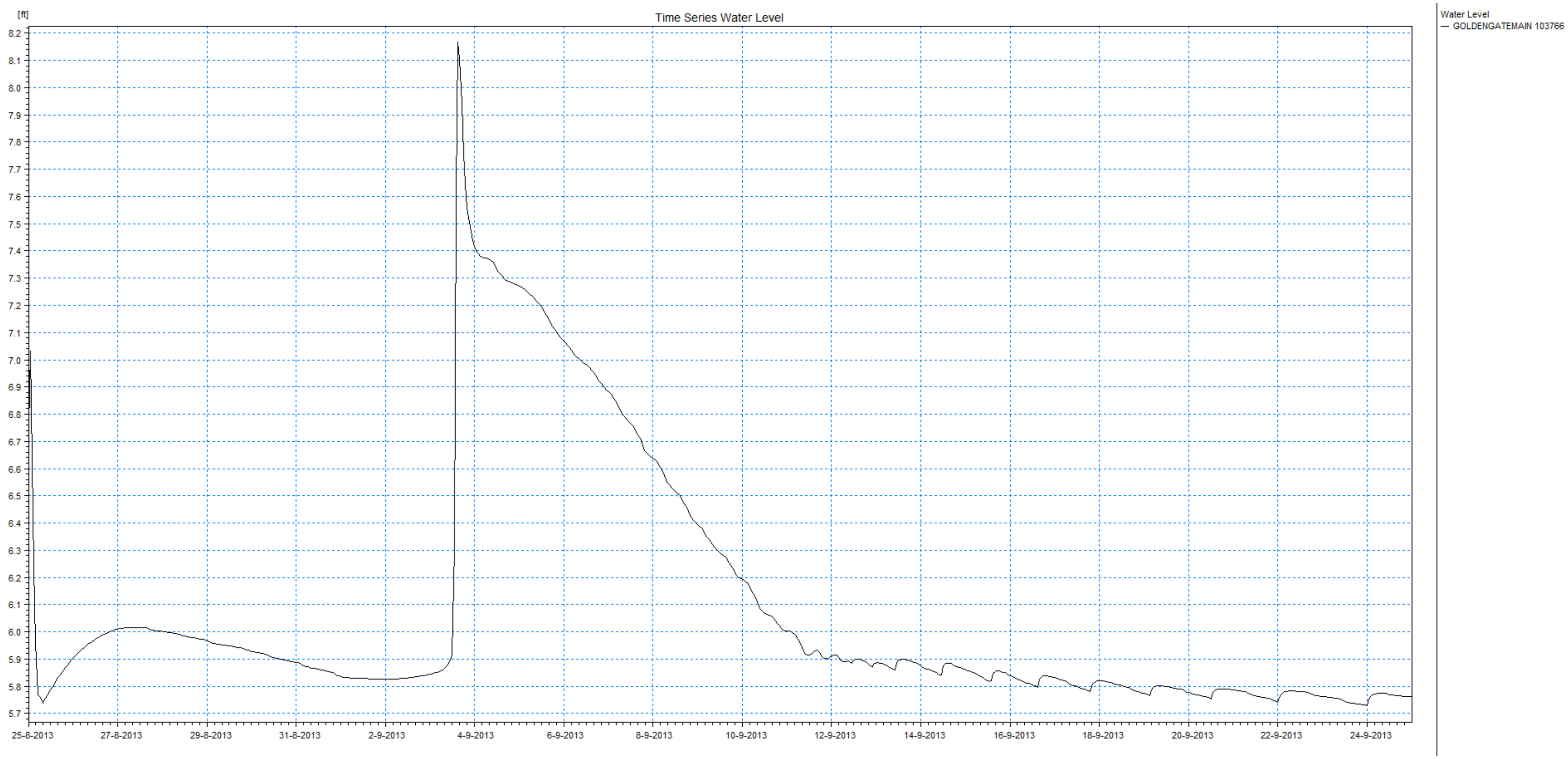

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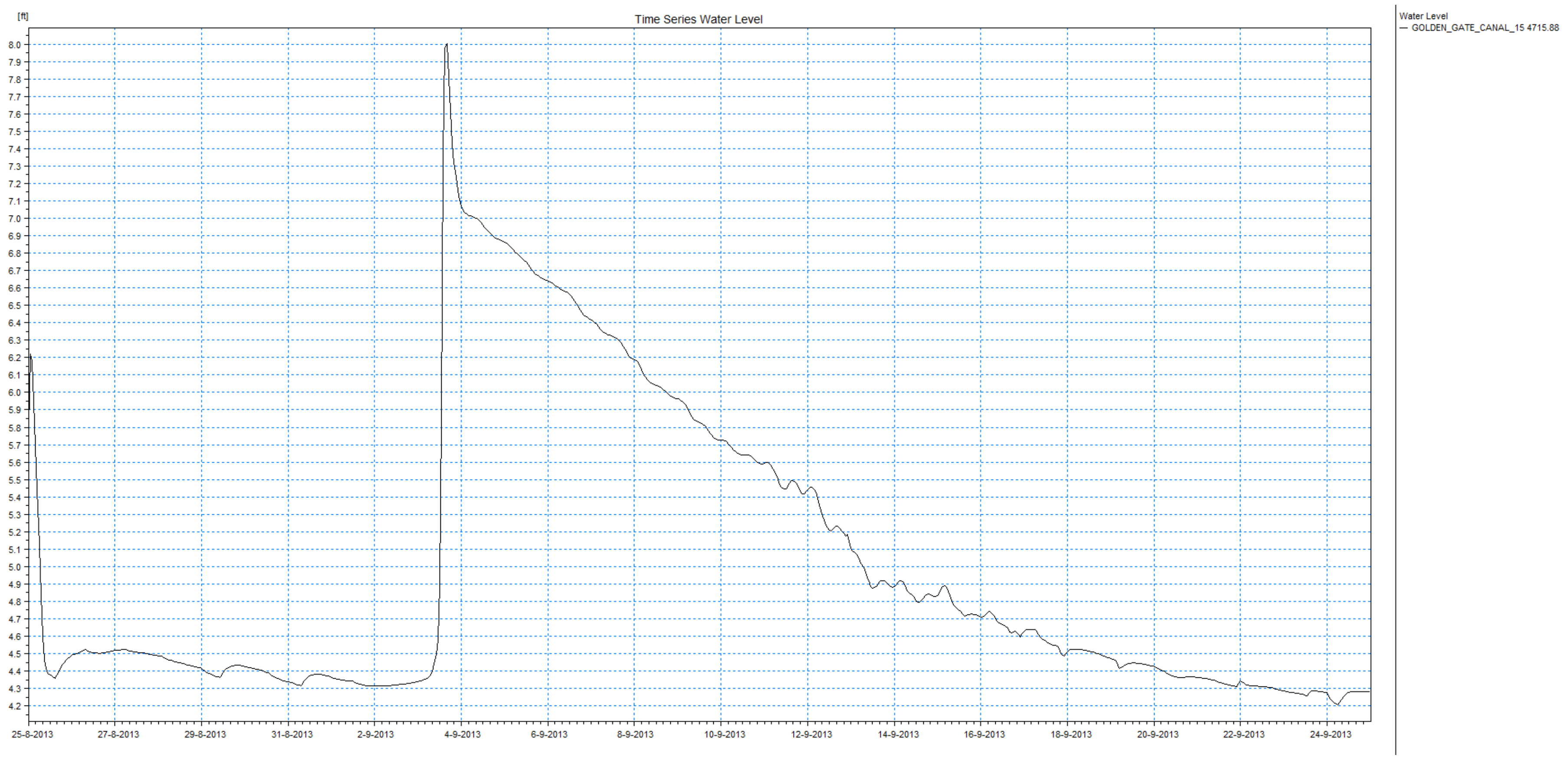

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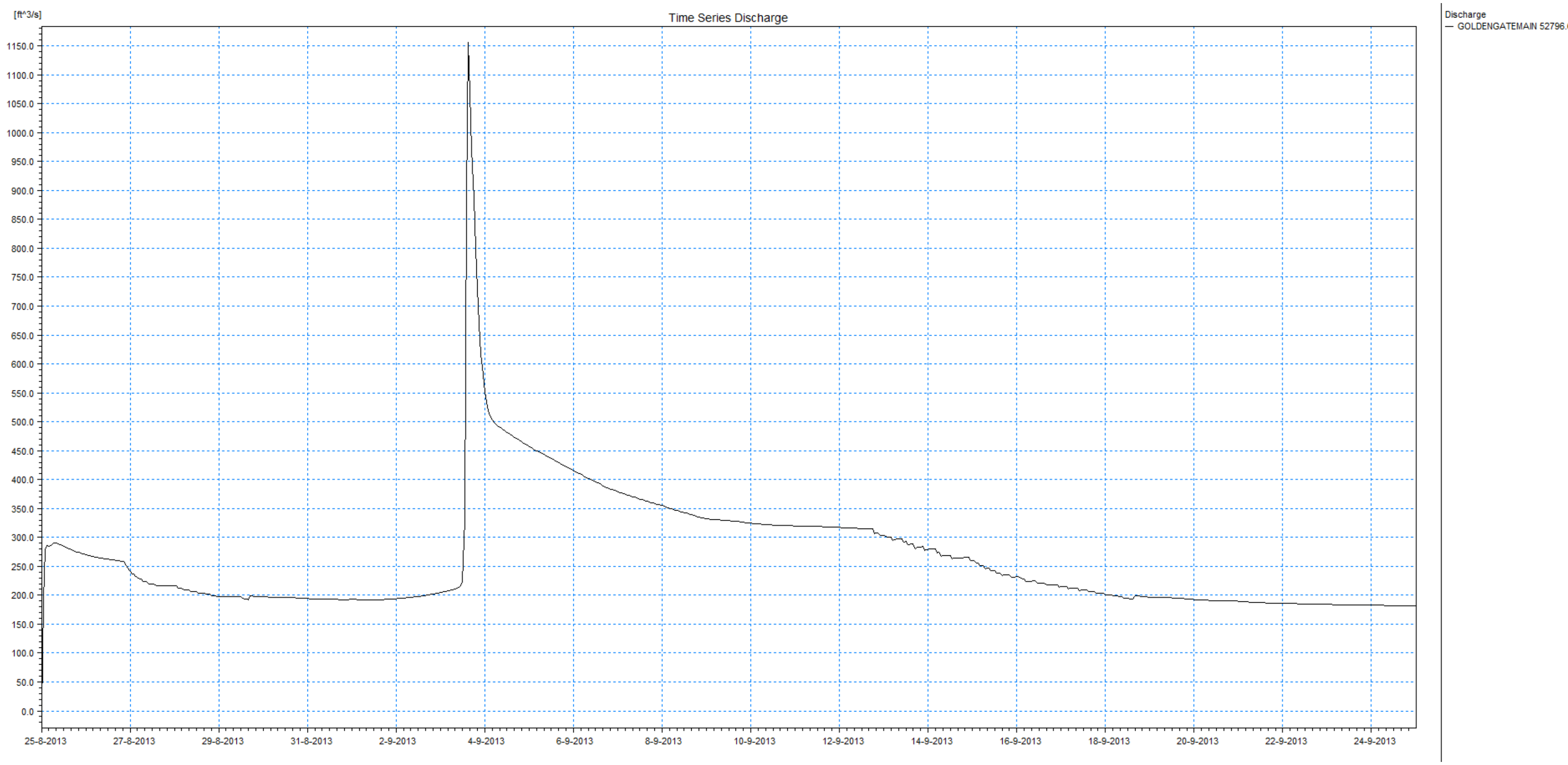

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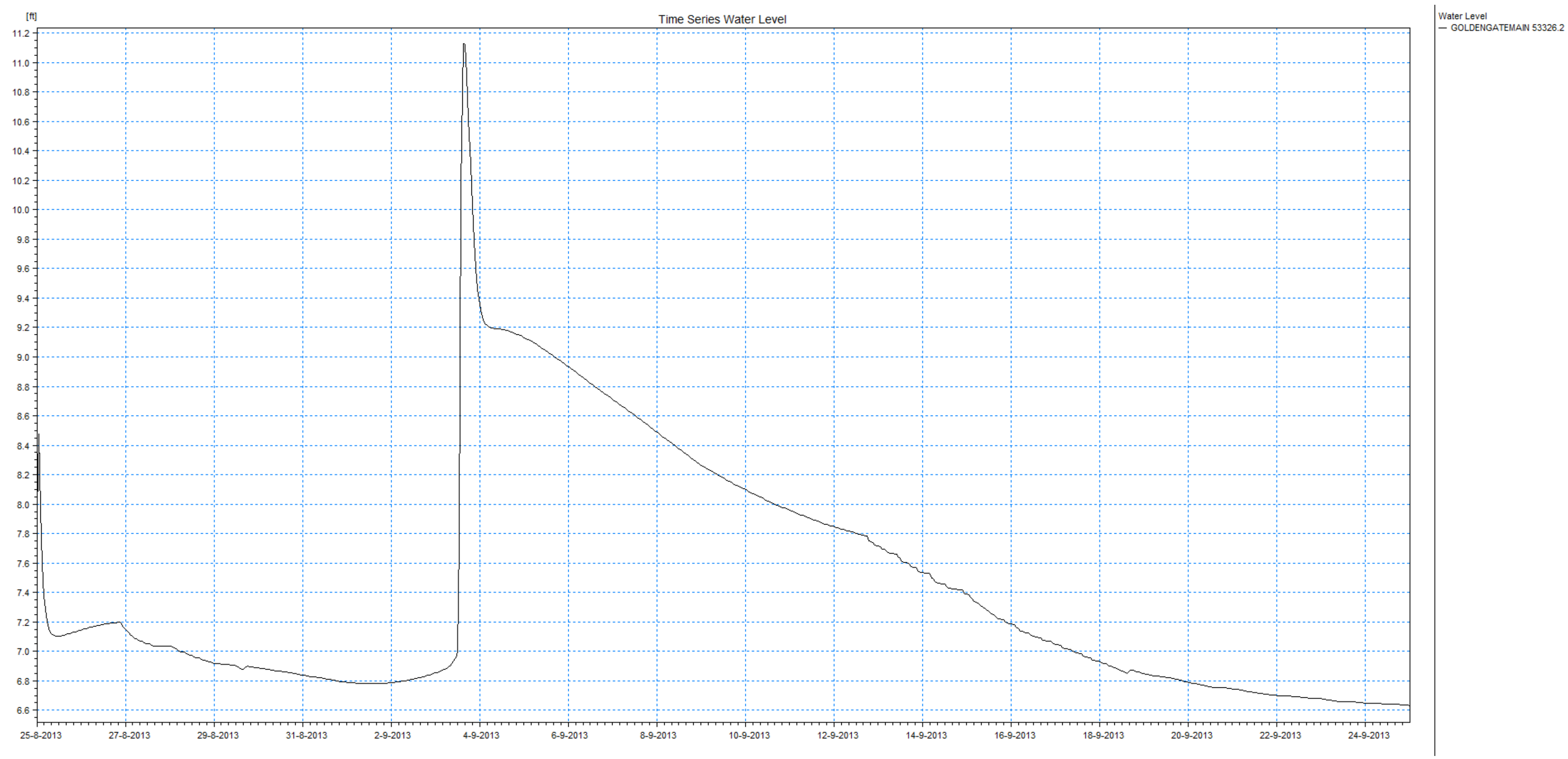

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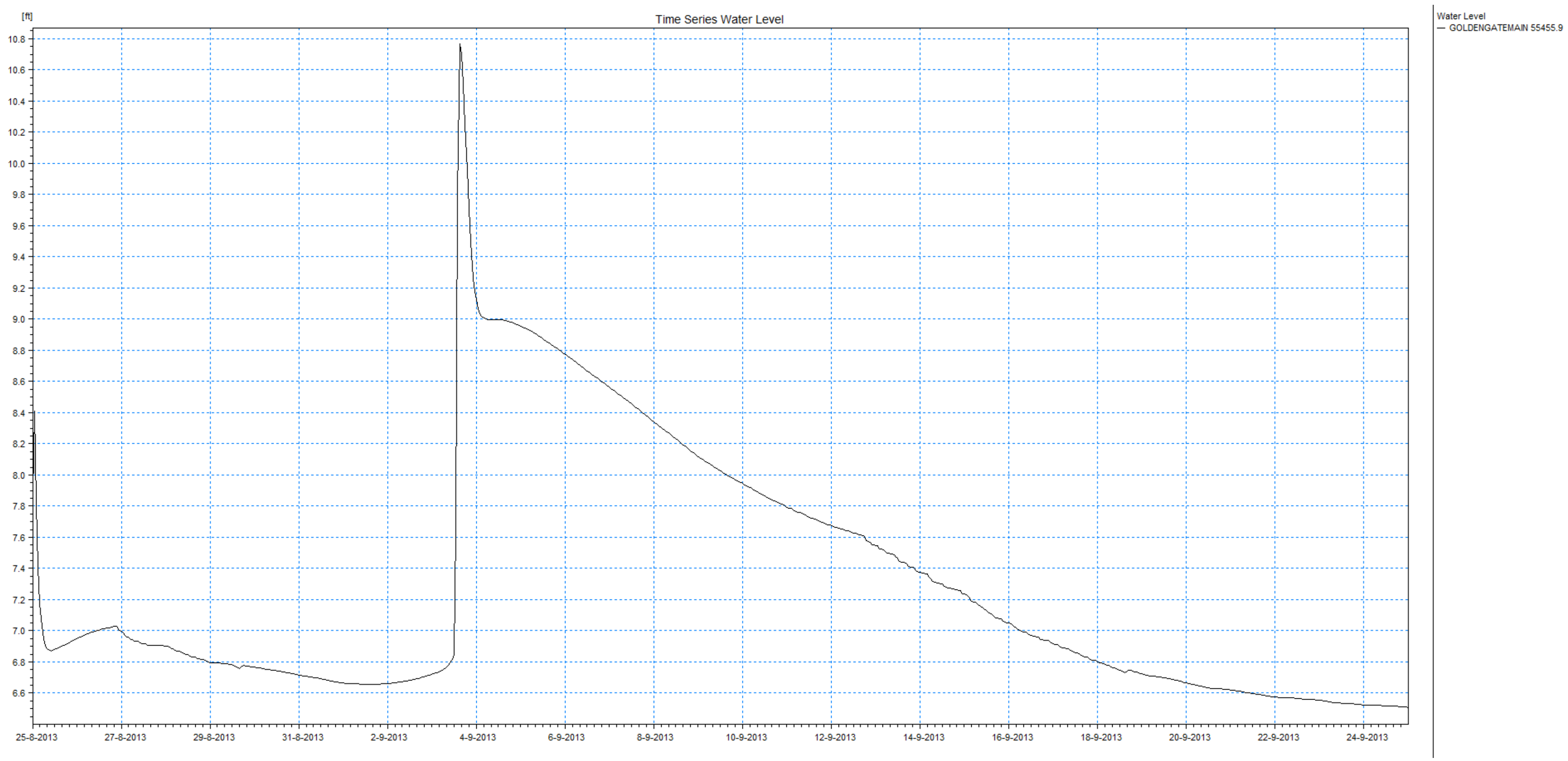

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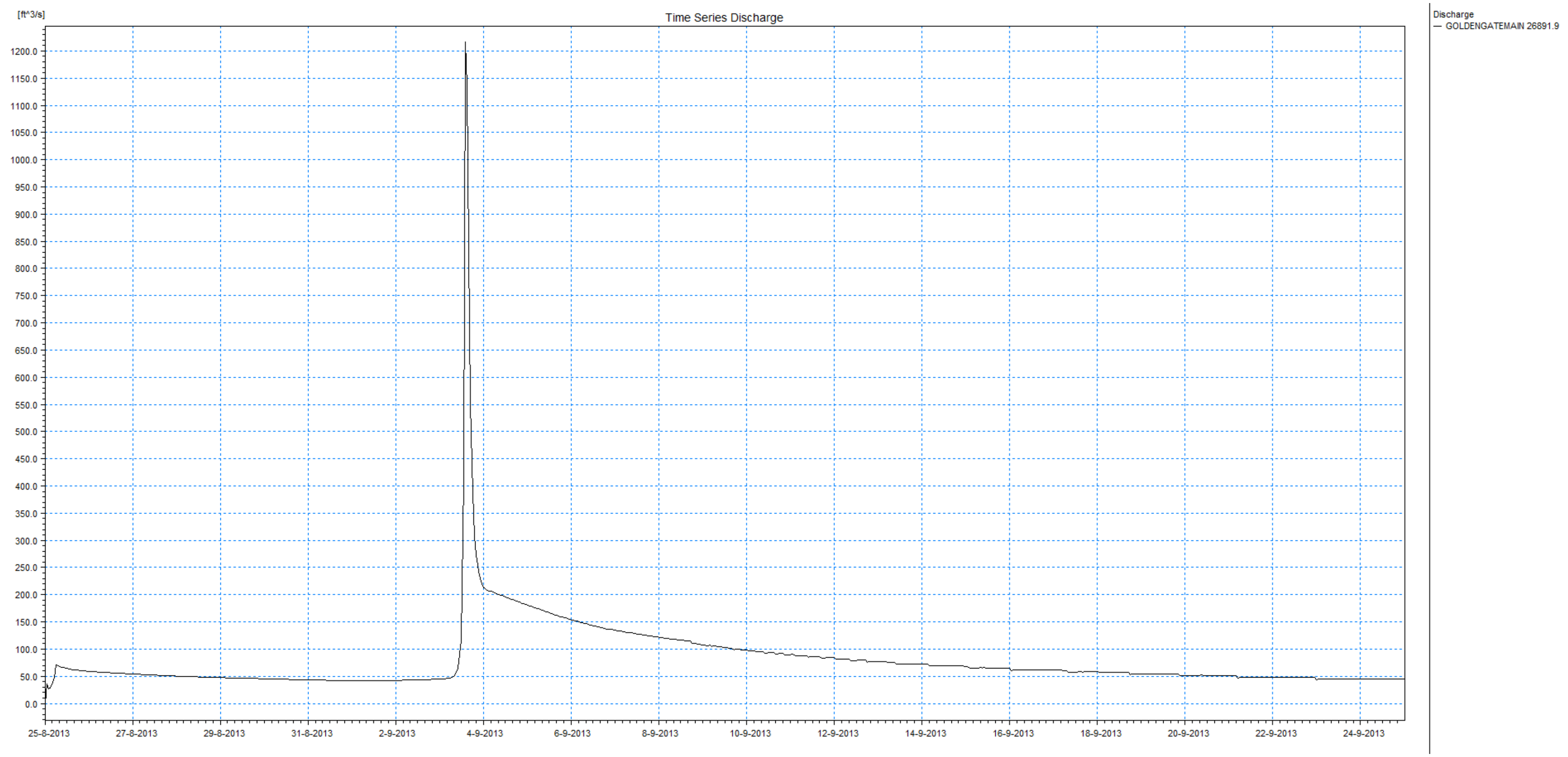

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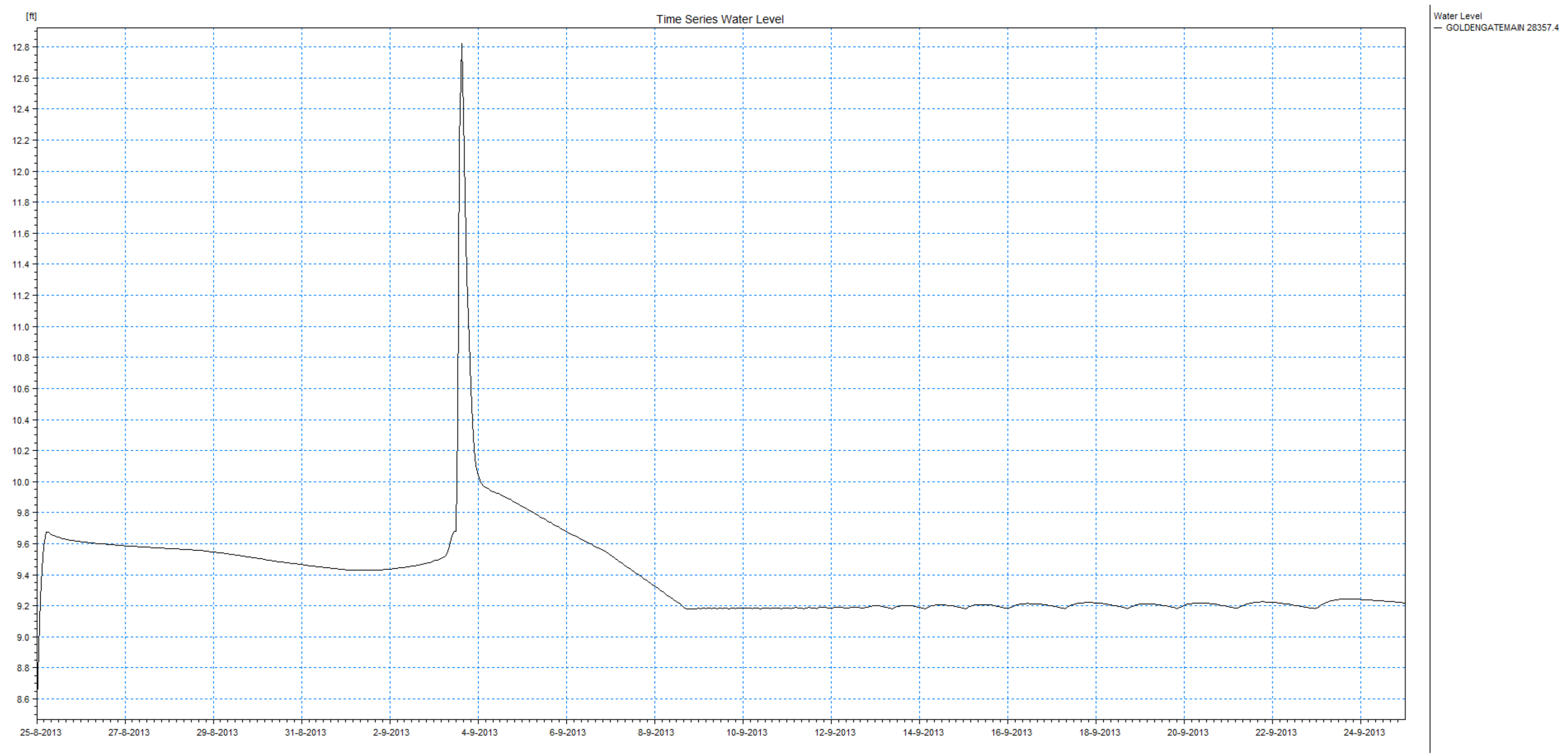

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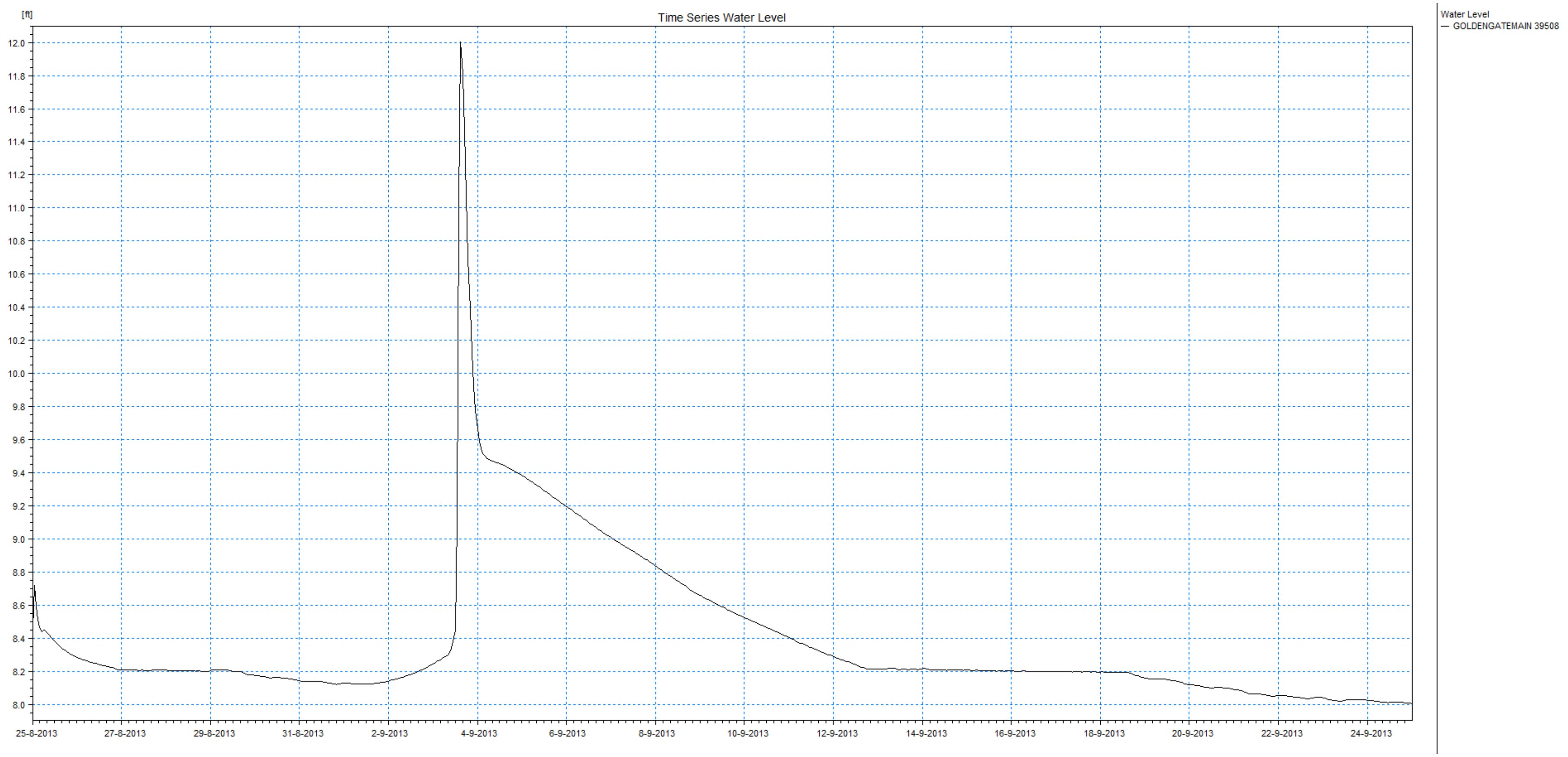

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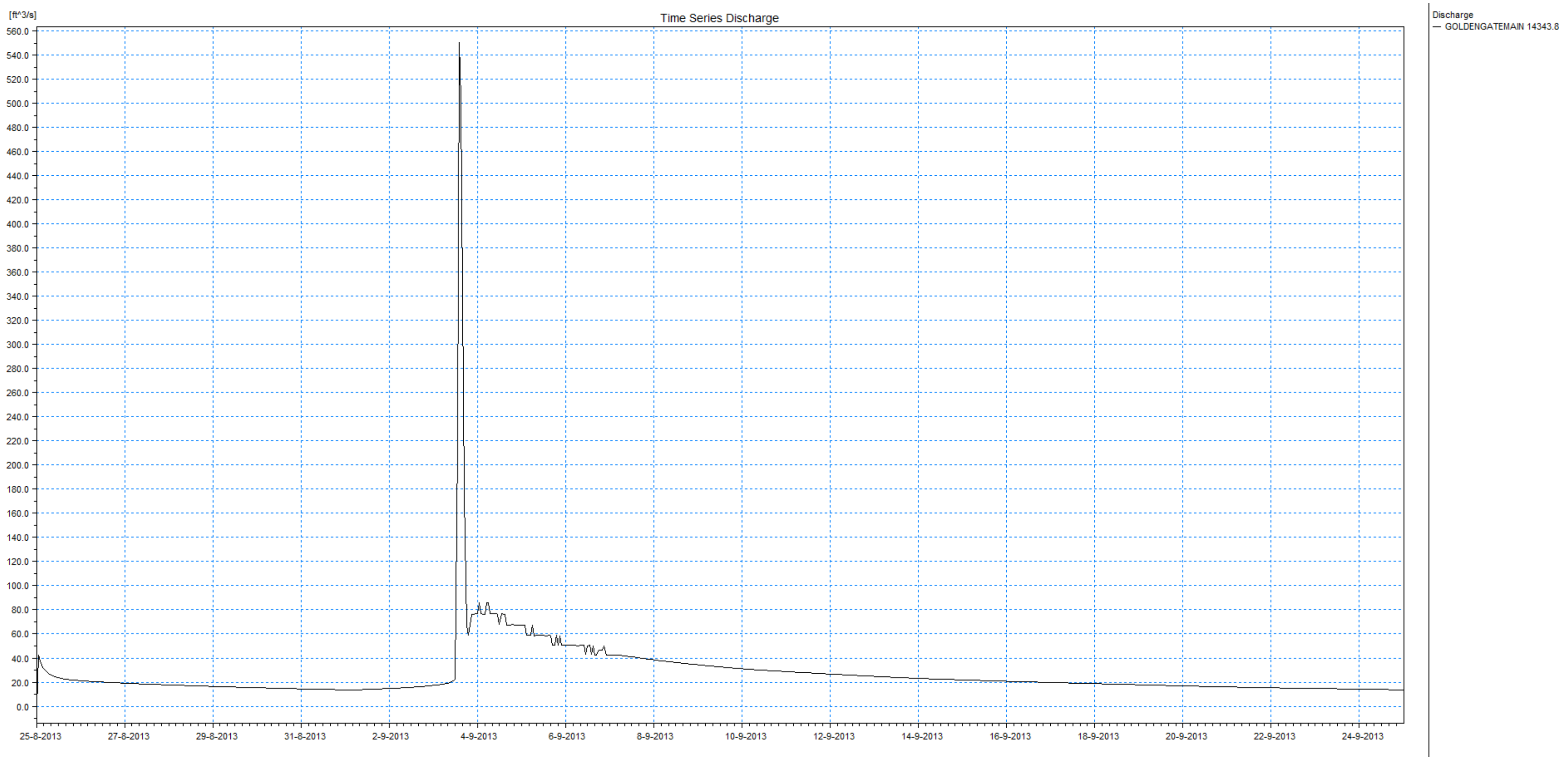

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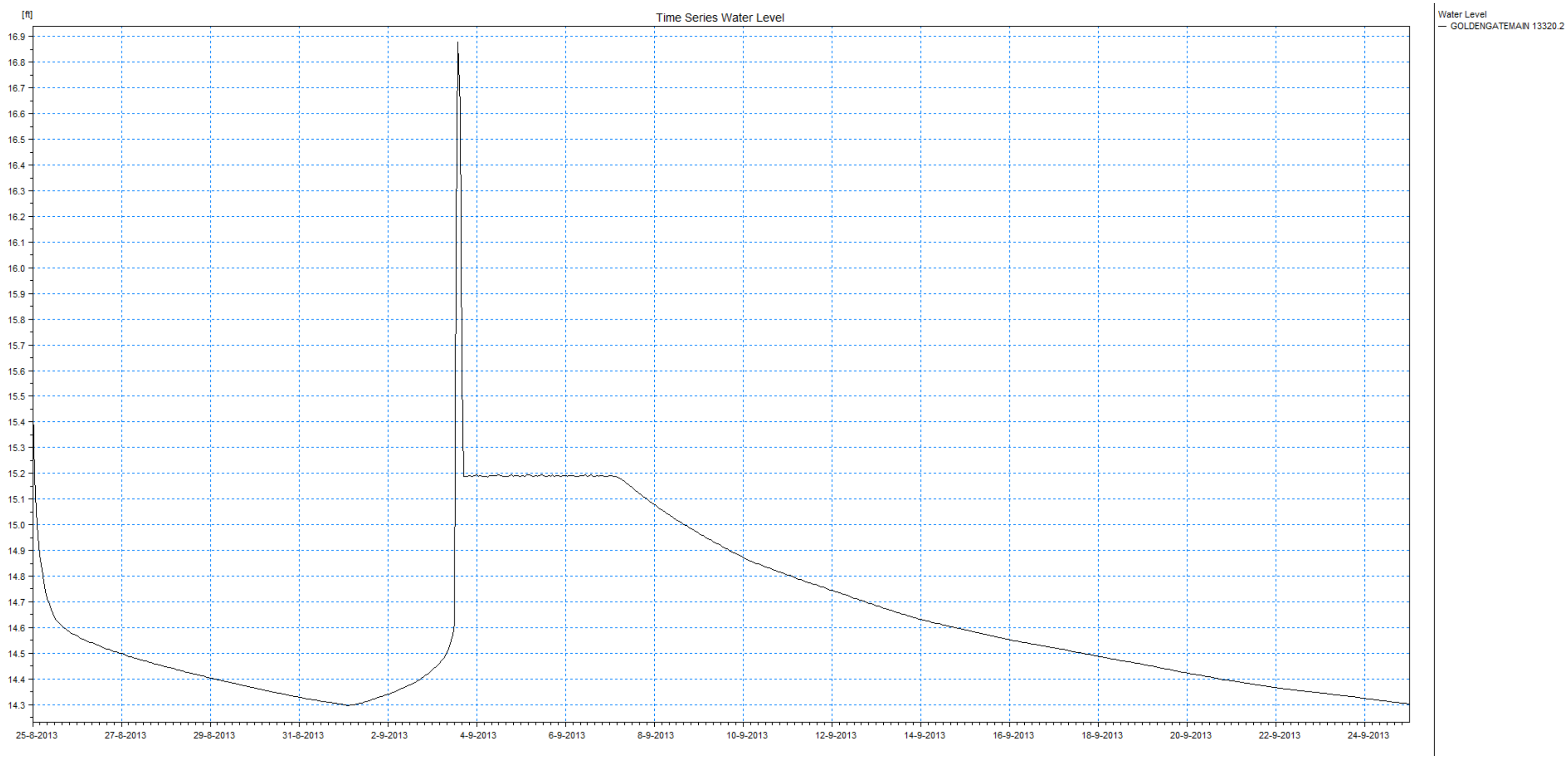

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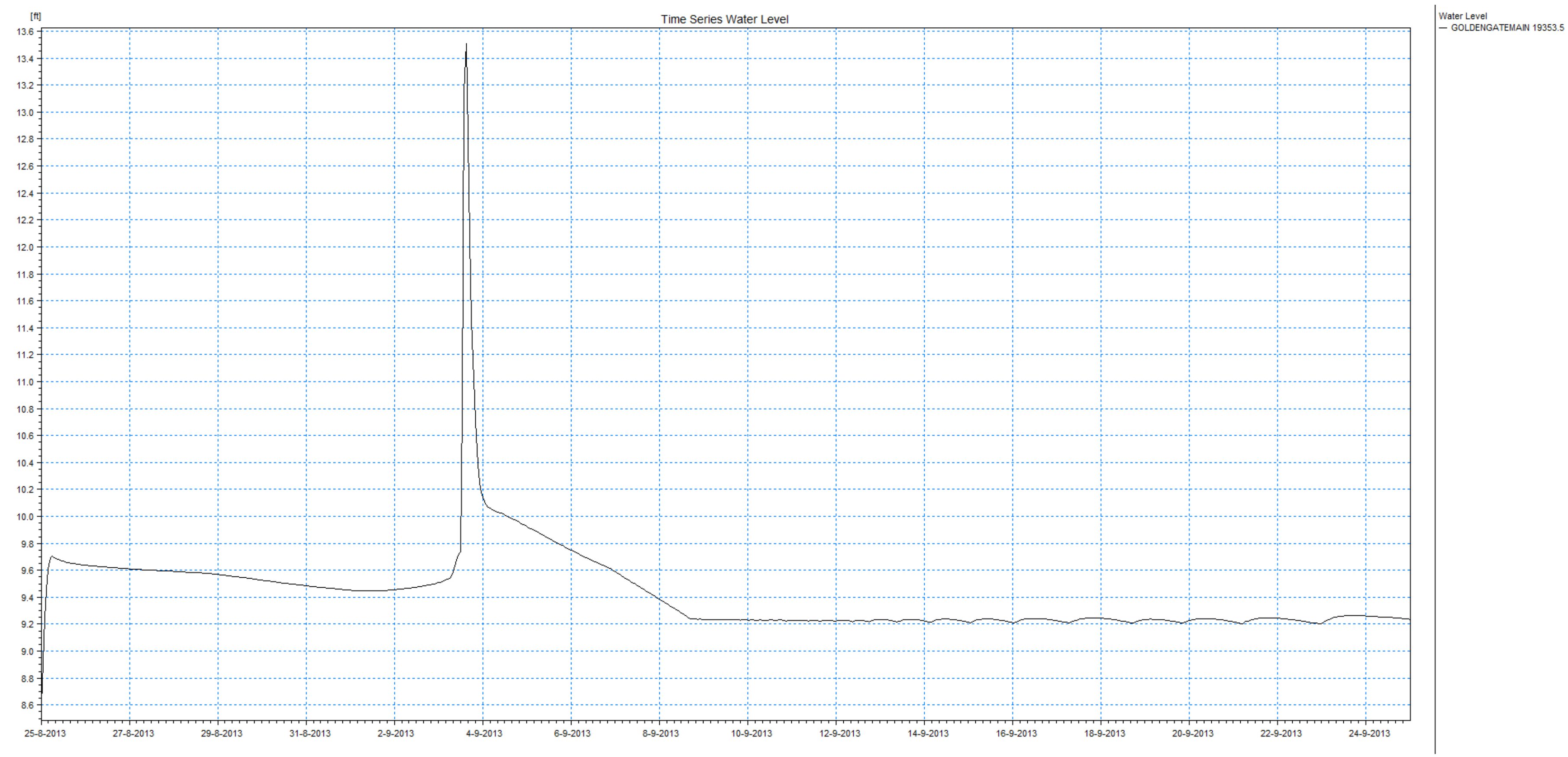

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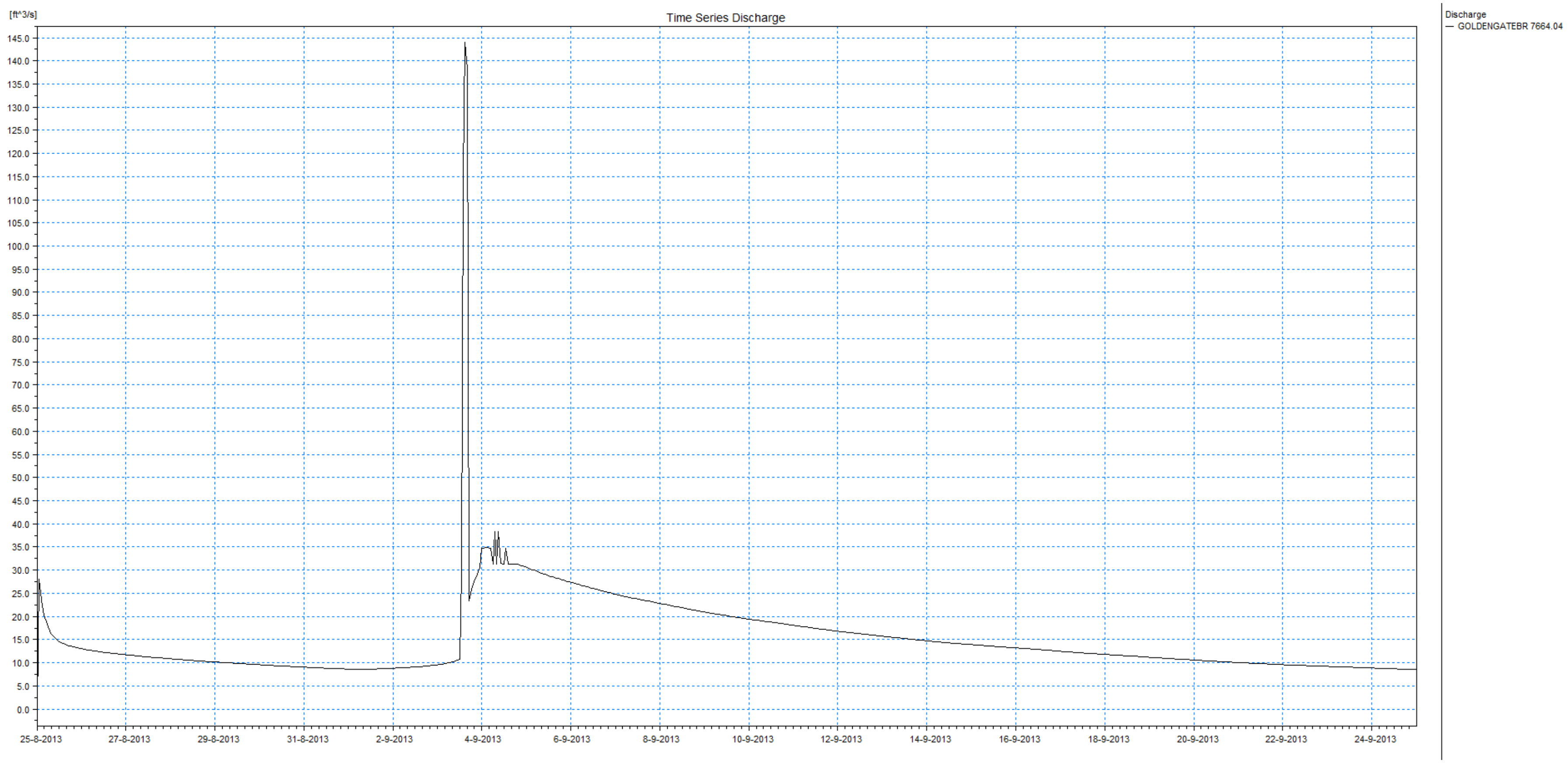

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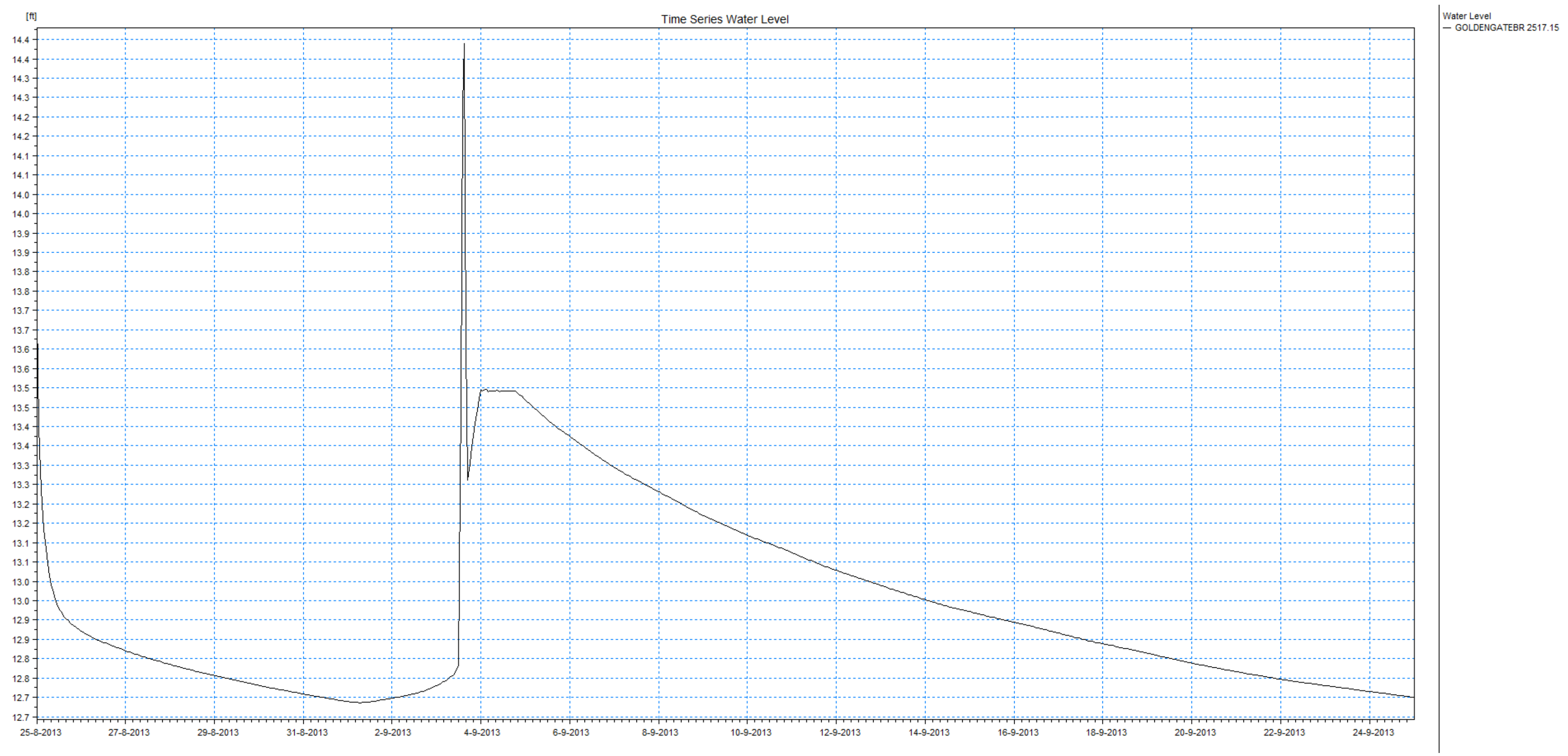

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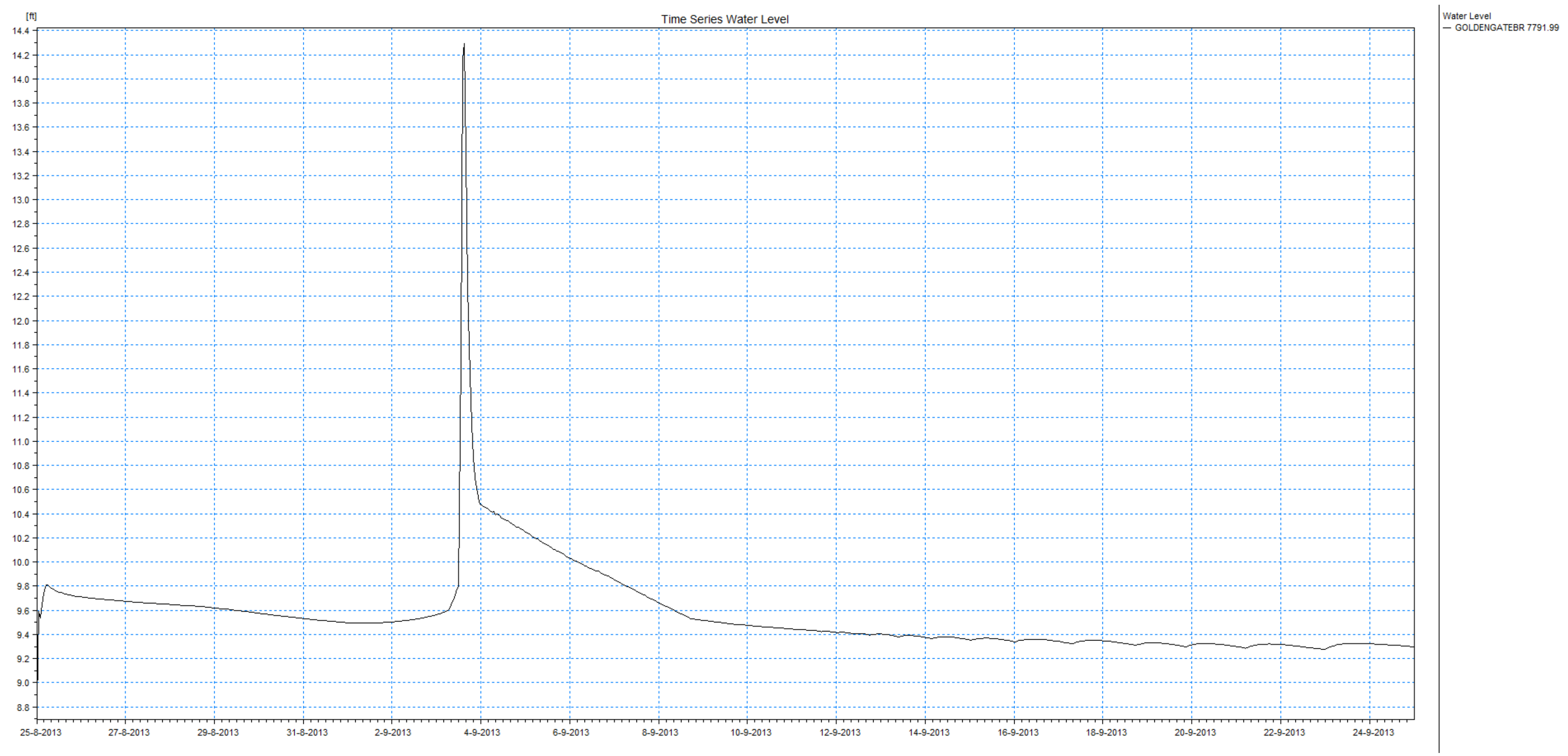

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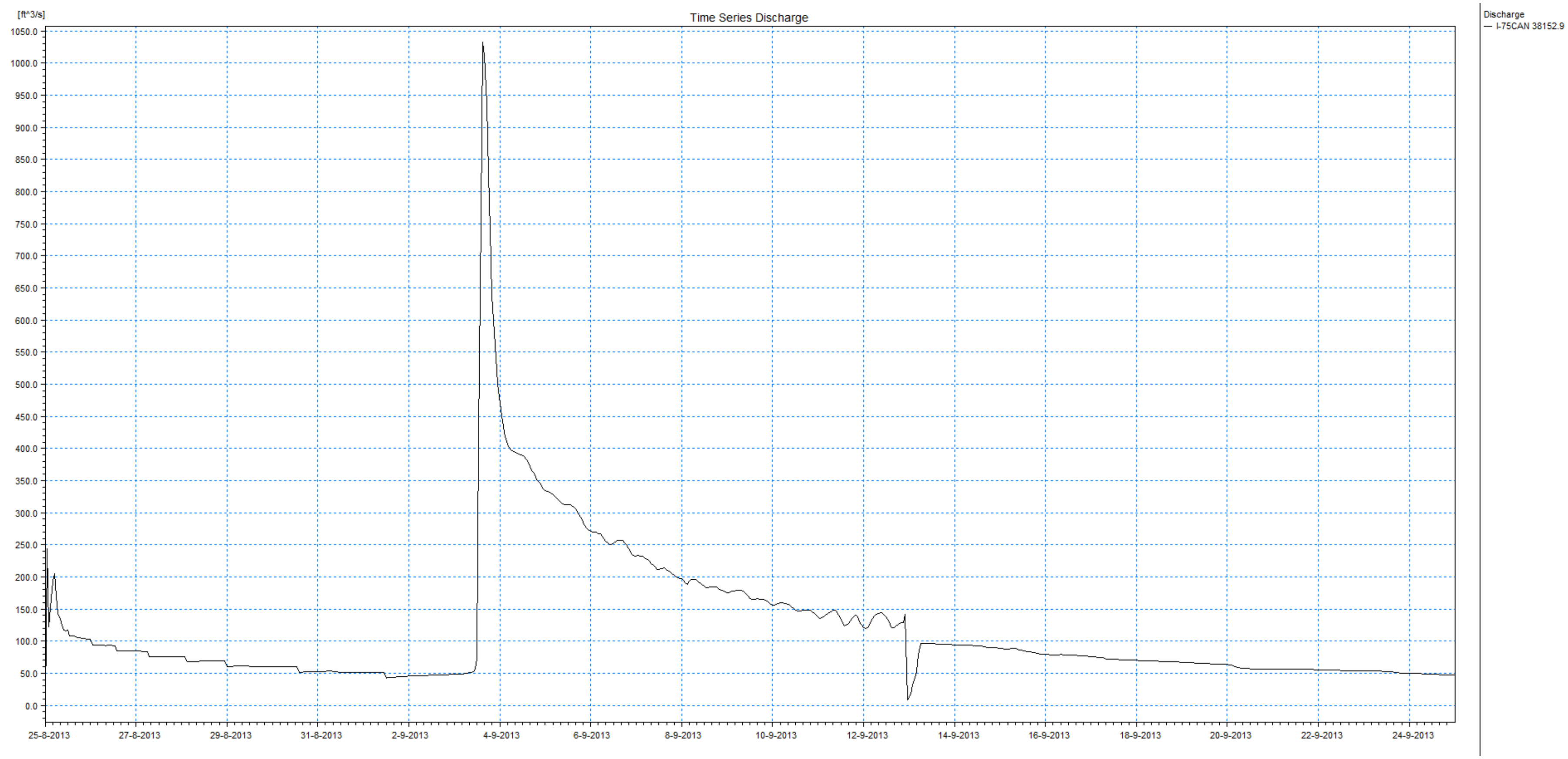

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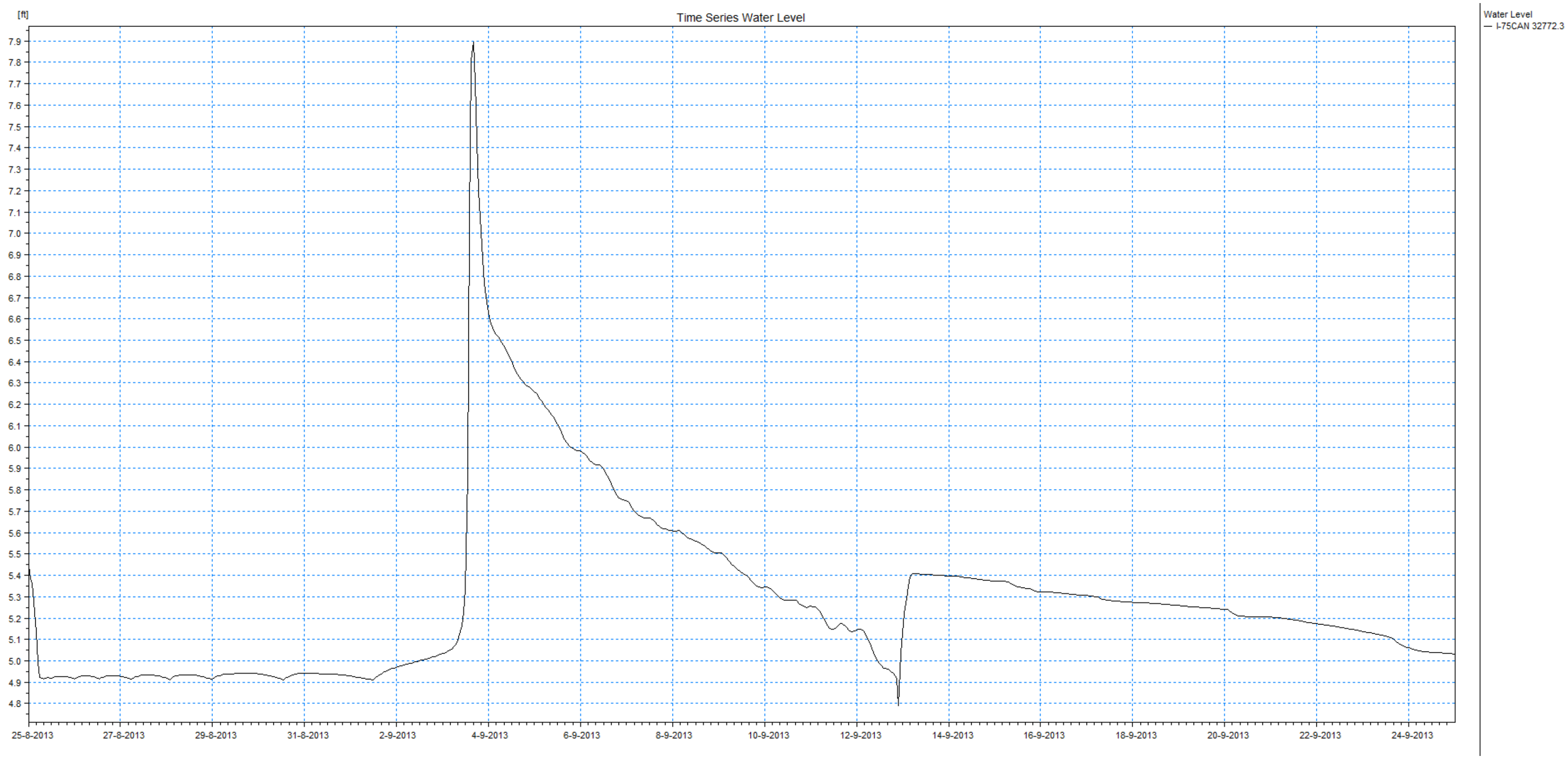

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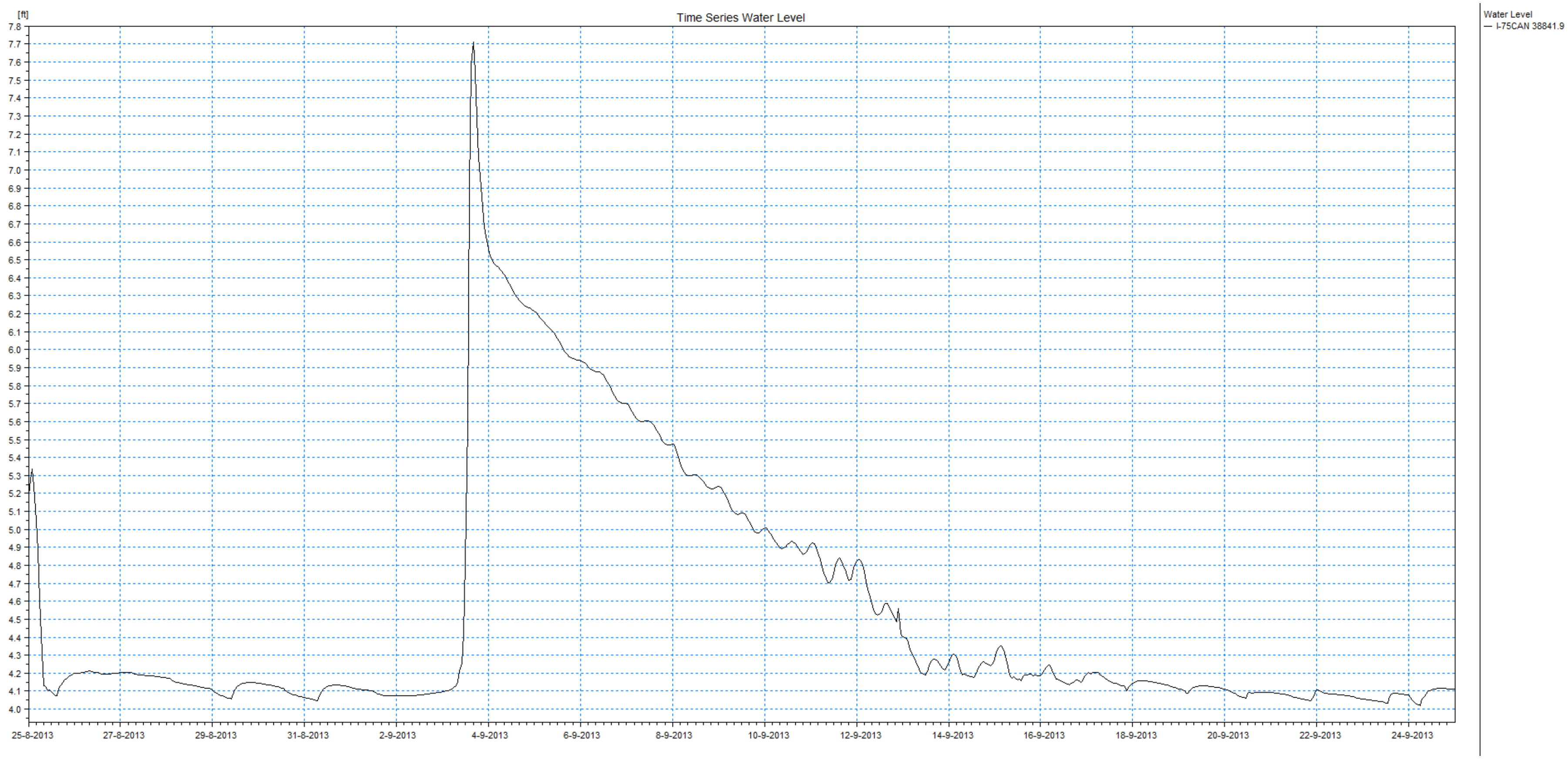

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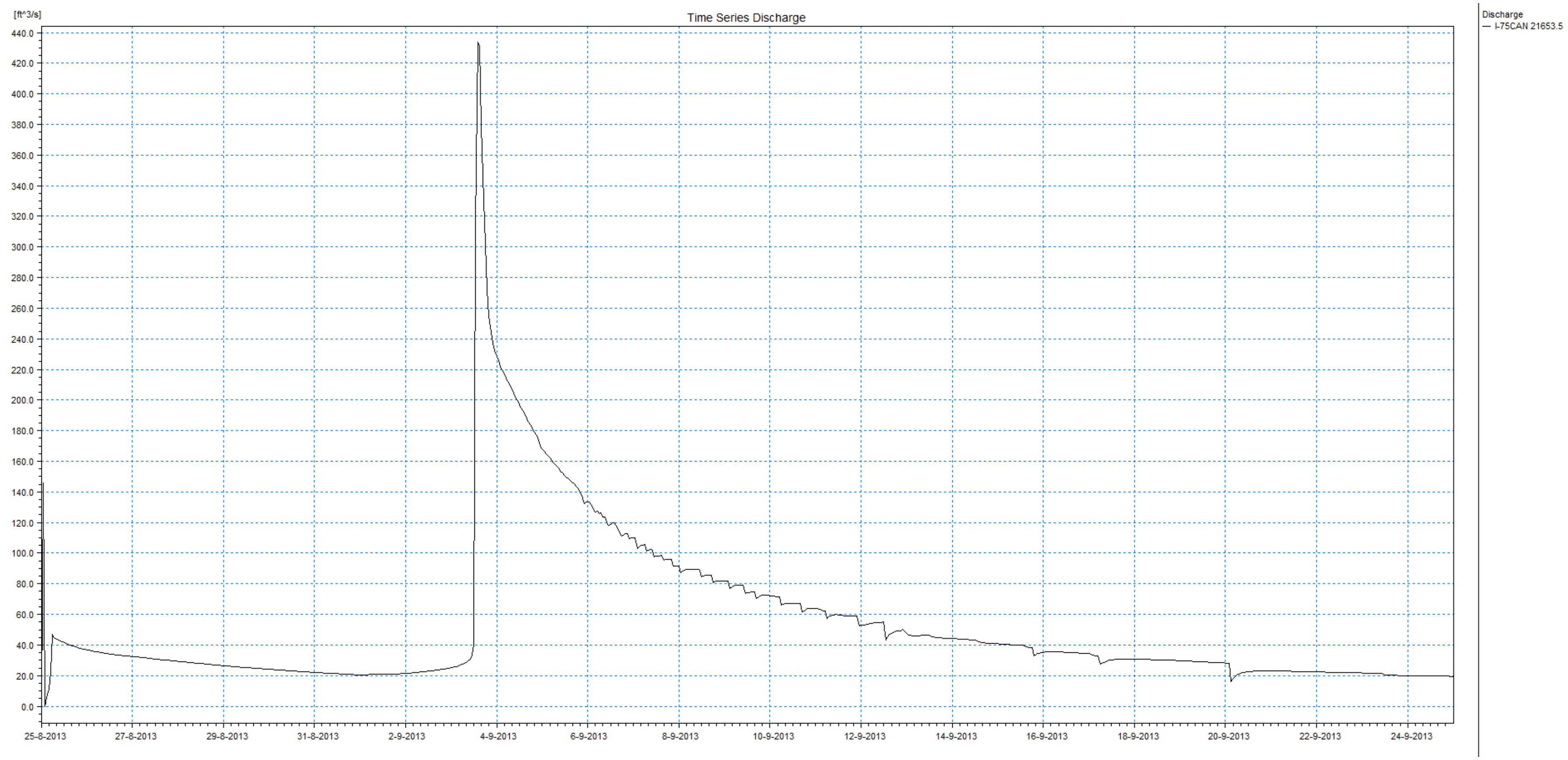

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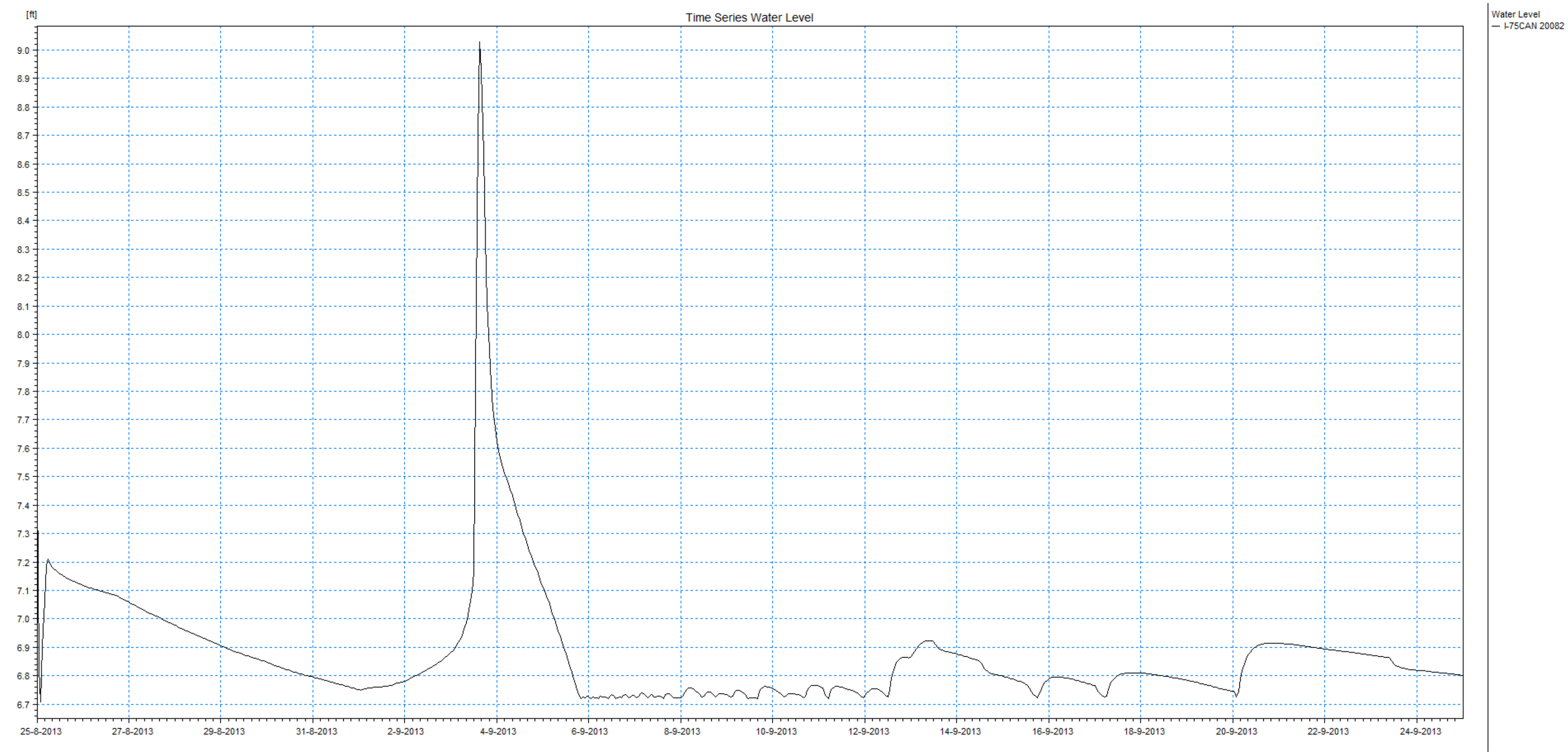

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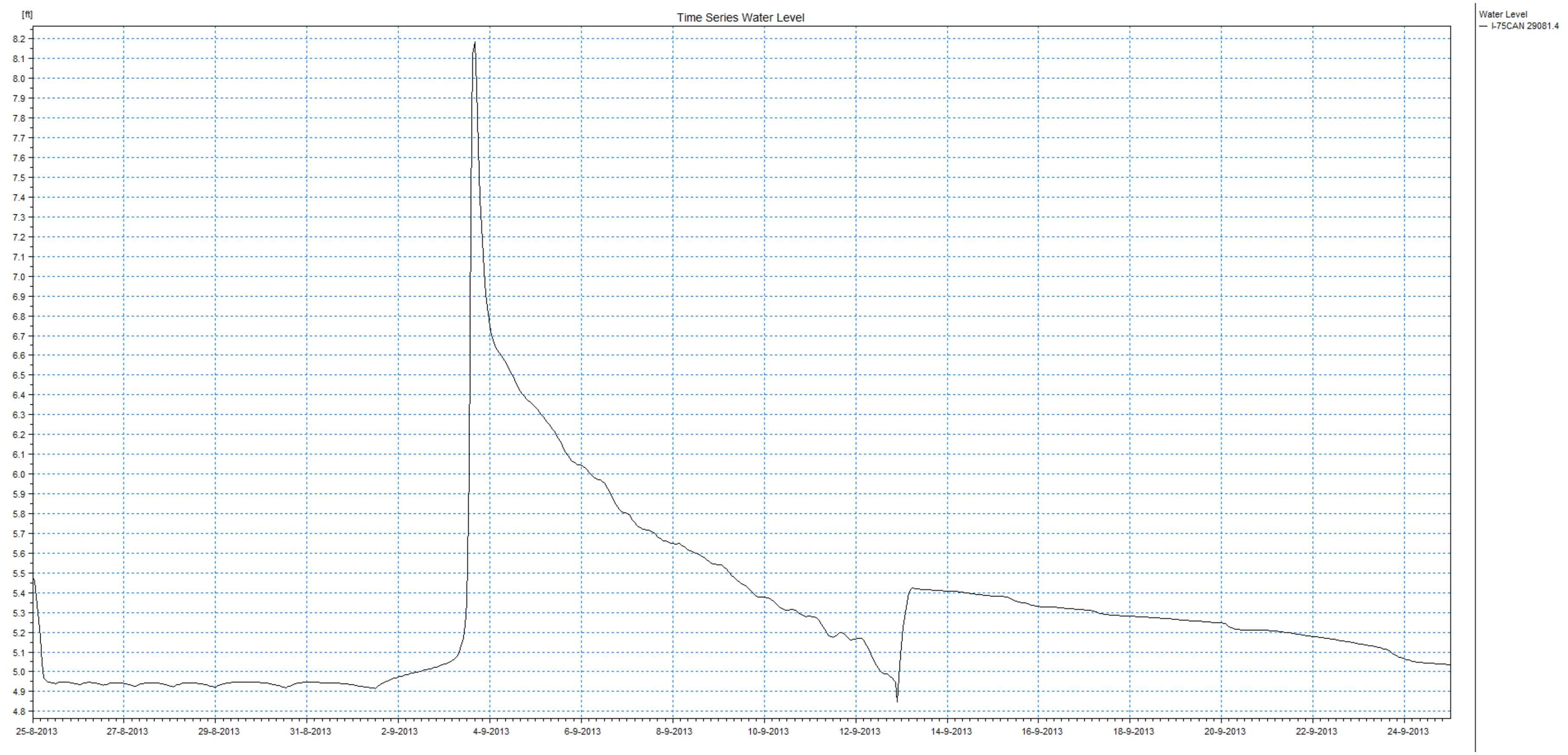

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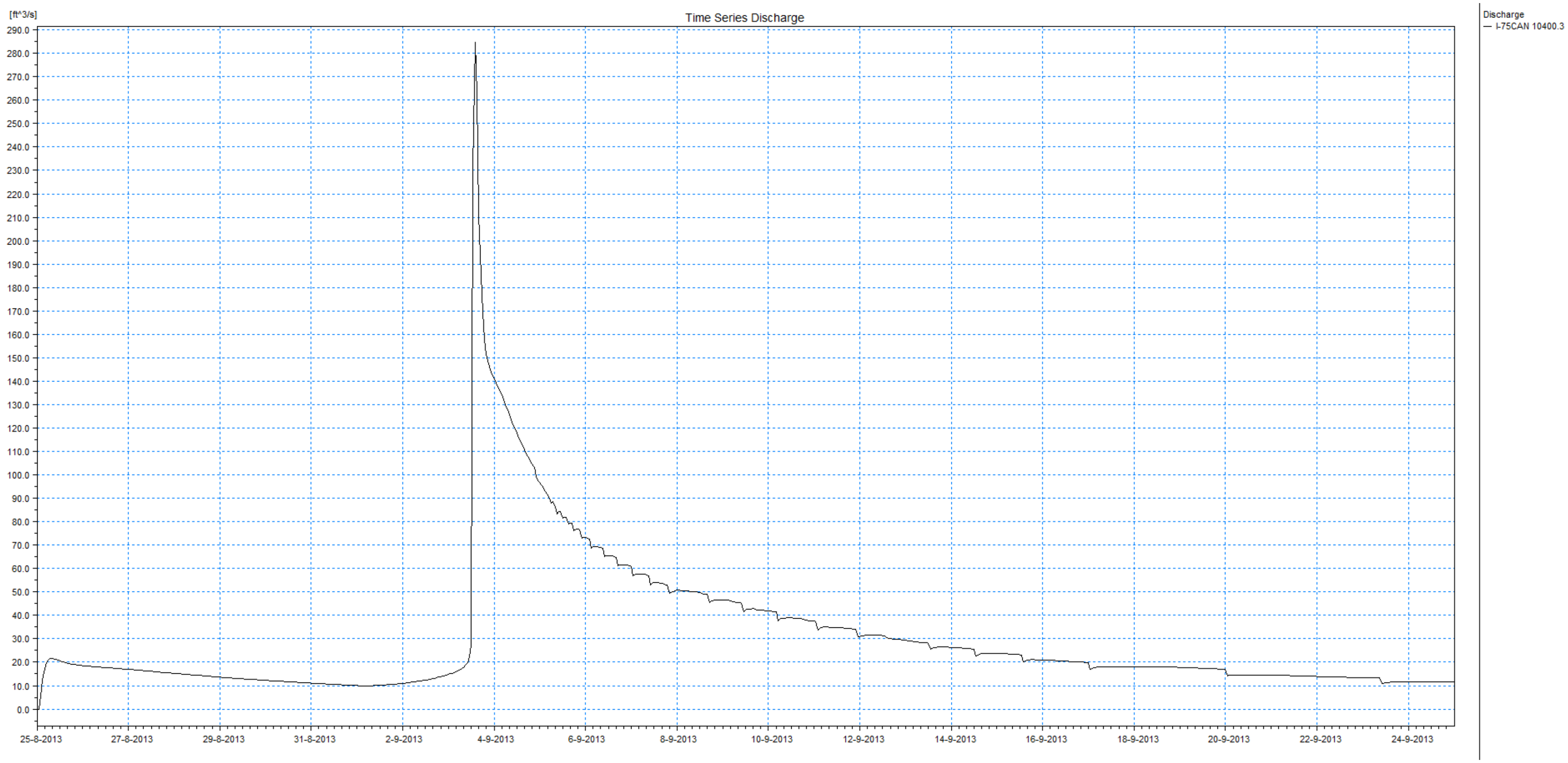

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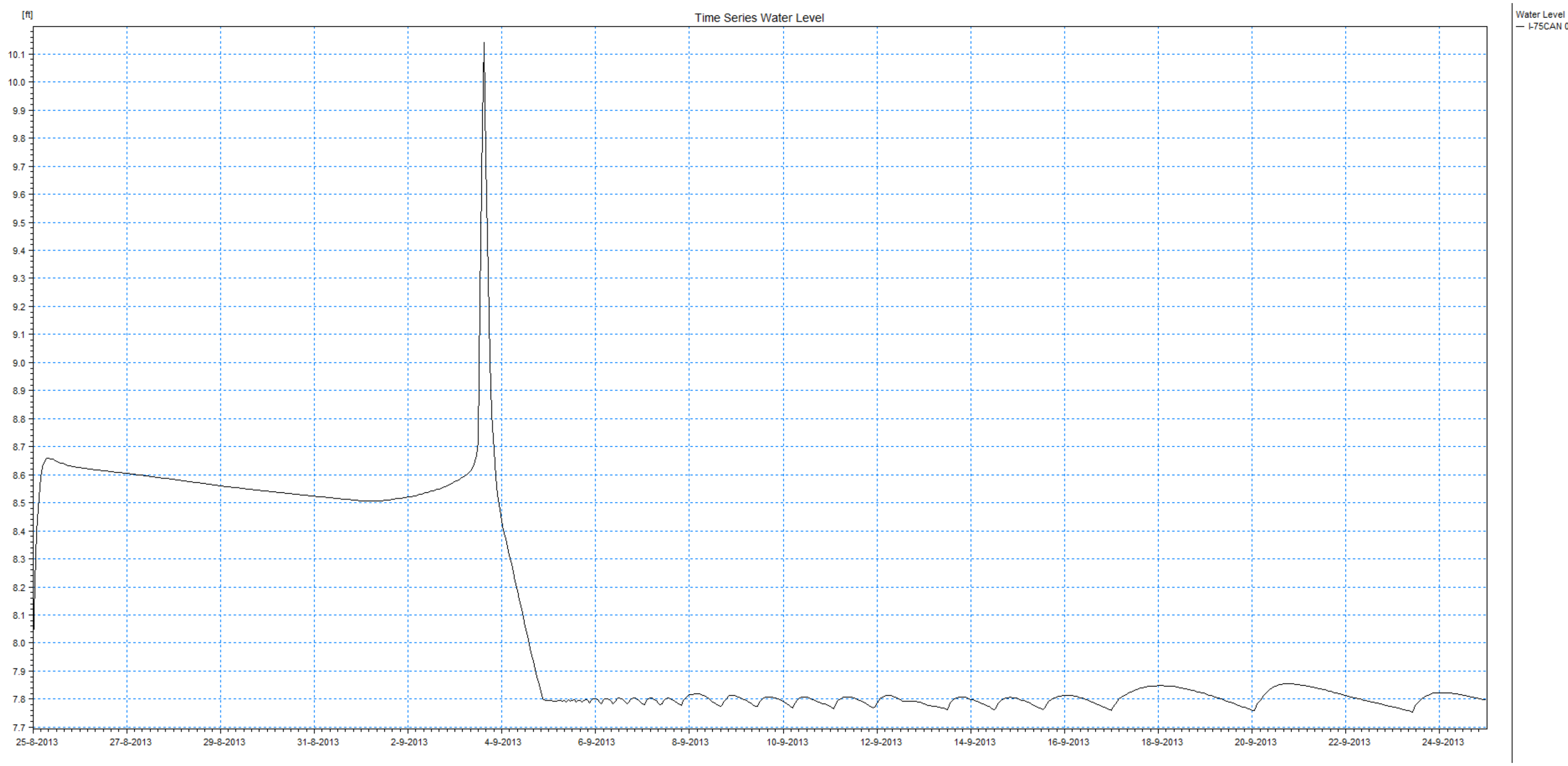

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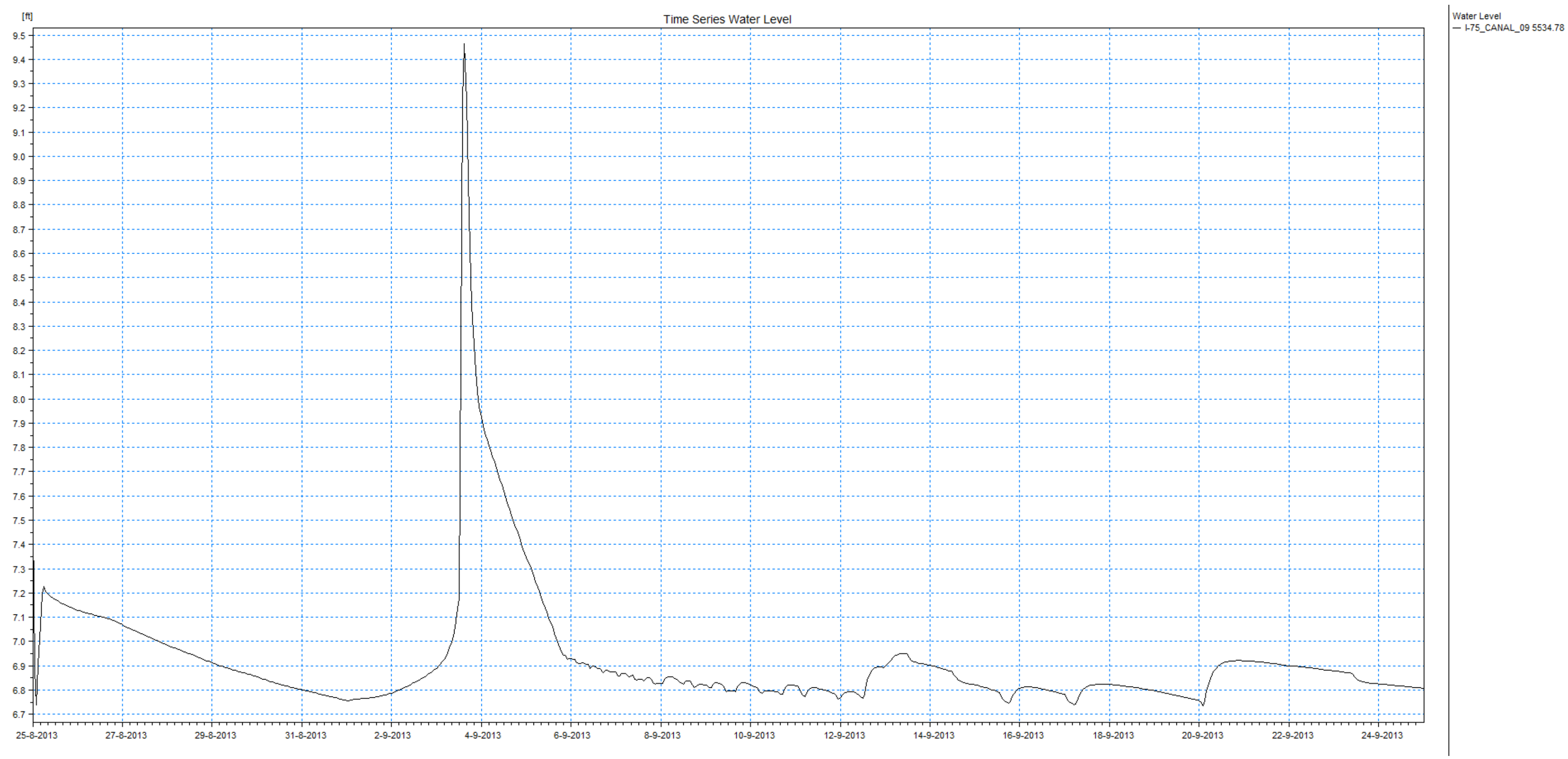

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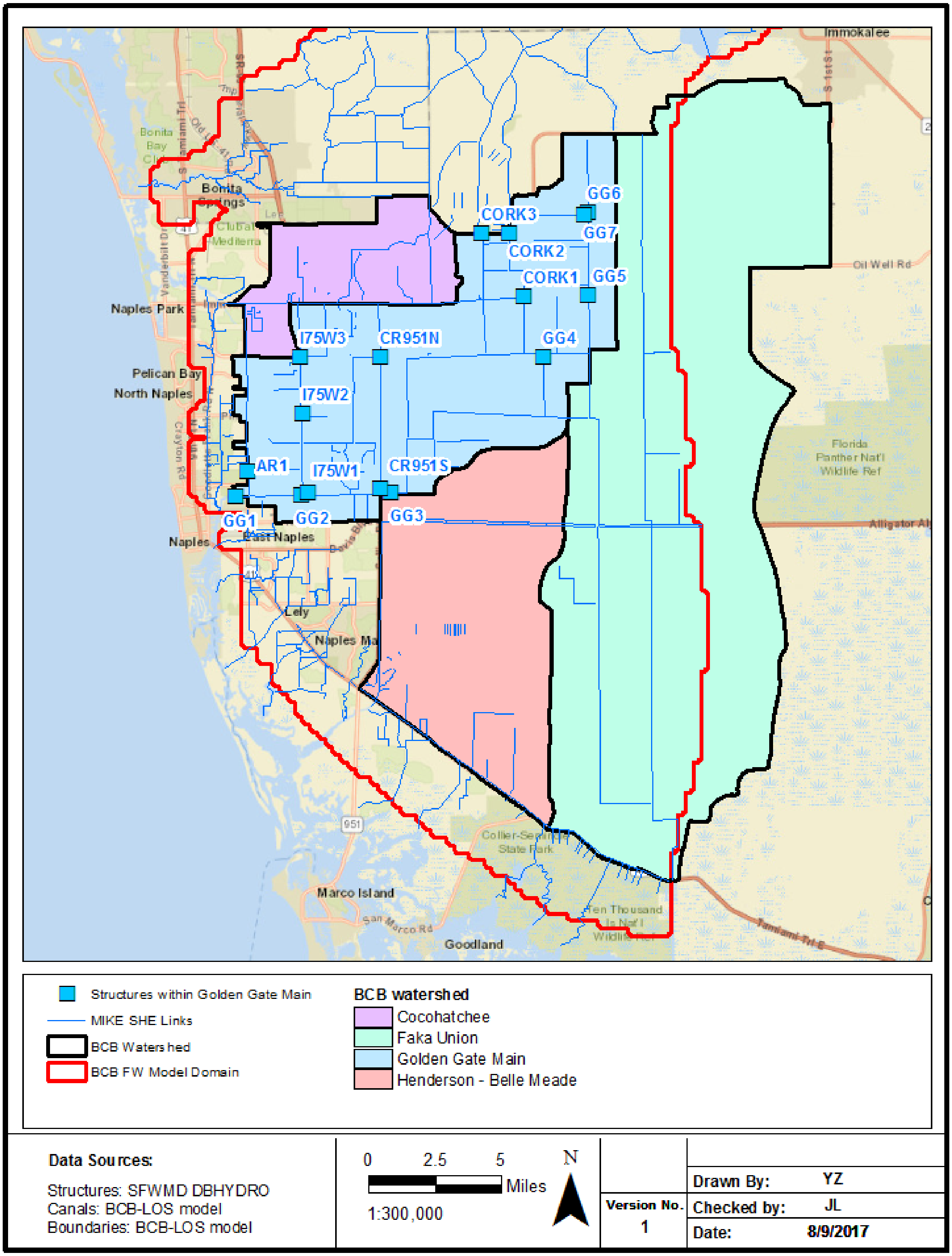

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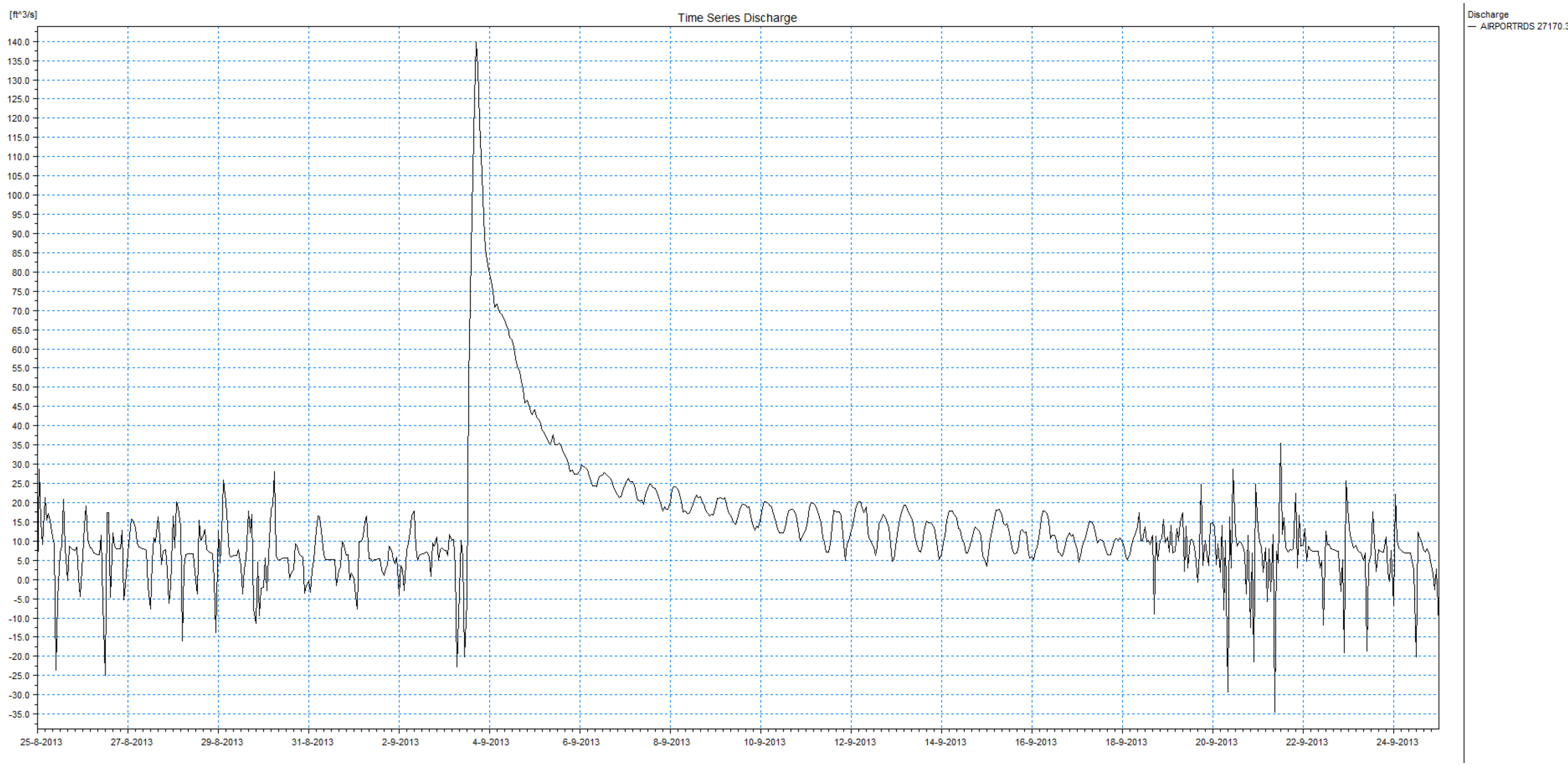

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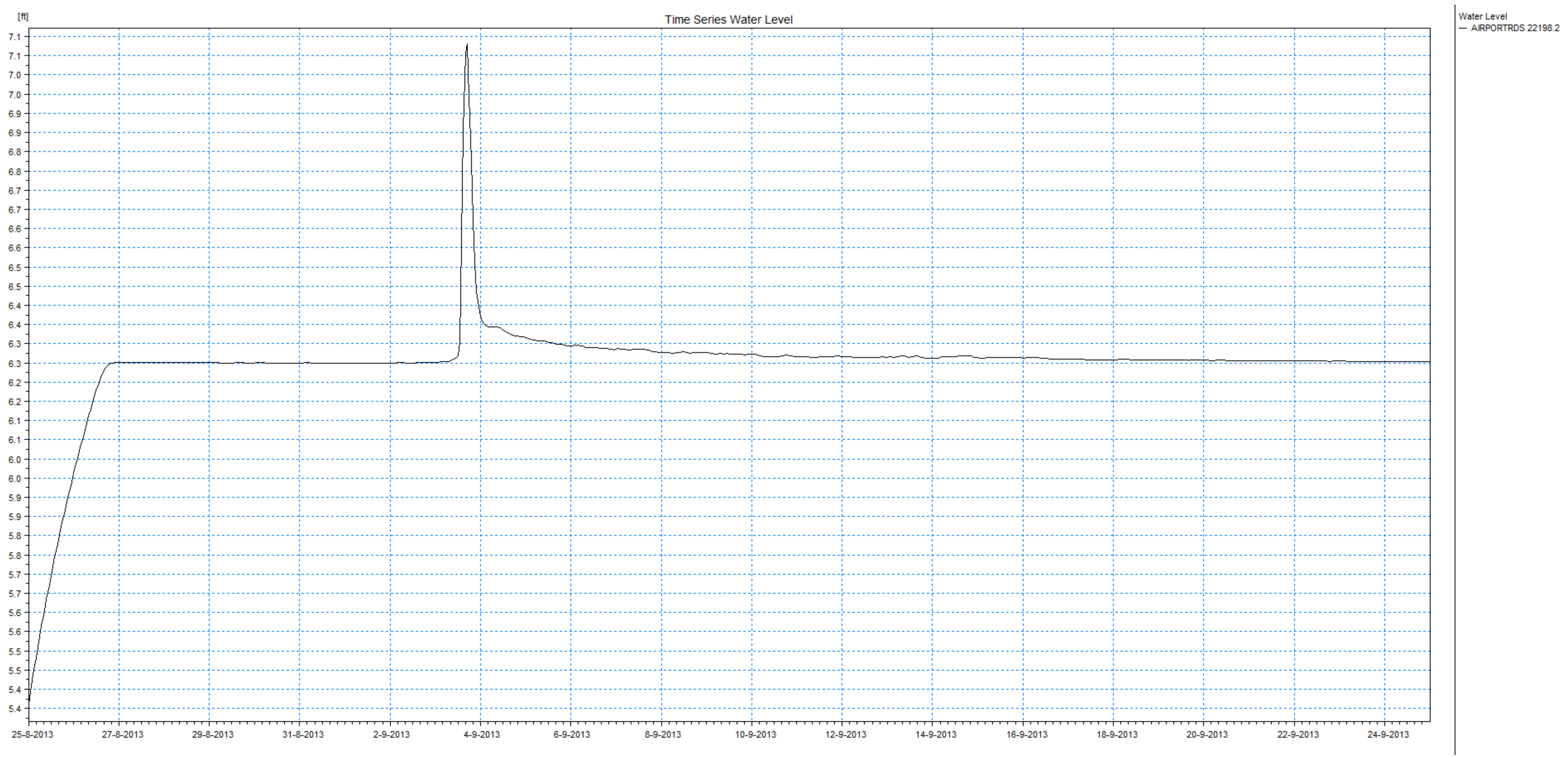

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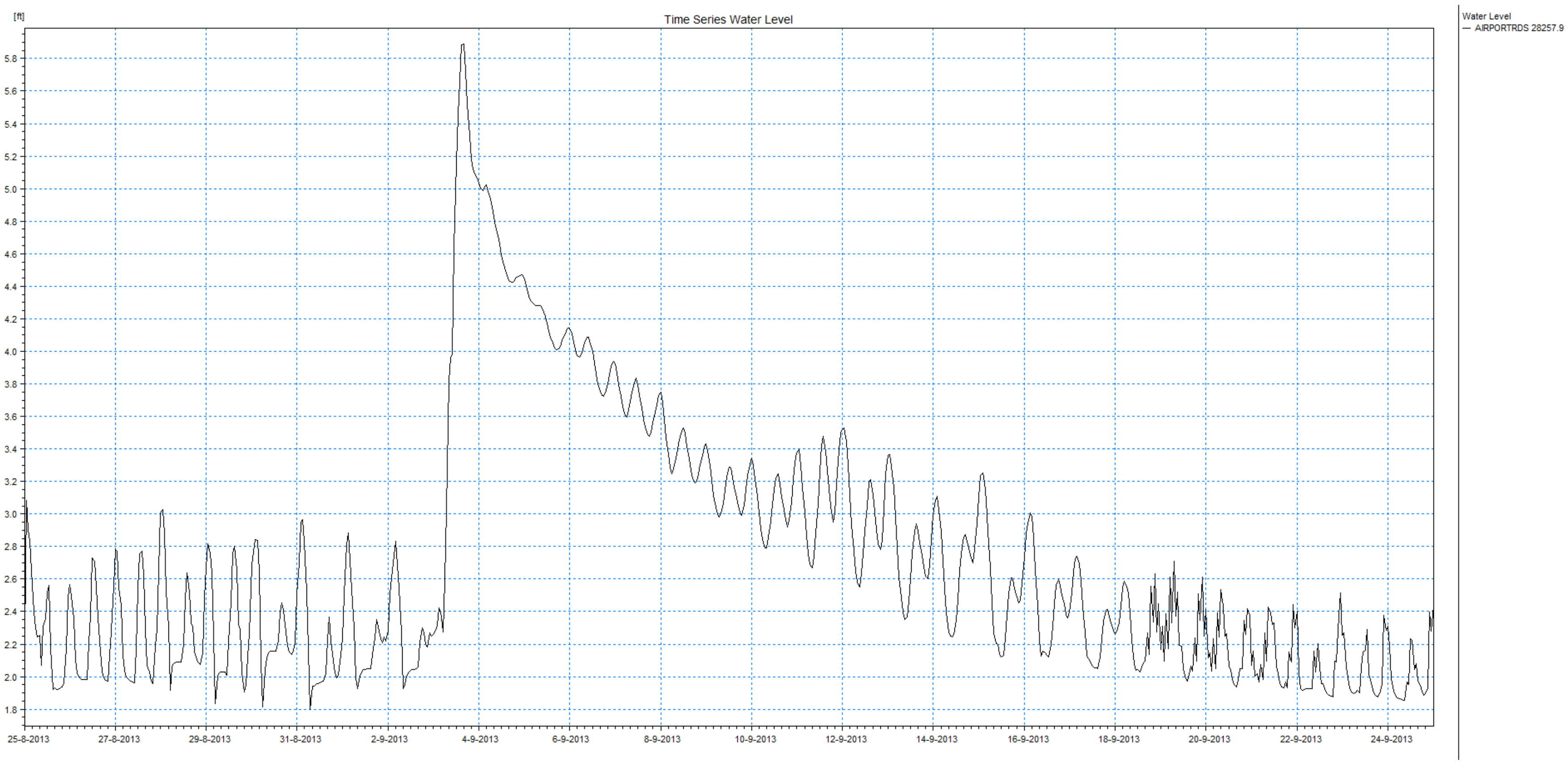

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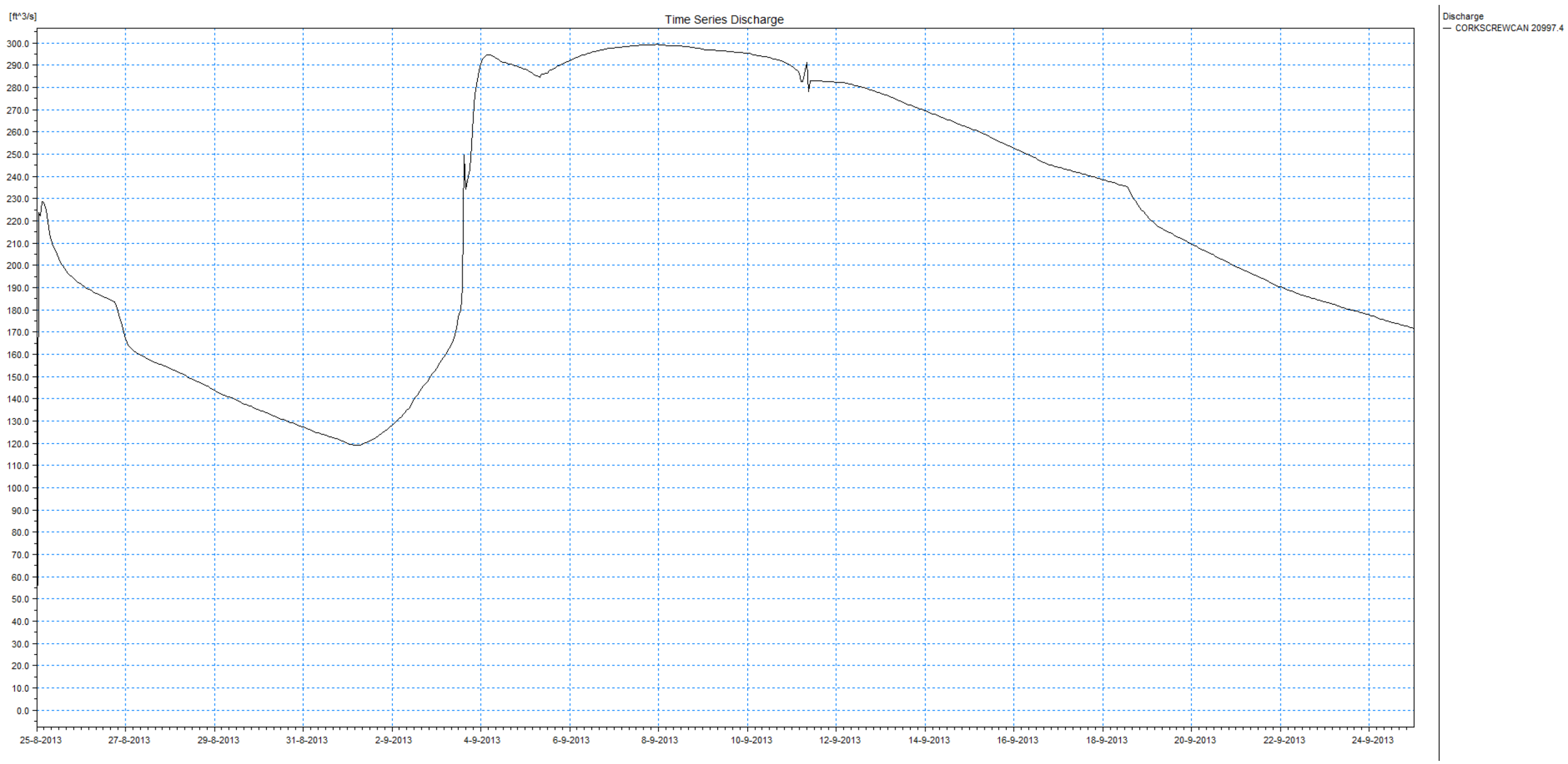

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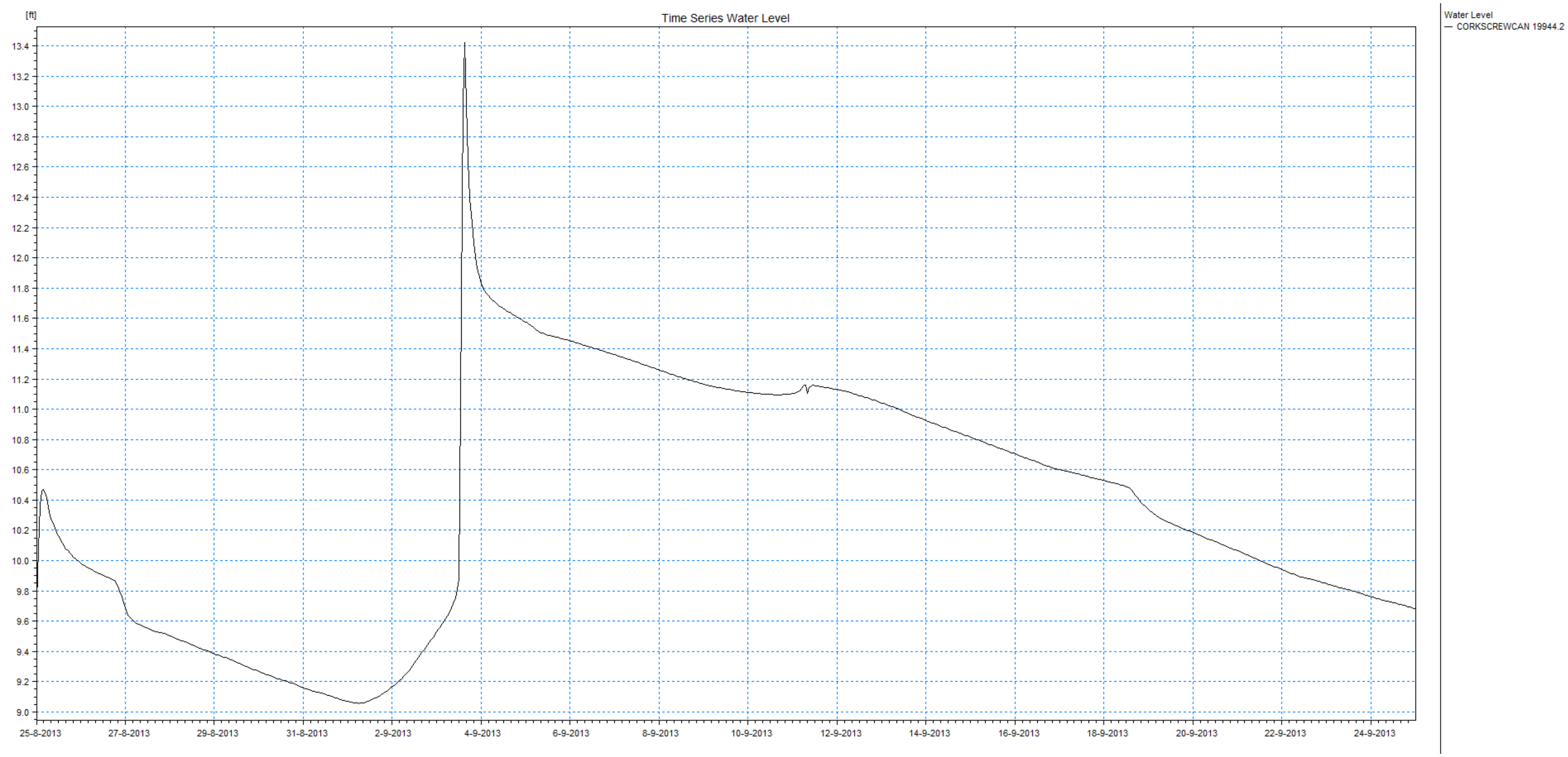

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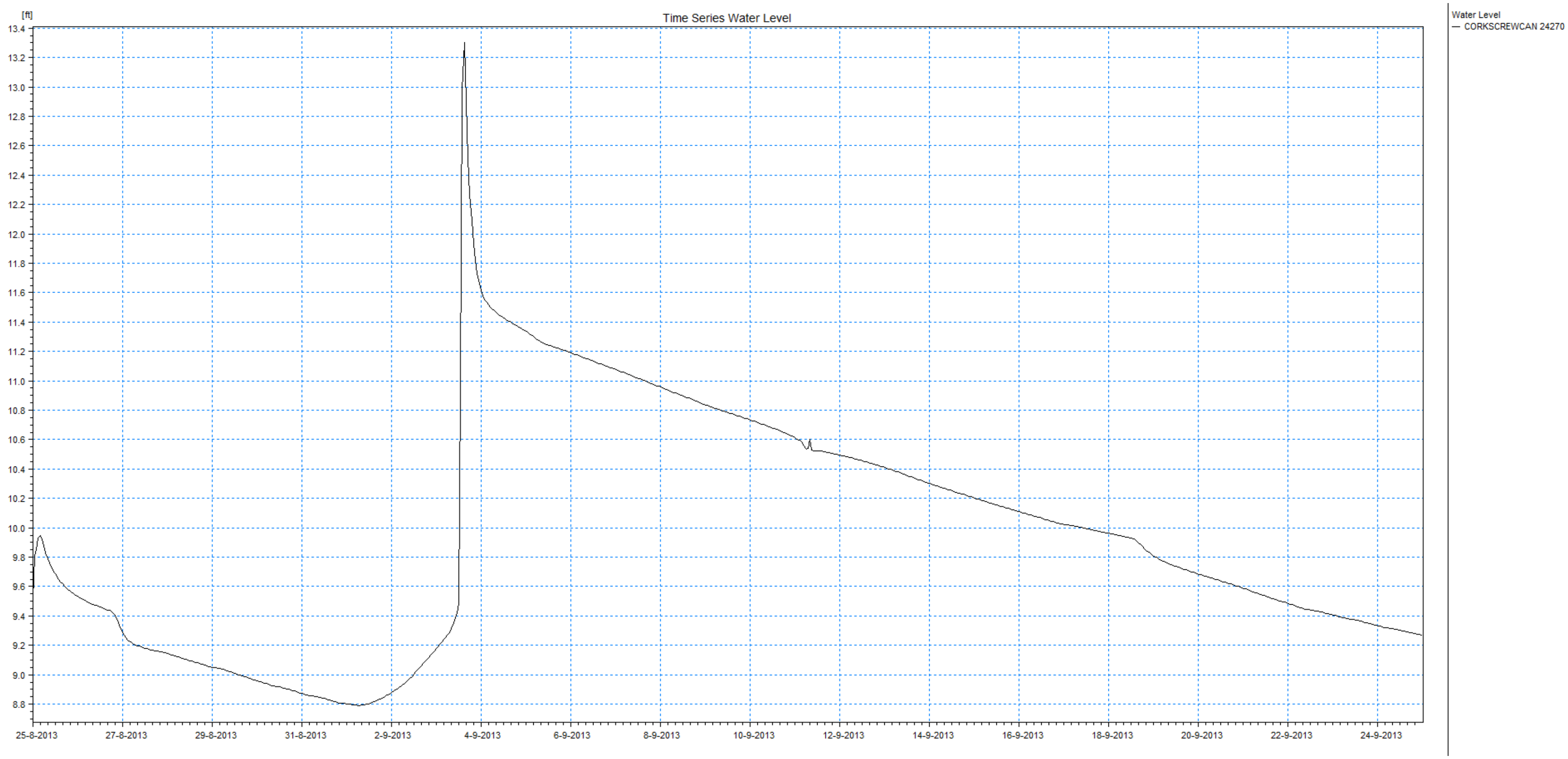

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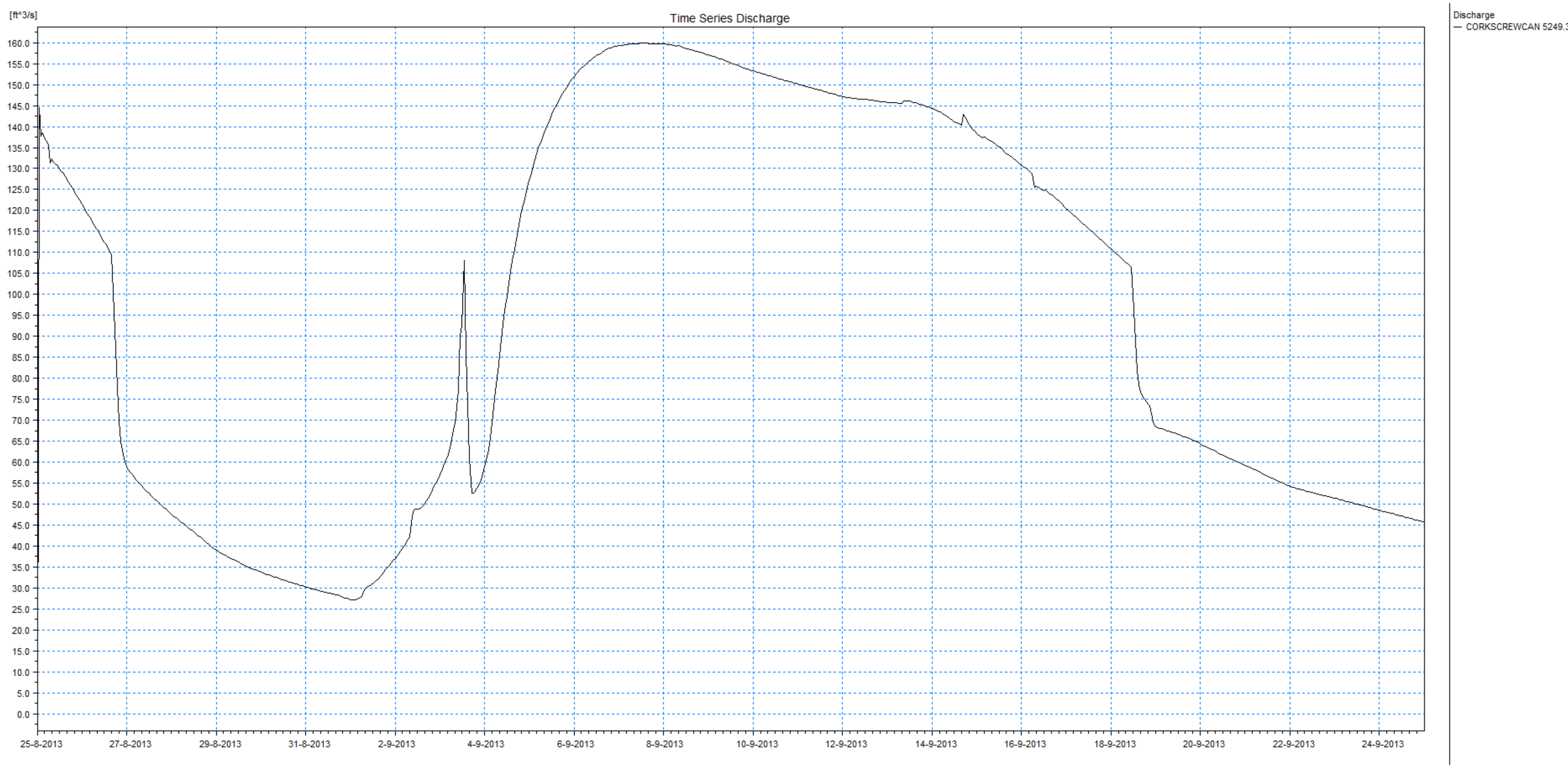

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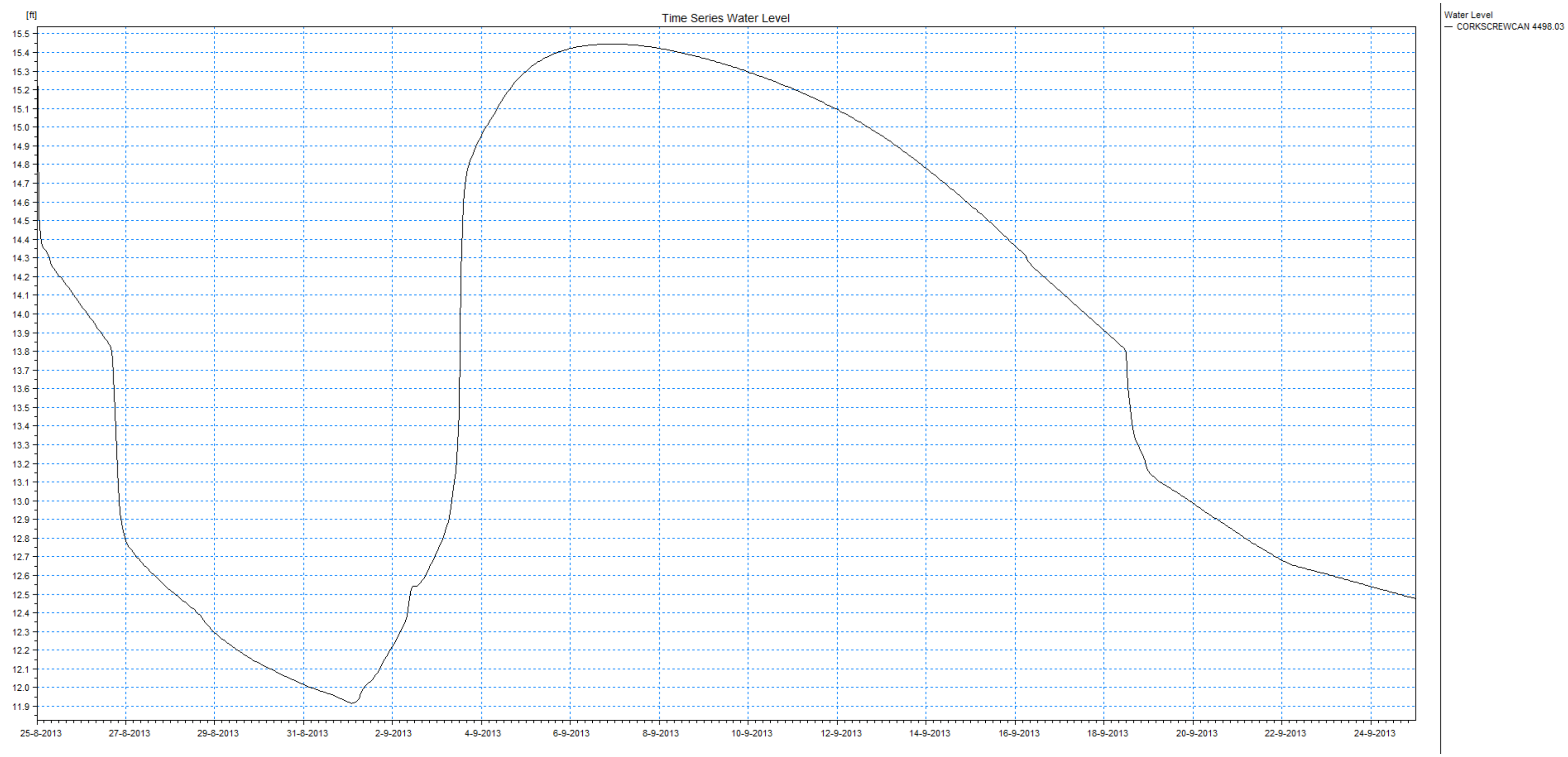

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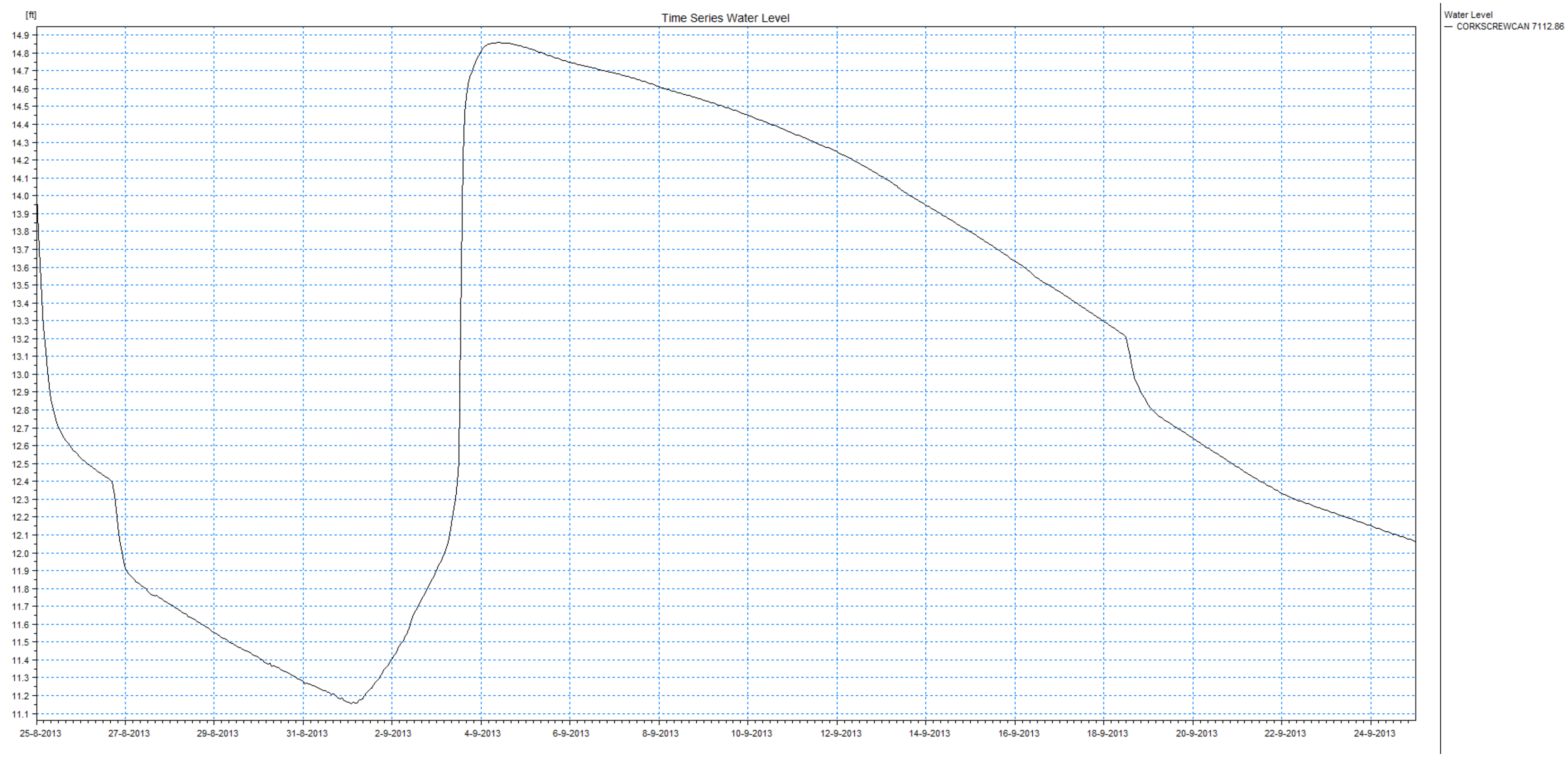

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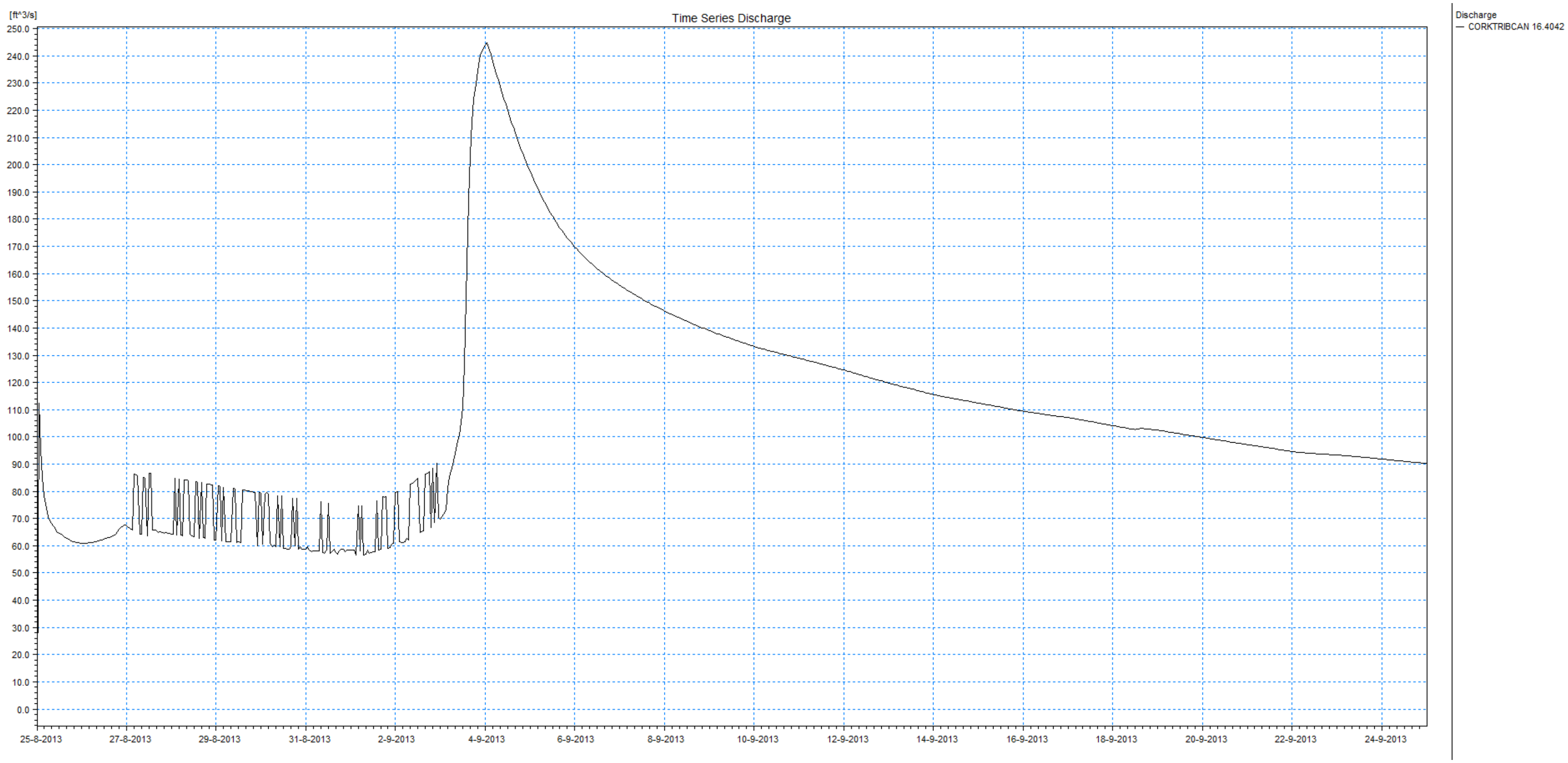

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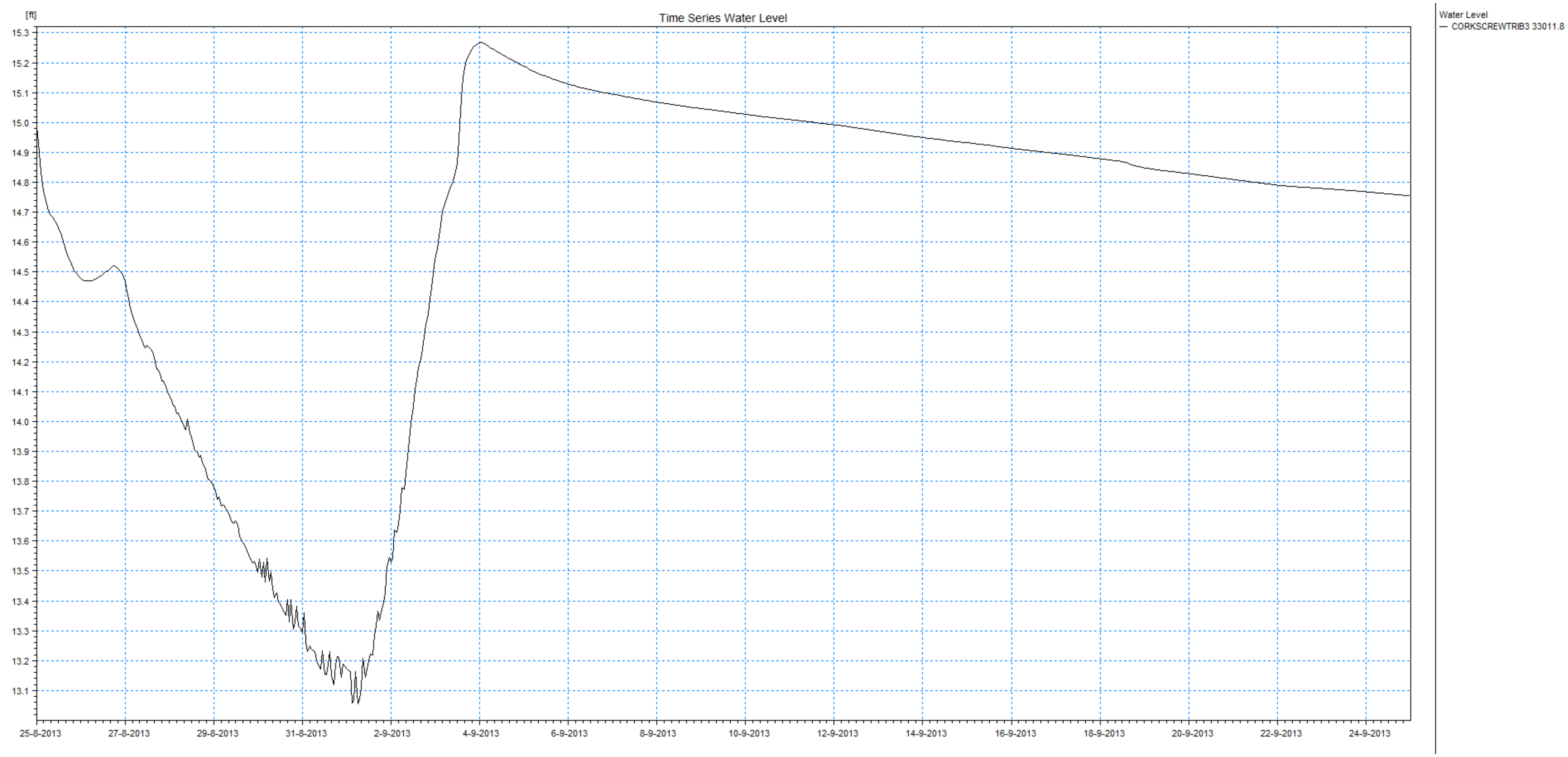

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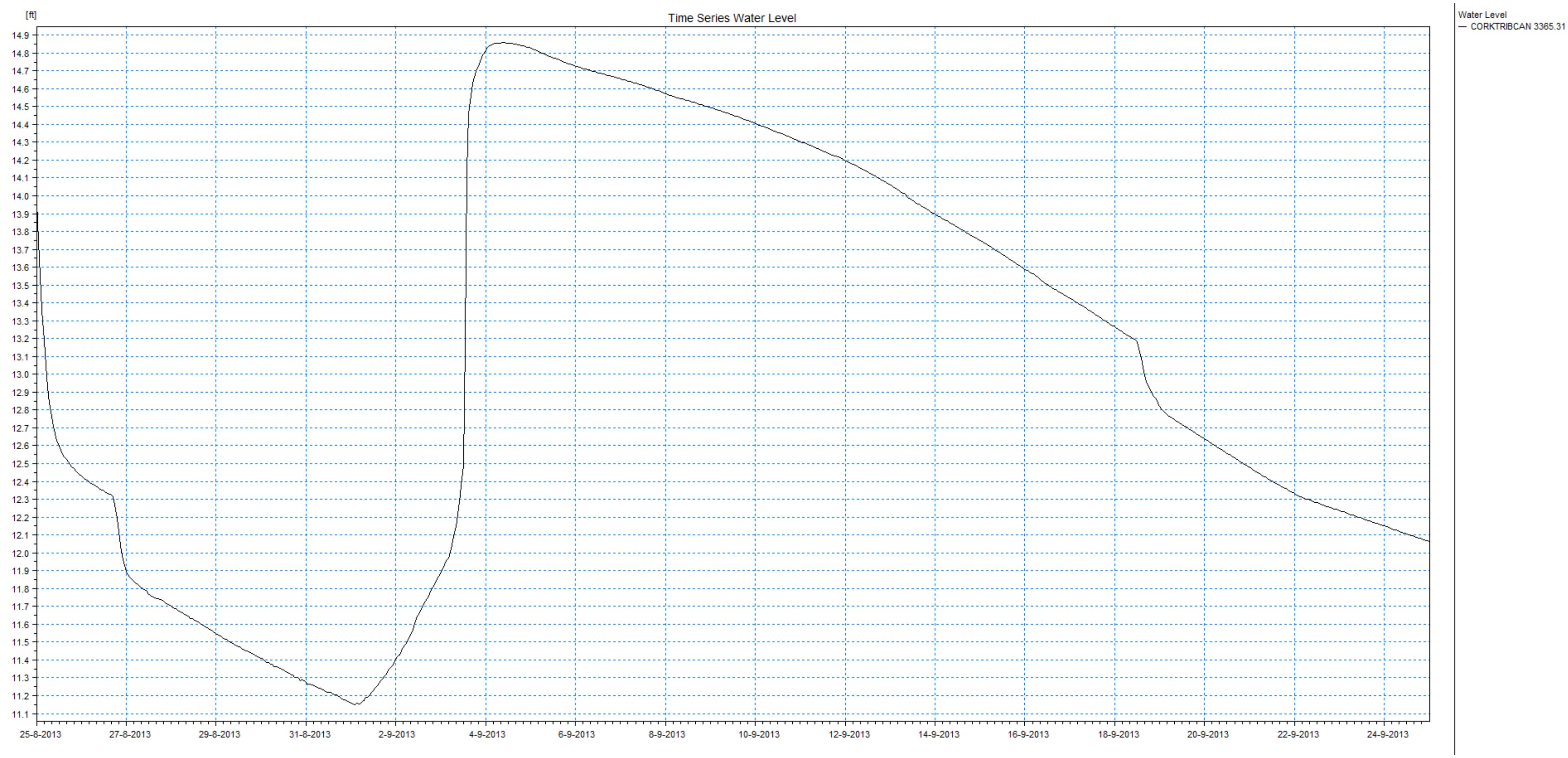

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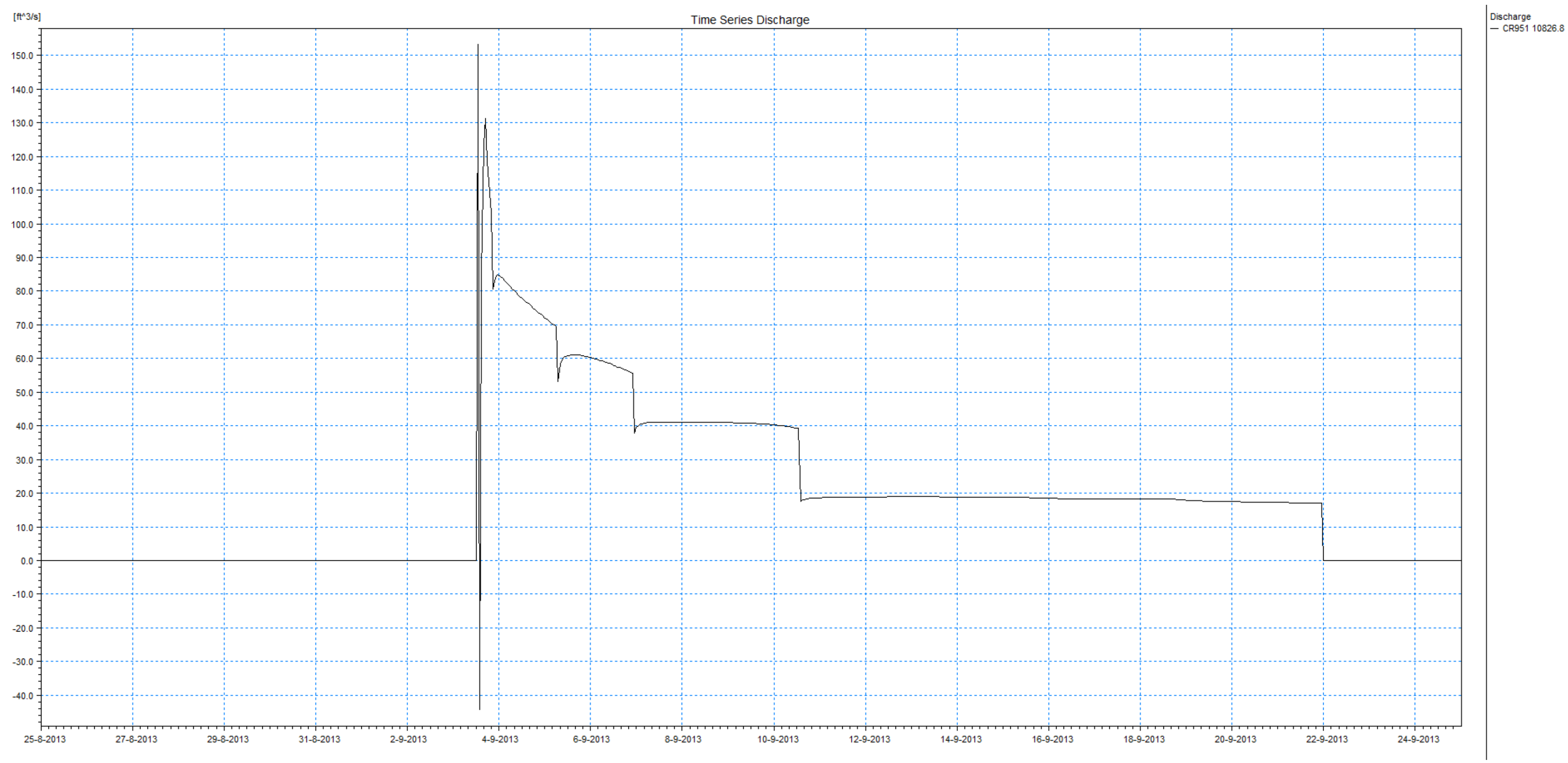

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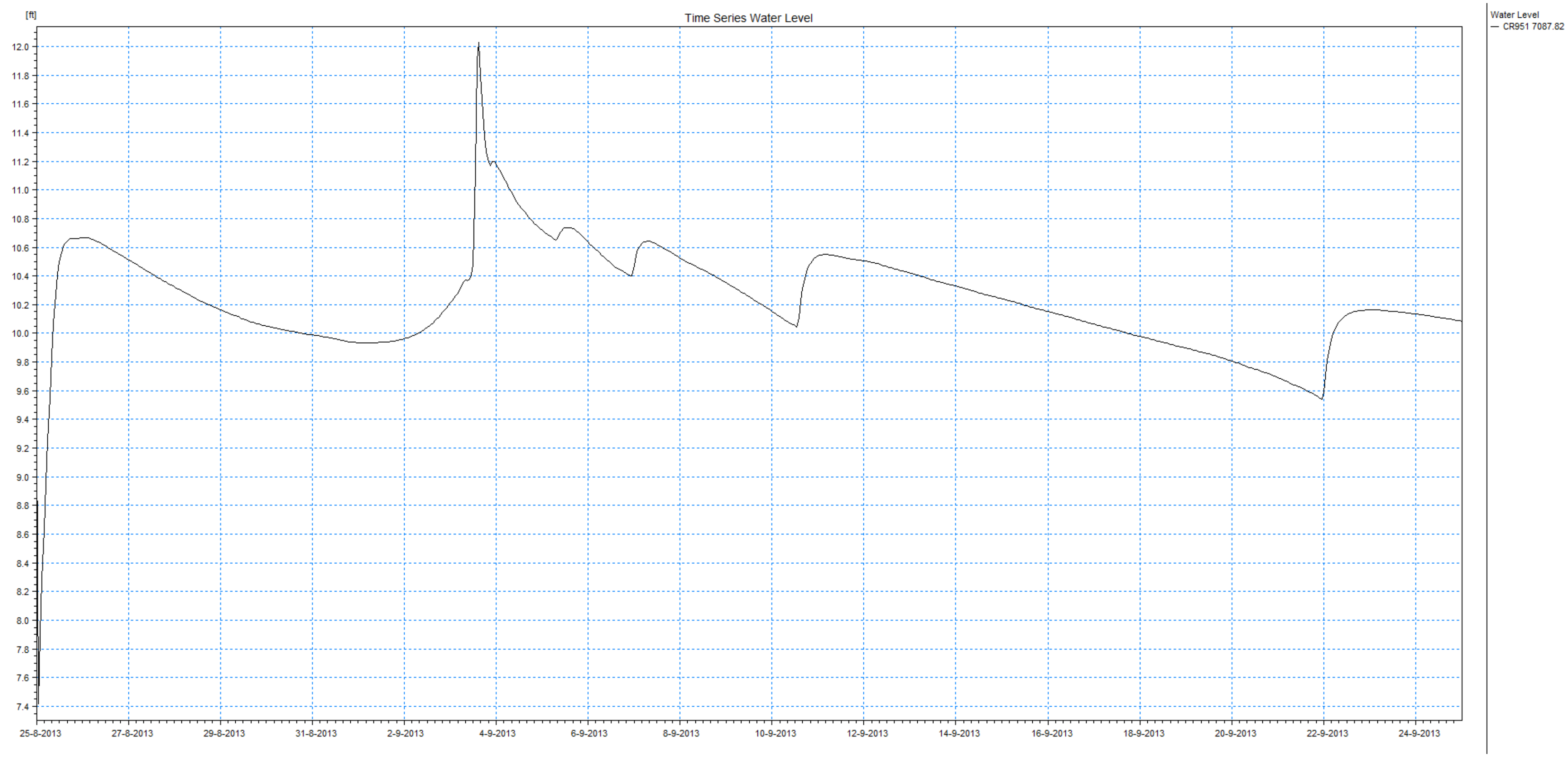

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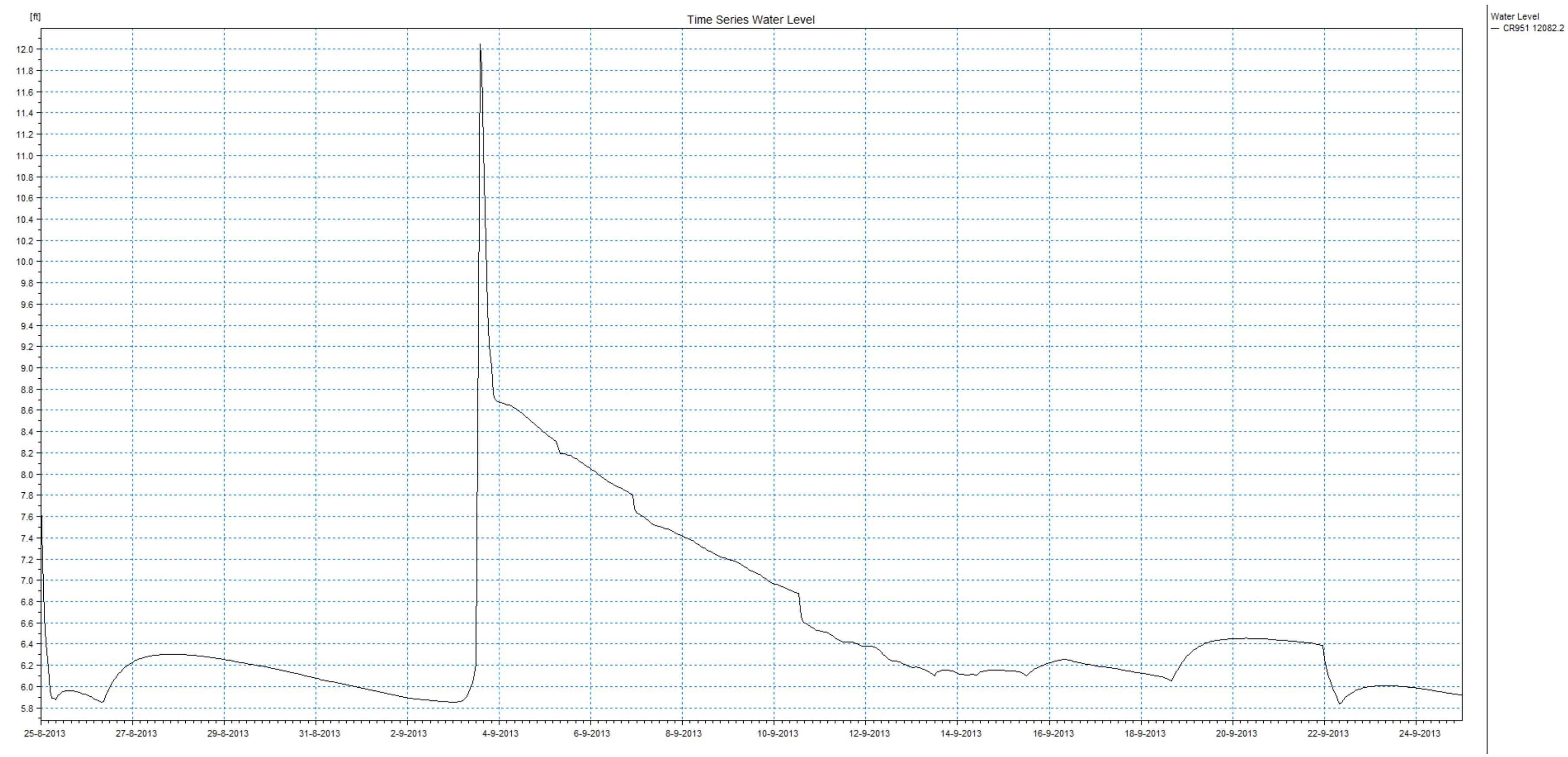

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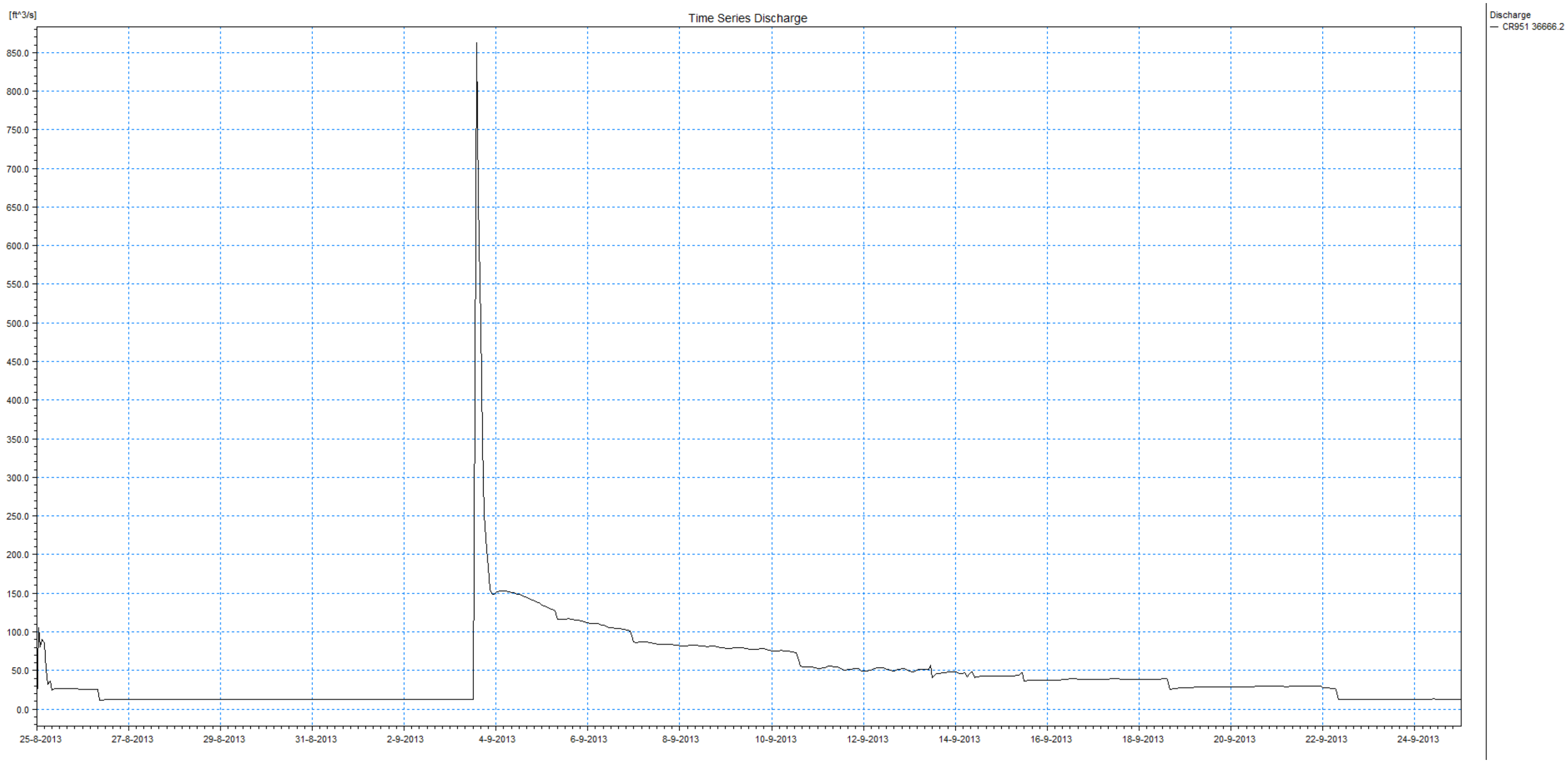

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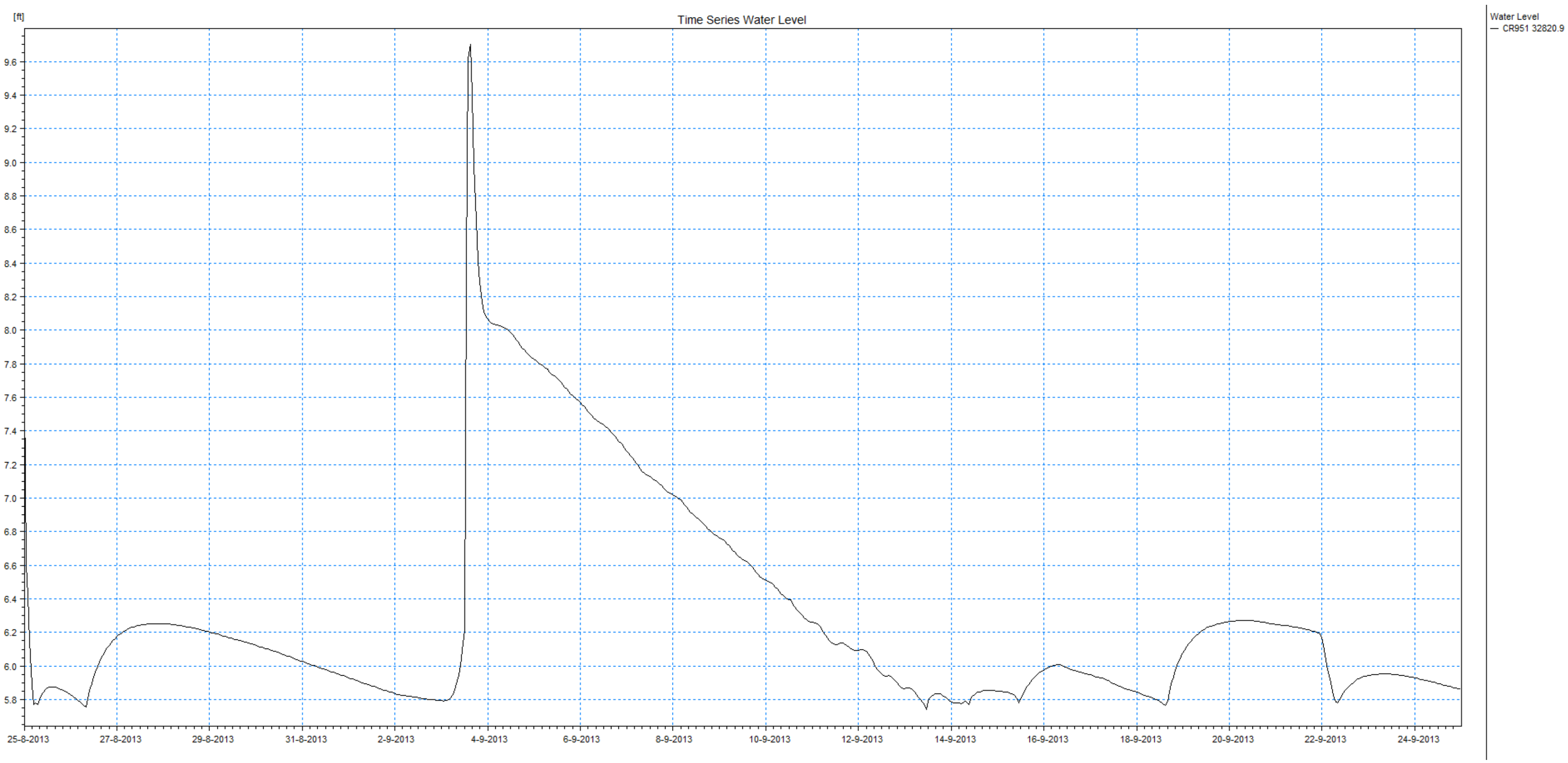

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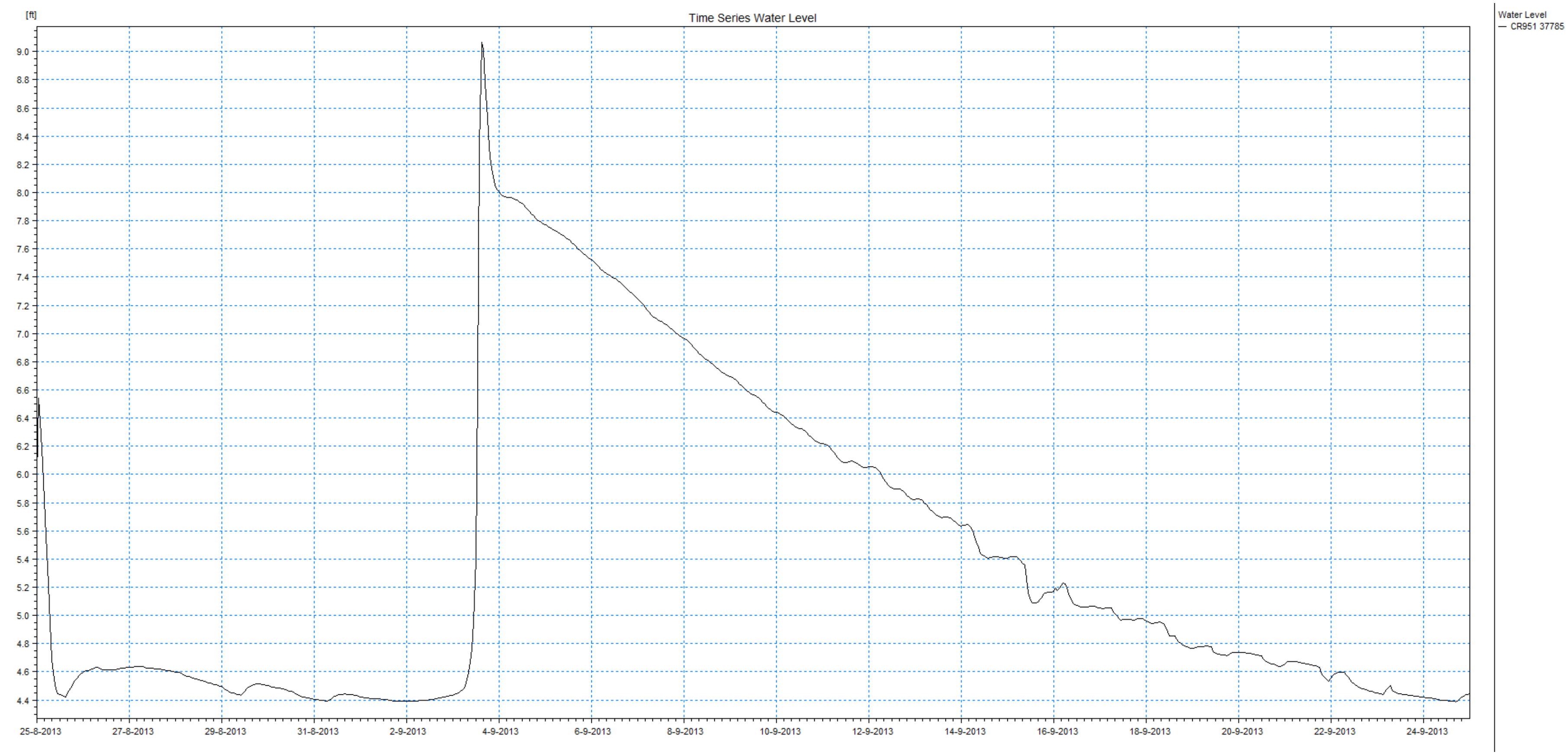

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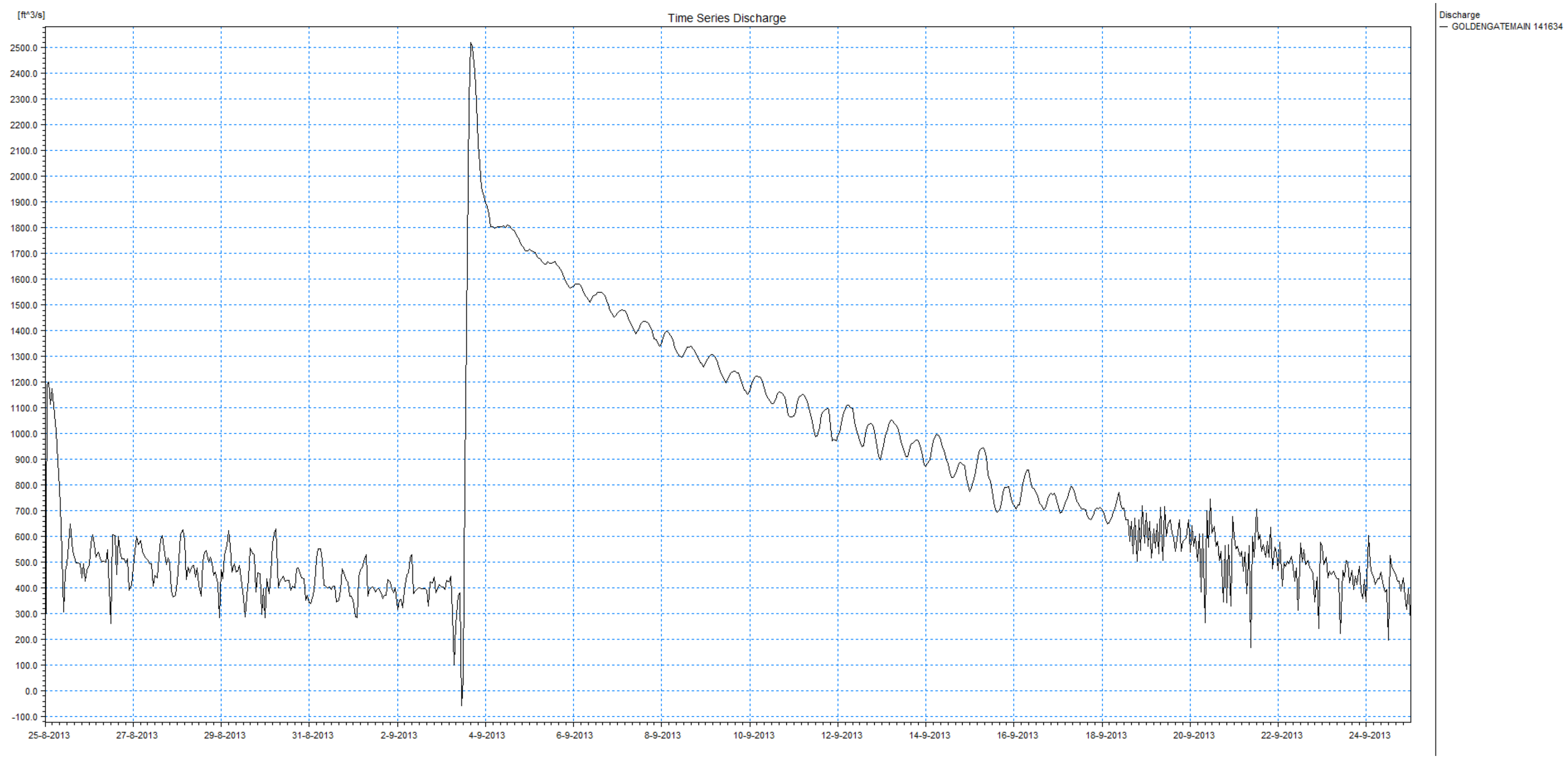

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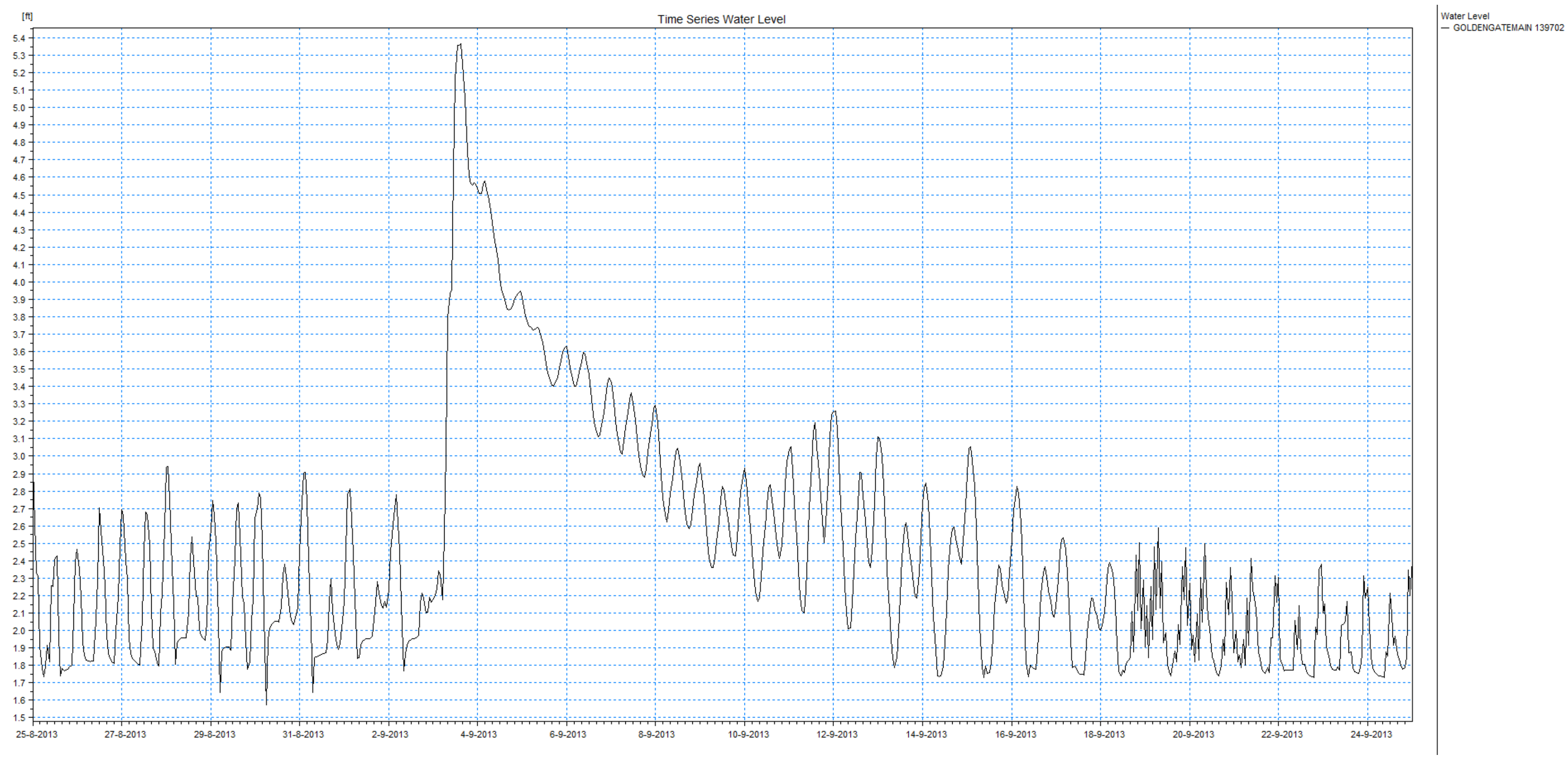

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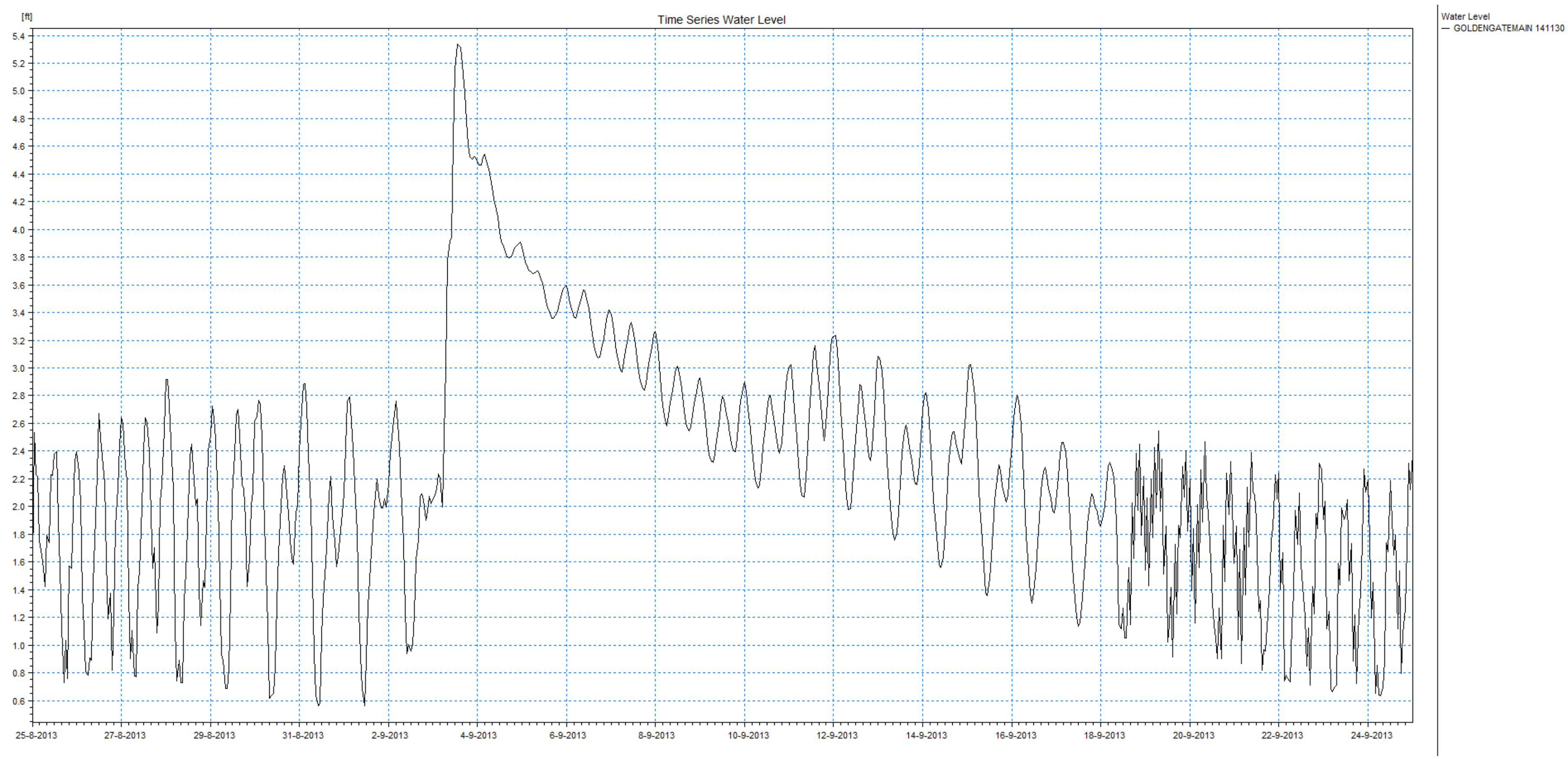

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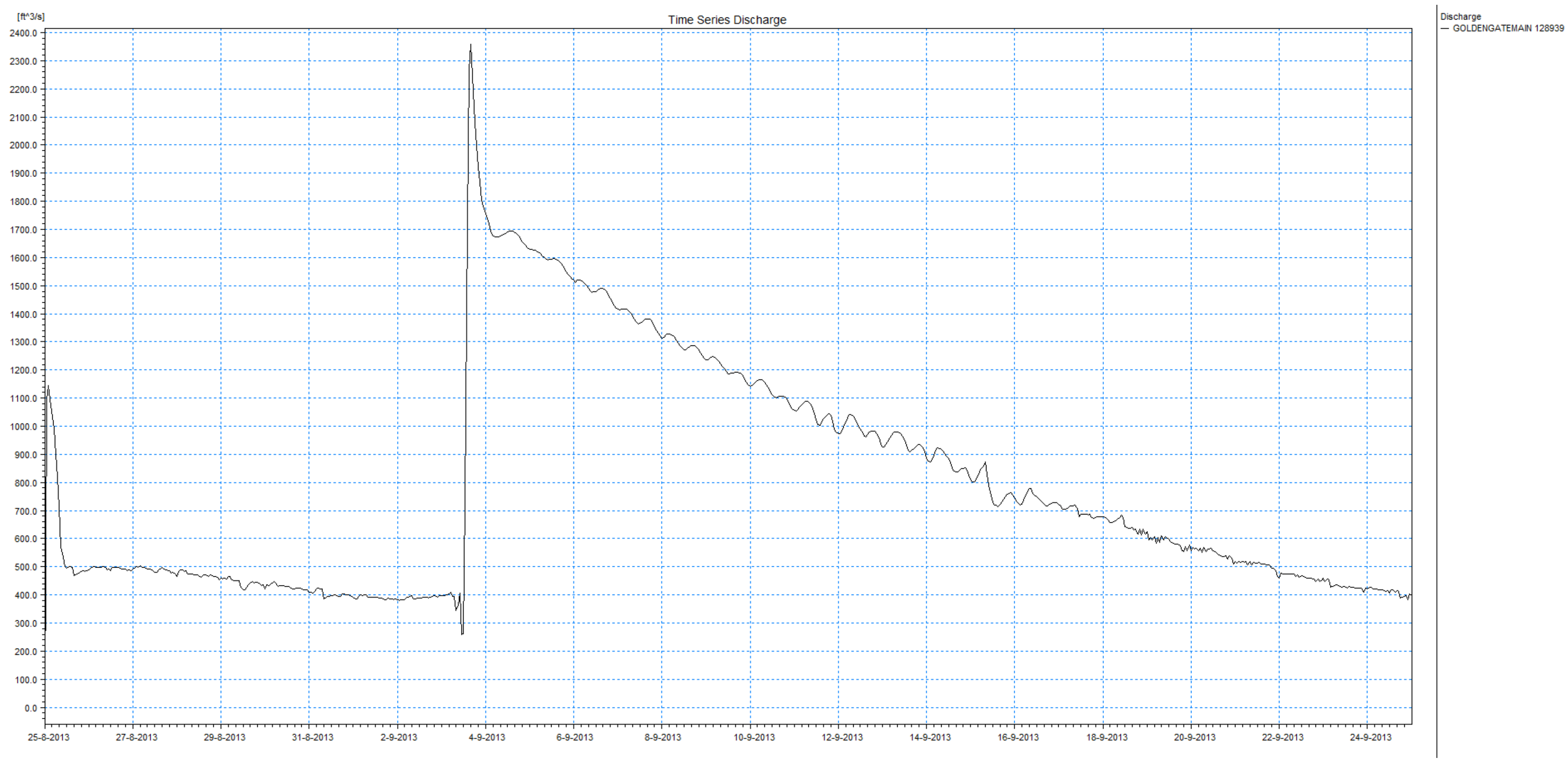

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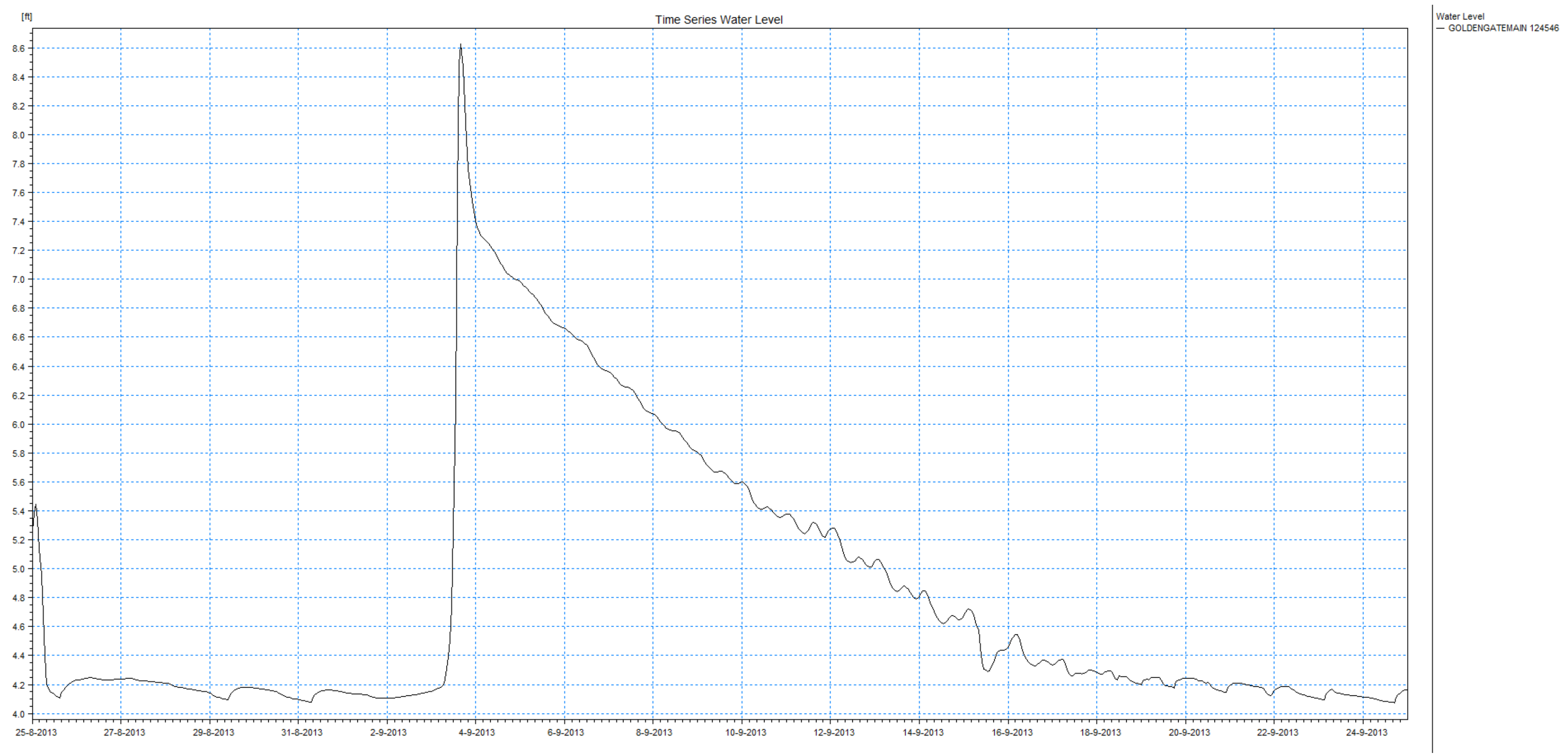

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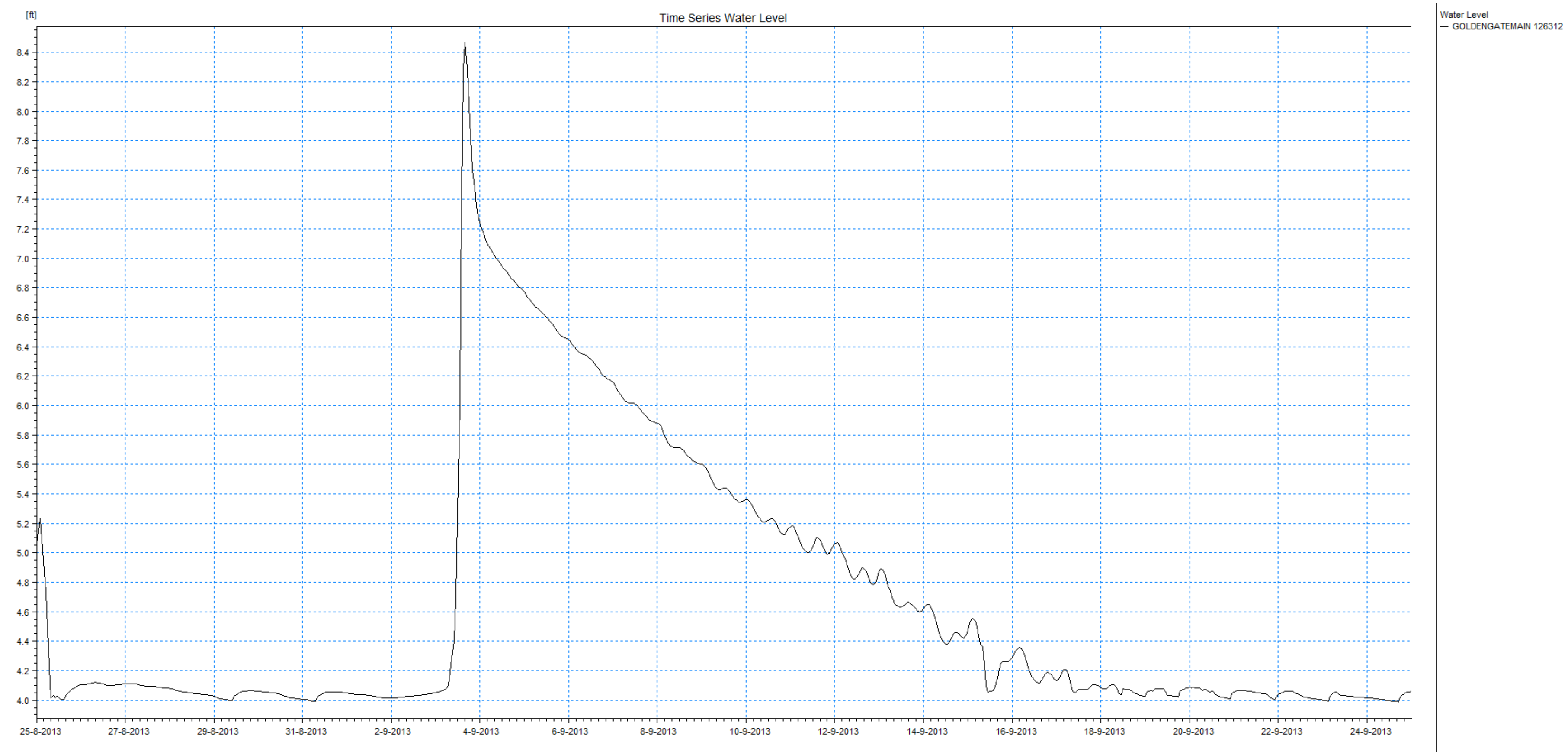

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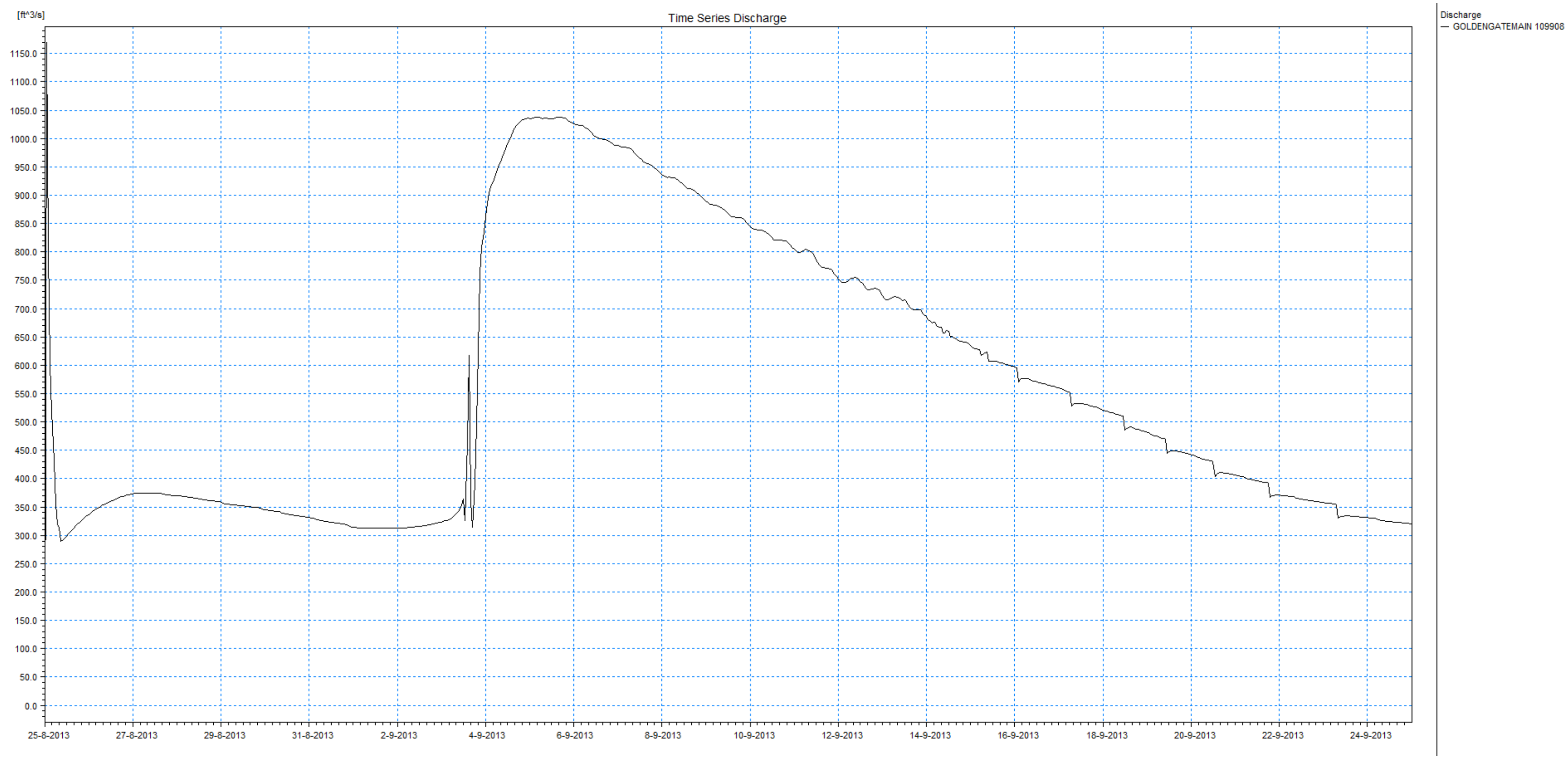

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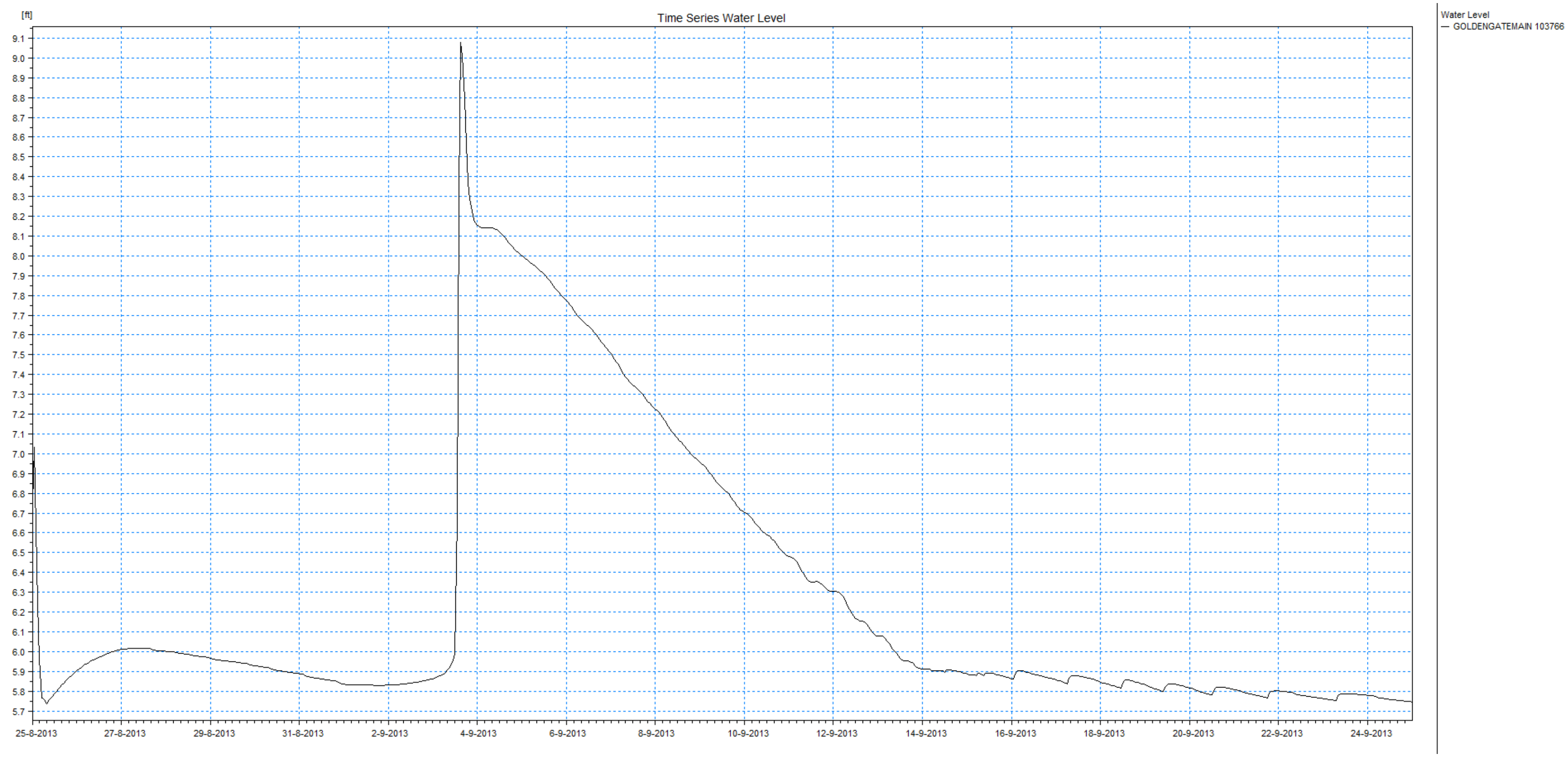

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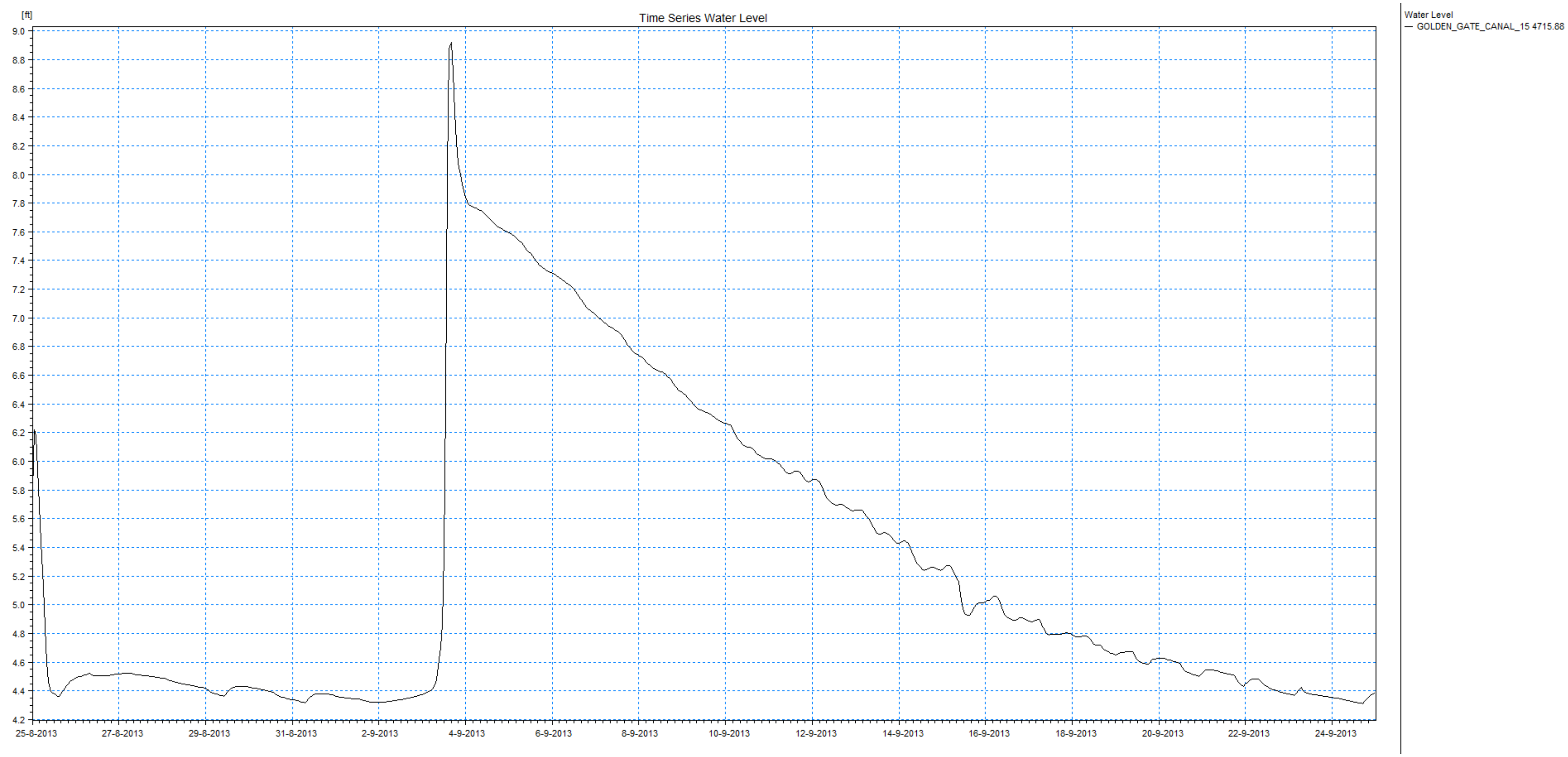

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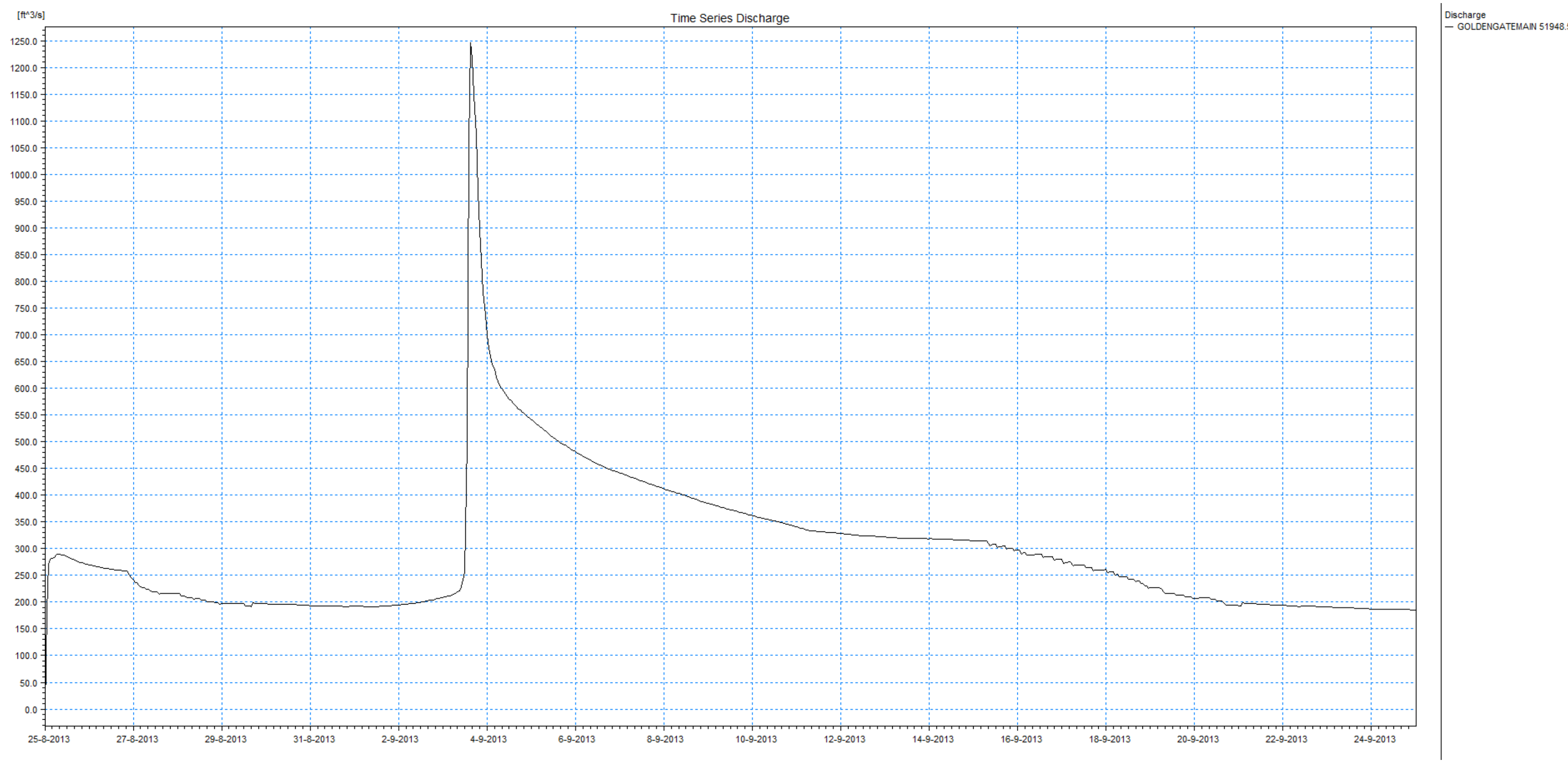

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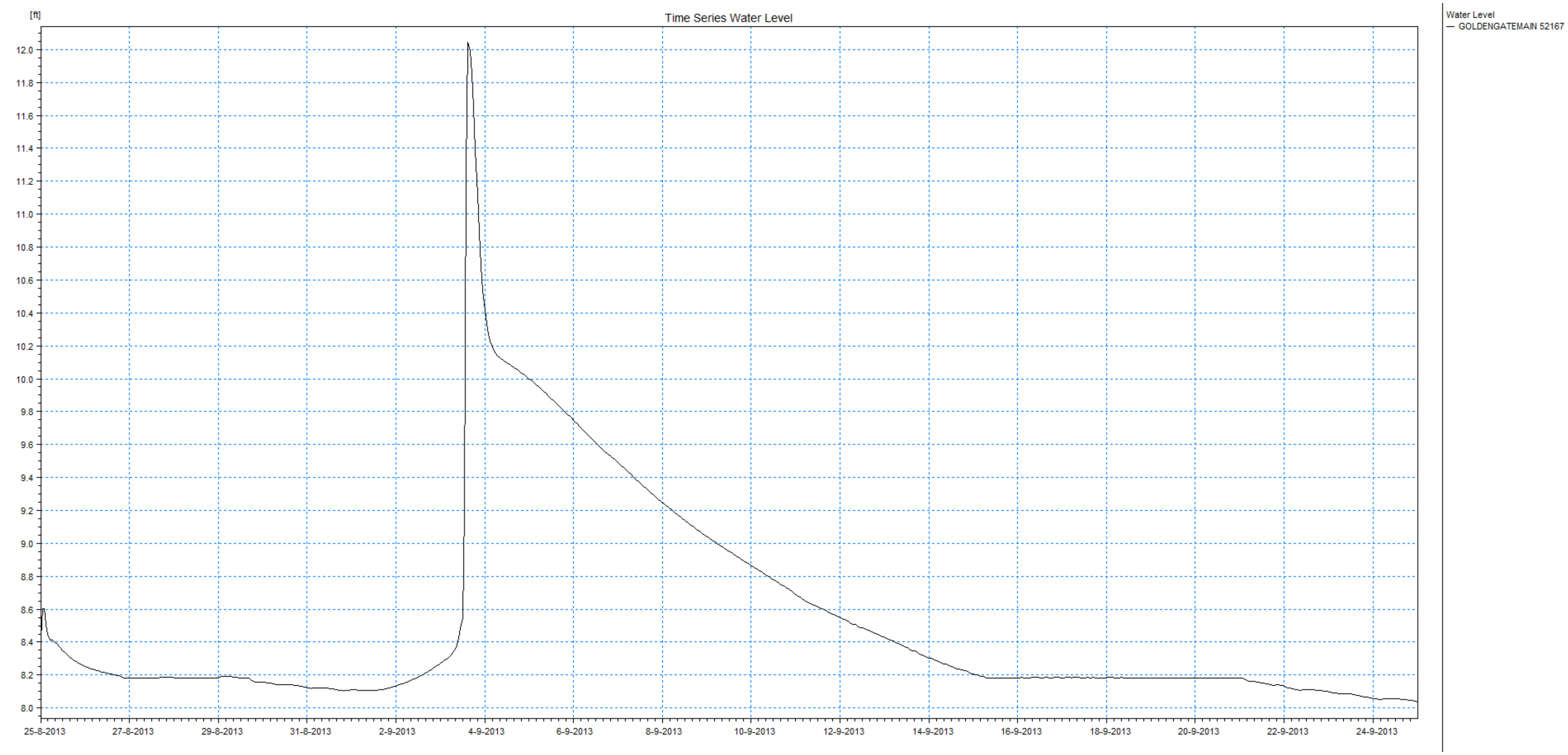

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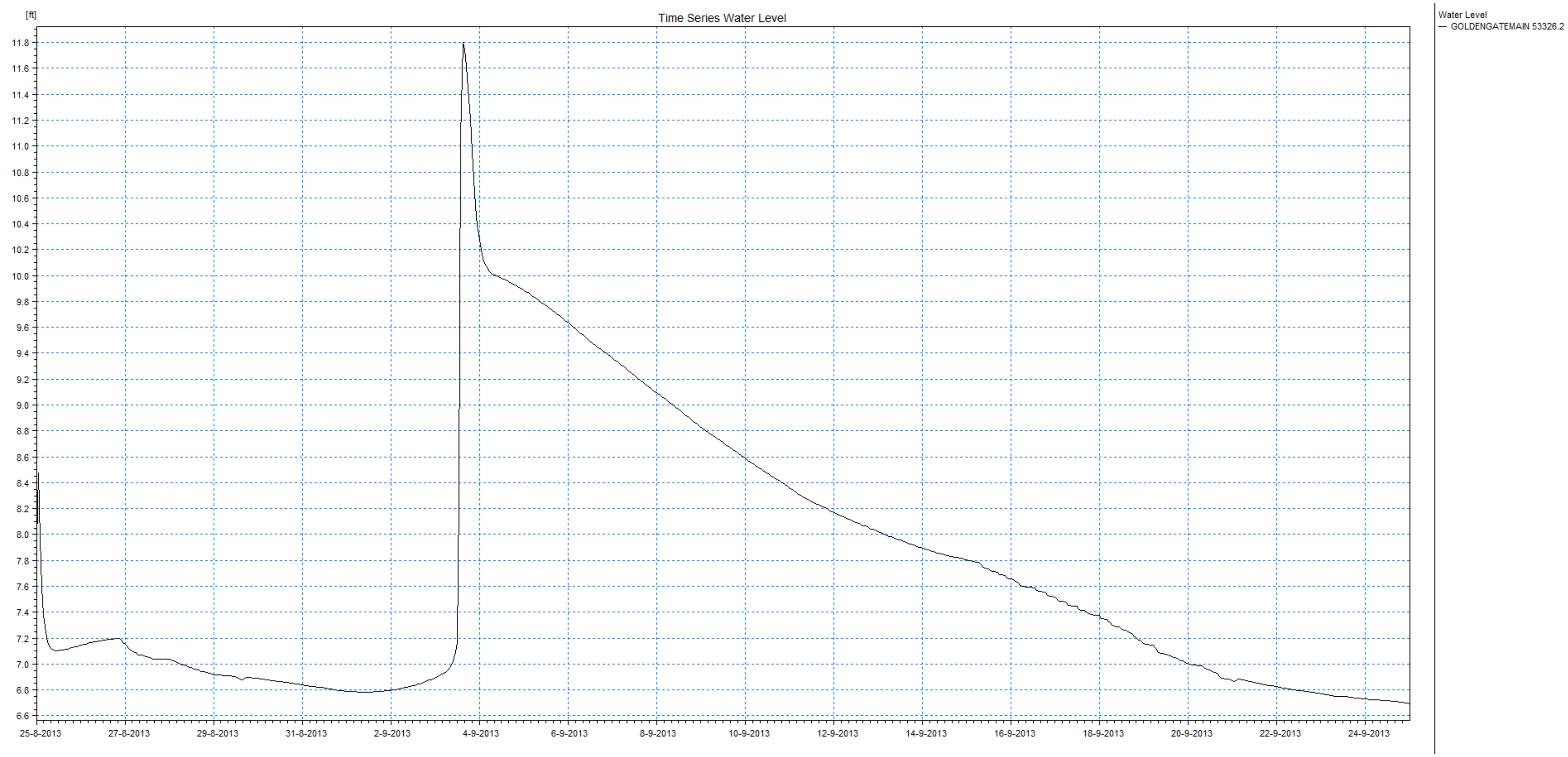

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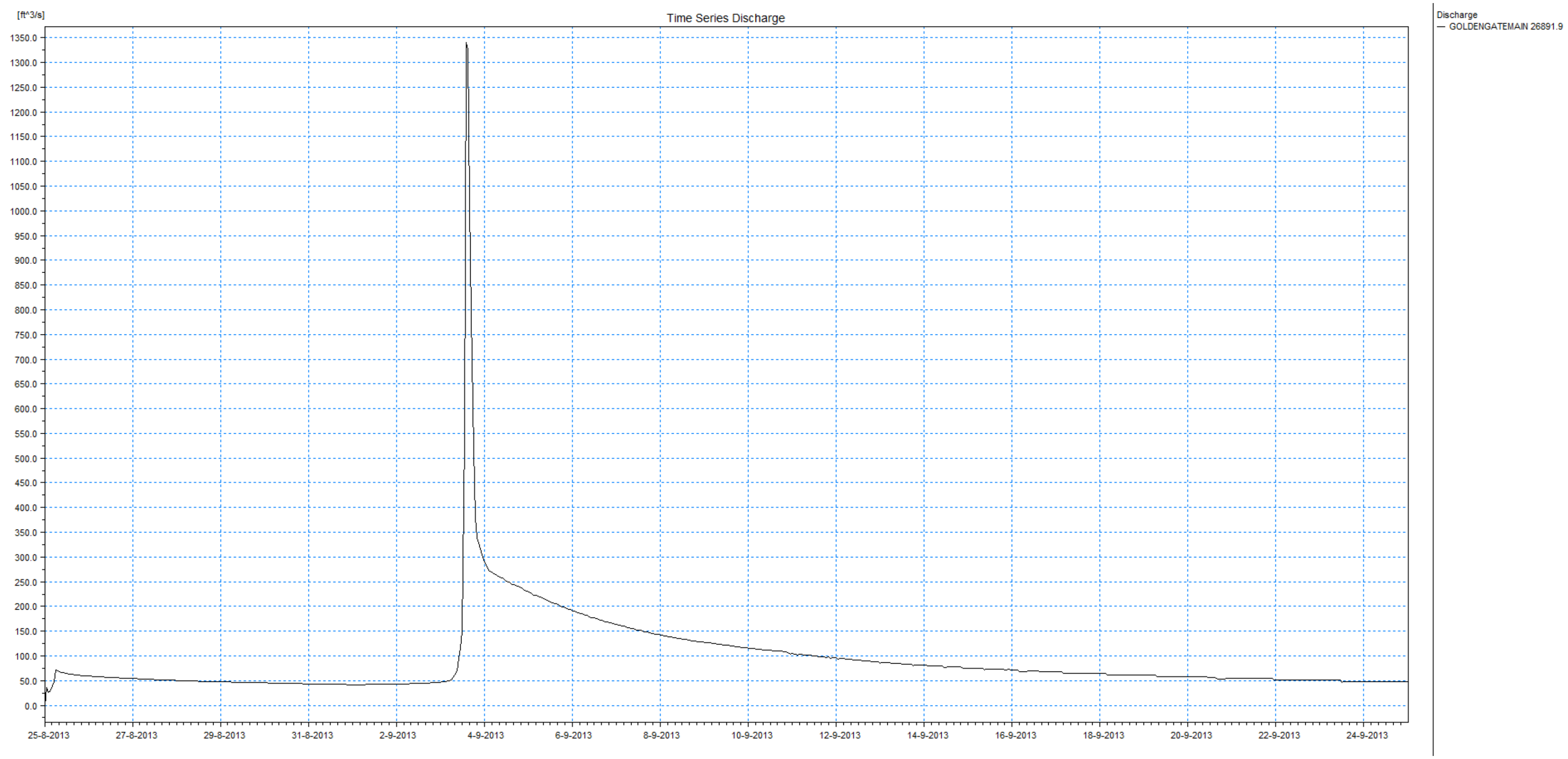

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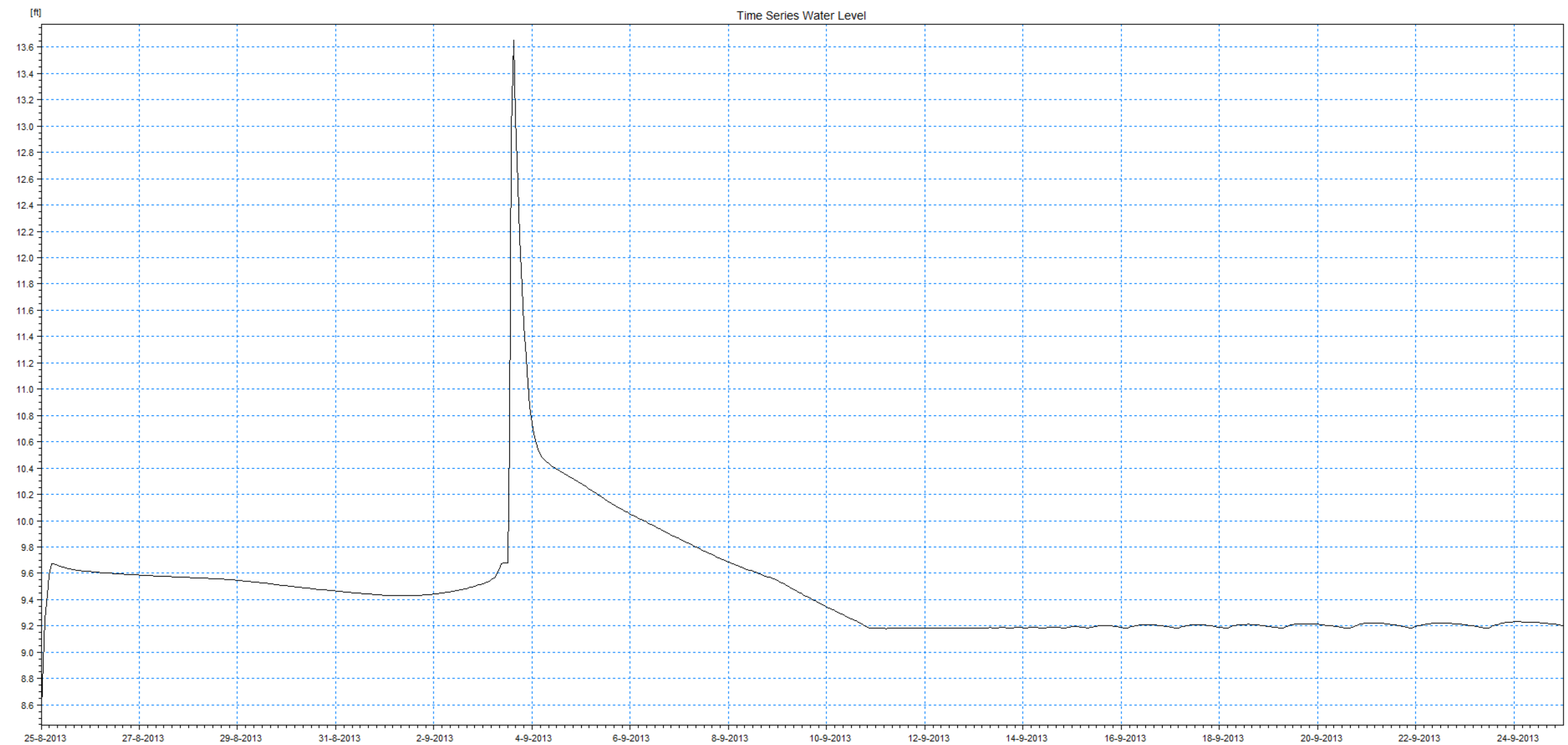

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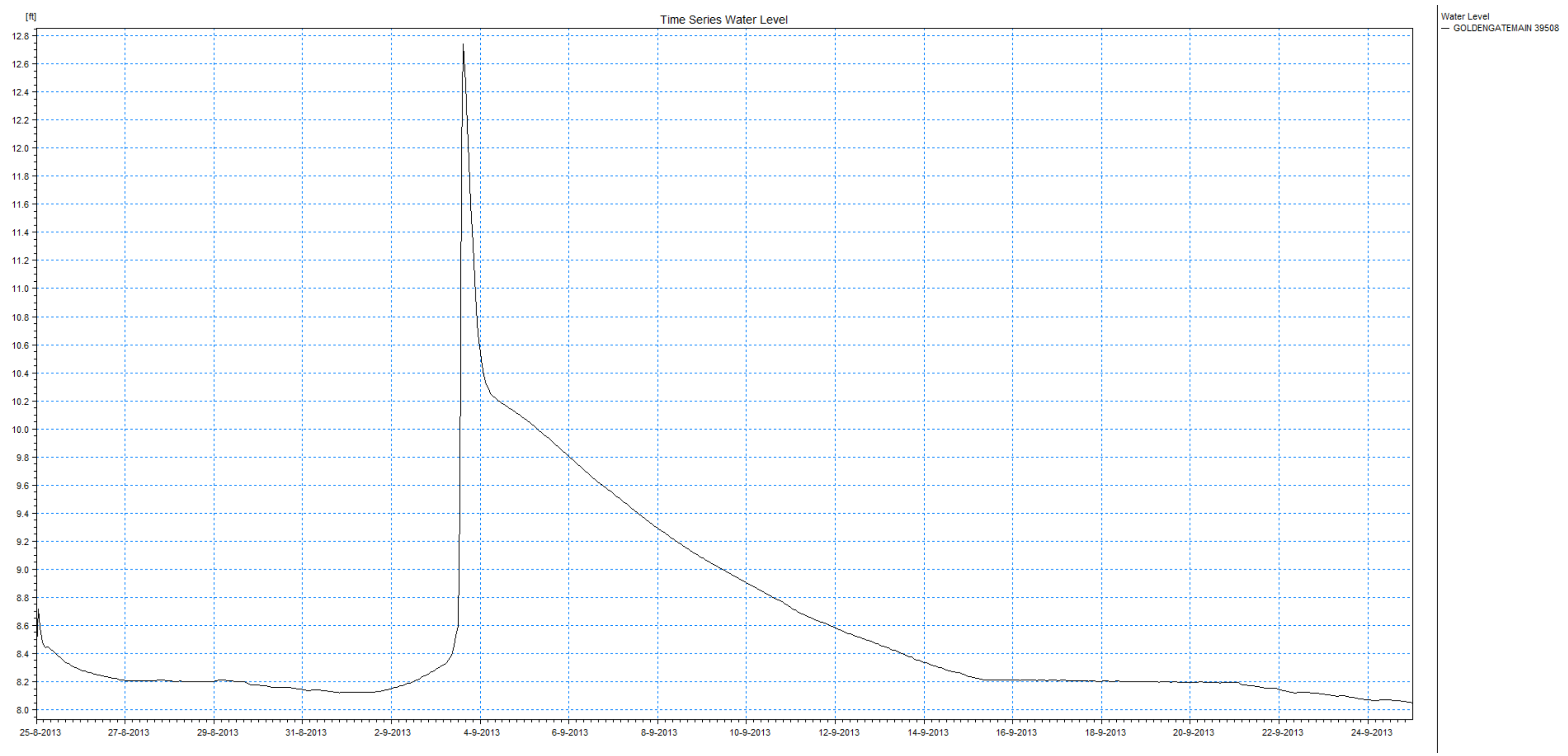

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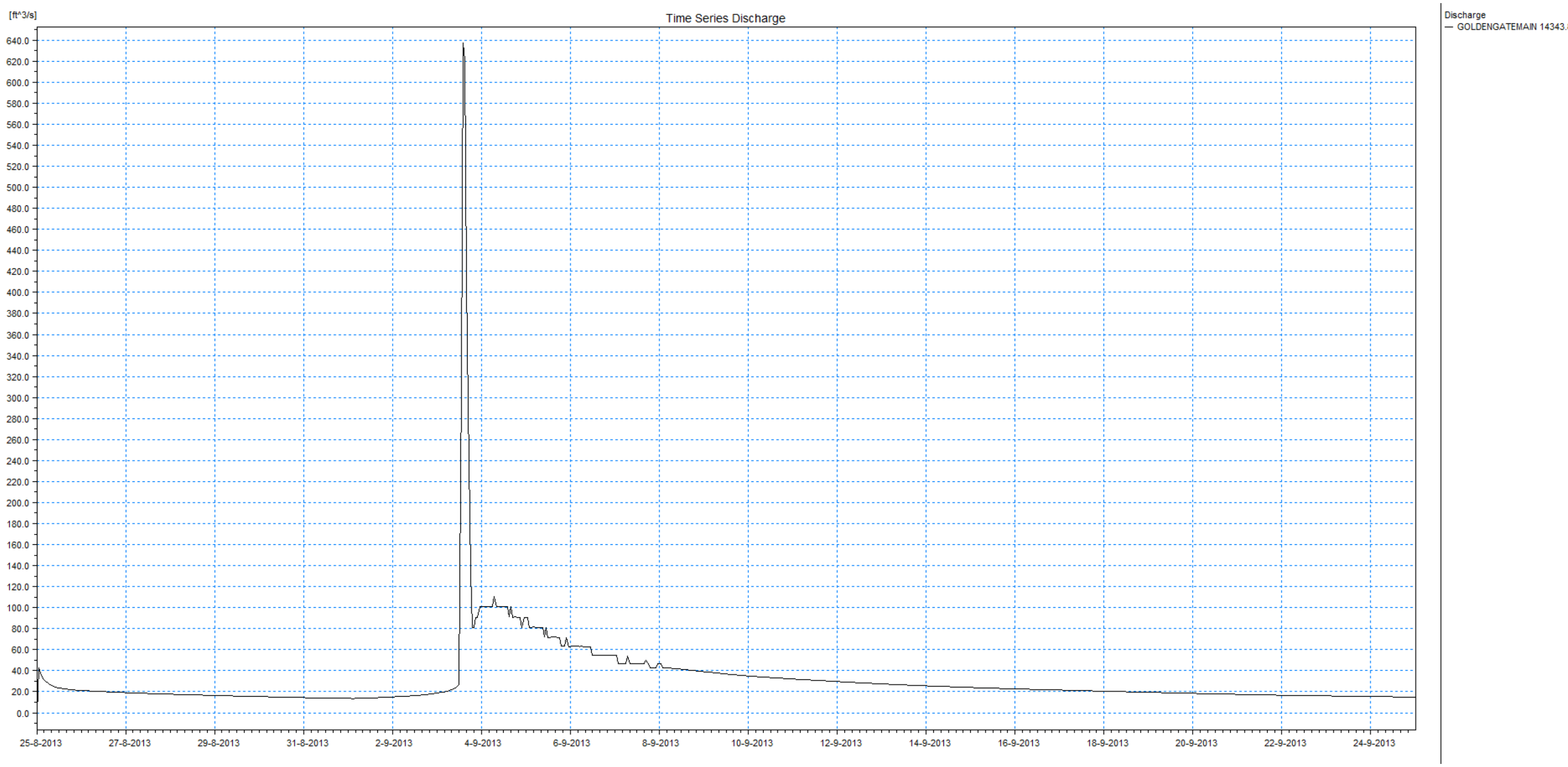

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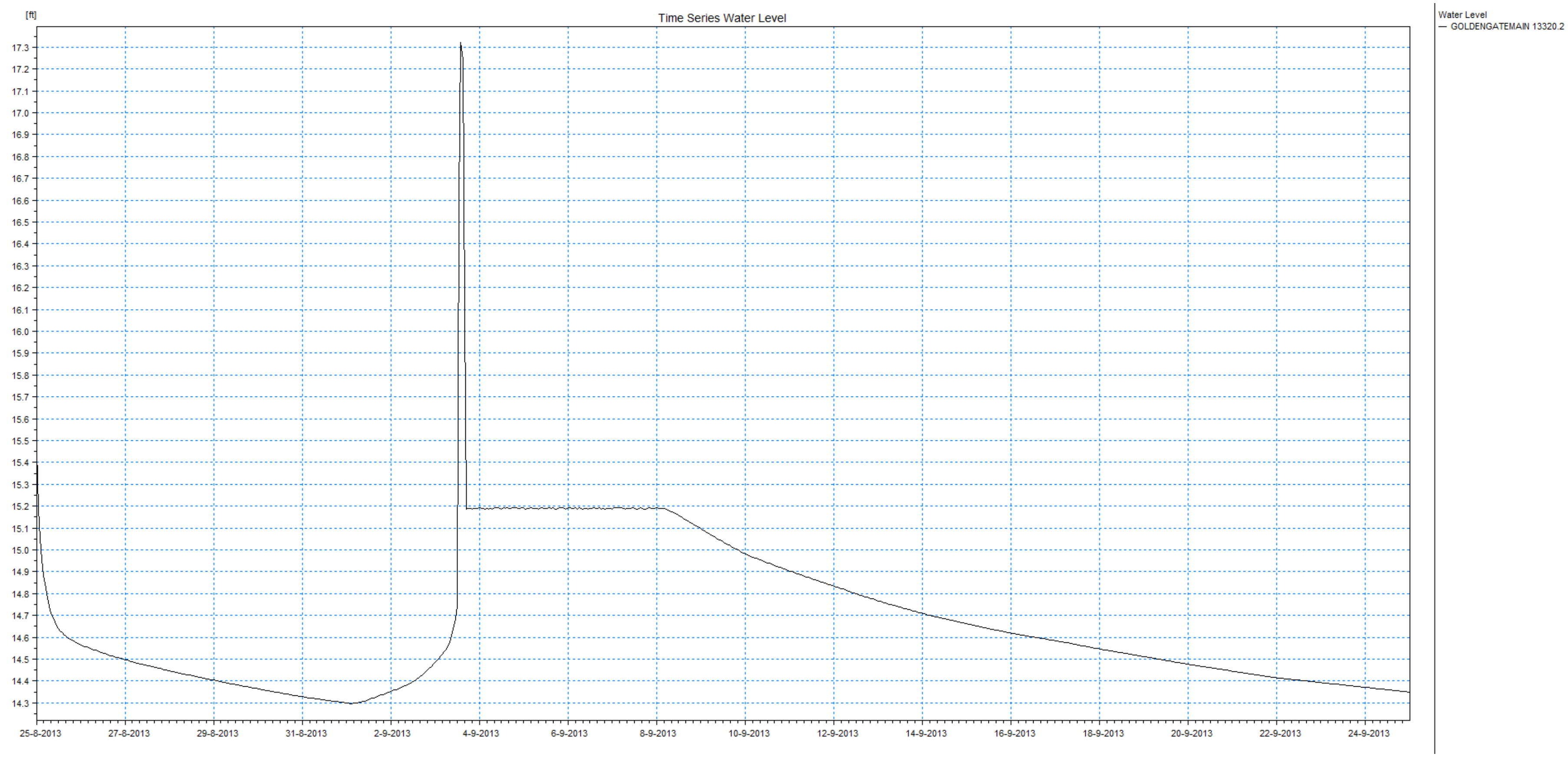

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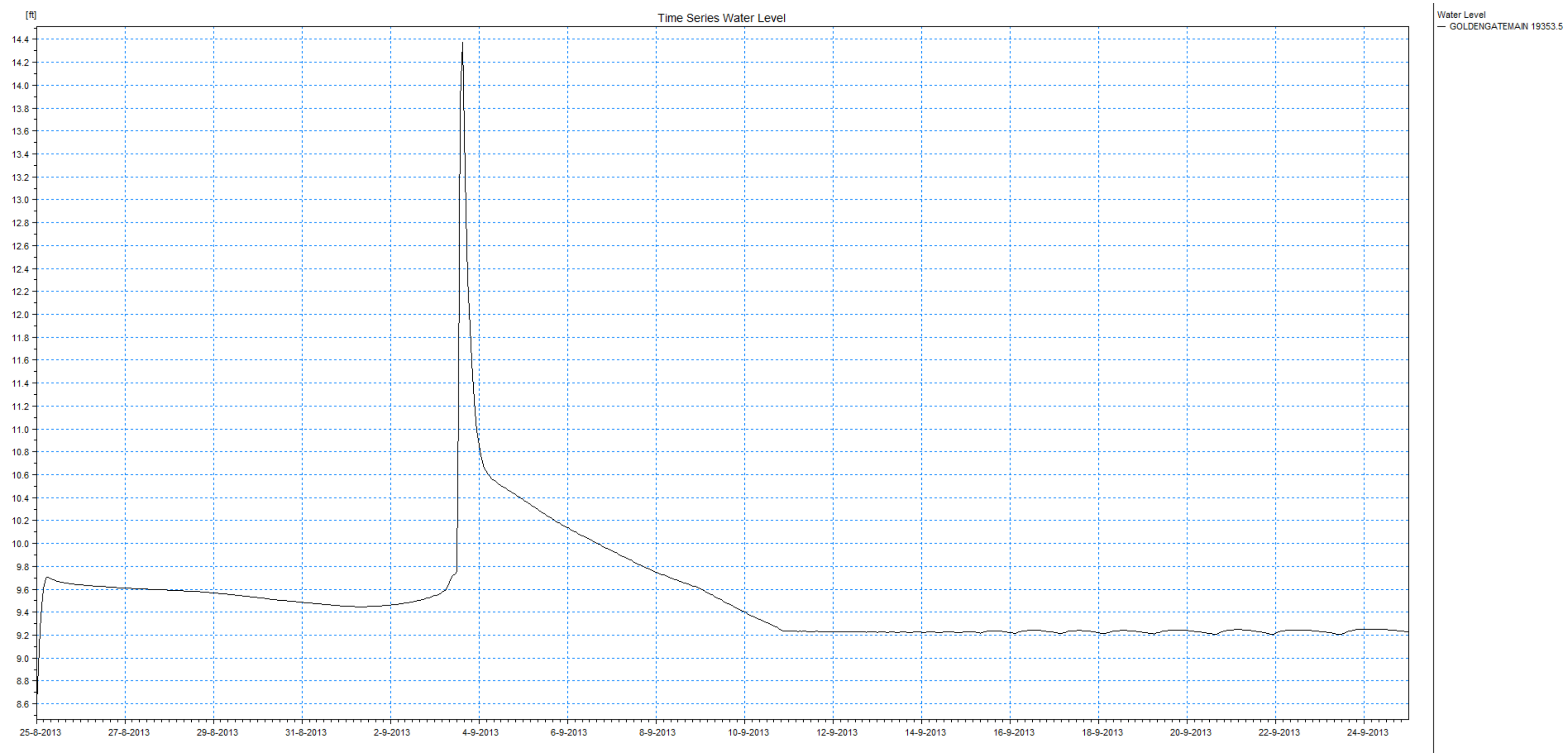

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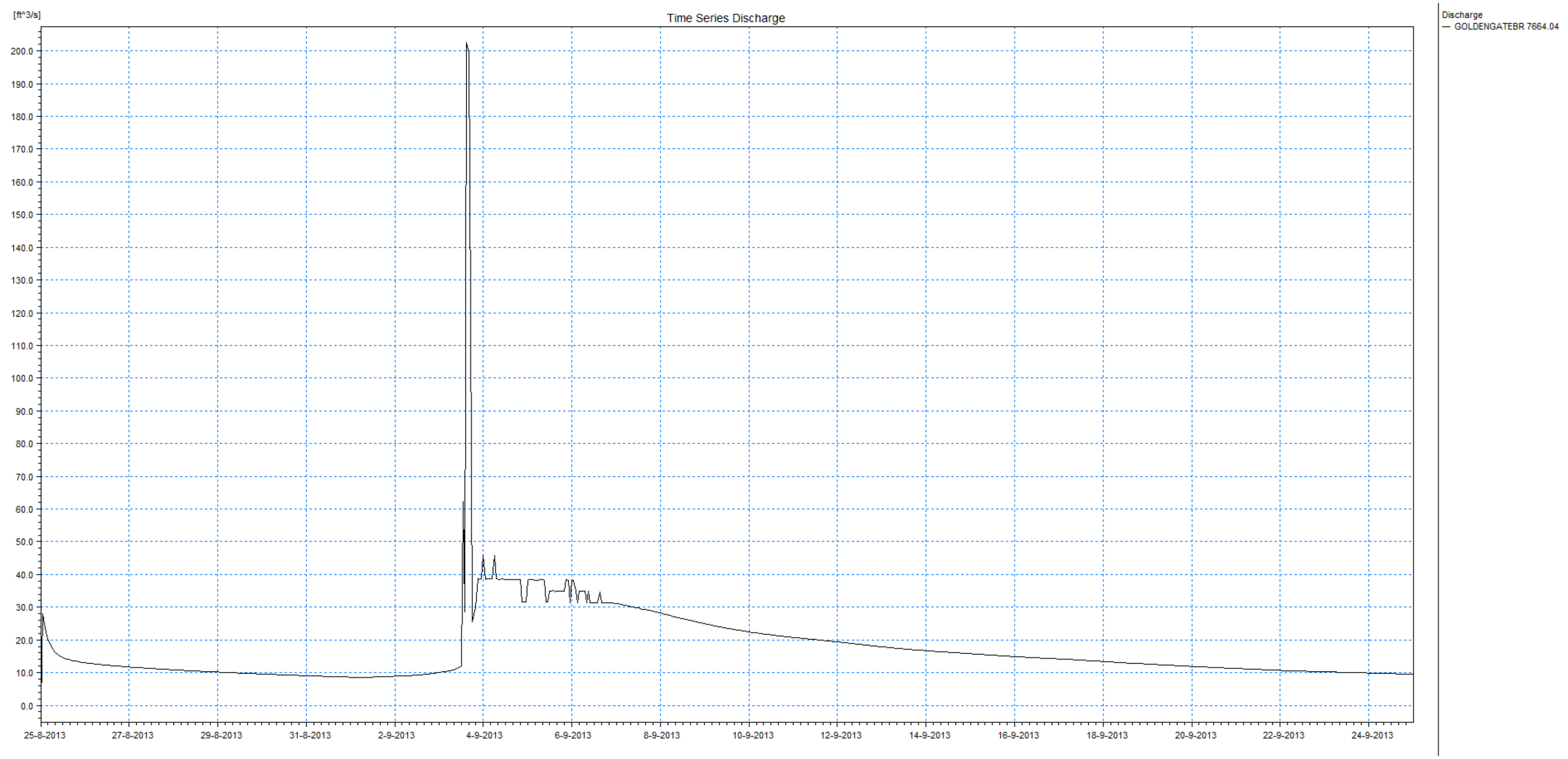

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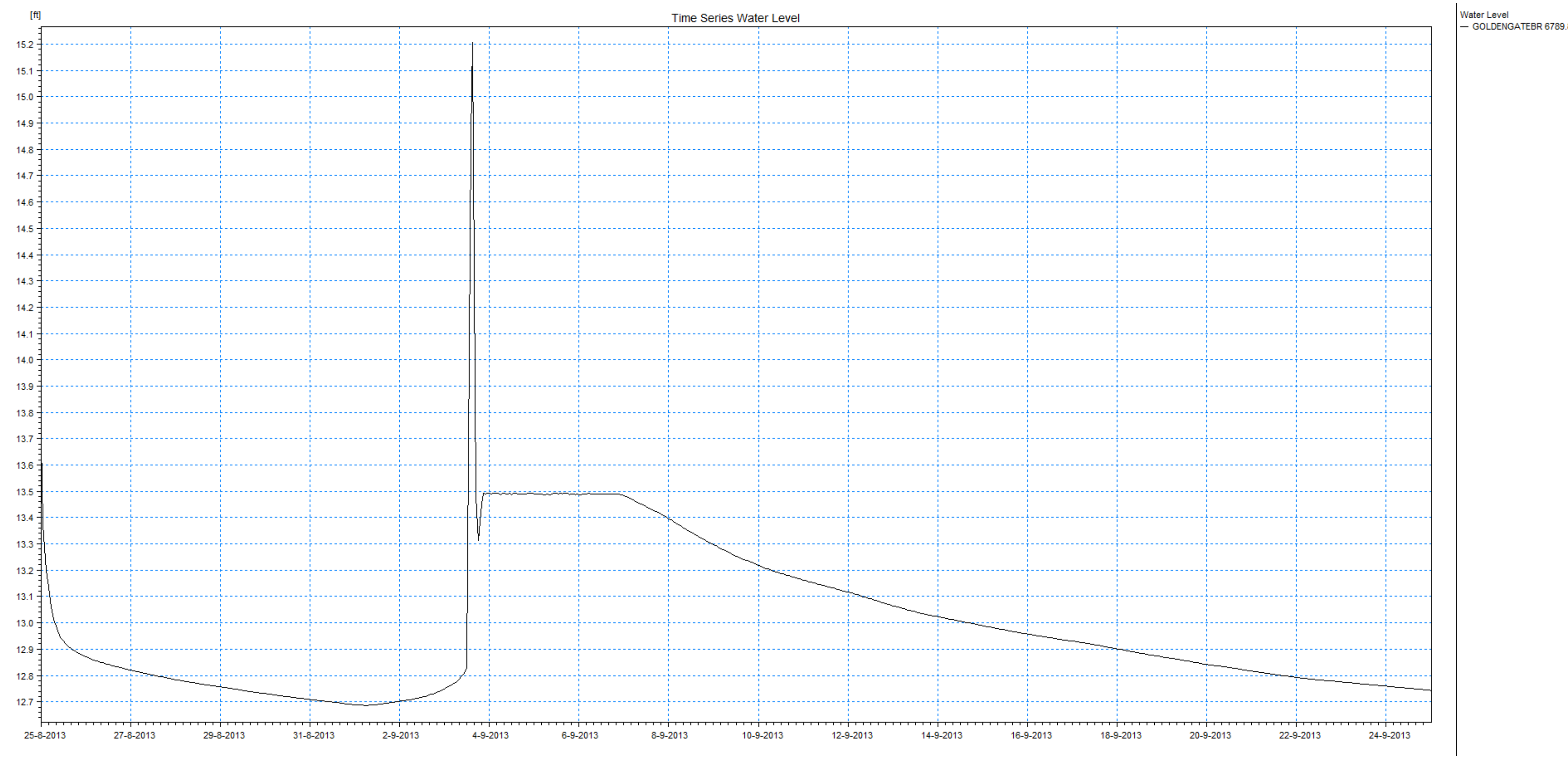

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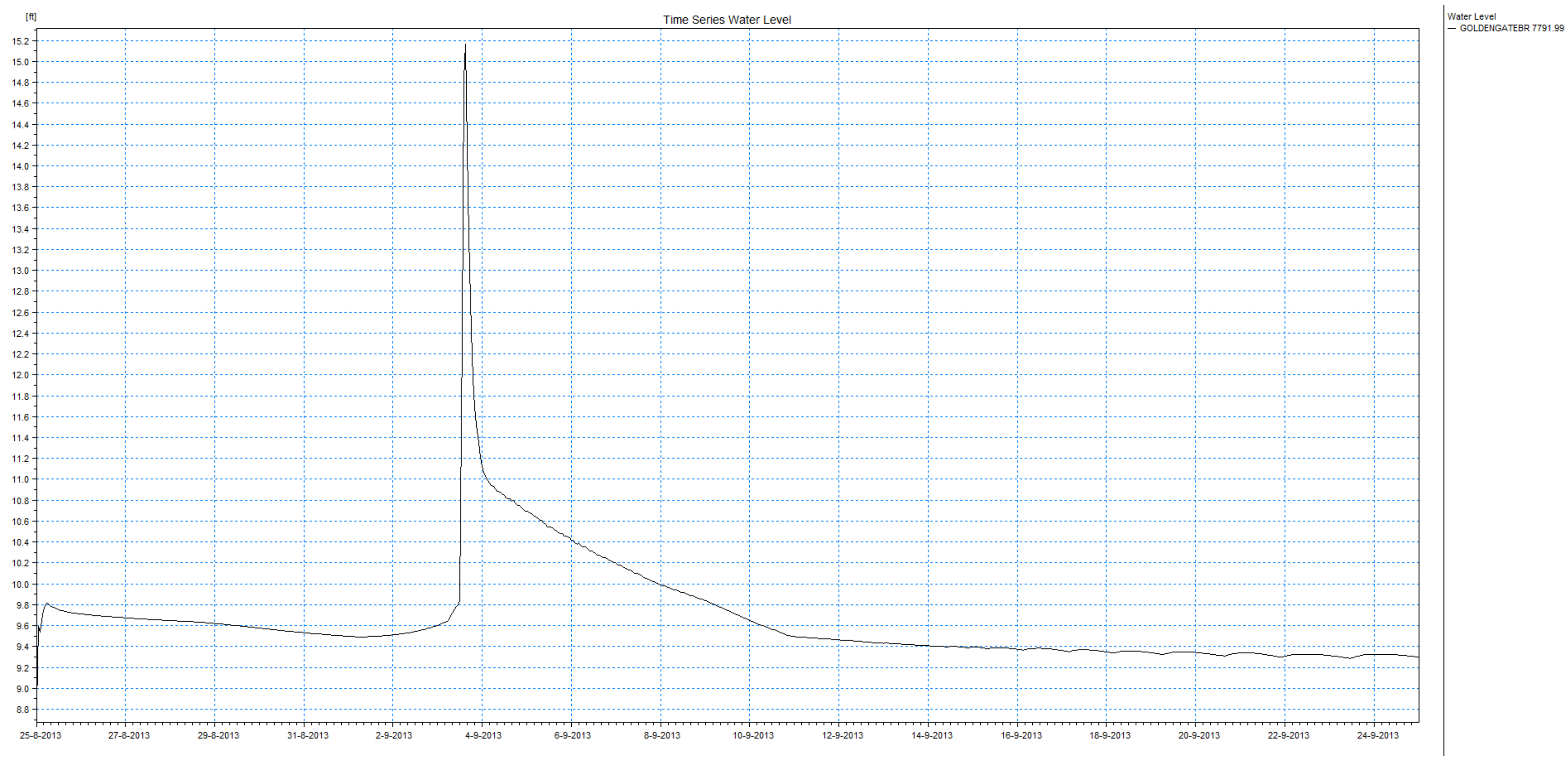

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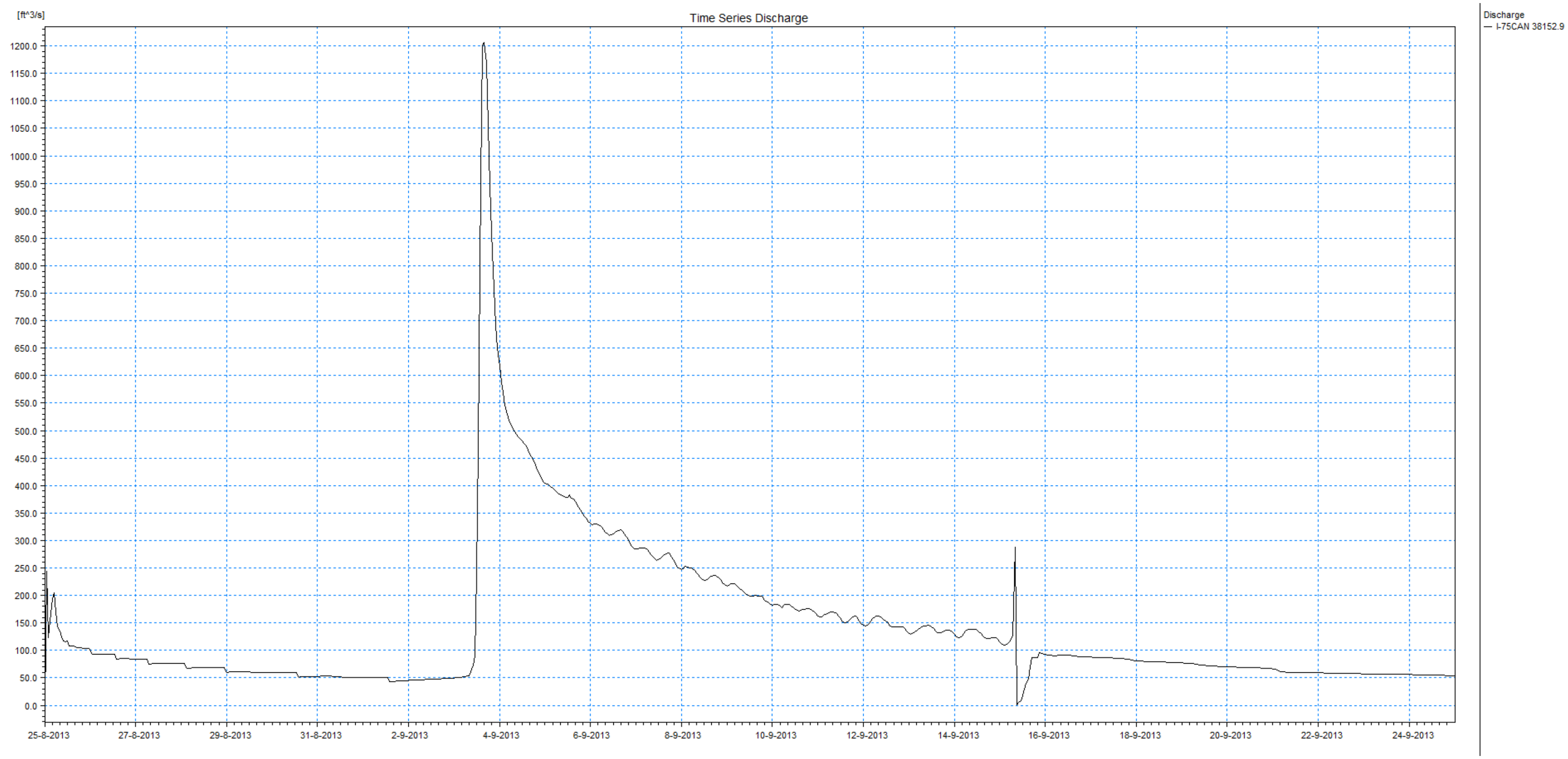

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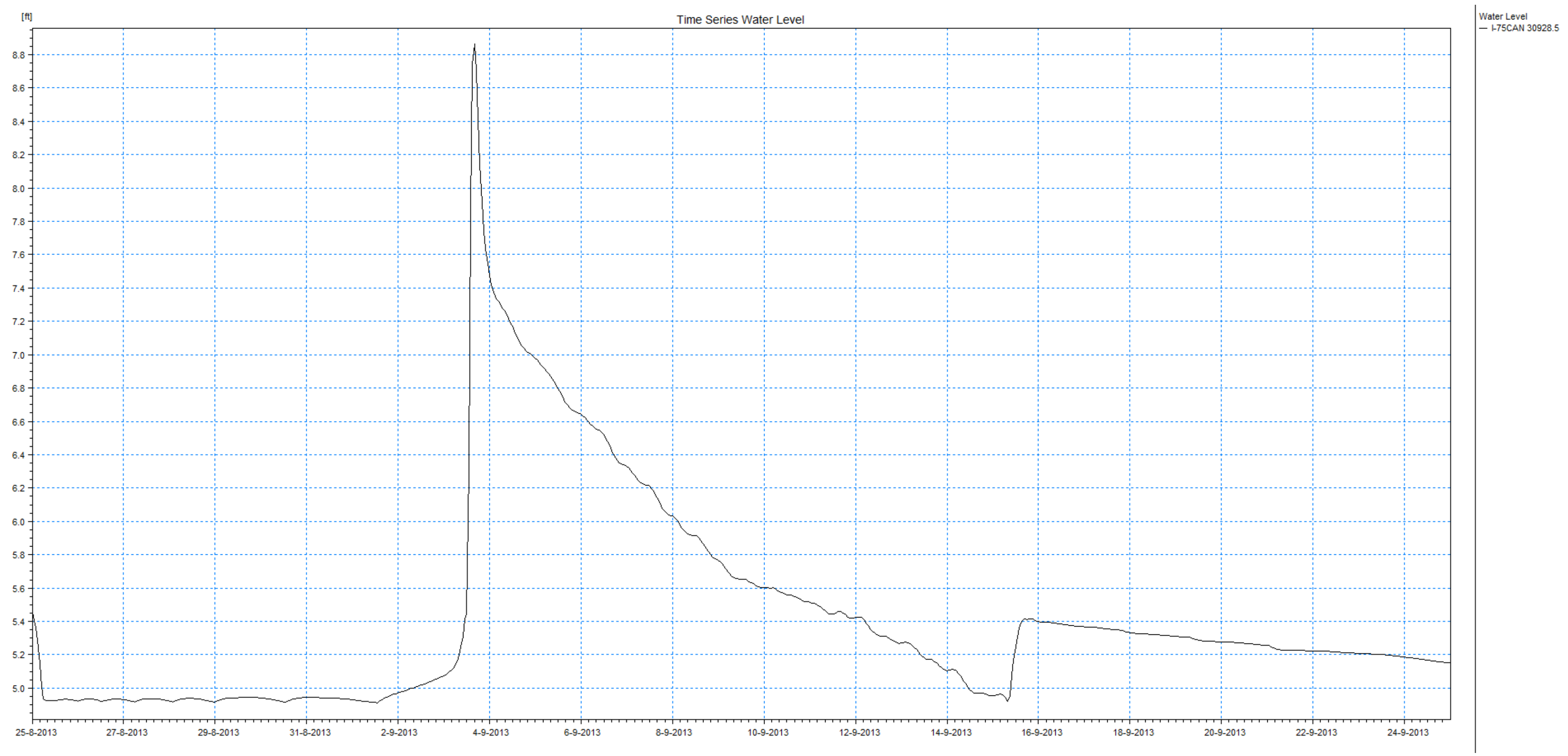

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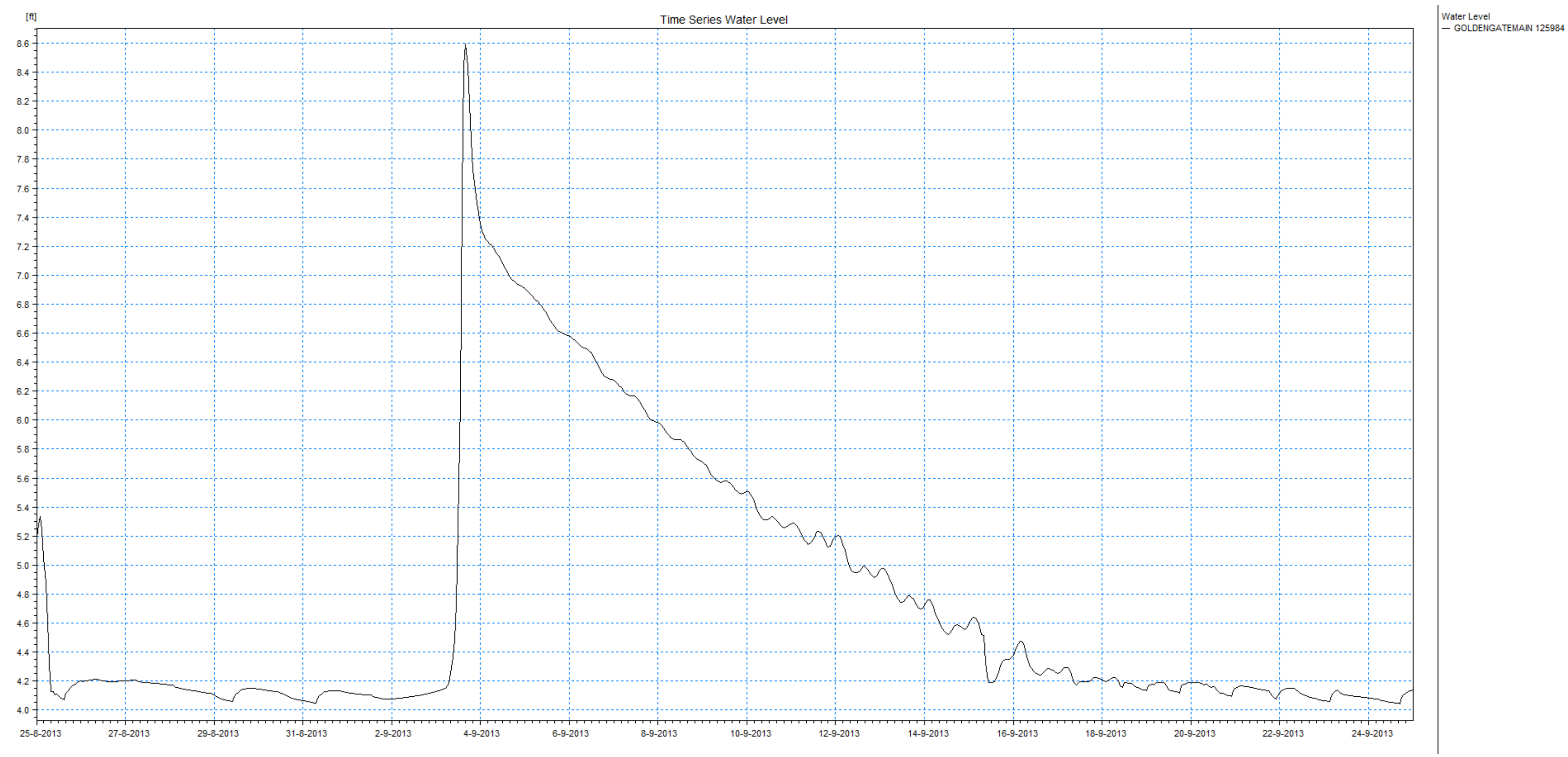

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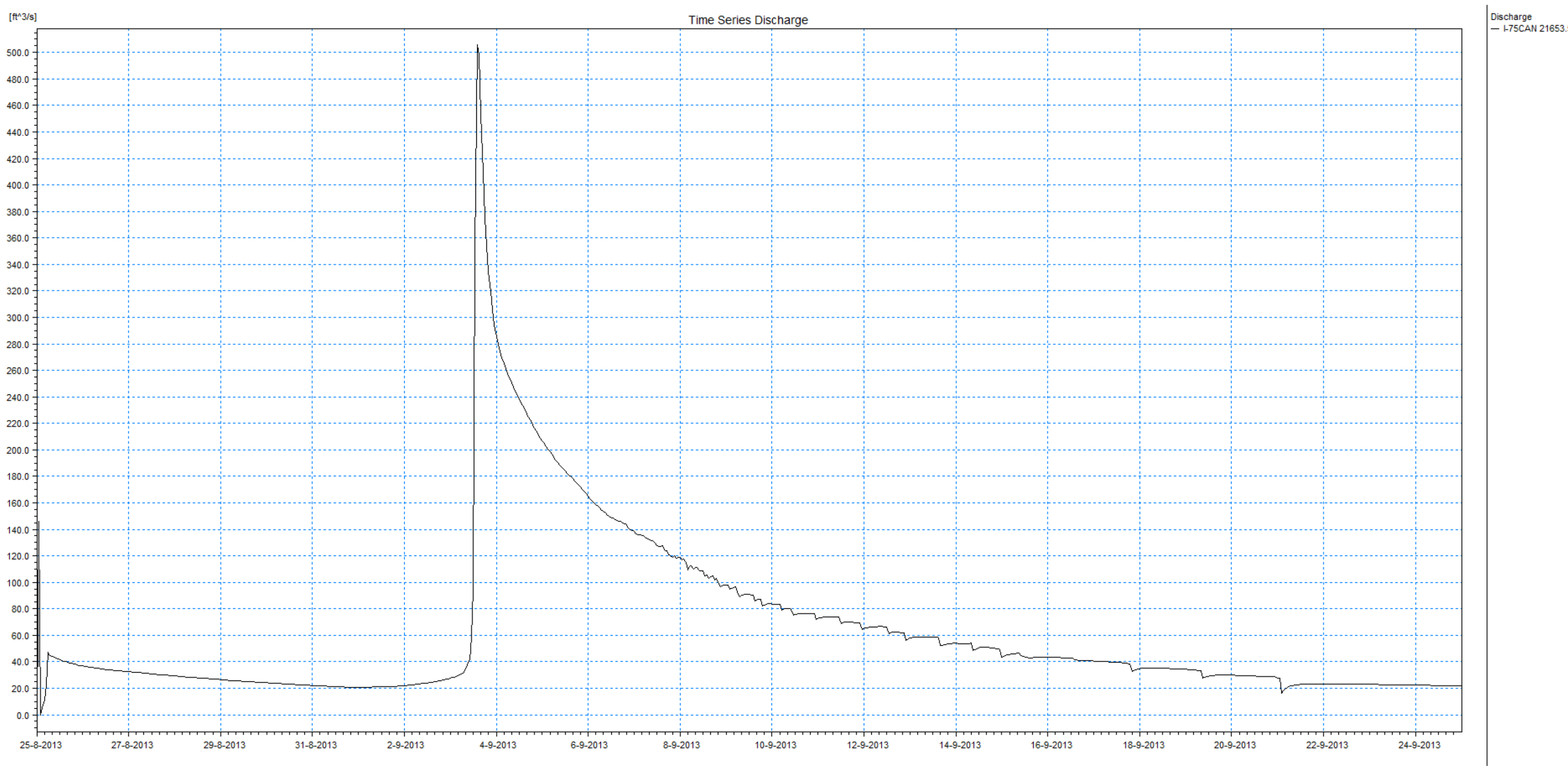

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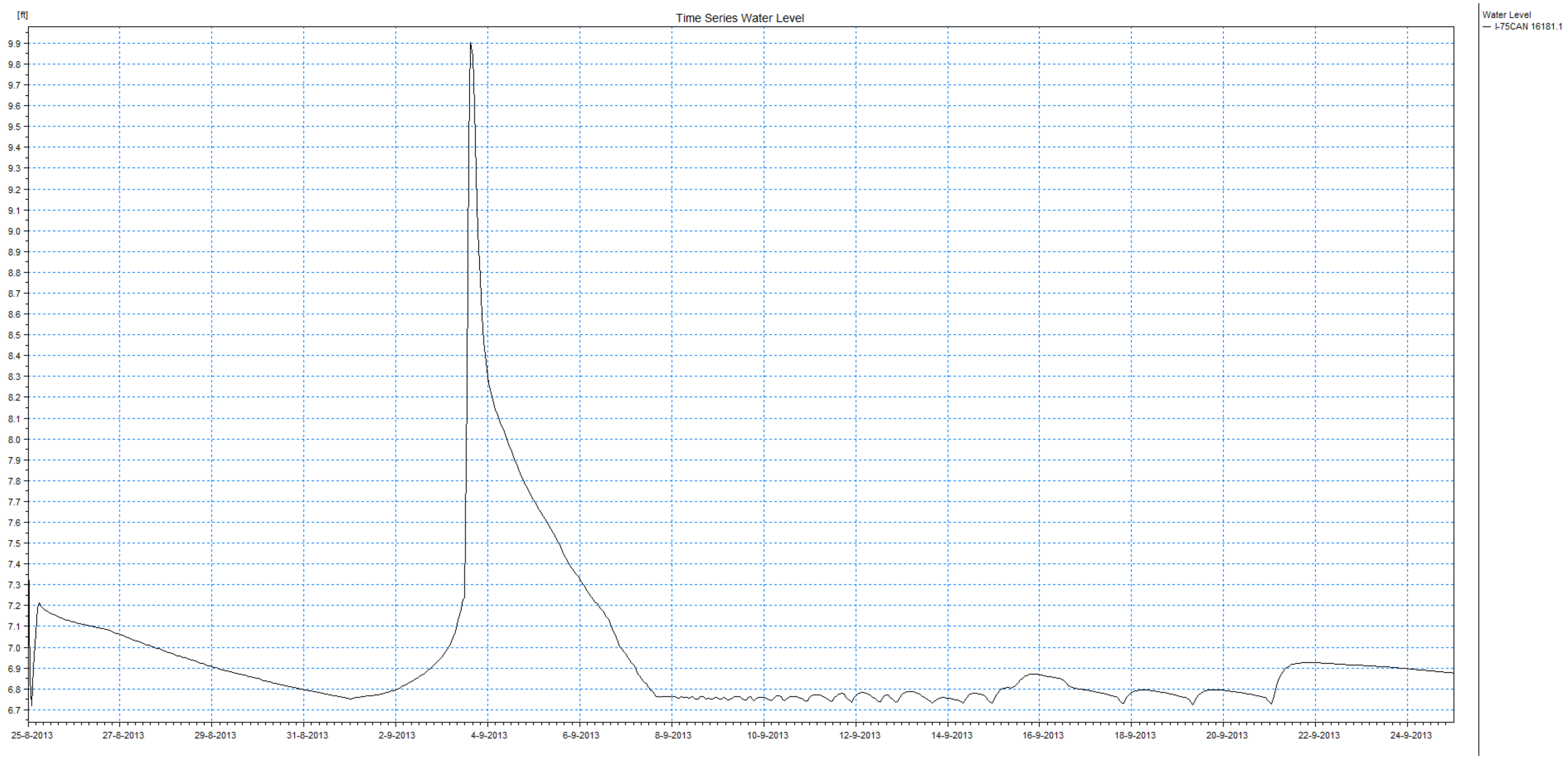

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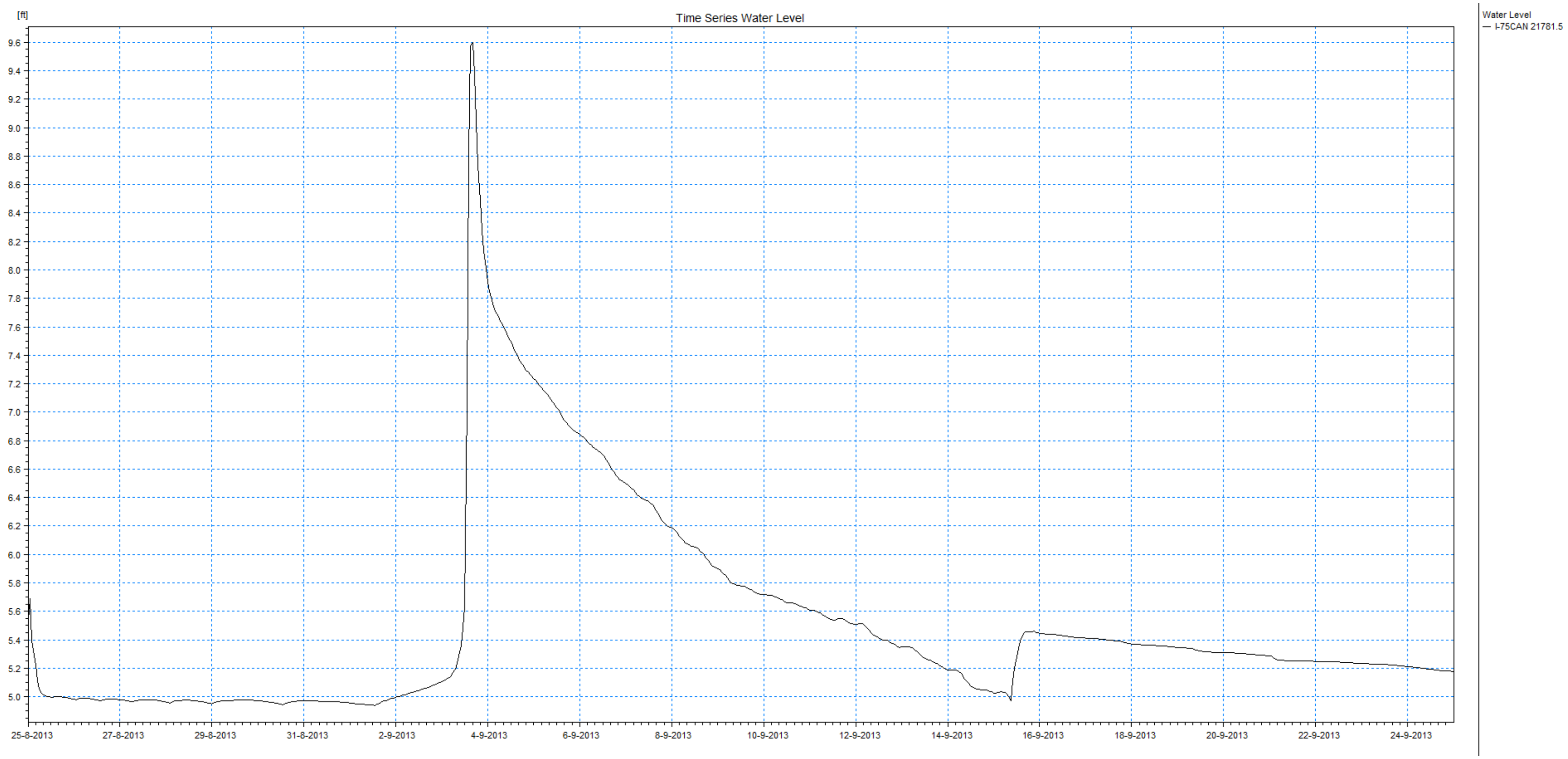

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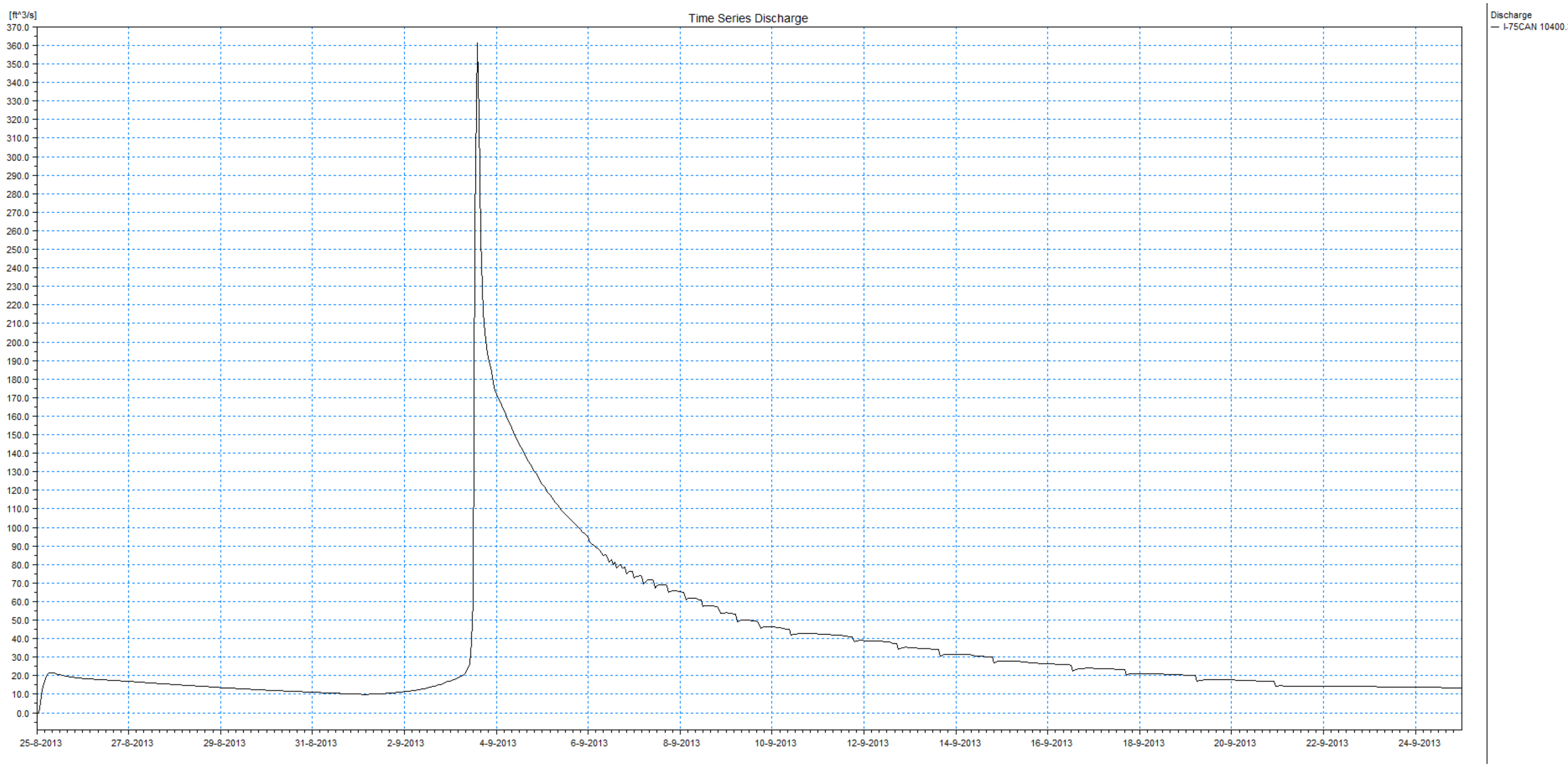

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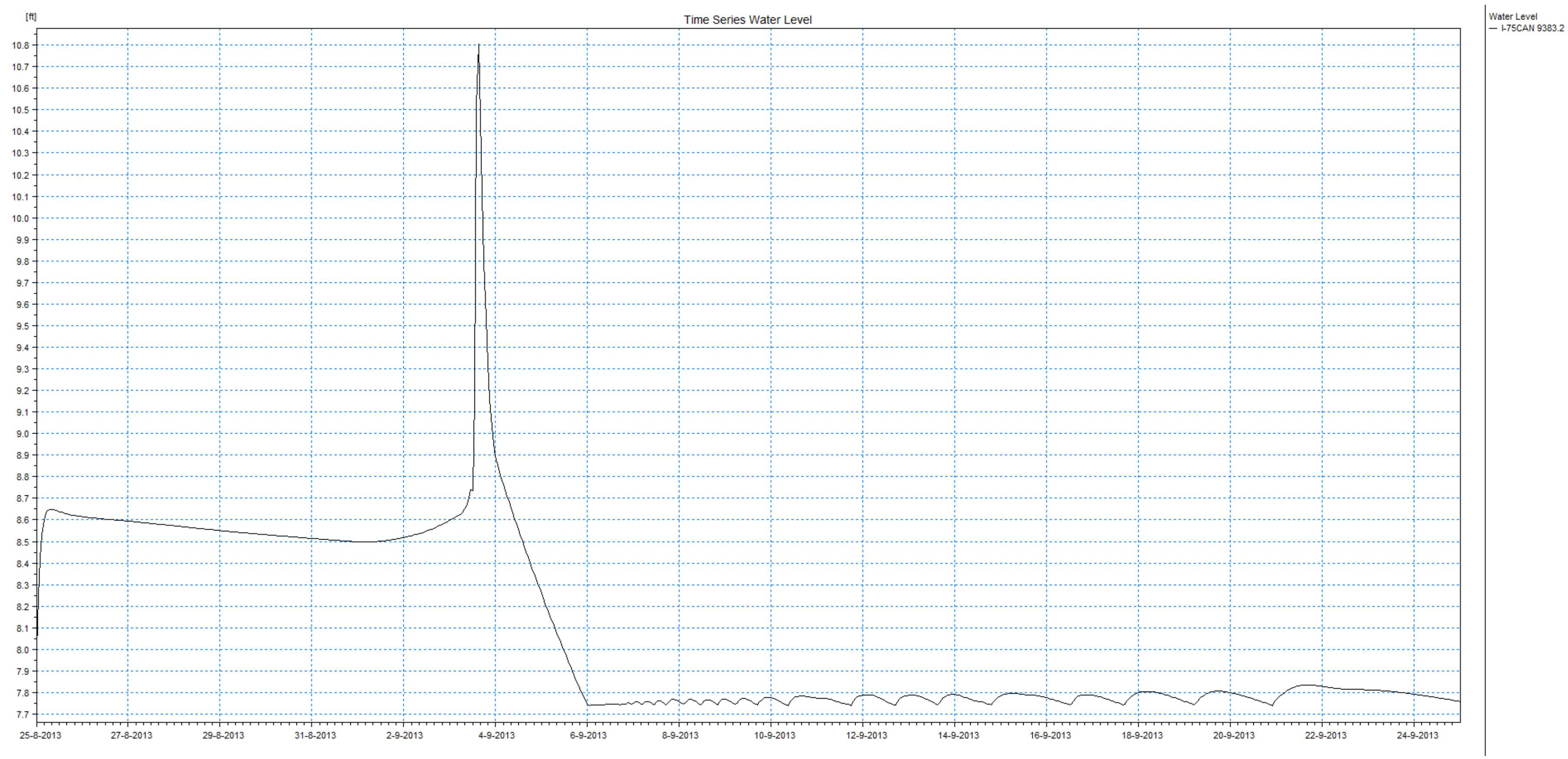

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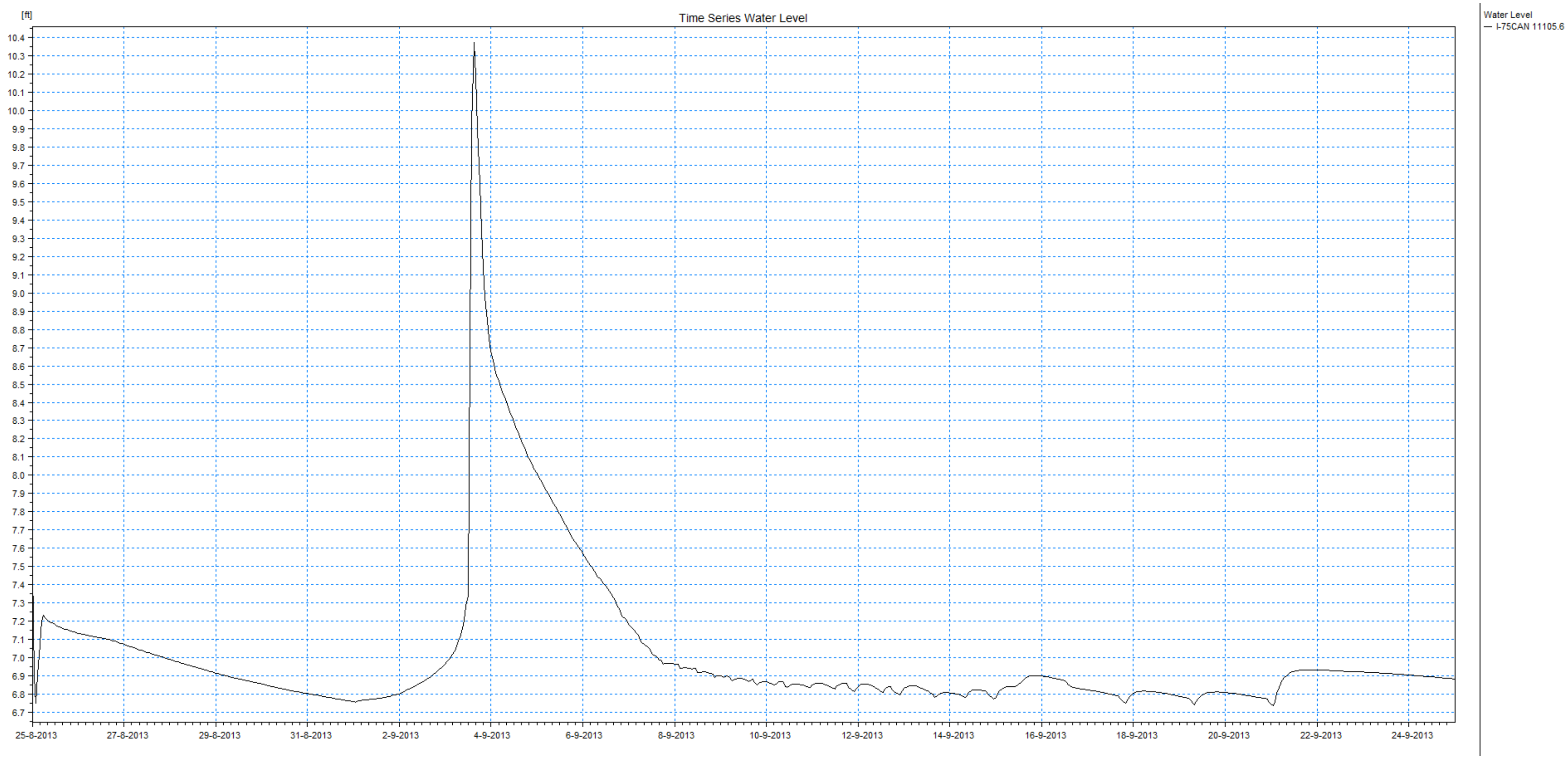

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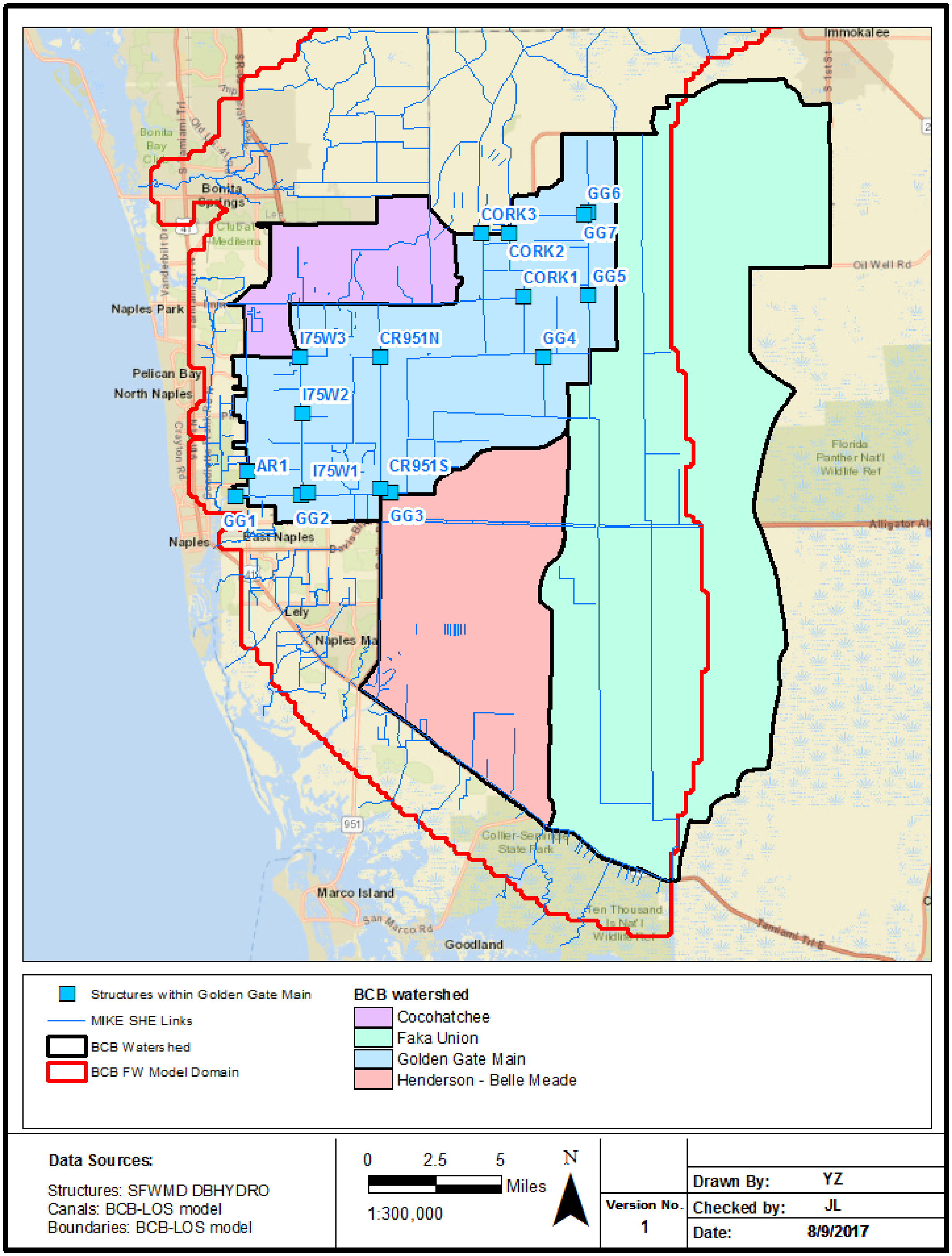

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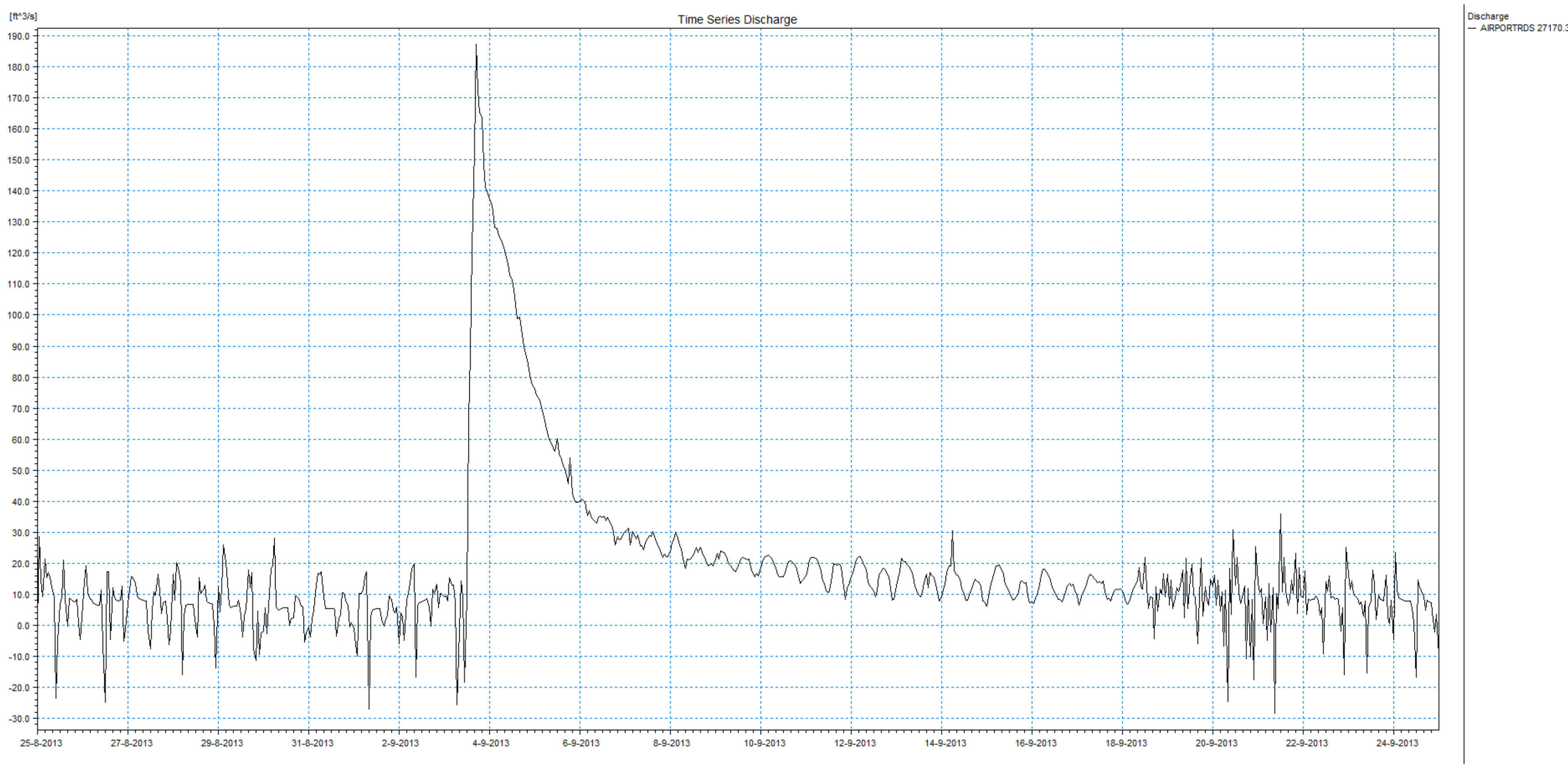

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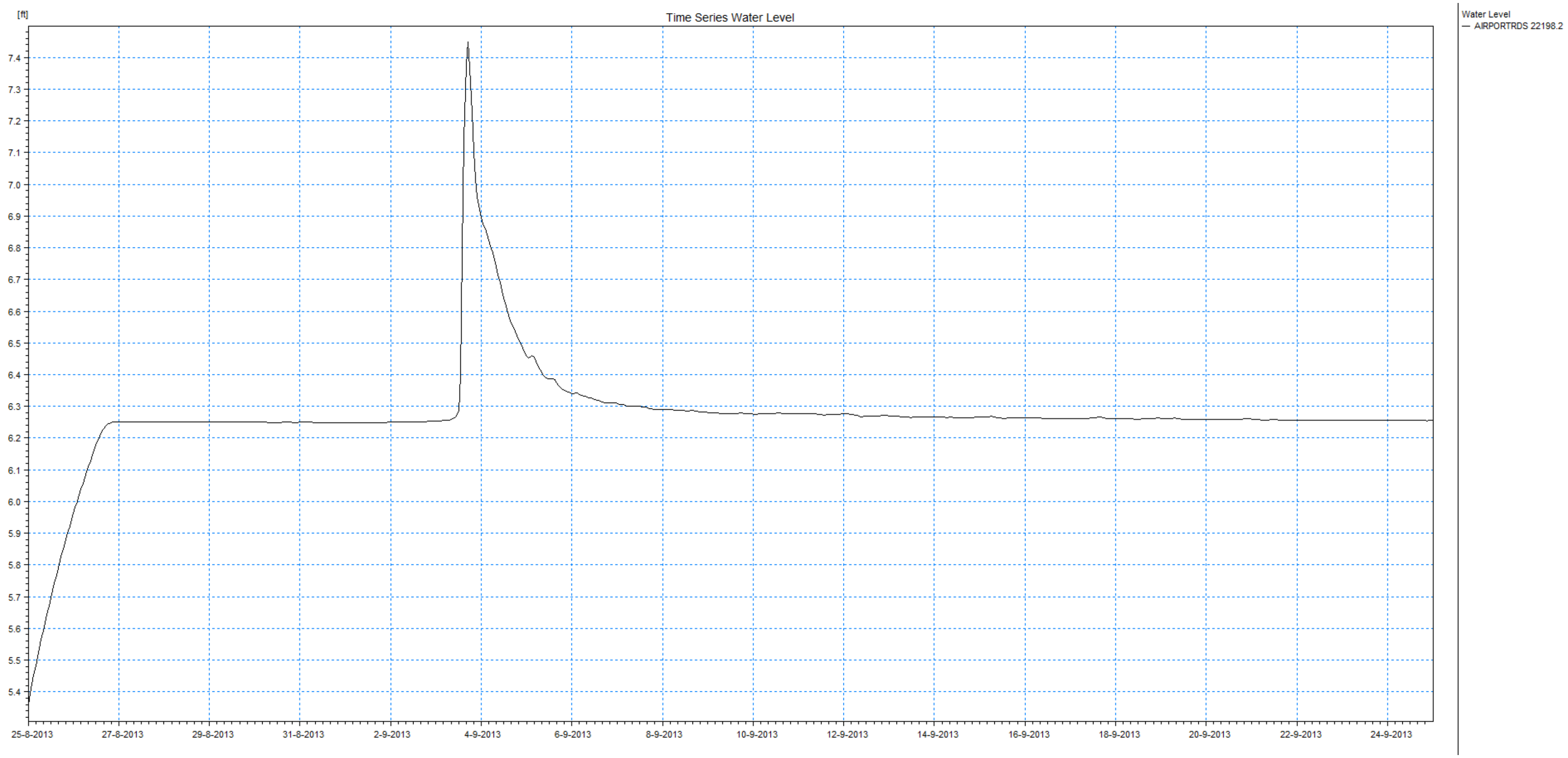

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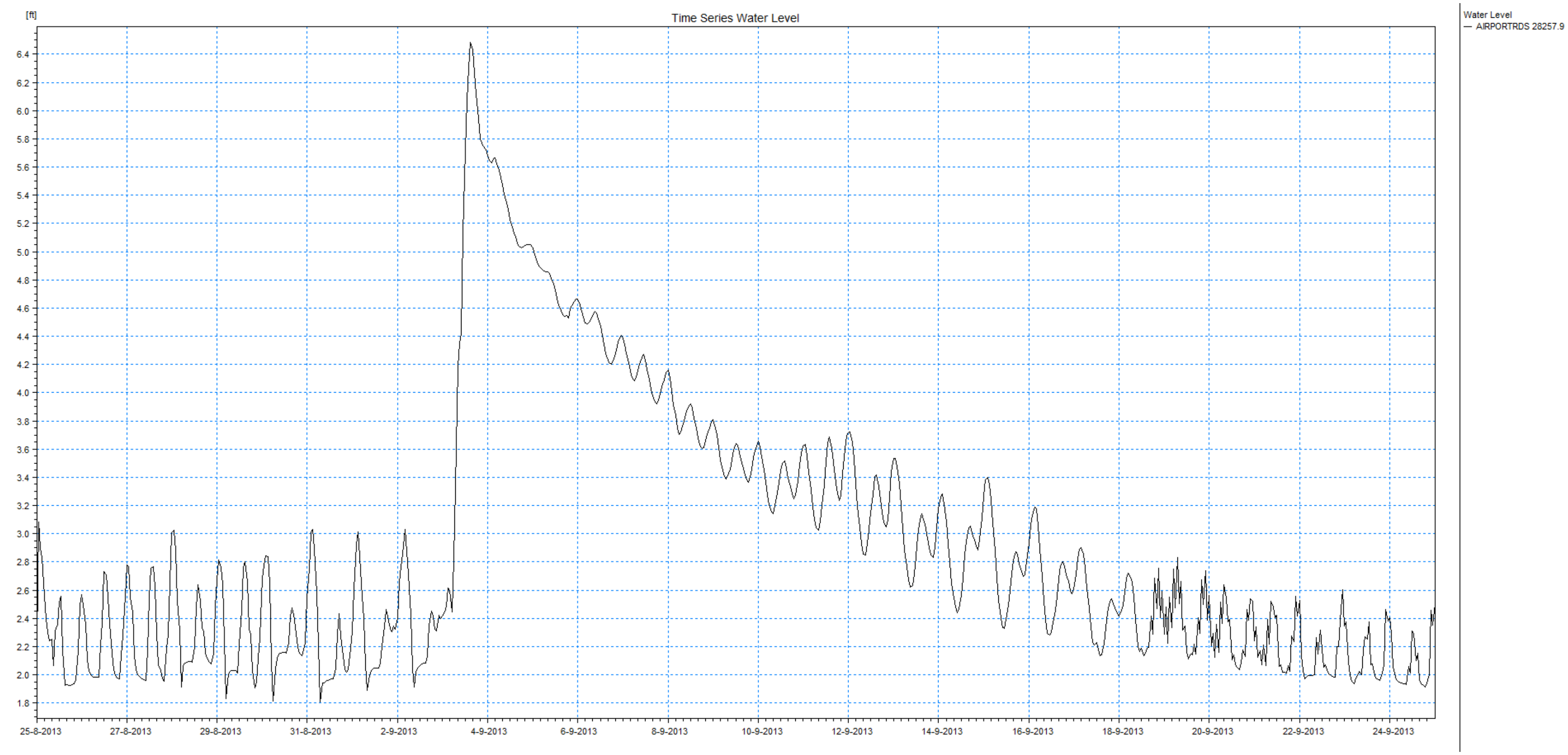
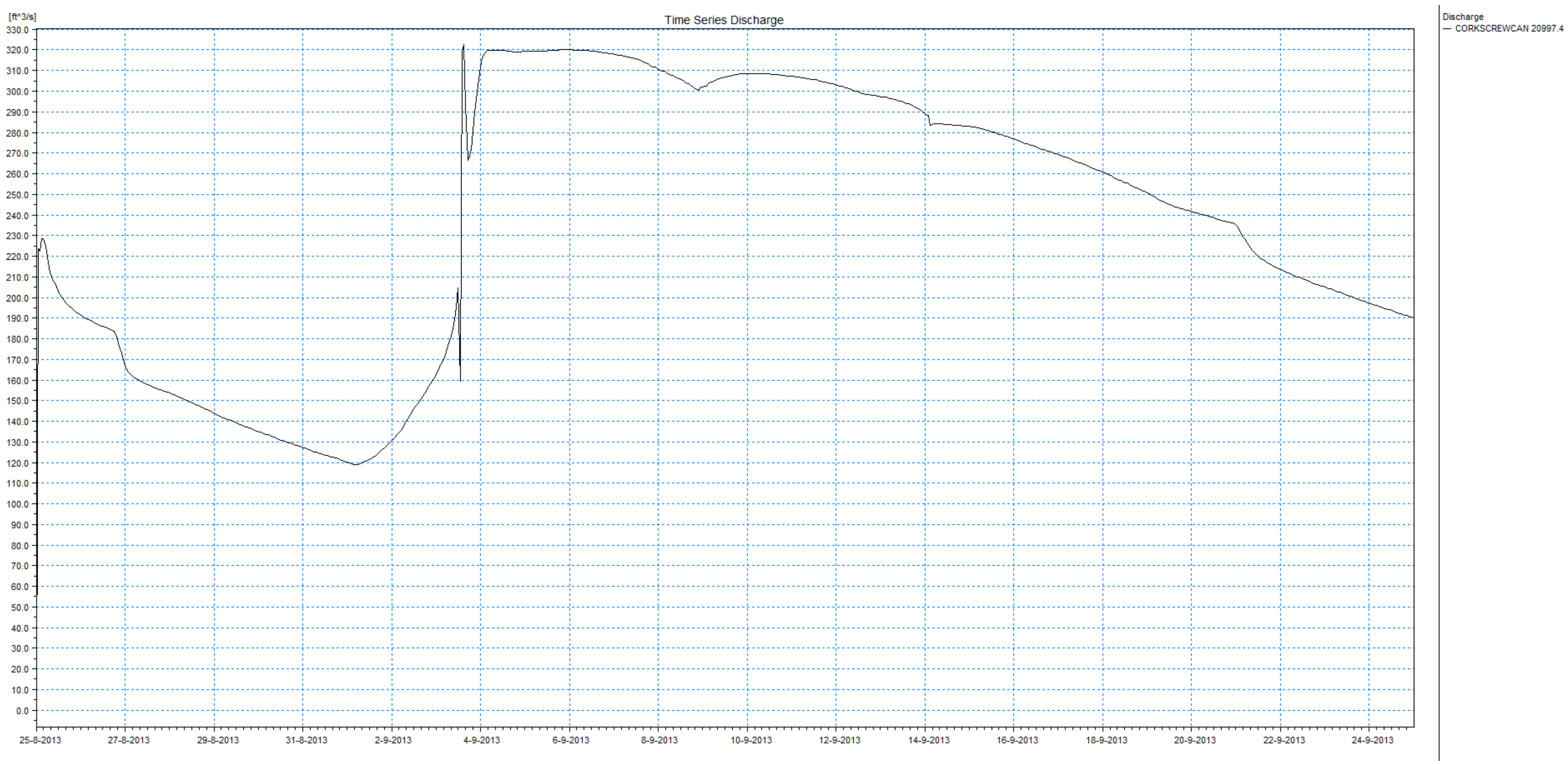


Figure 4. AR1 Tailwater Stage (ft)



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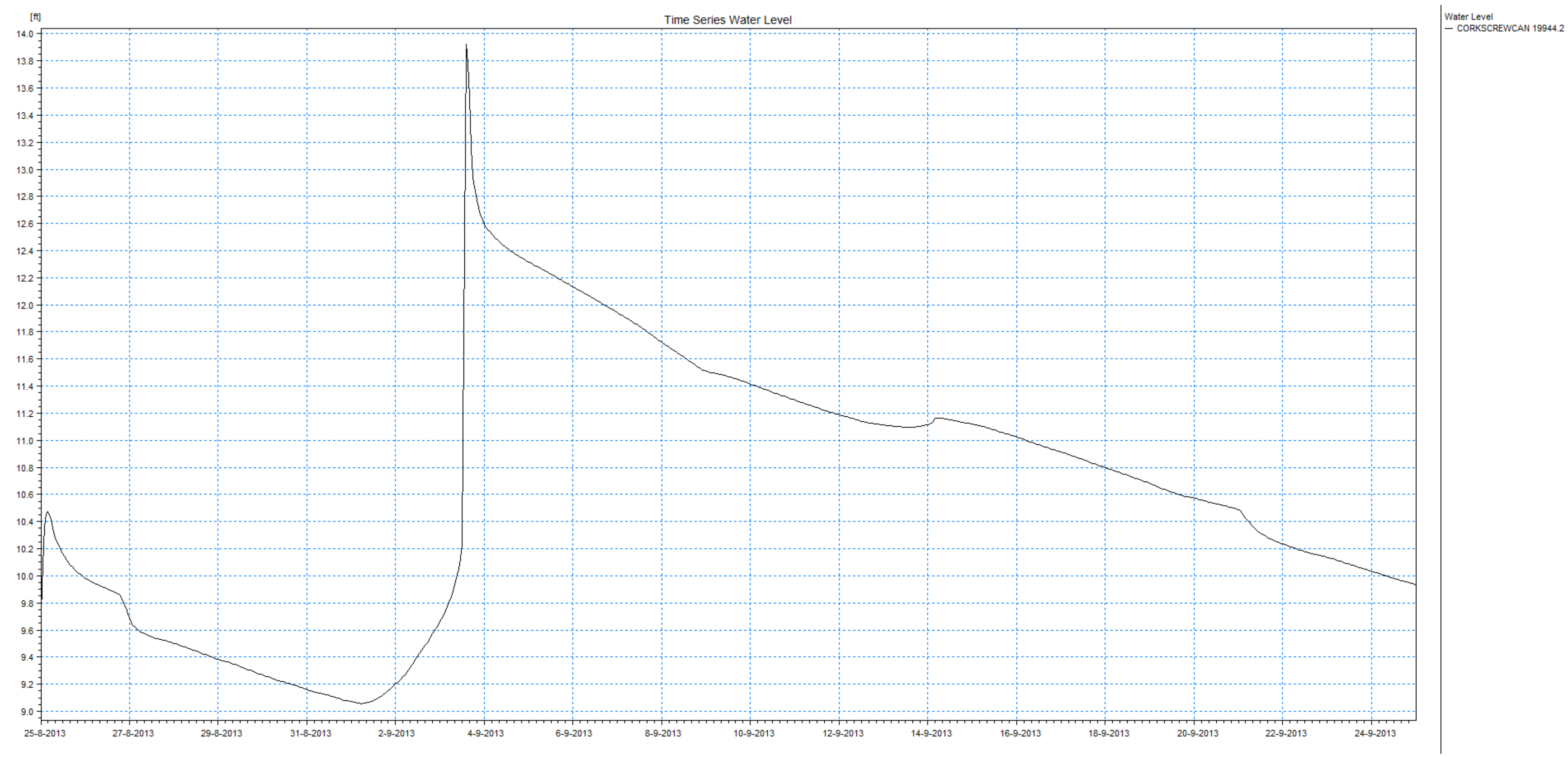


Figure 6. CORK 1 Headwater Stage (ft)

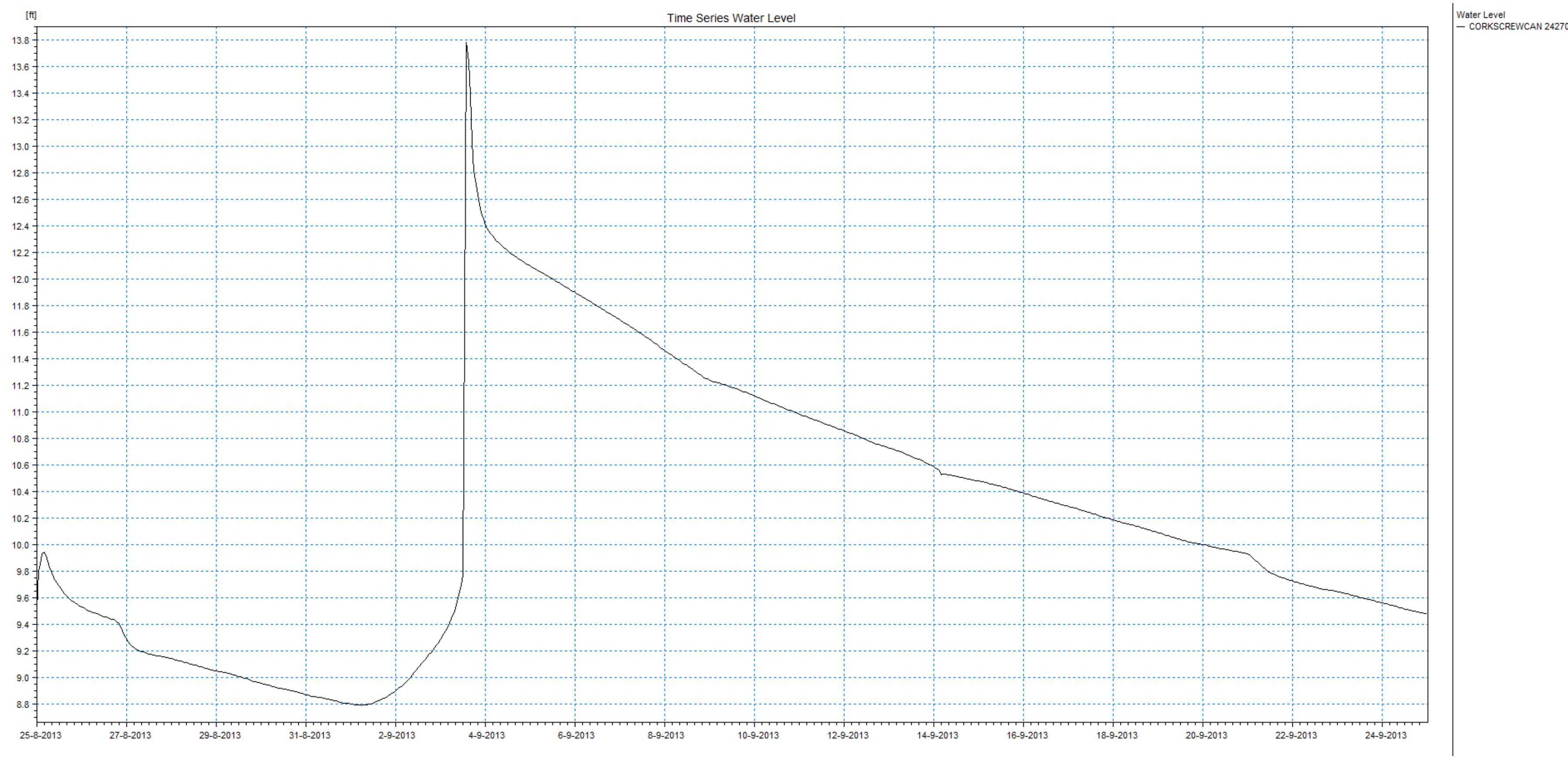


Figure 7. CORK 1 Tailwater Stage (ft)

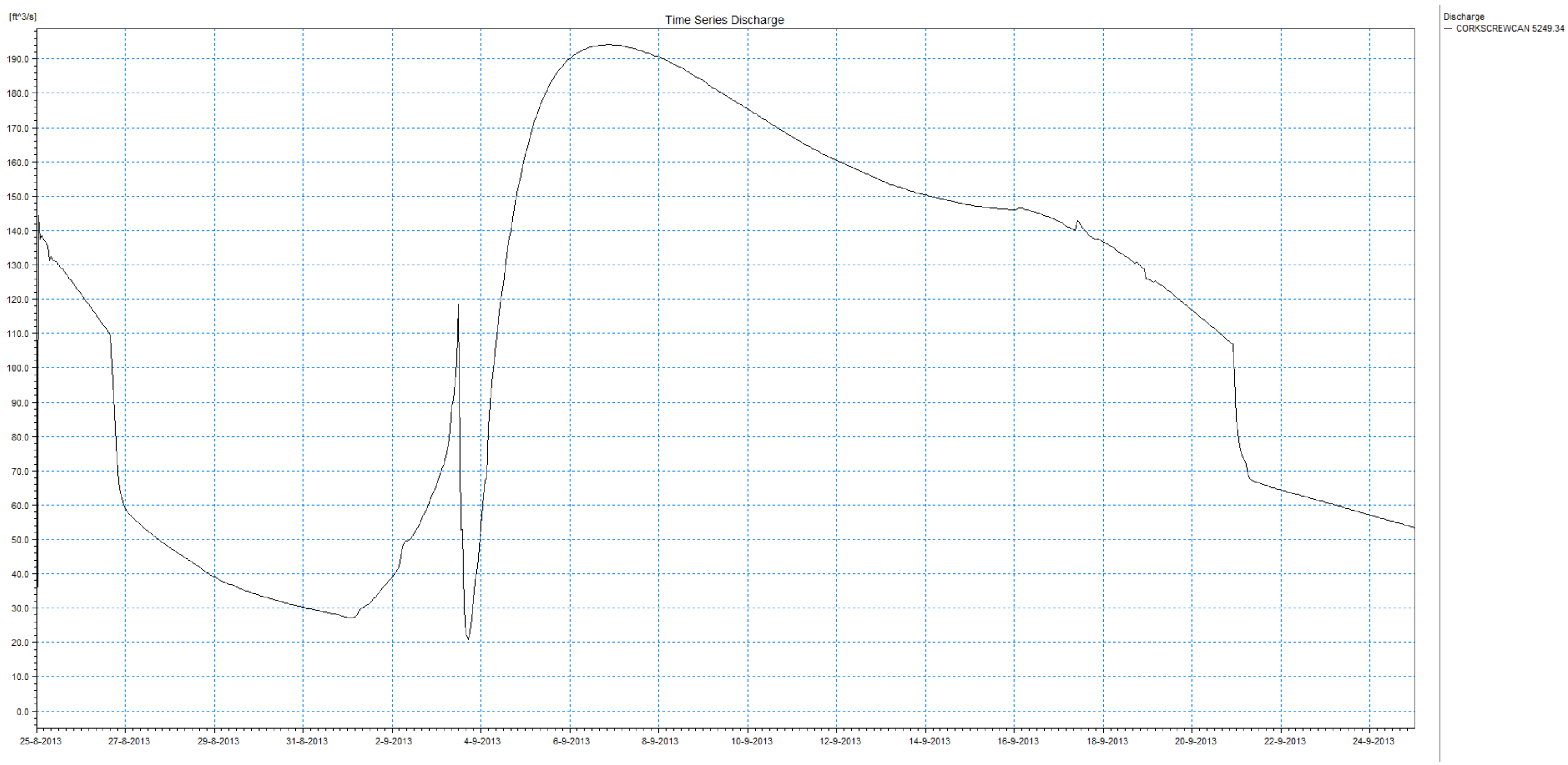


Figure 8. CORK 2 Discharge (cfs)

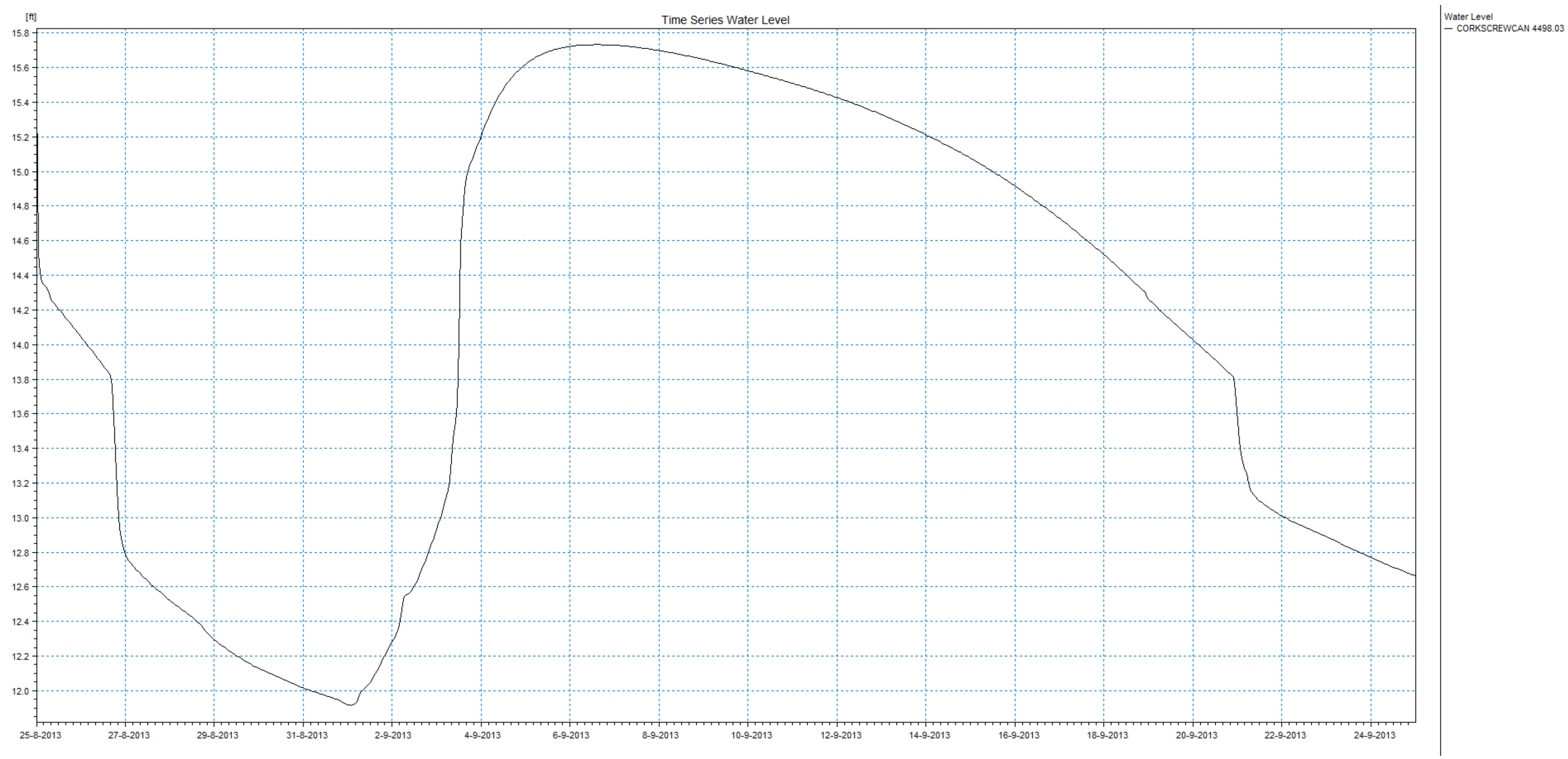


Figure 9. CORK 2 Headwater Stage (ft)

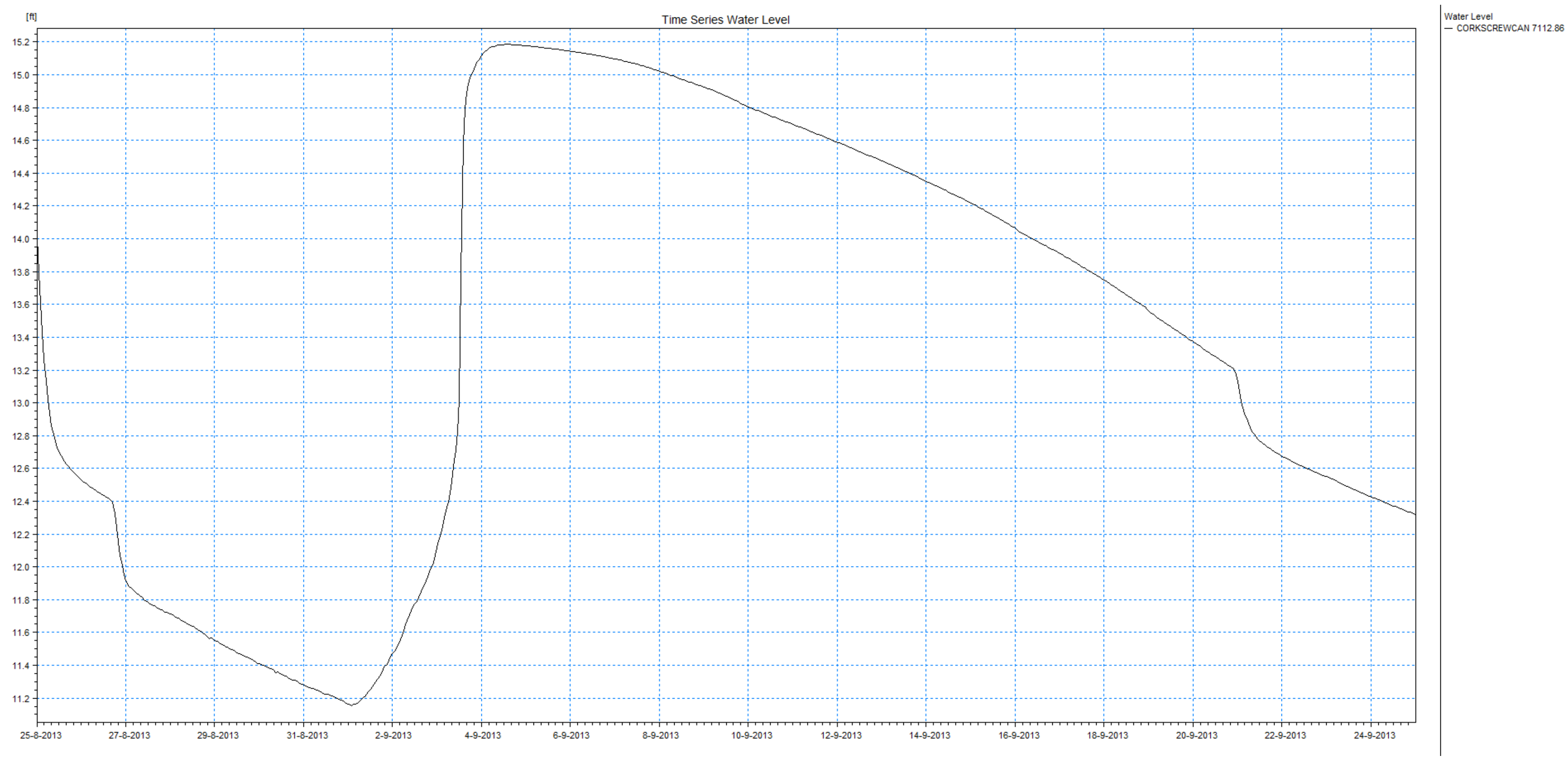
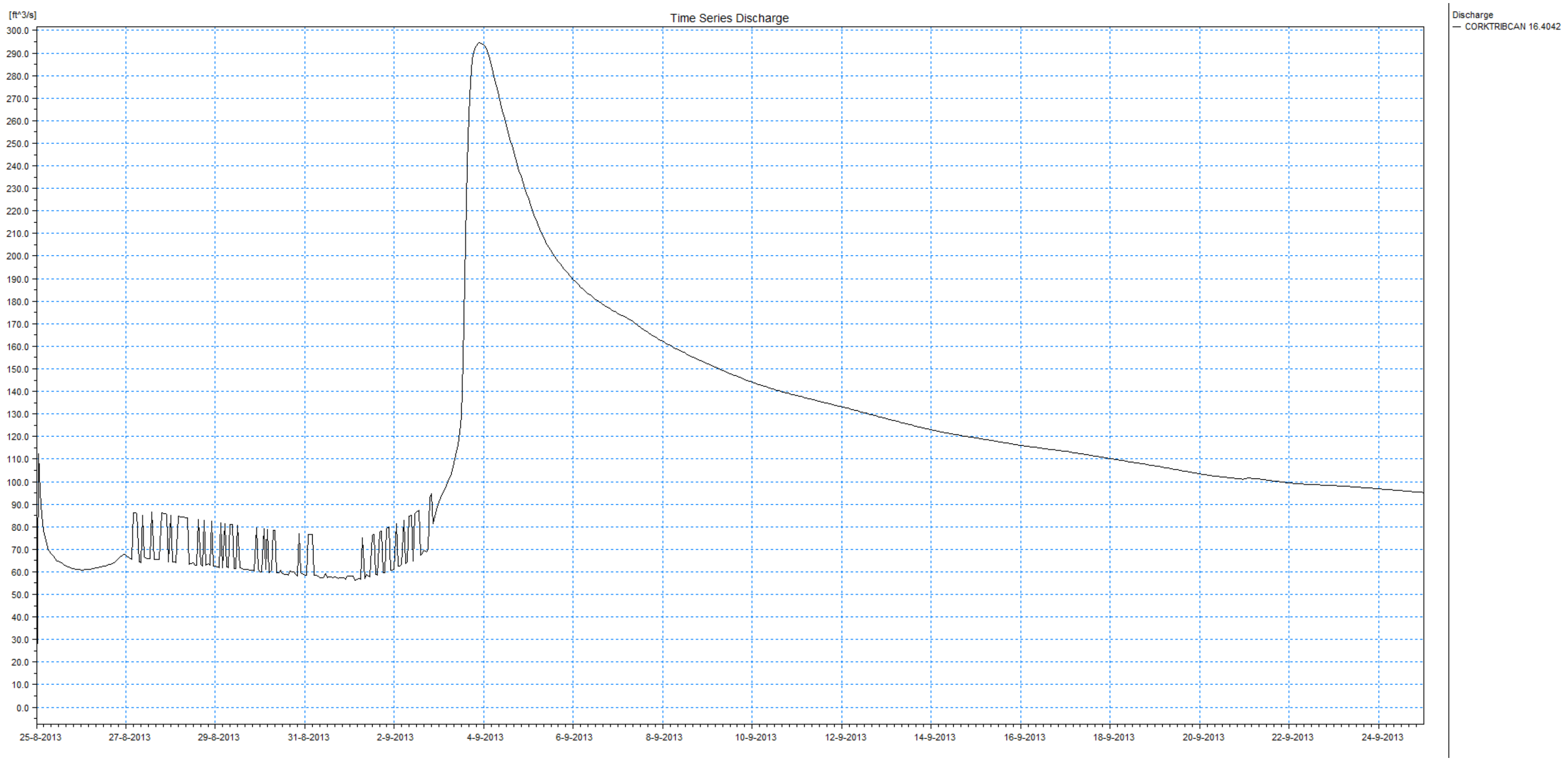


Figure 10. CORK 2 Tailwater Stage (ft)



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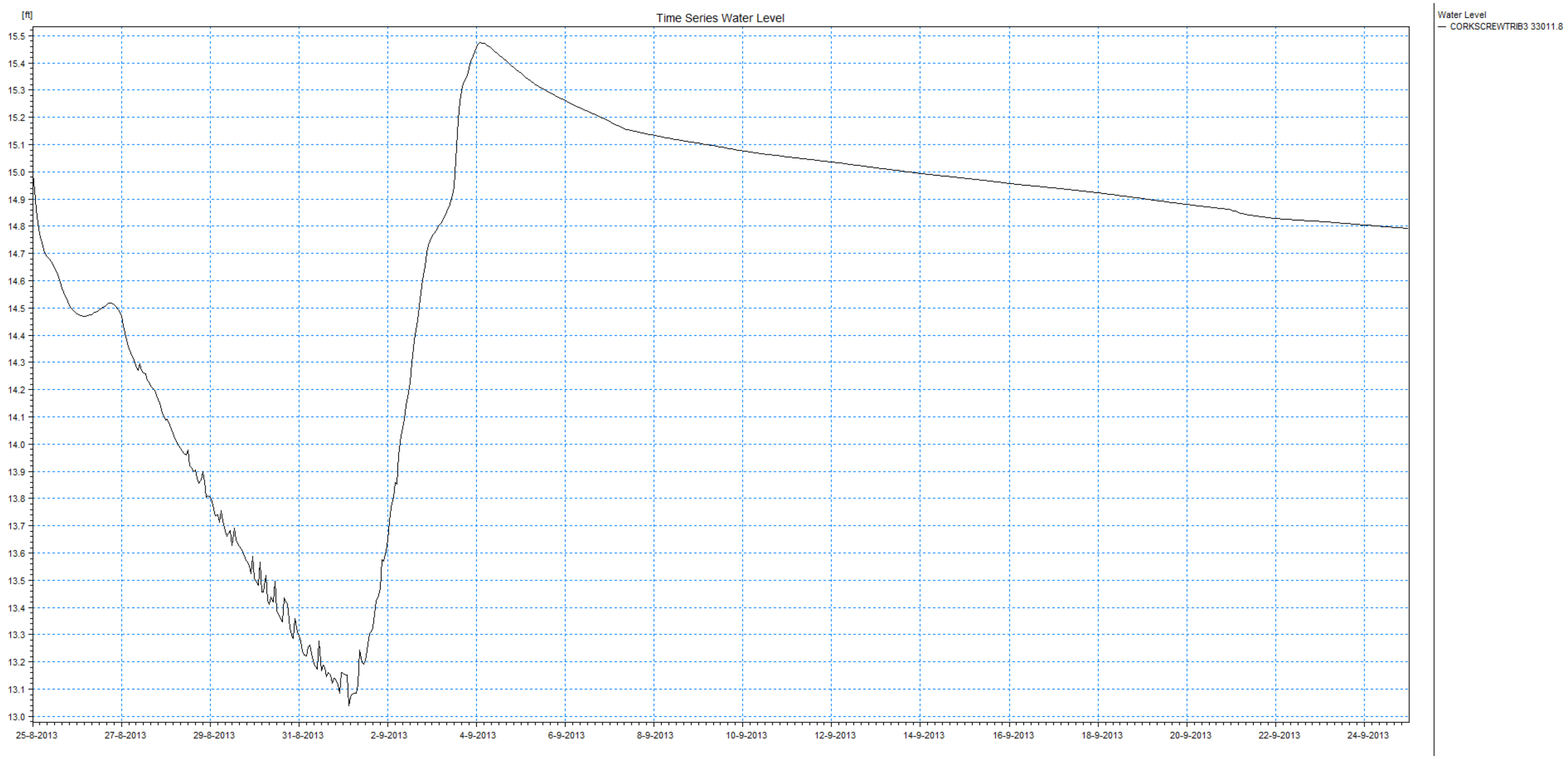

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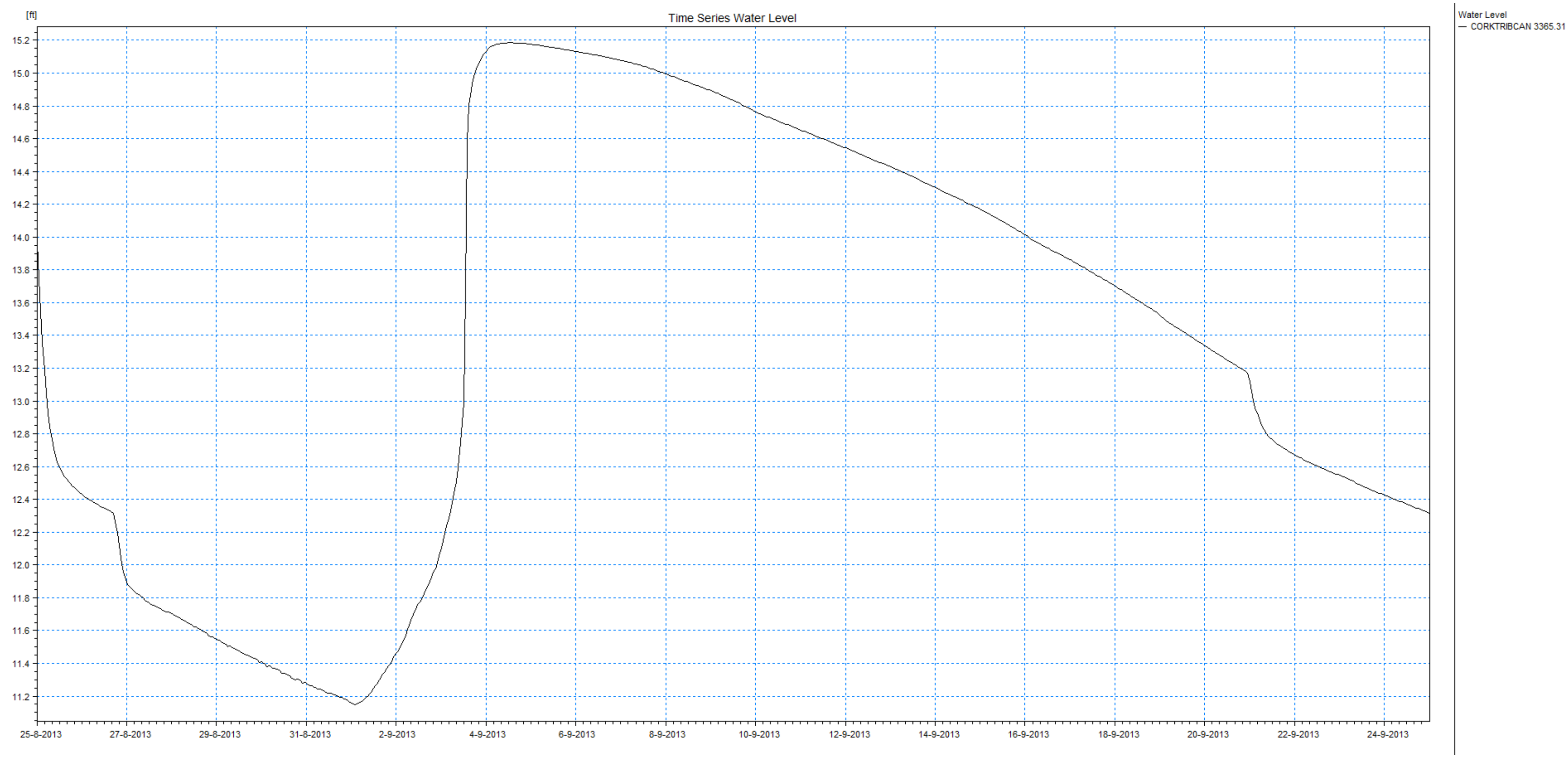
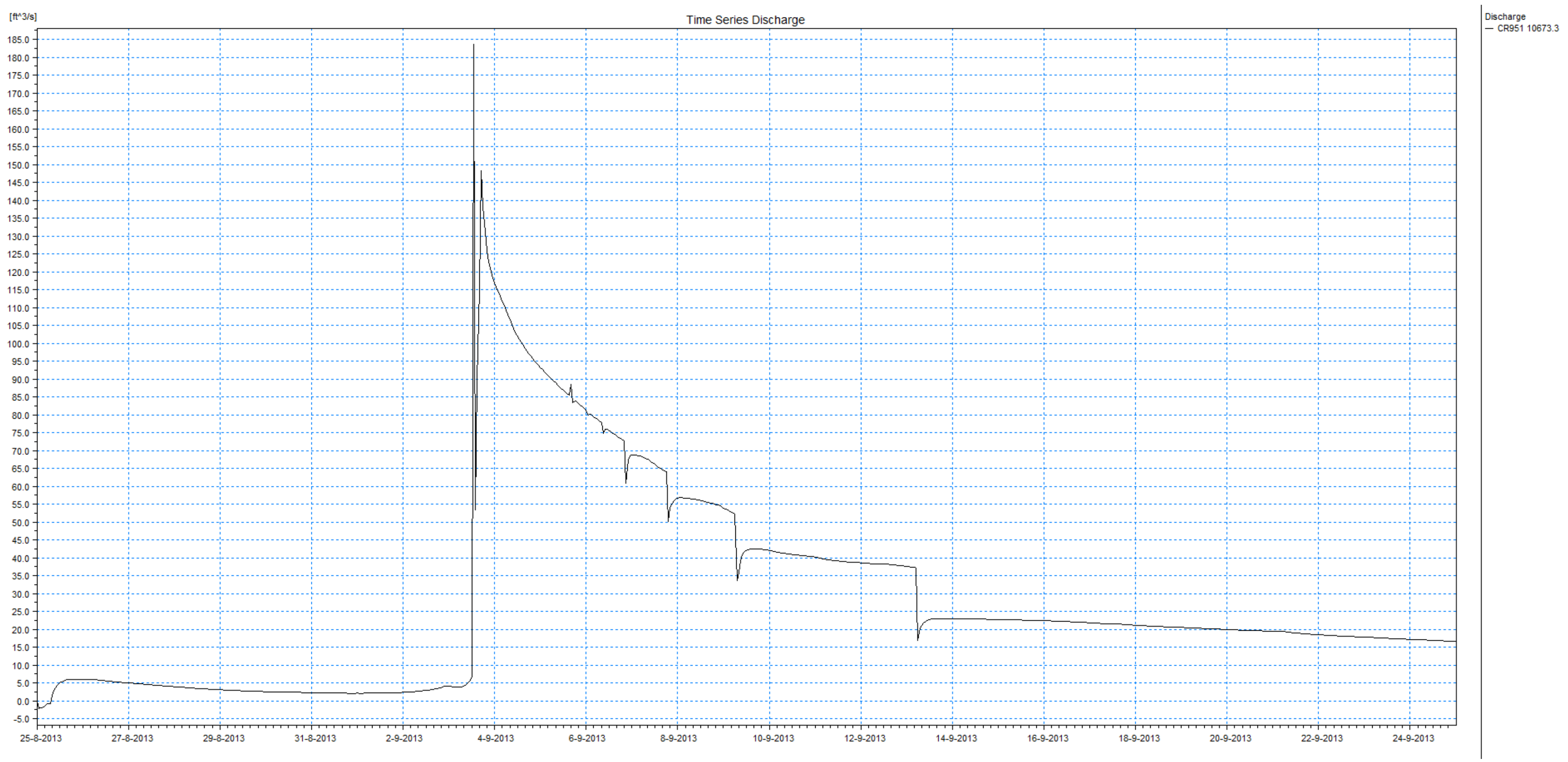


Figure 13. CORK 3 Tailwater Stage (ft)



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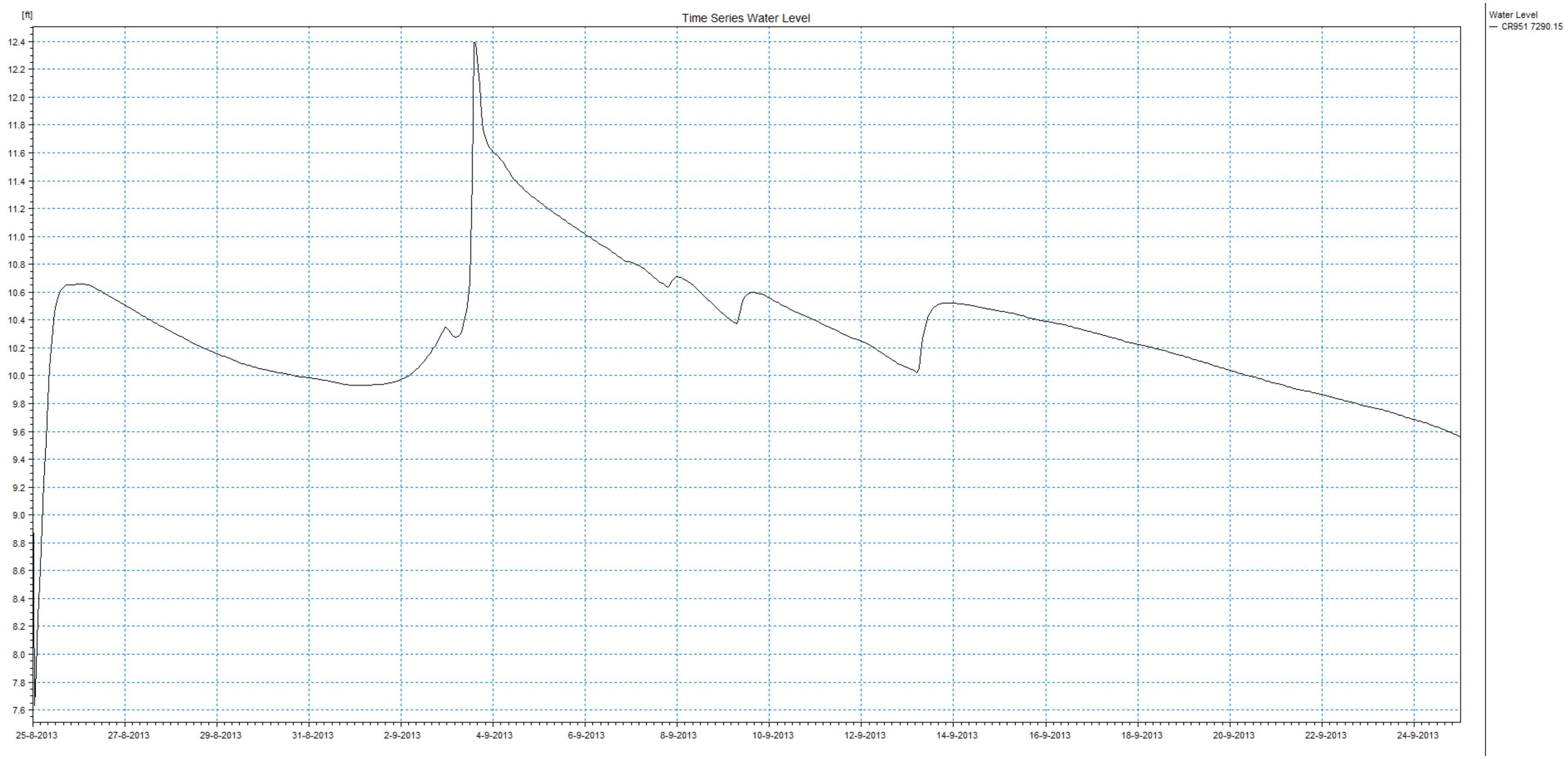

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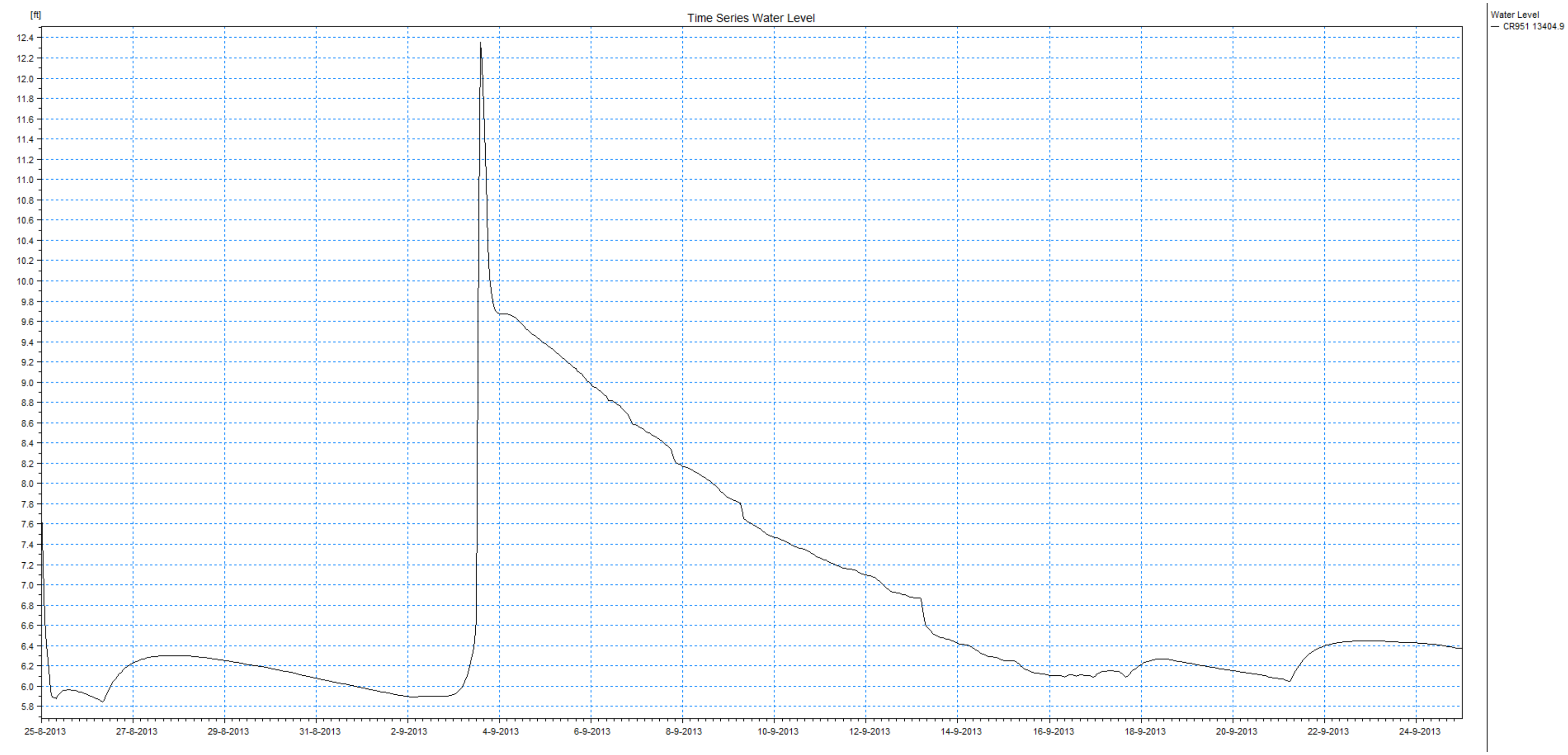

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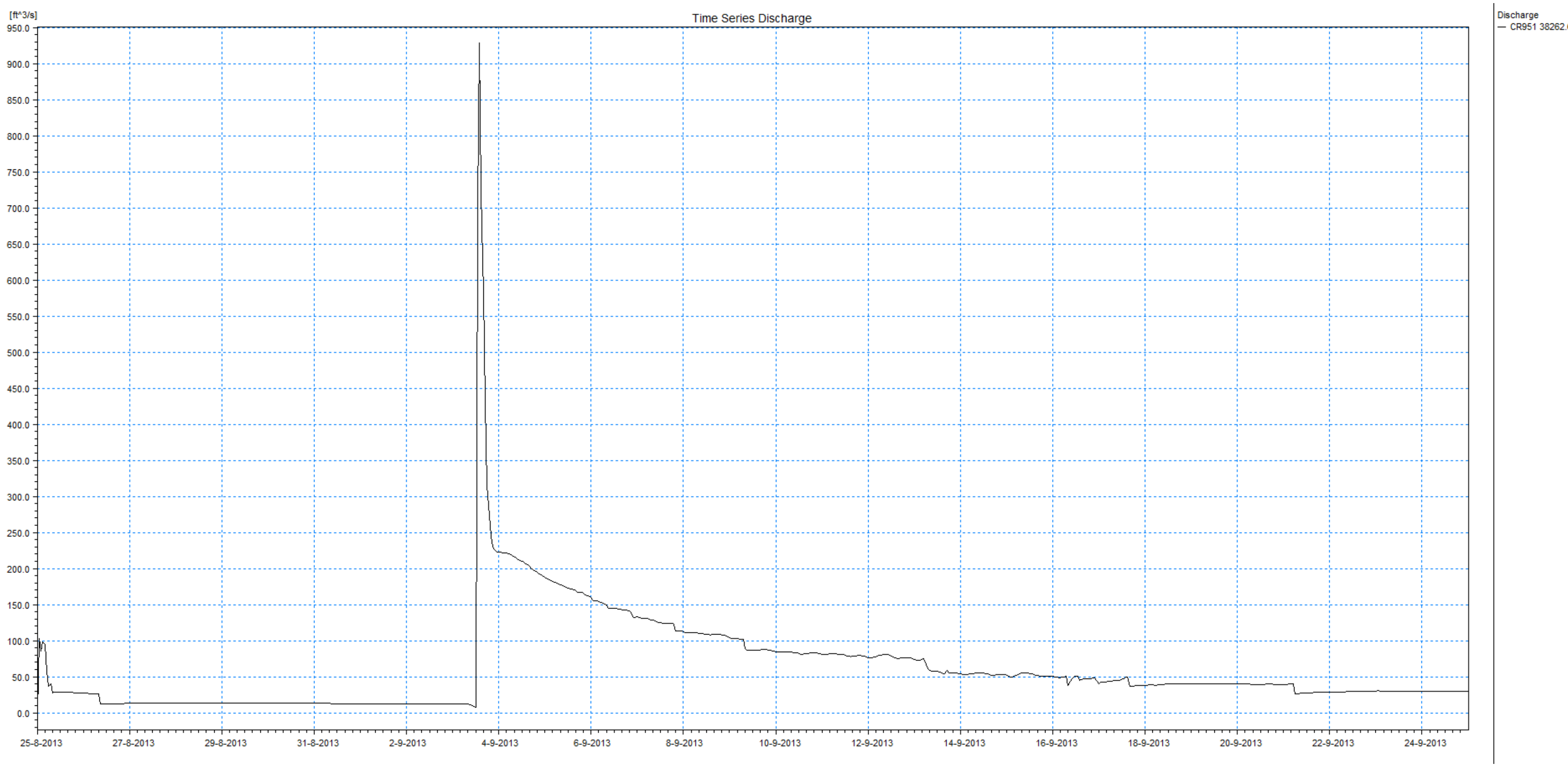

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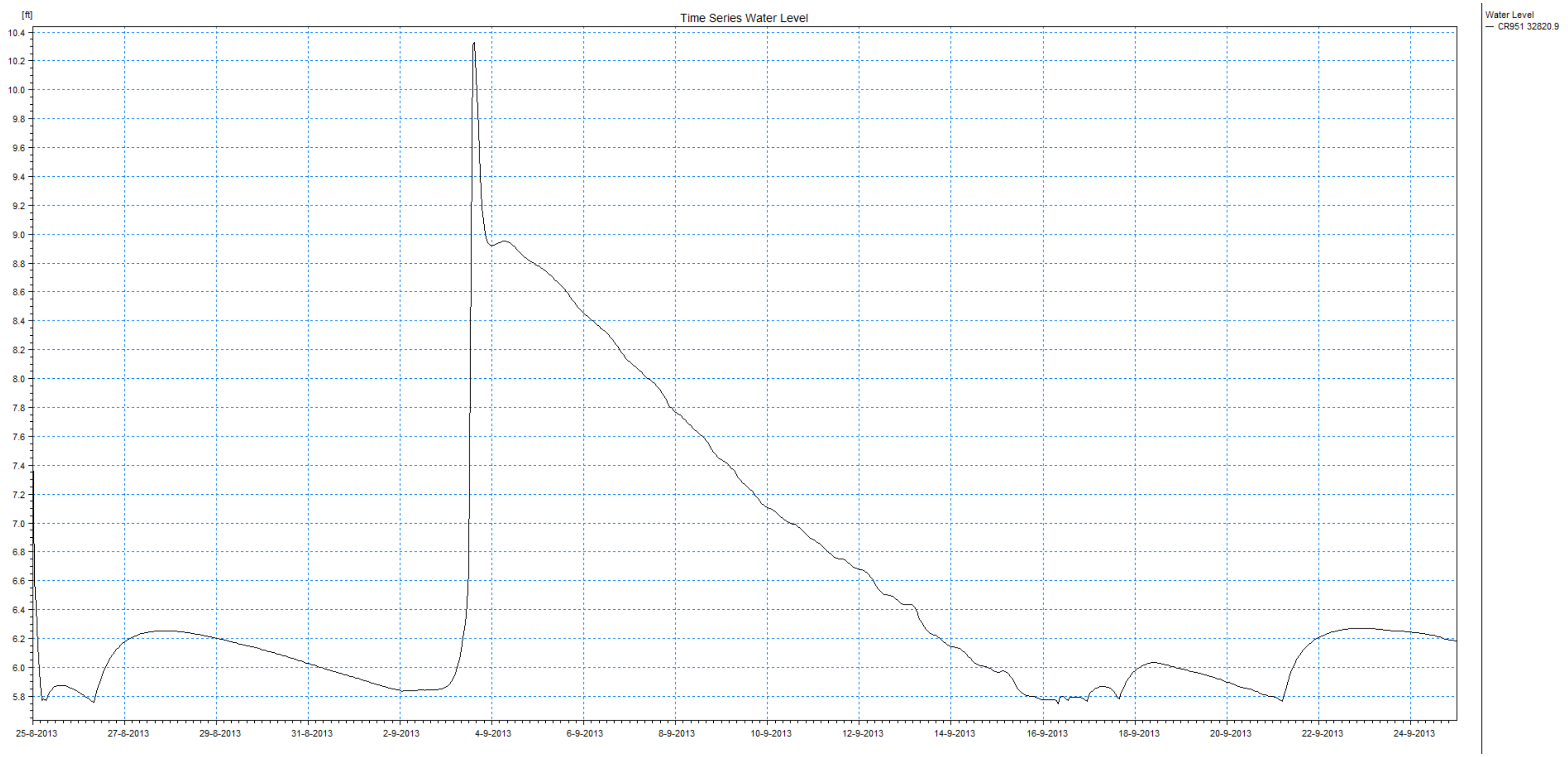

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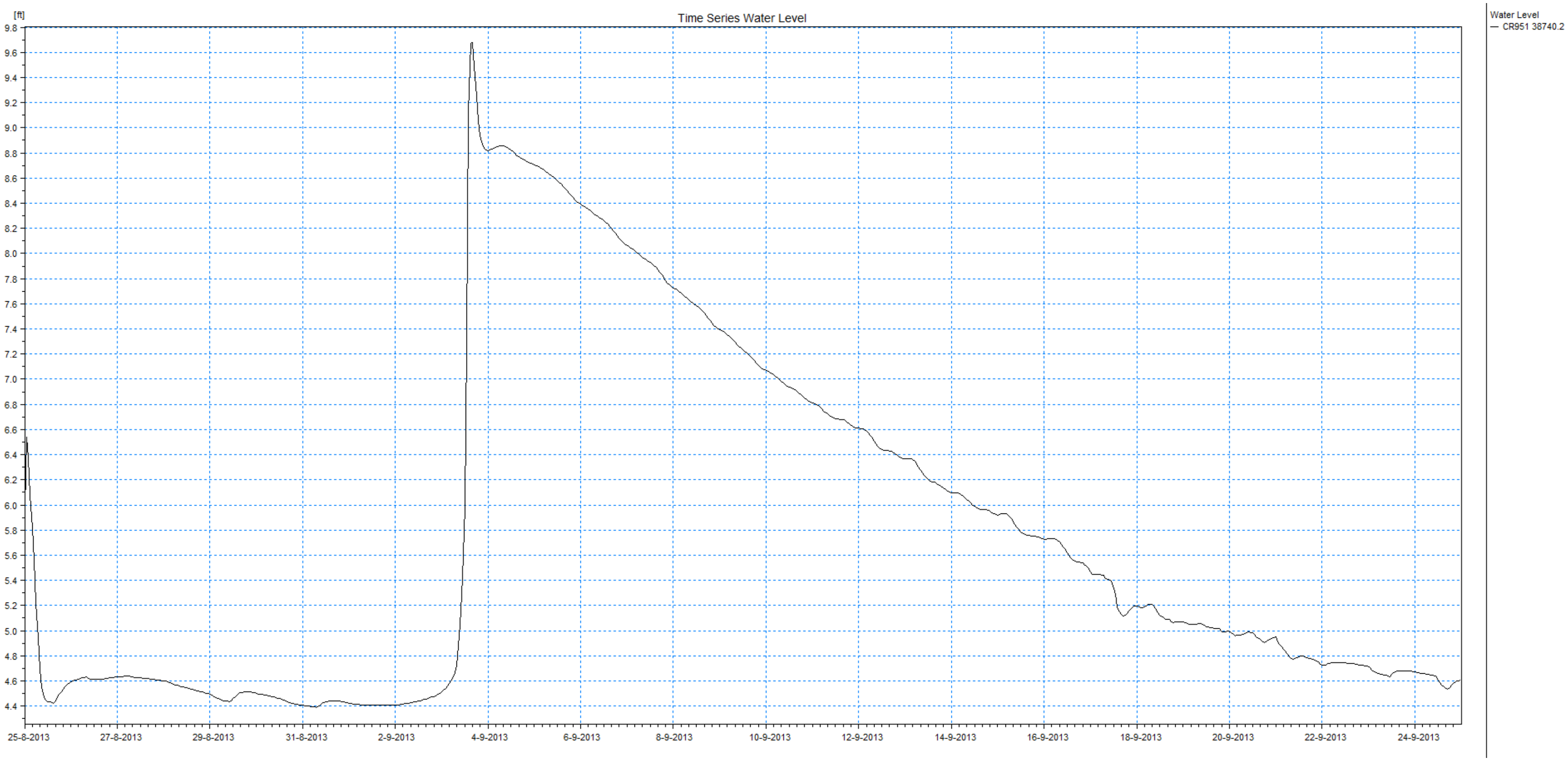

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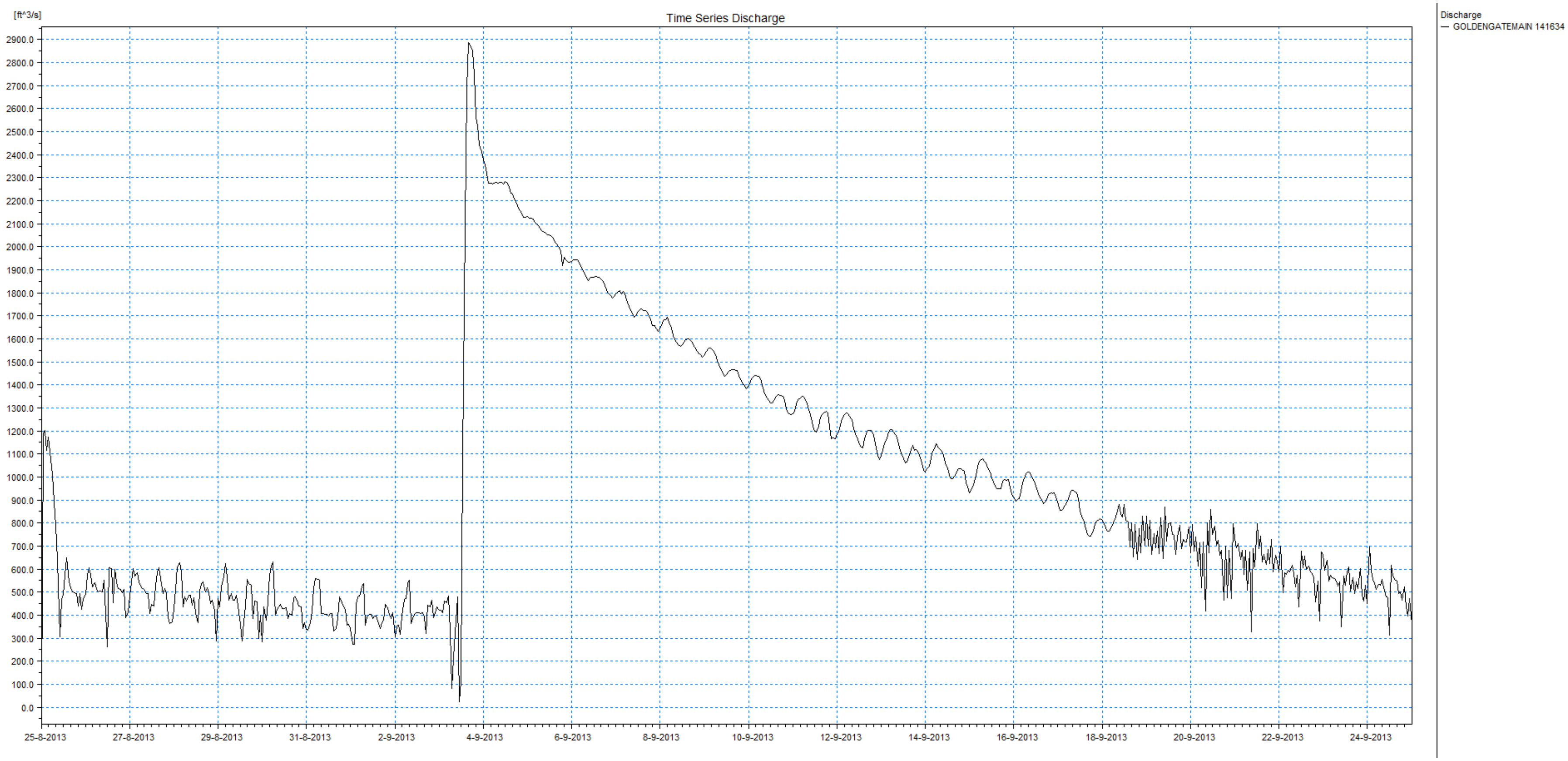

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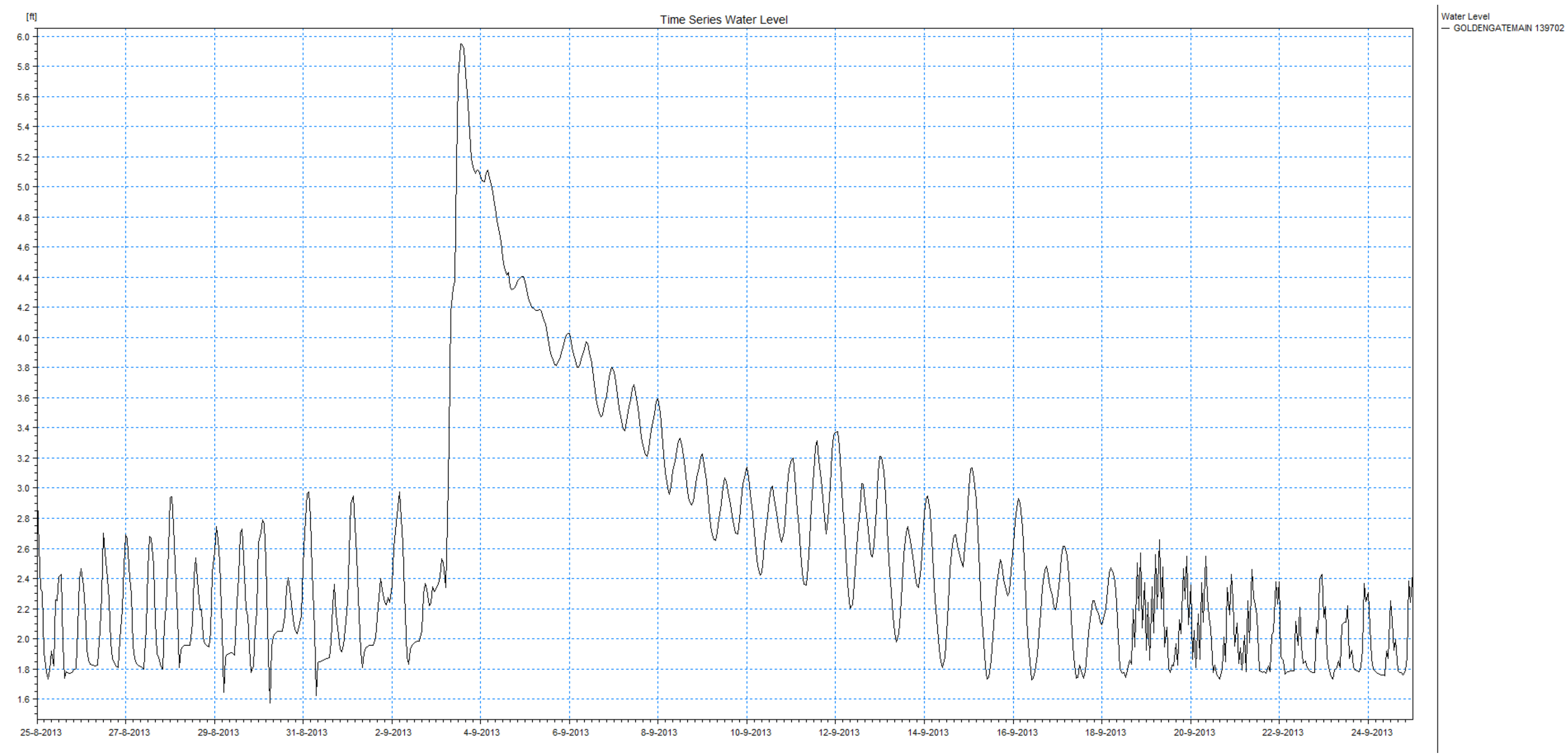

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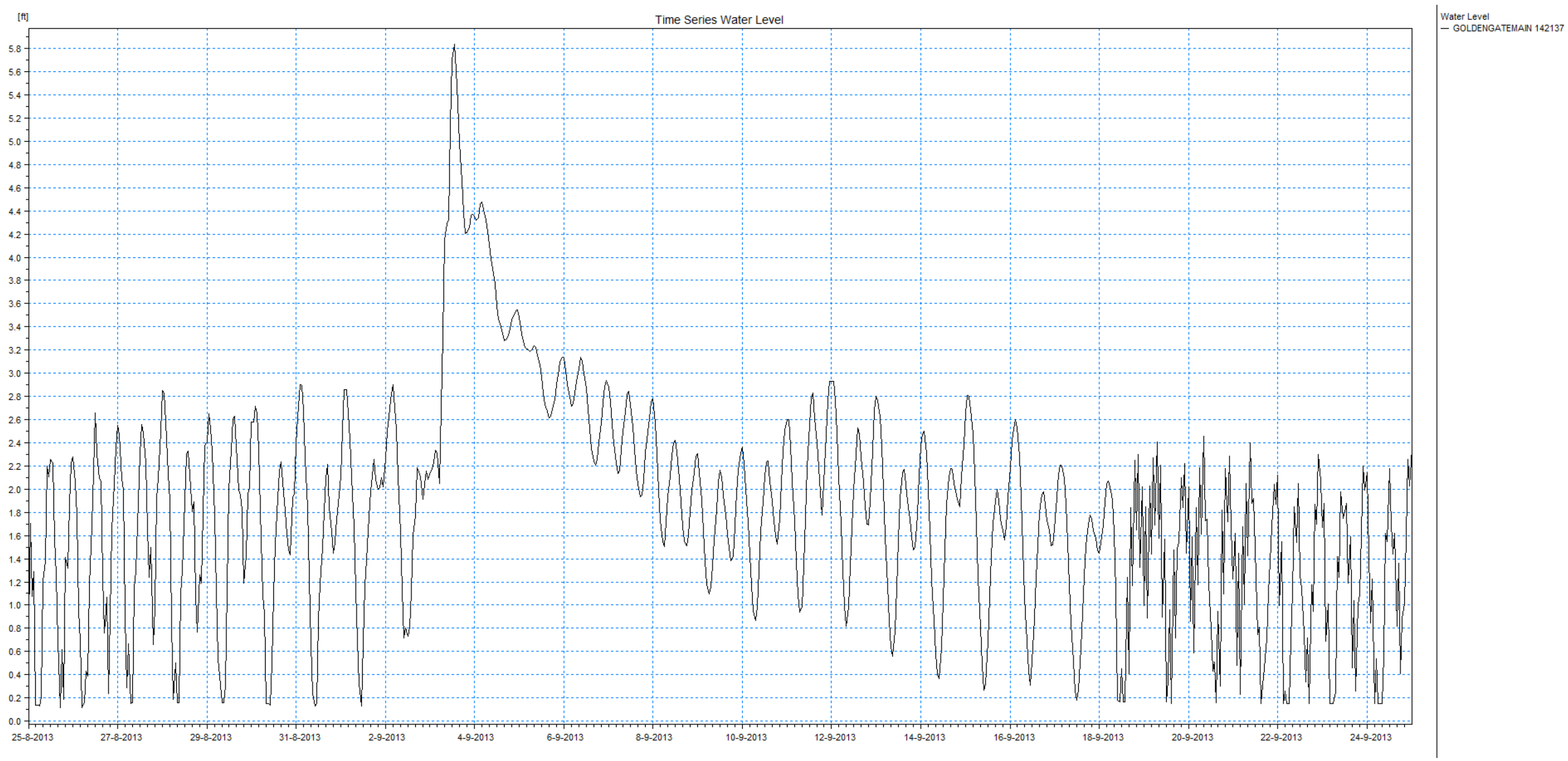

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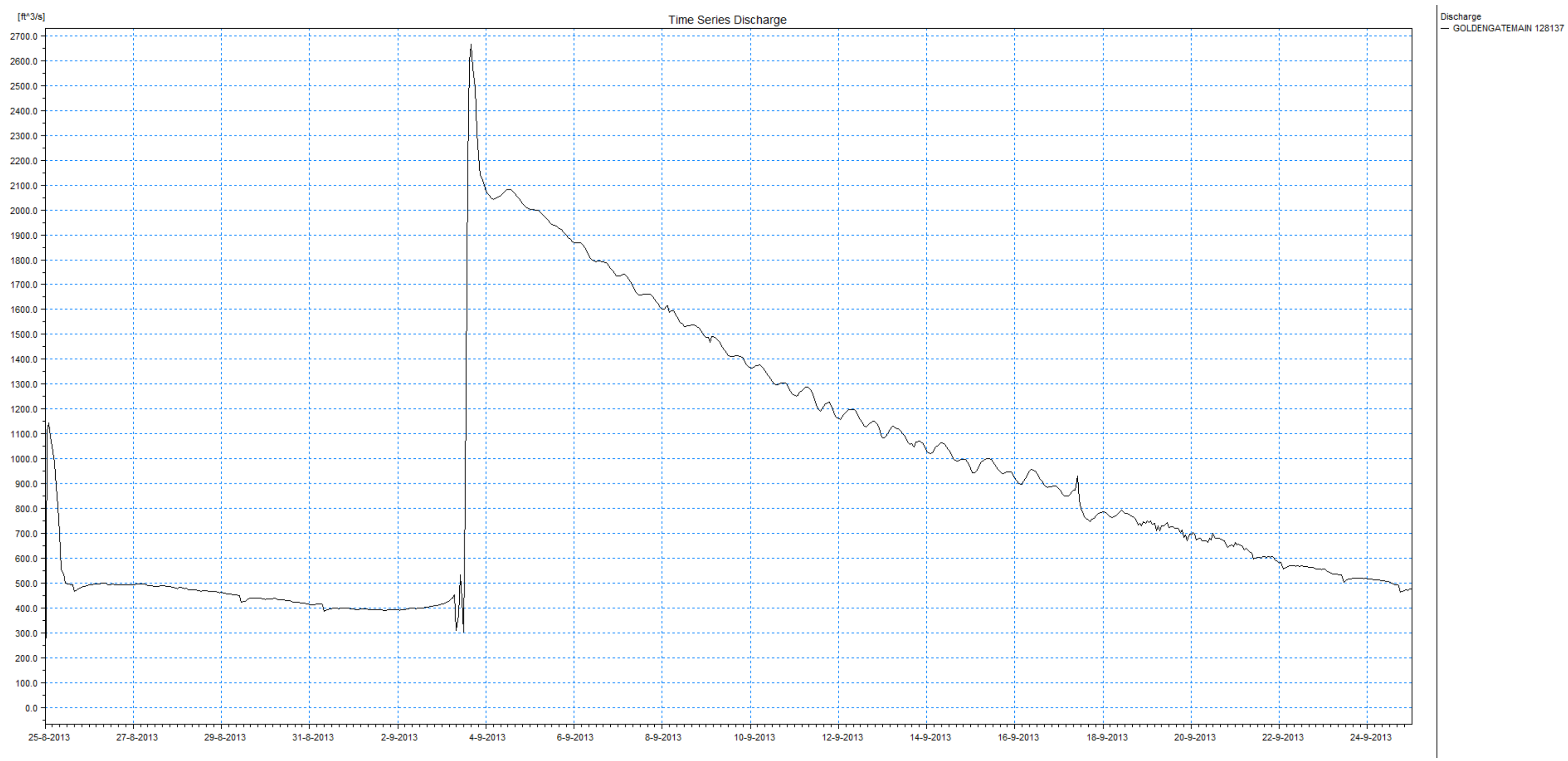
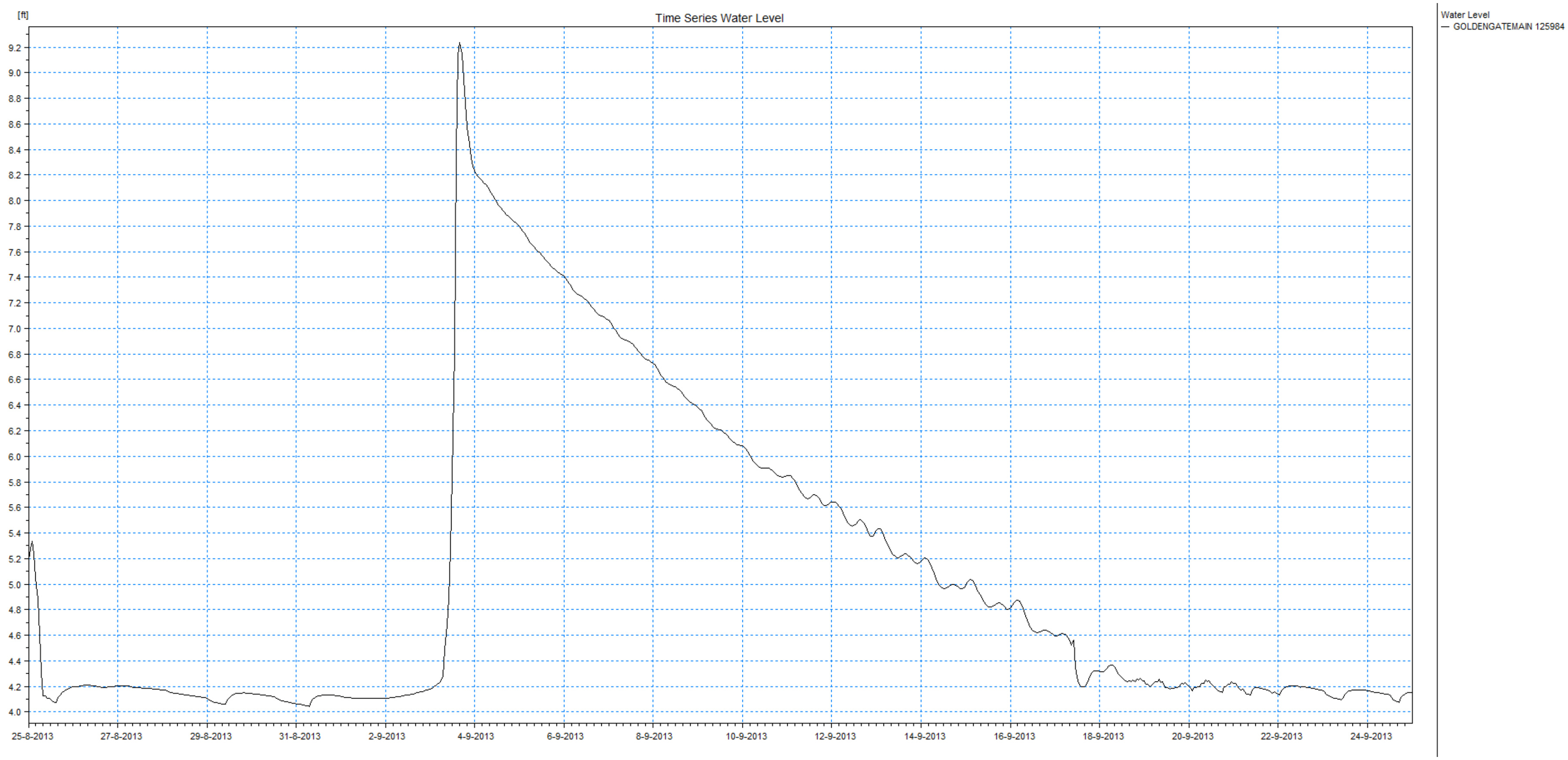


Figure 23. GG 2 Discharge (cfs)



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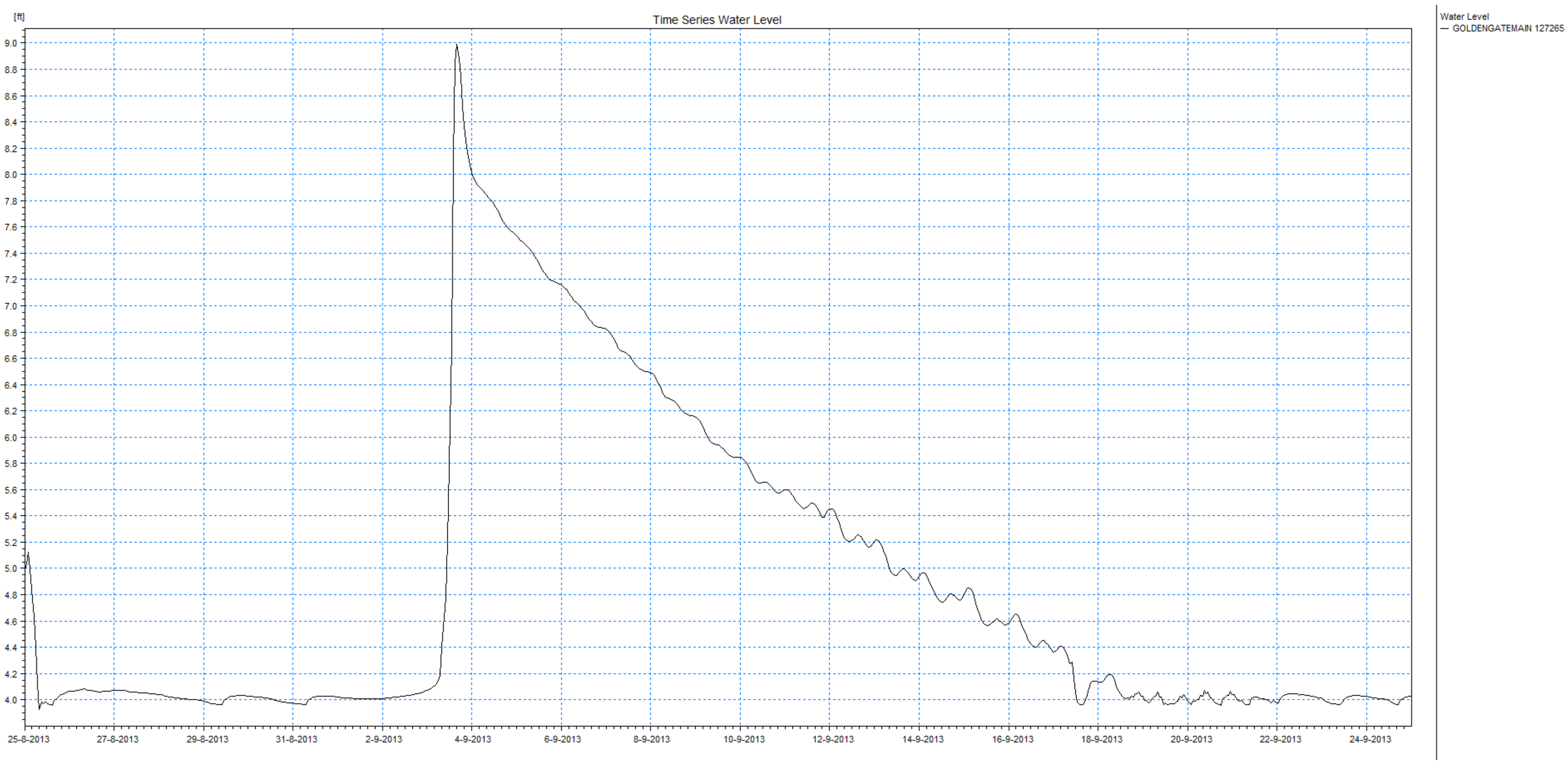

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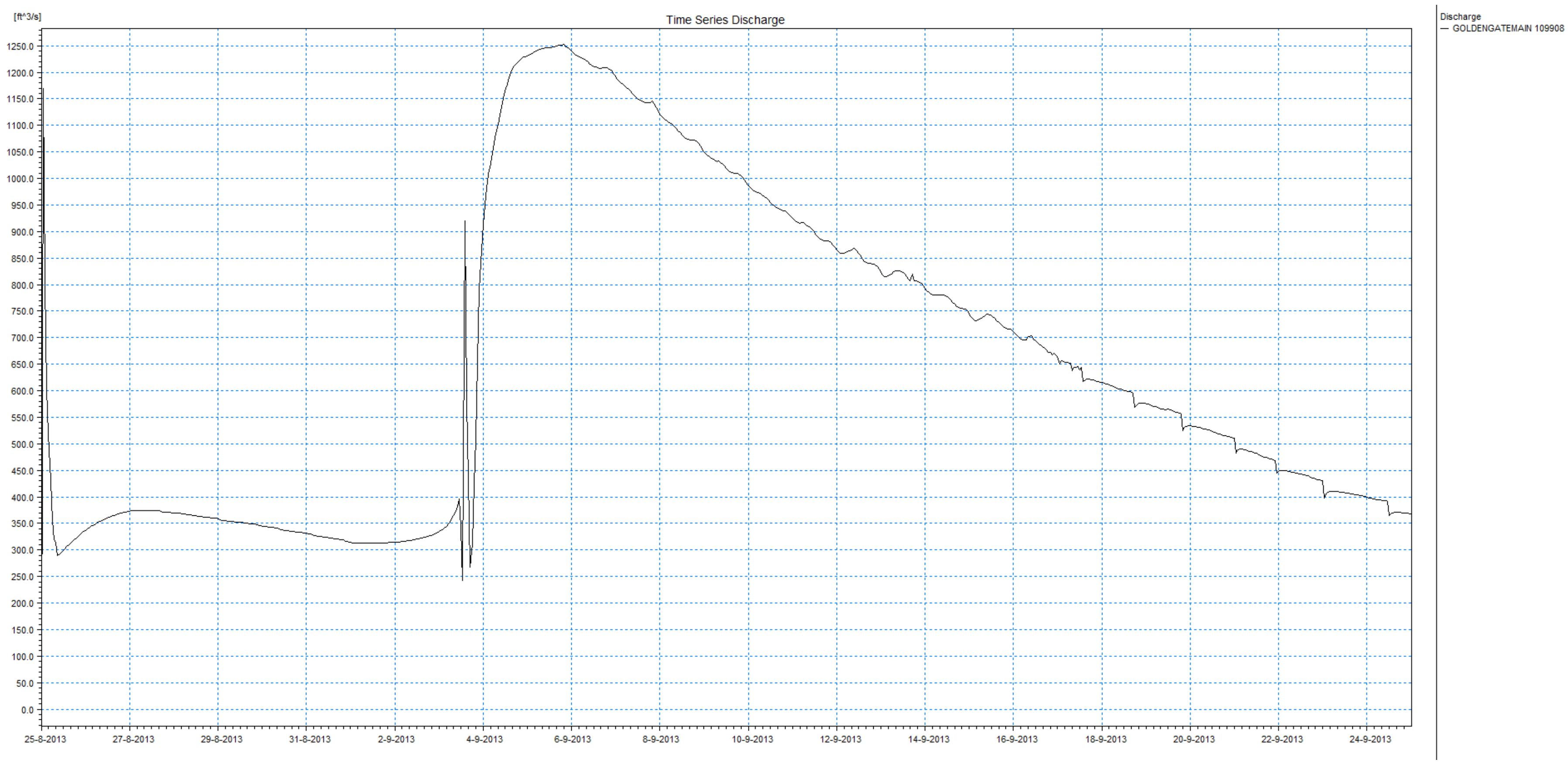


Figure 26. GG 3 Discharge (cfs)

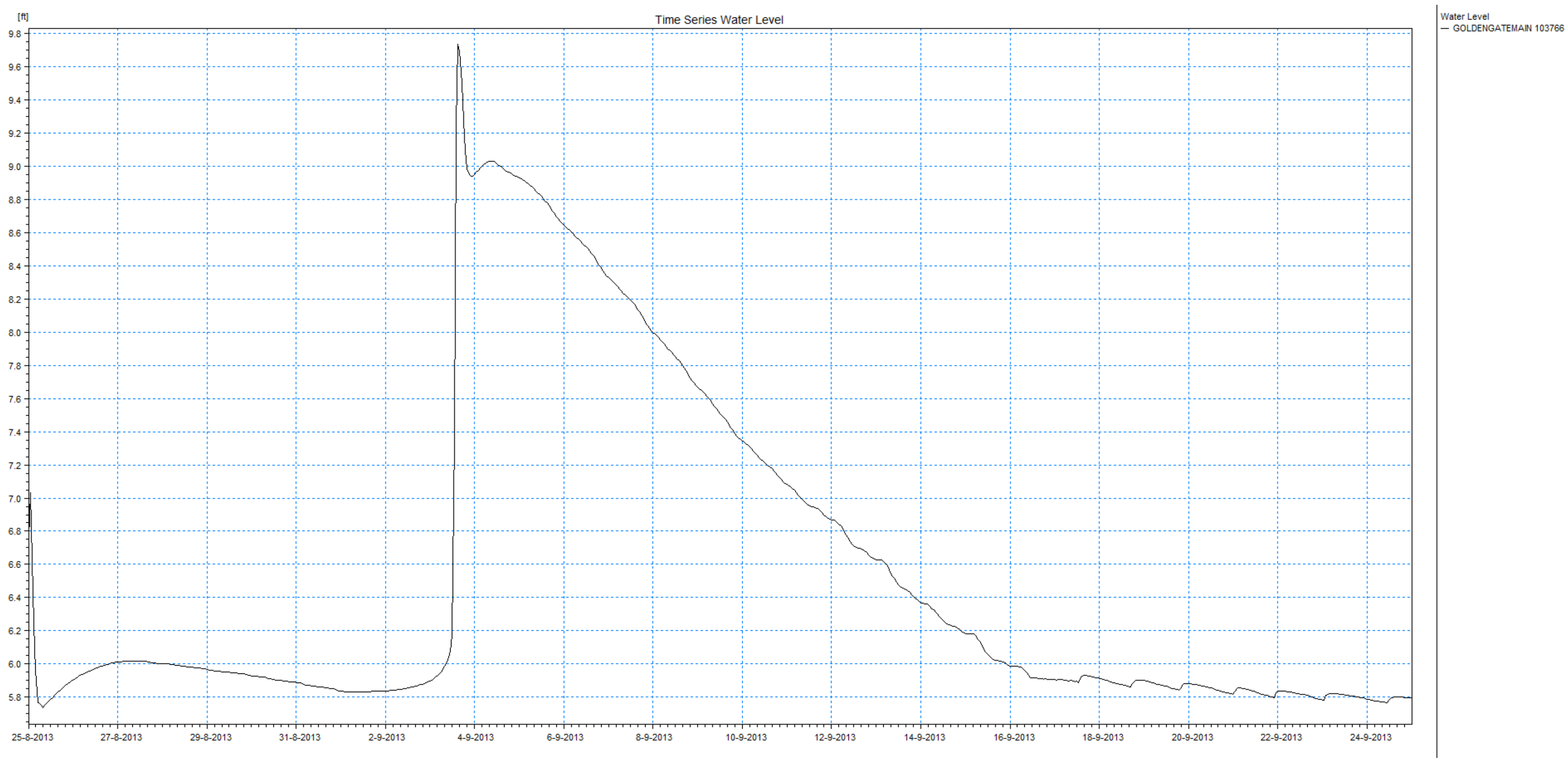
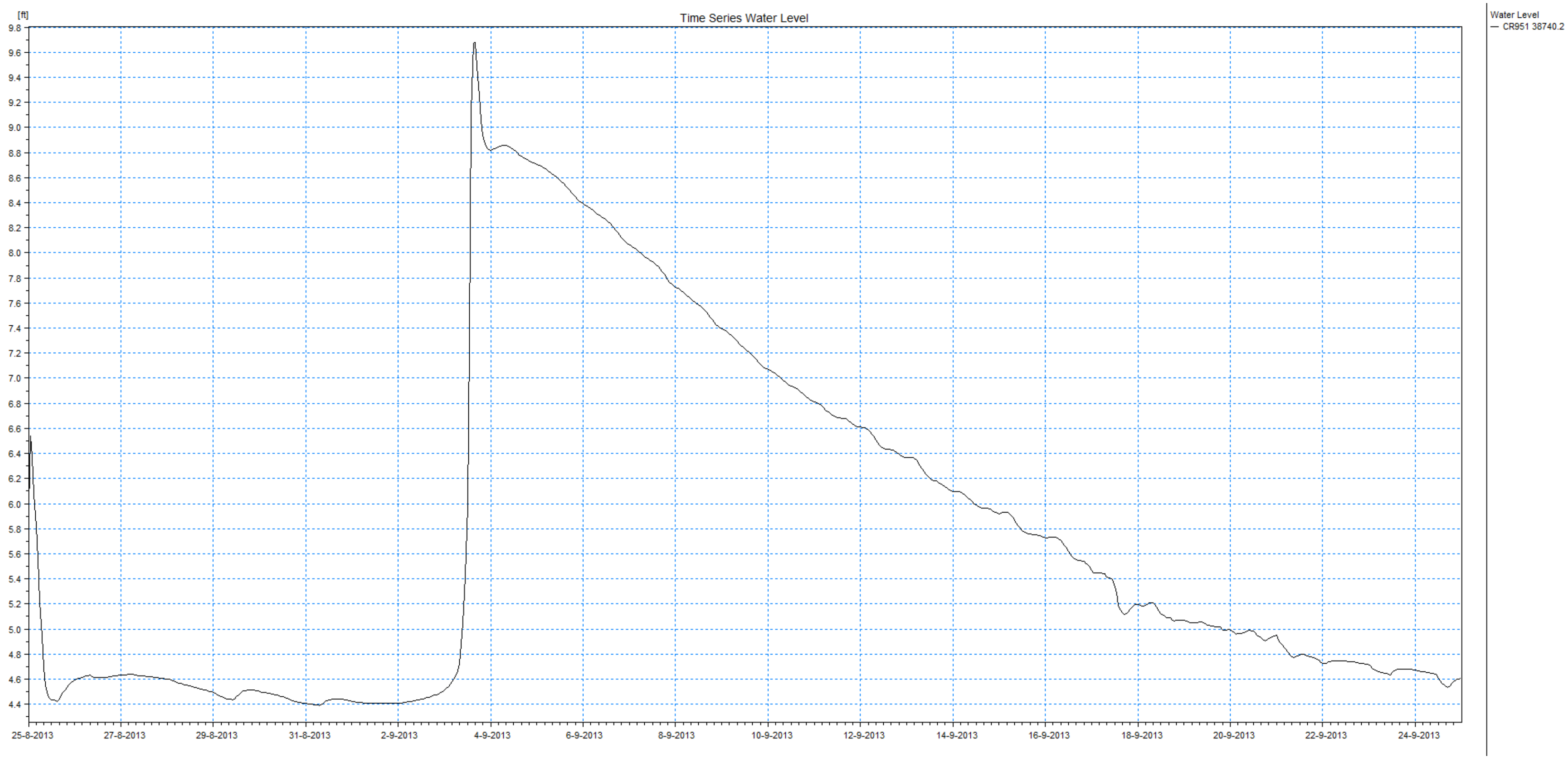


Figure 27. GG 3 Headwater Stage (ft)



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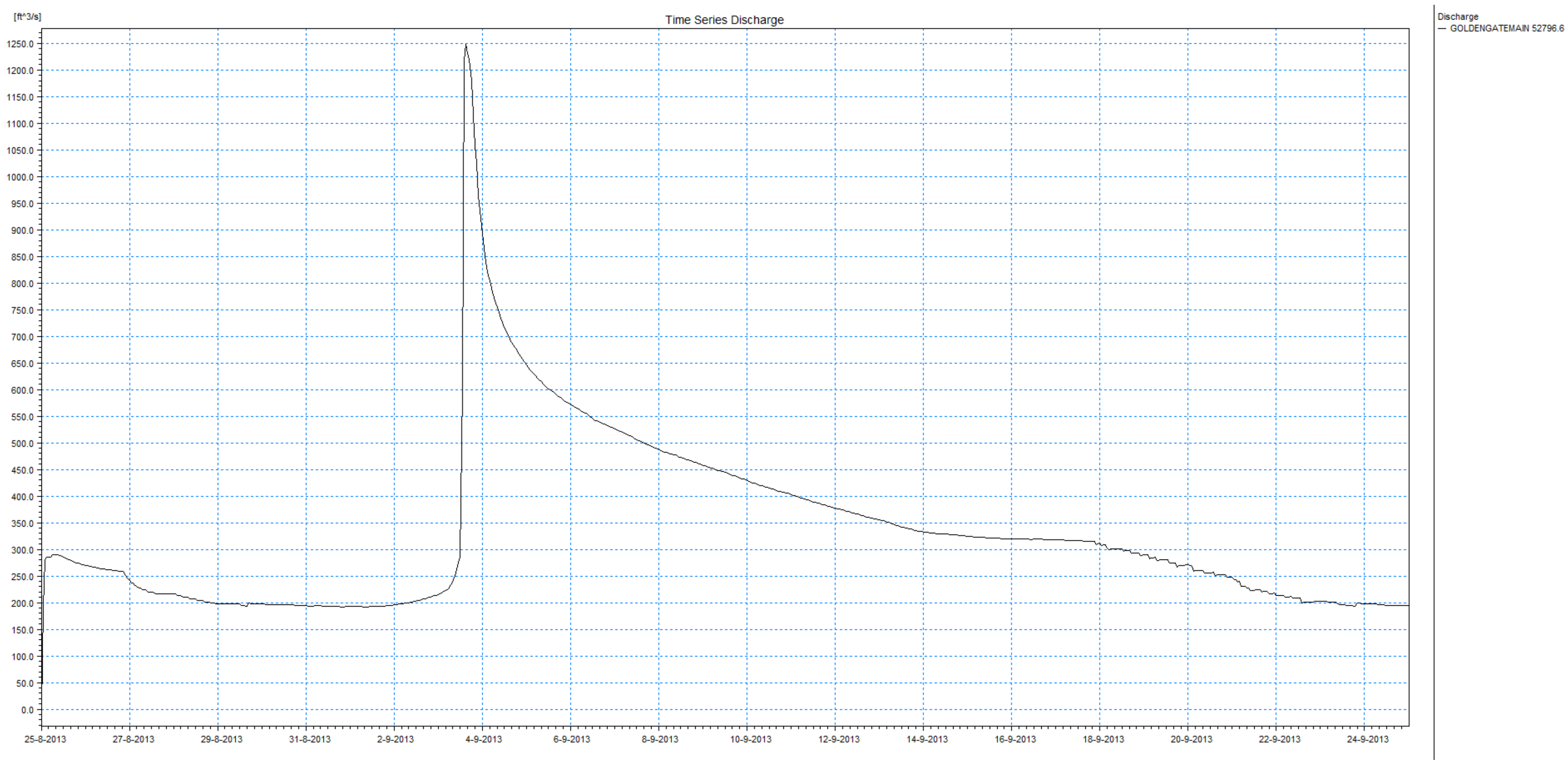

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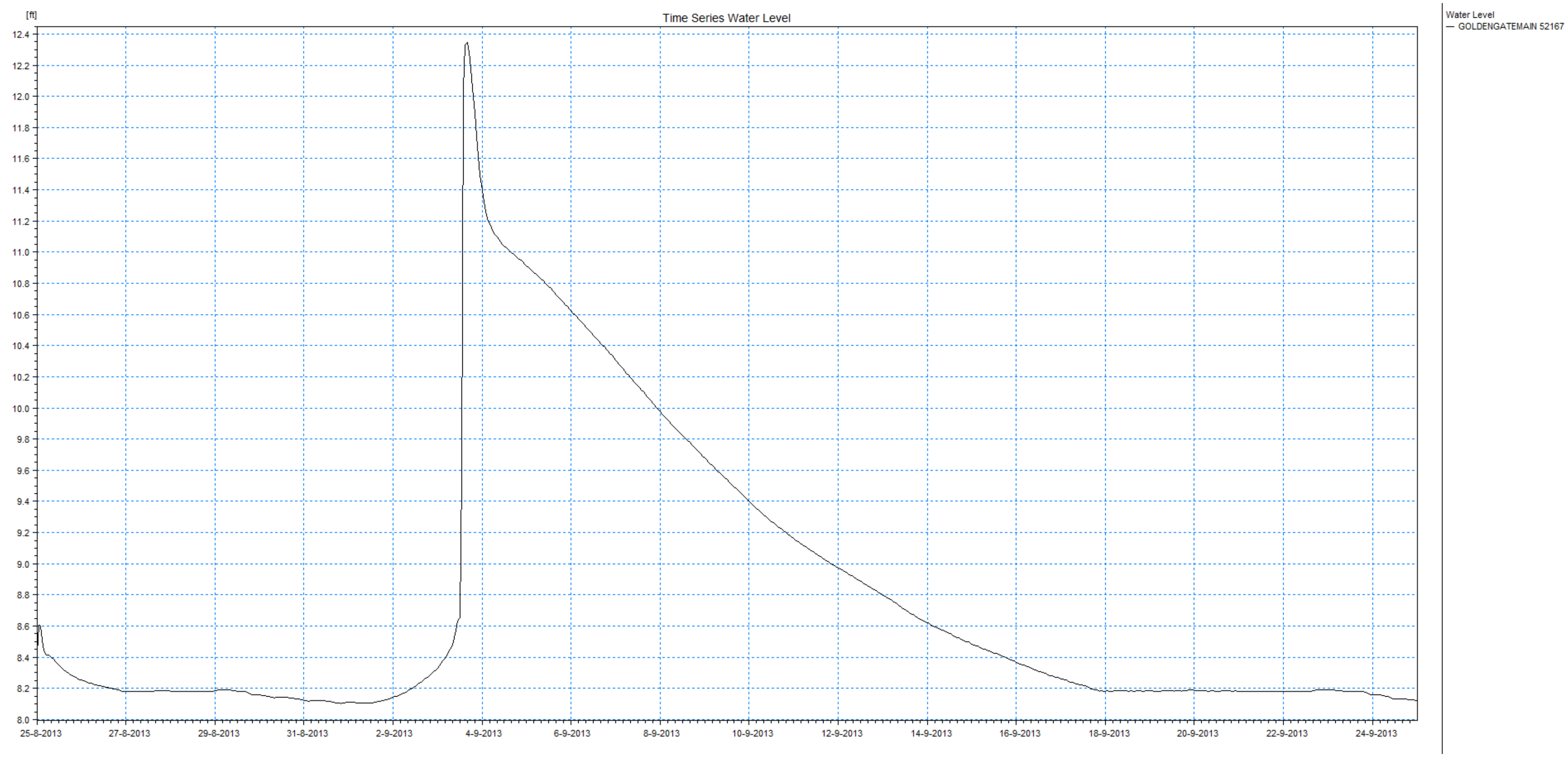

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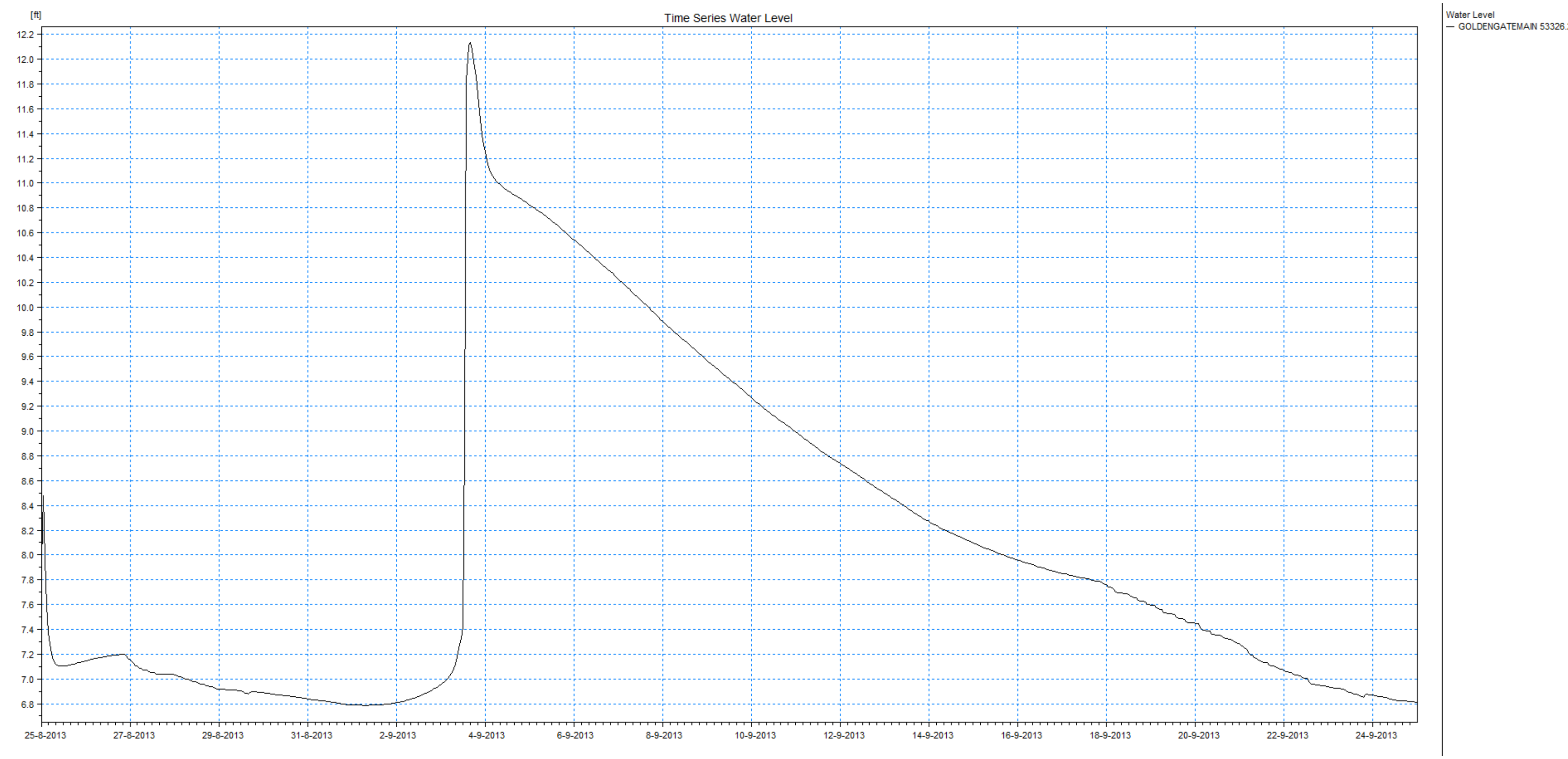


Figure 31. GG 4 Tailwater Stage (ft)

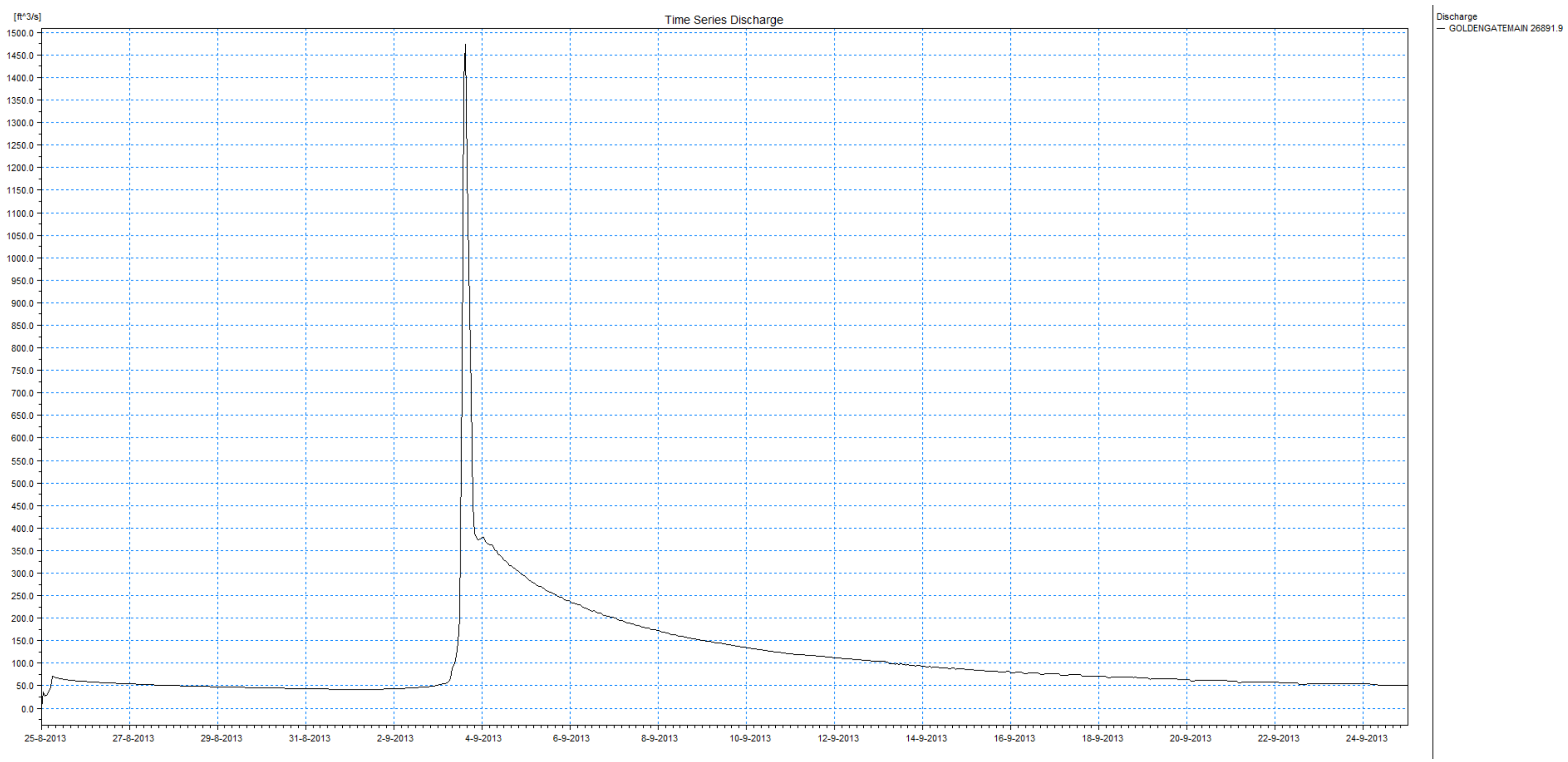


Figure 32. GG 5 Discharge (cfs)

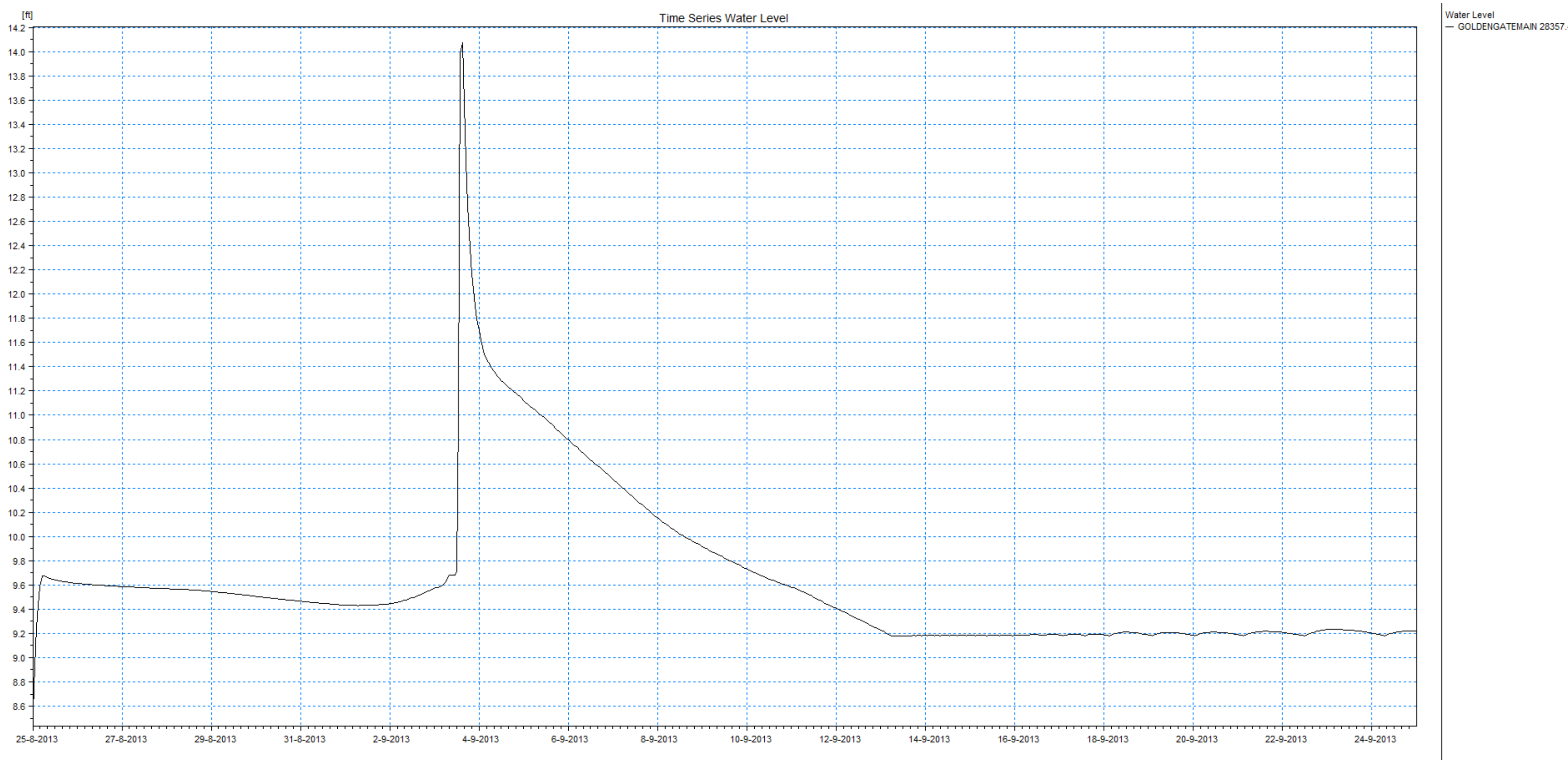
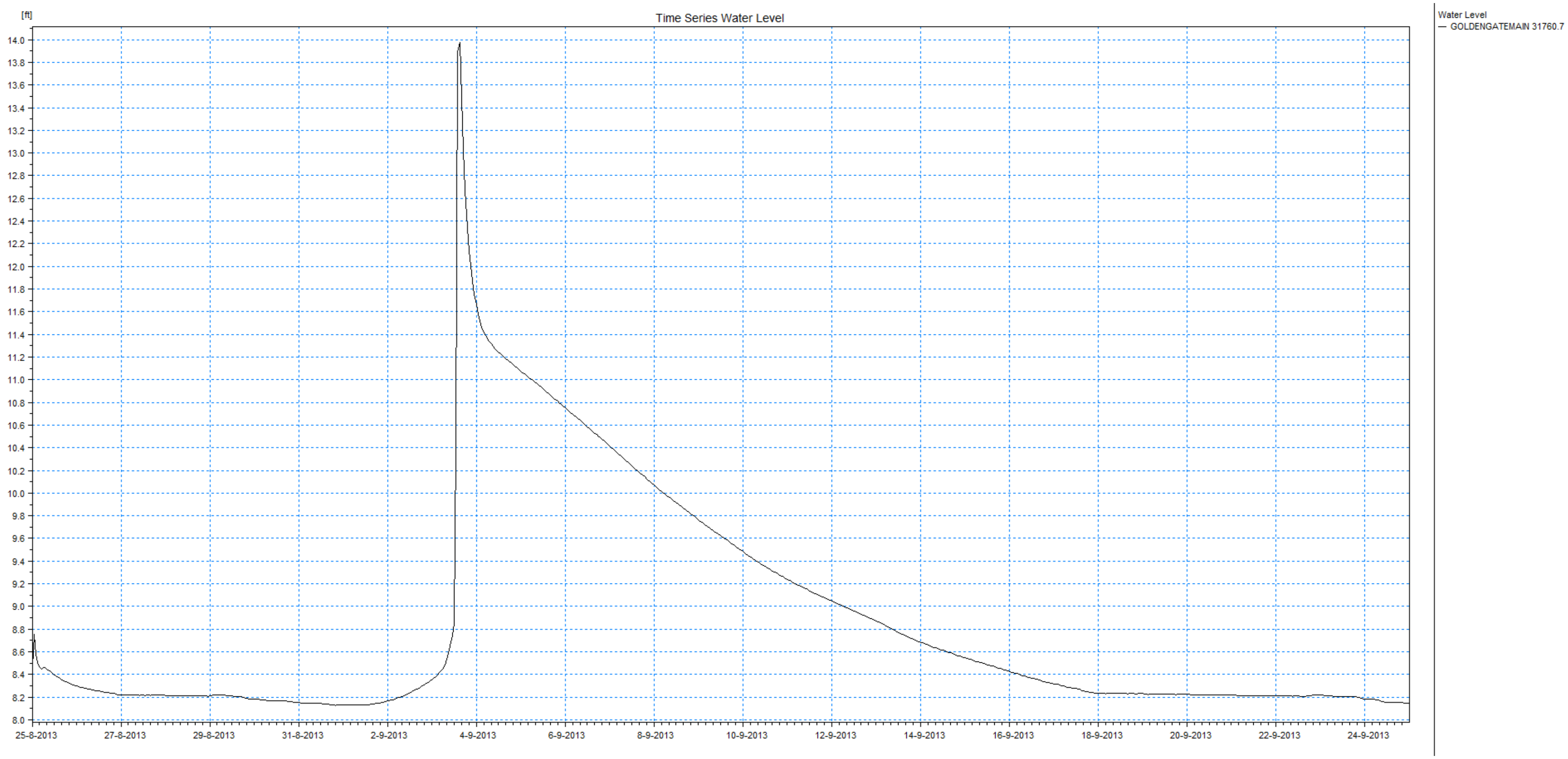


Figure 33. GG 5 Headwater Stage (ft)



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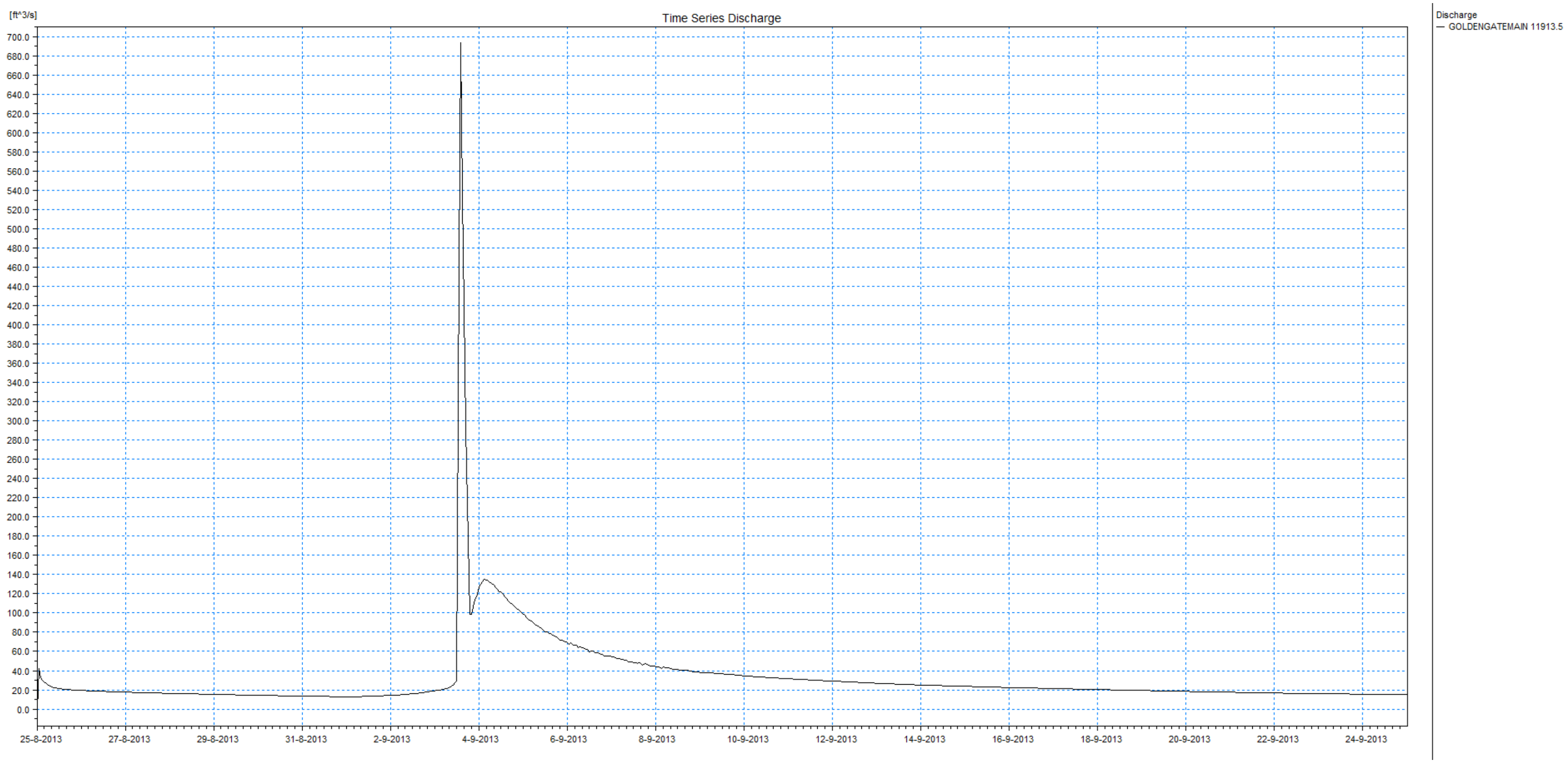


Figure 35. GG 6 Discharge (cfs)

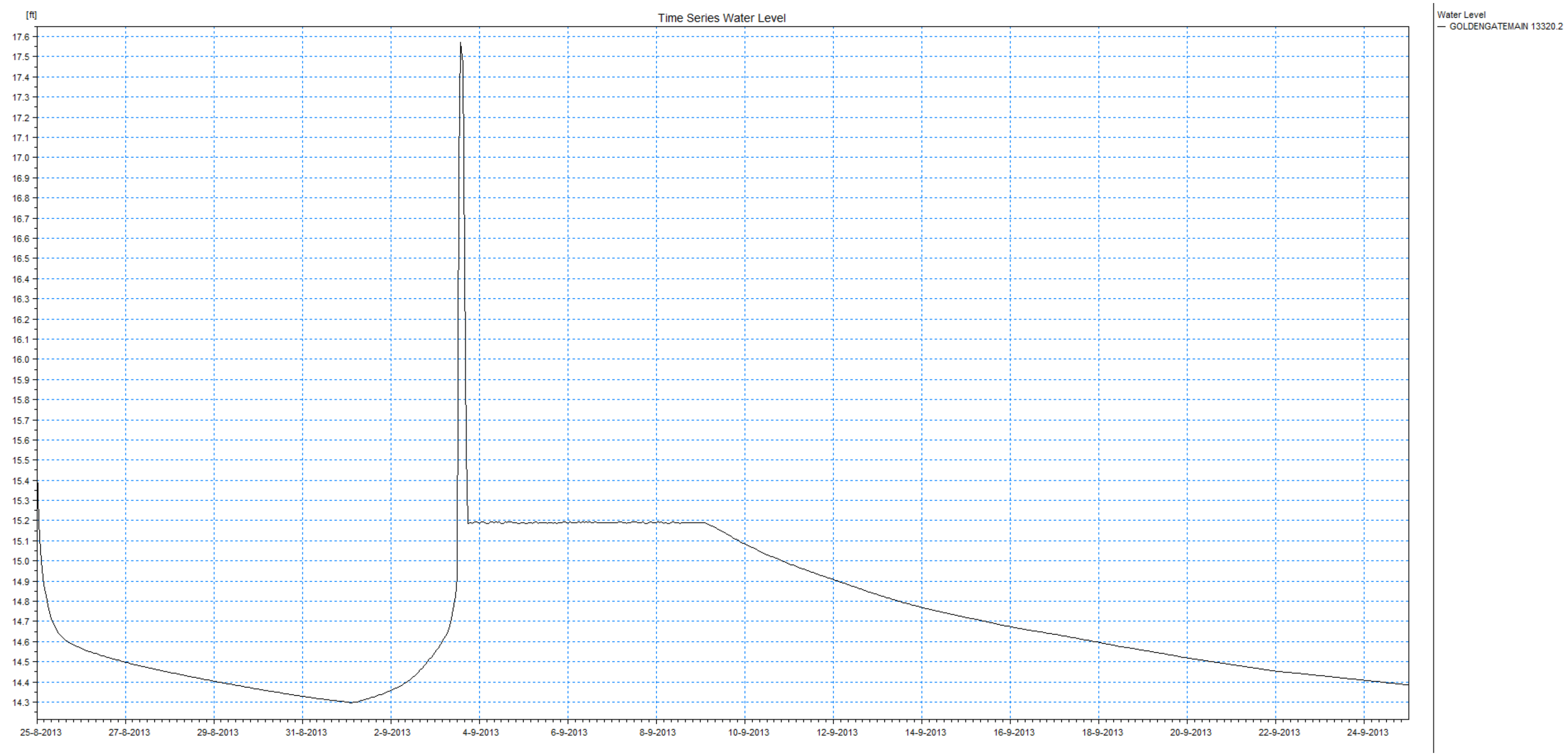
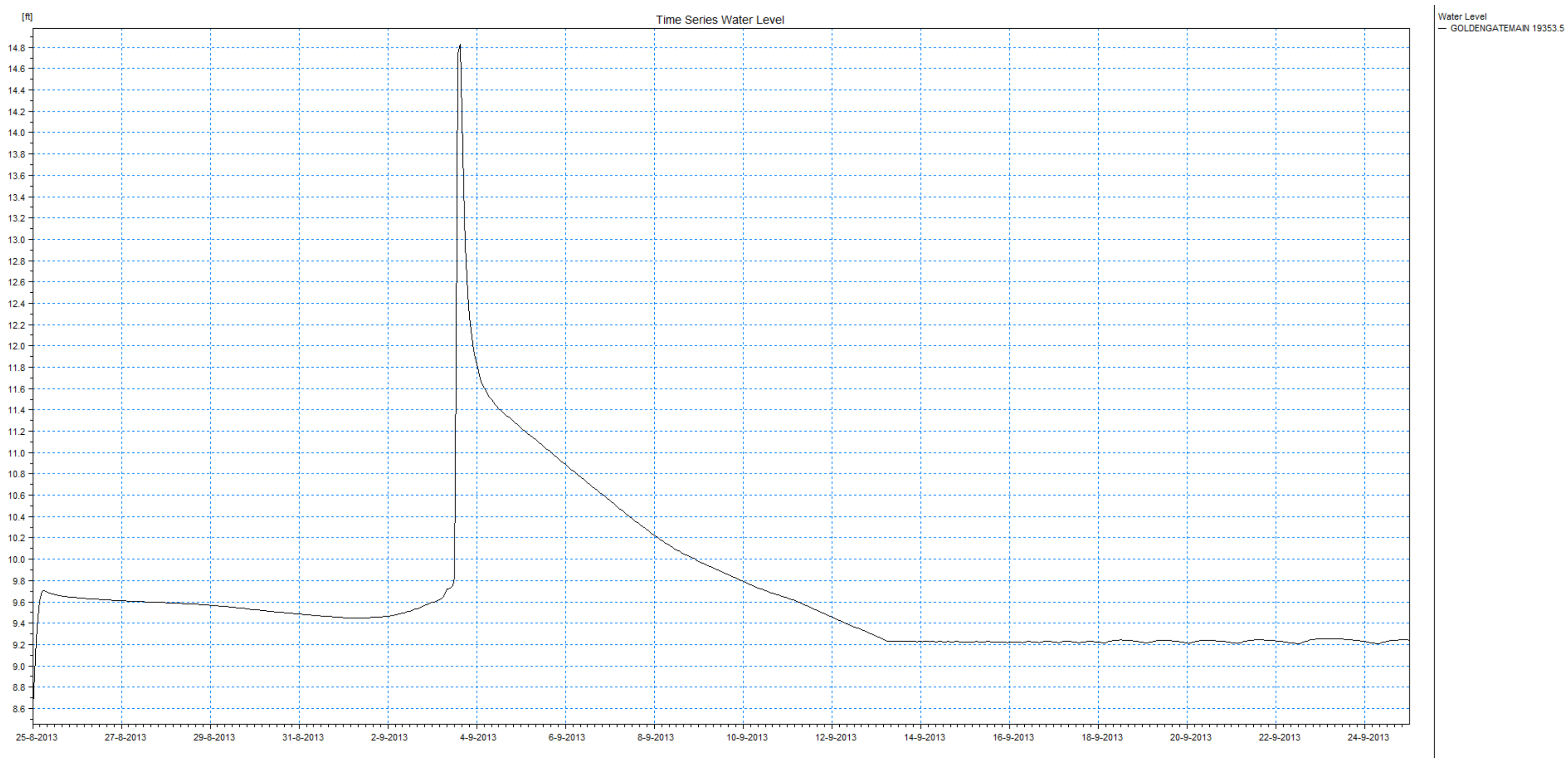


Figure 36. GG 6 Headwater Stage (ft)



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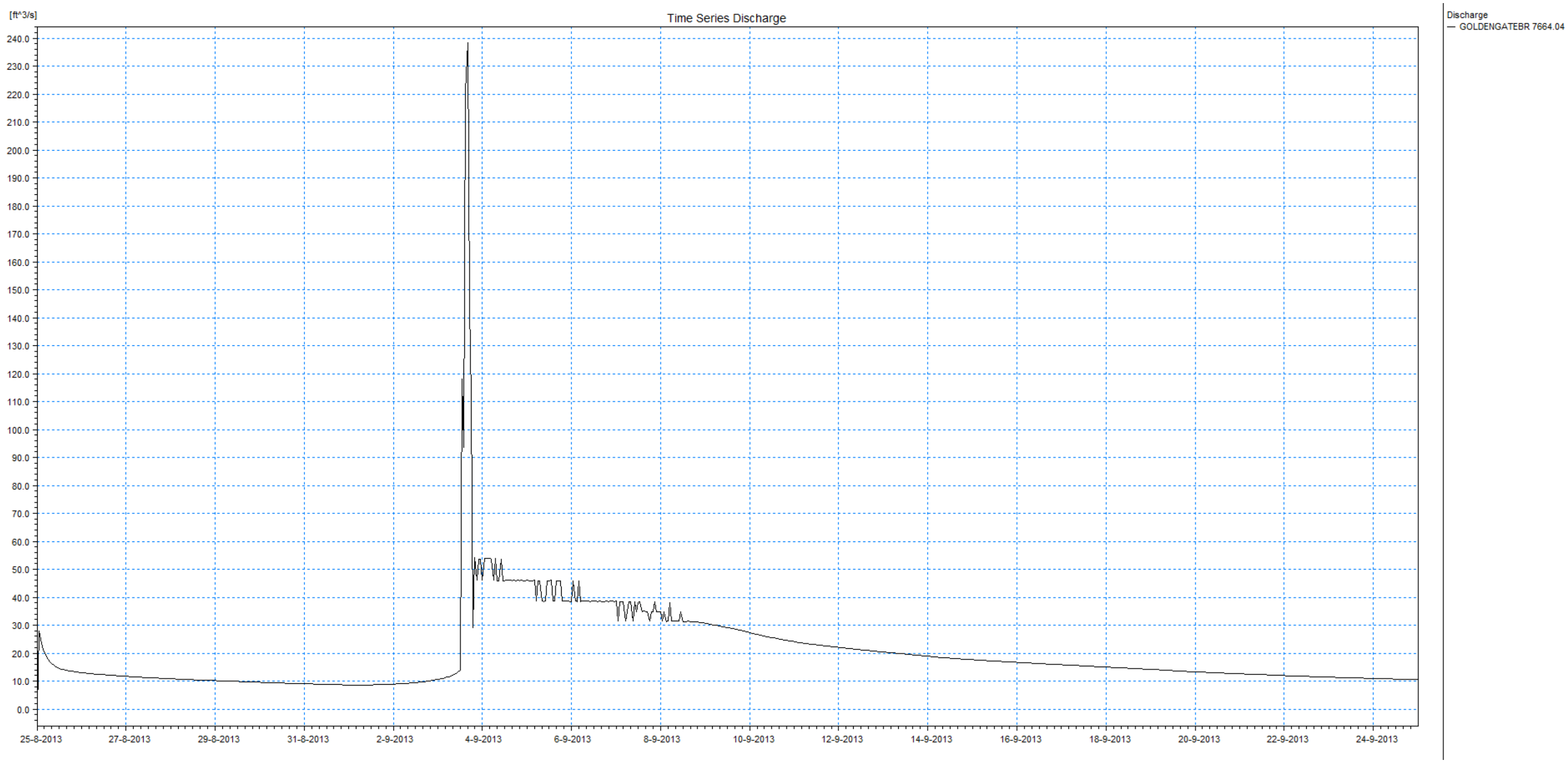


Figure 38. GG 7 Discharge (cfs)

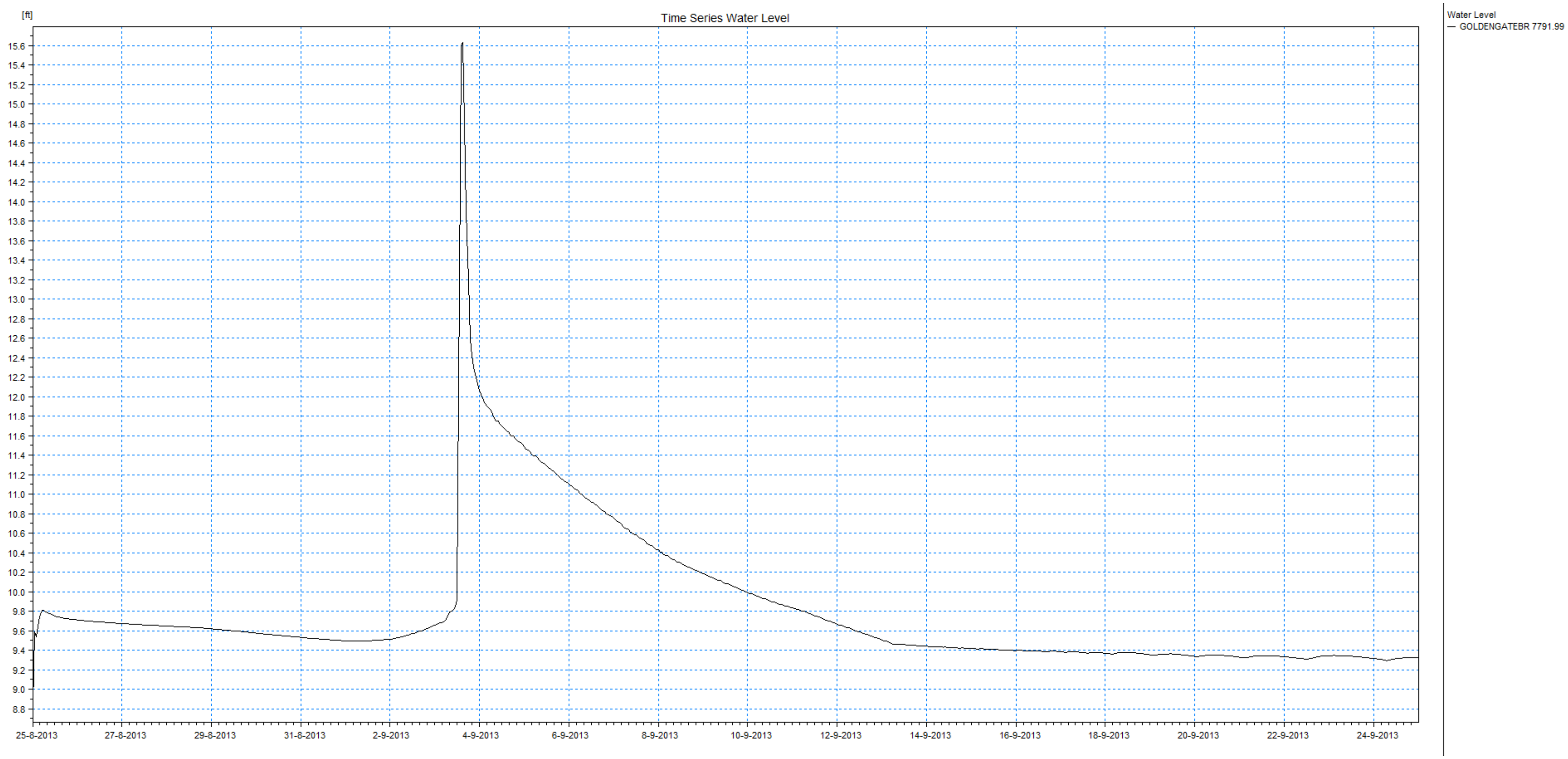


Figure 39. GG 7 Headwater Stage (ft)

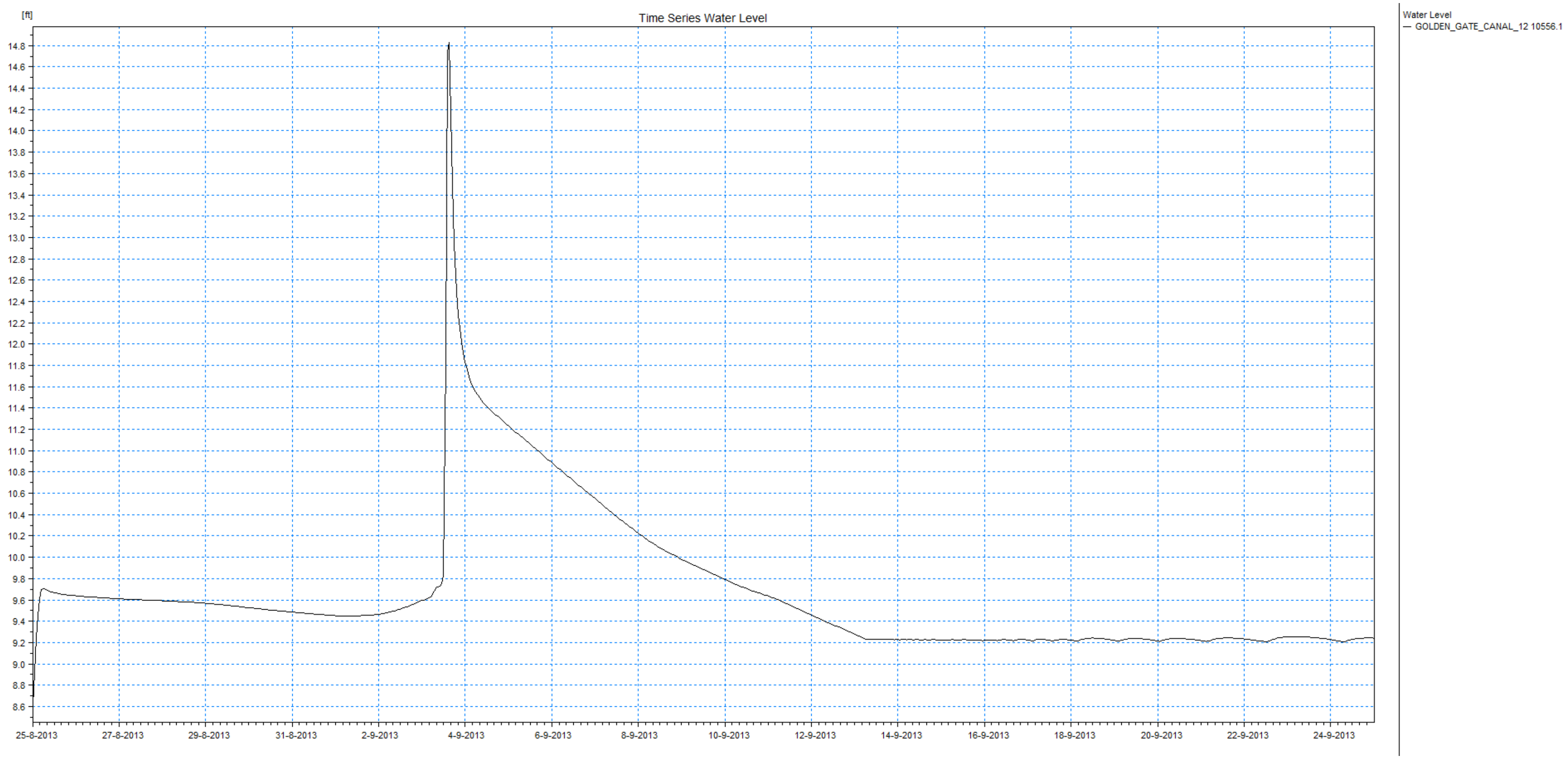


Figure 40. GG 7 Tailwater Stage (ft)

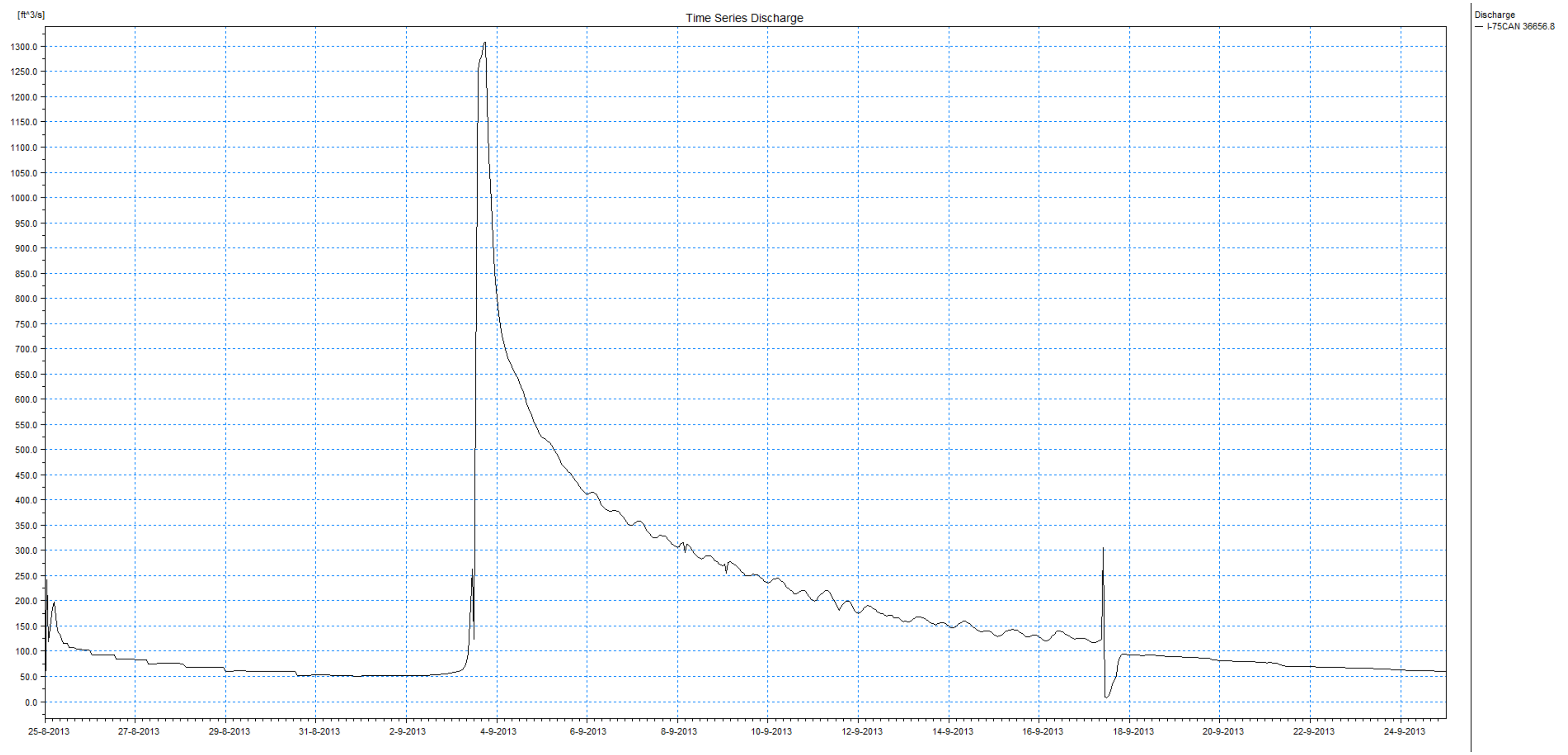
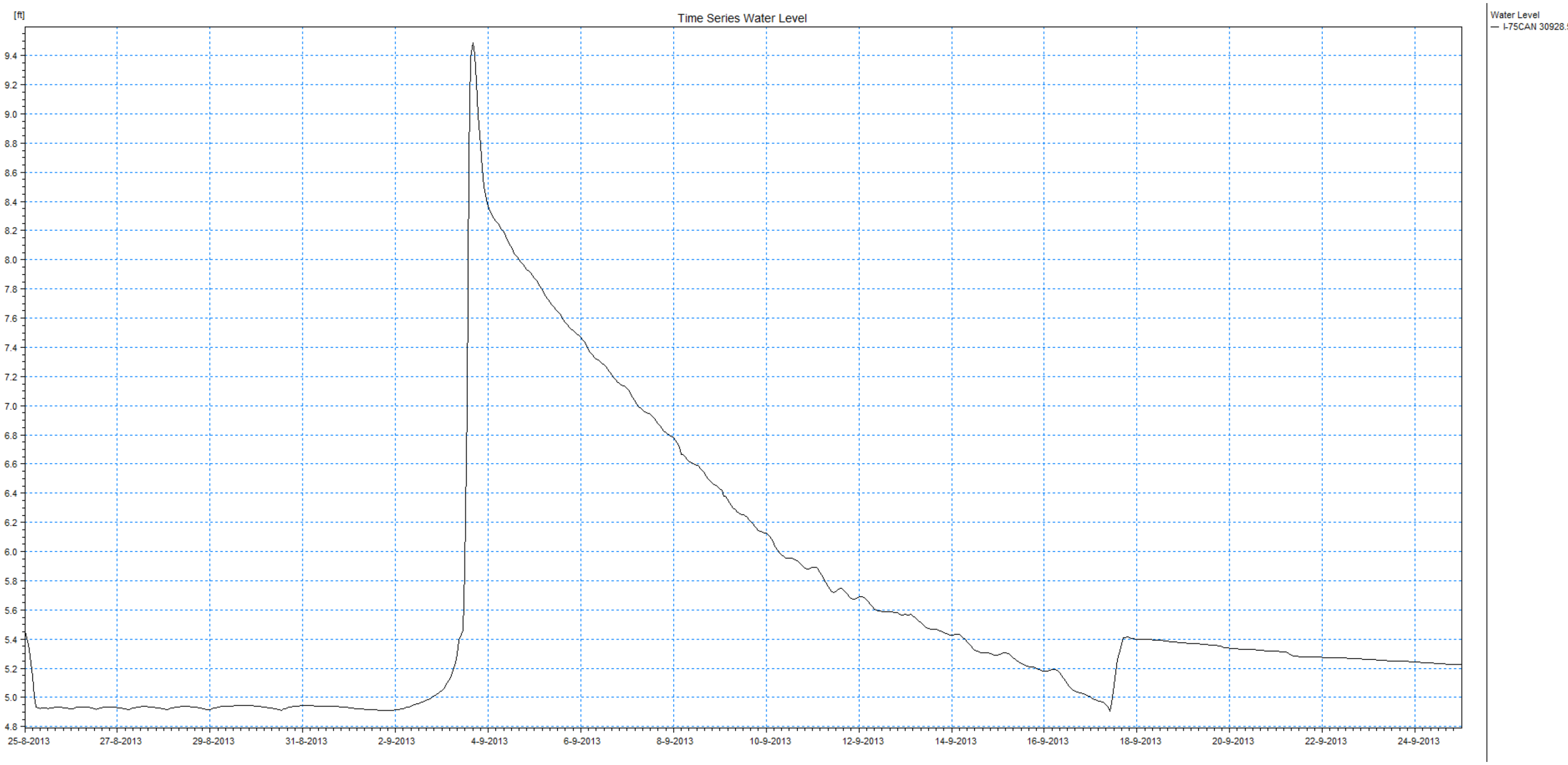


Figure 41. I75W1 Discharge (cfs)



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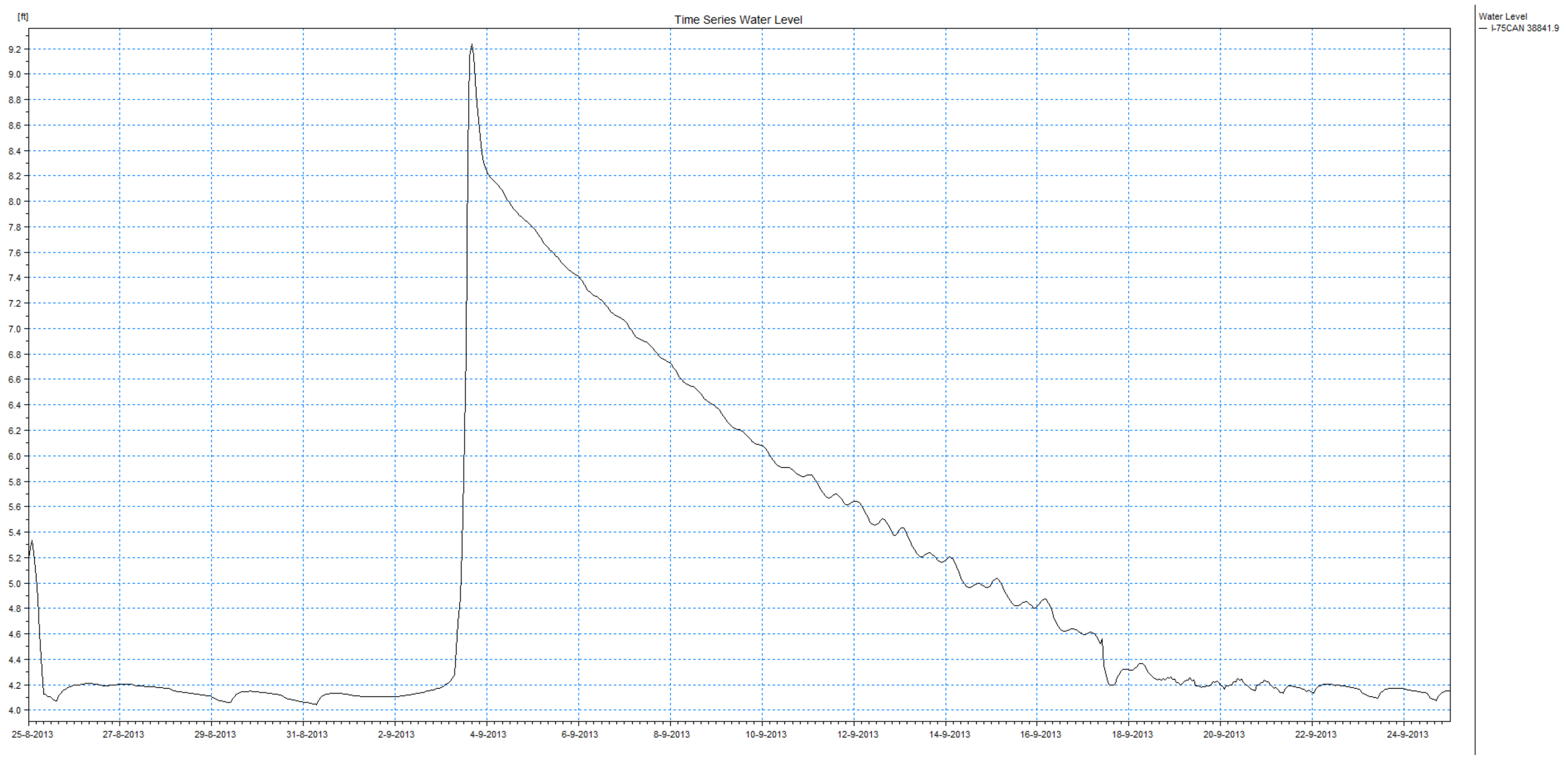

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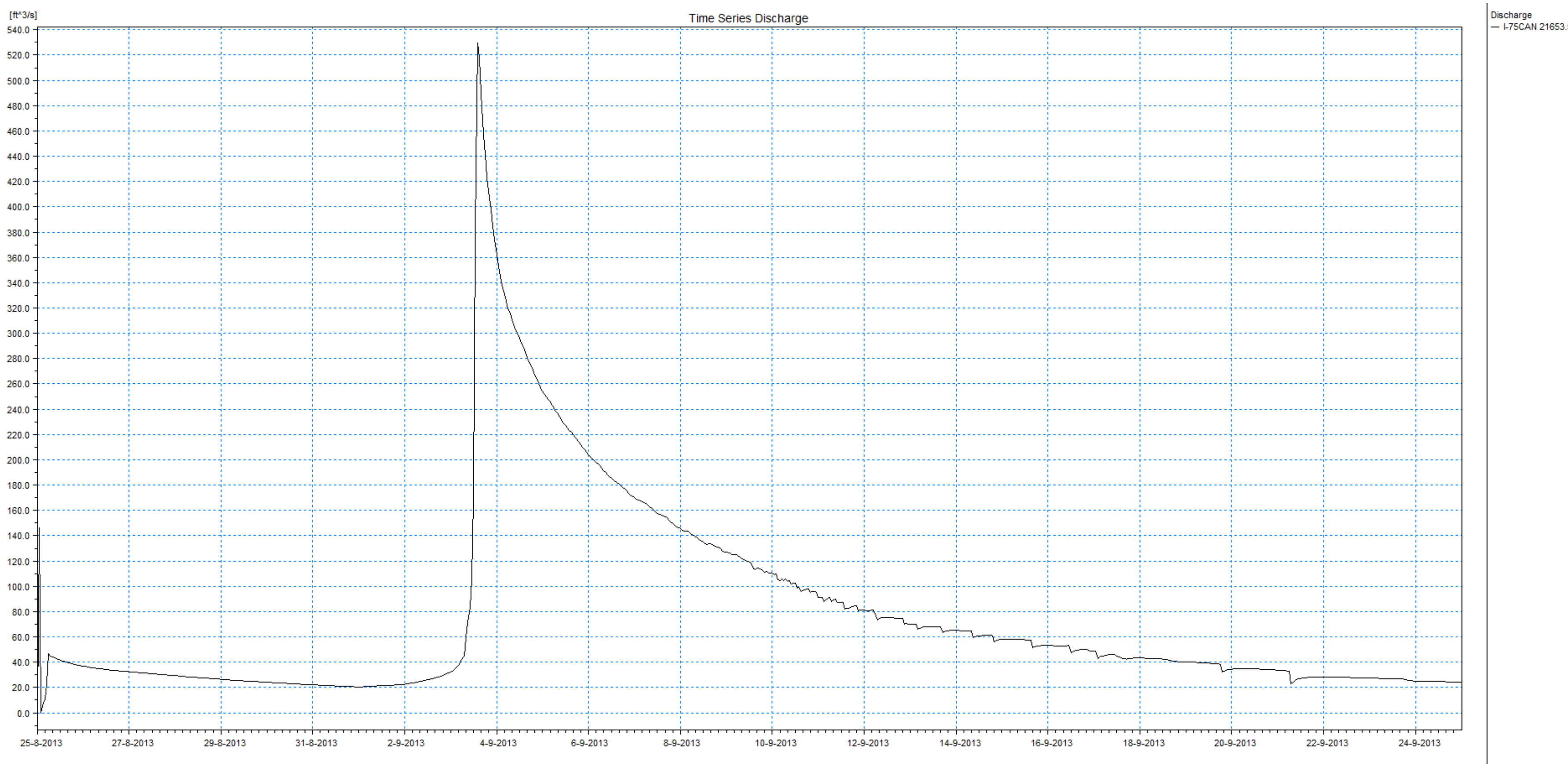

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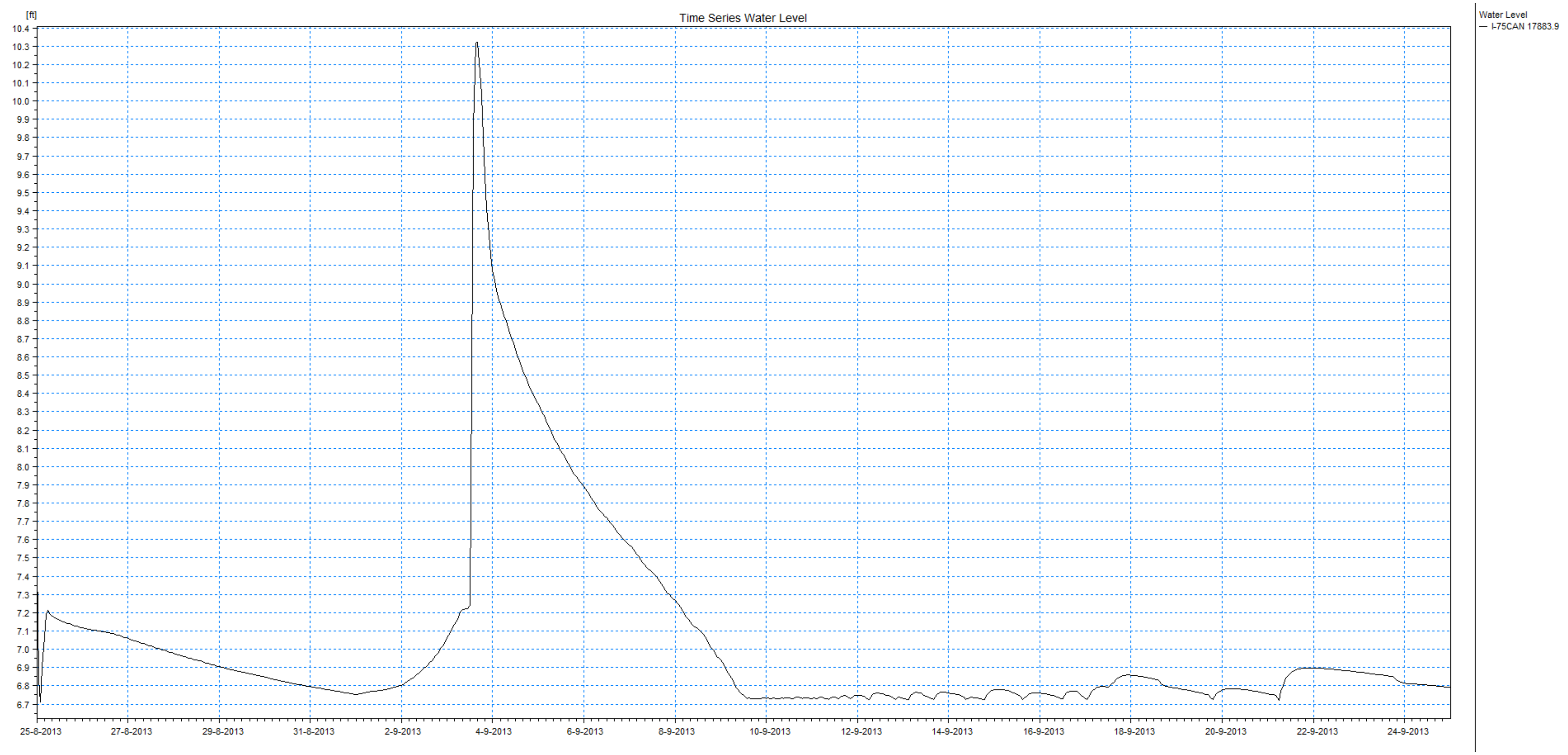


Figure 45. I75W2 Headwater Stage (ft)

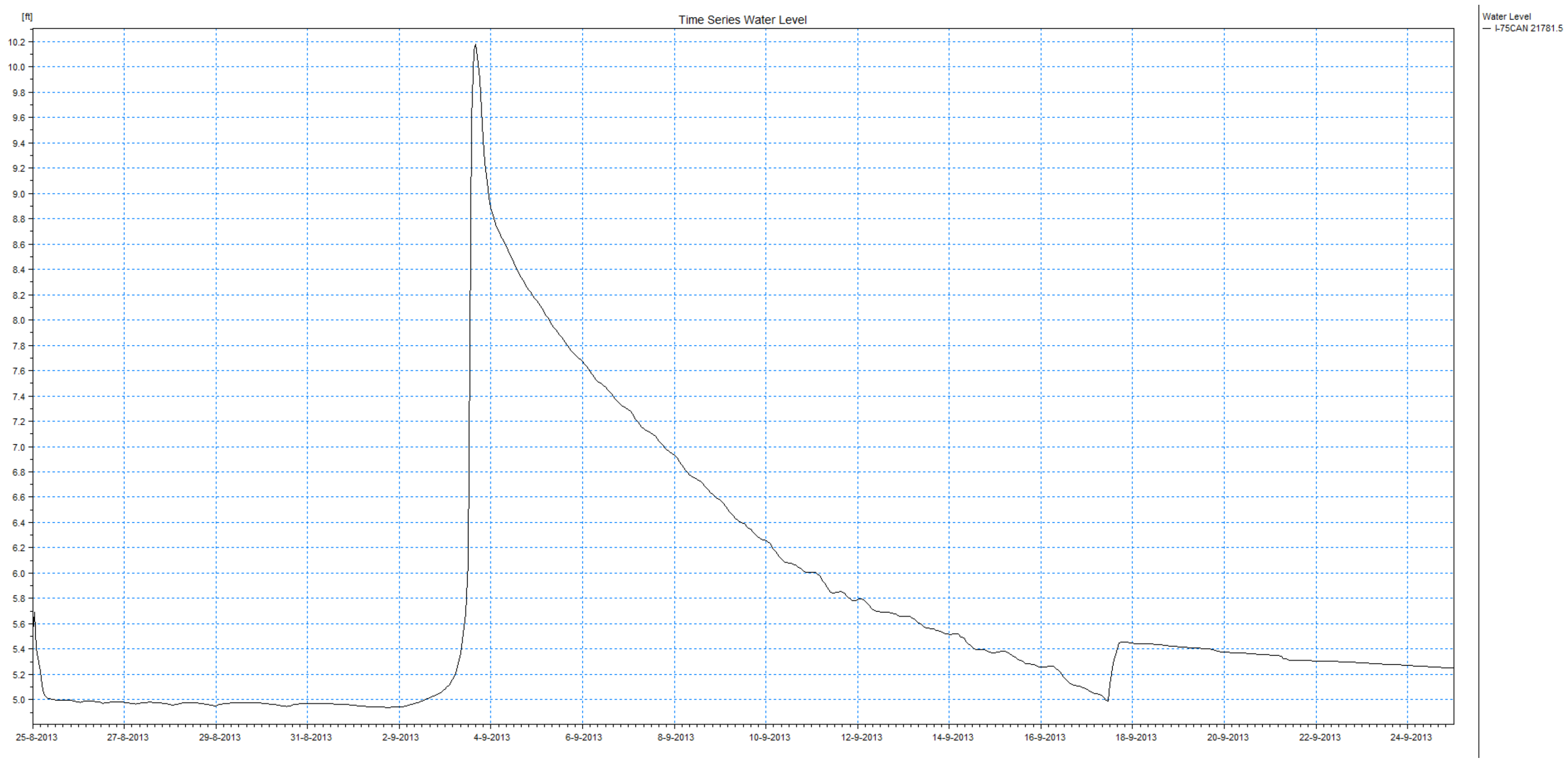


Figure 46. I75W2 Tailwater Stage (ft)

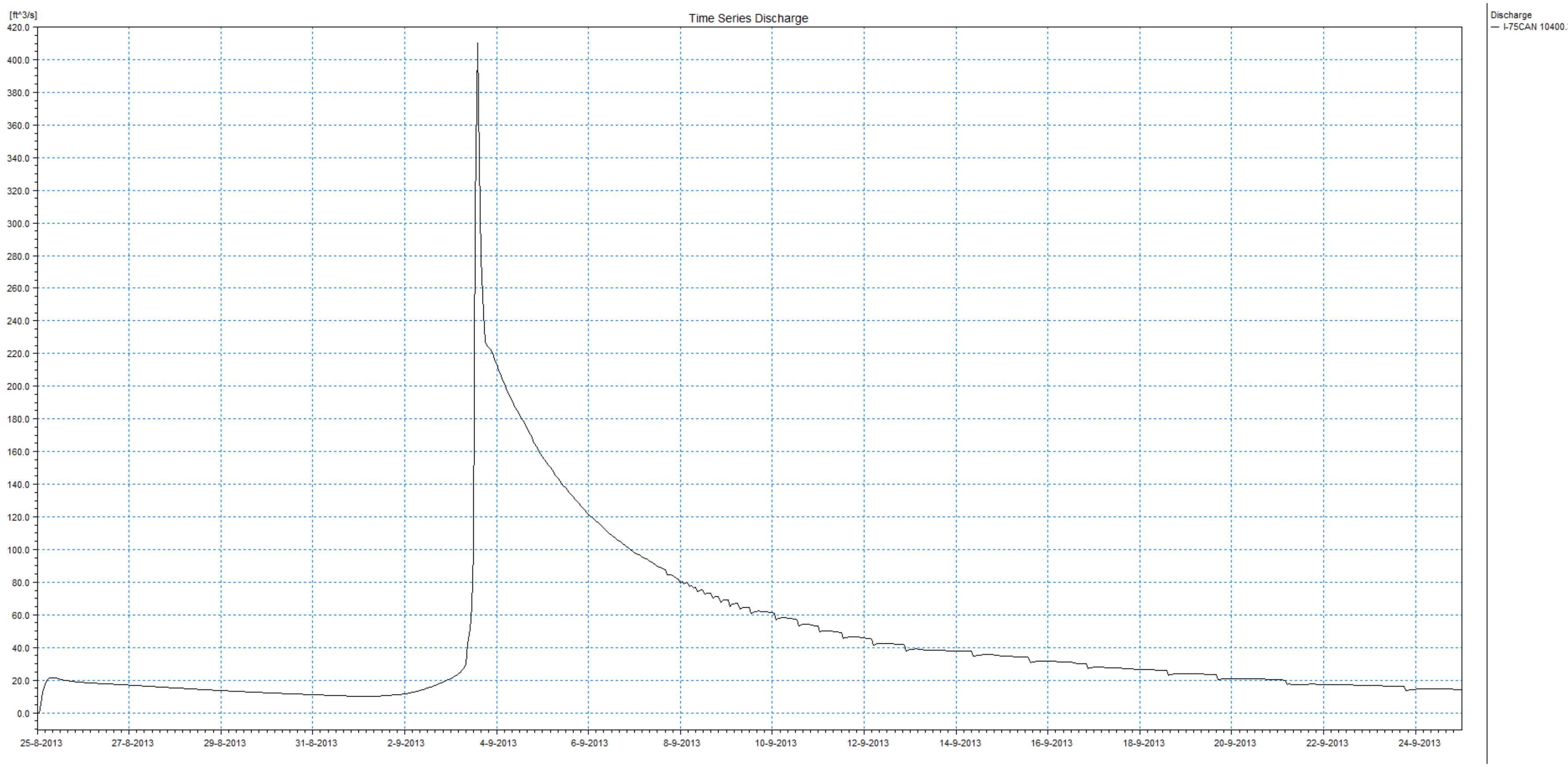
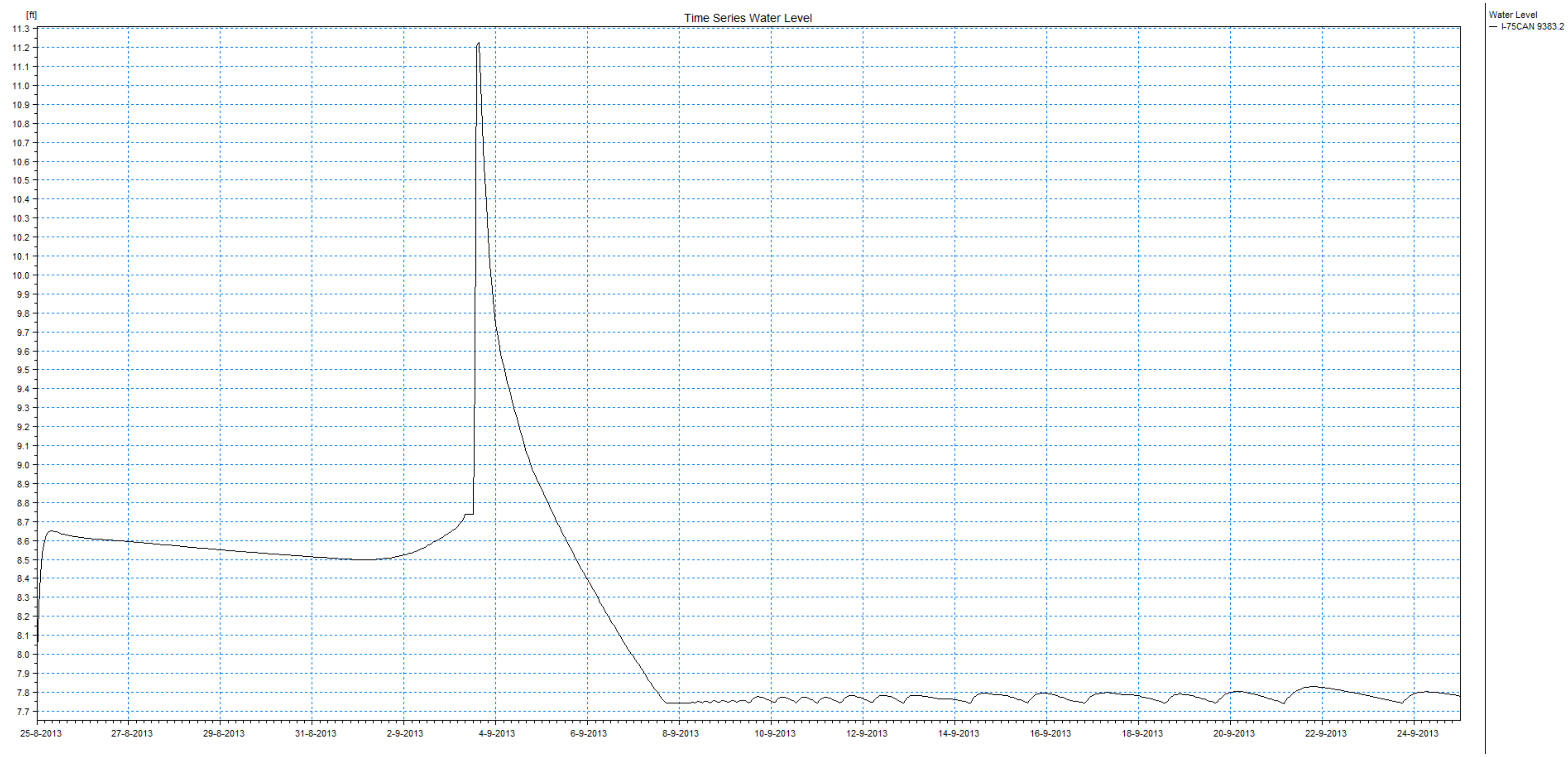


Figure 47. I75W3 Discharge (cfs)



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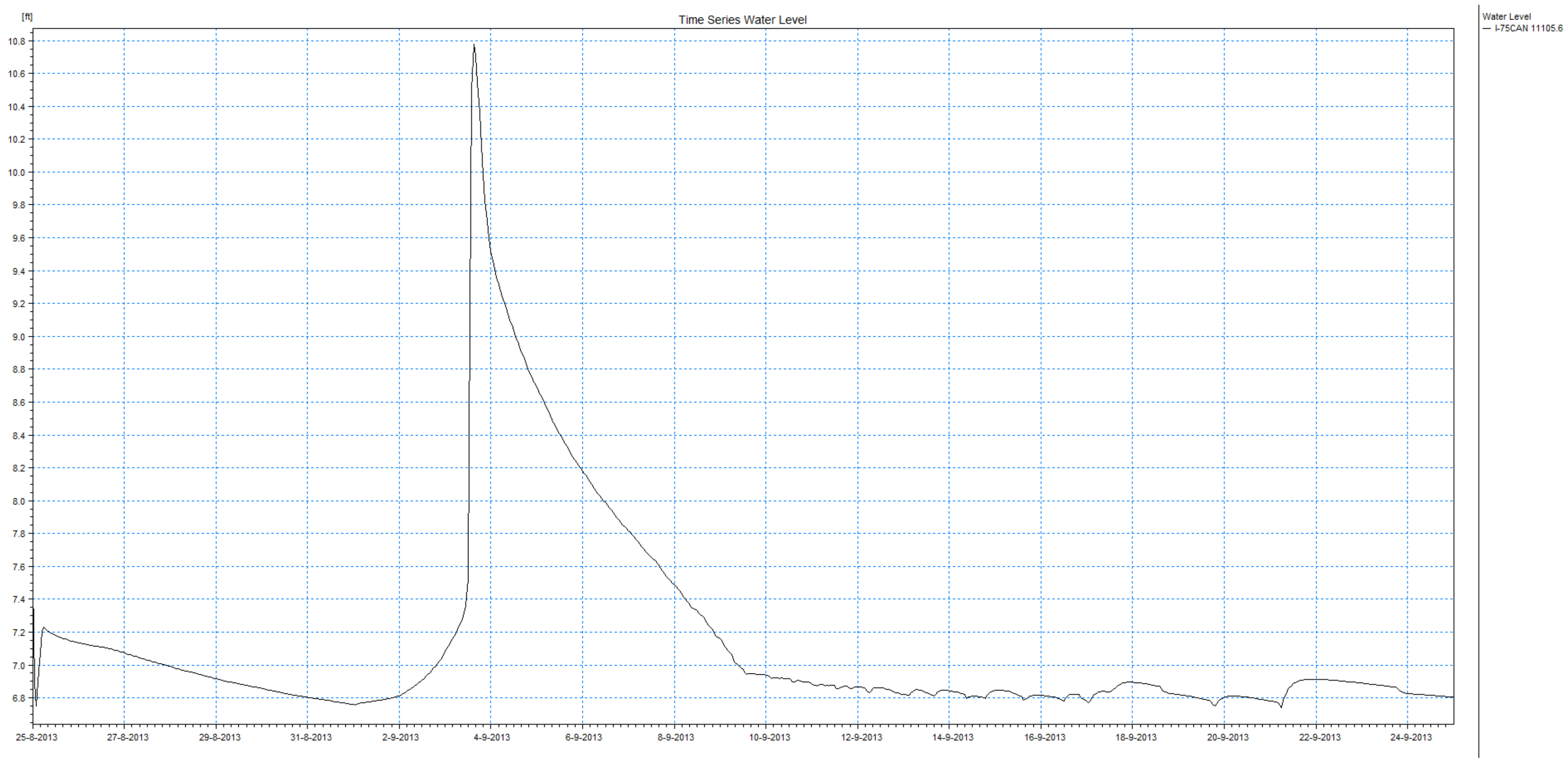

Figure 49. I75W3 Tailwater Stage (ft)

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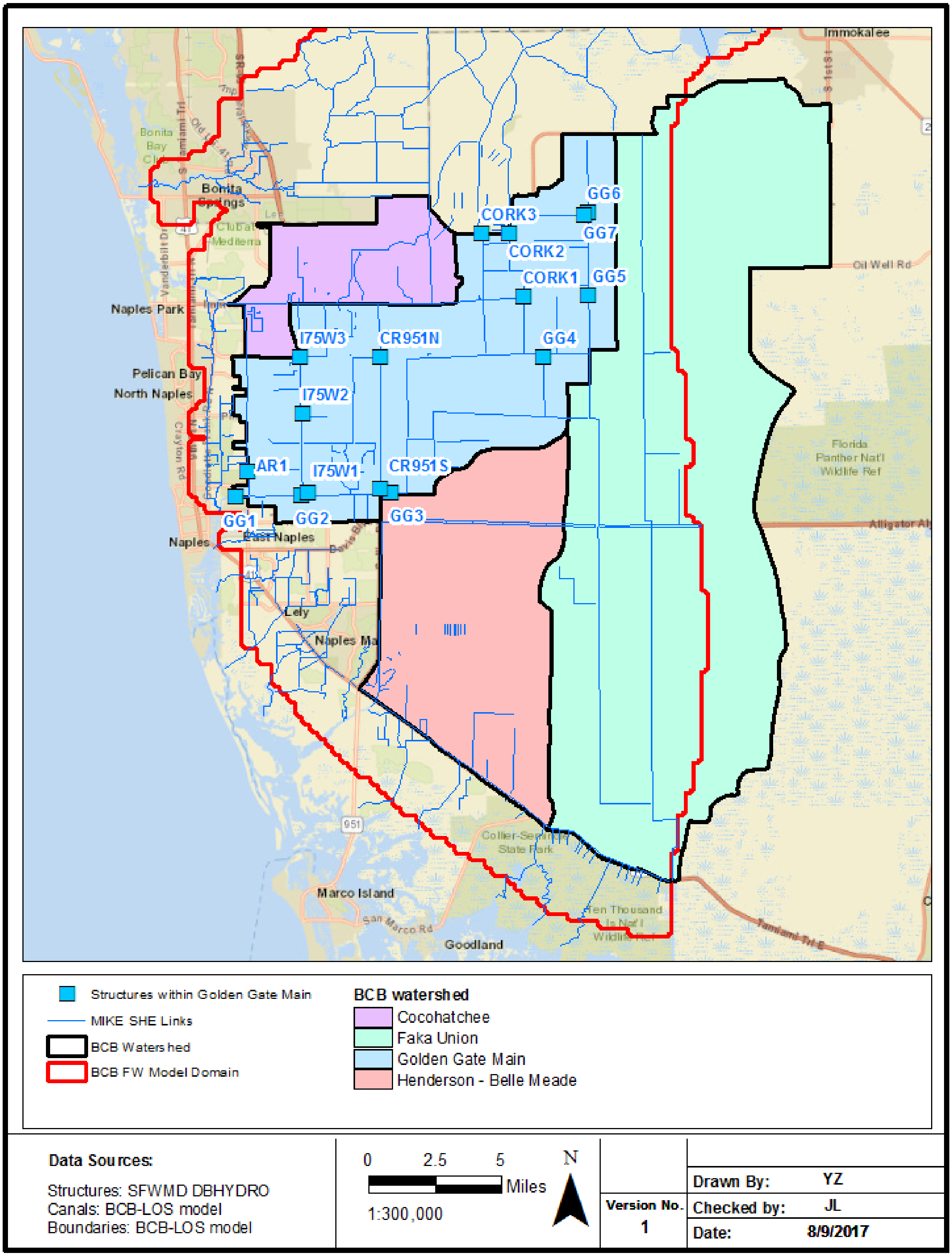

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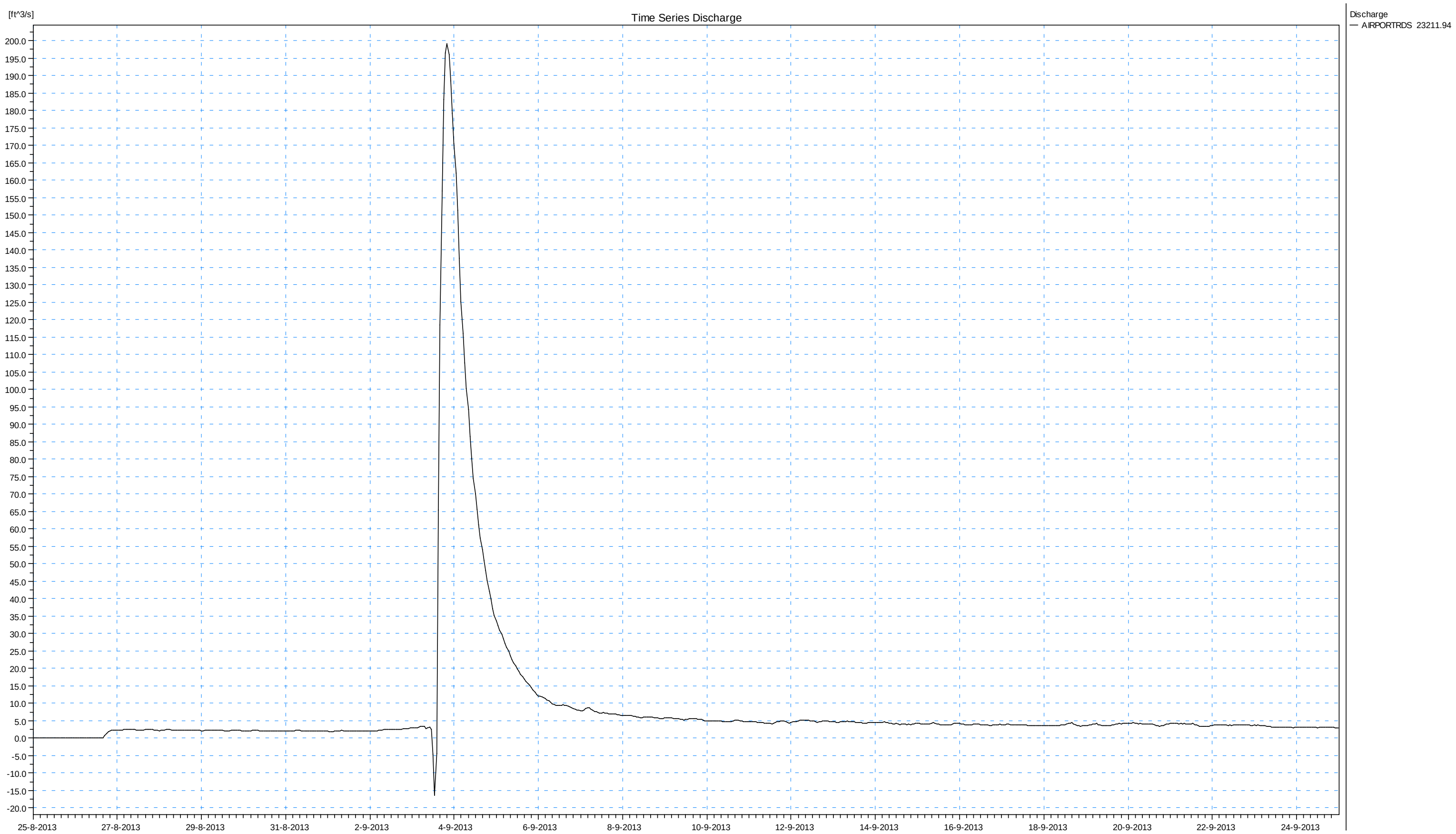
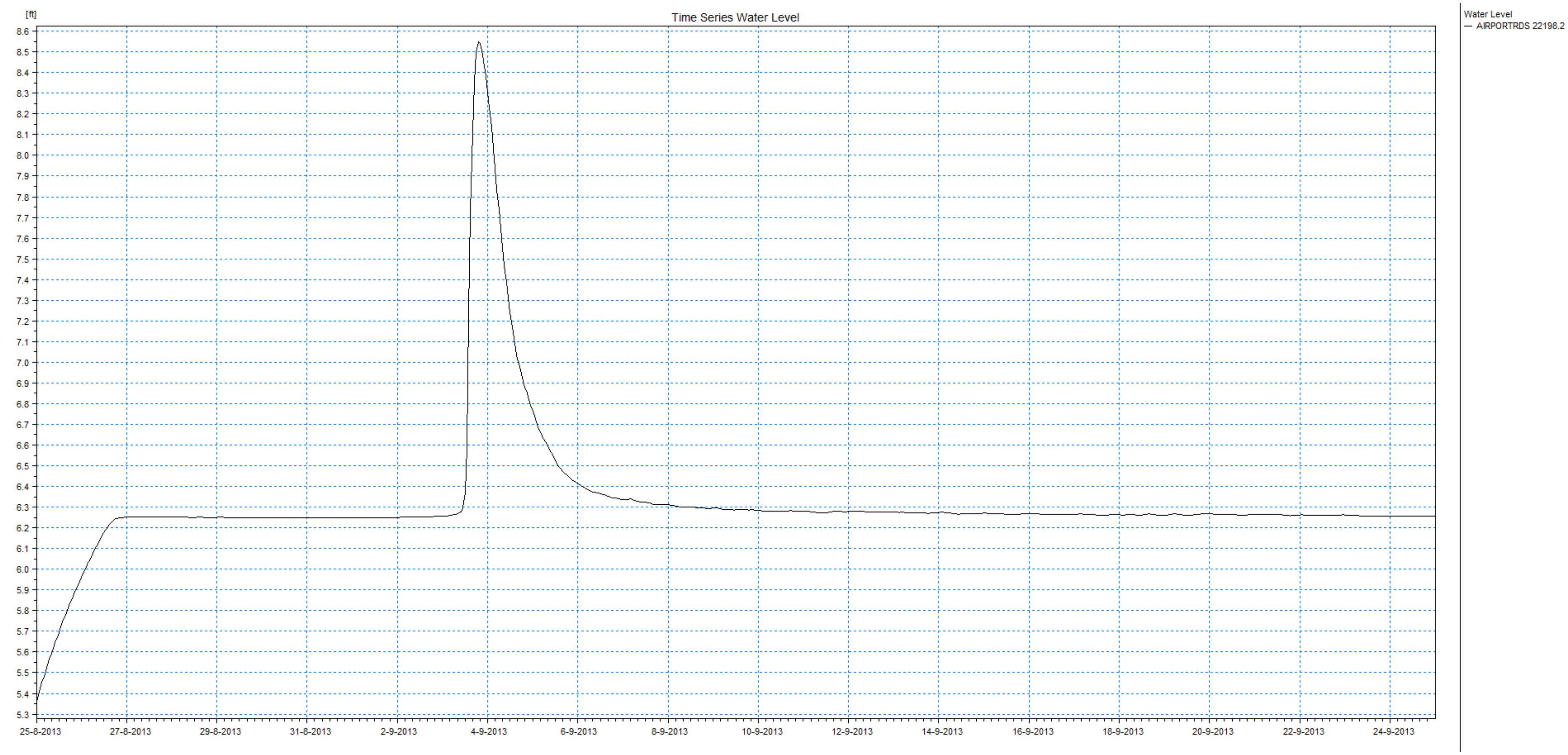


Figure 2. AR1 Discharge (cfs)



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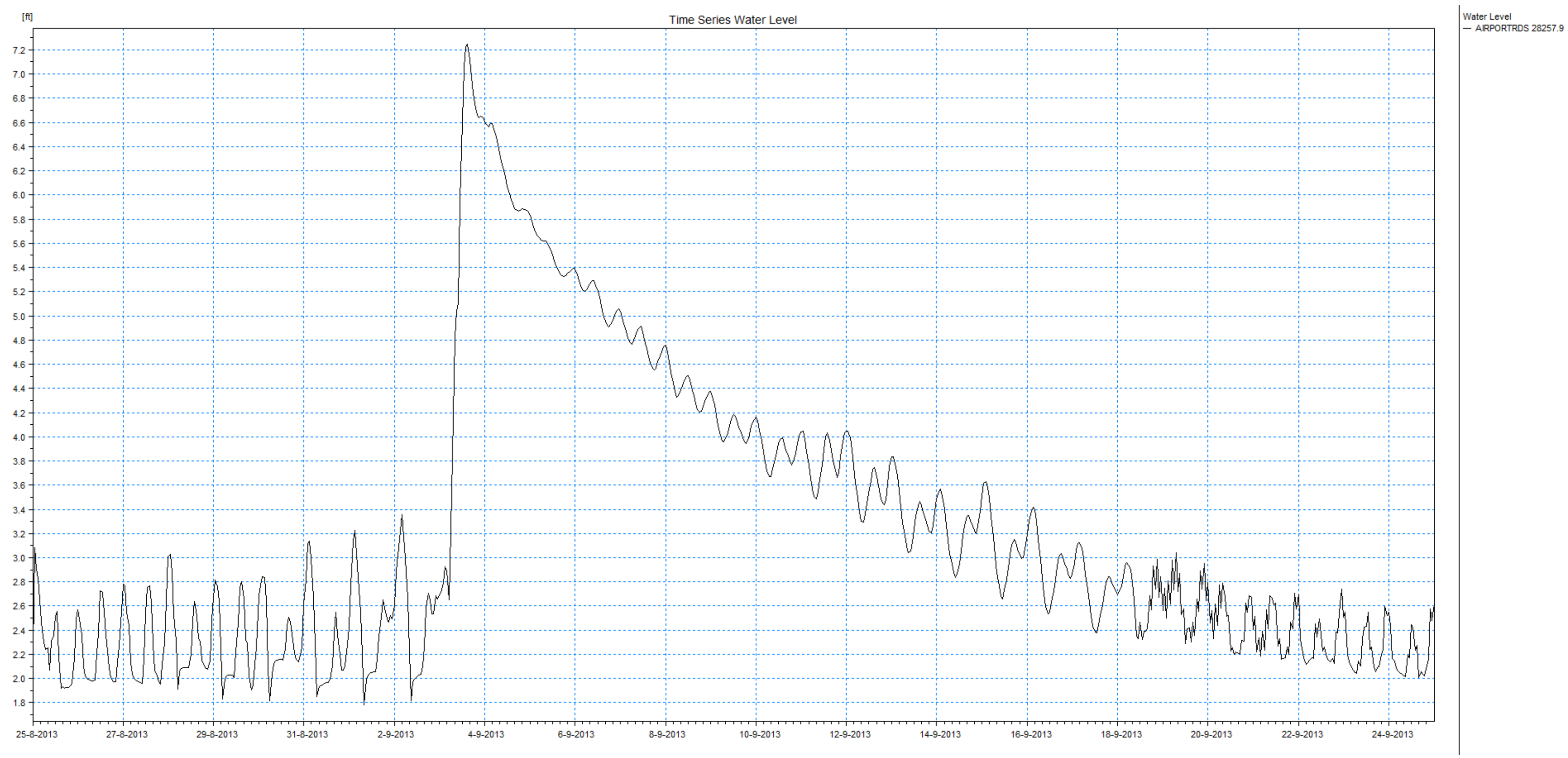

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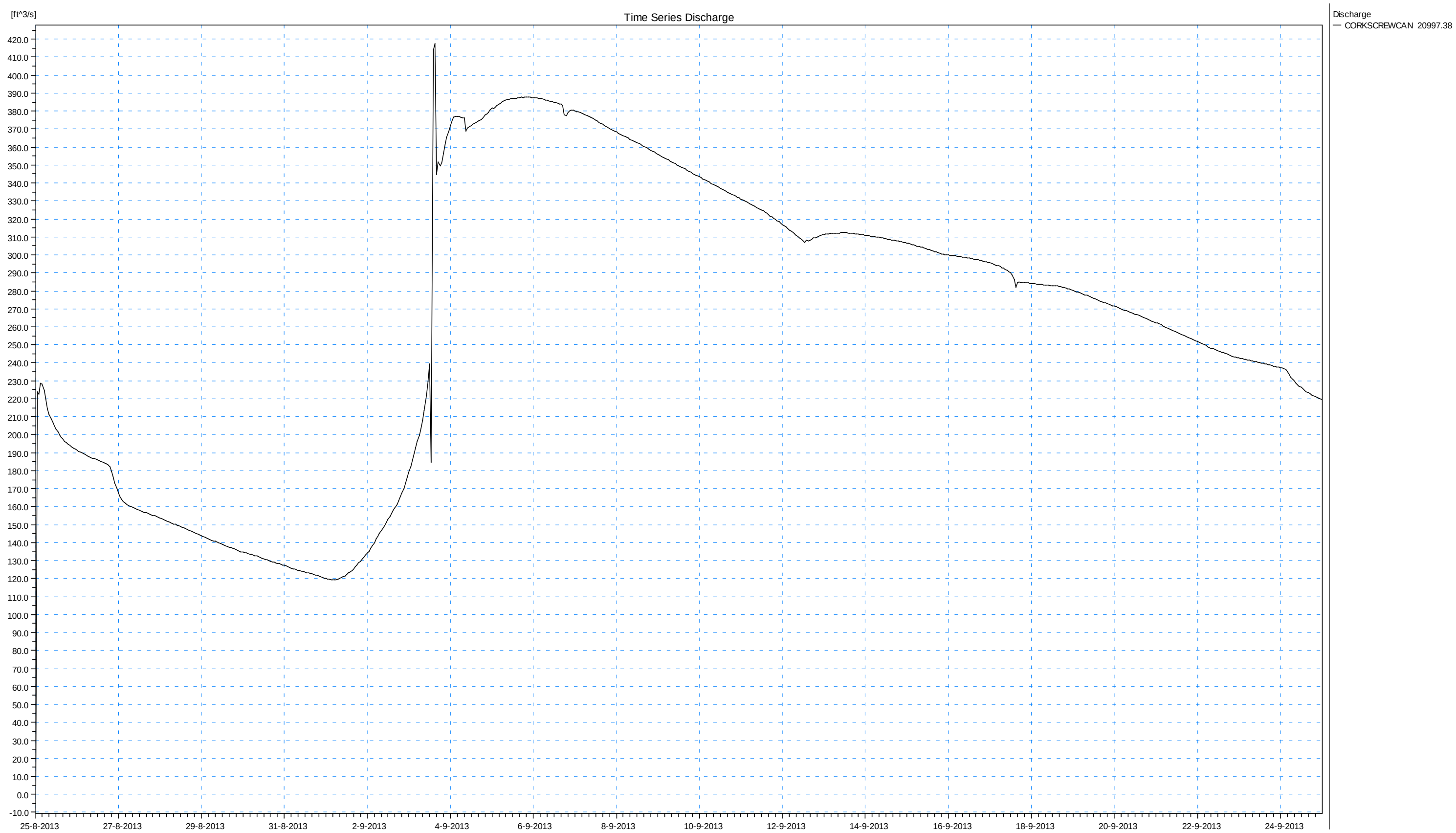

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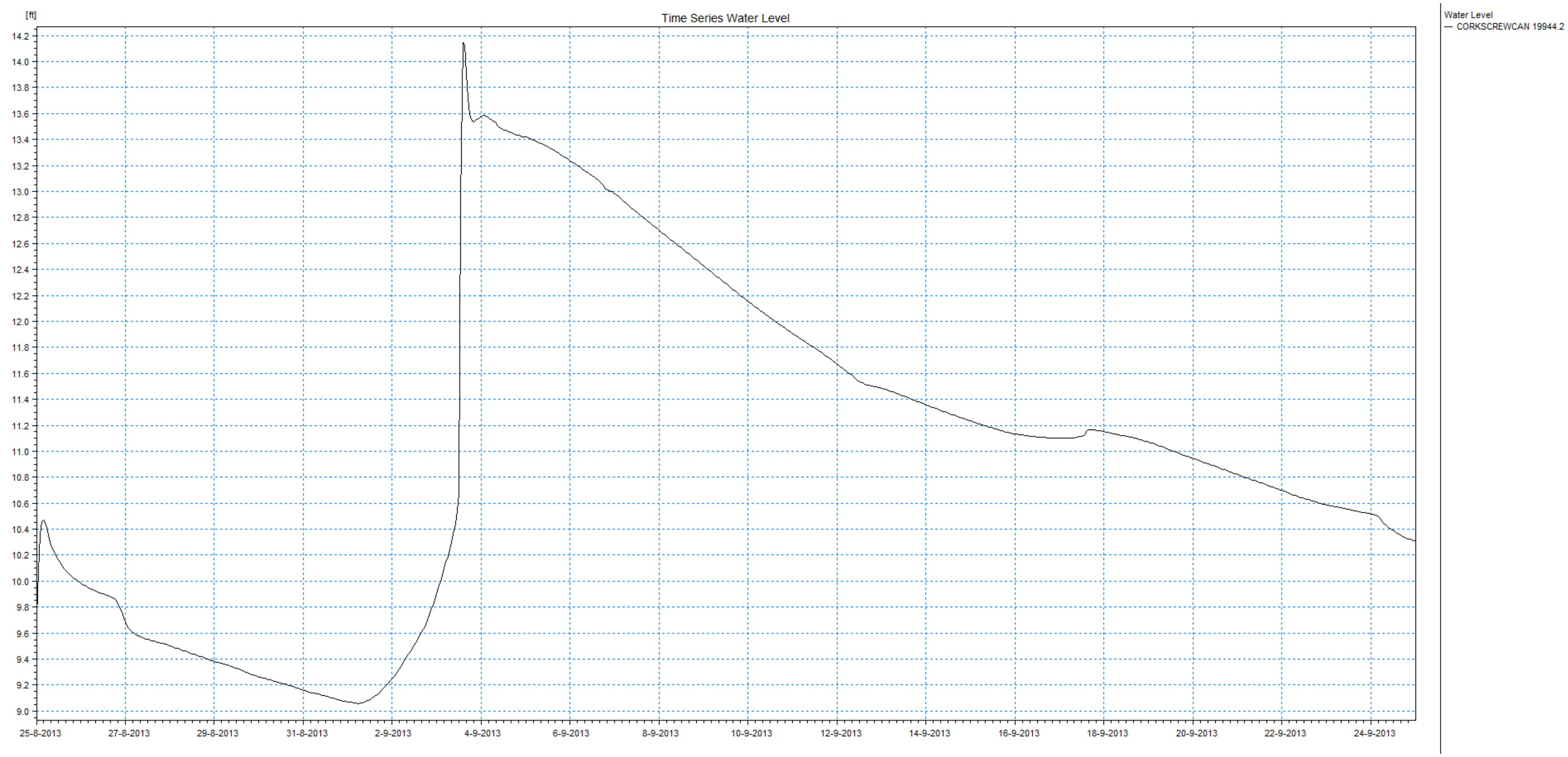

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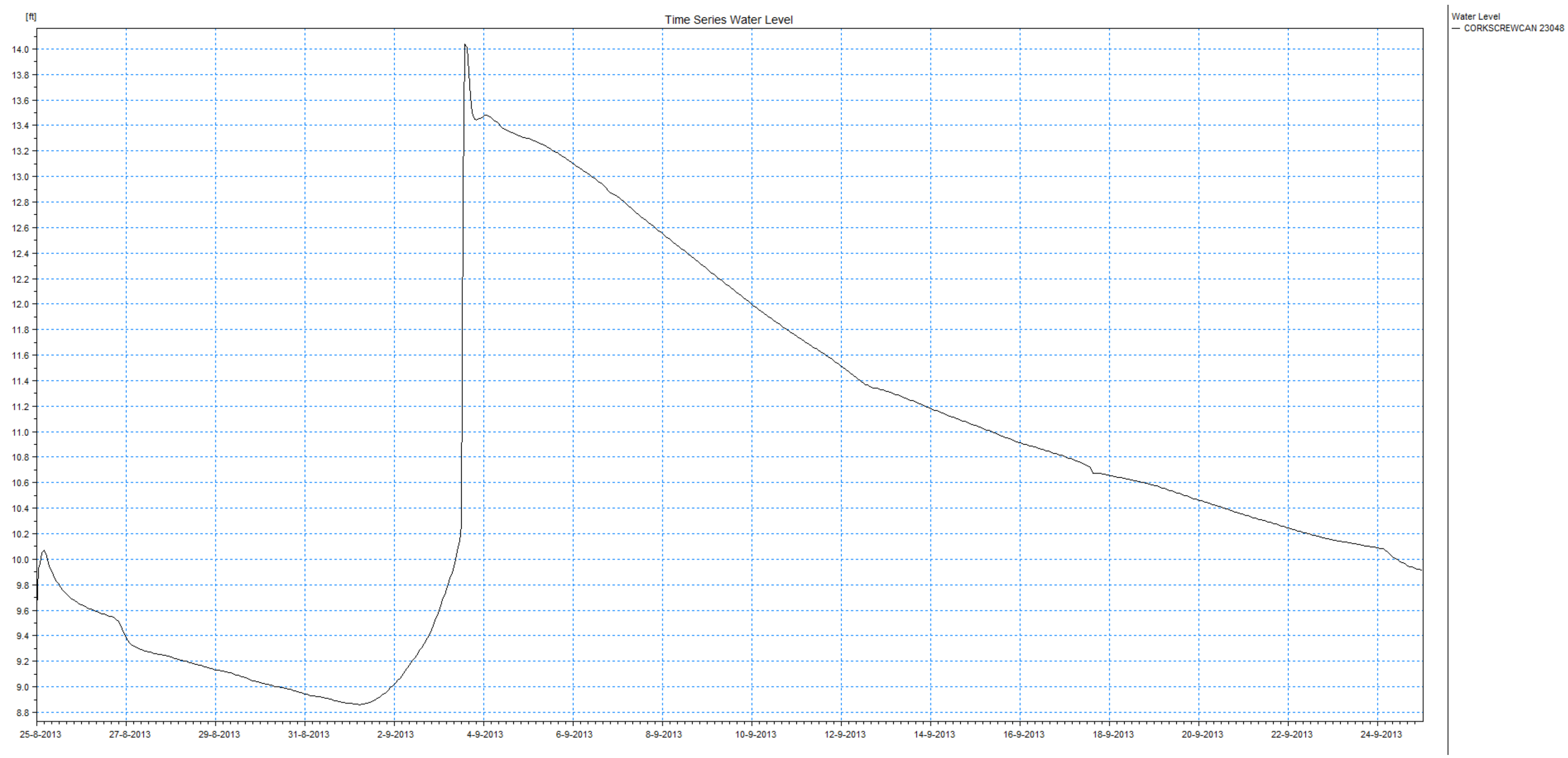

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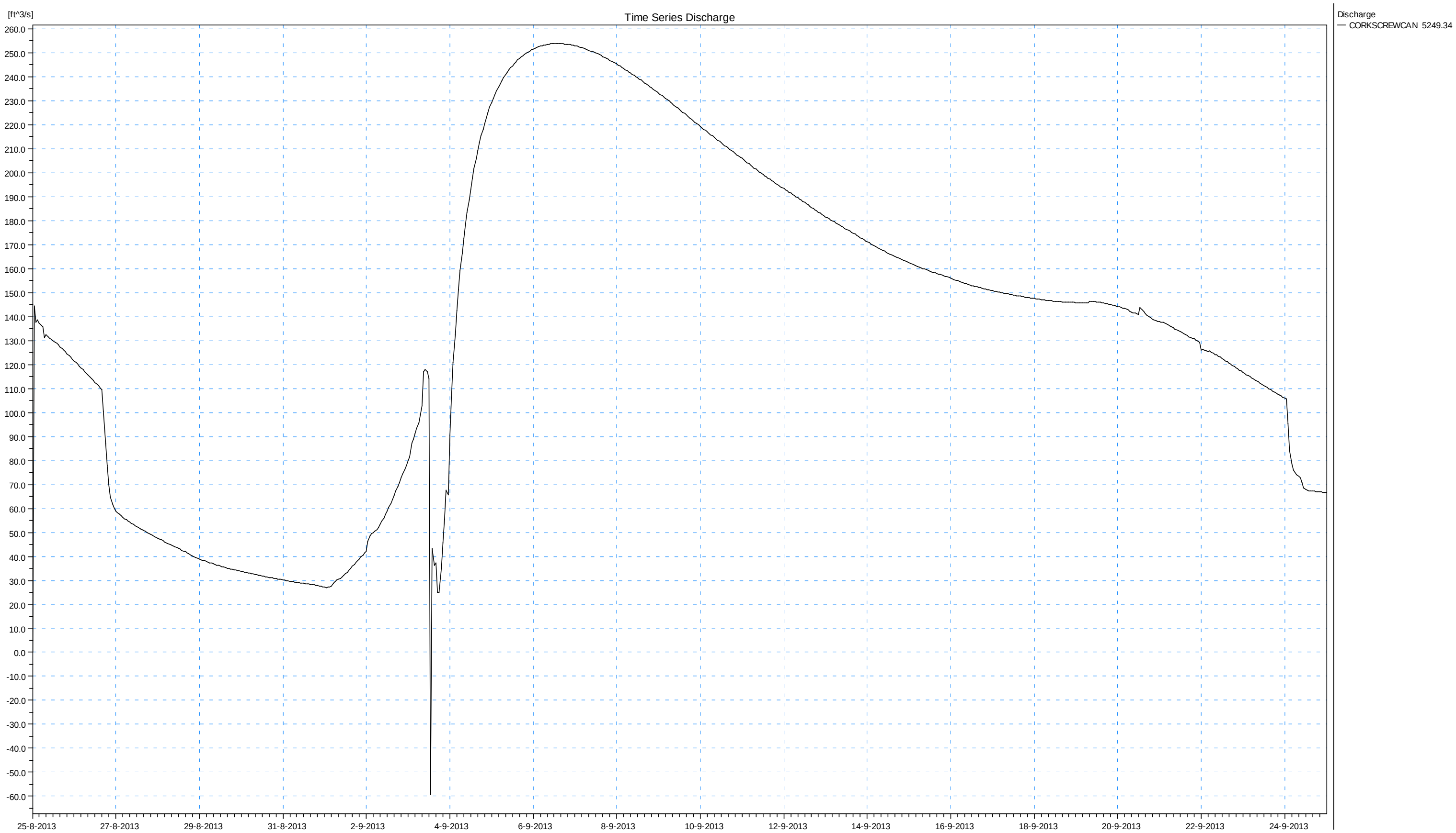

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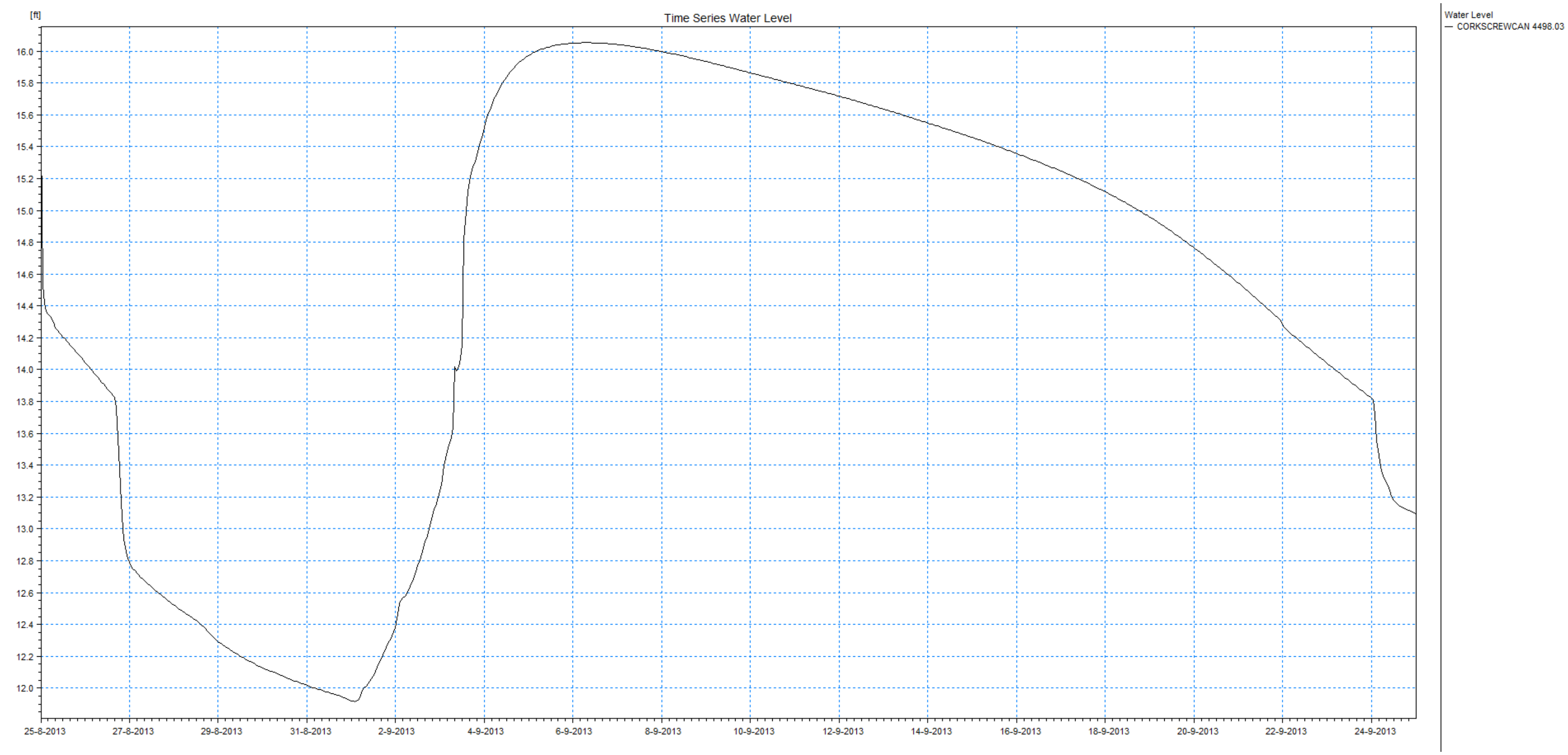

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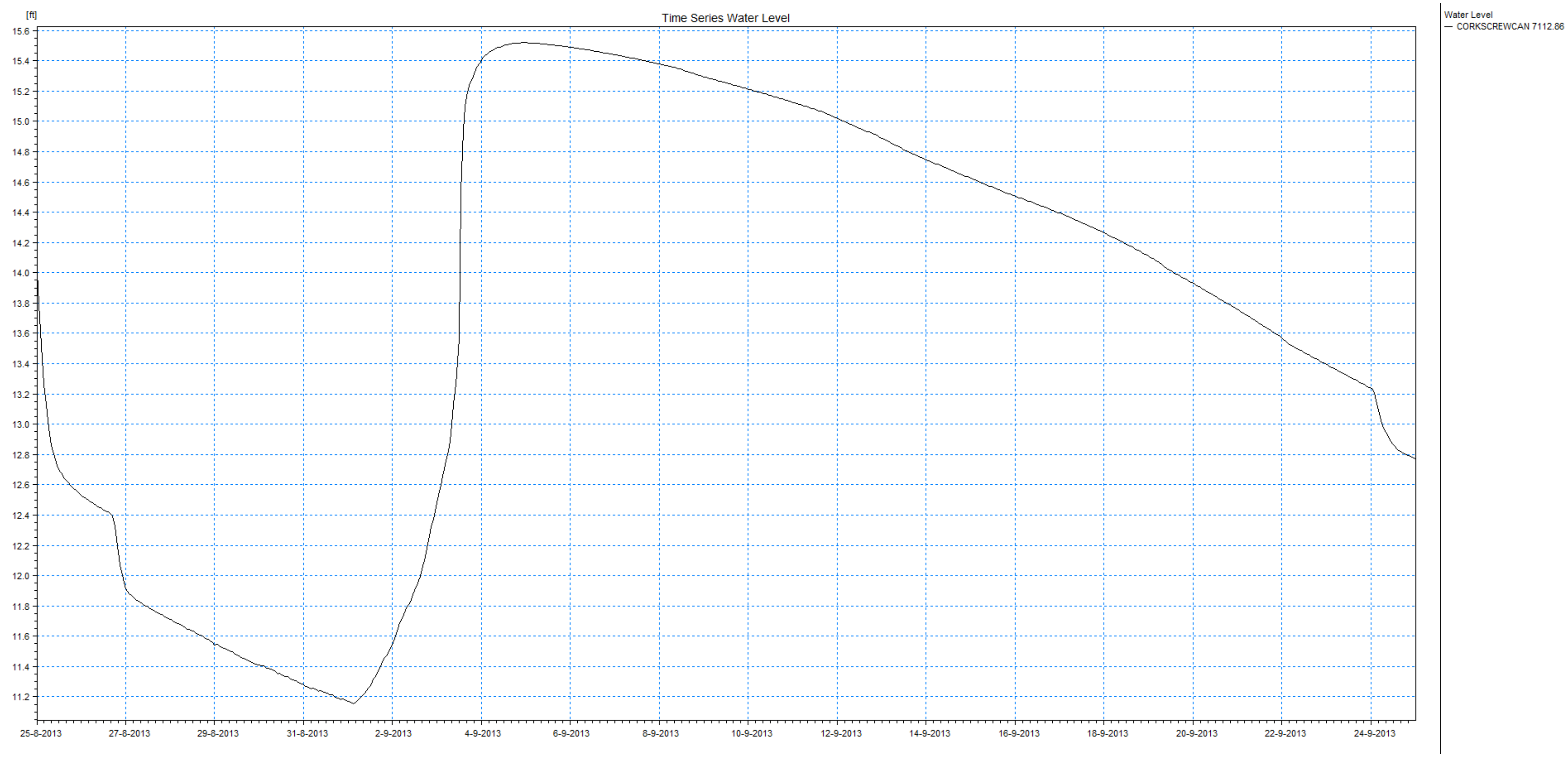

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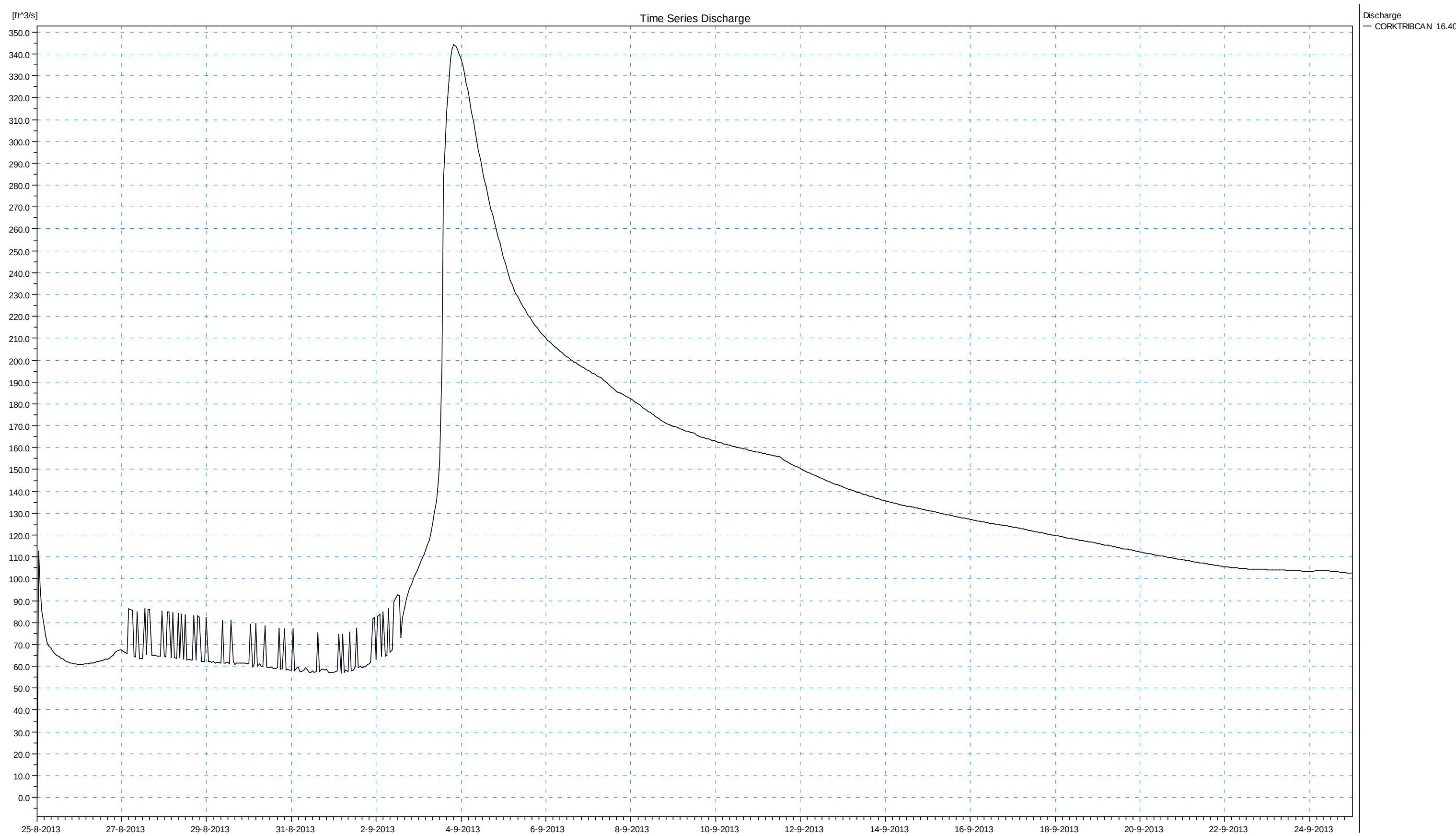

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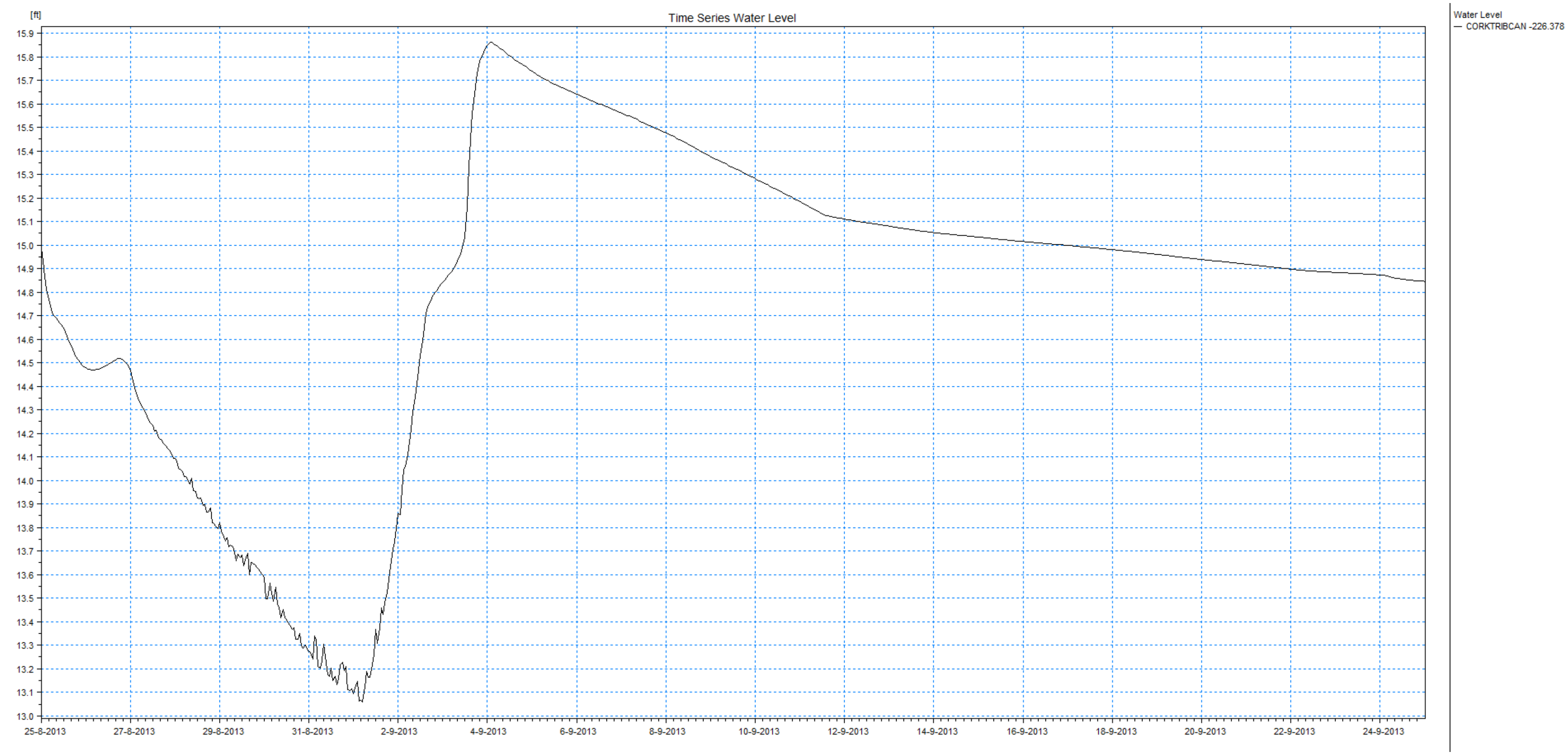

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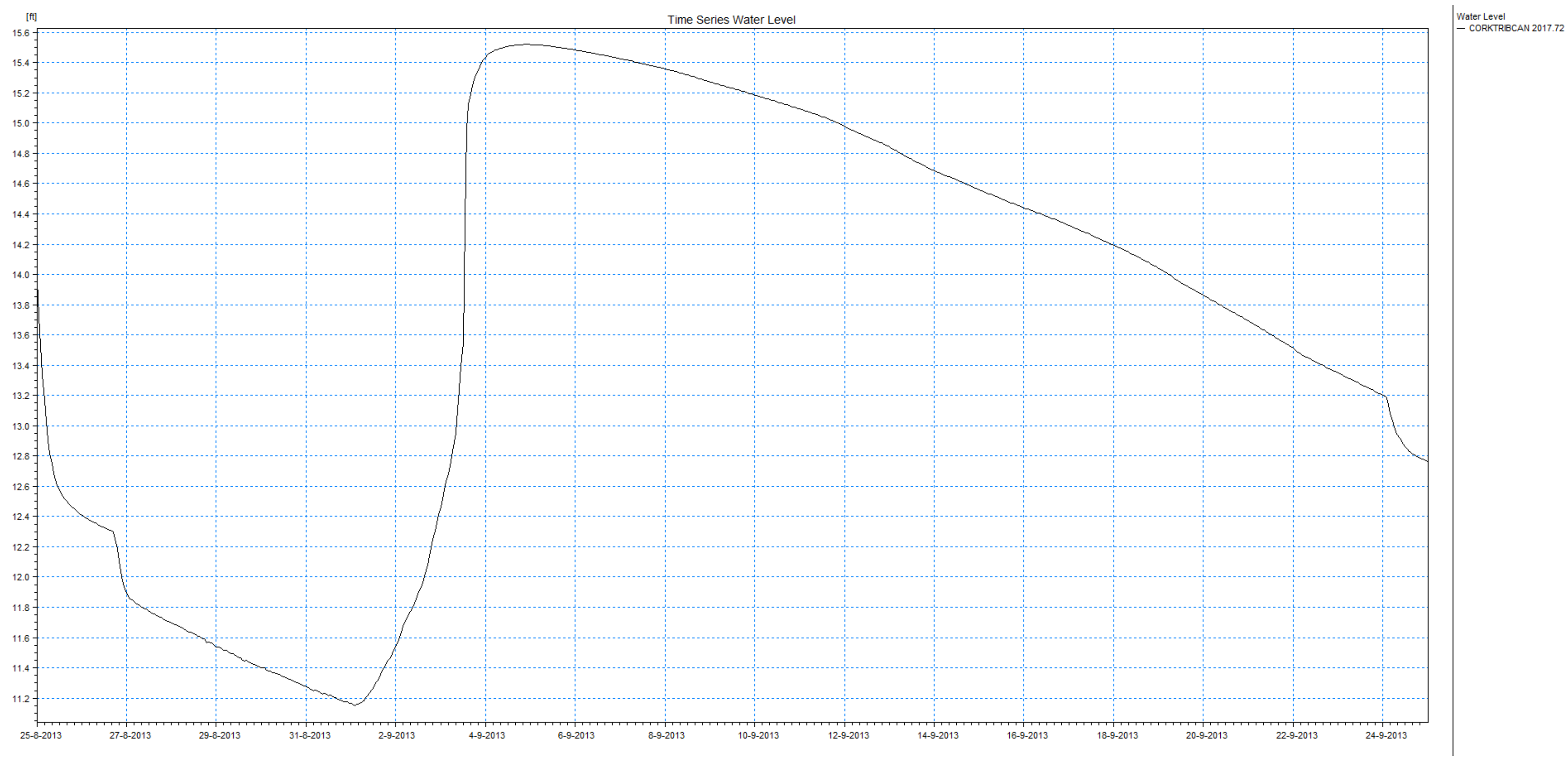

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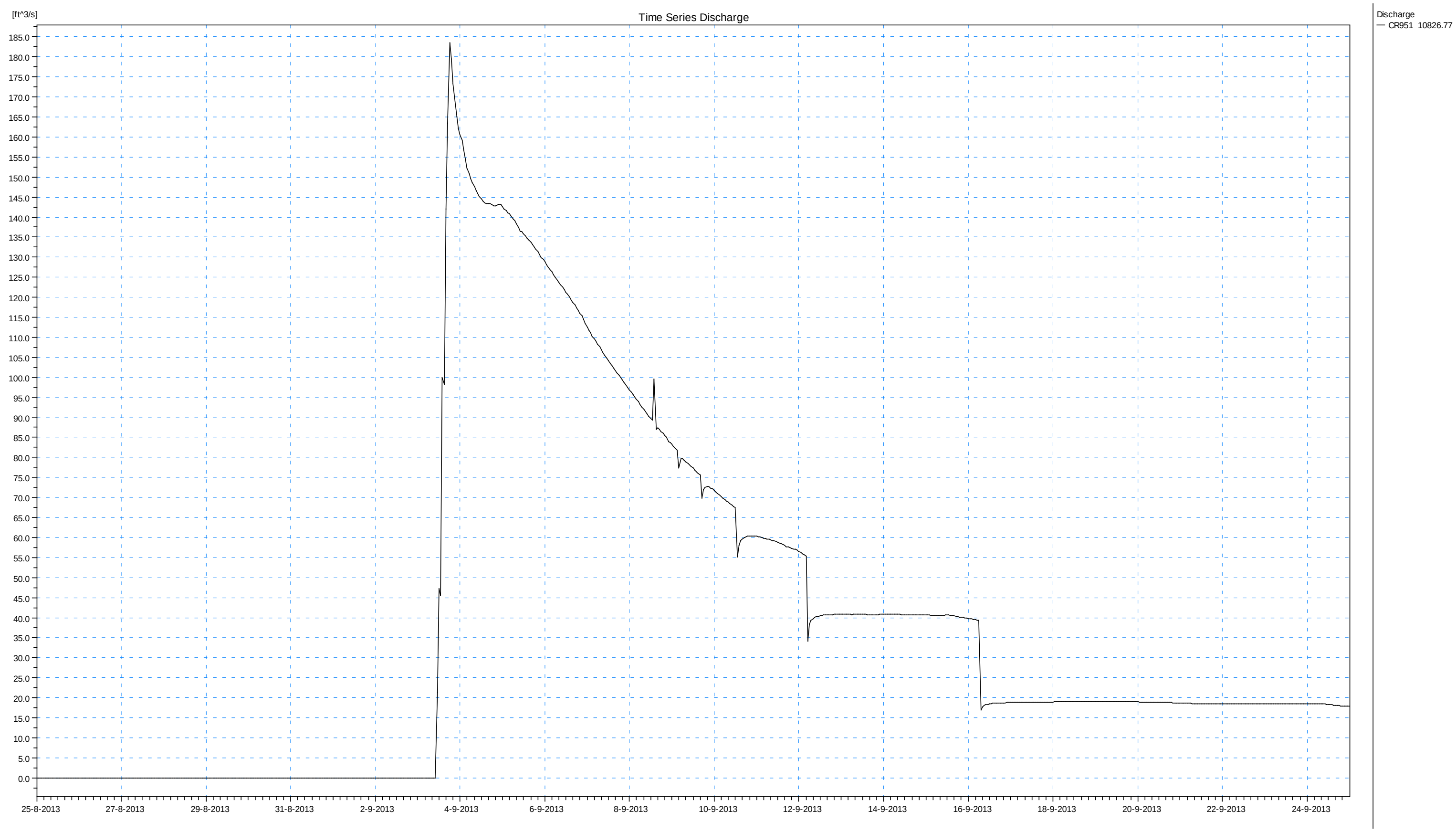

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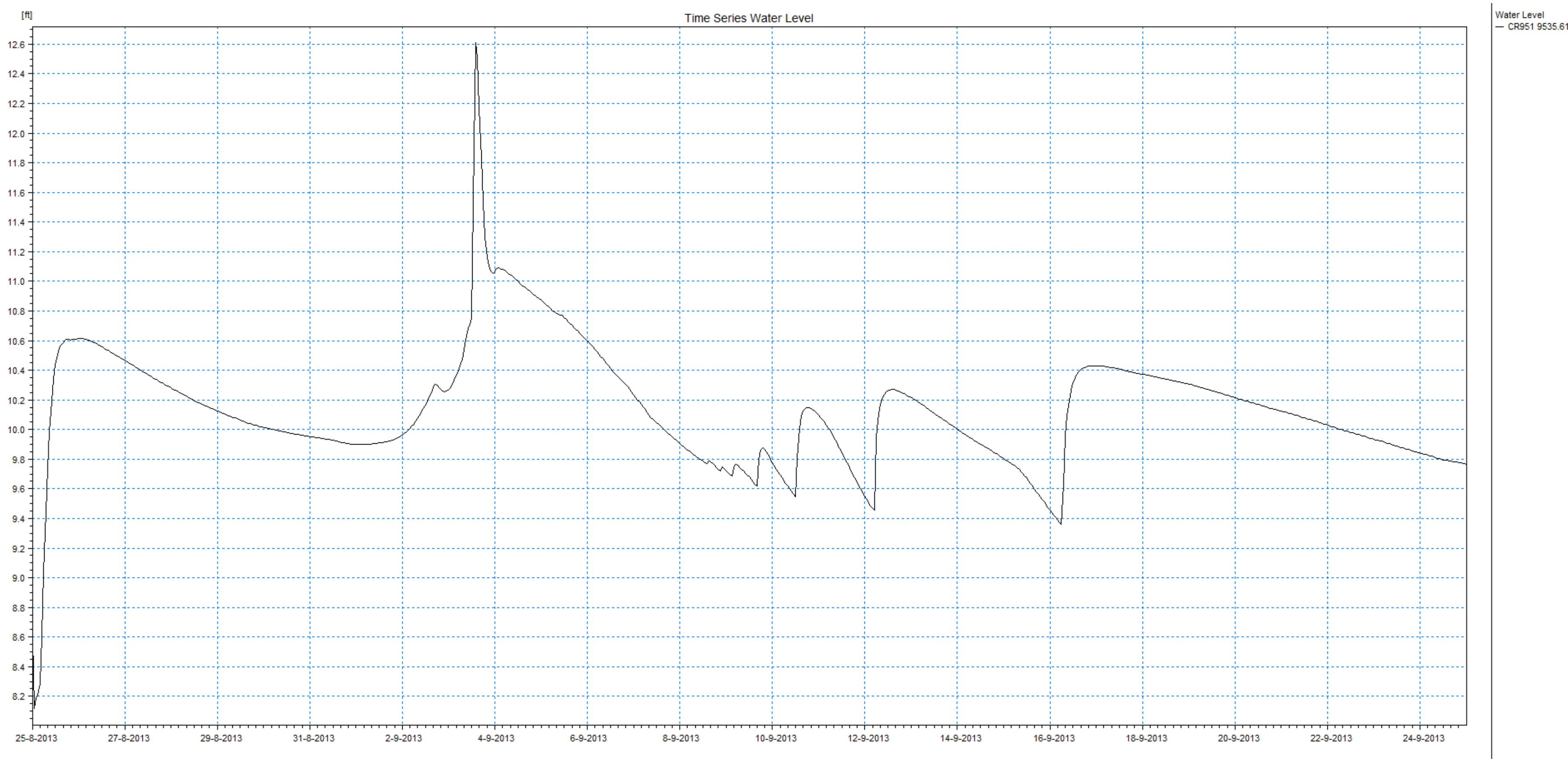

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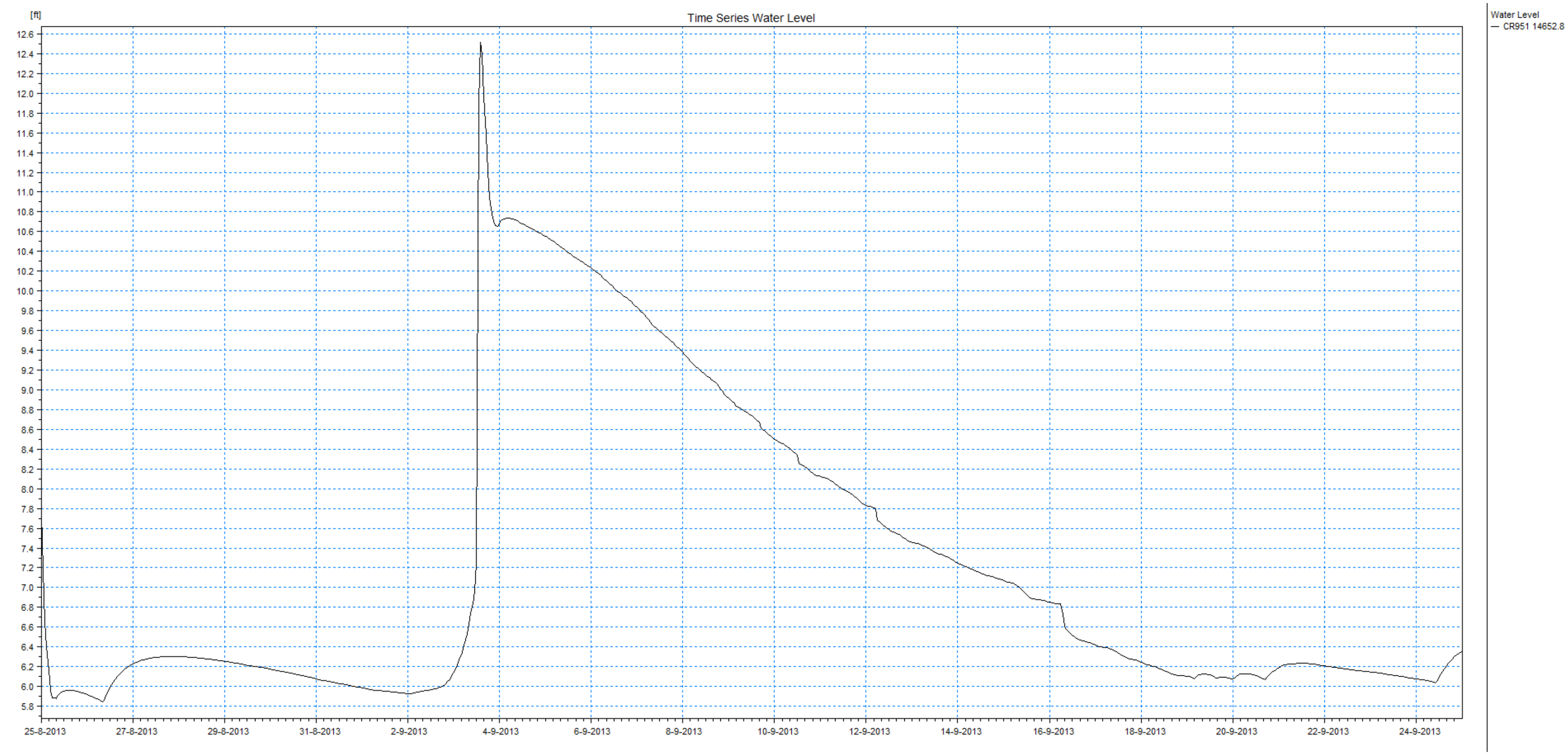

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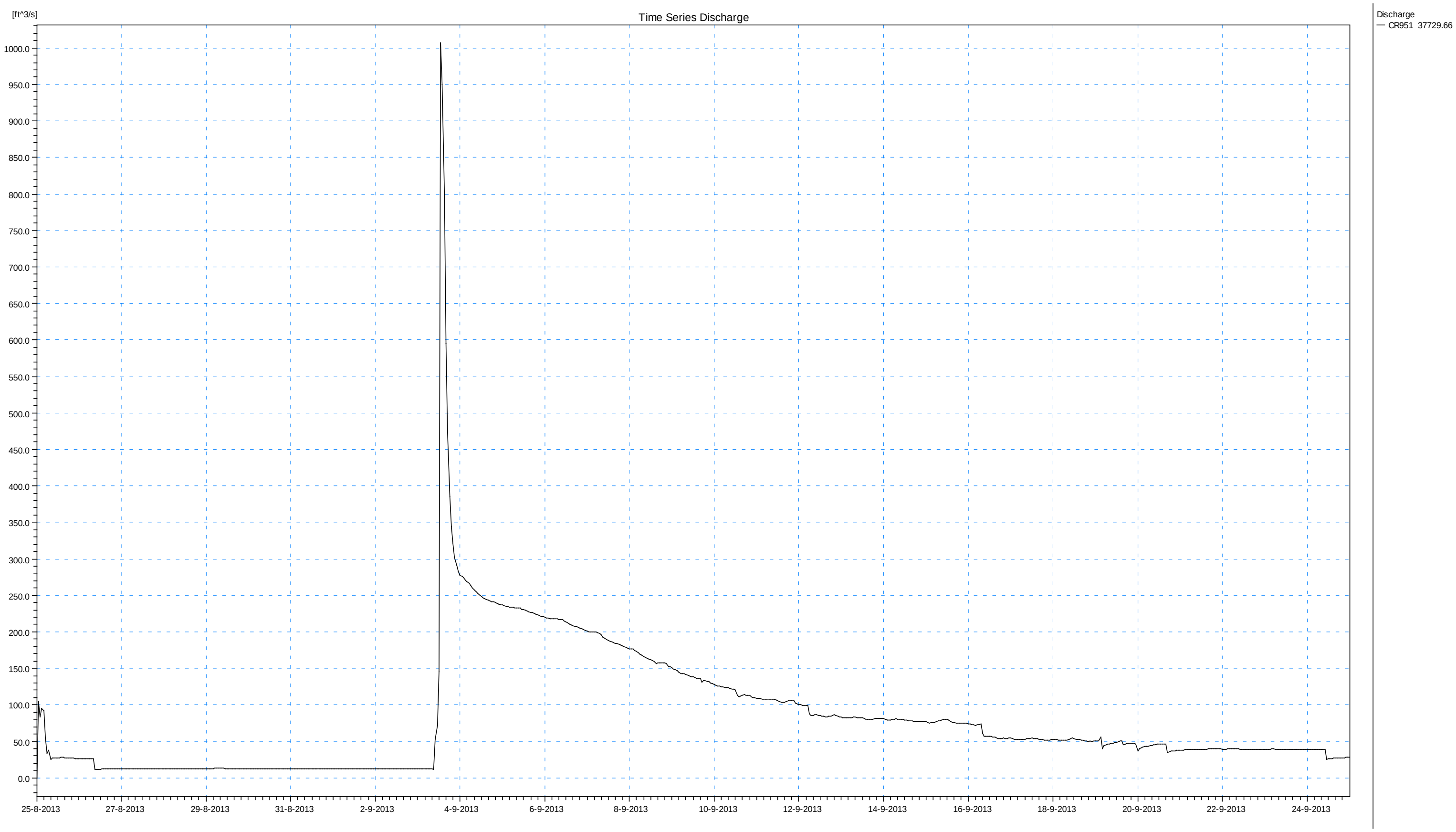

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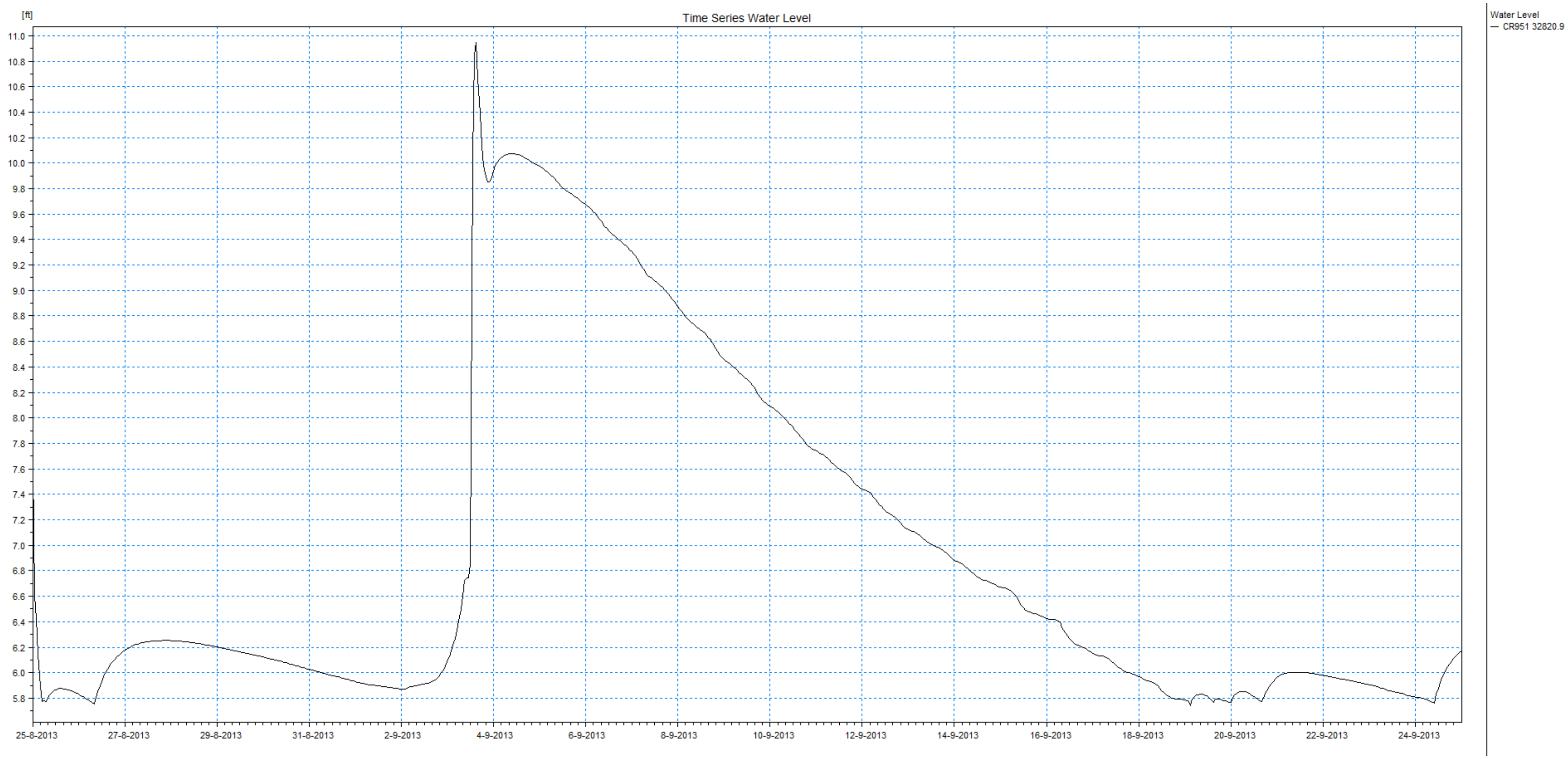

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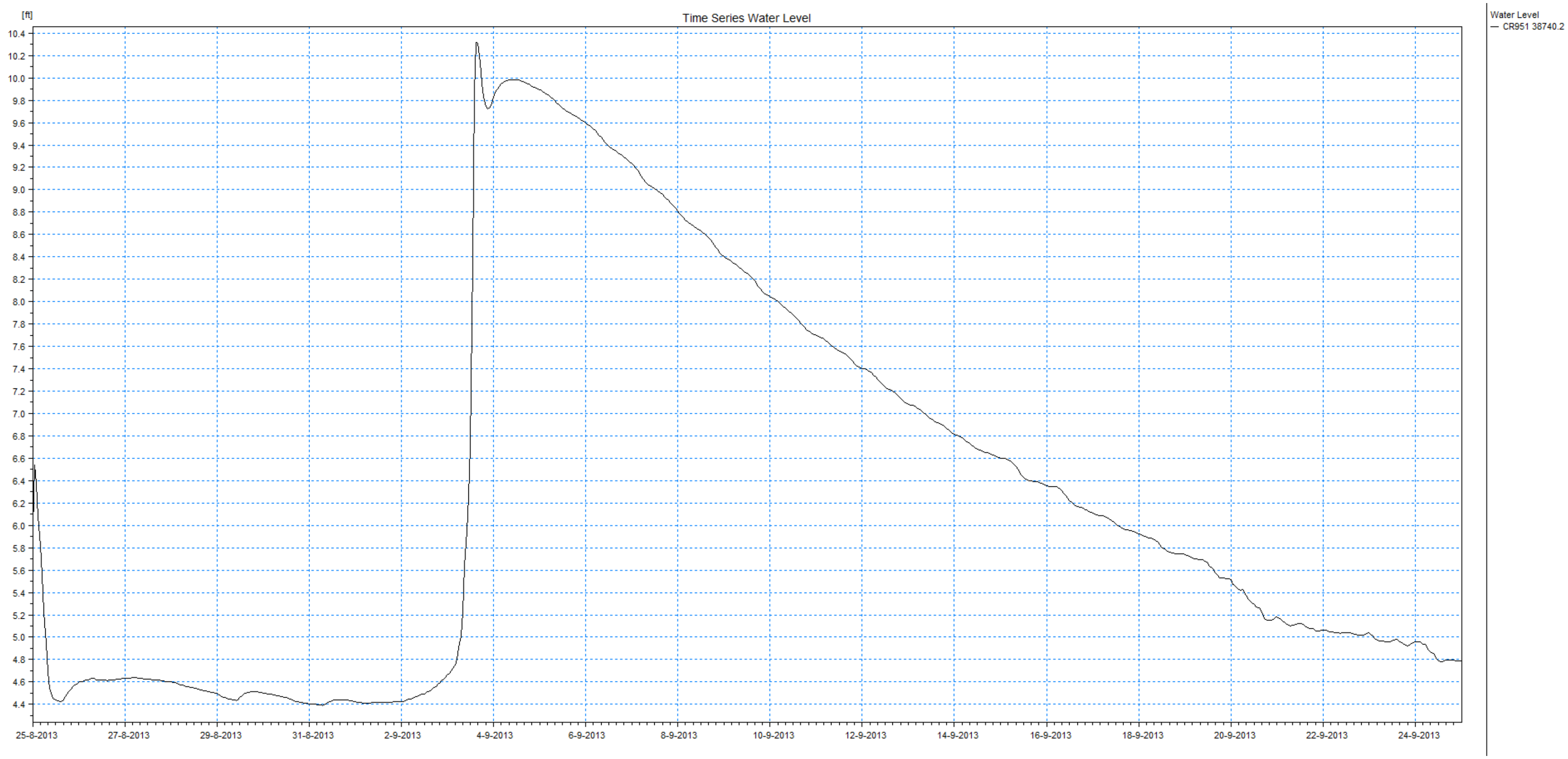

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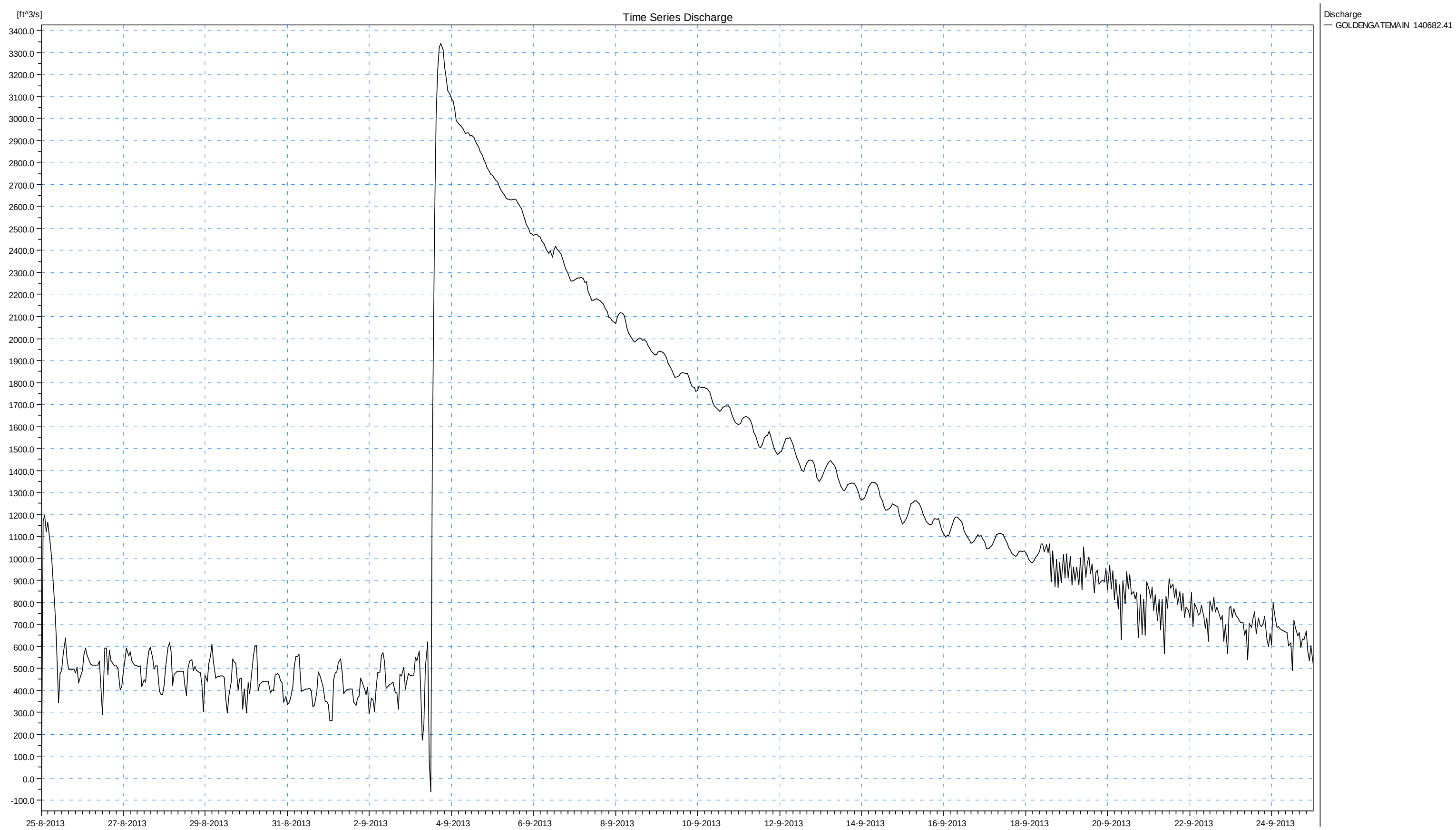

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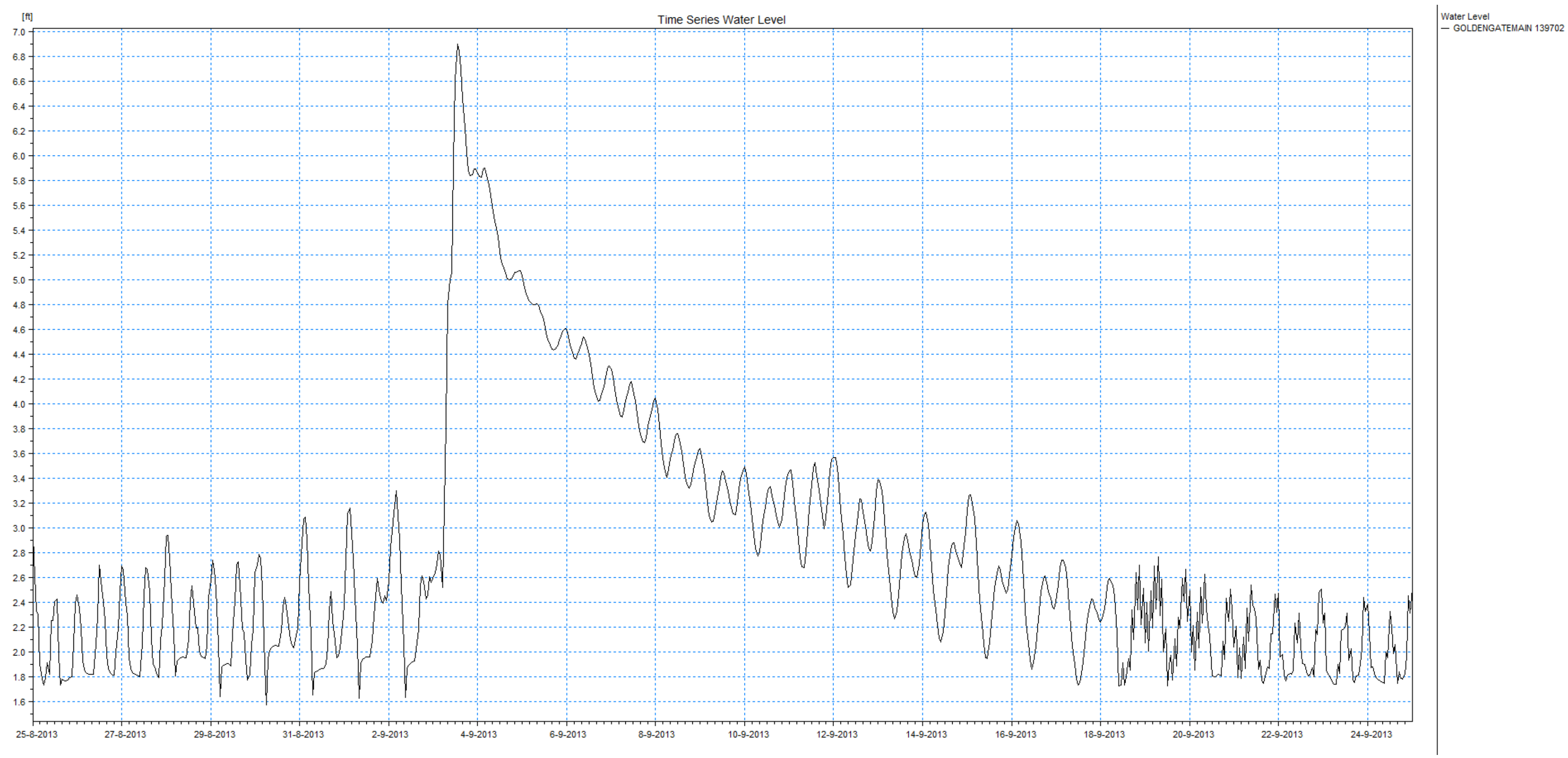

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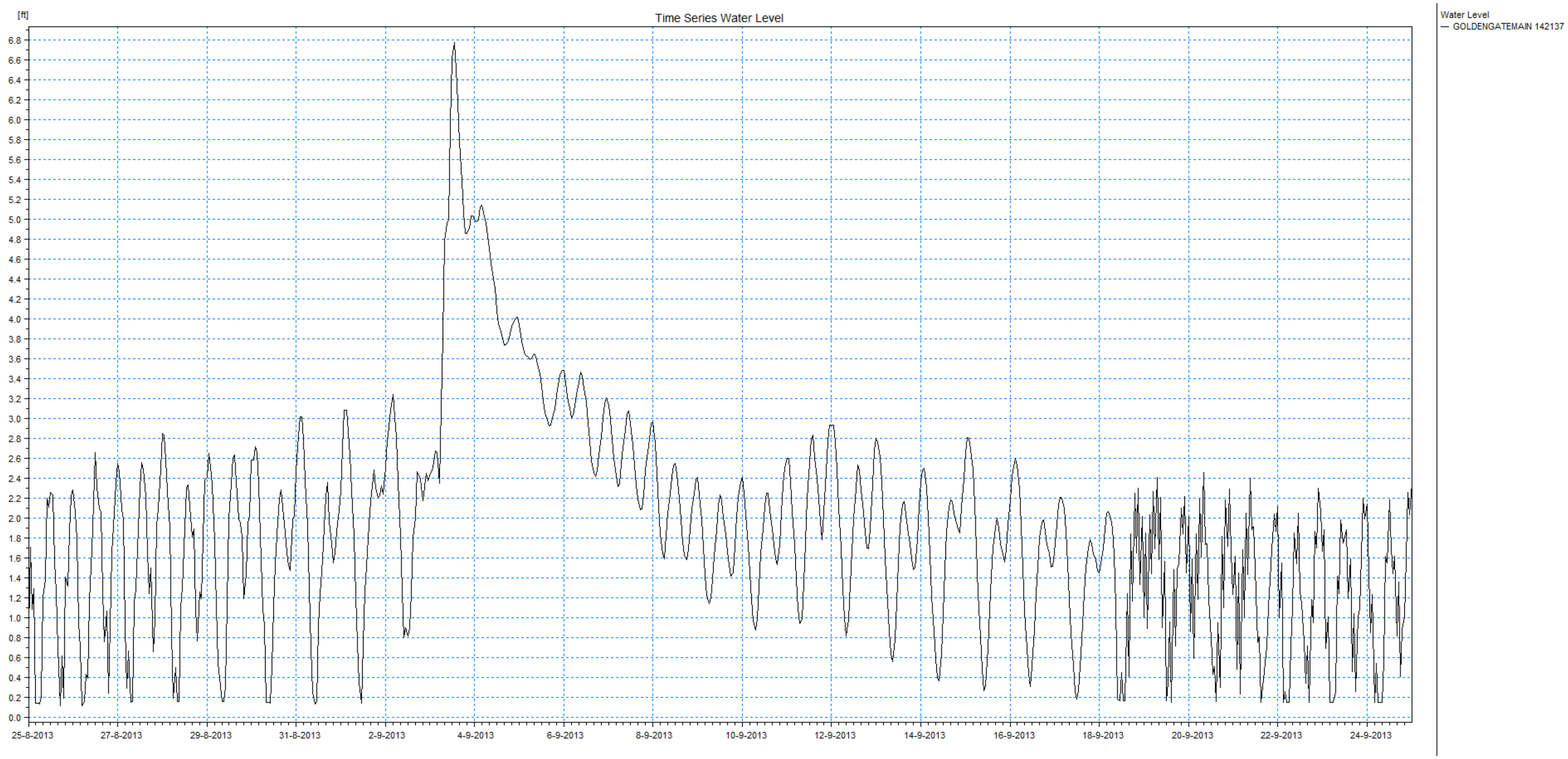

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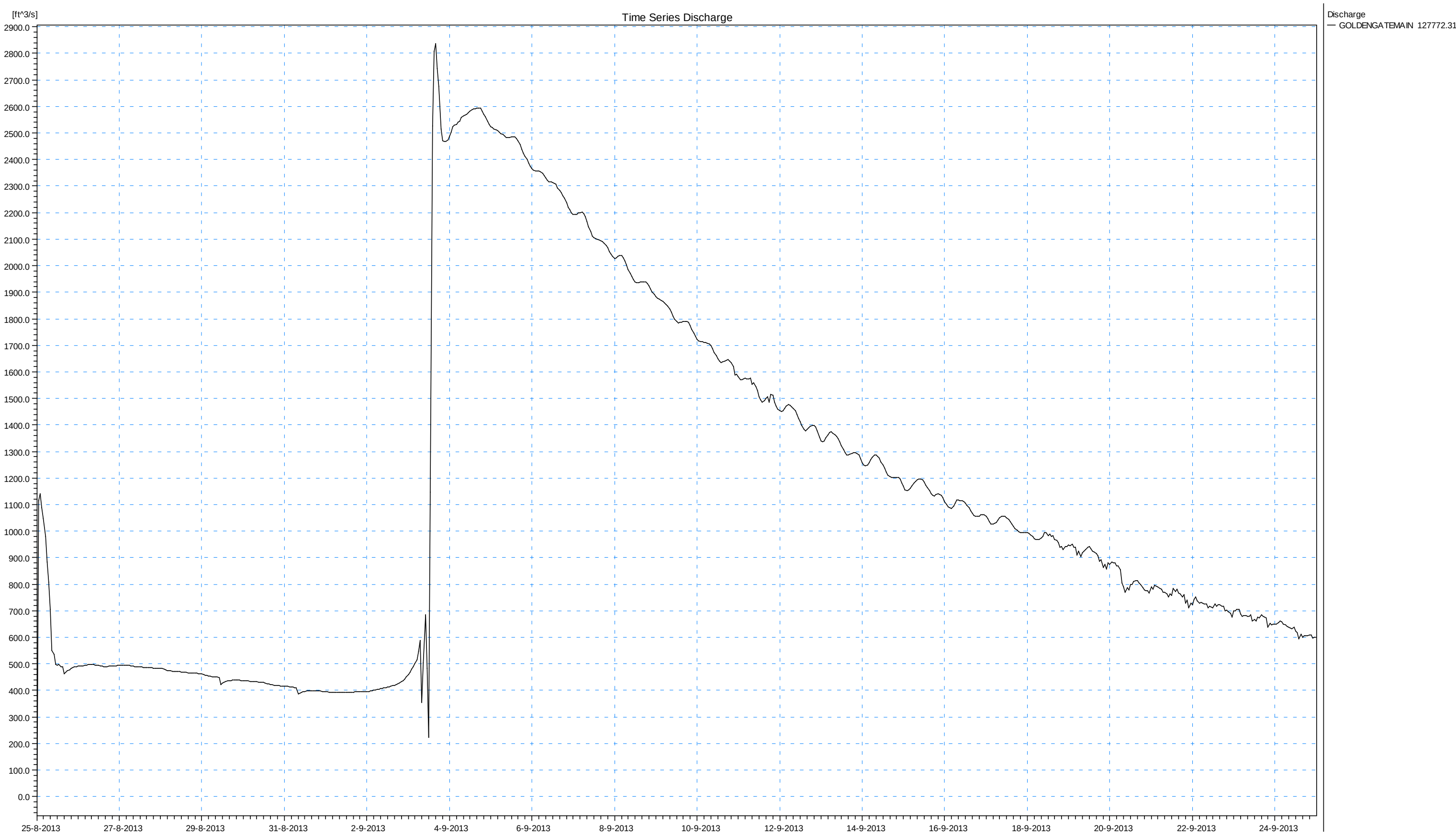

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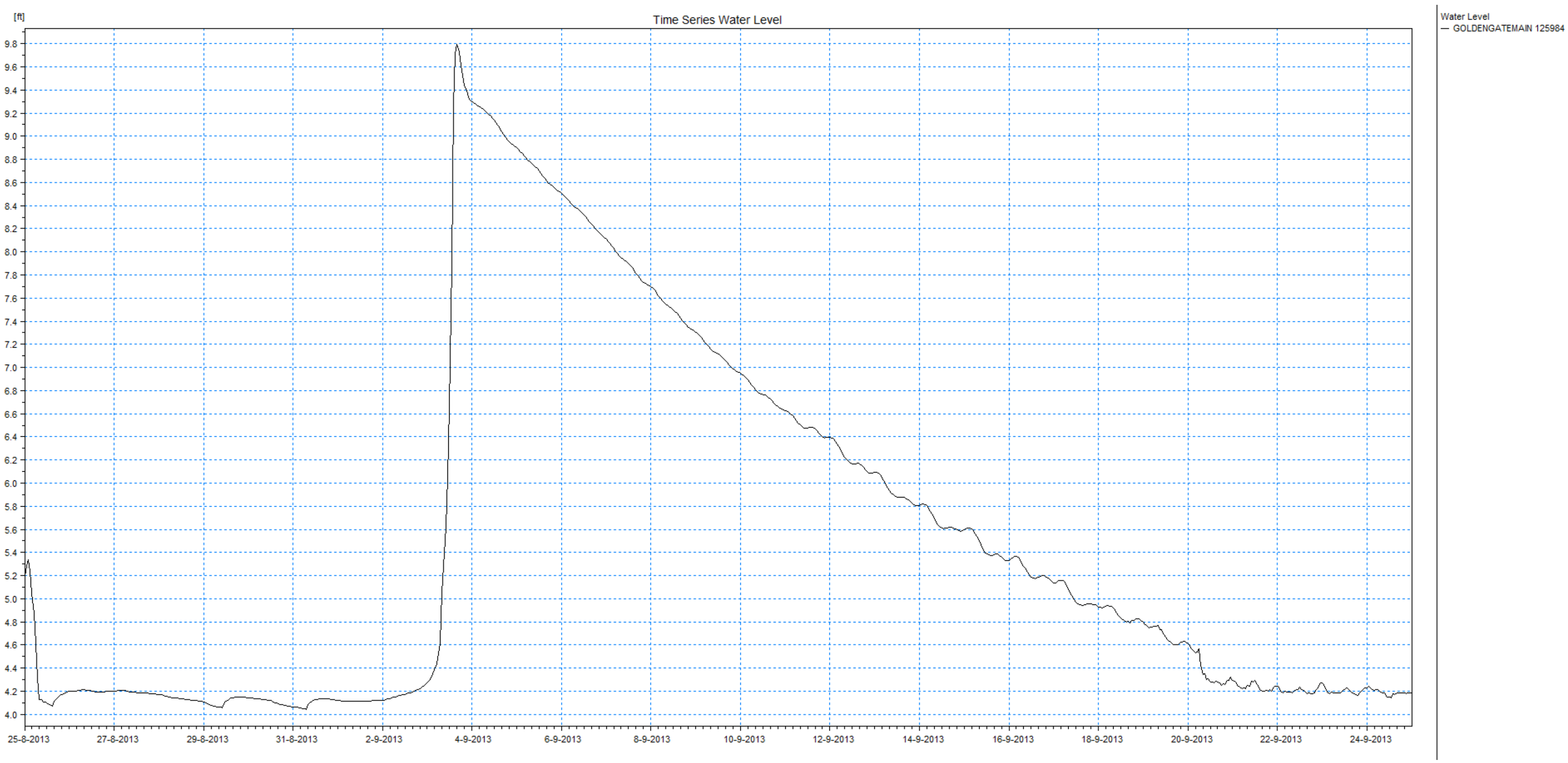

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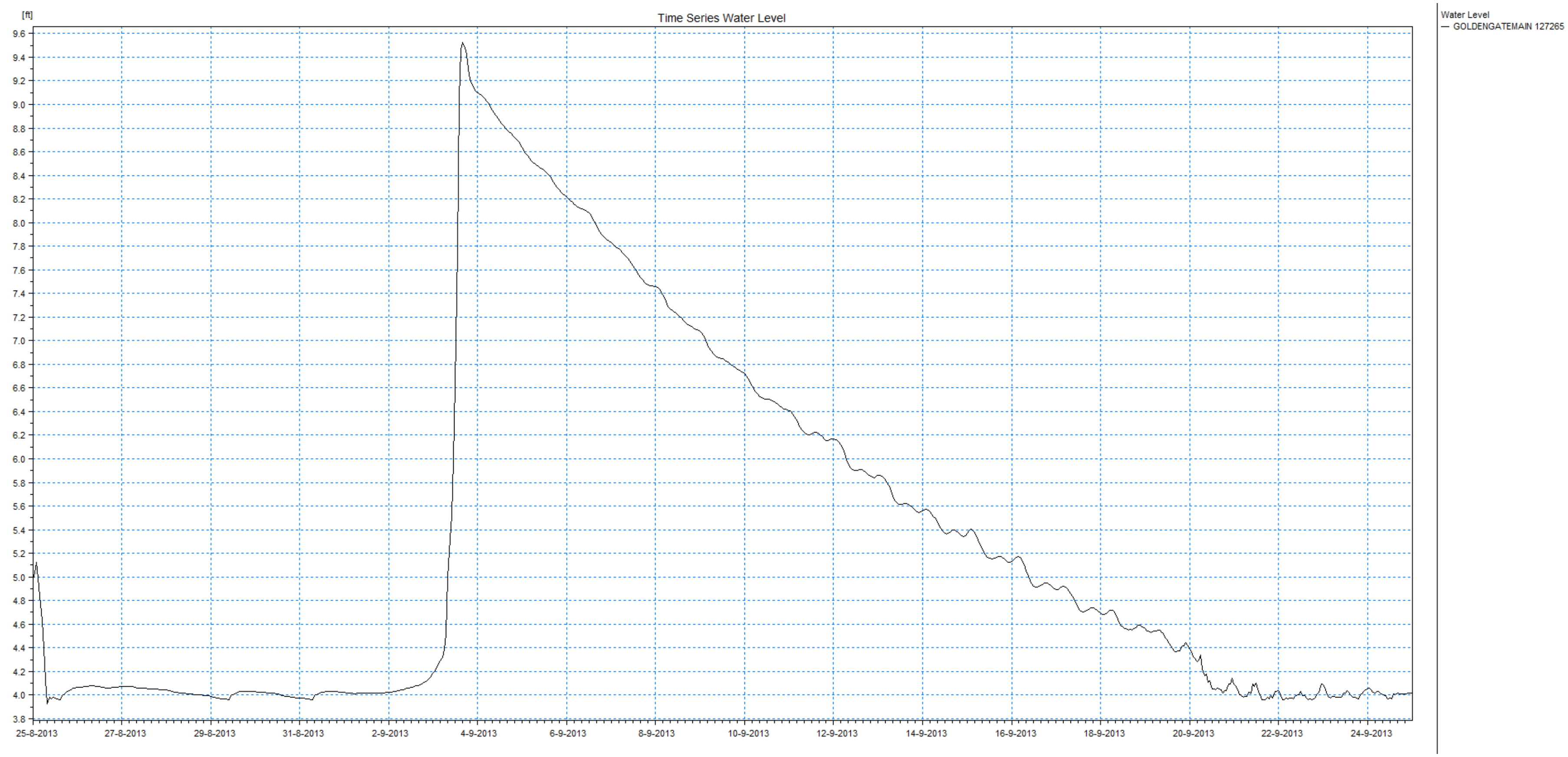

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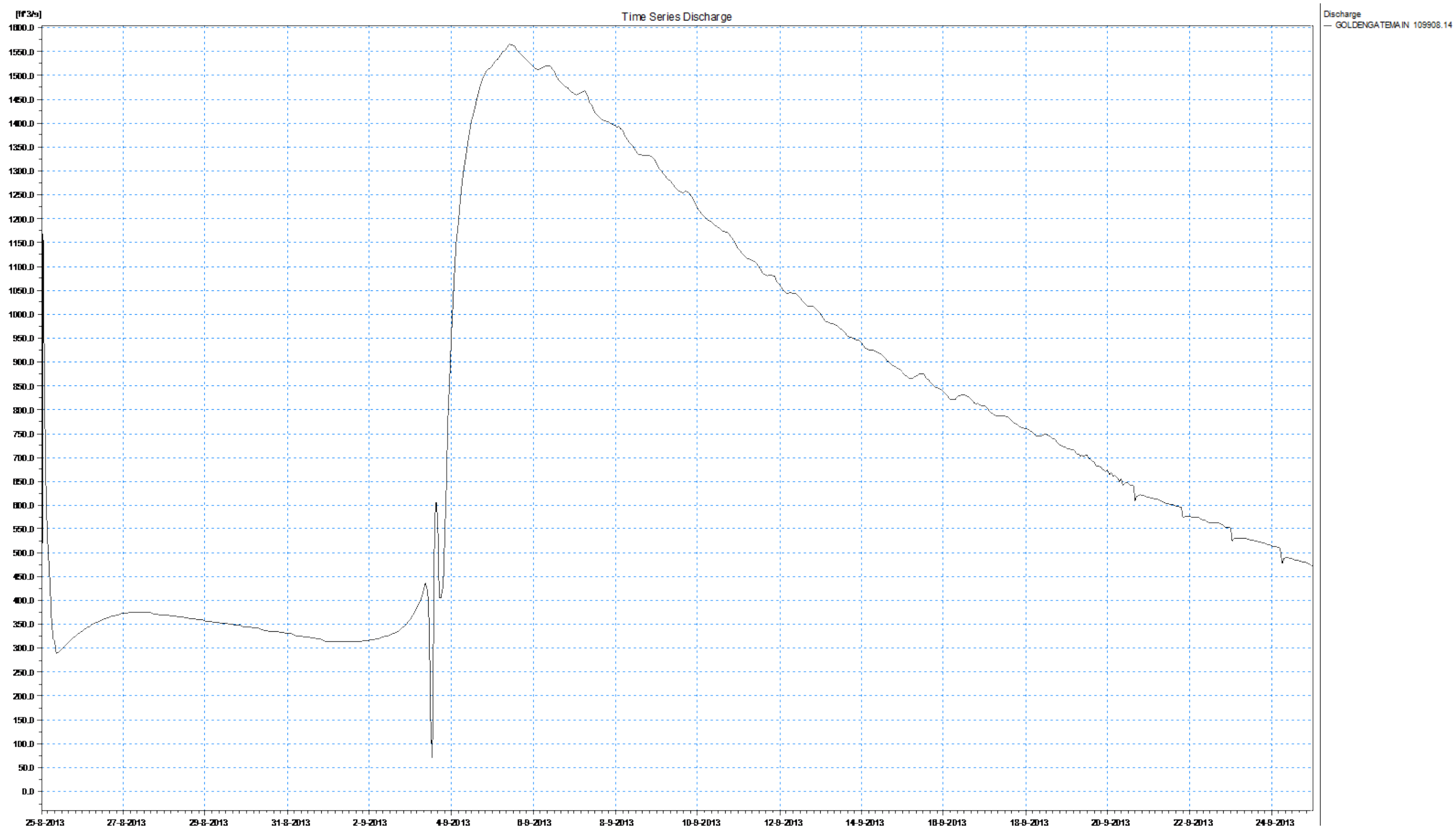

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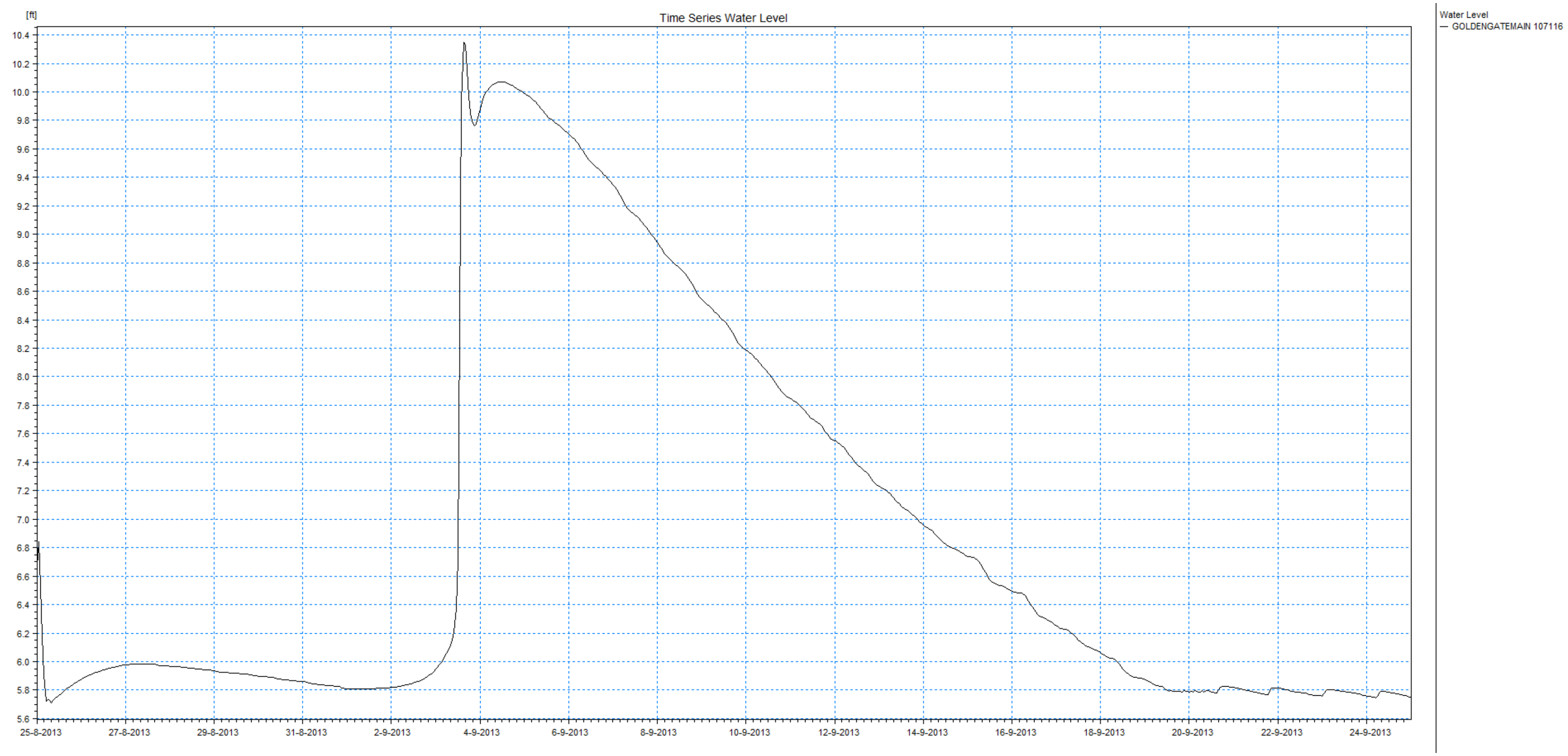

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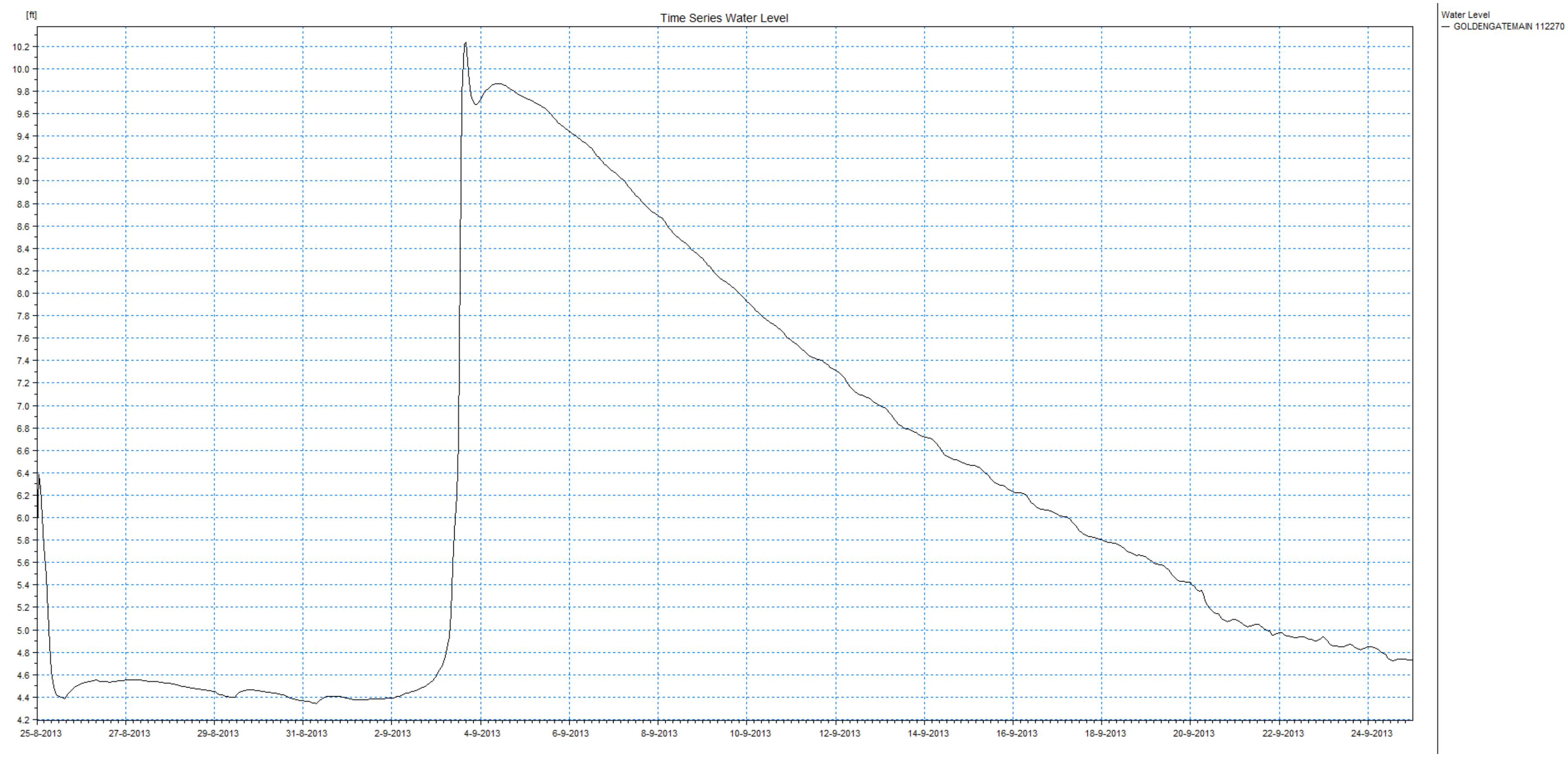

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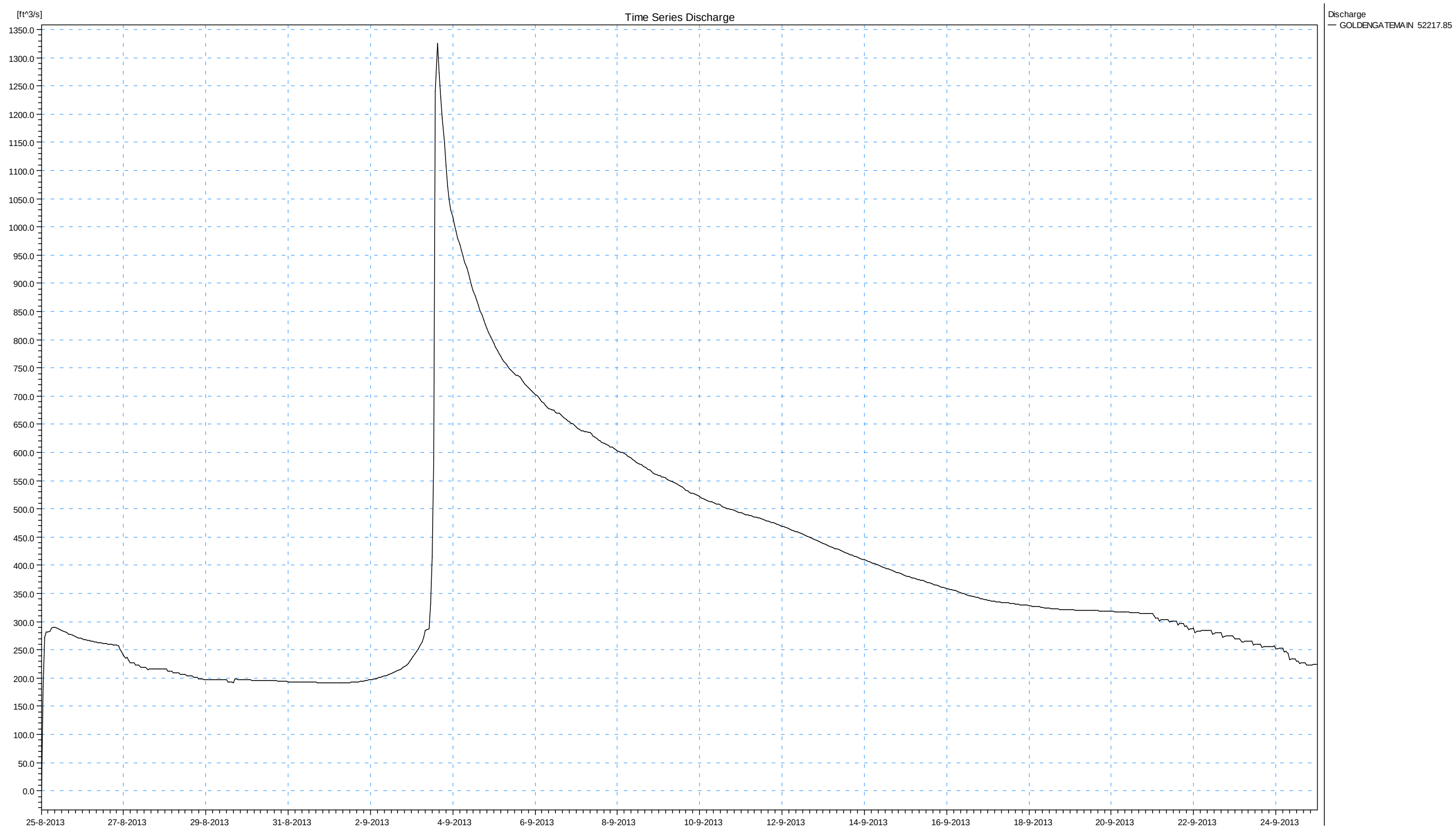
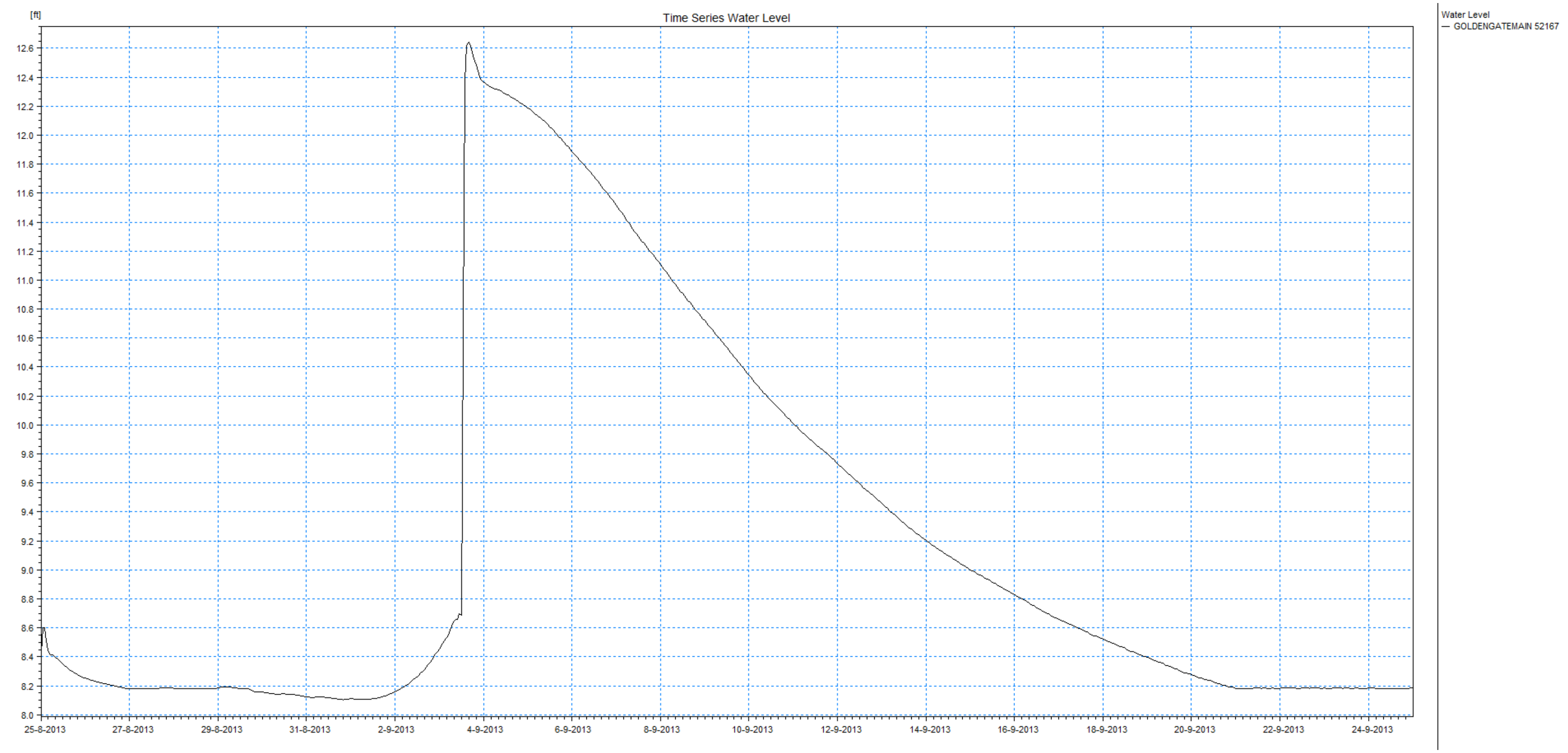


Figure 29. GG 4 Discharge (cfs)



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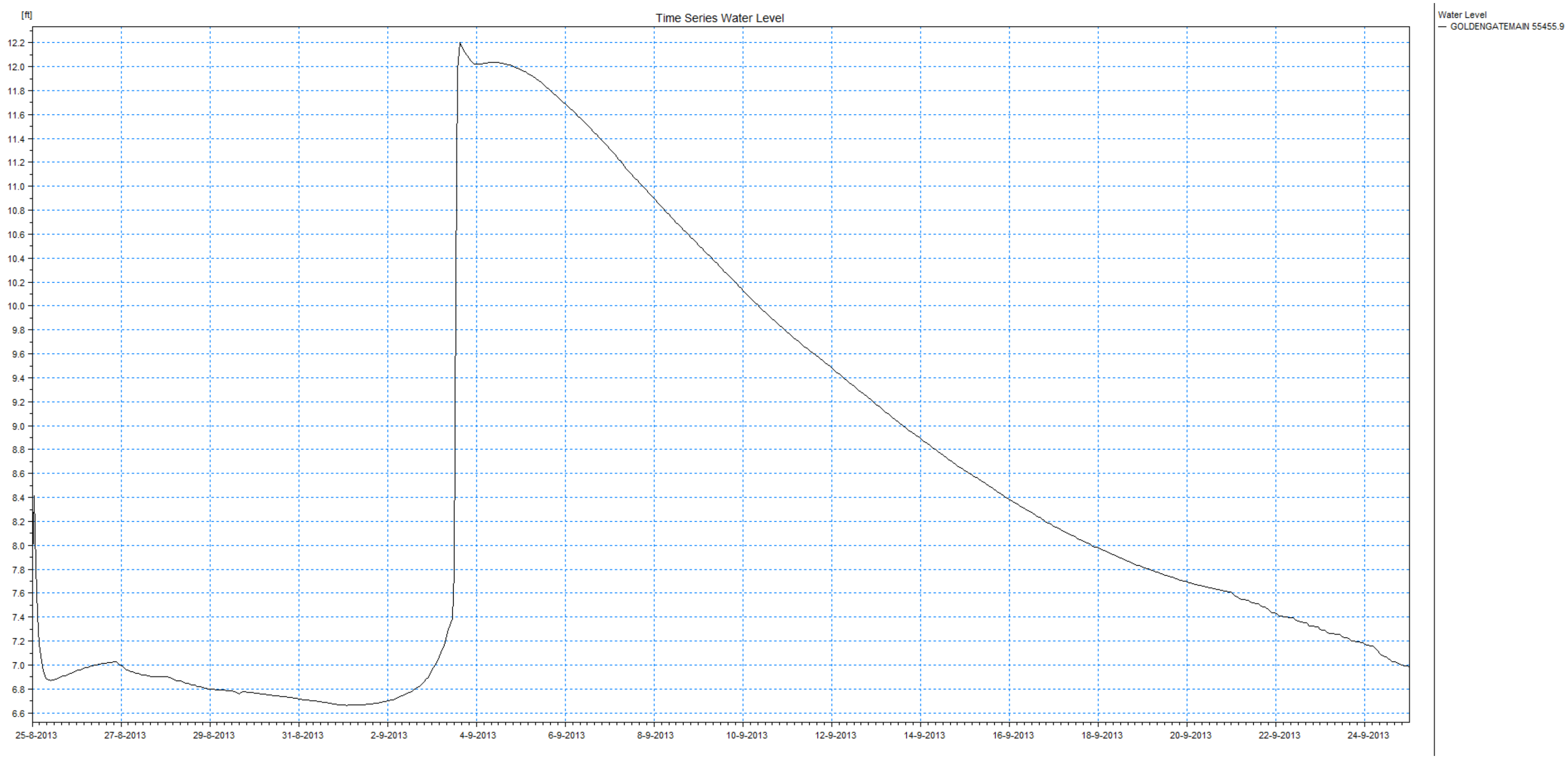
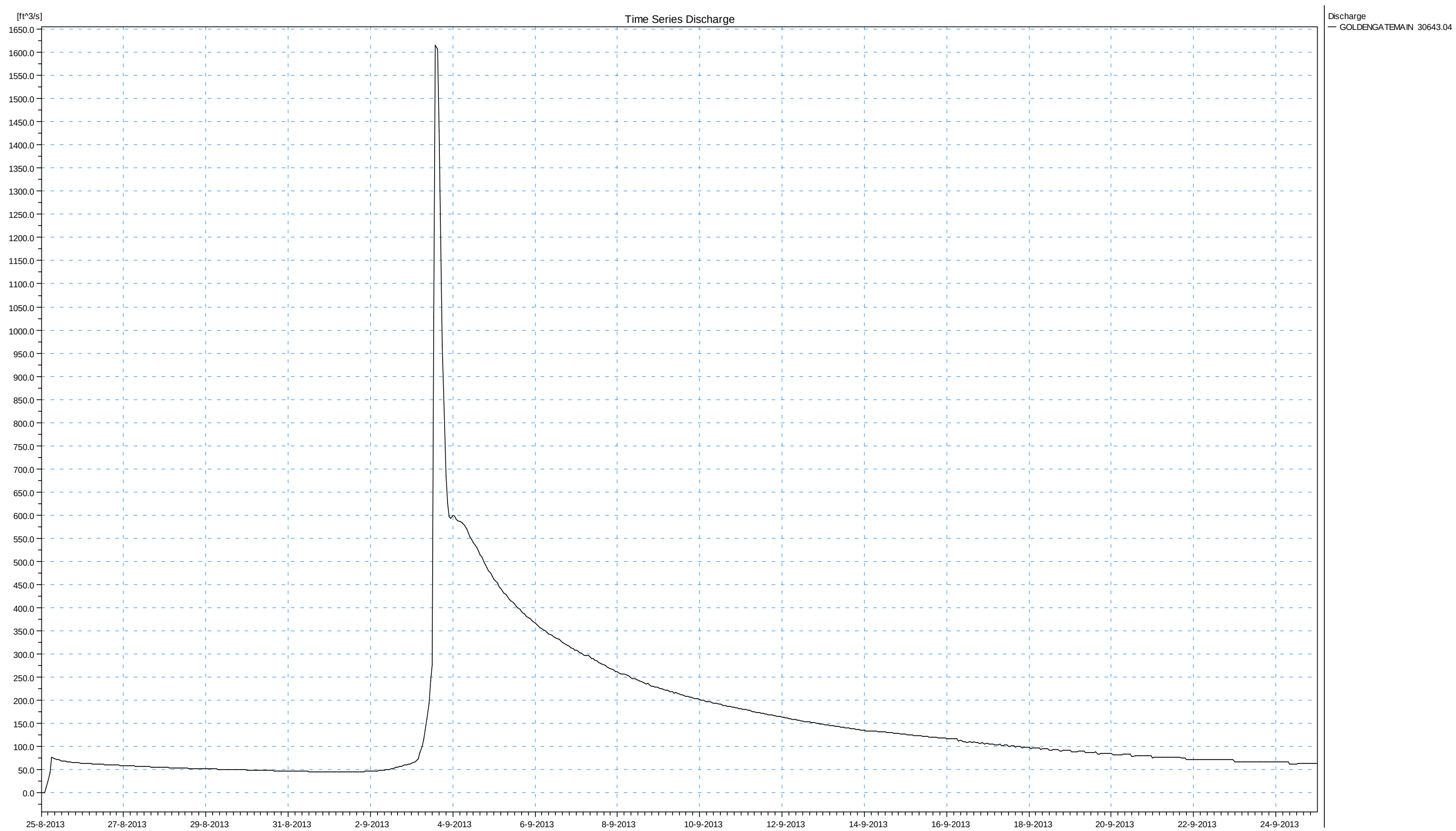


Figure 31. GG 4 Tailwater Stage (ft)



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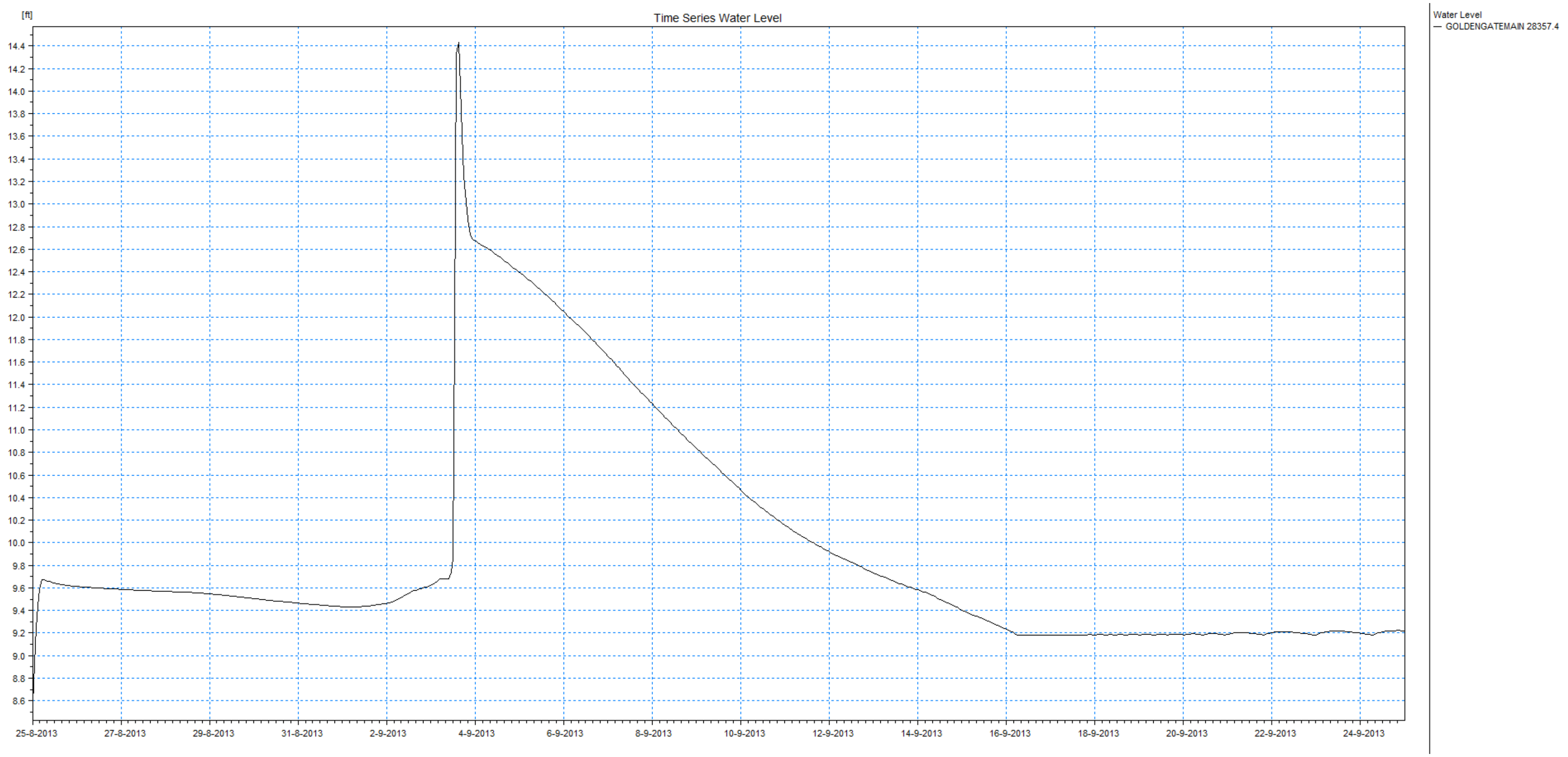

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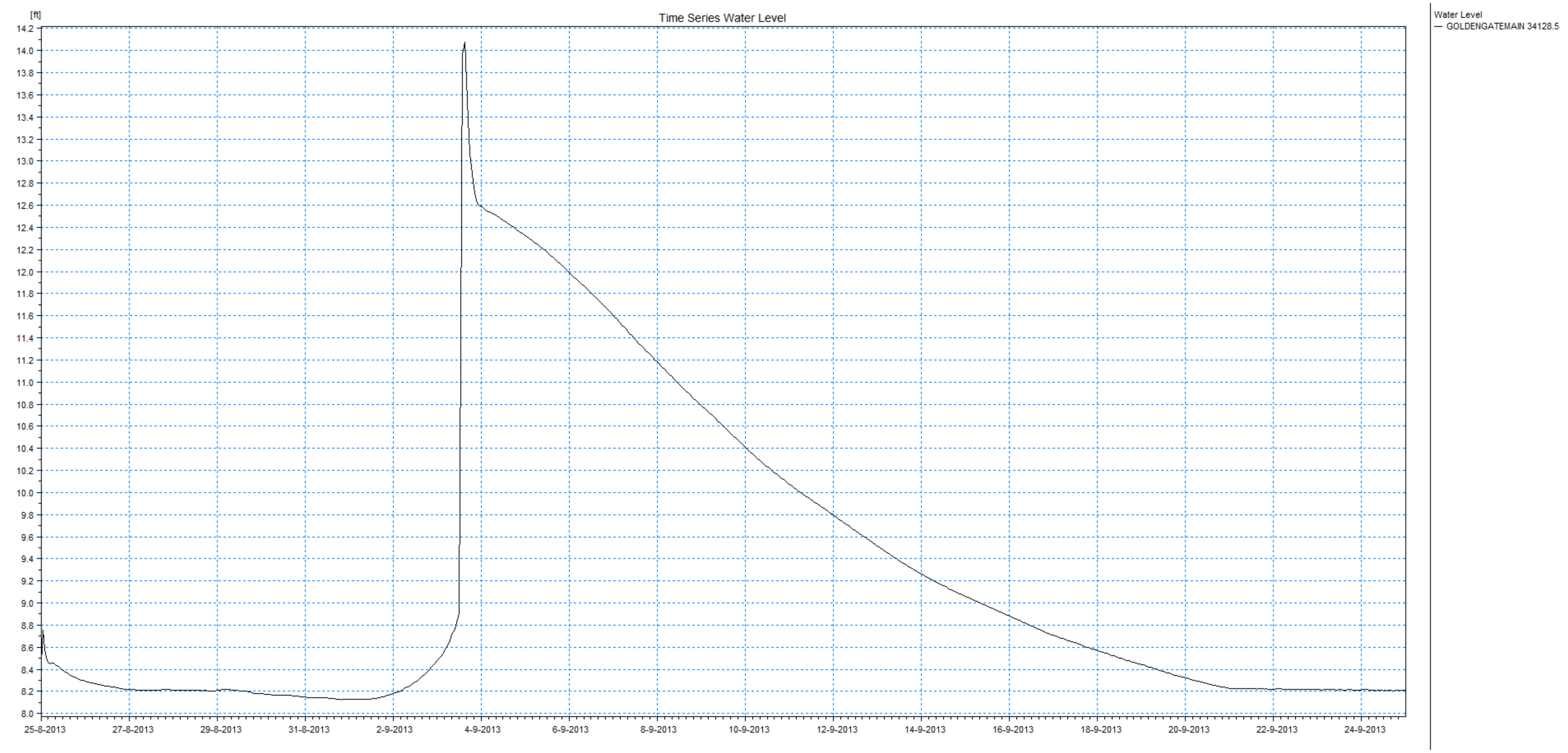

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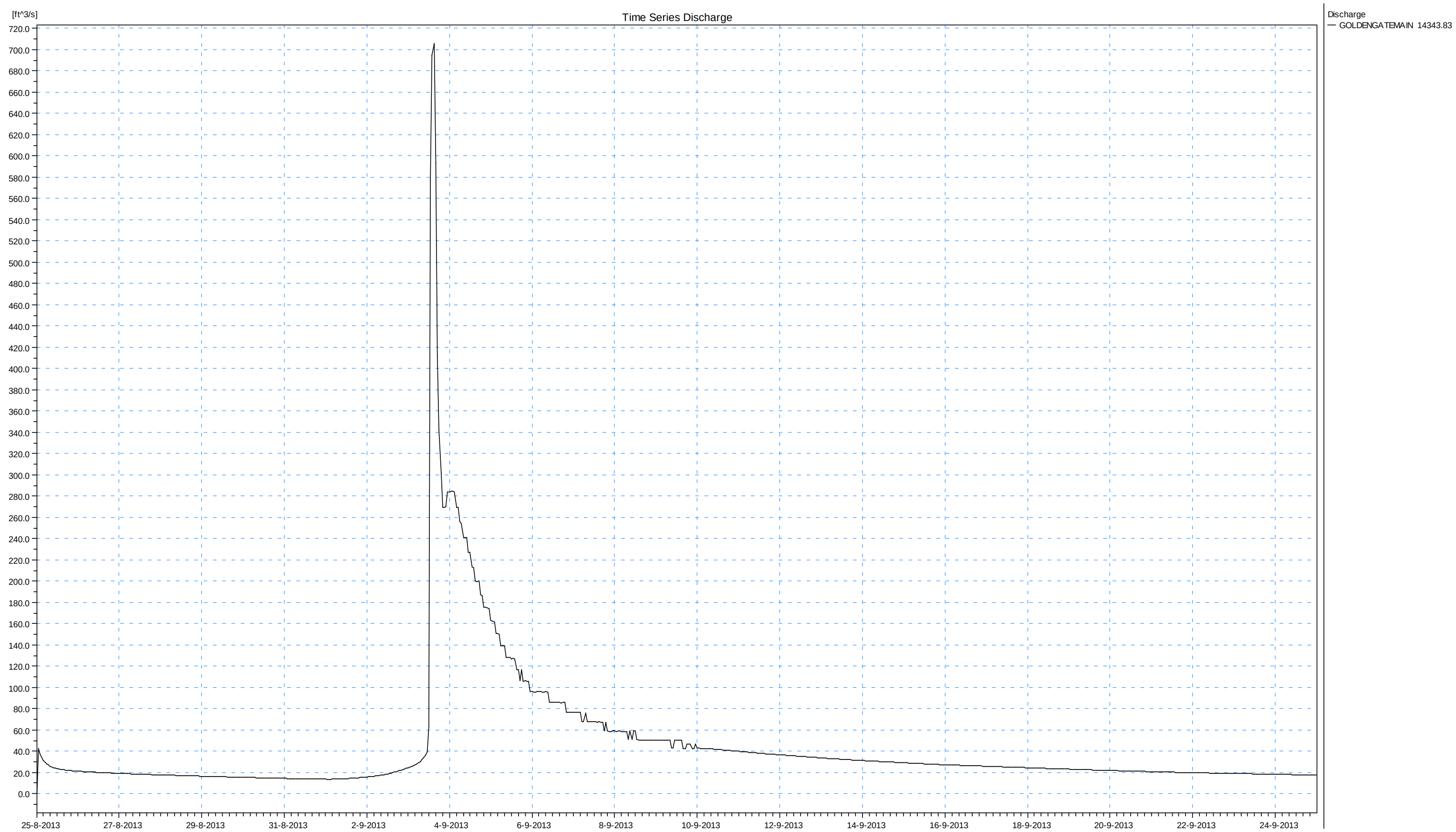

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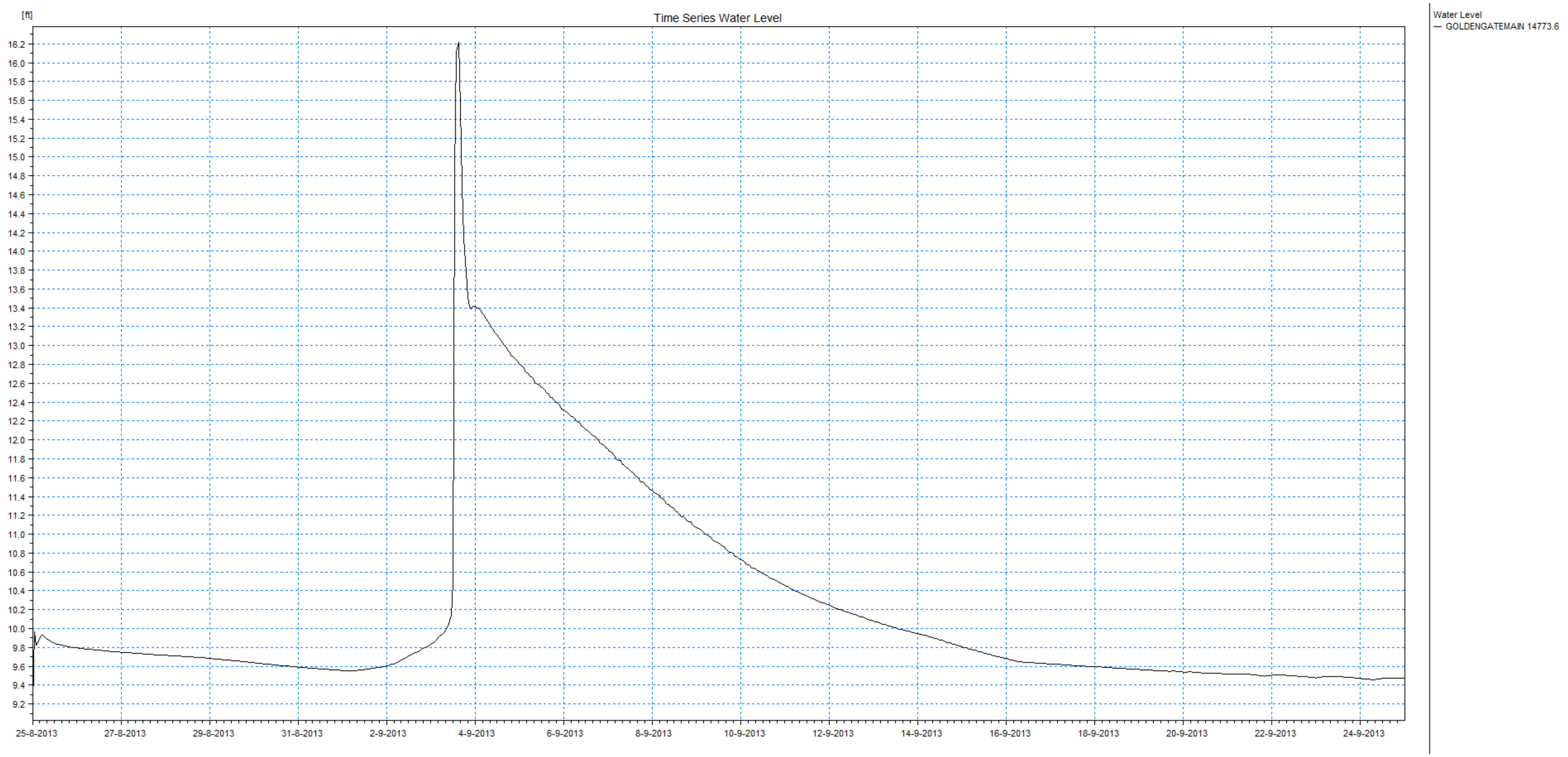

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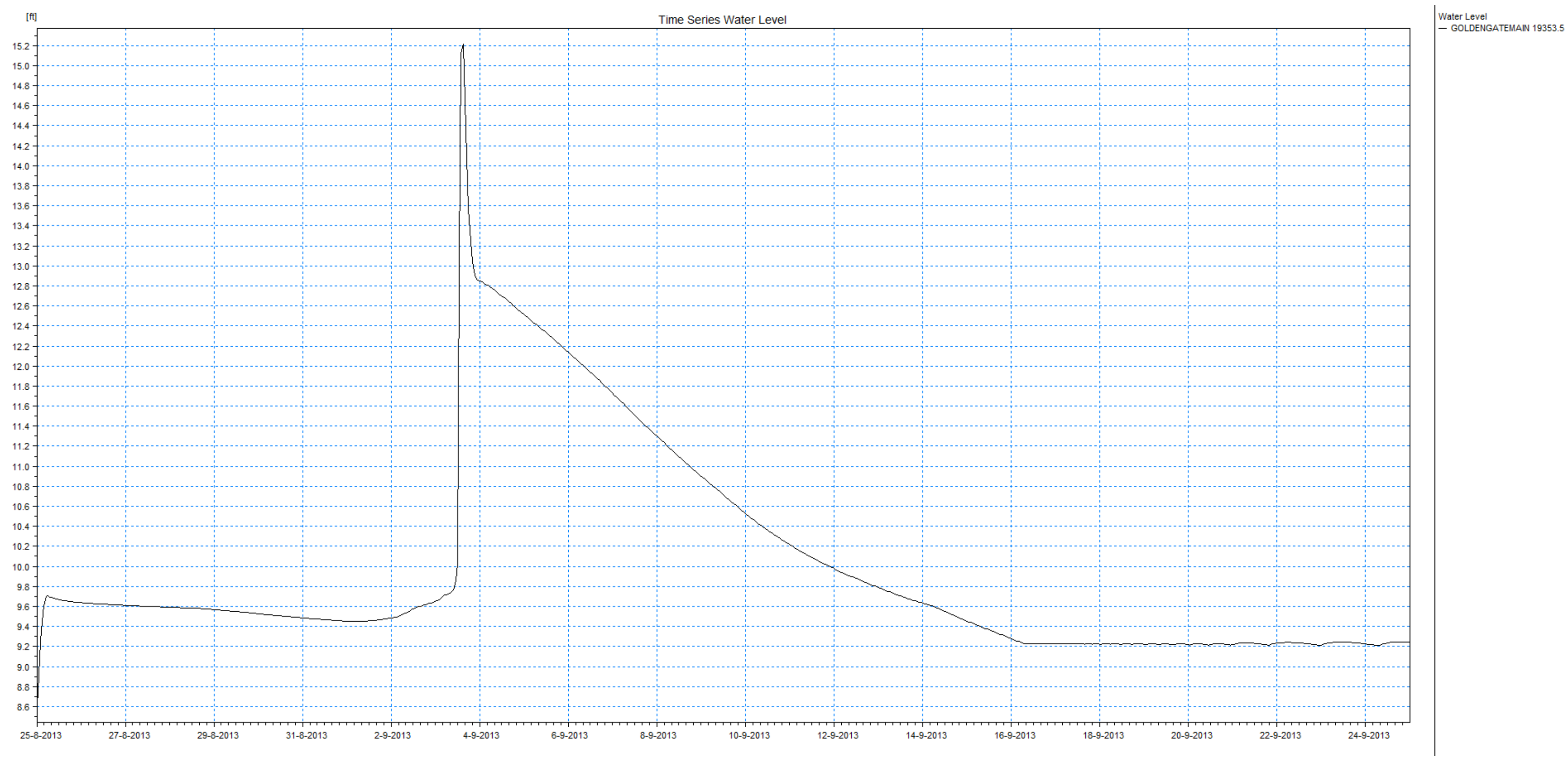

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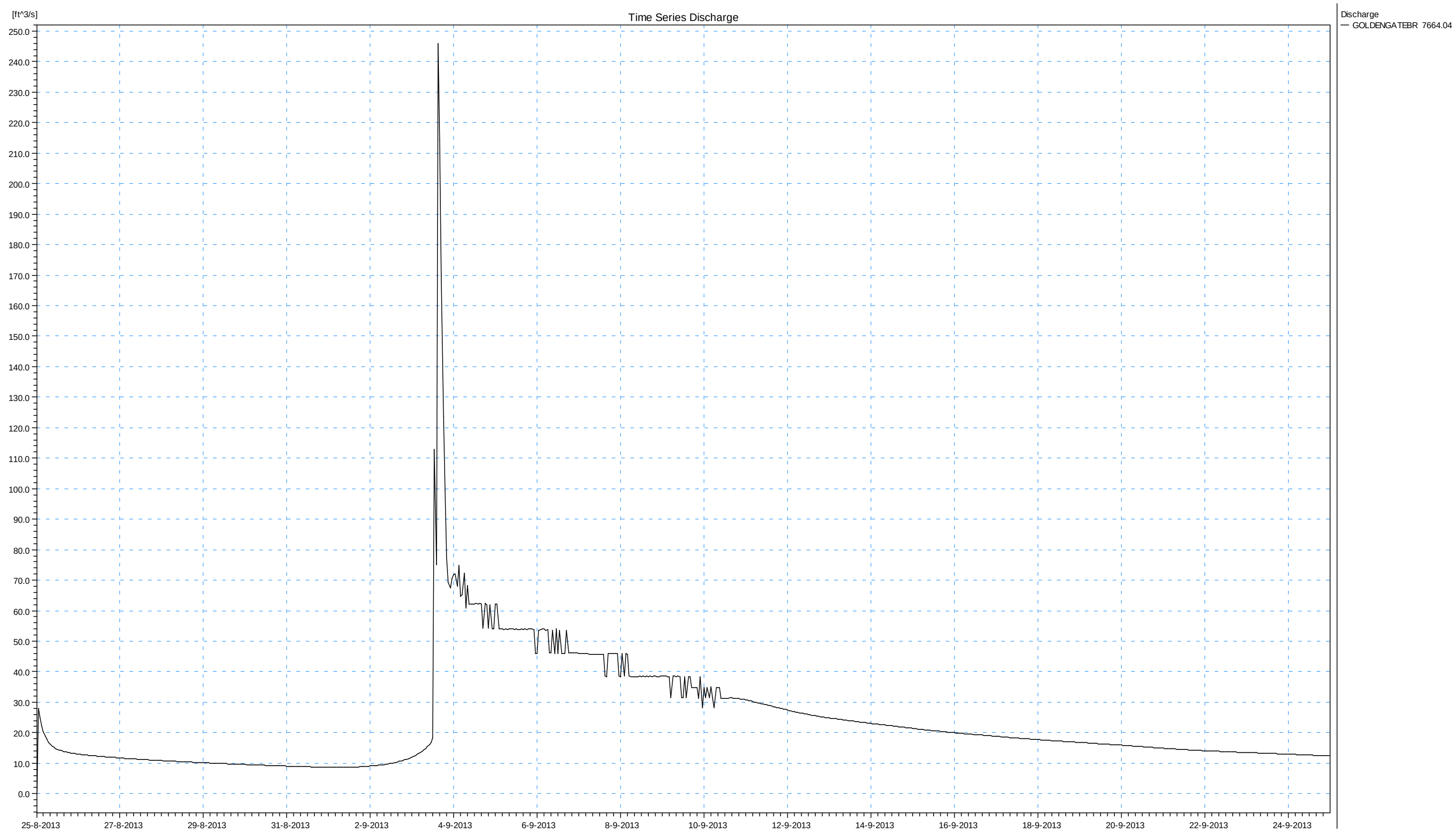

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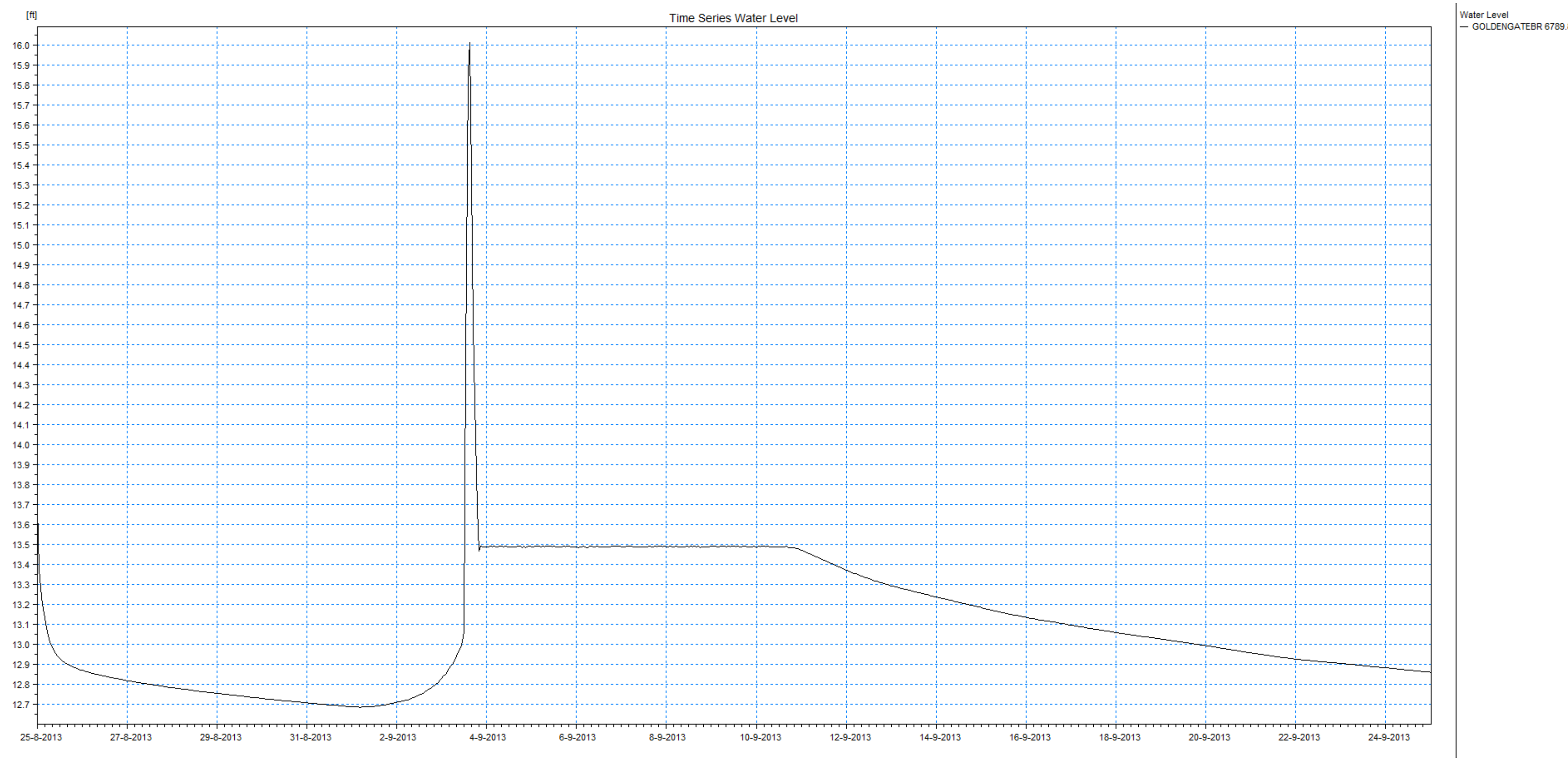

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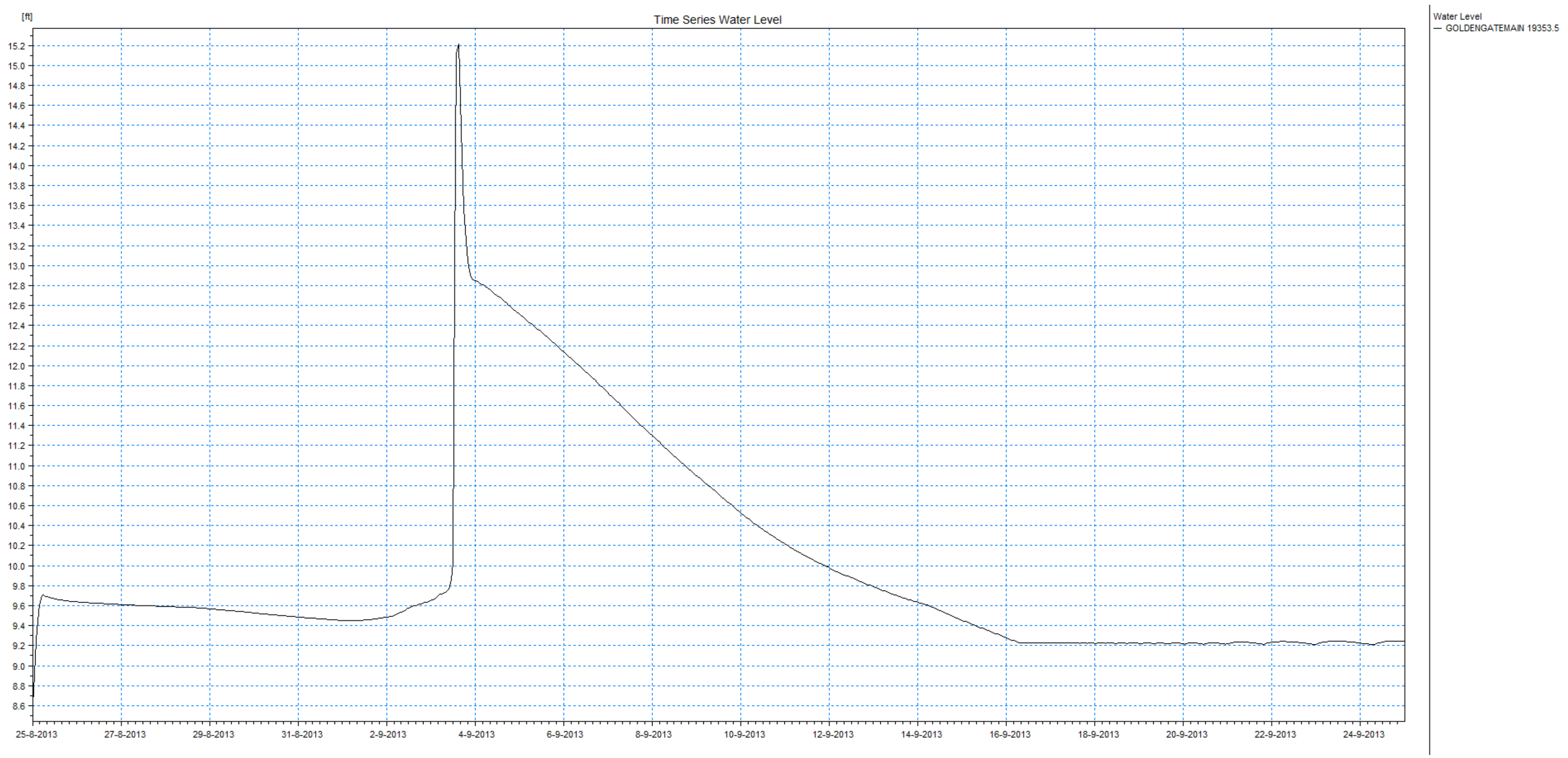

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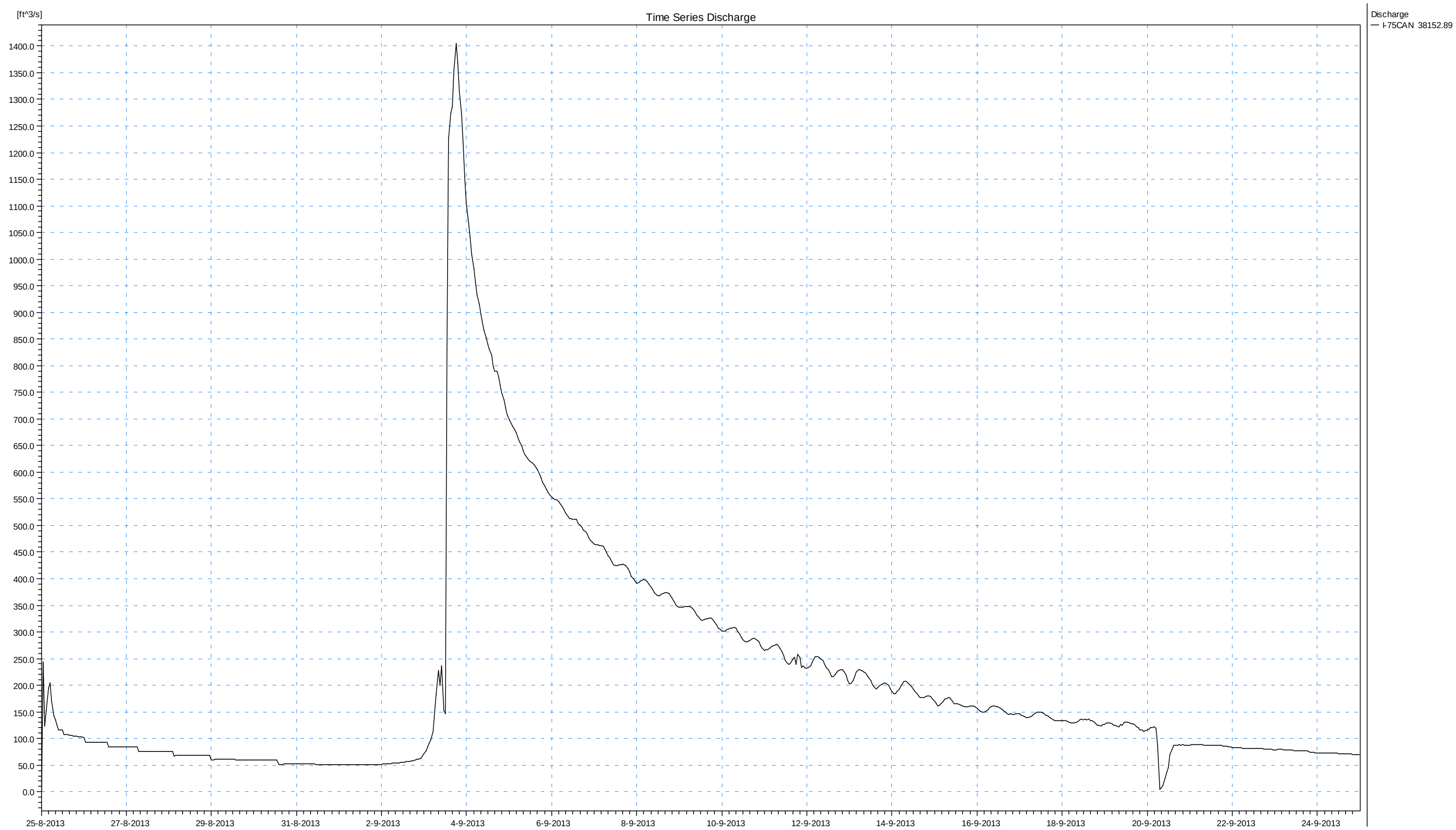

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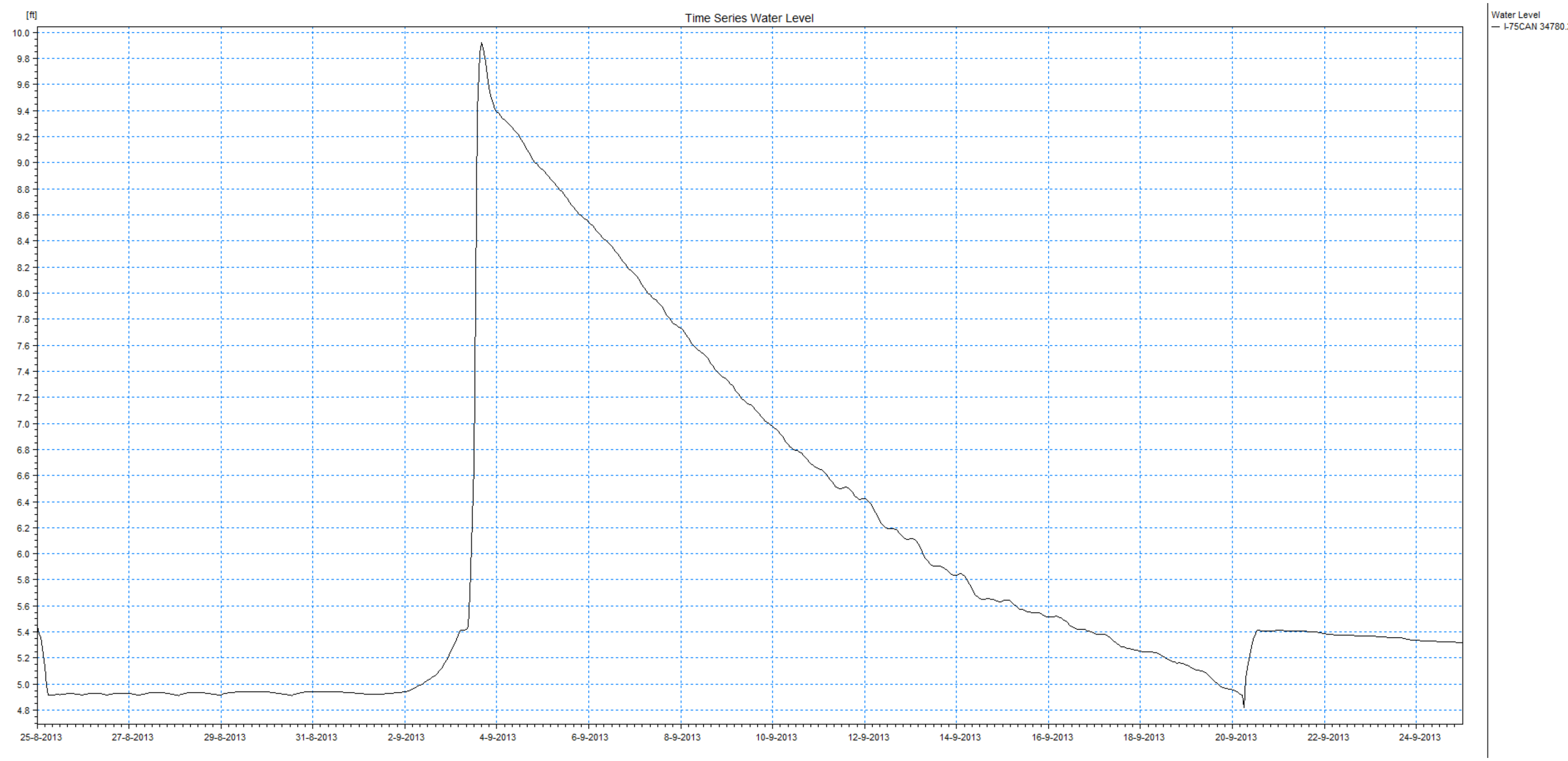

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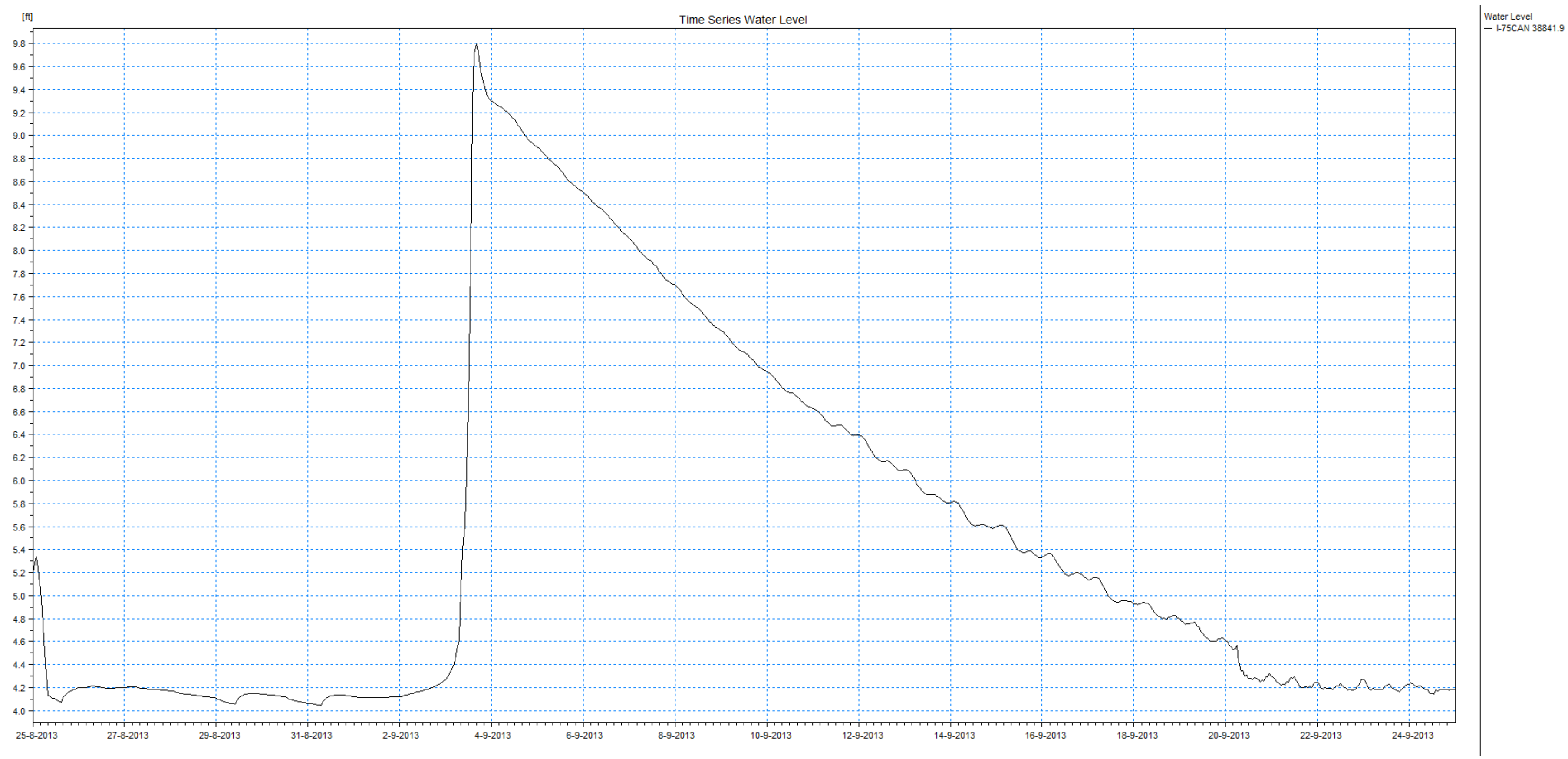

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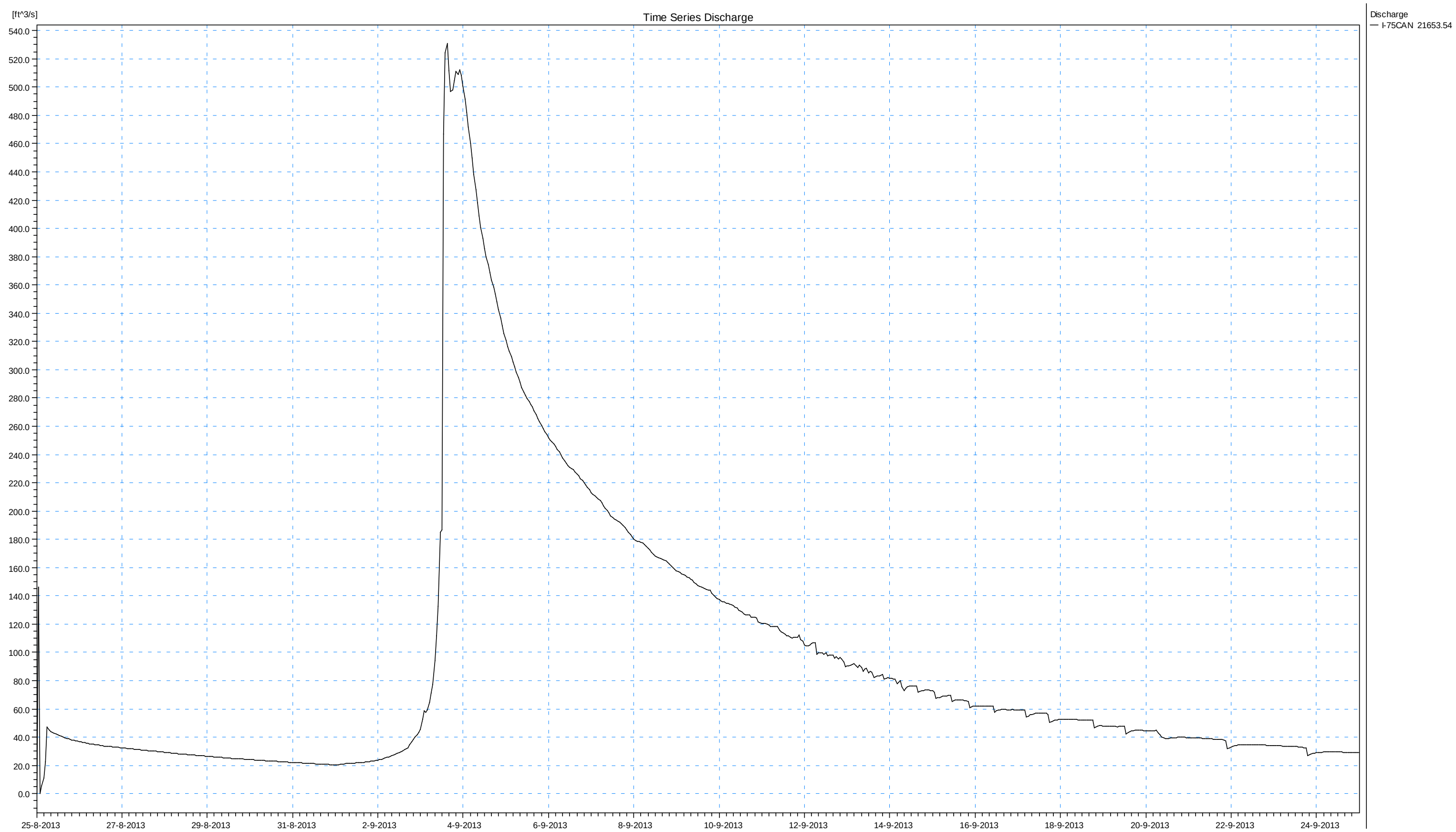

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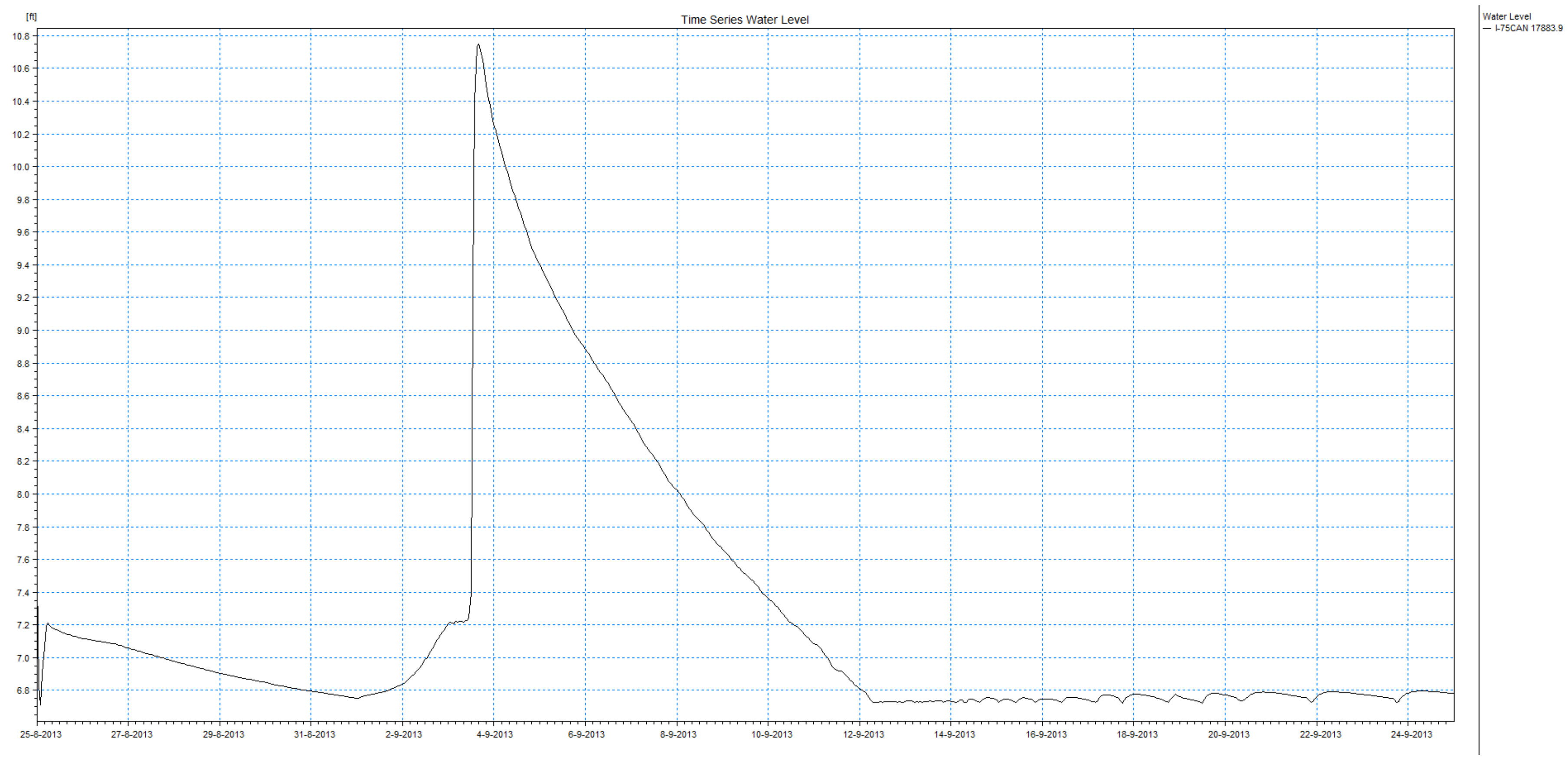

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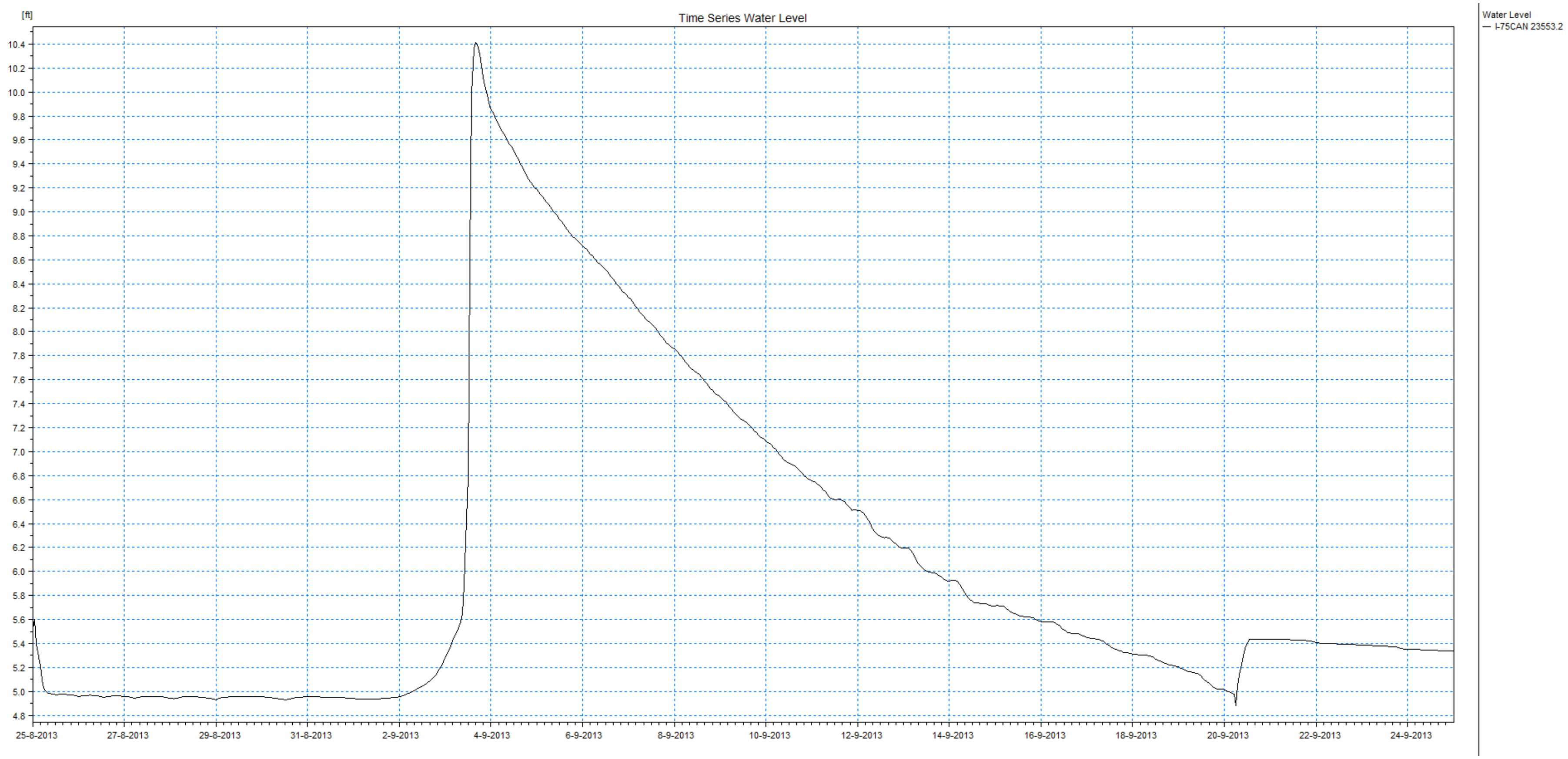

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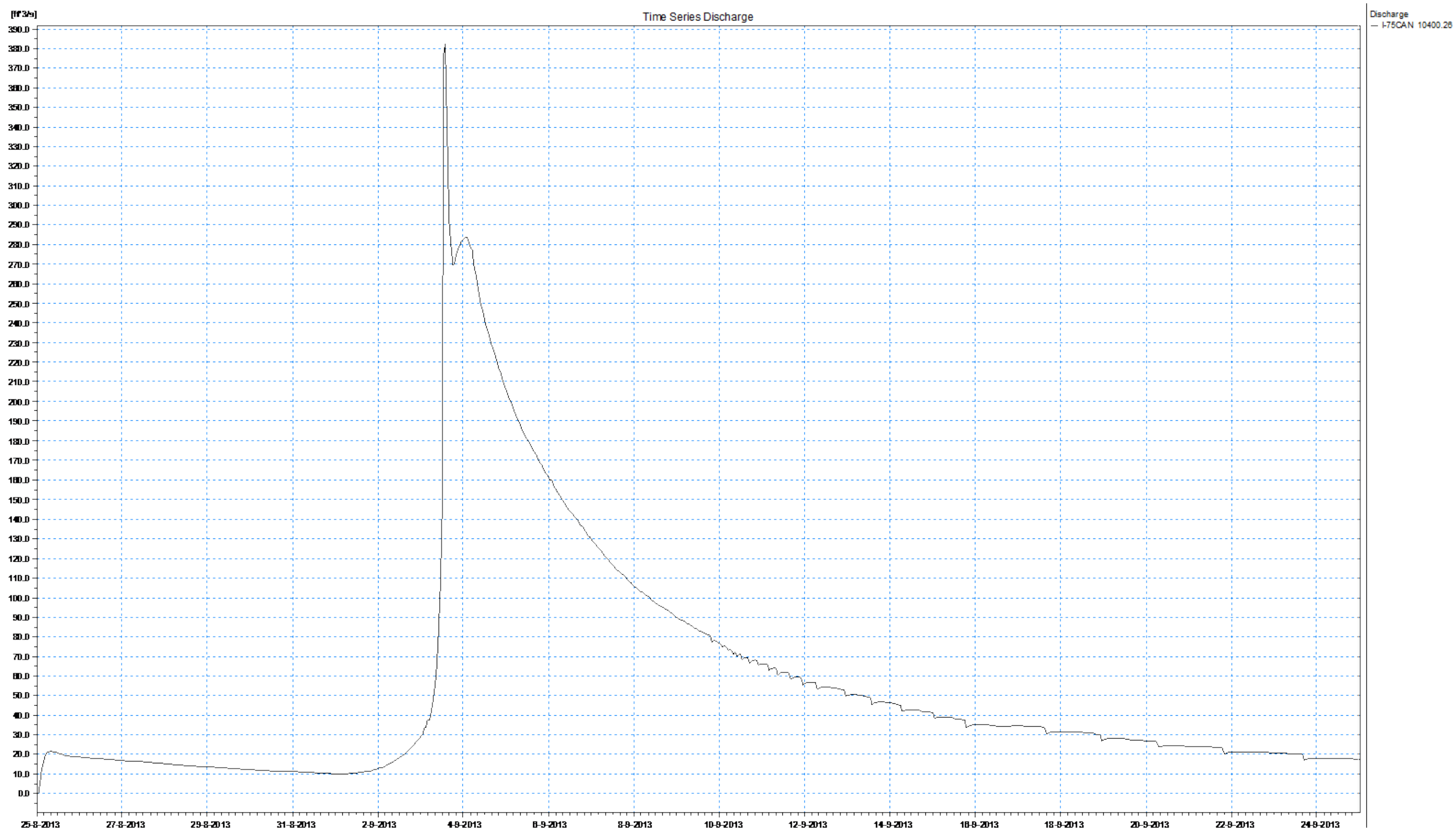

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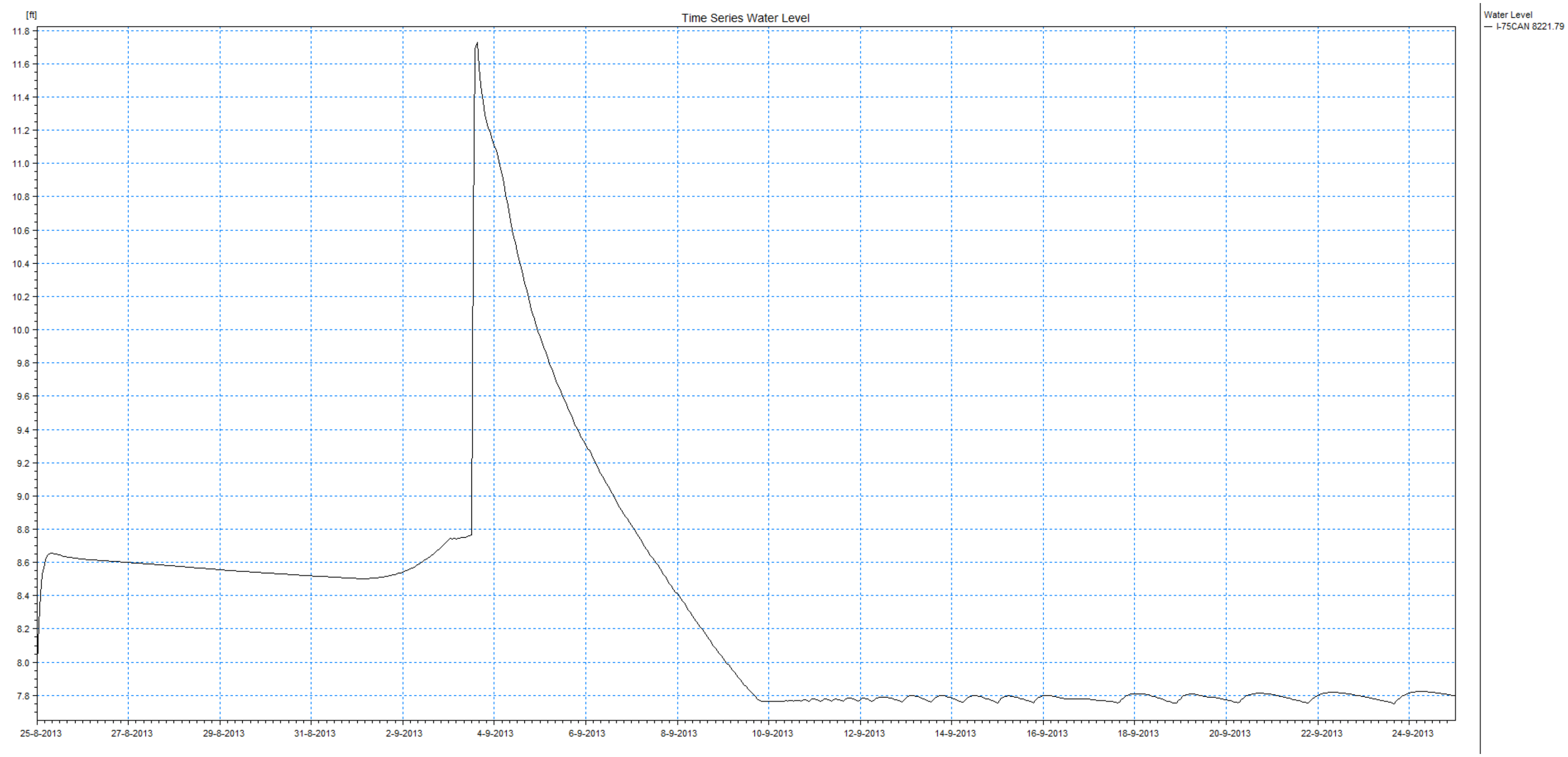

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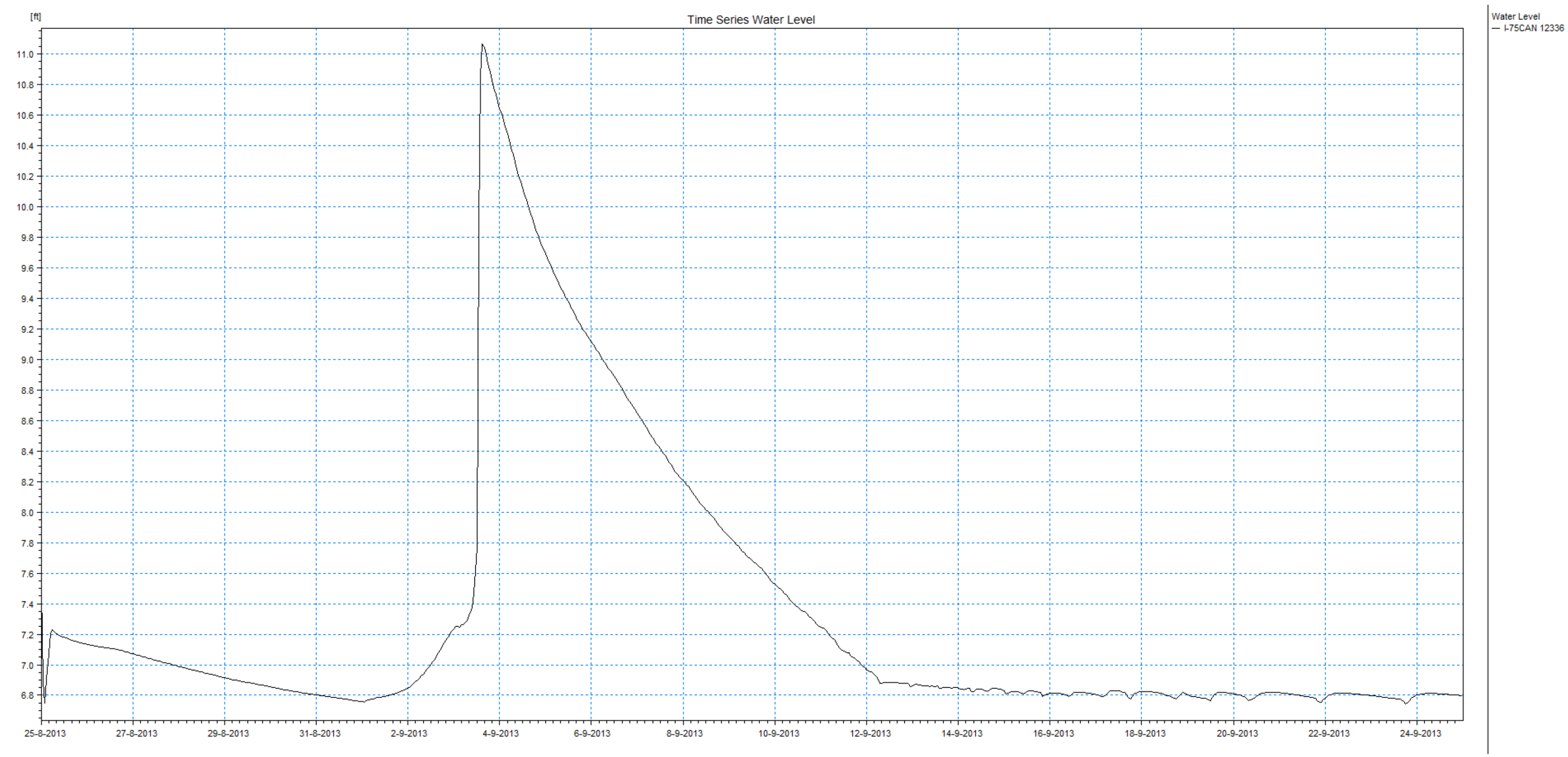

Figure 49. I75W3 Tailwater Stage (ft)

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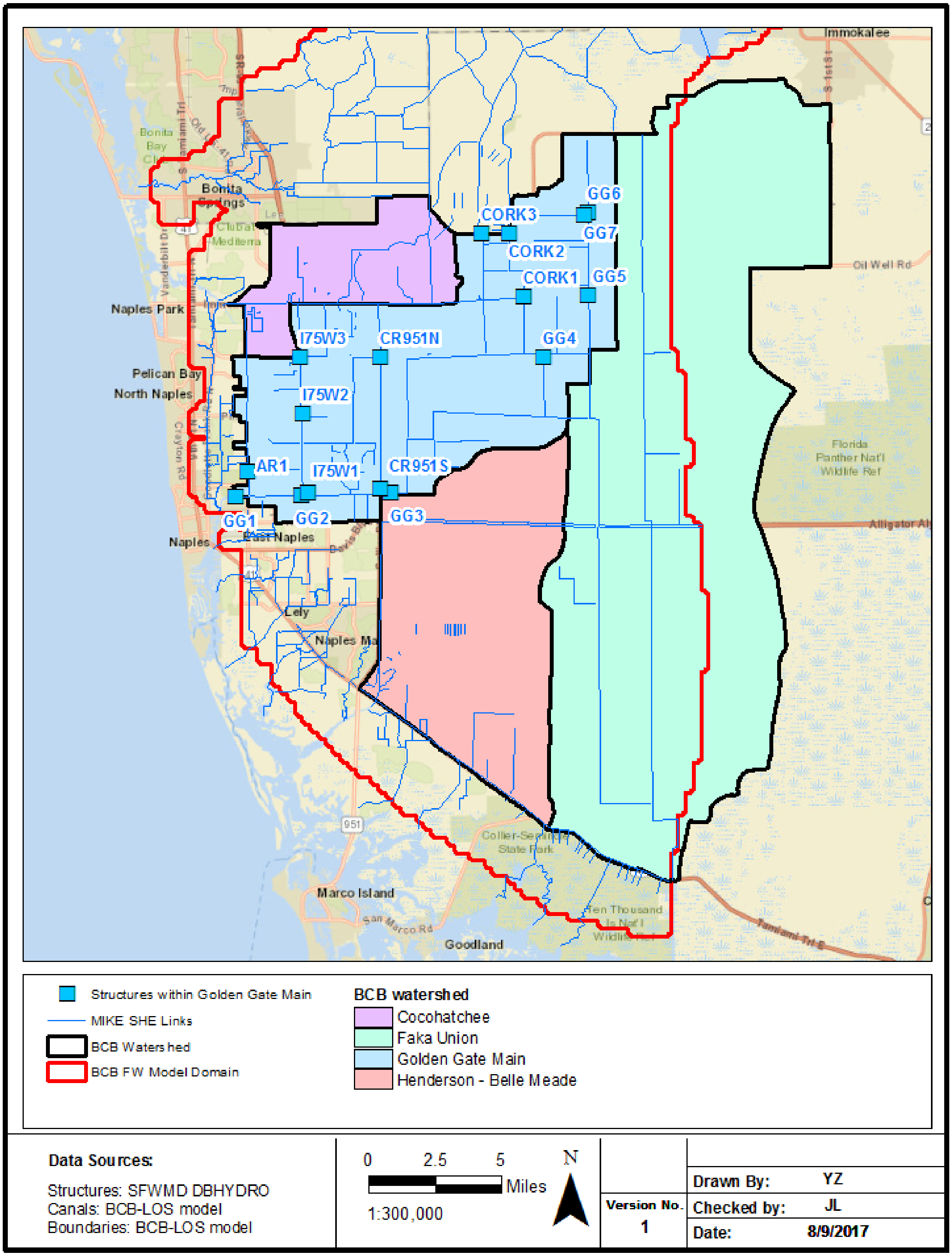

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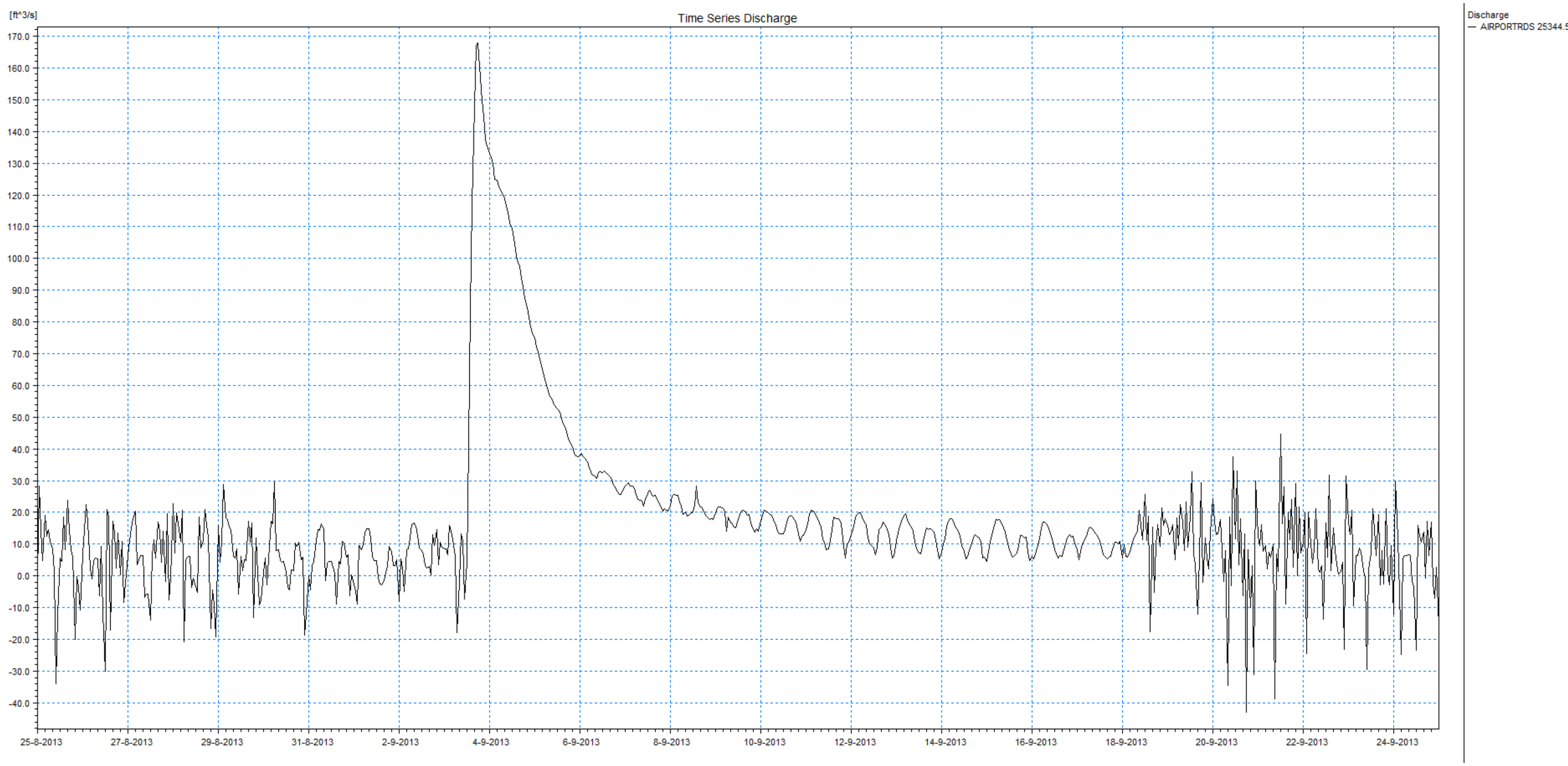

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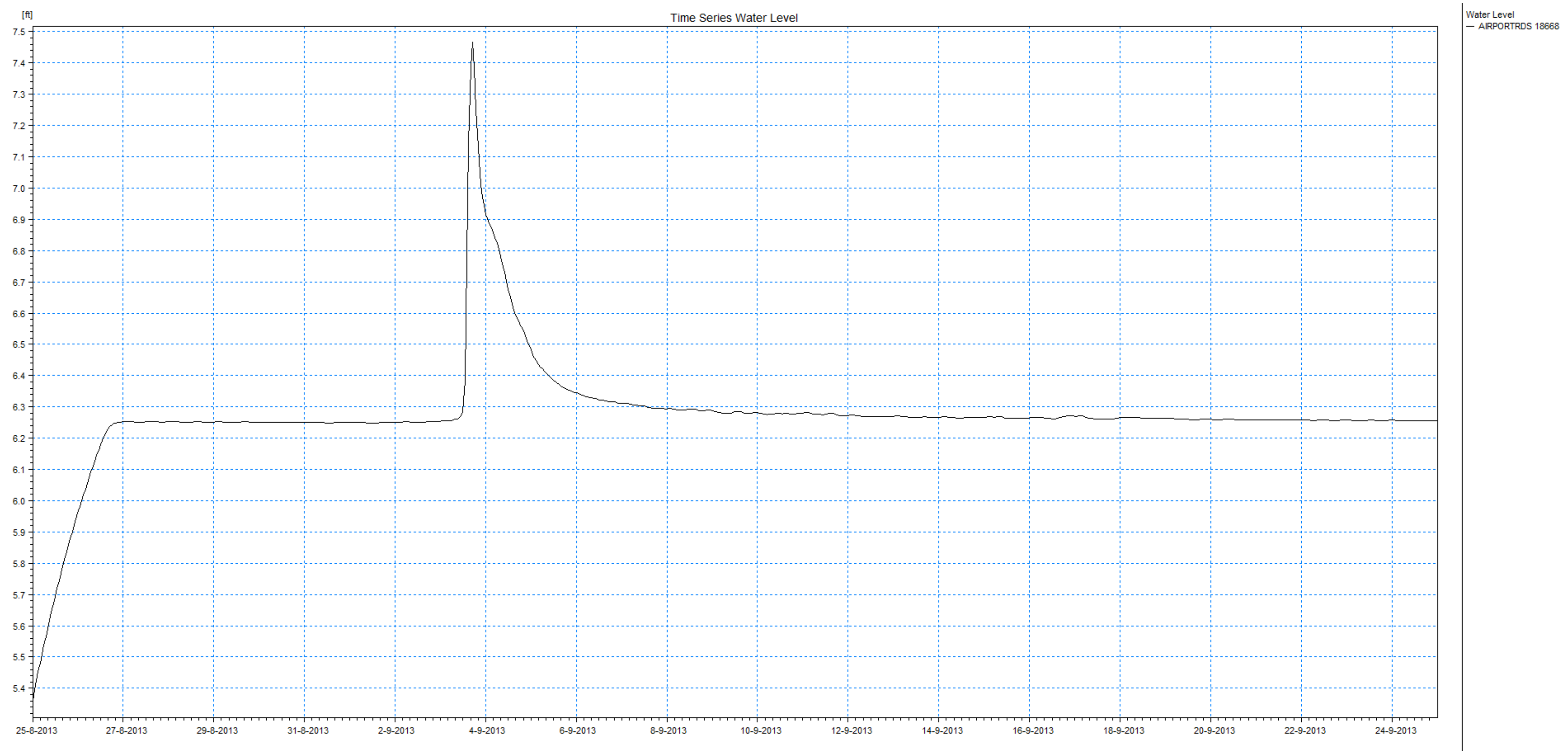

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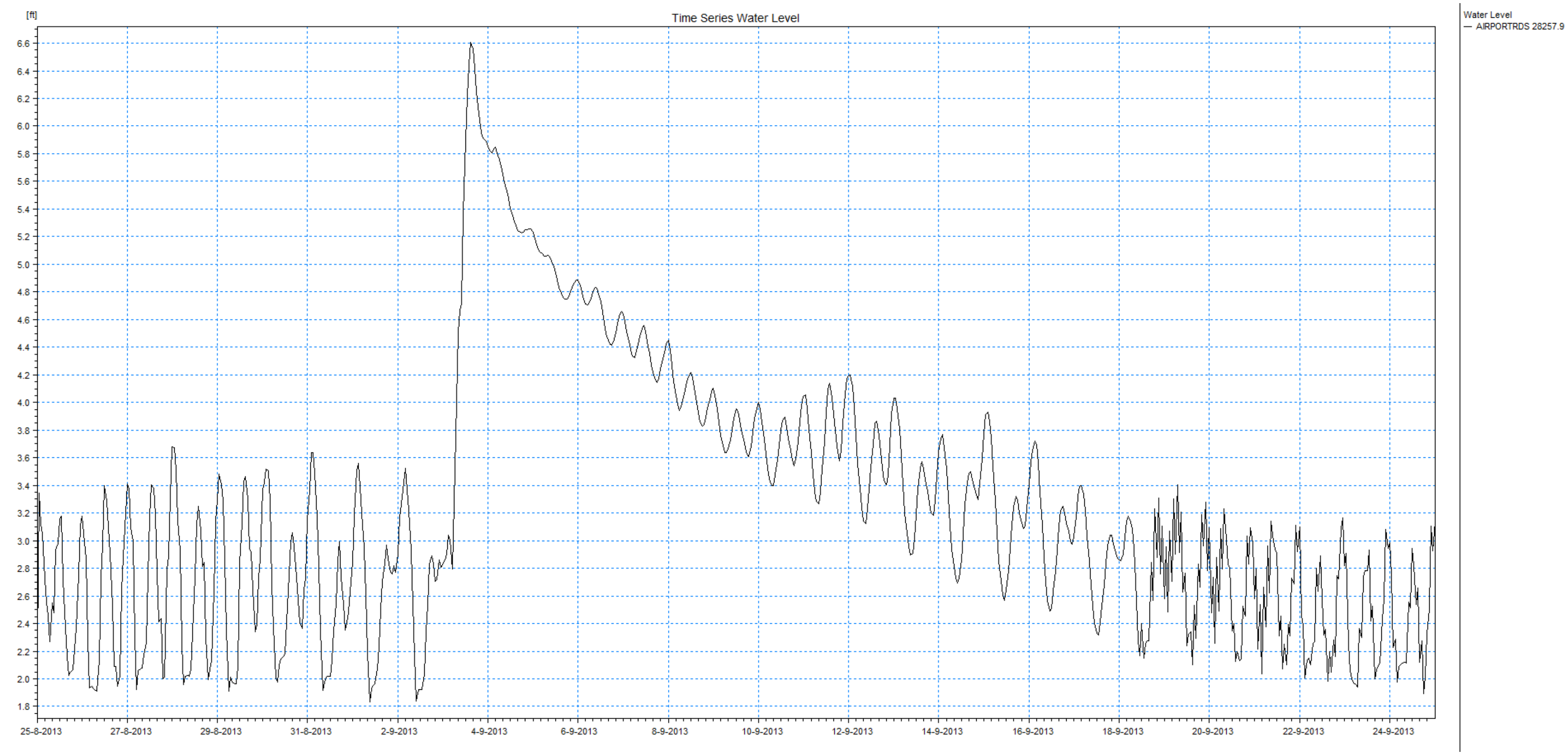

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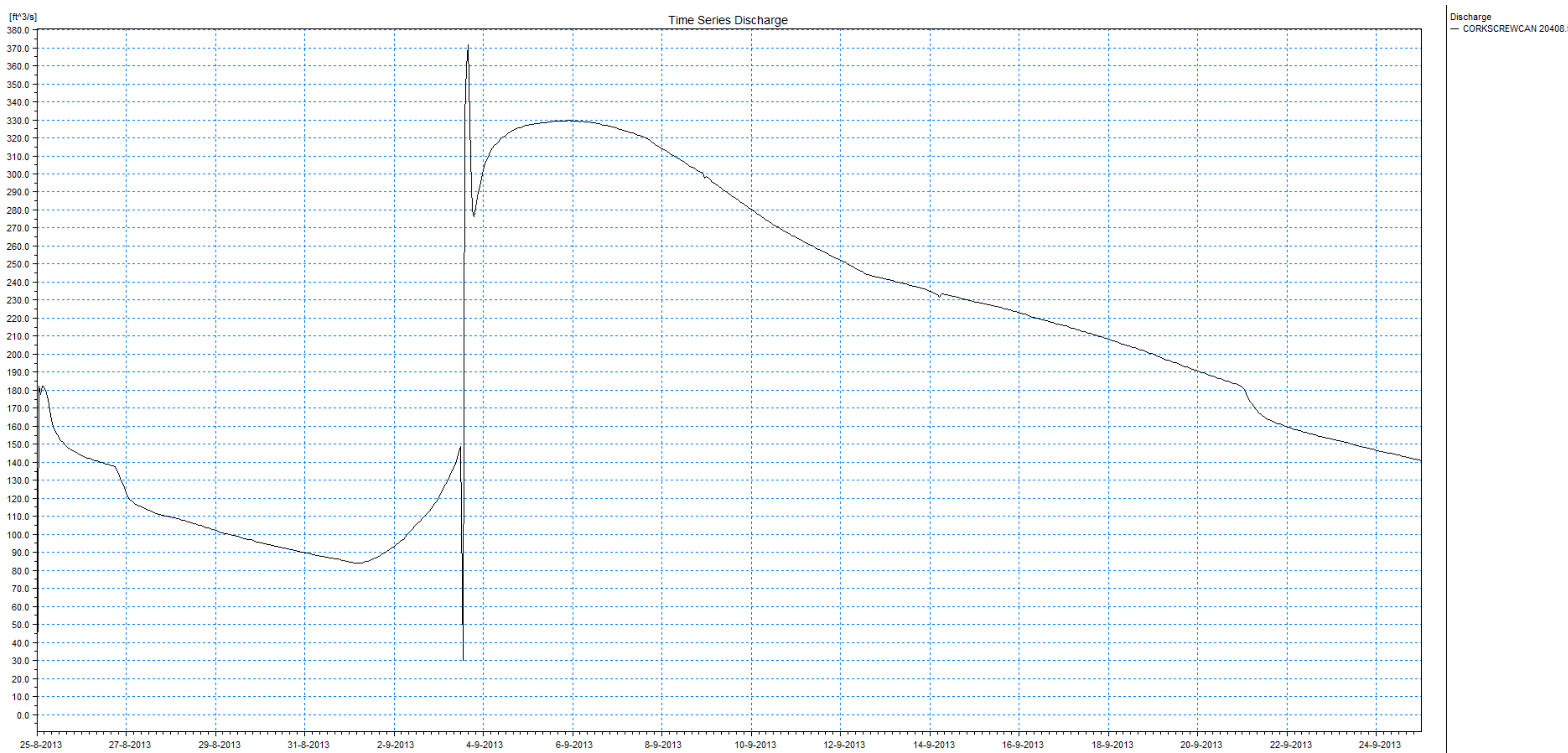

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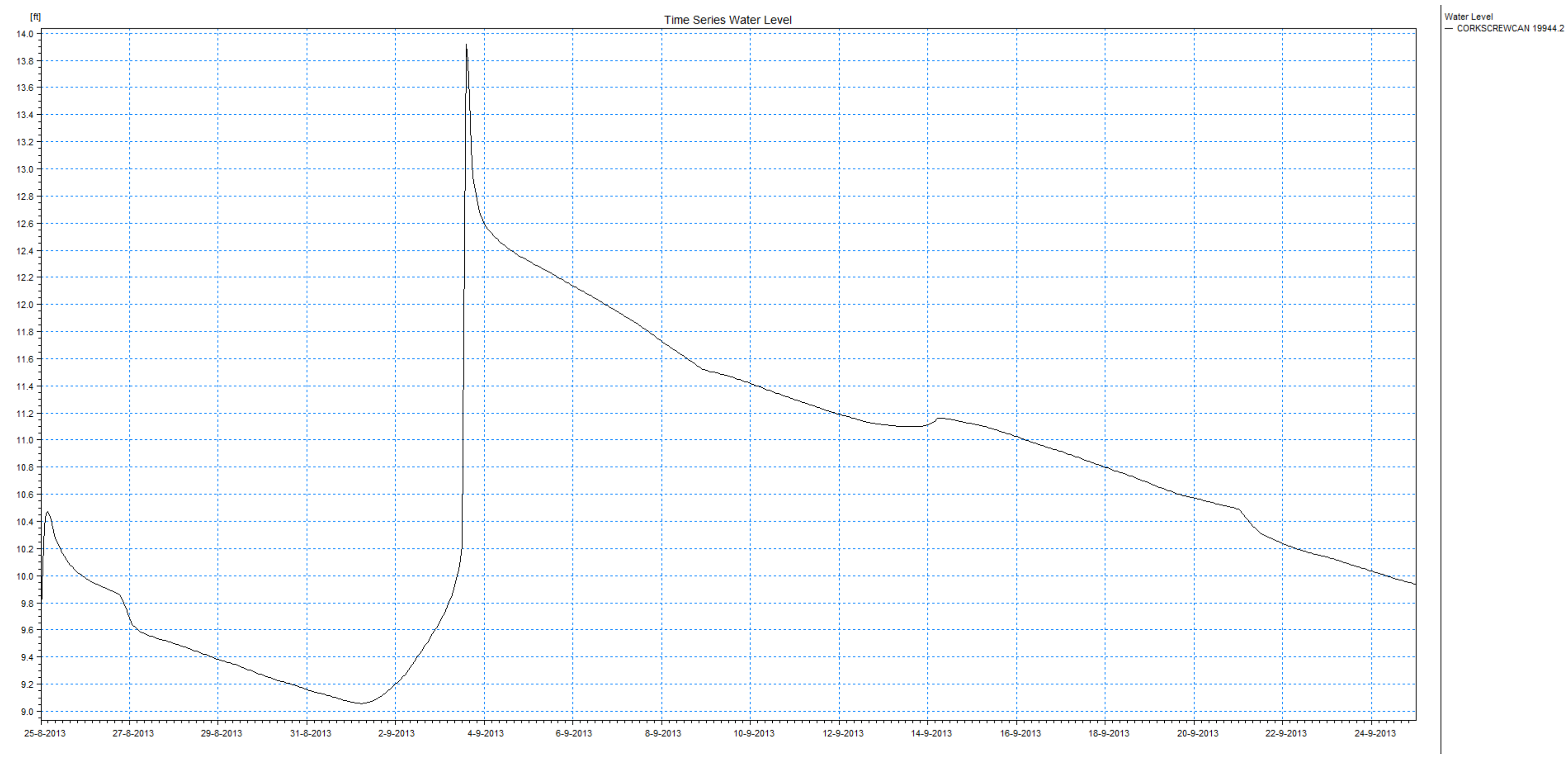

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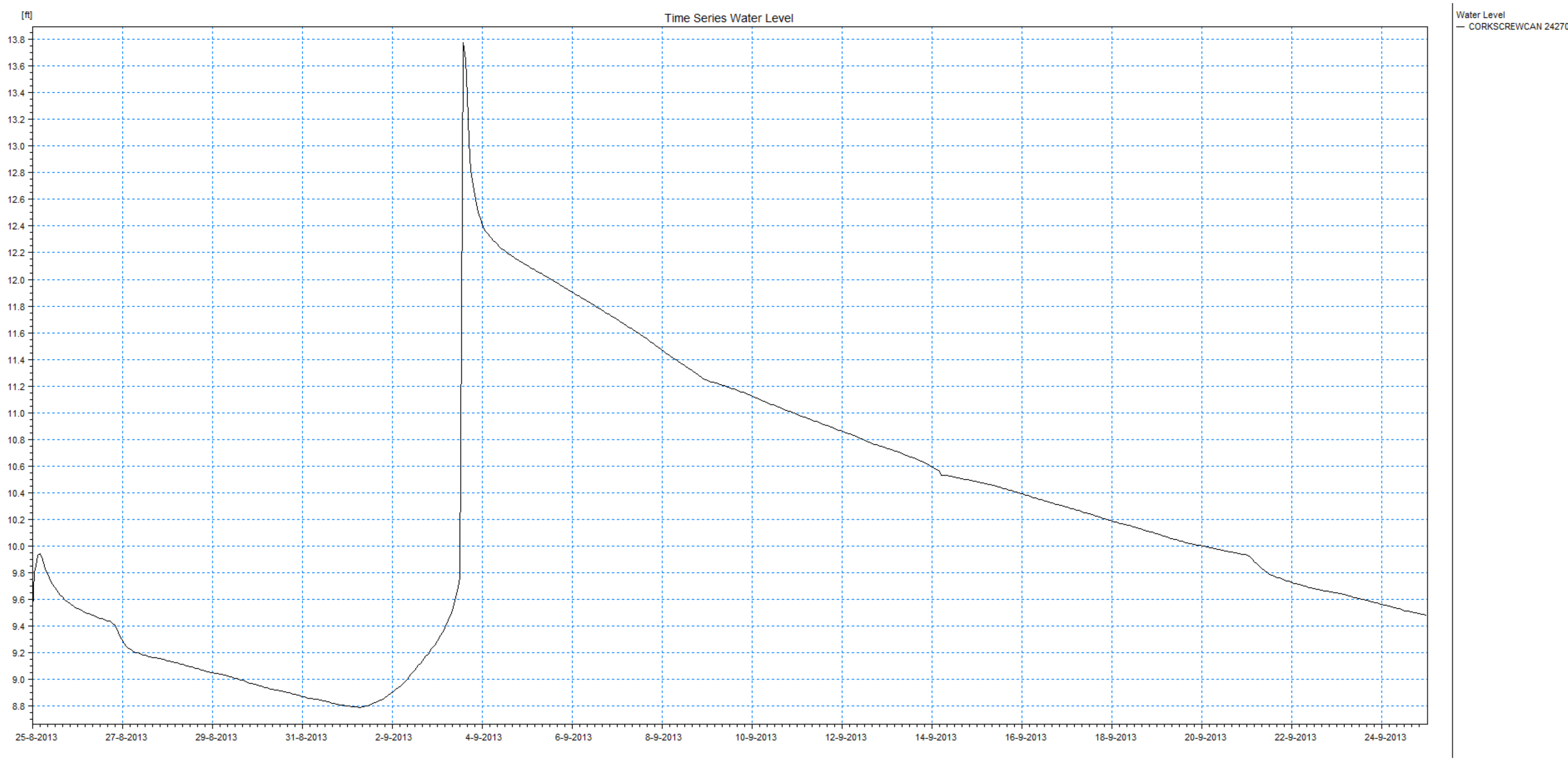

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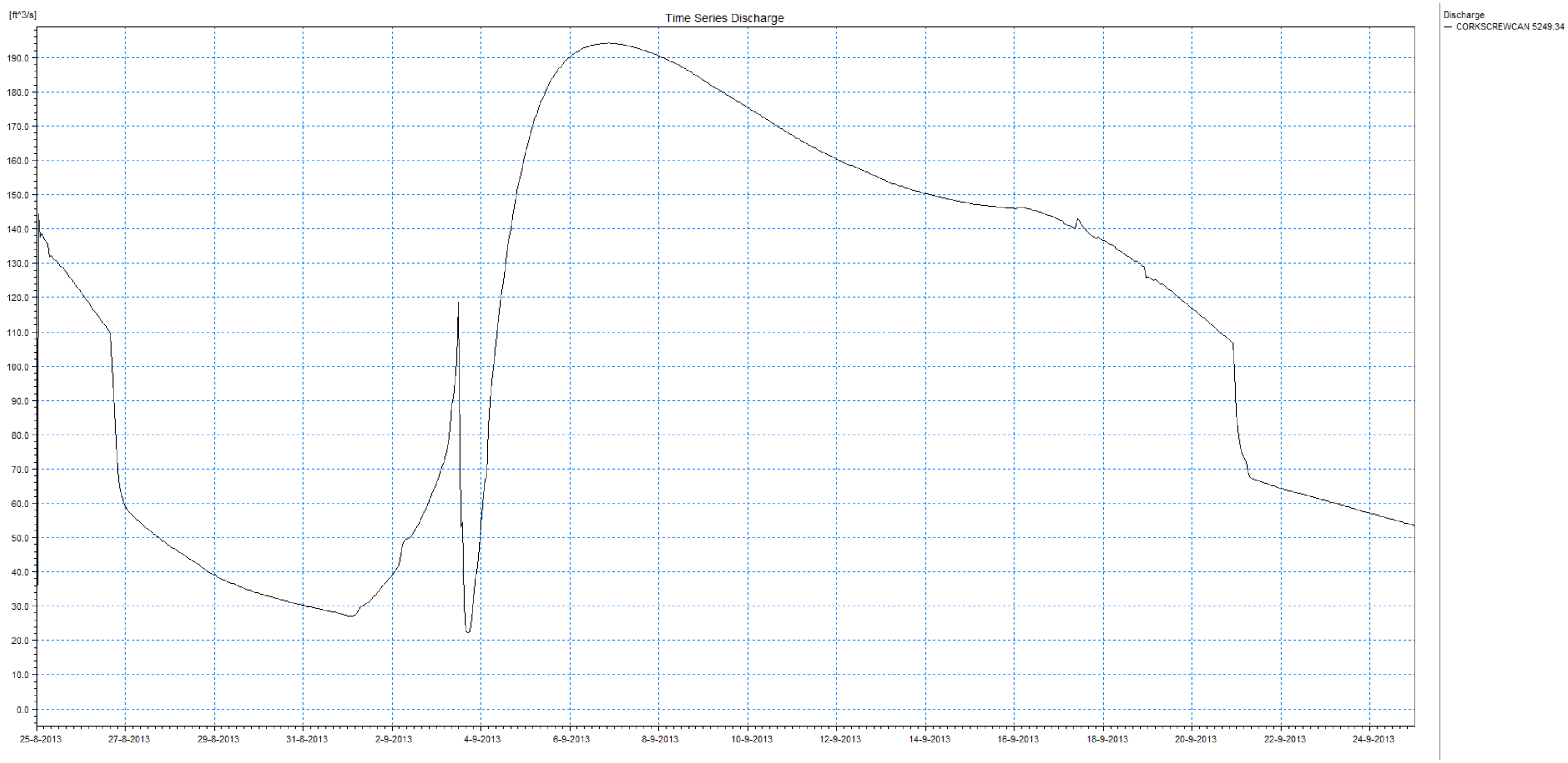

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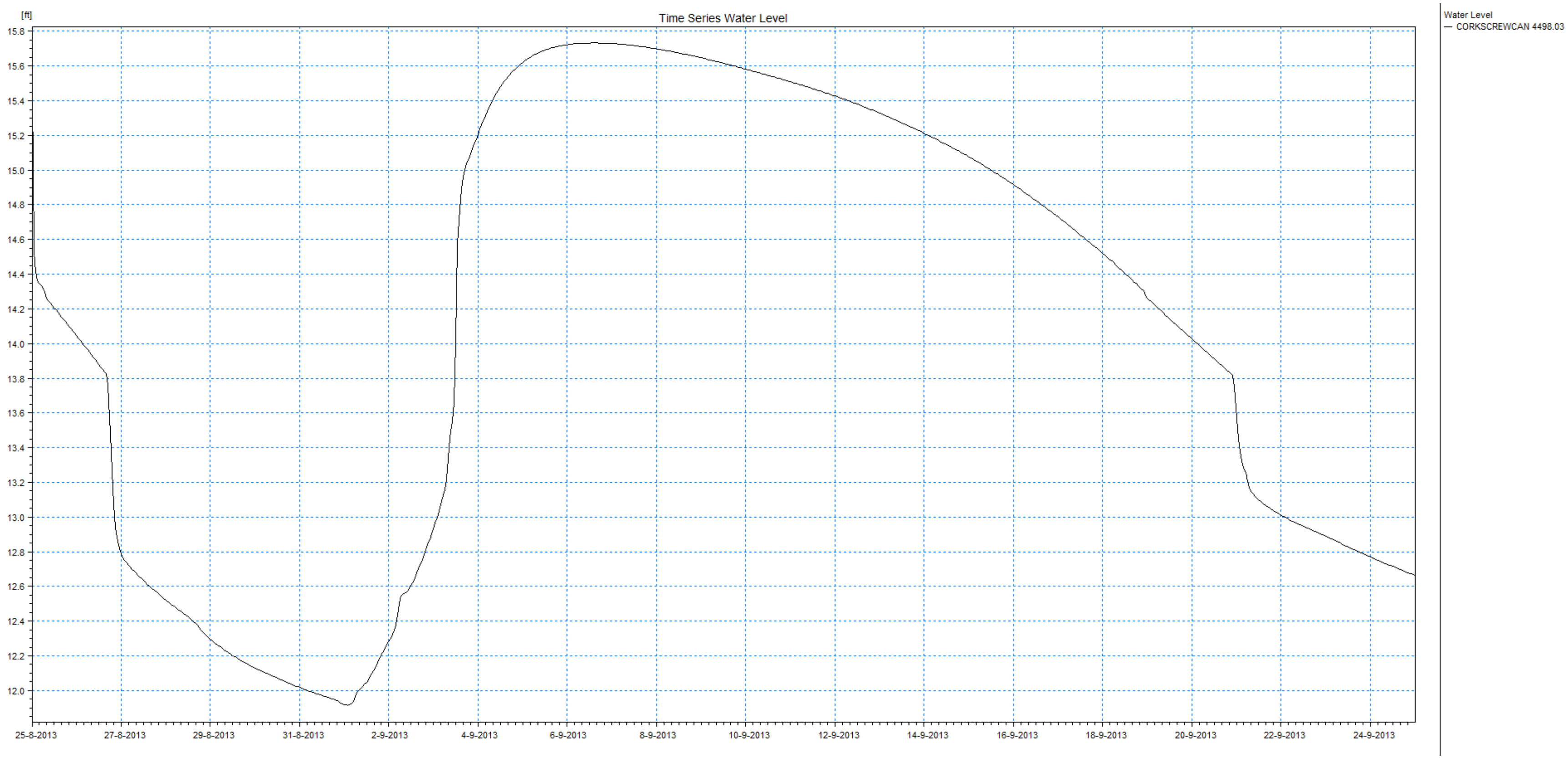

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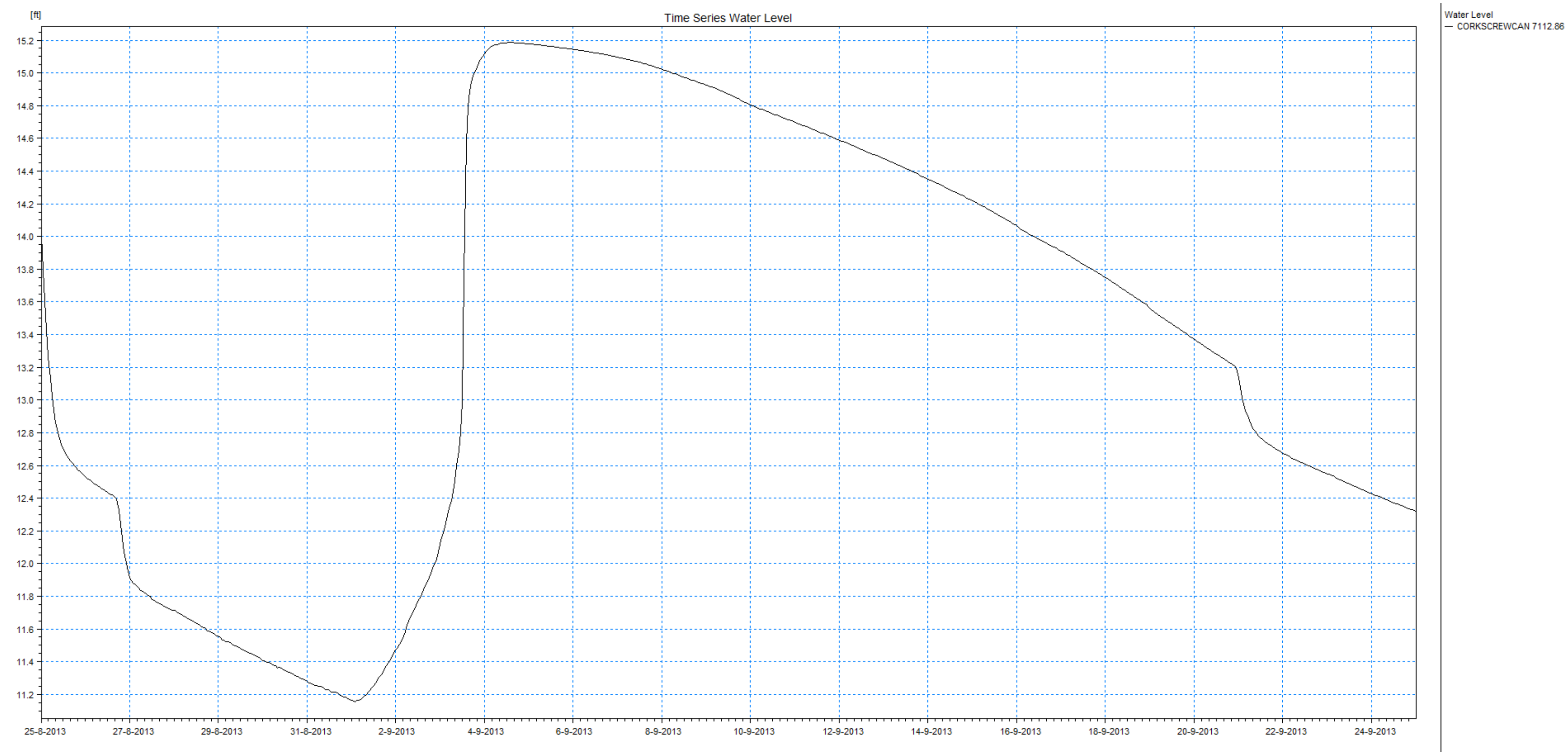

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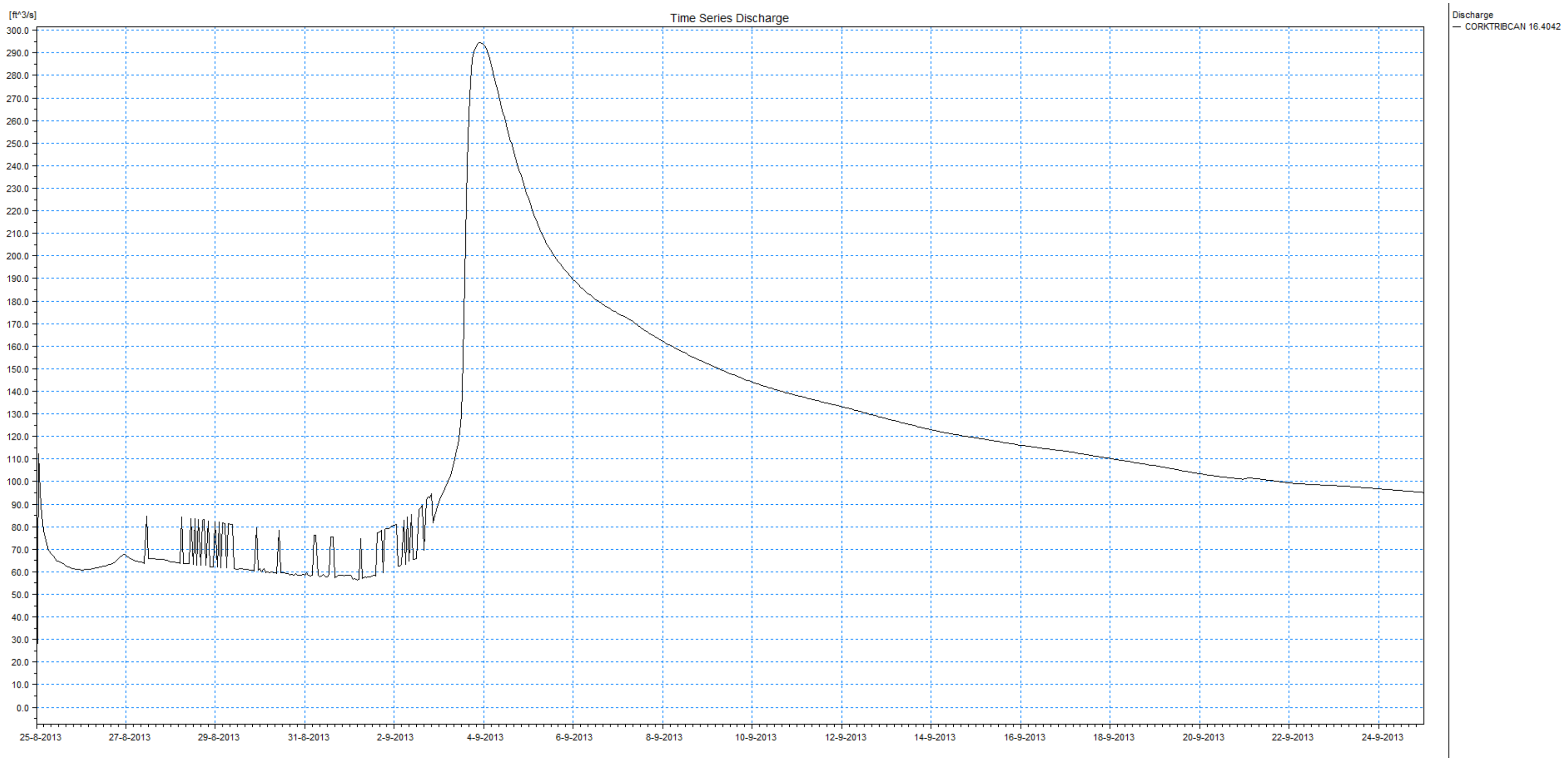

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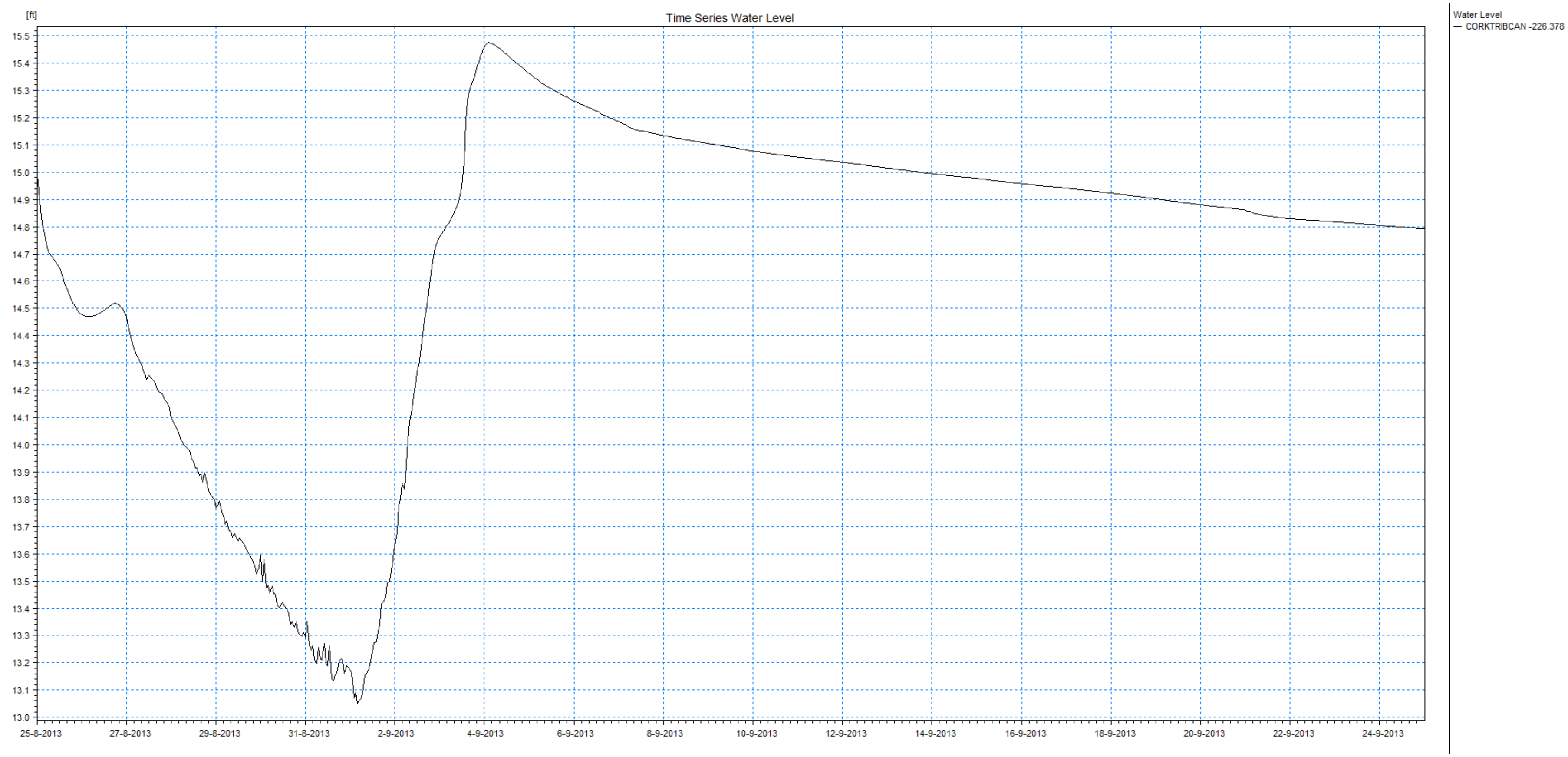

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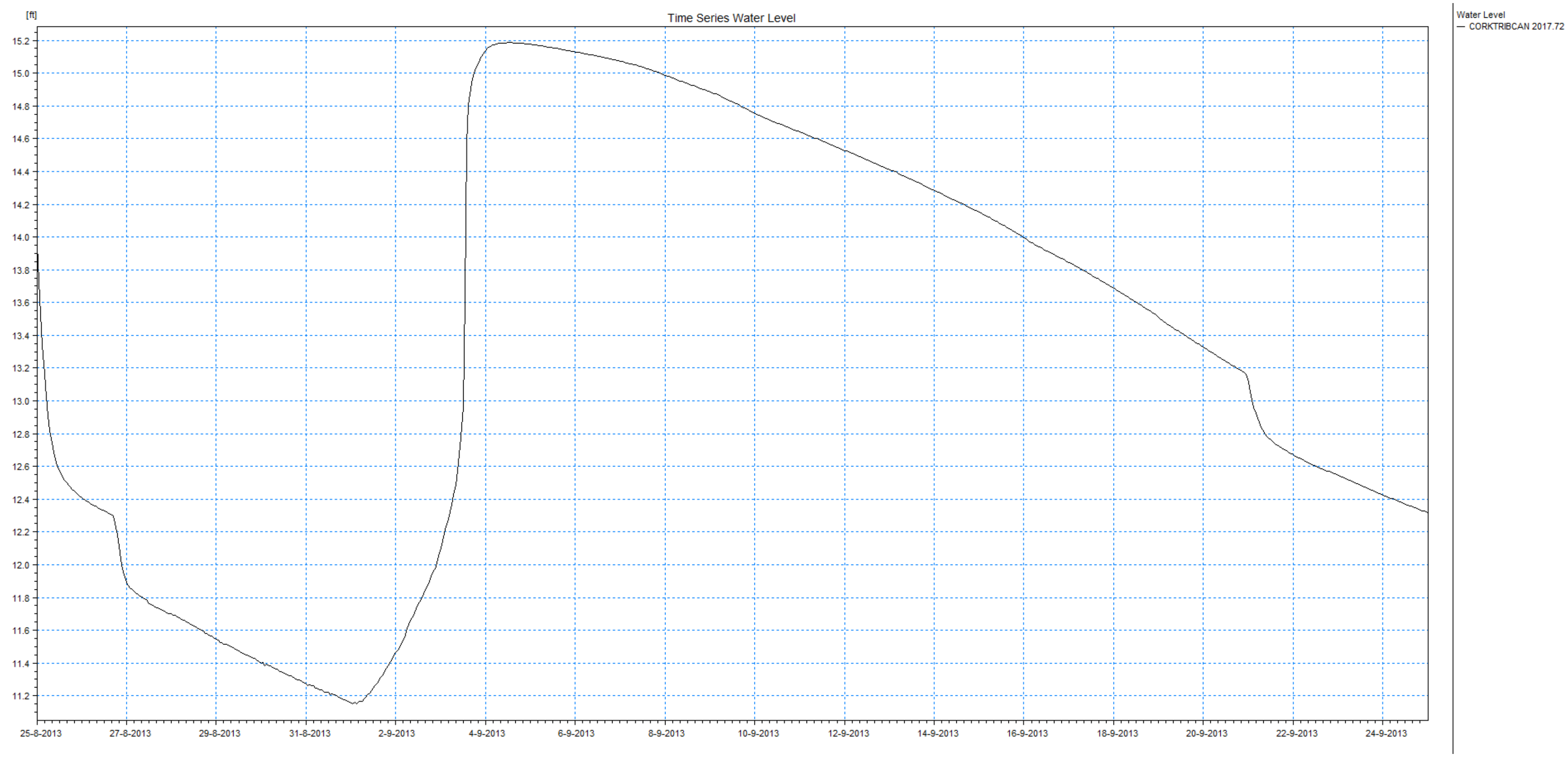

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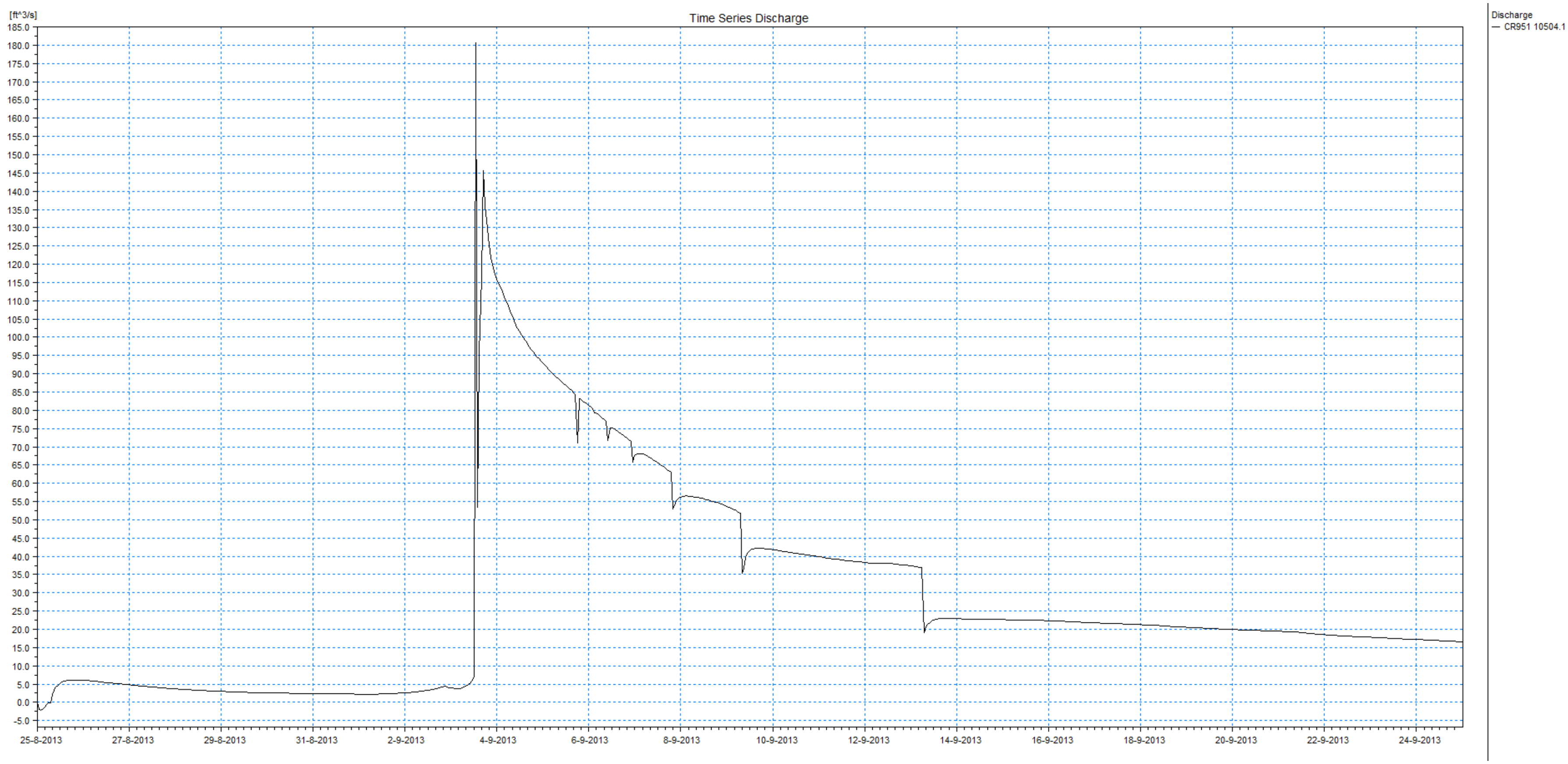

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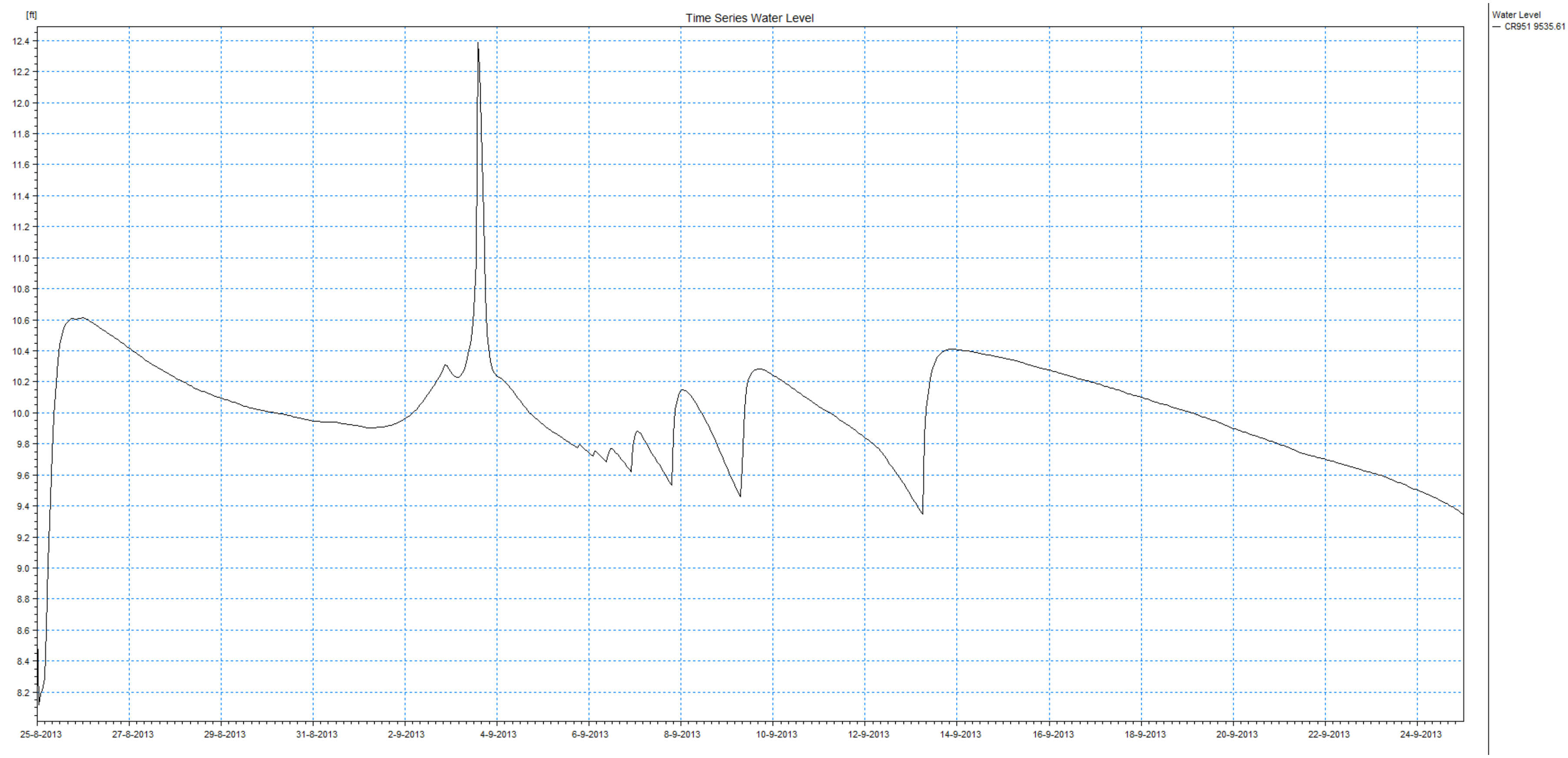

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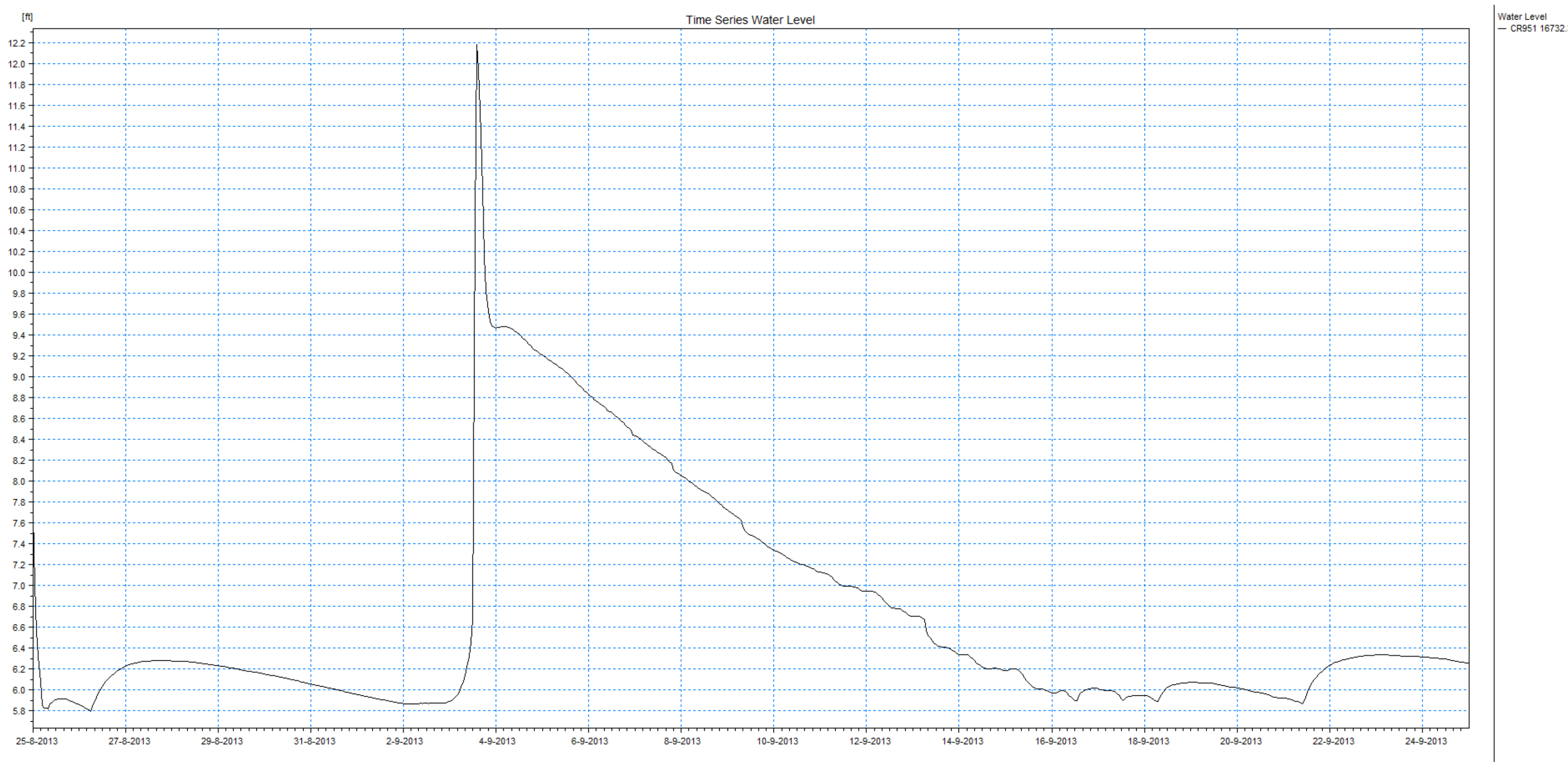

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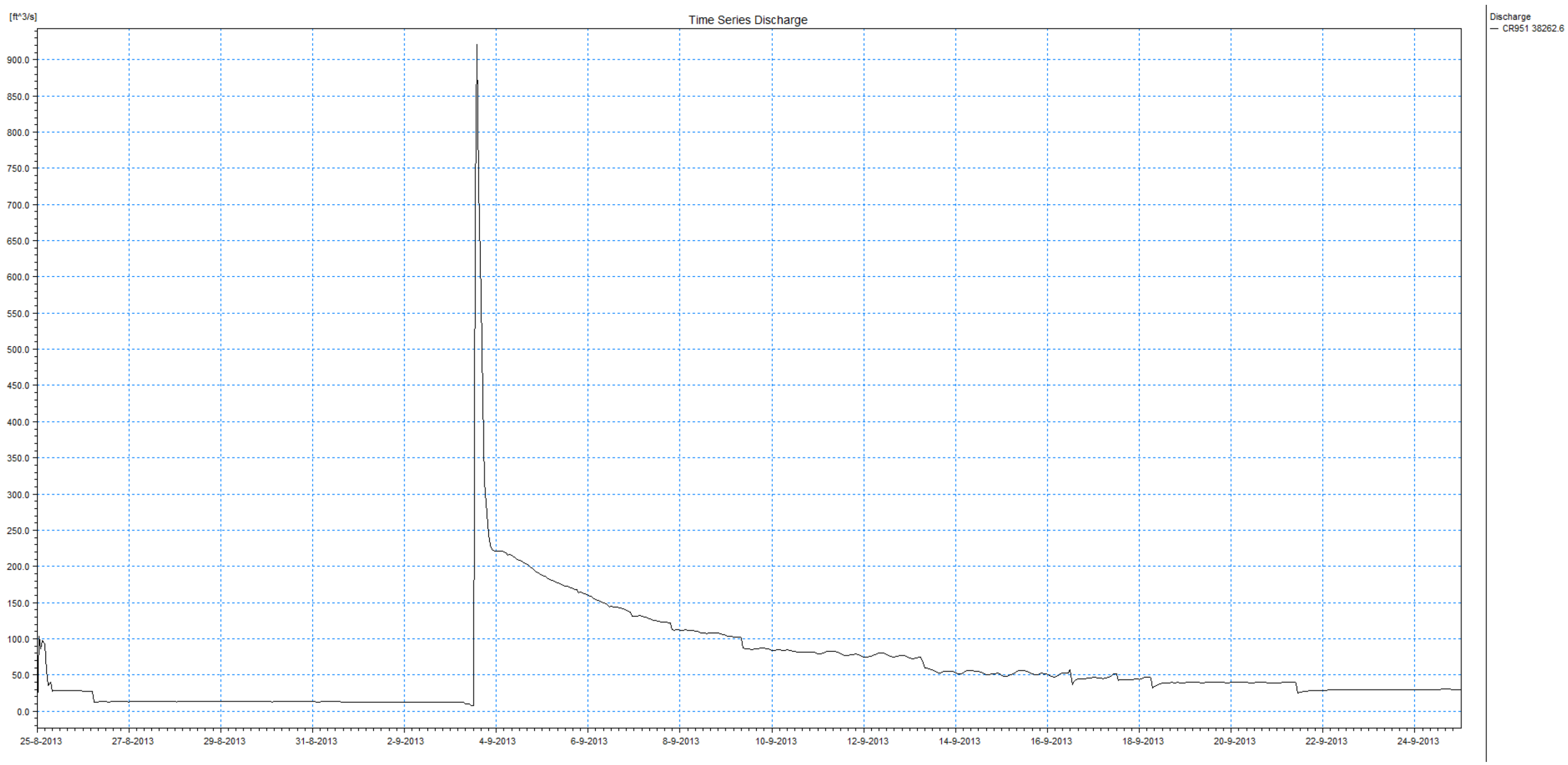

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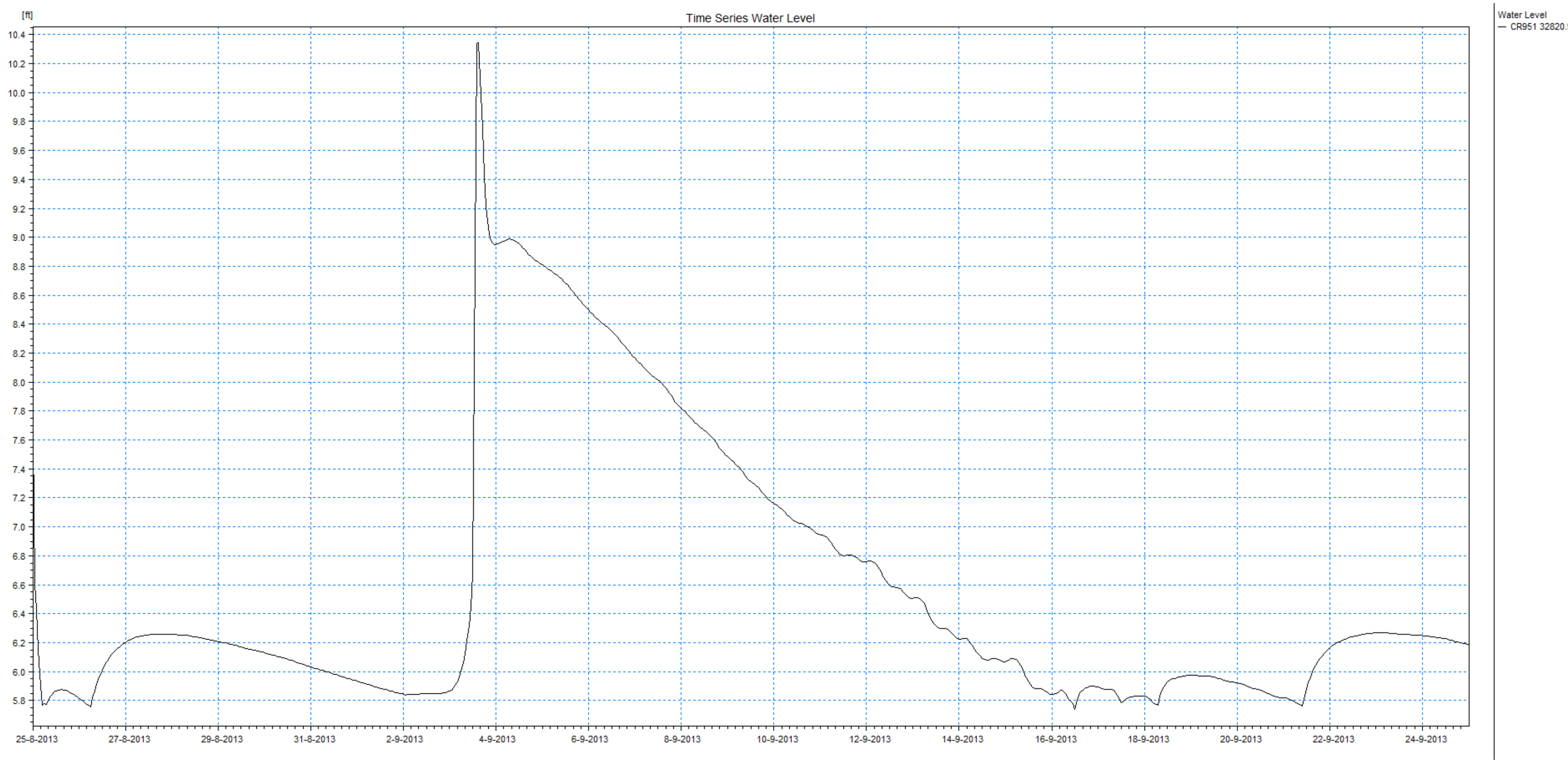

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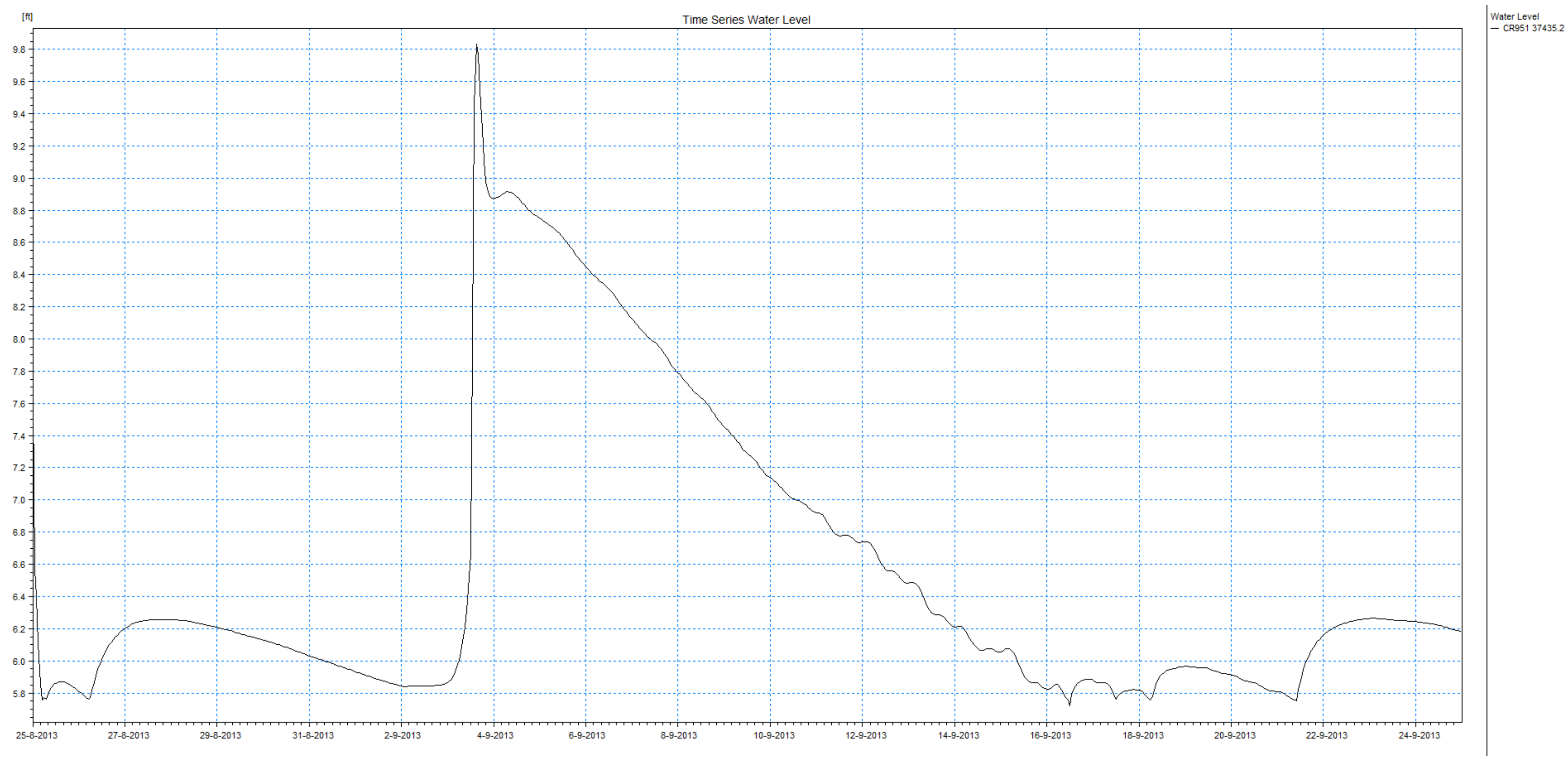

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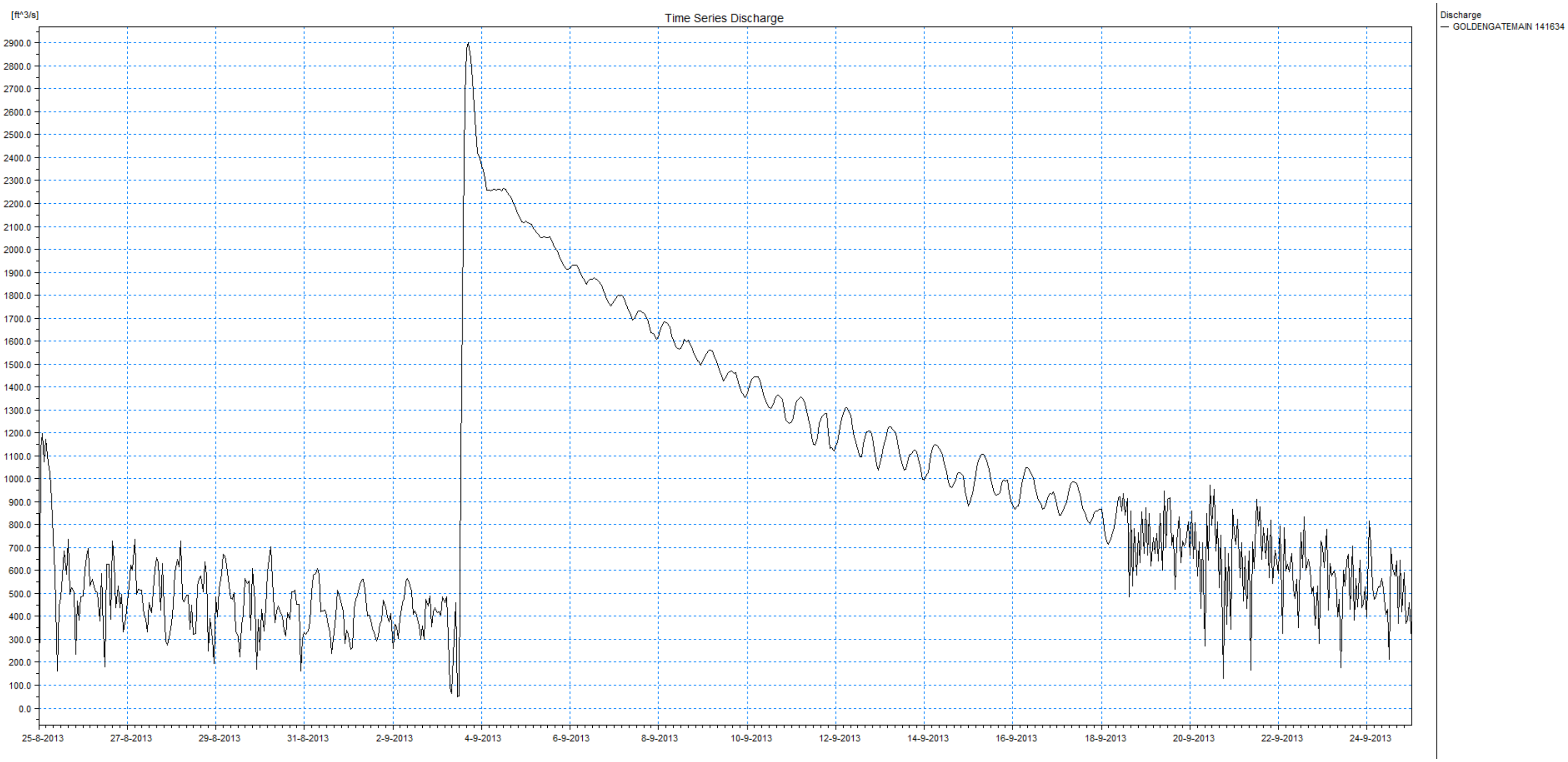

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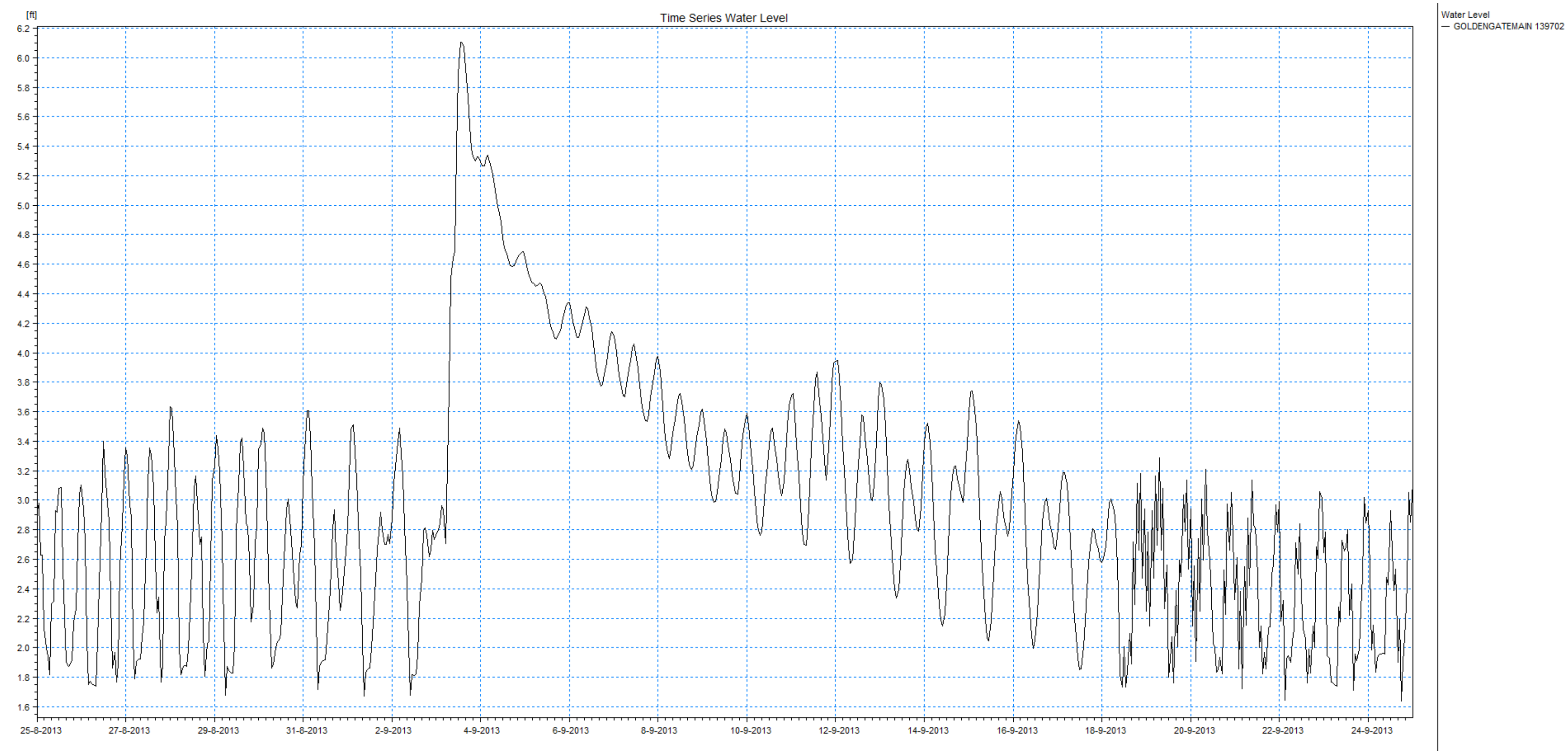

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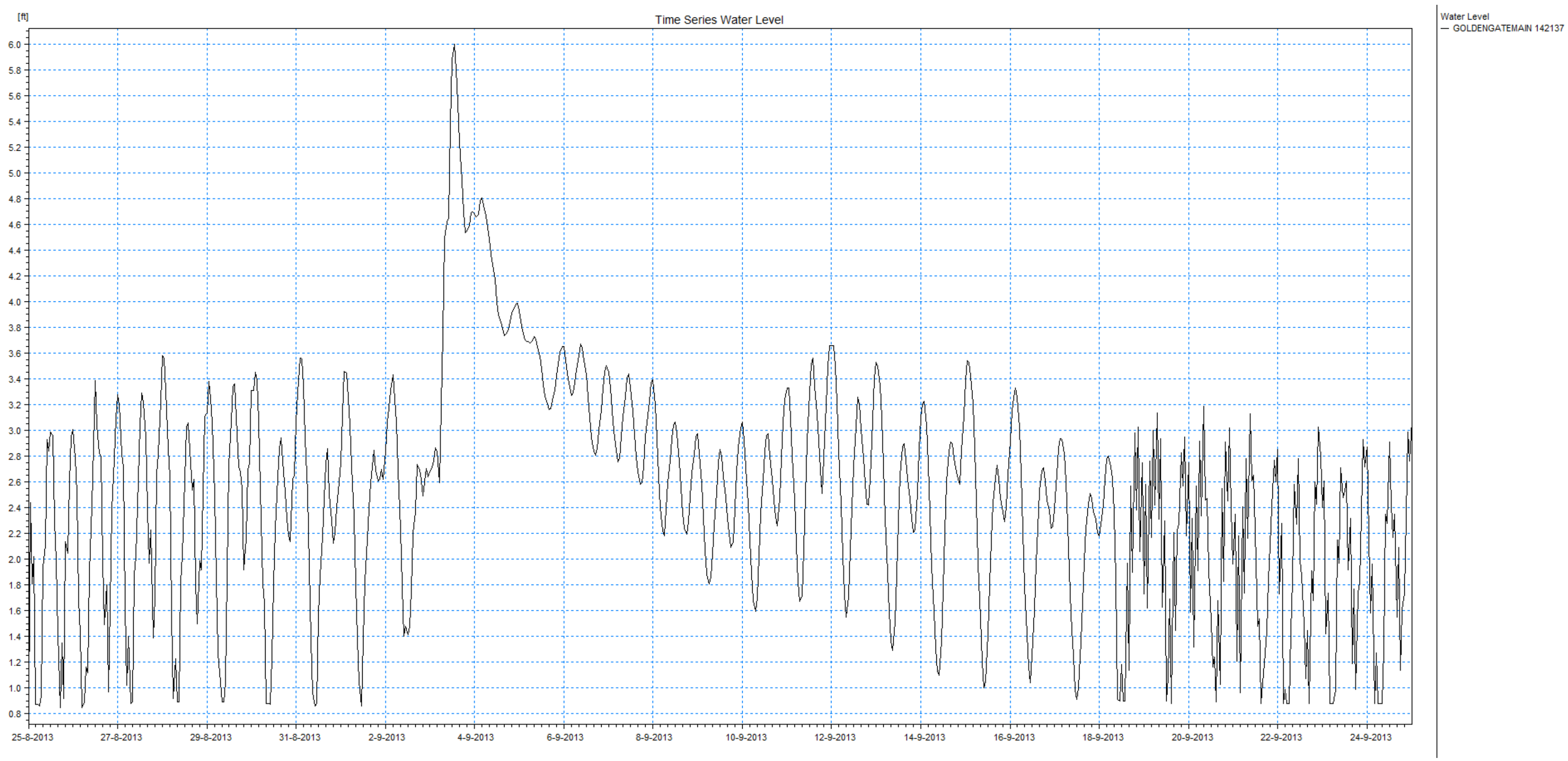

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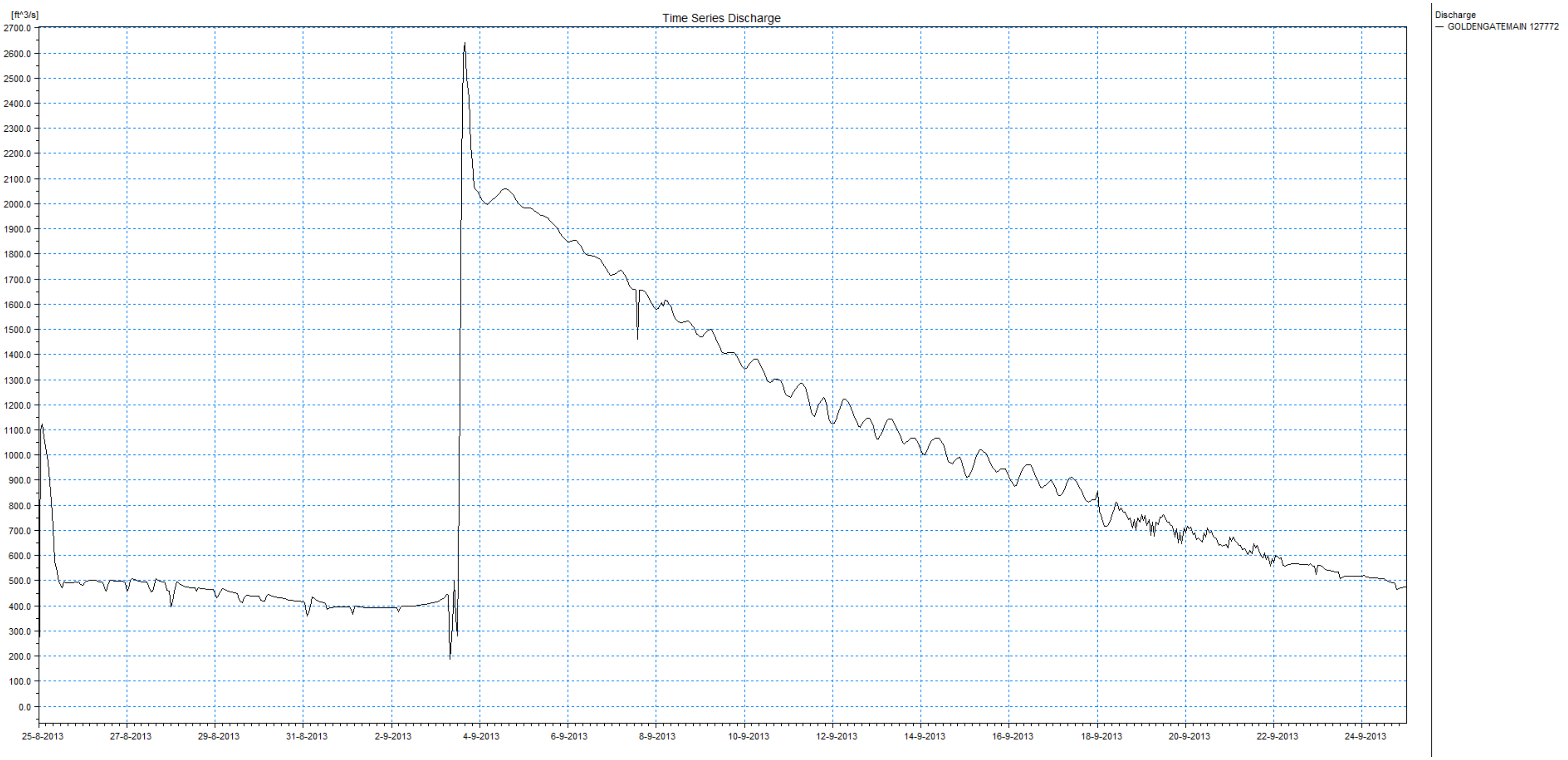

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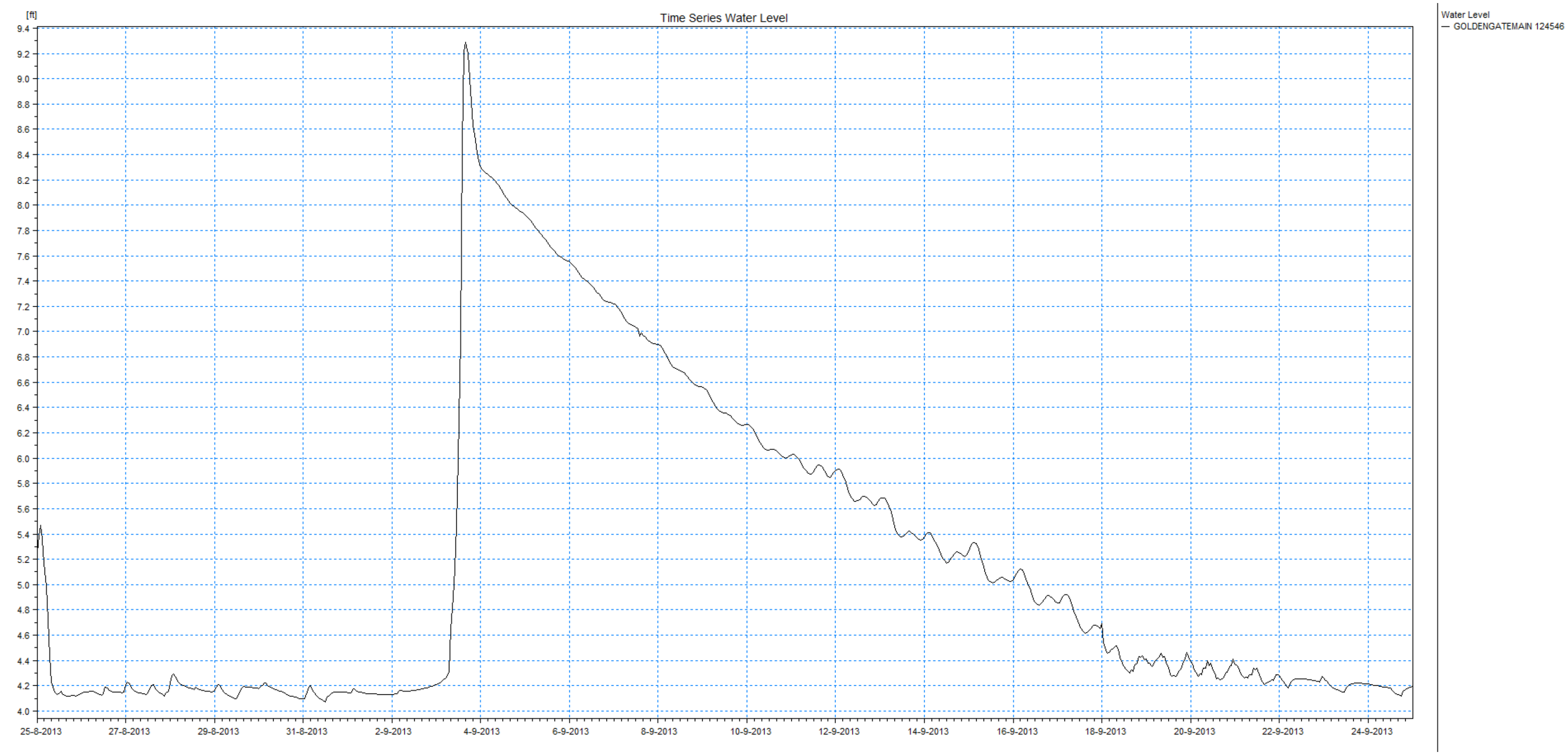

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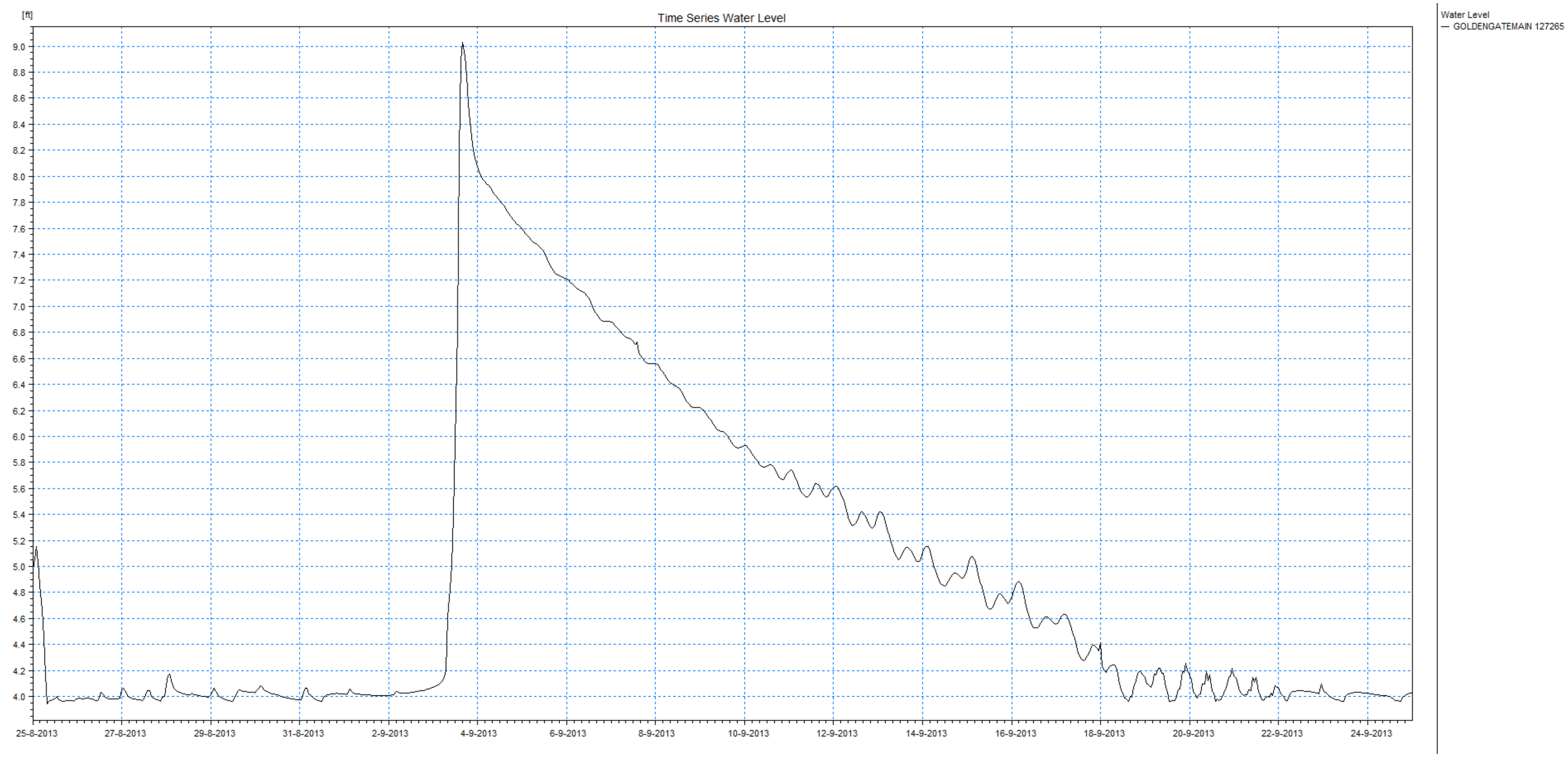

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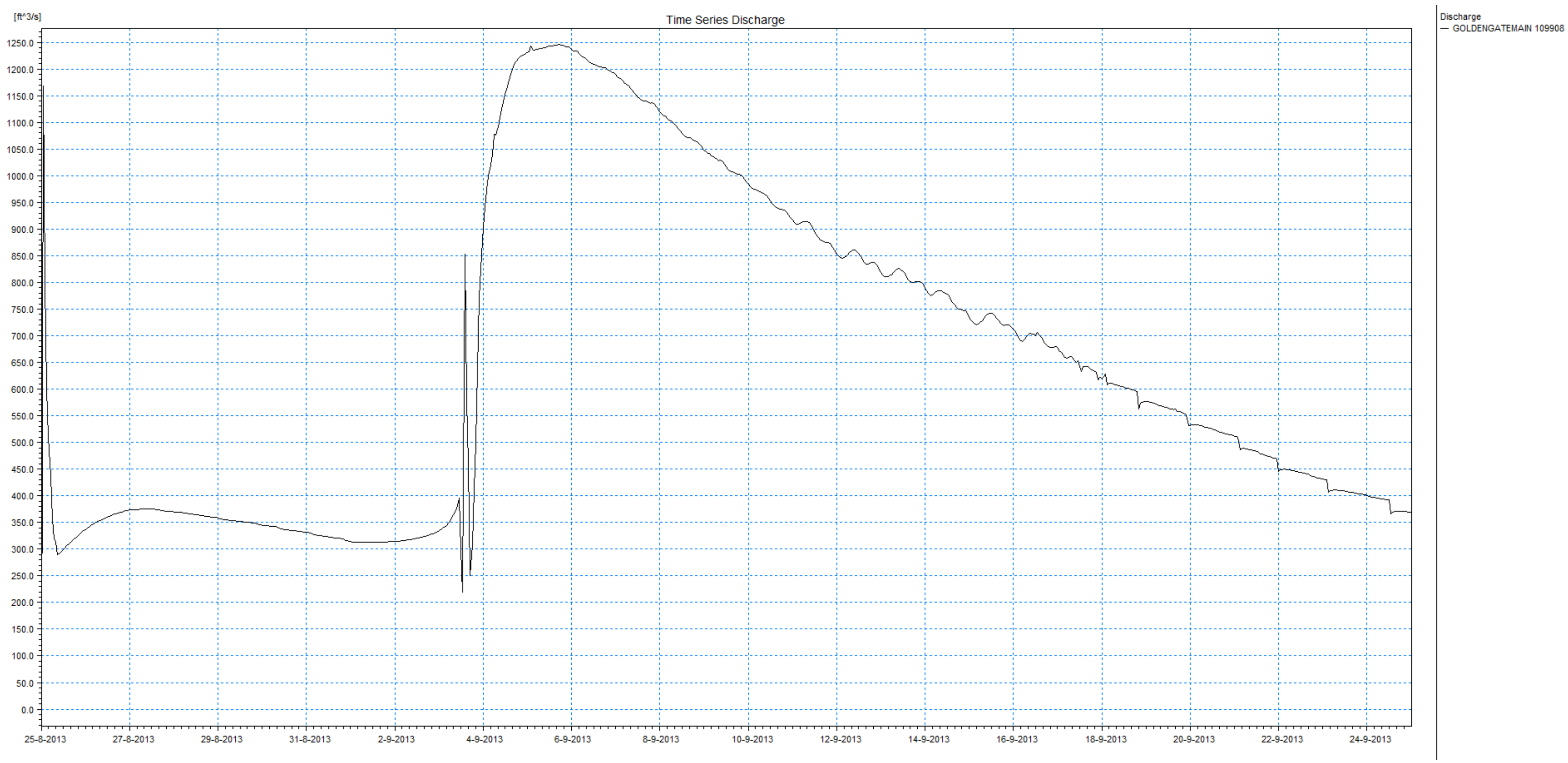

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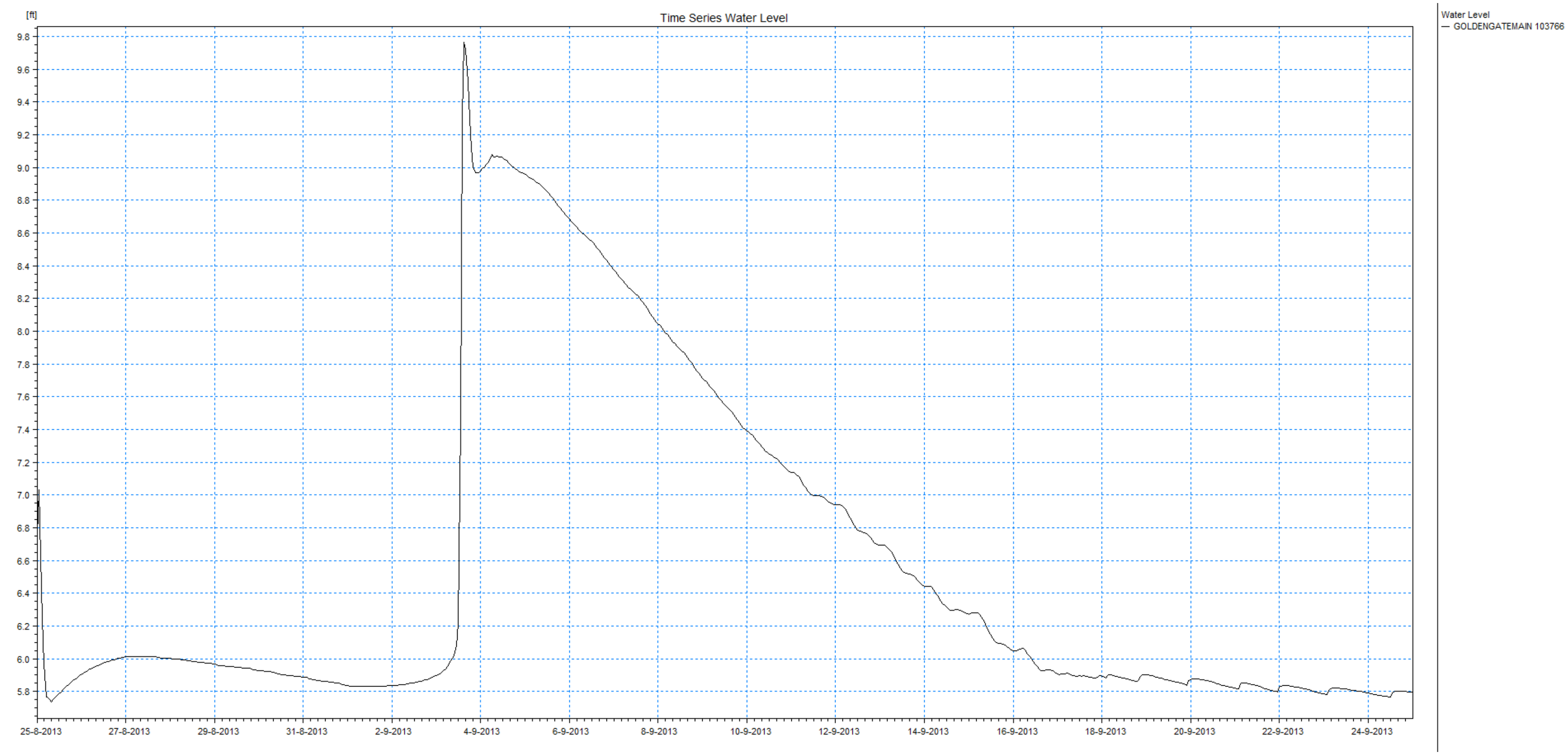

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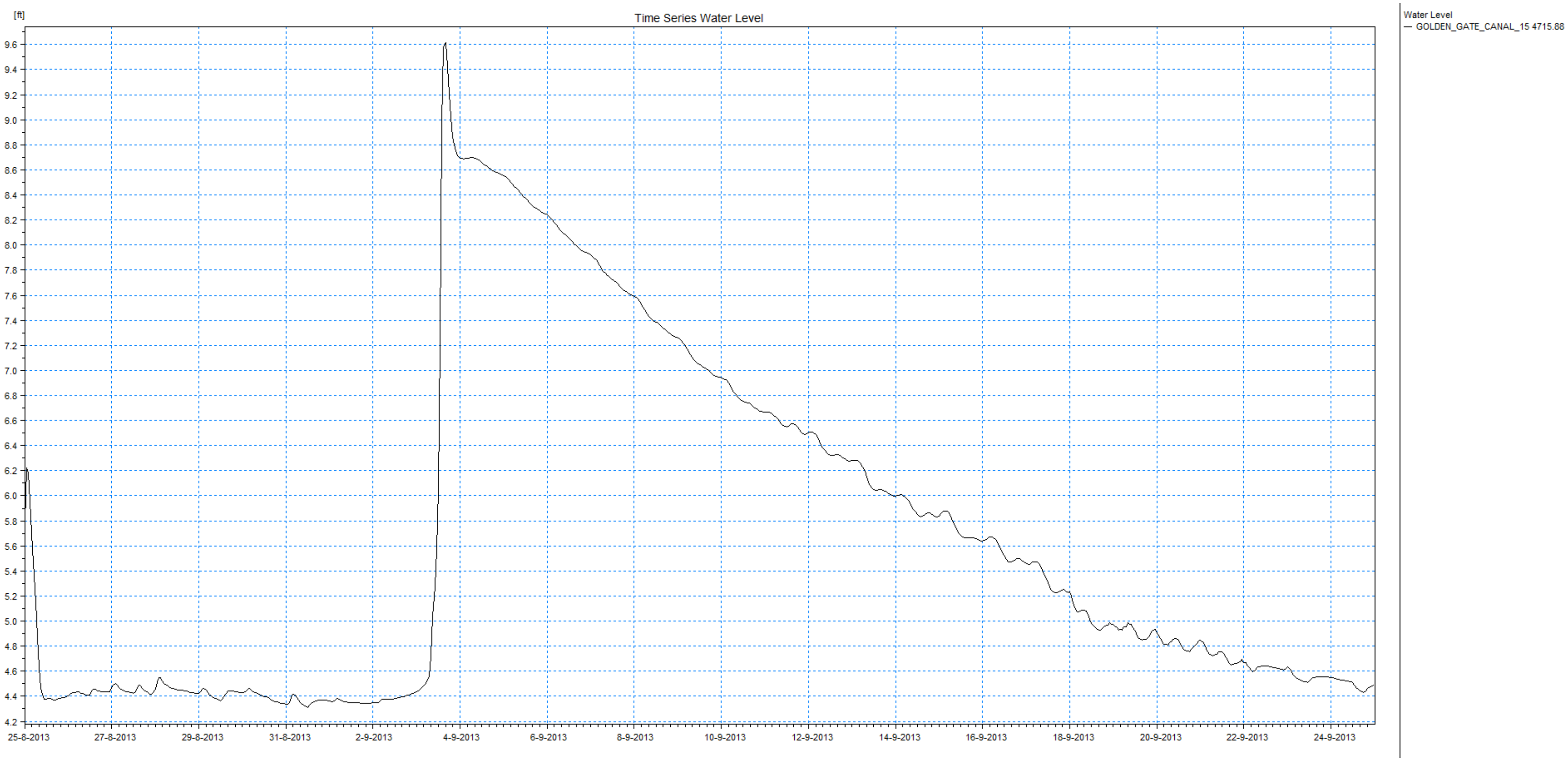

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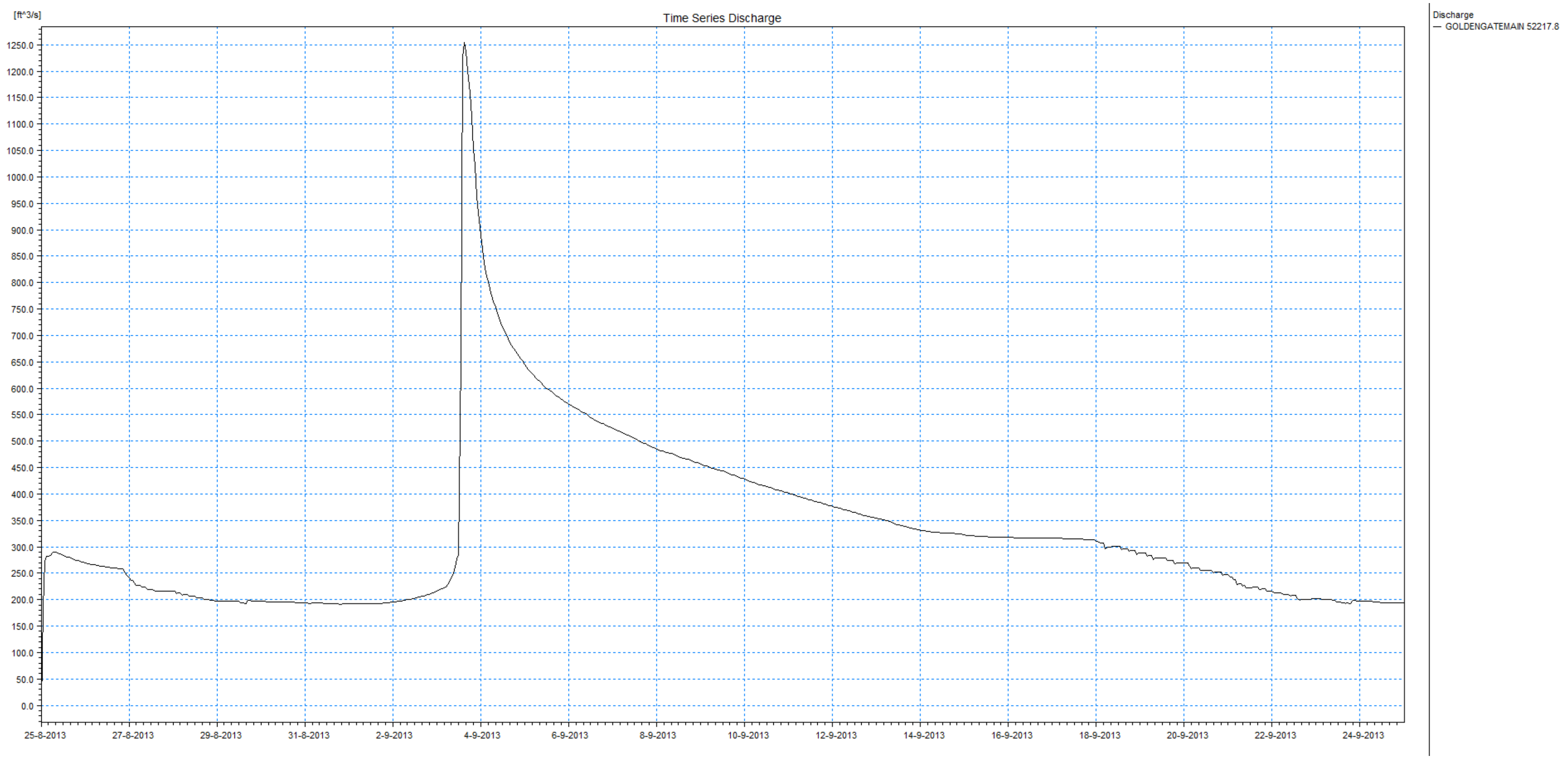

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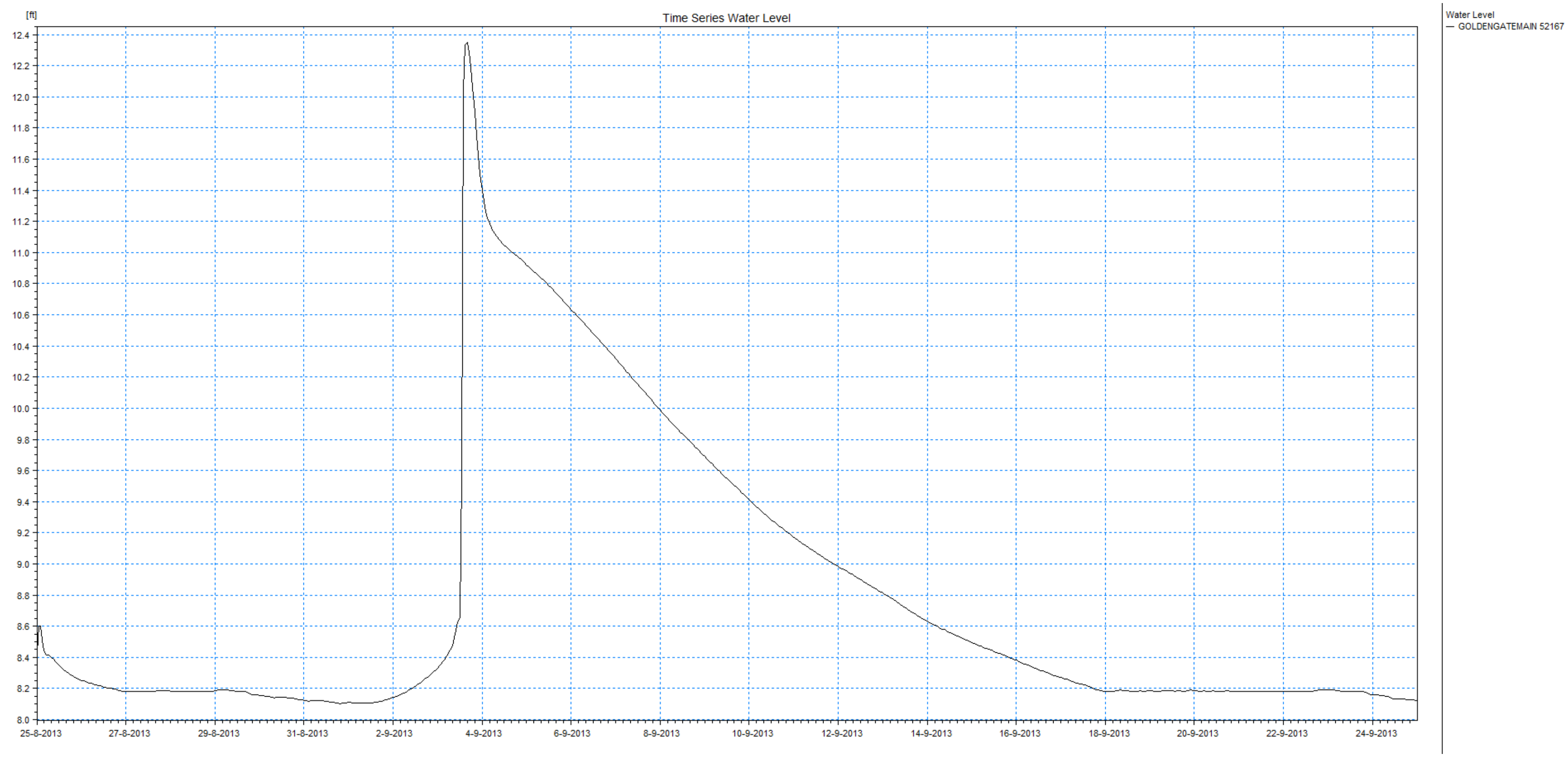

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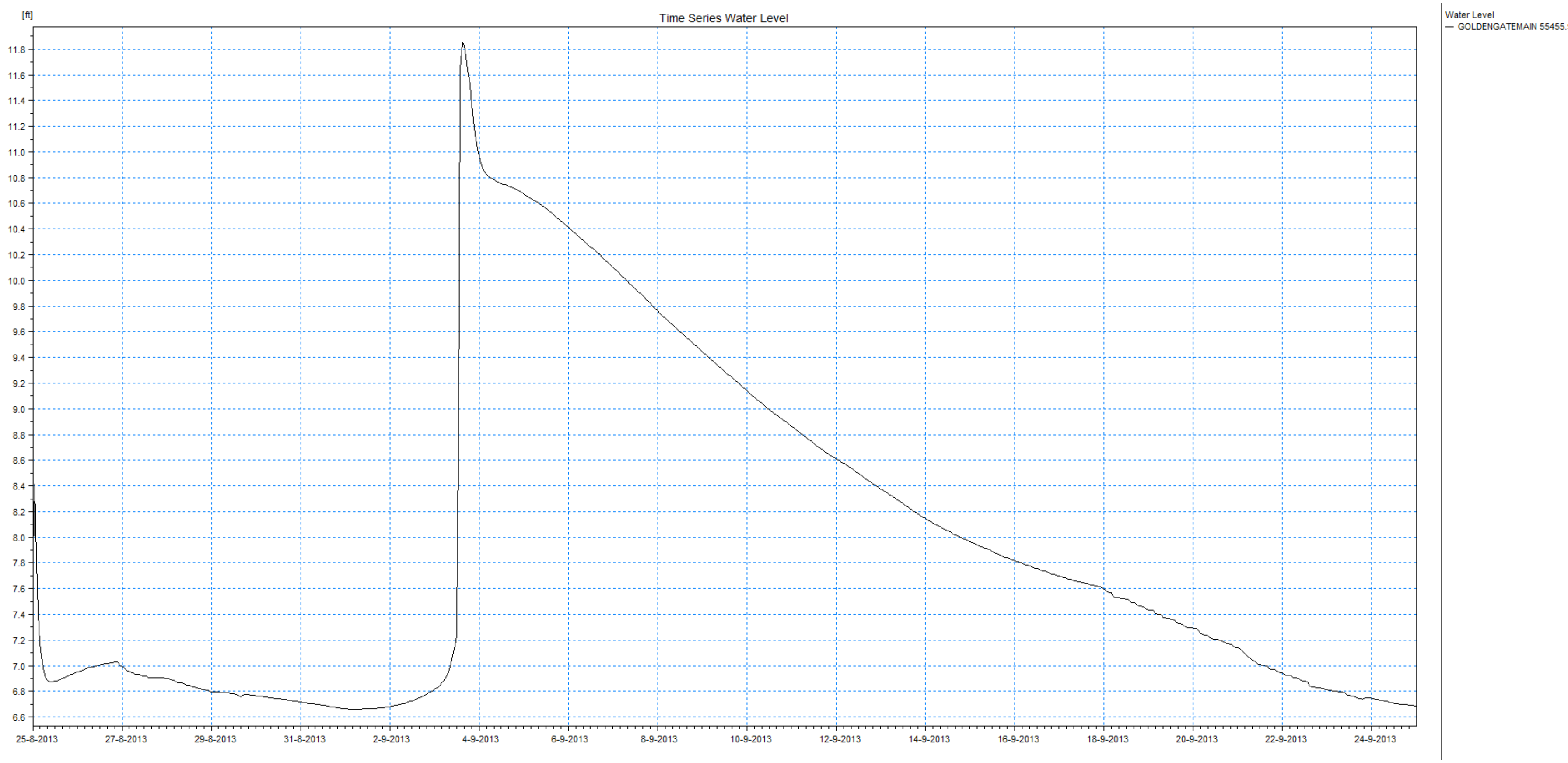

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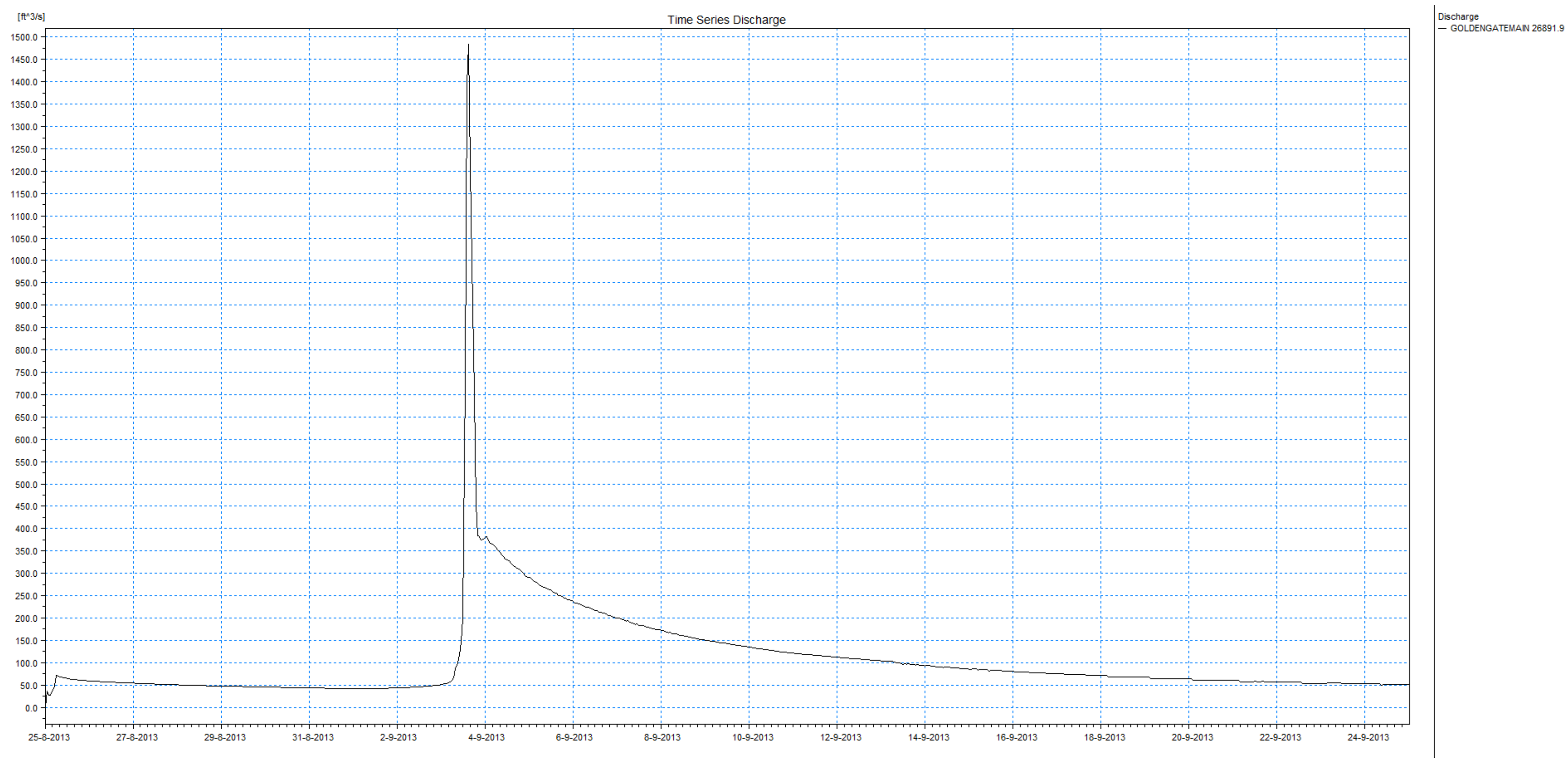

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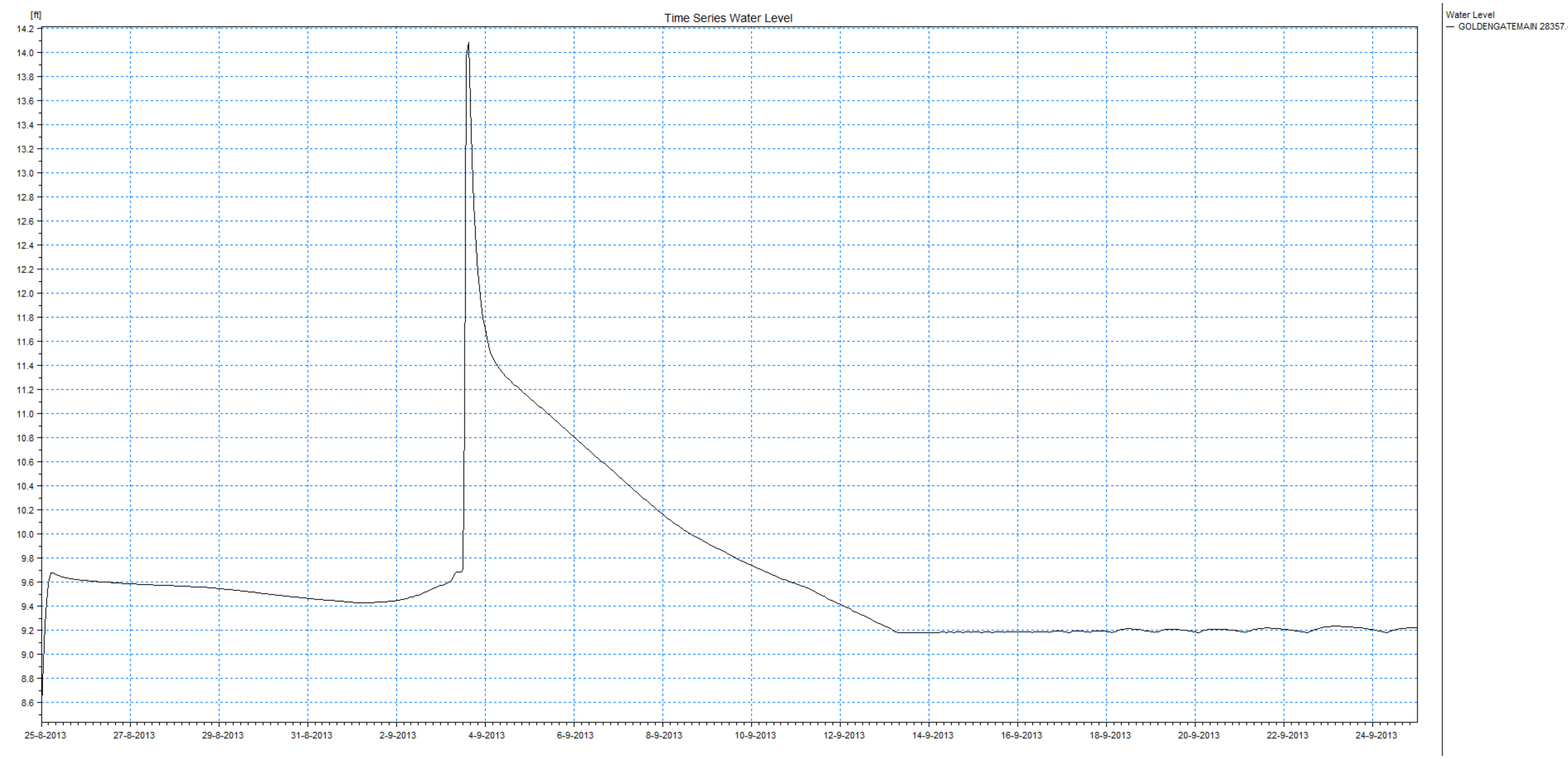

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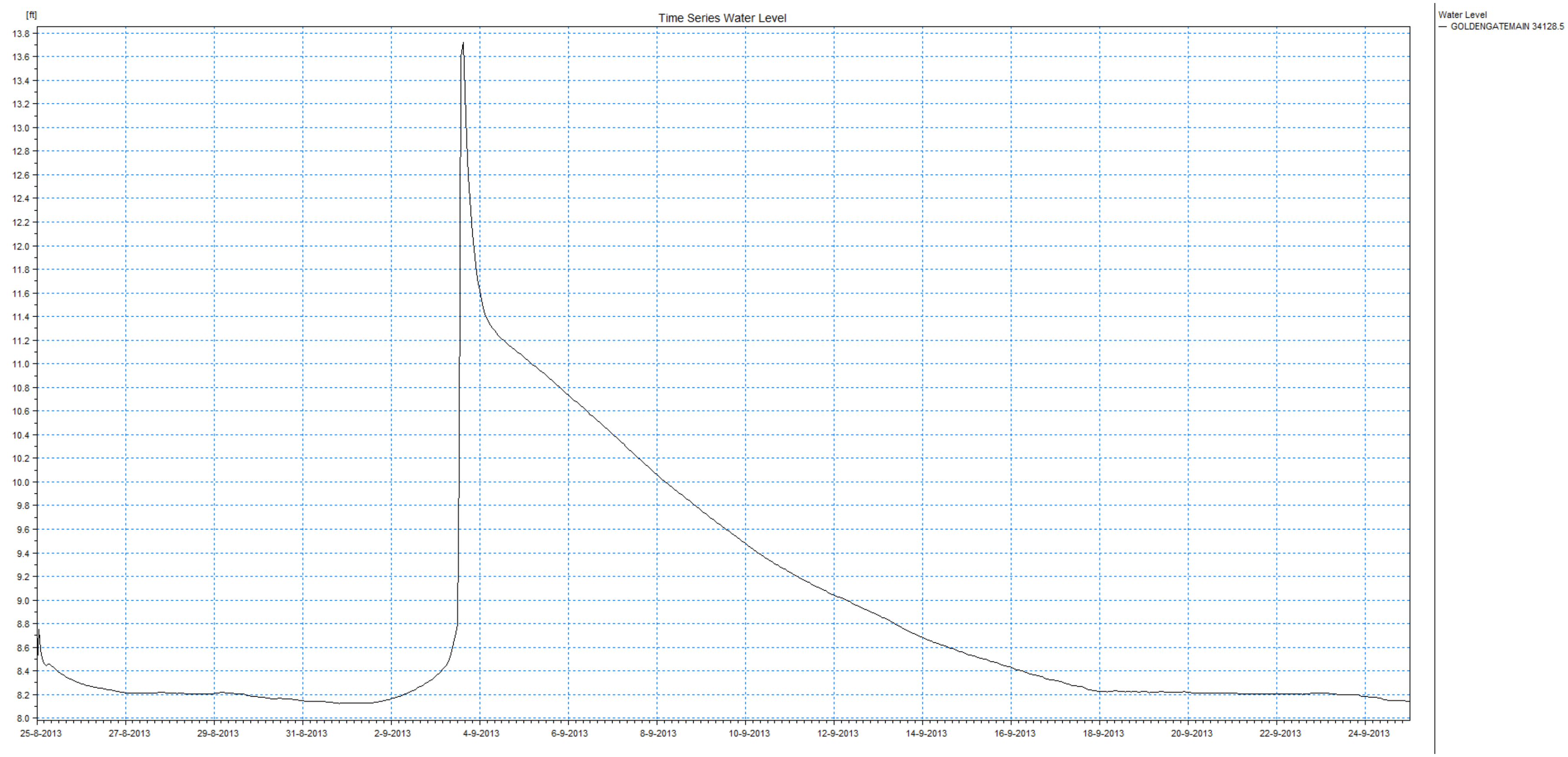

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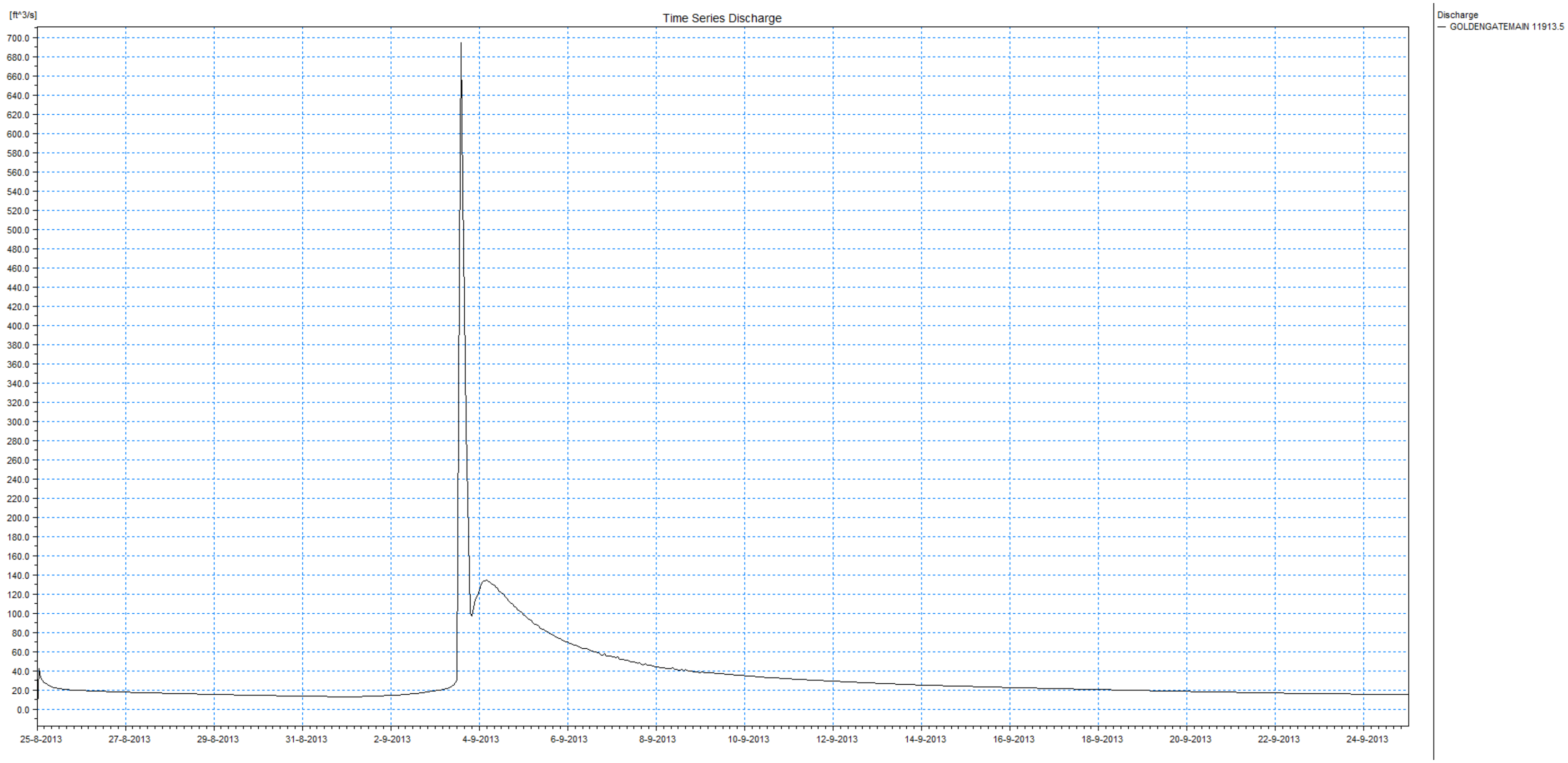

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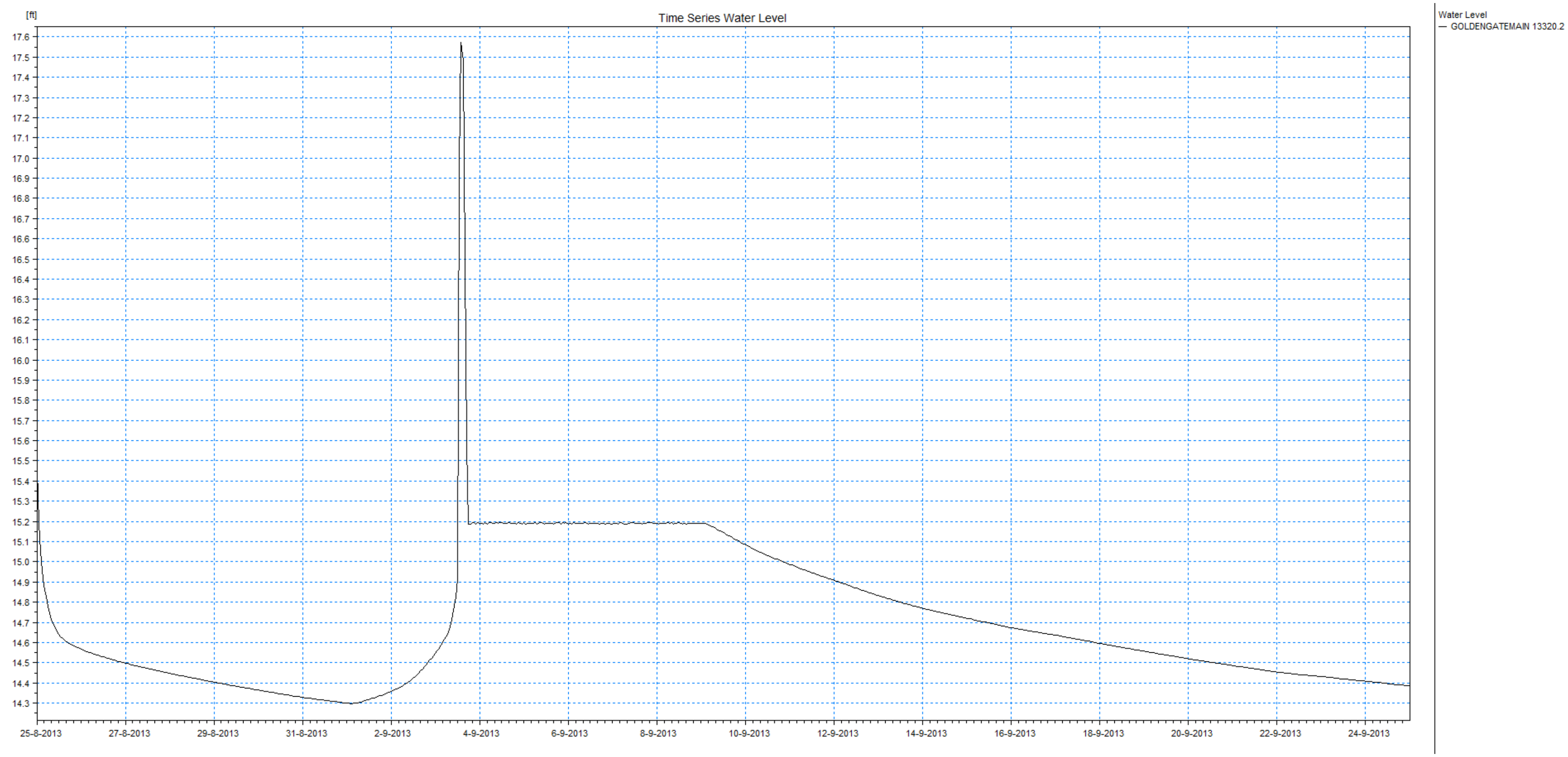

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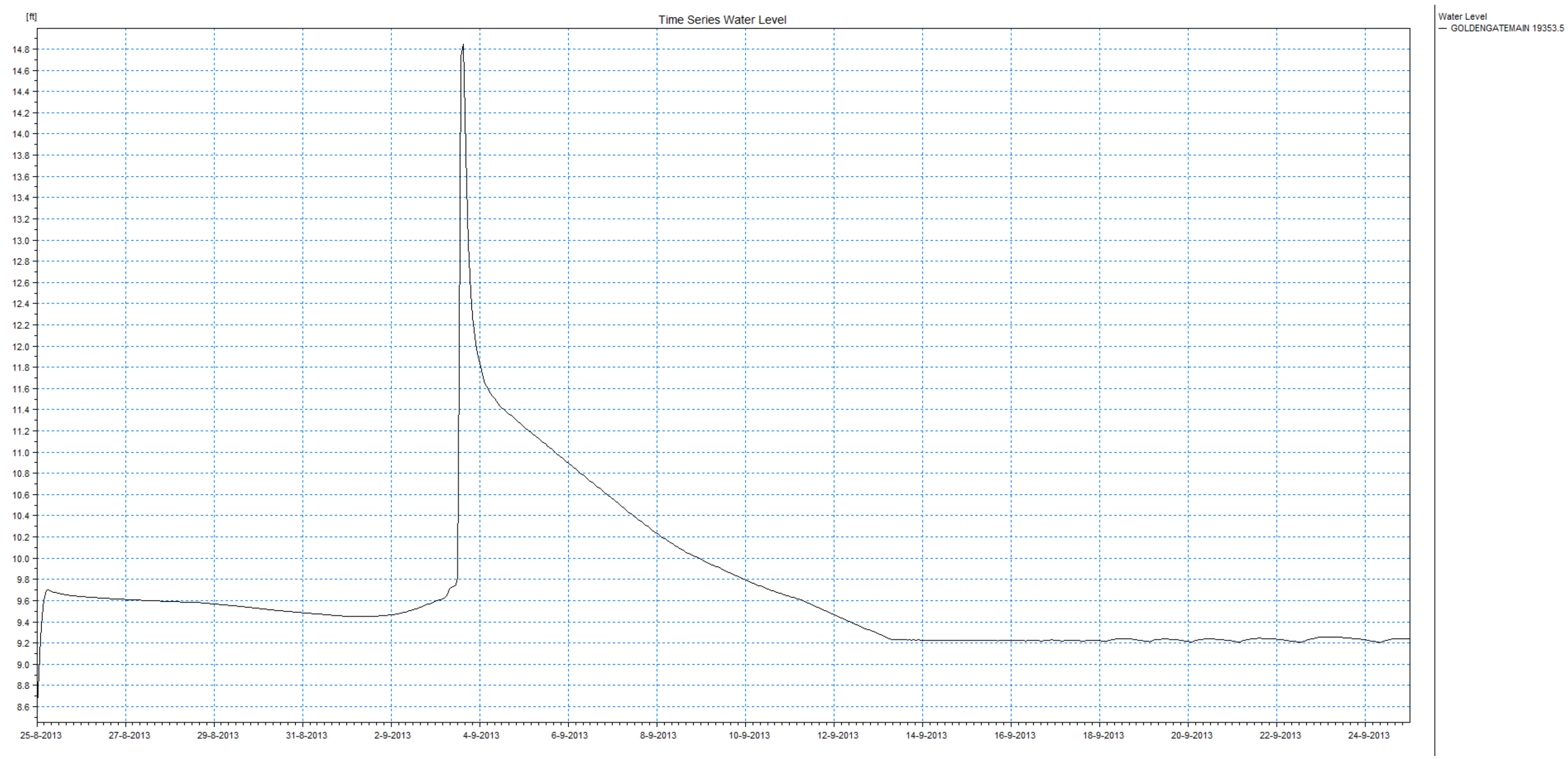

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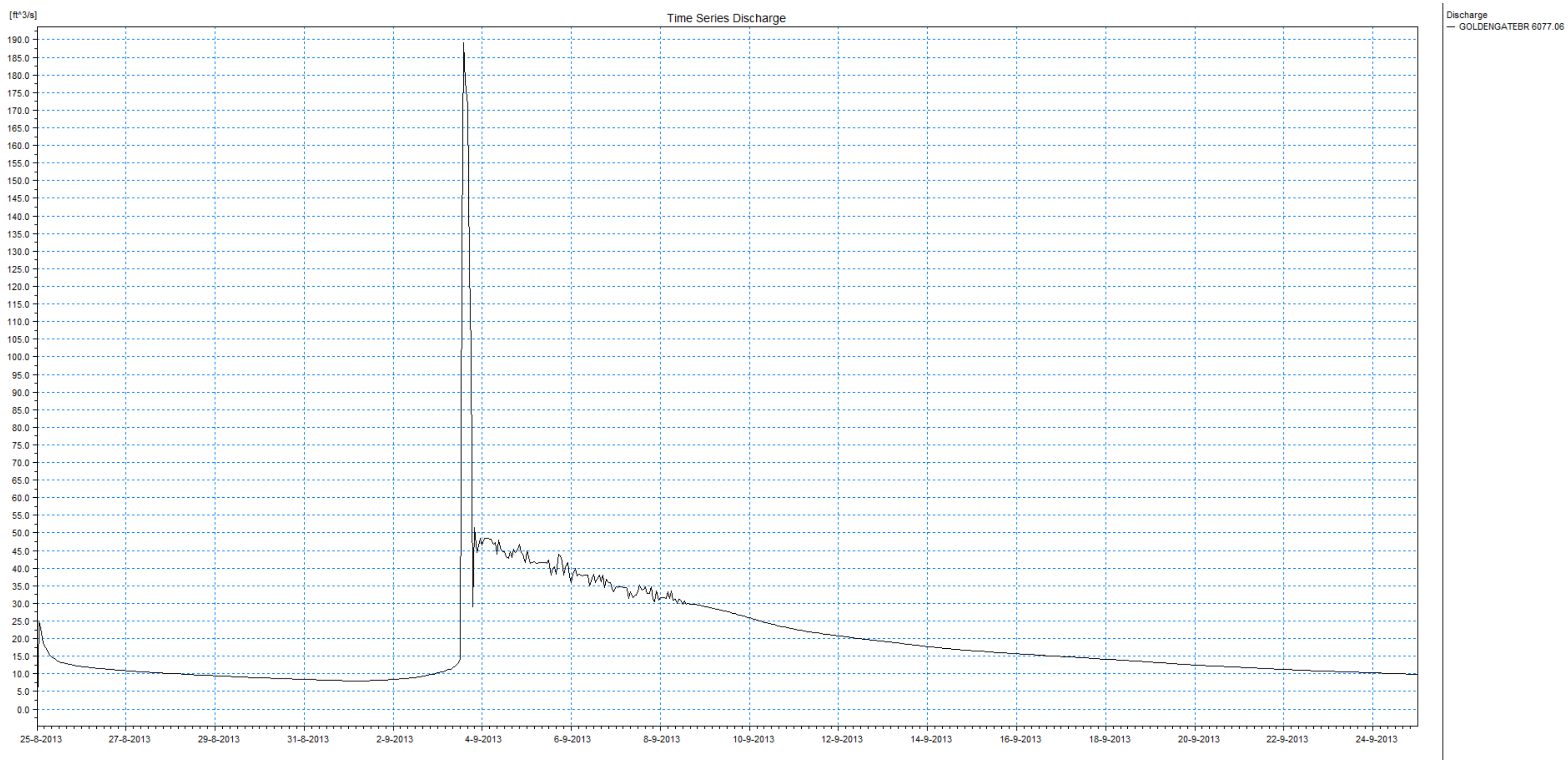

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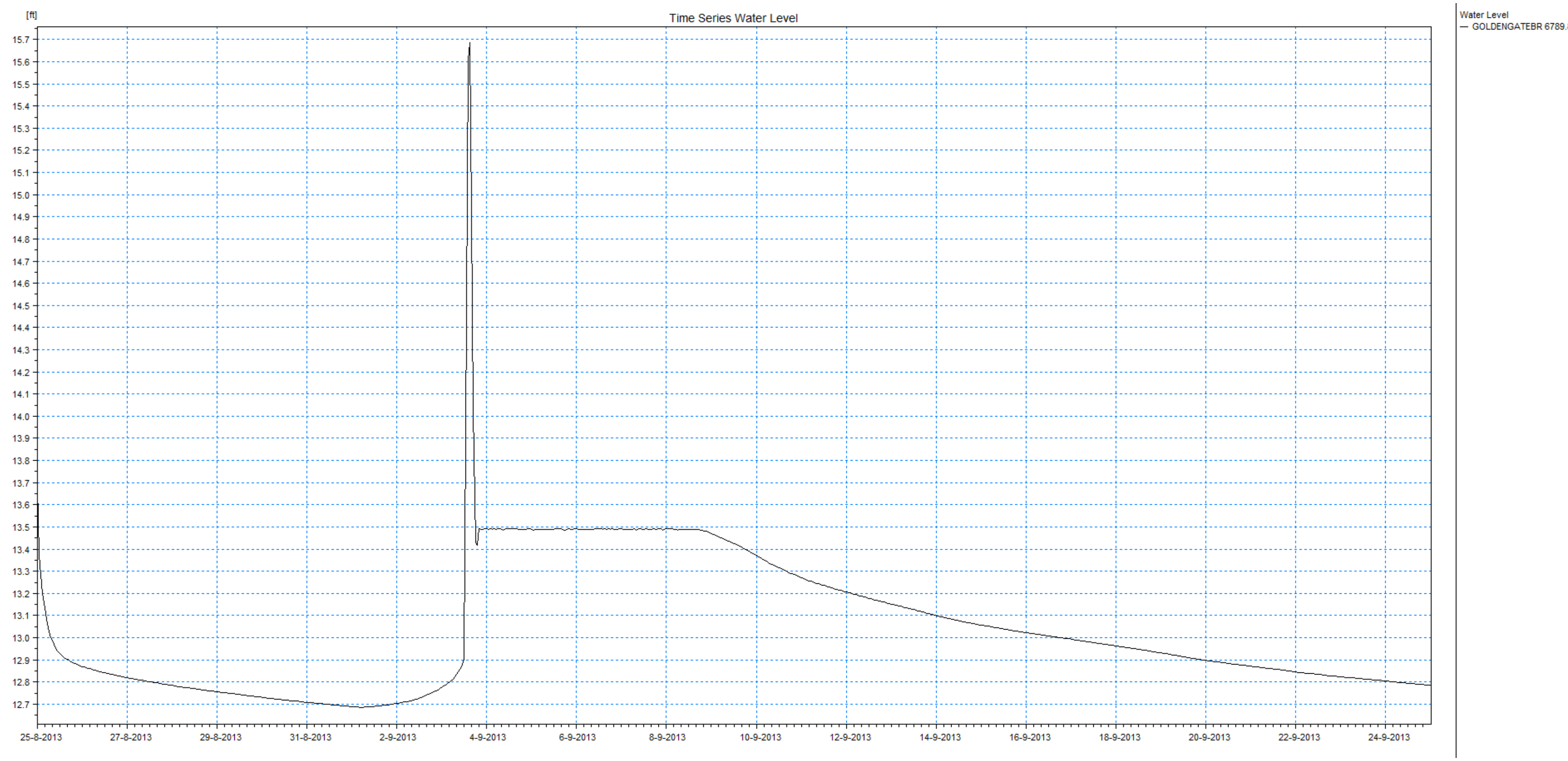

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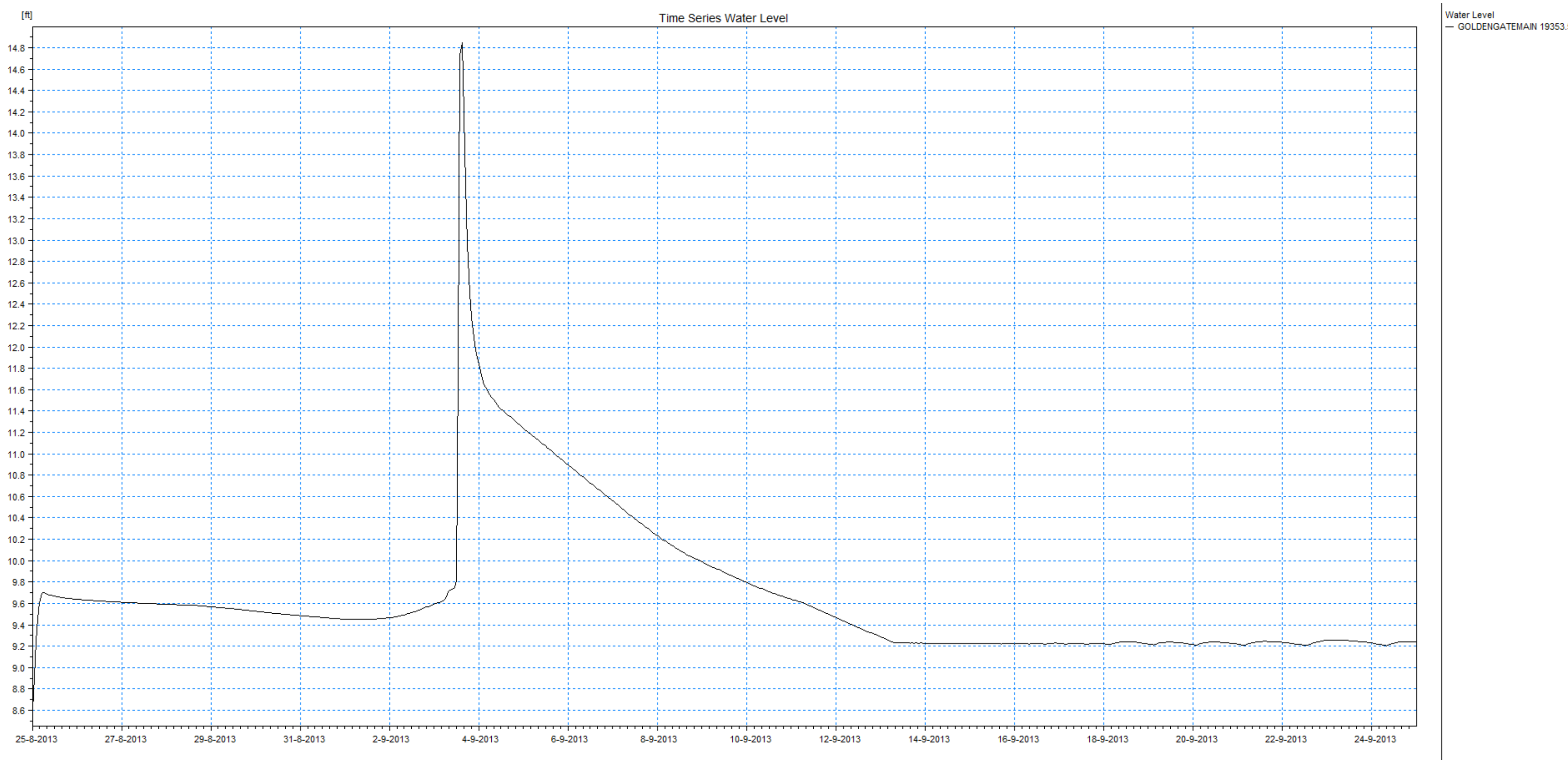

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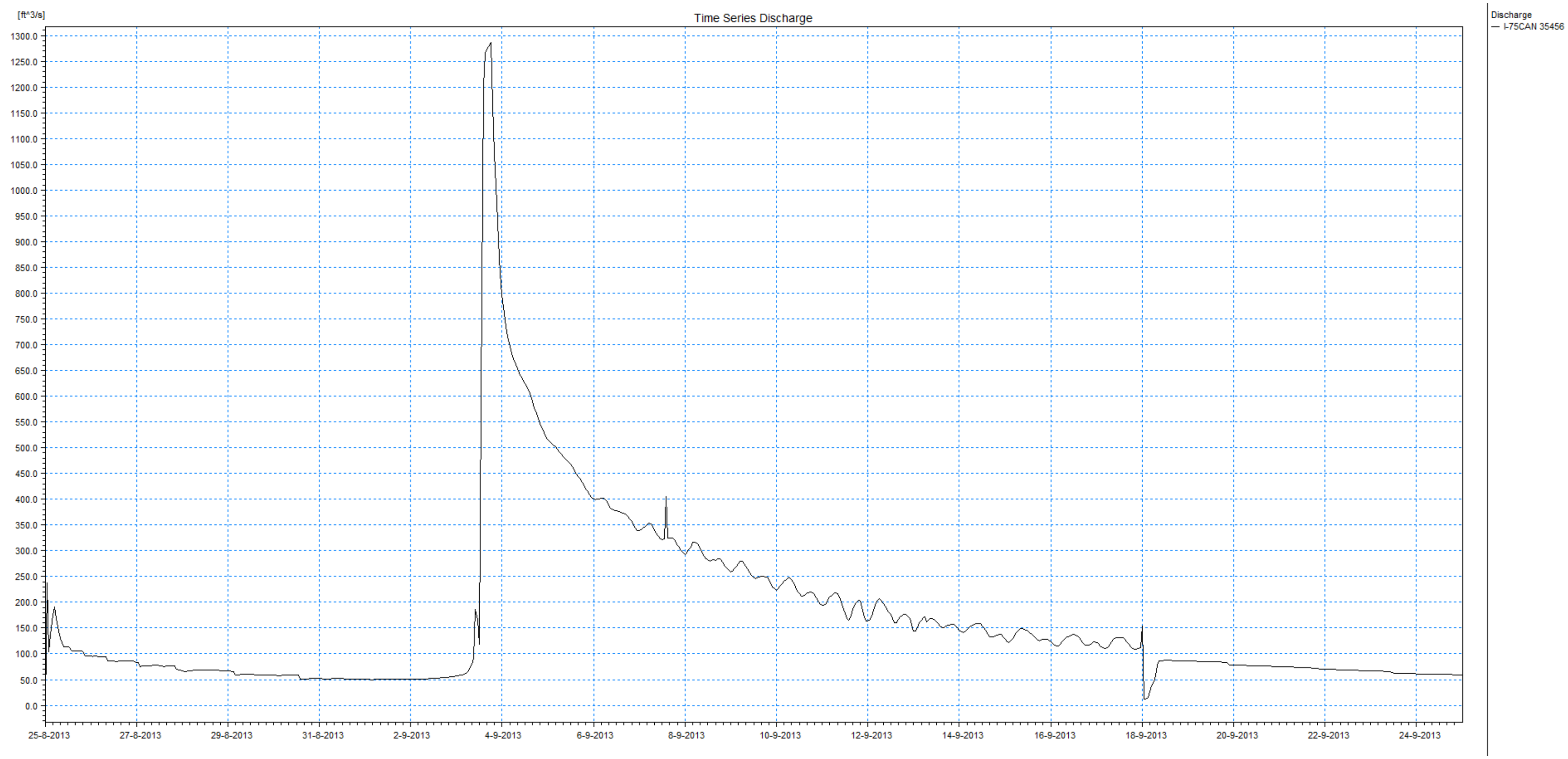

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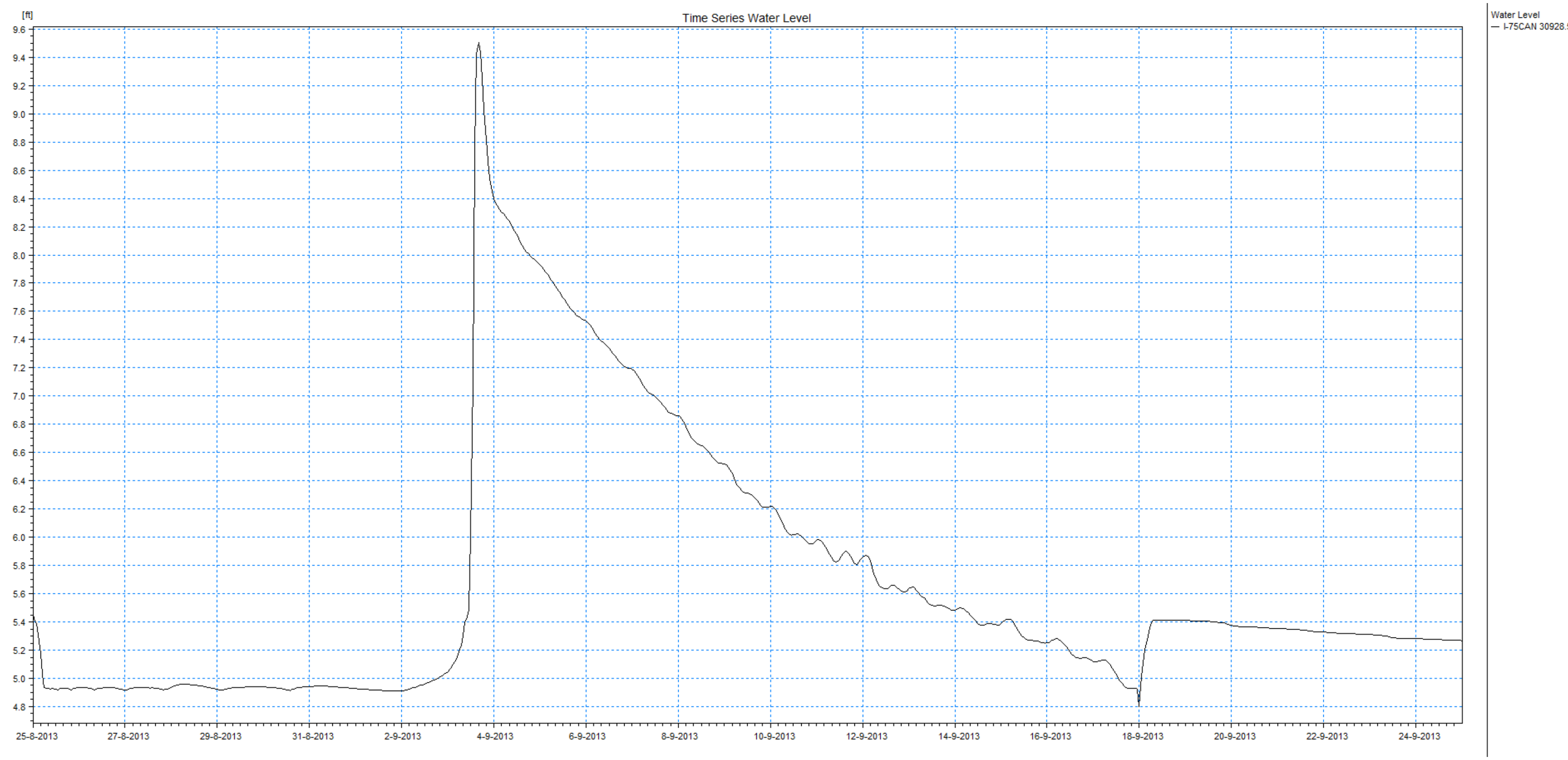

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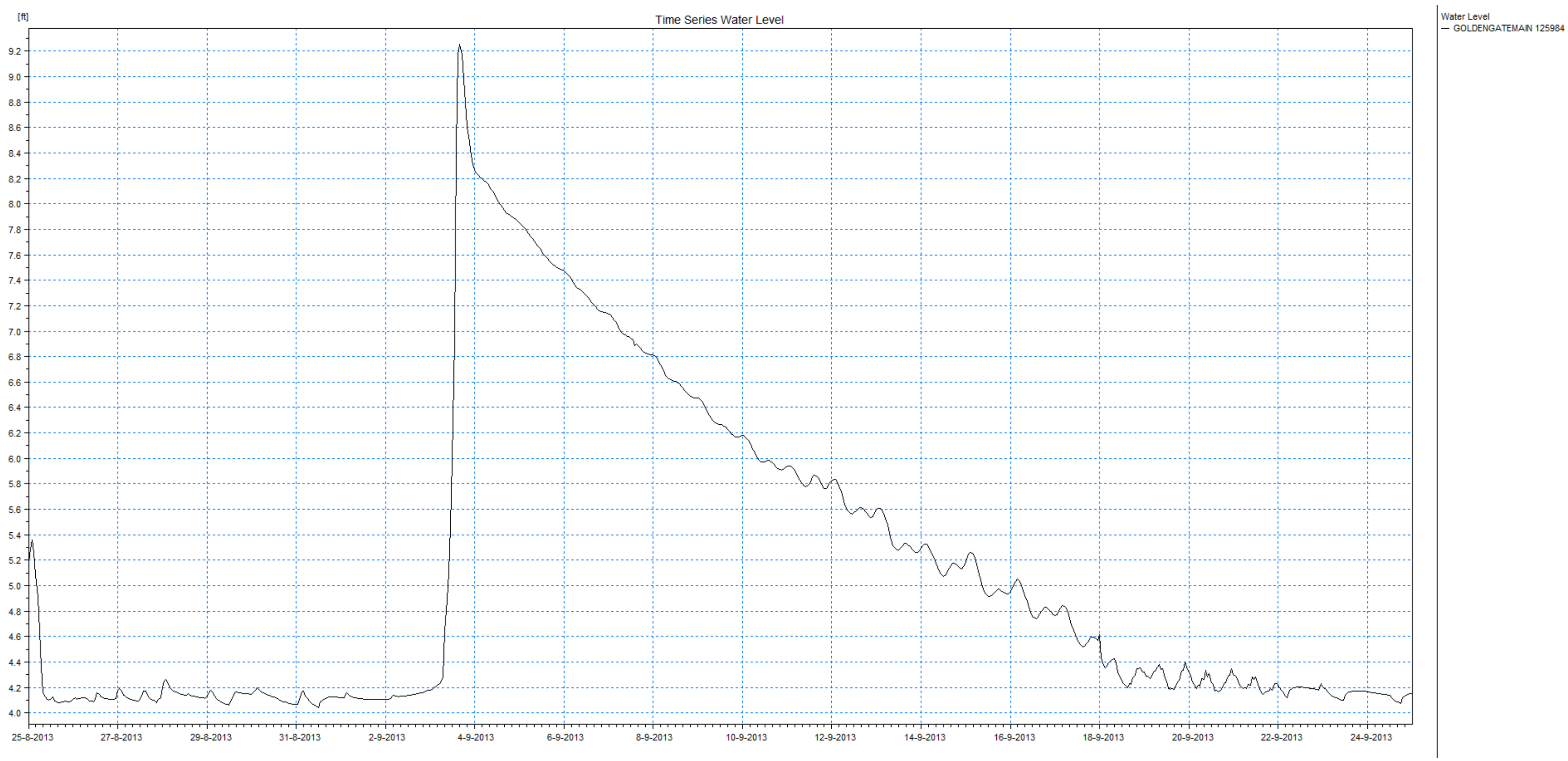
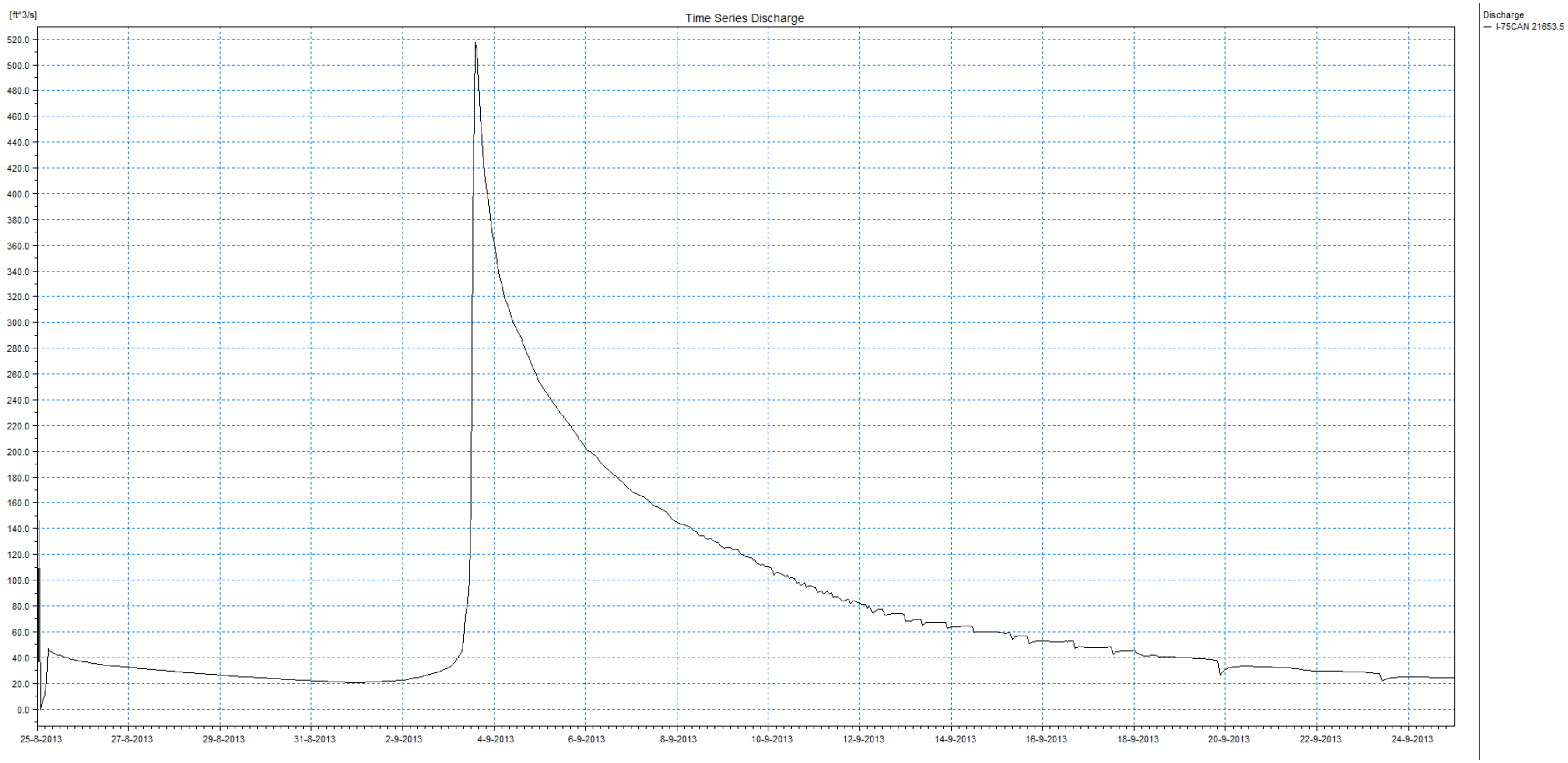


Figure 43. I75W1 Tailwater Stage (ft)



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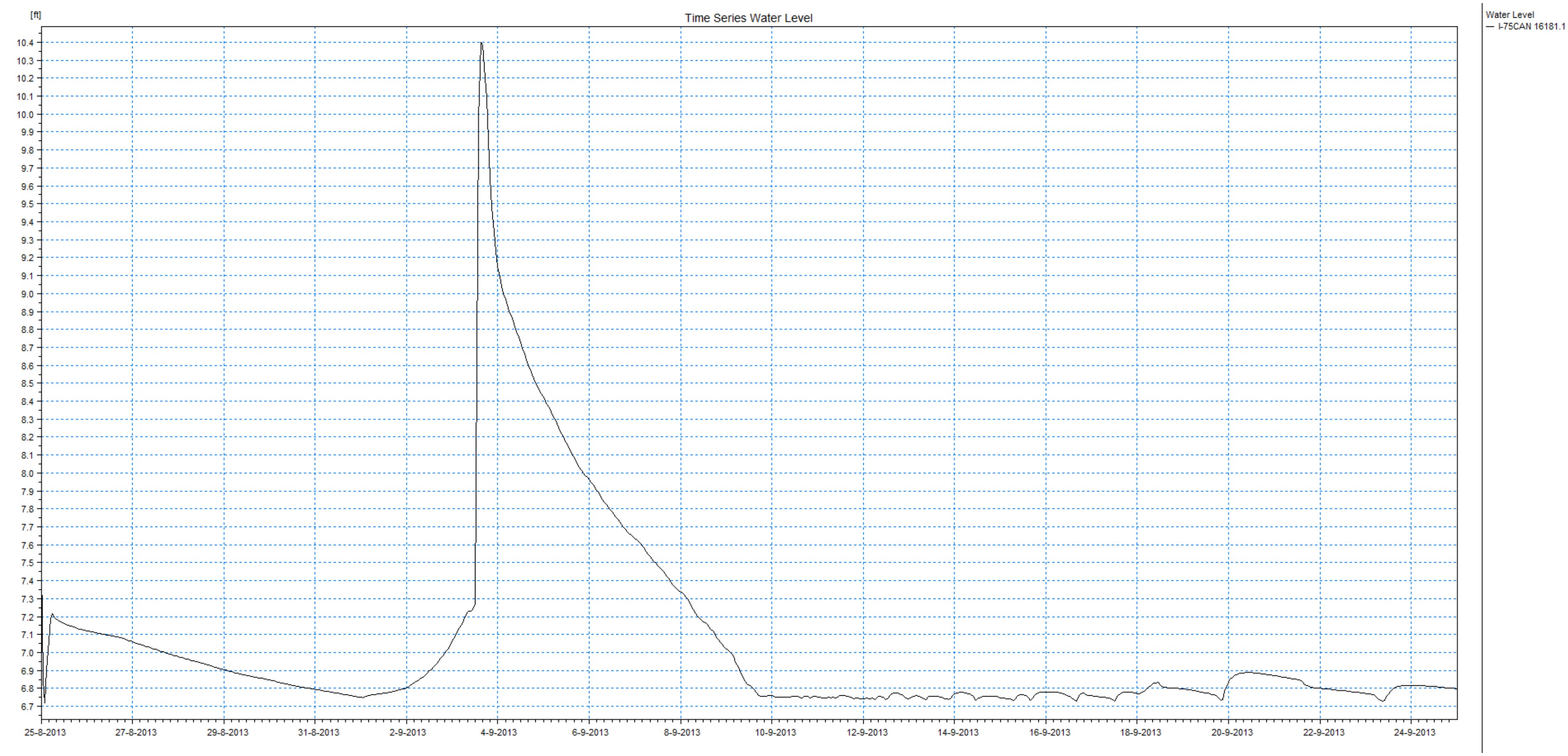


Figure 45. I75W2 Headwater Stage (ft)

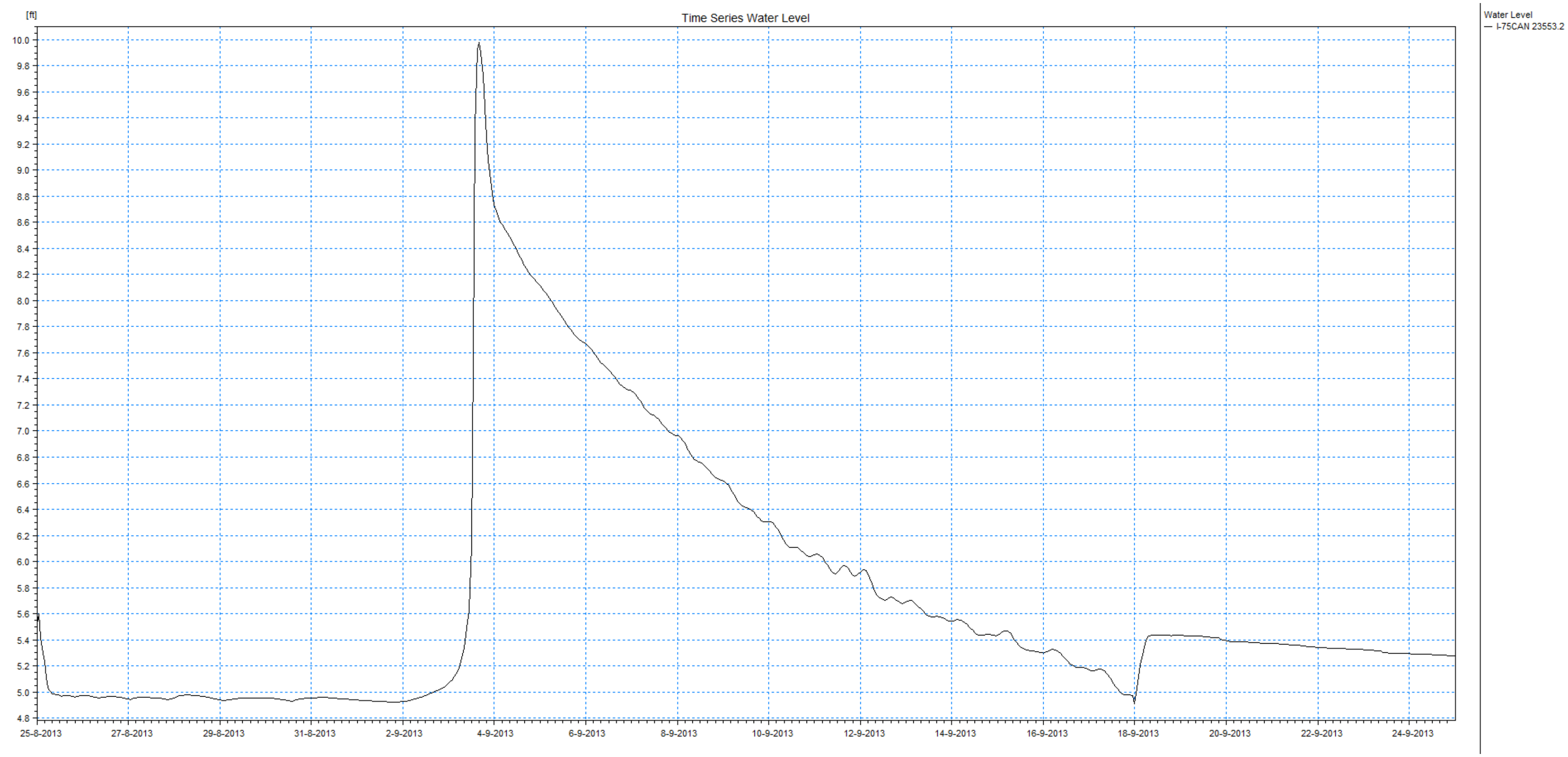
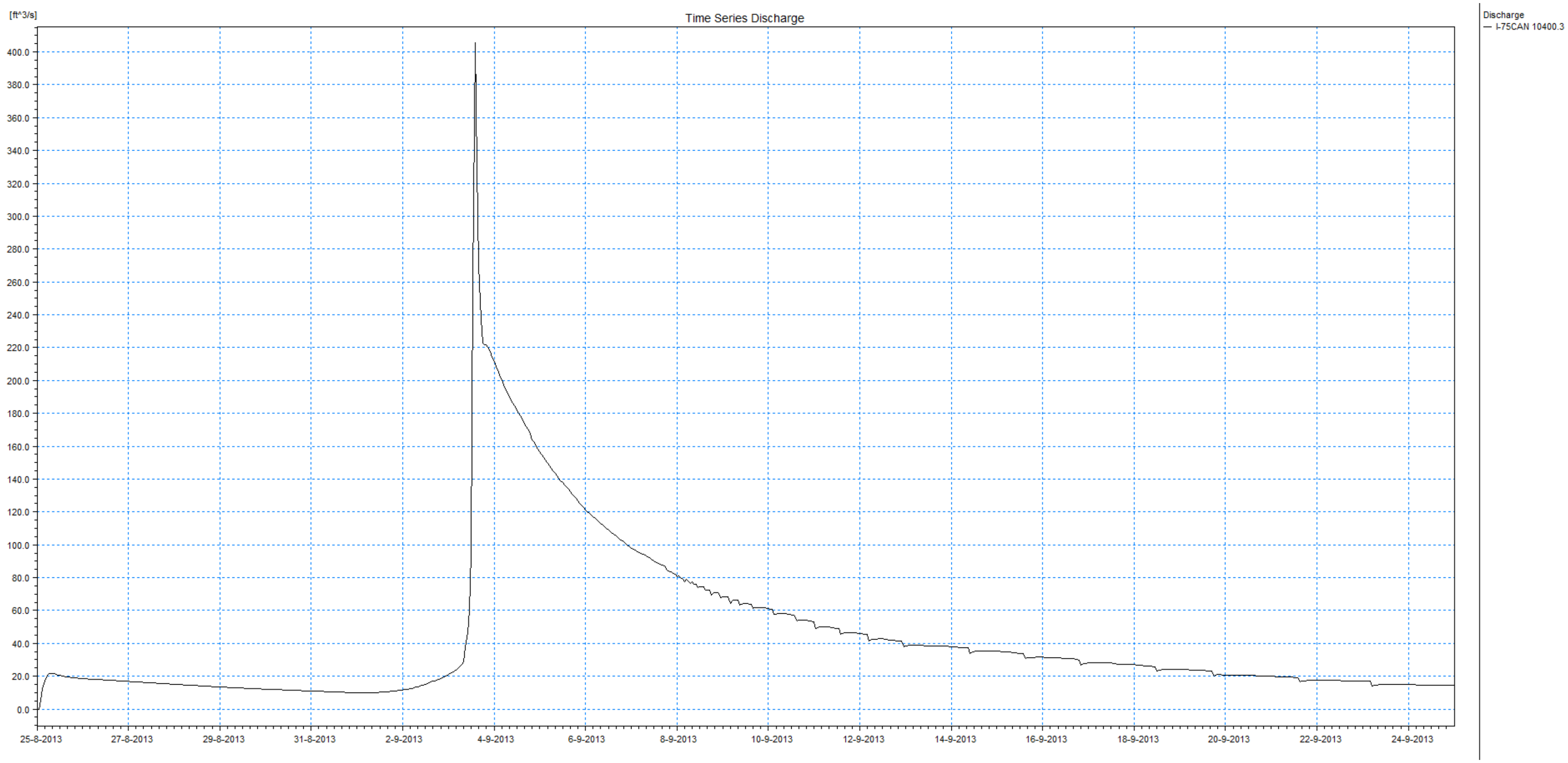


Figure 46. I75W2 Tailwater Stage (ft)



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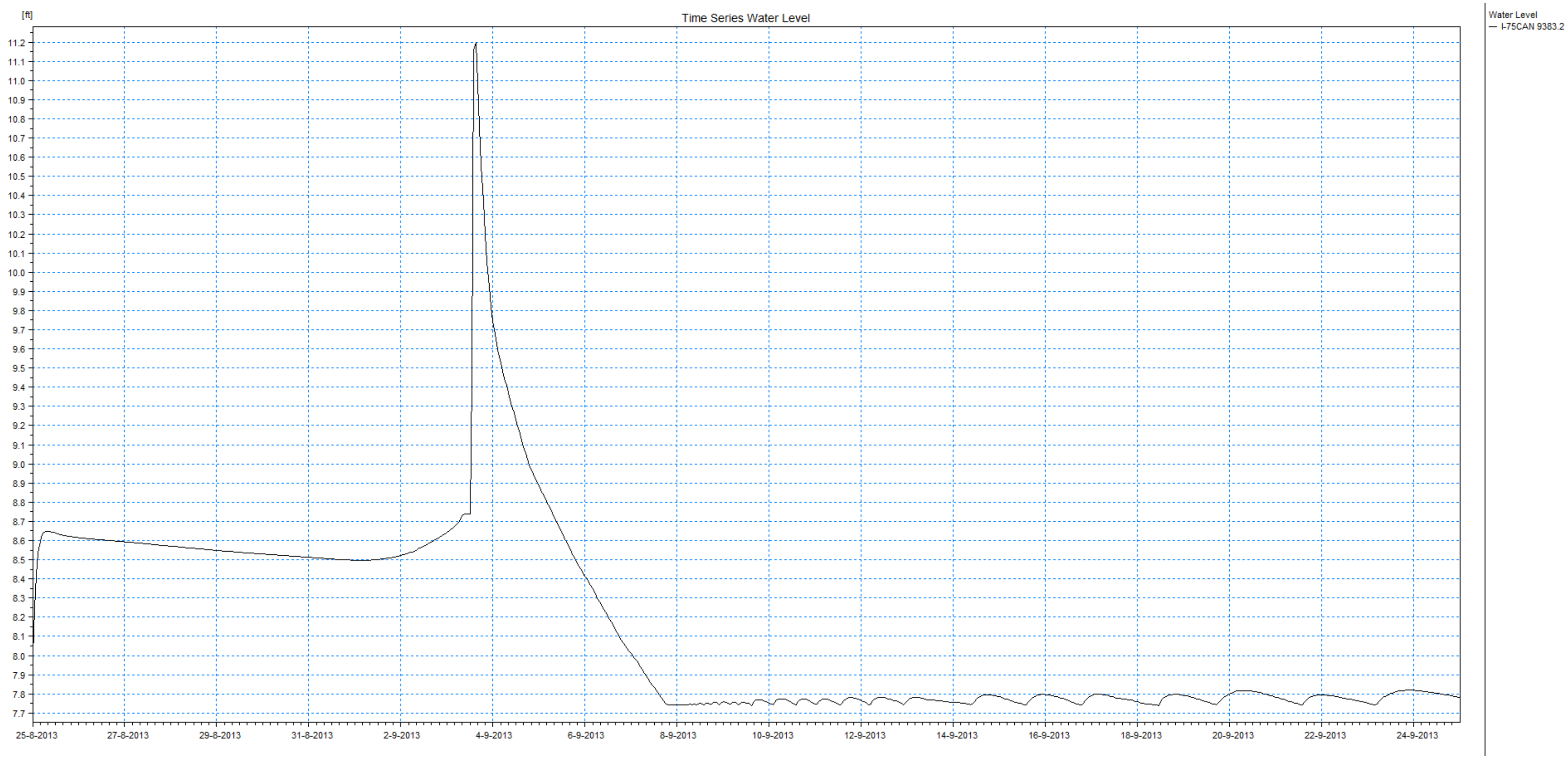

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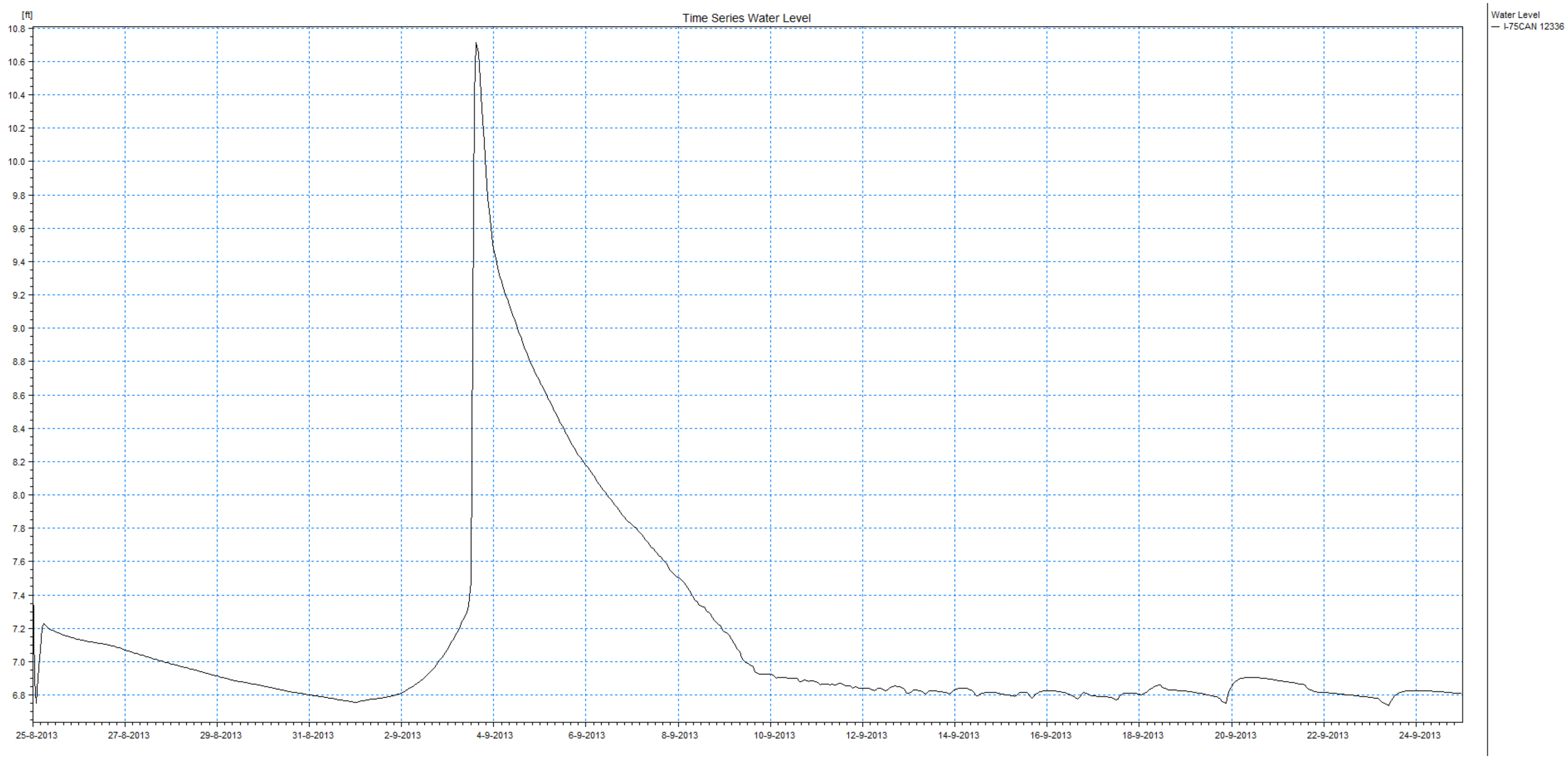

Figure 49. I75W3 Tailwater Stage (ft)

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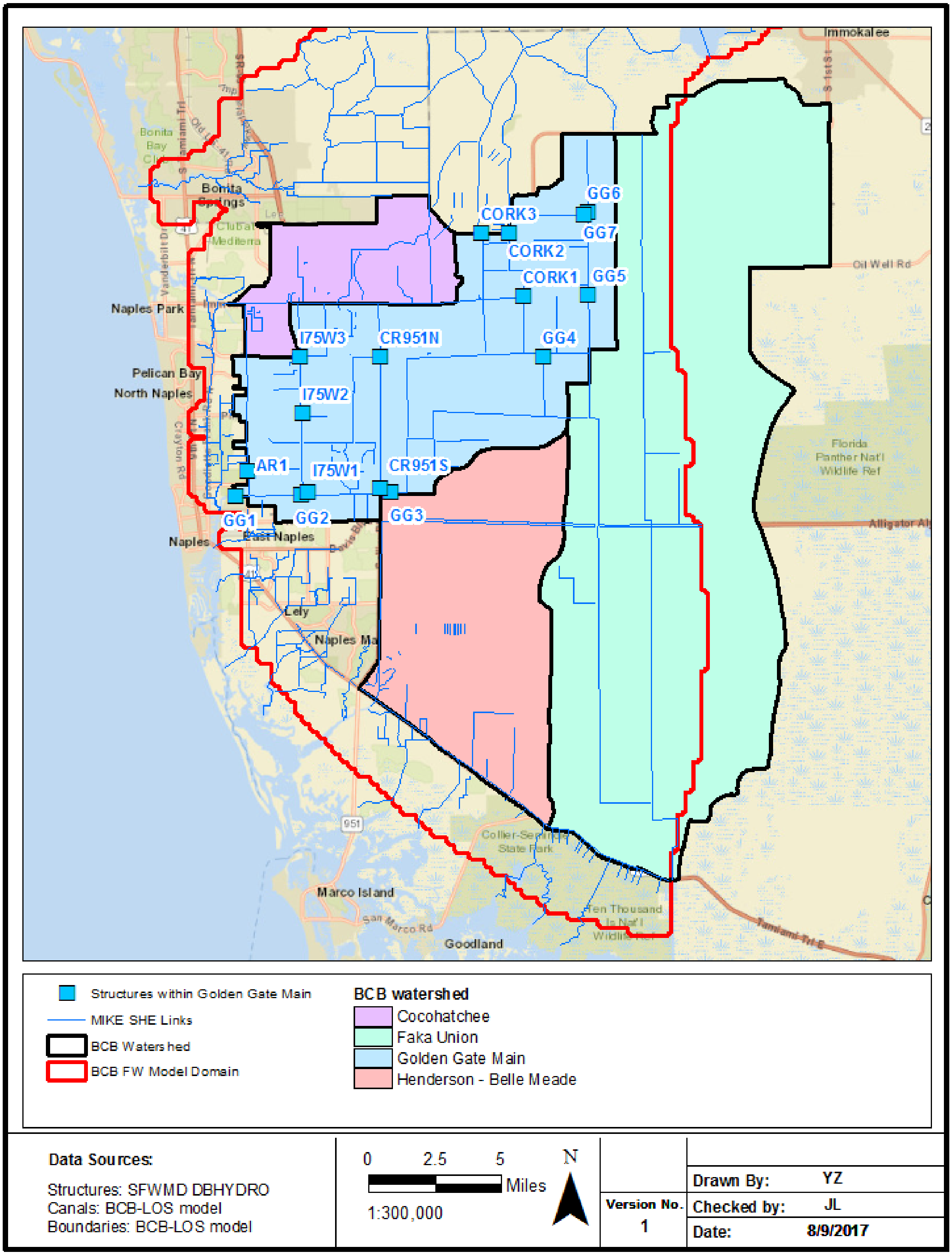

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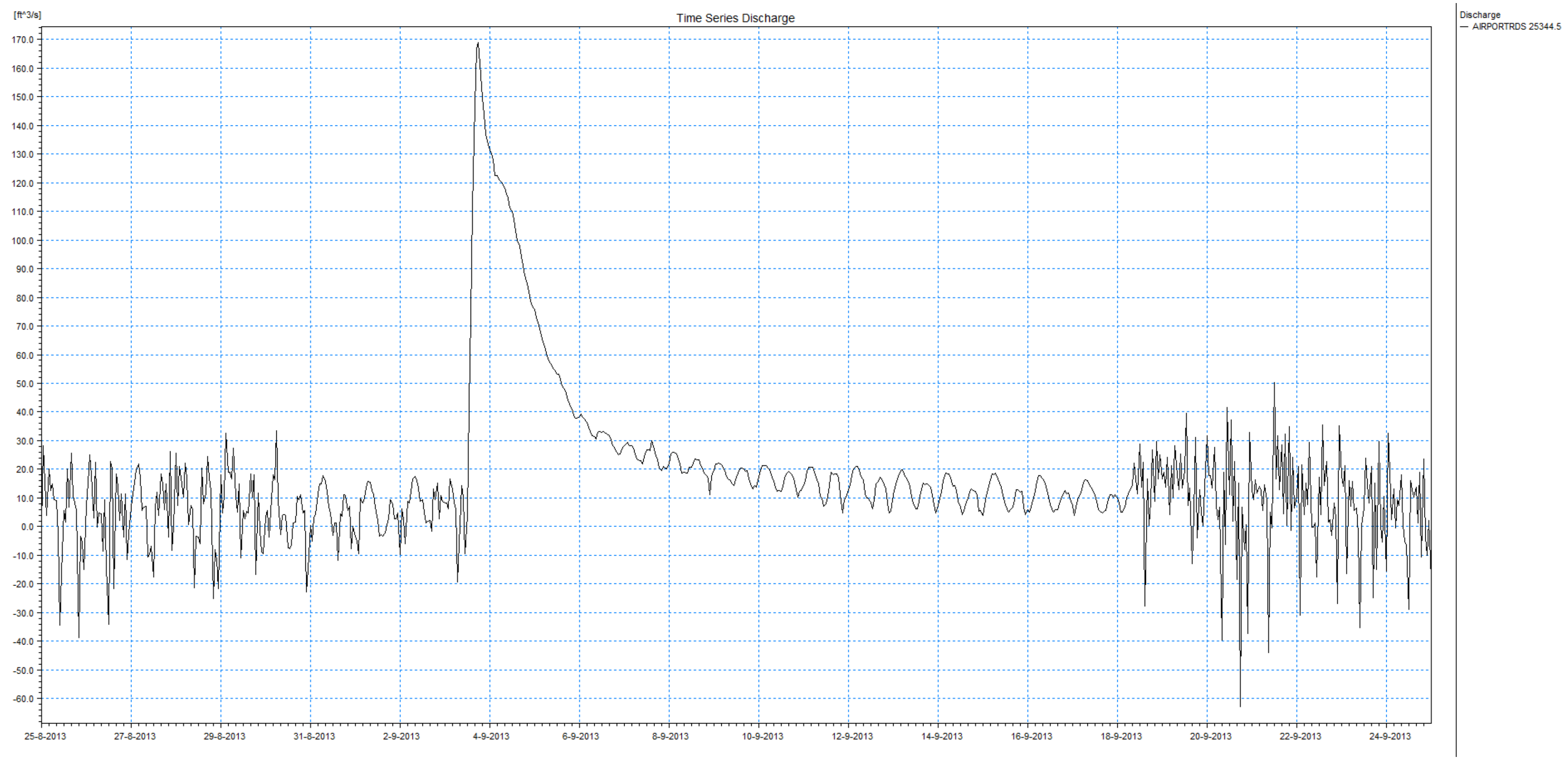

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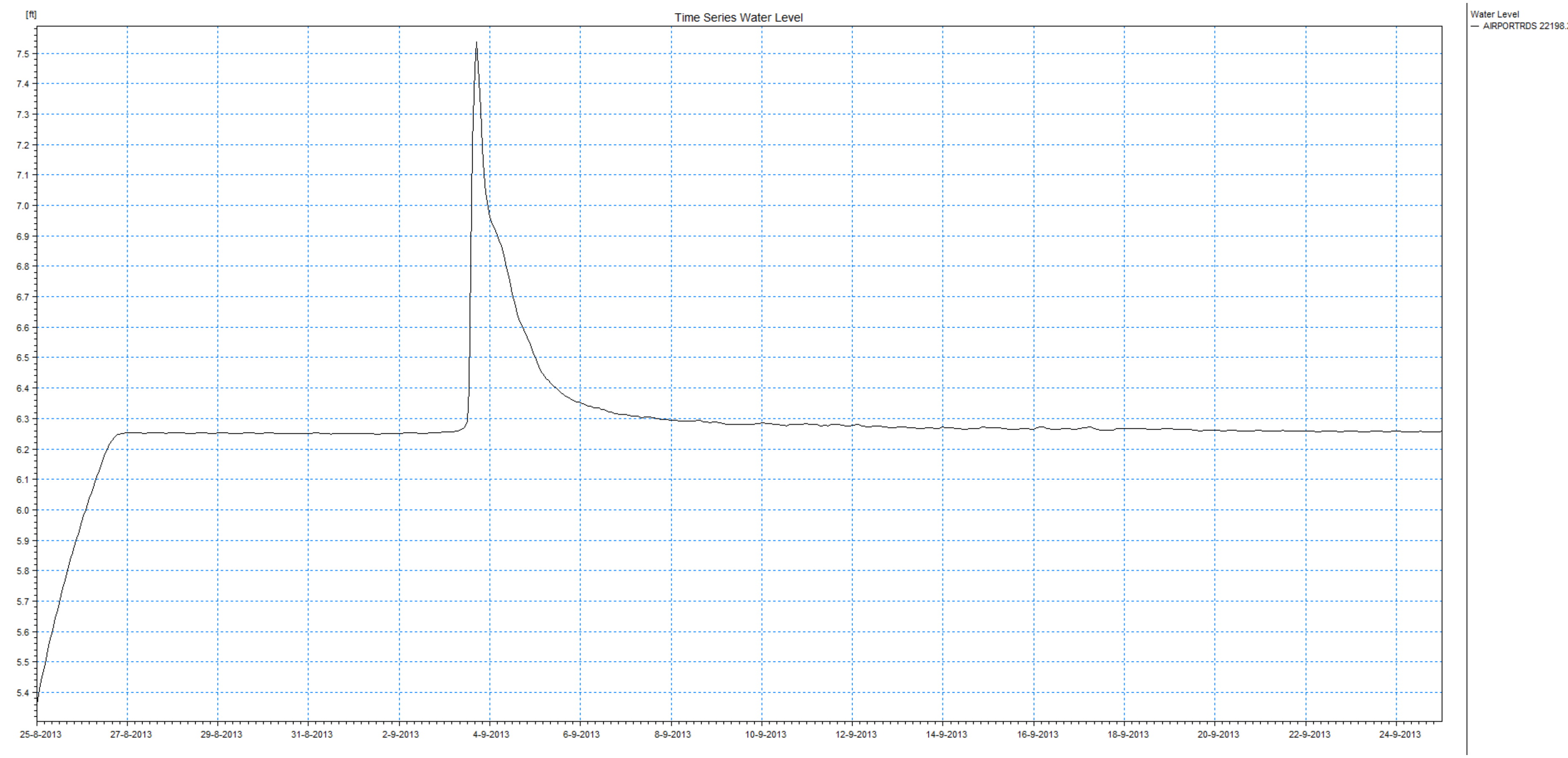

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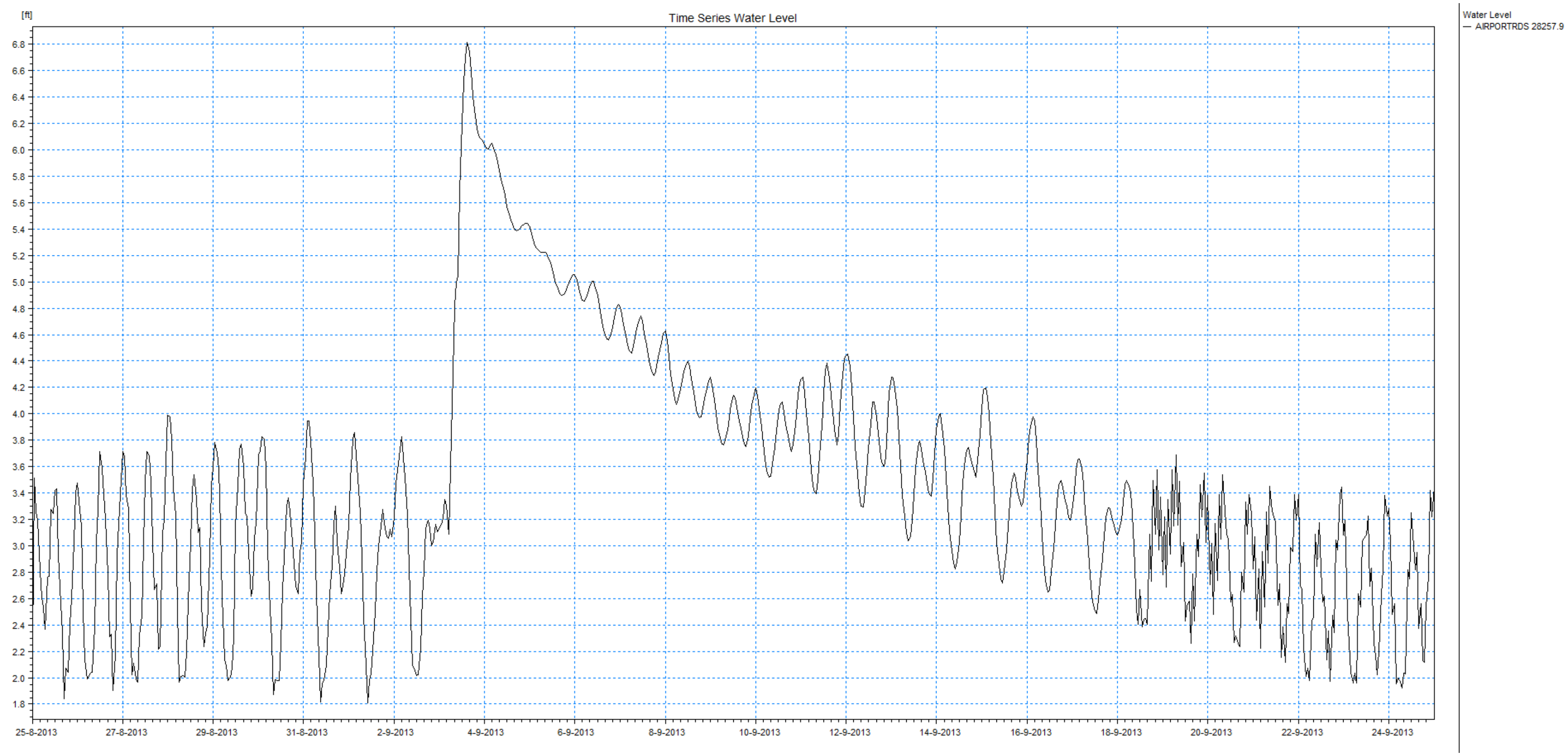

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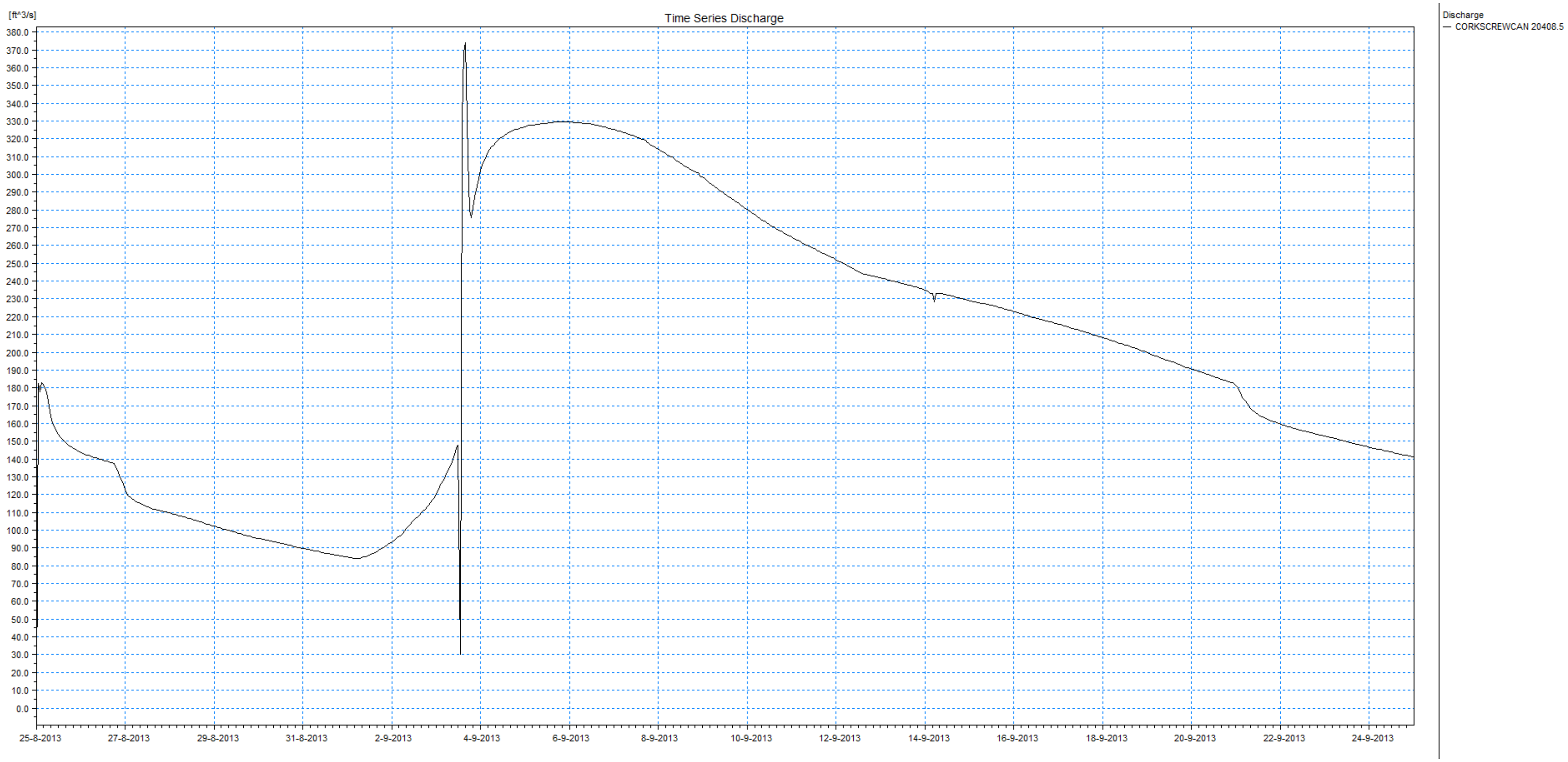

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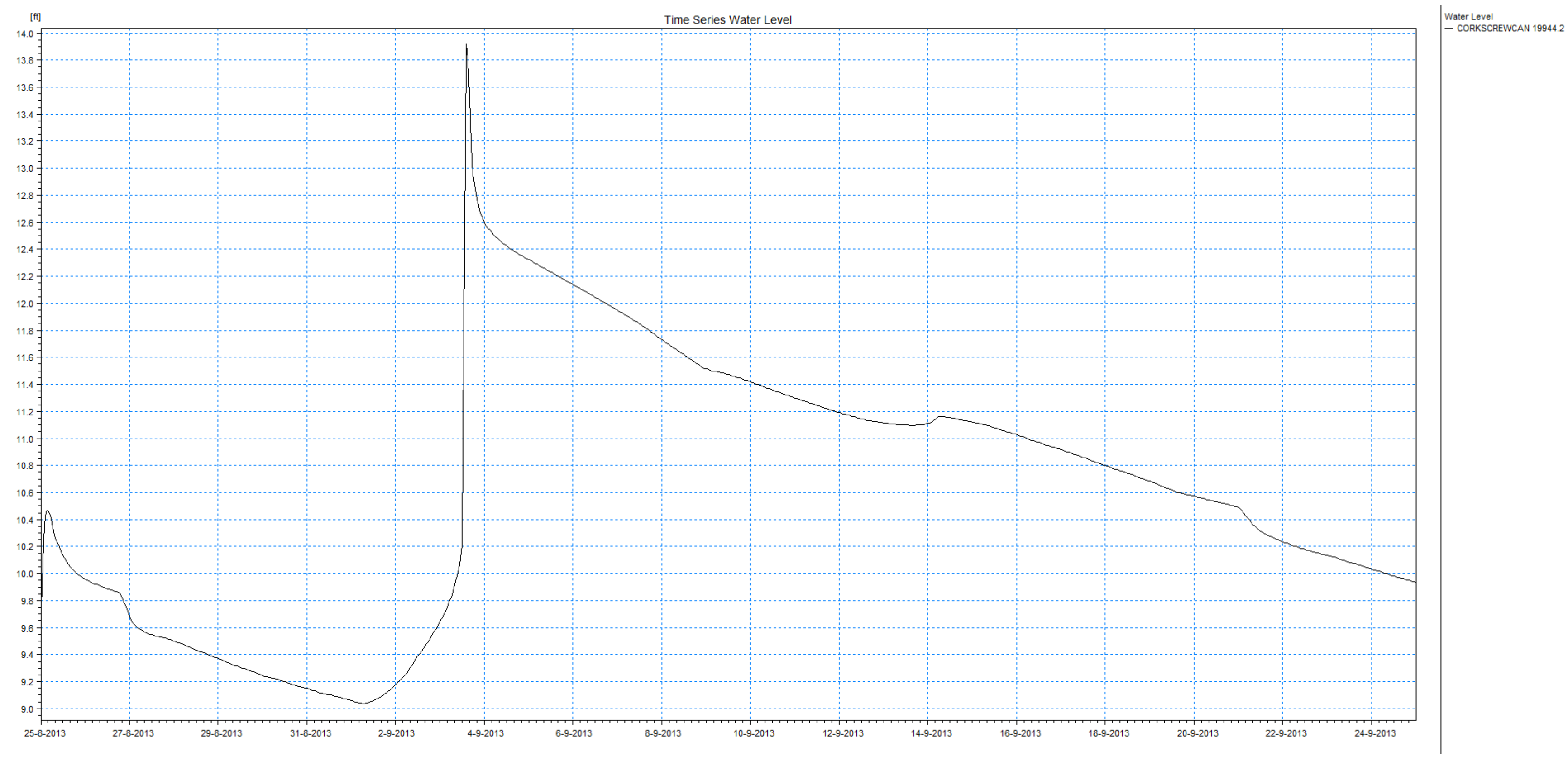

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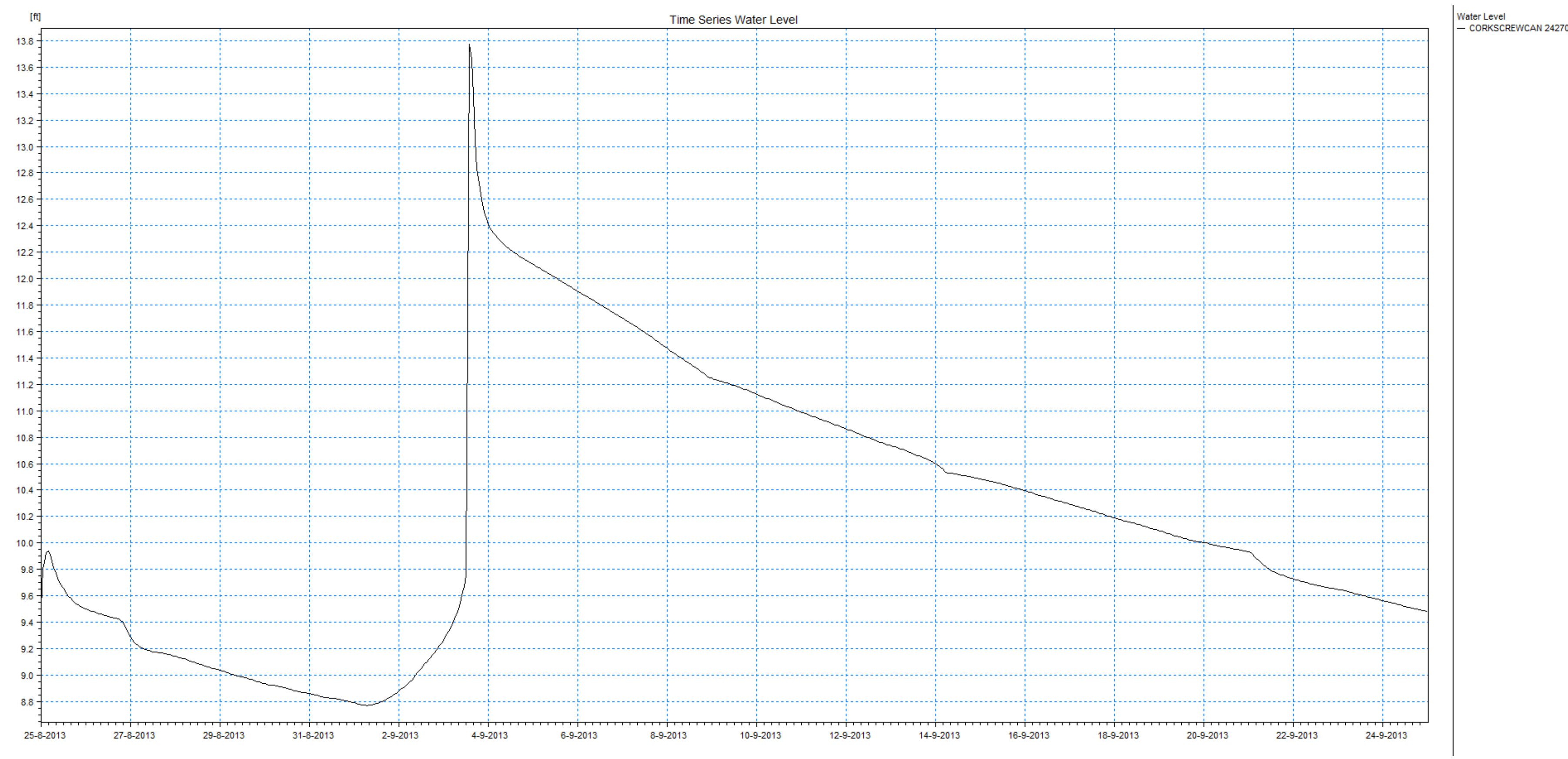

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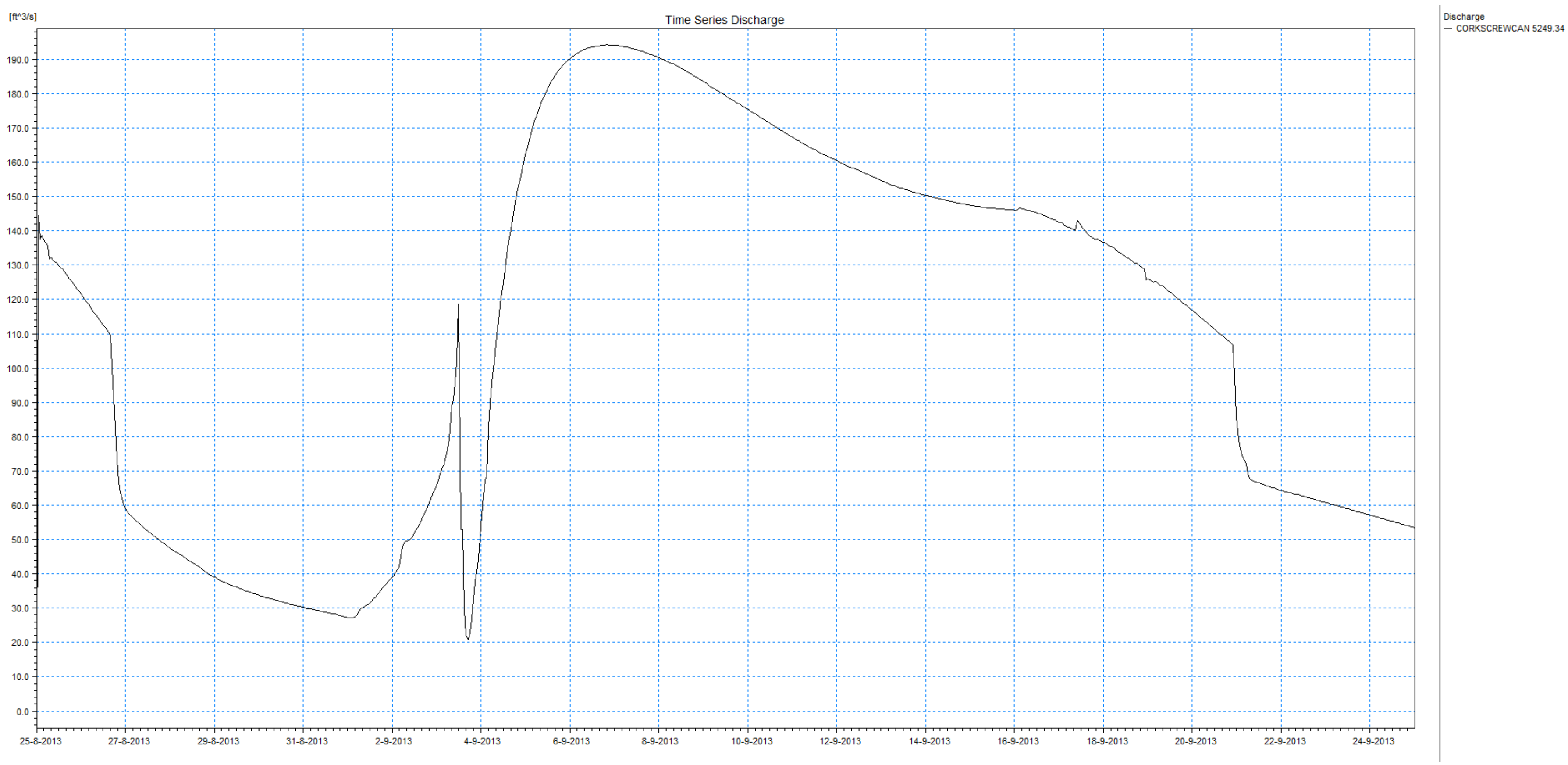

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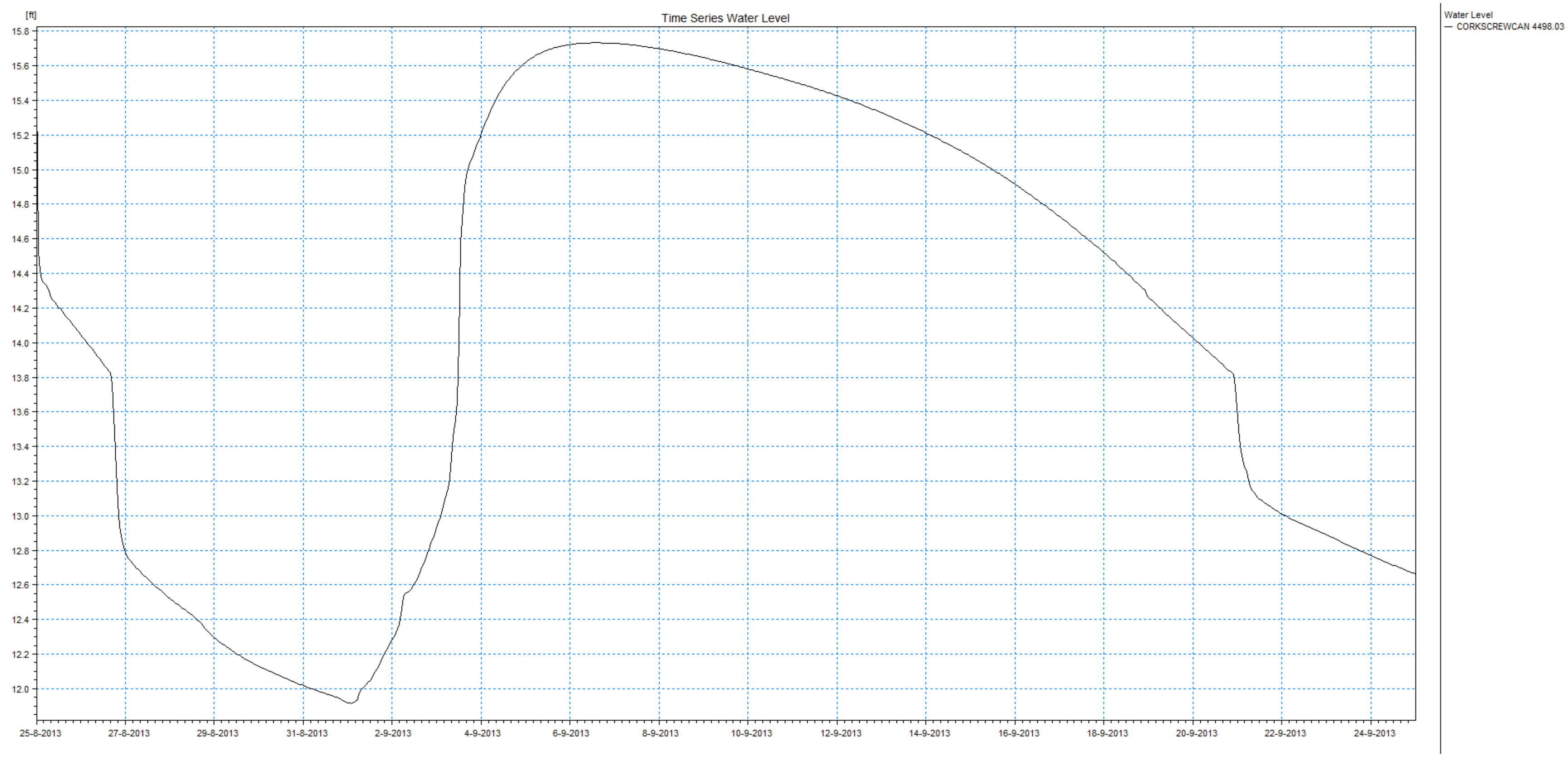

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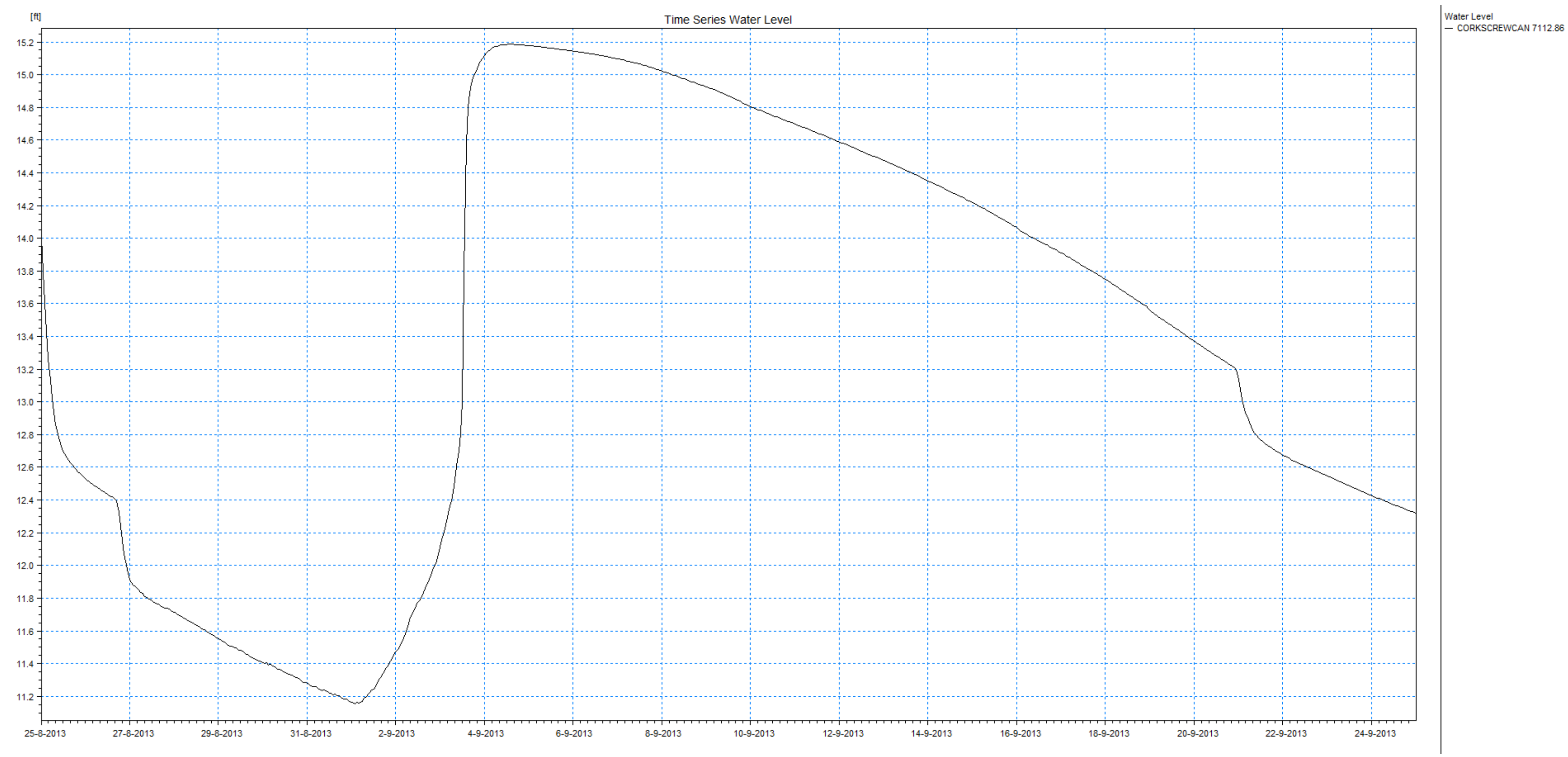

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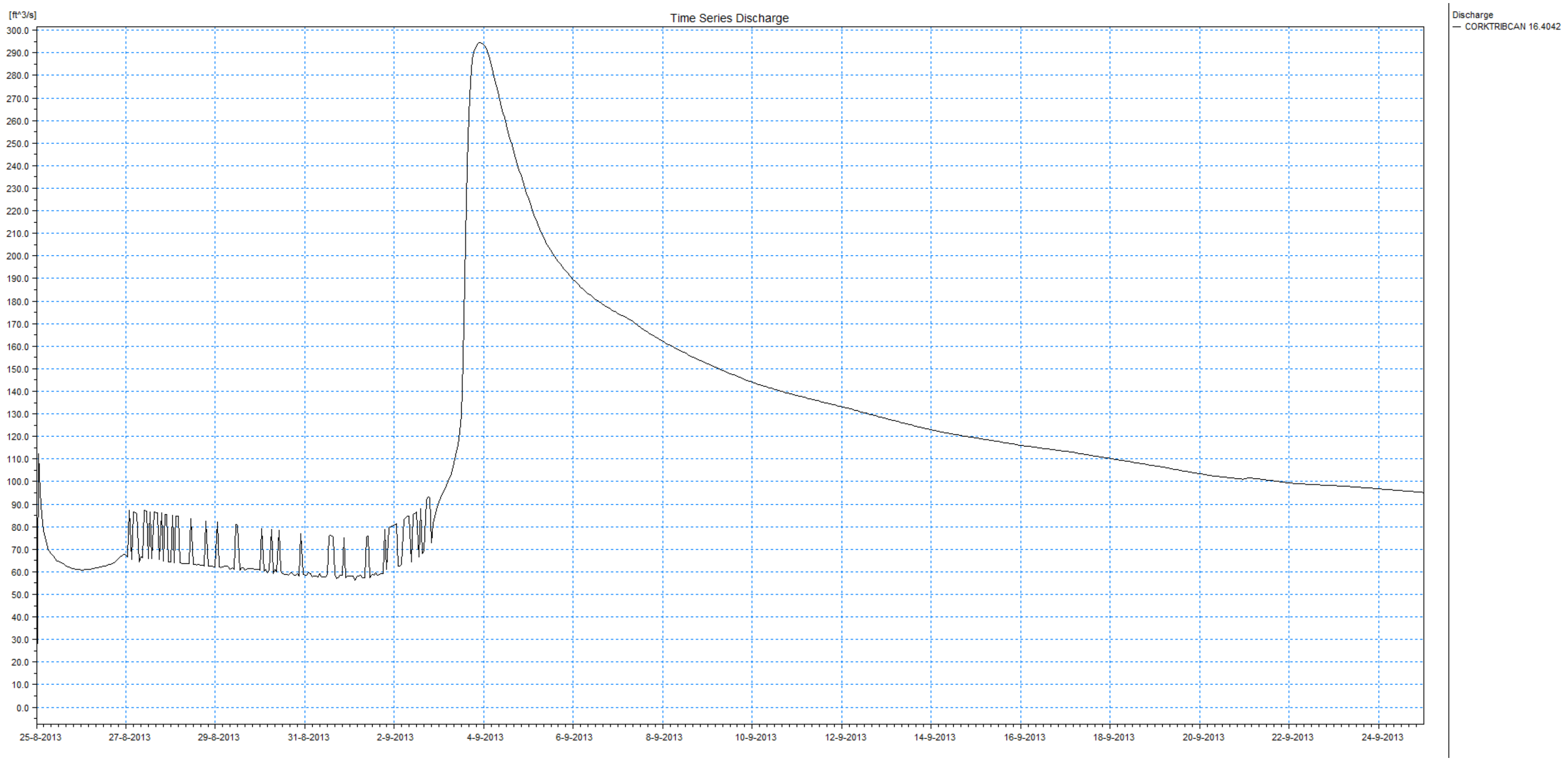

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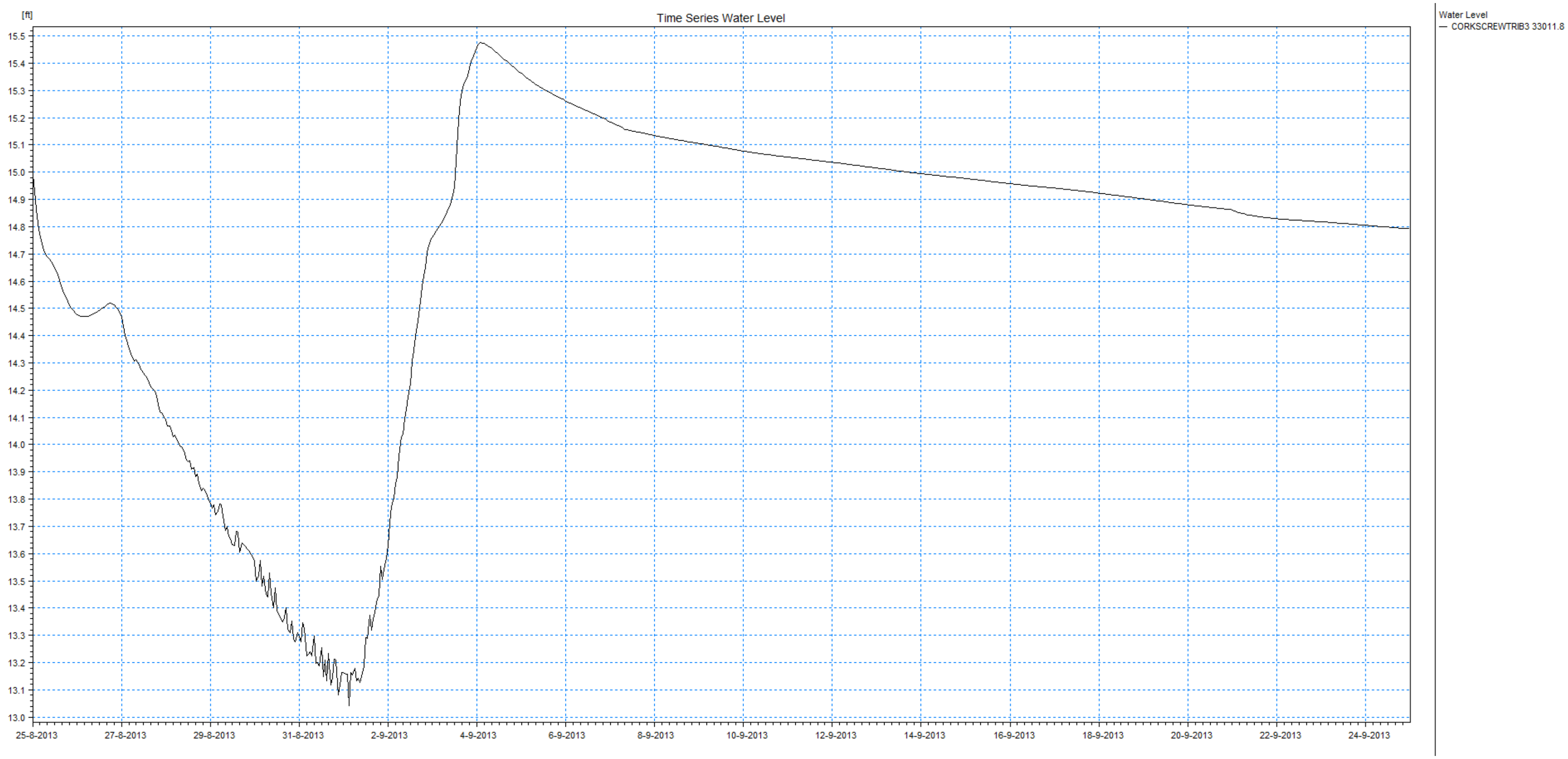

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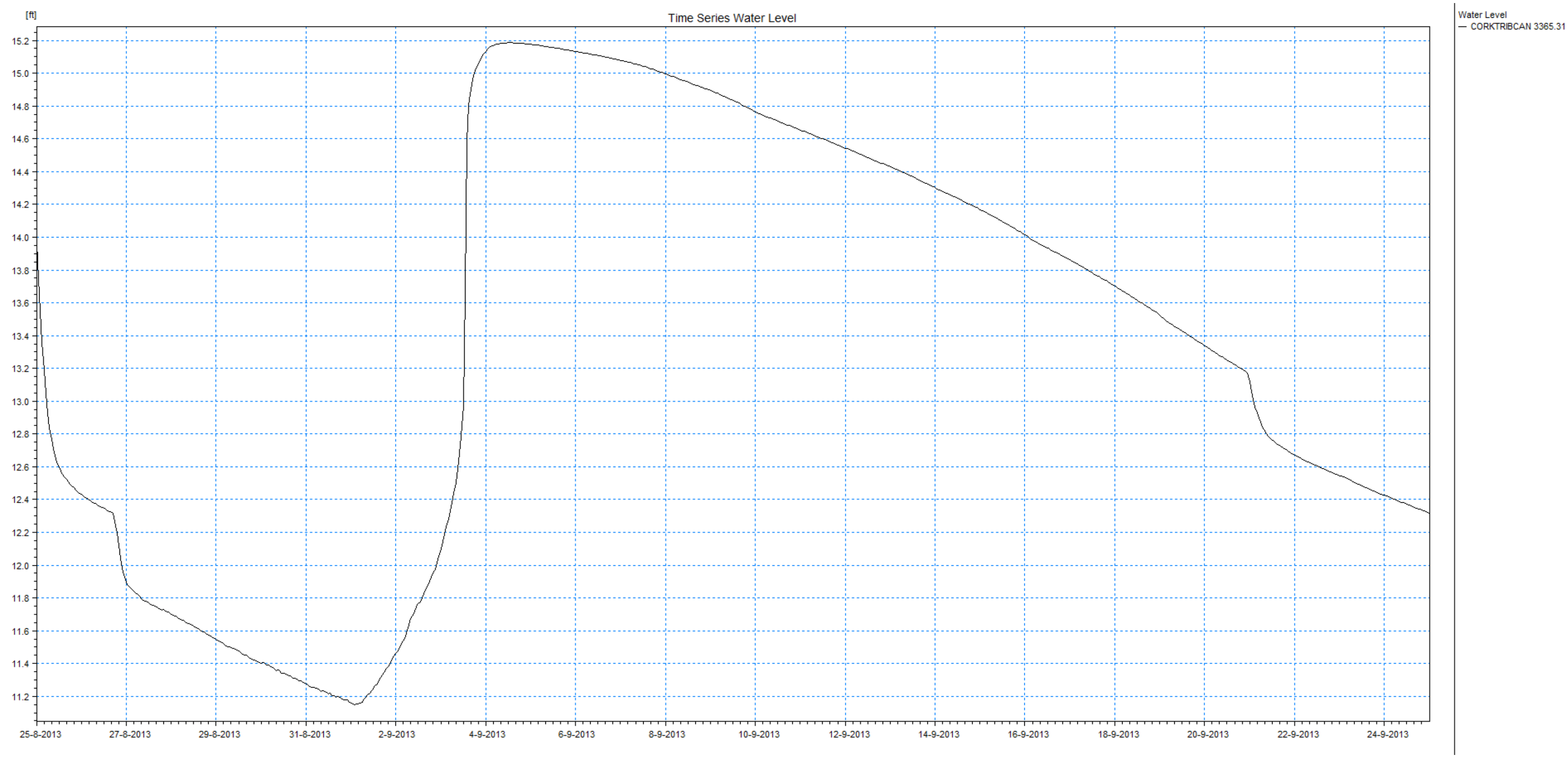

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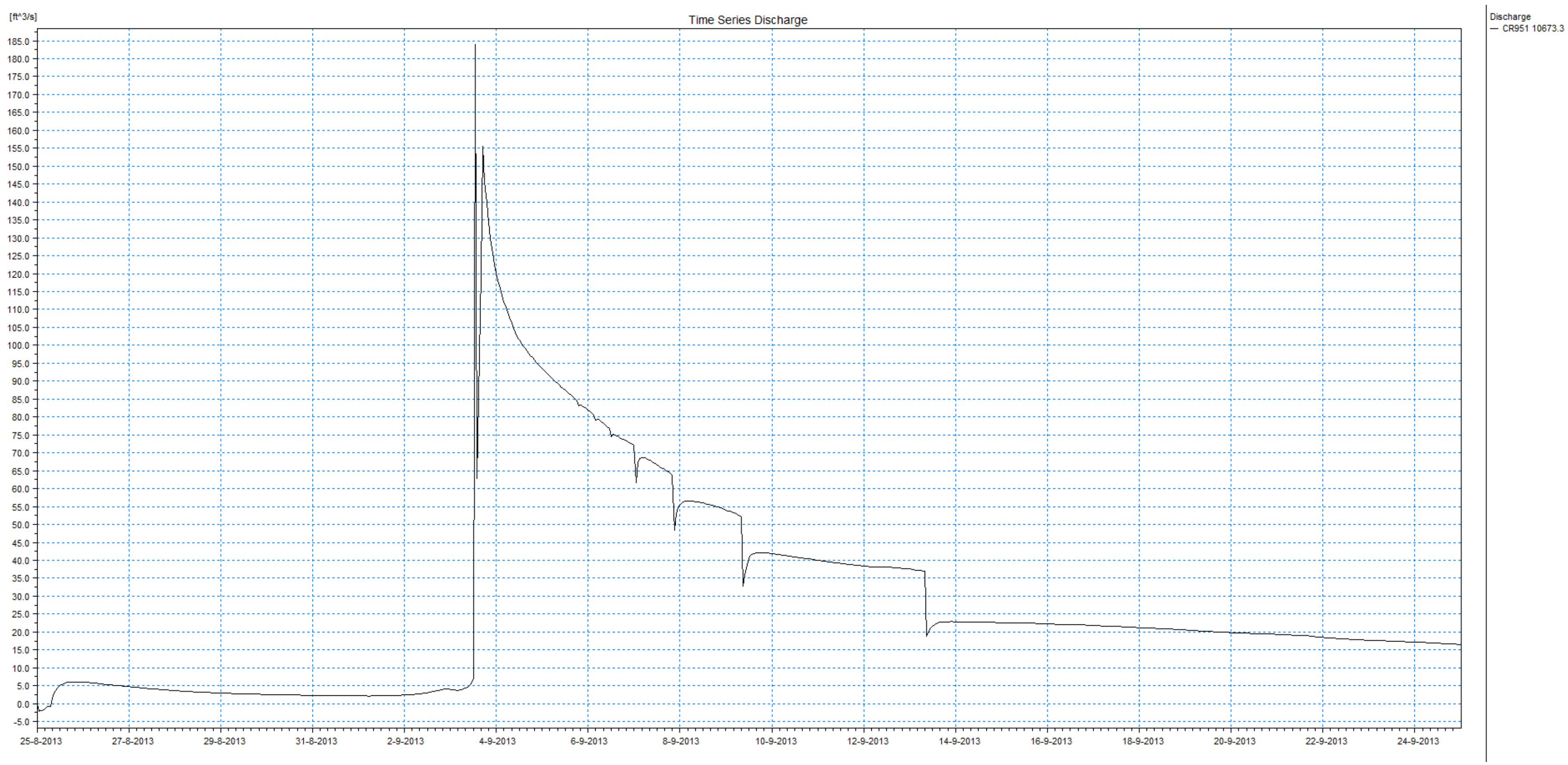

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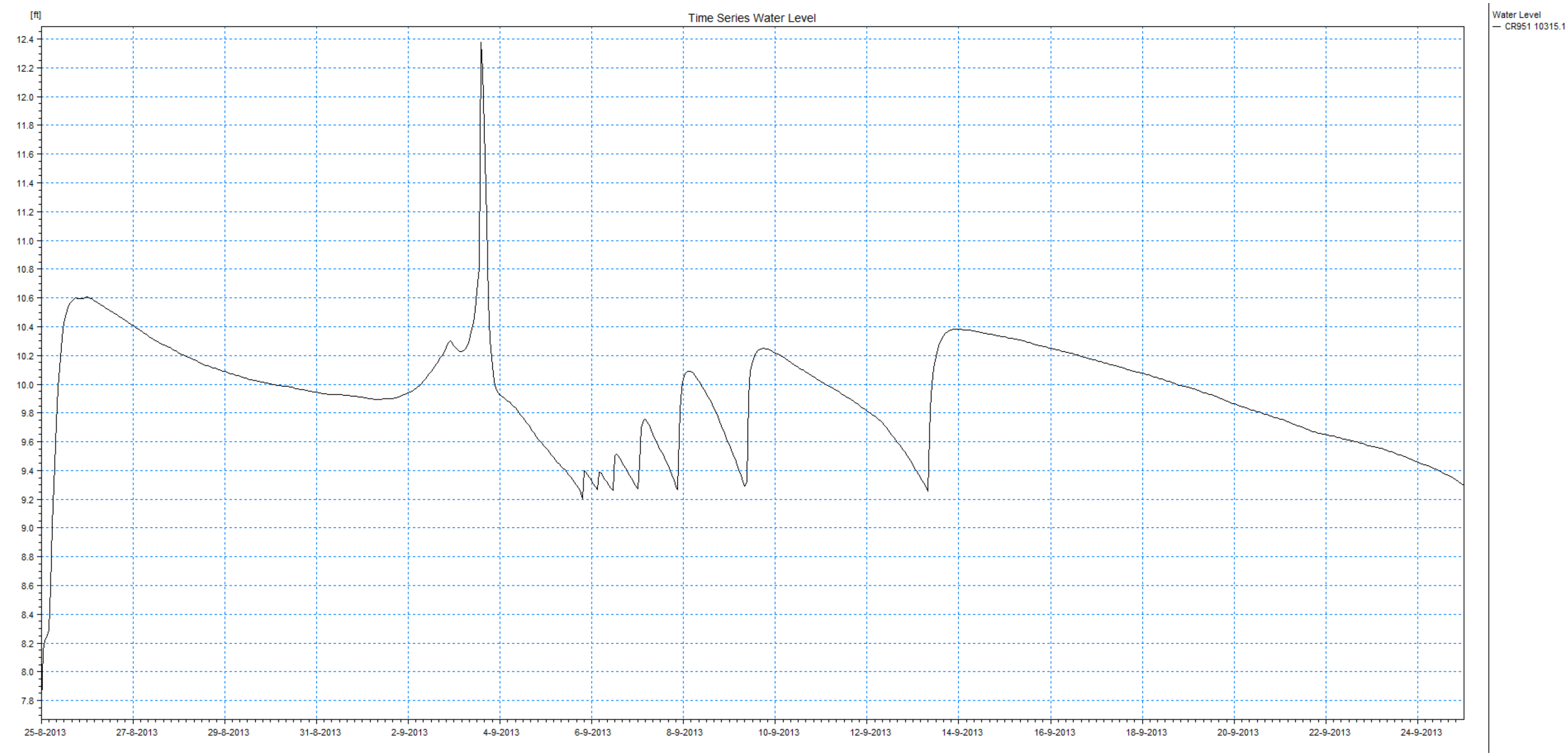

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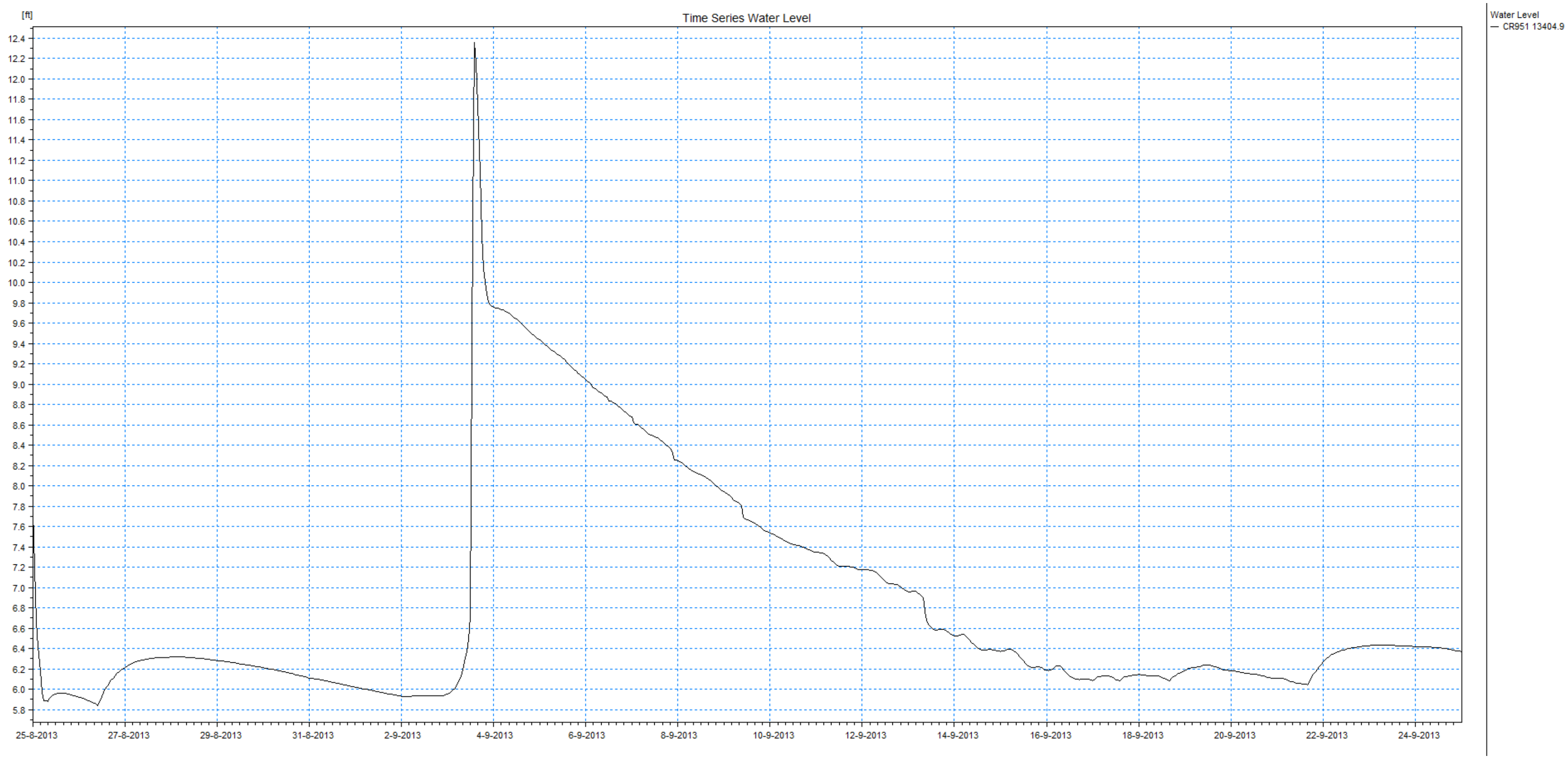

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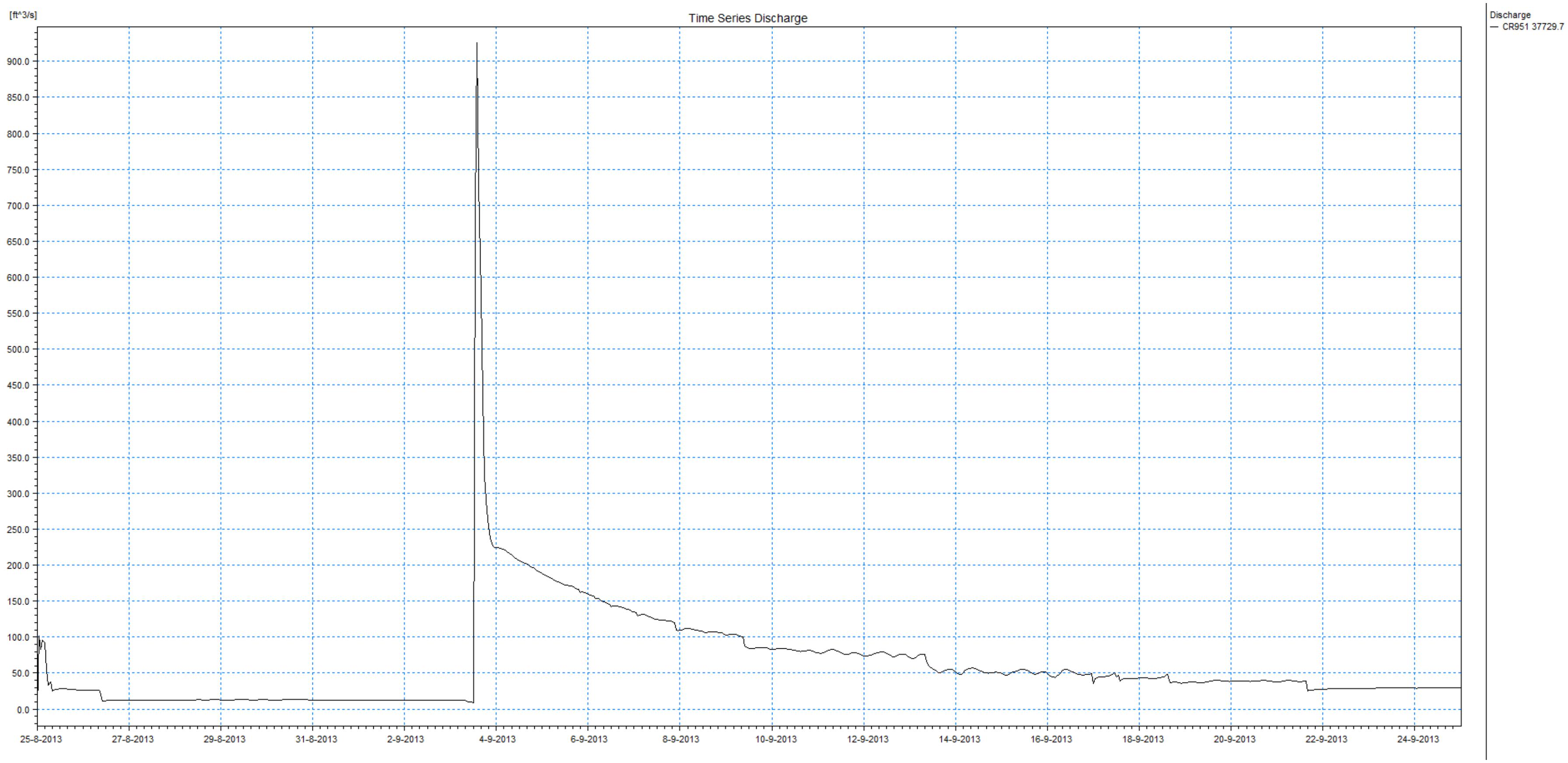

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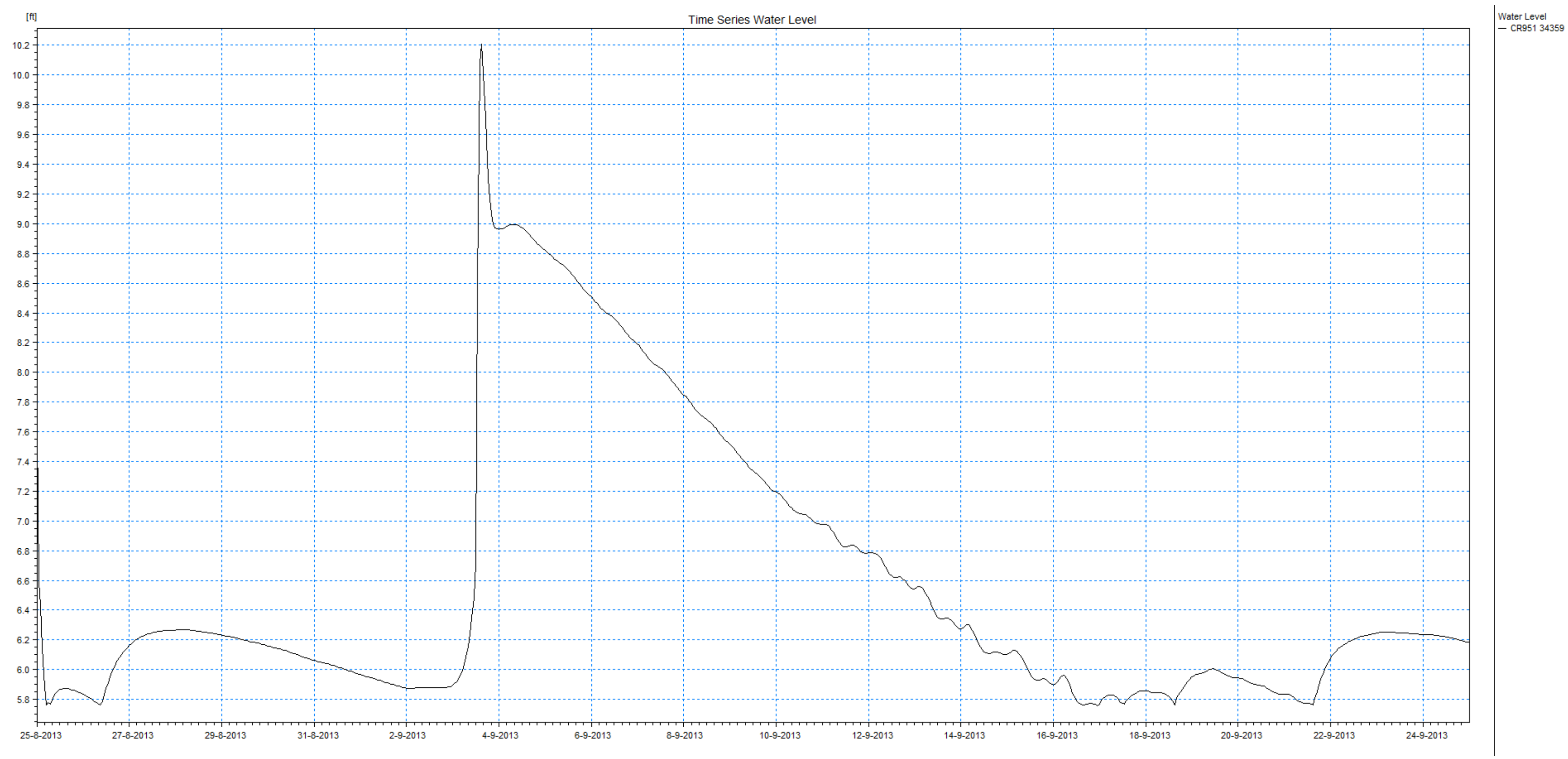

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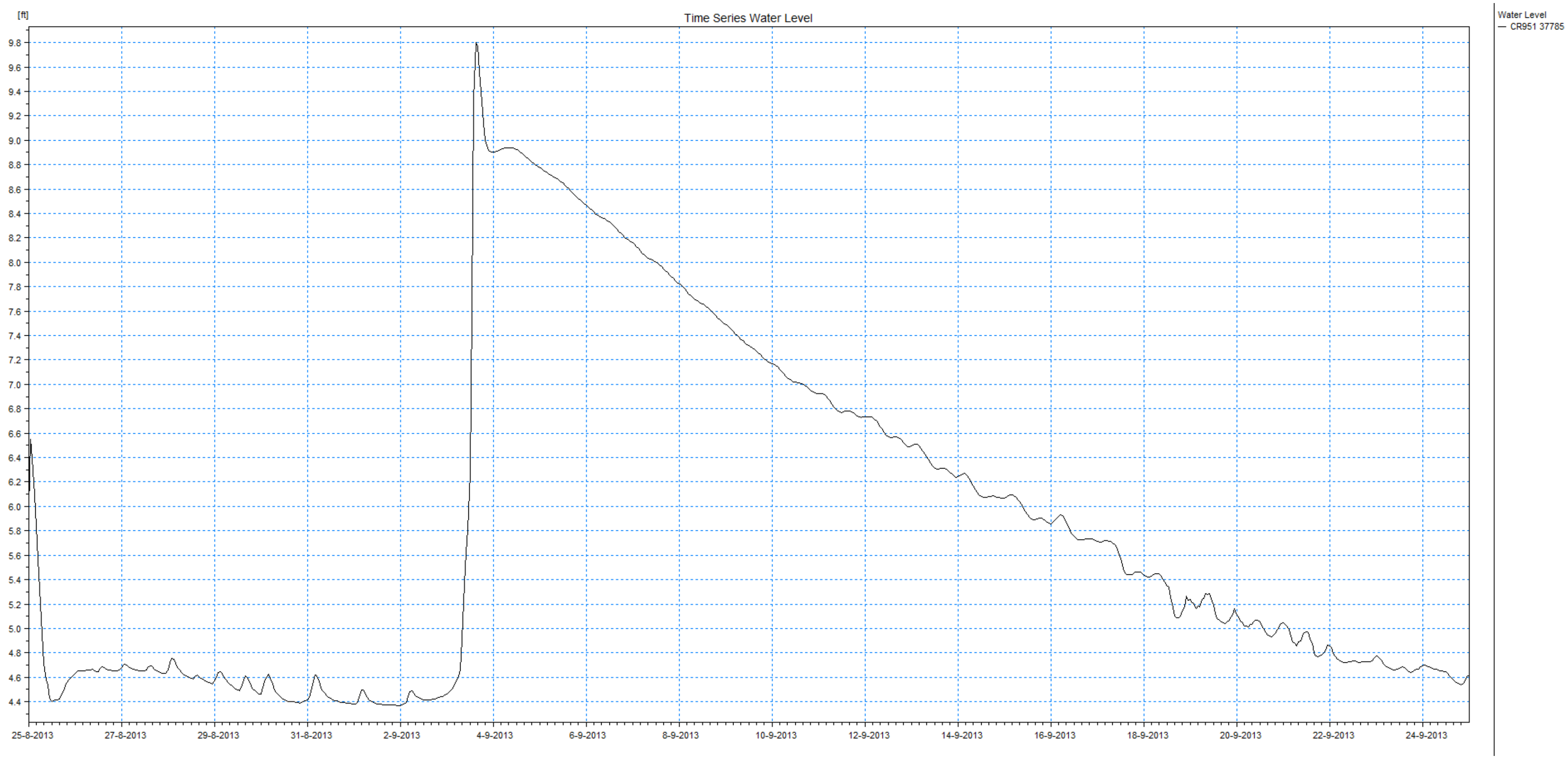


Figure 19. CR951S Tailwater Stage (ft)

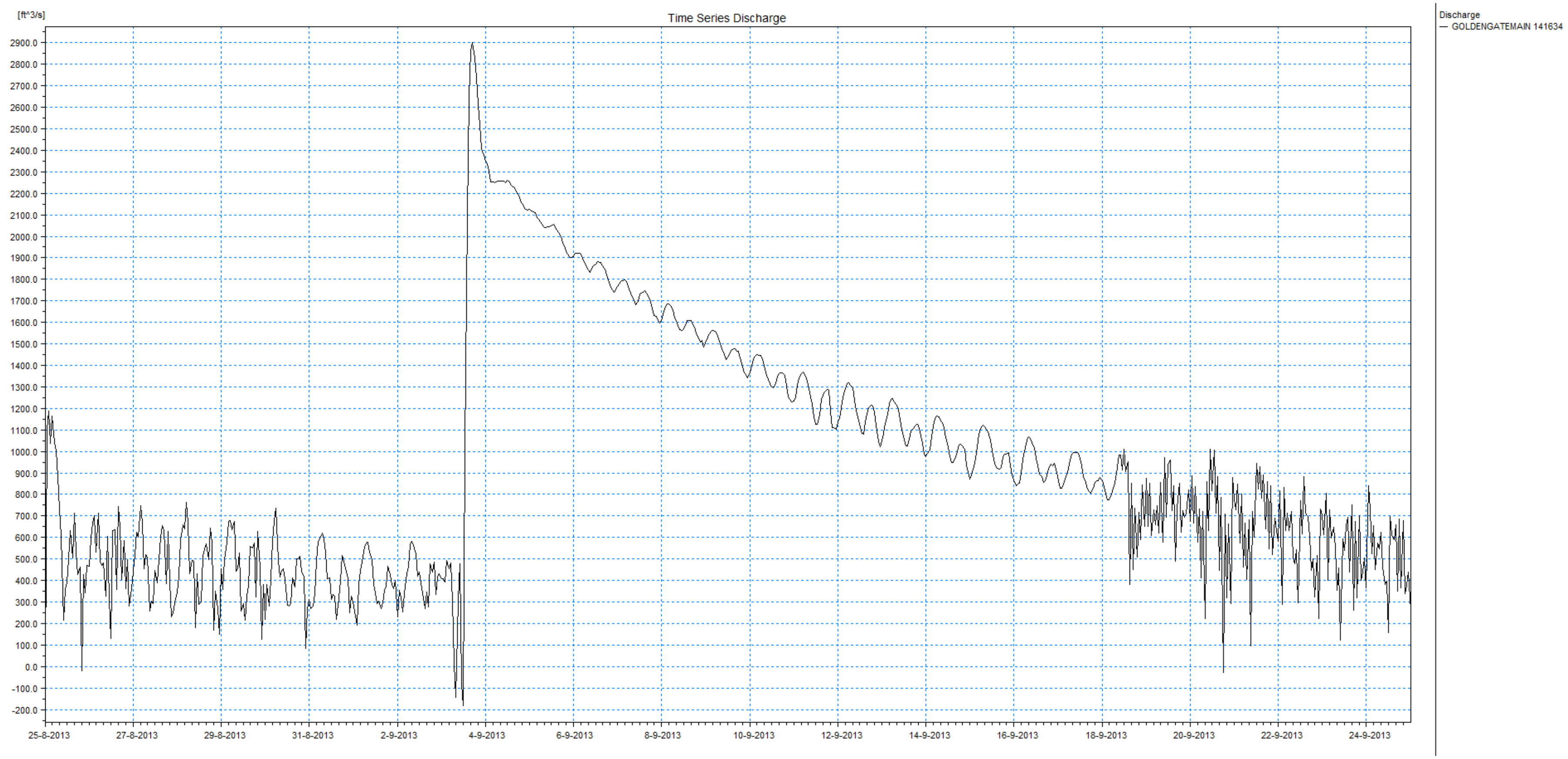


Figure 20. GG1 Discharge (cfs)

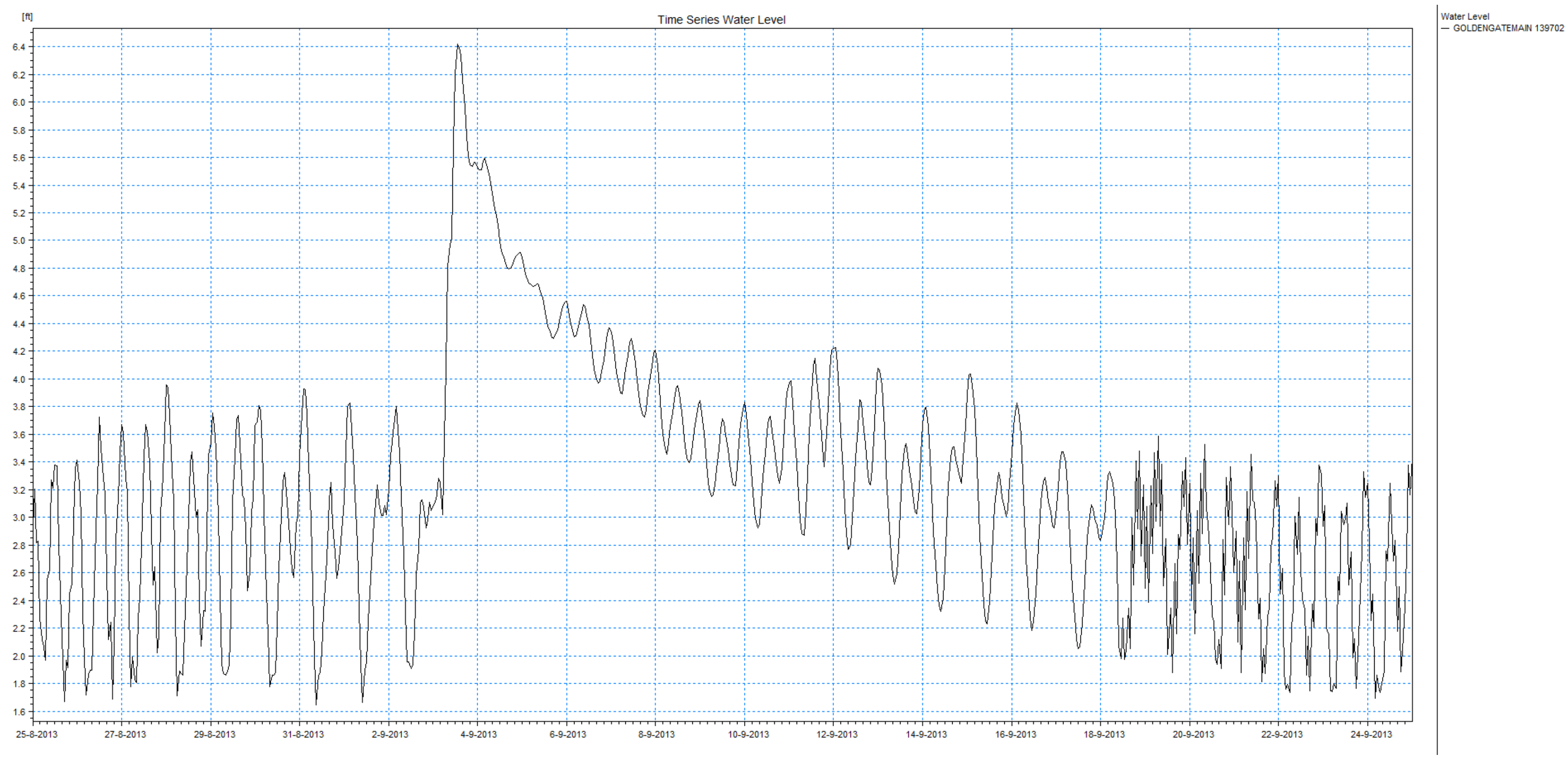
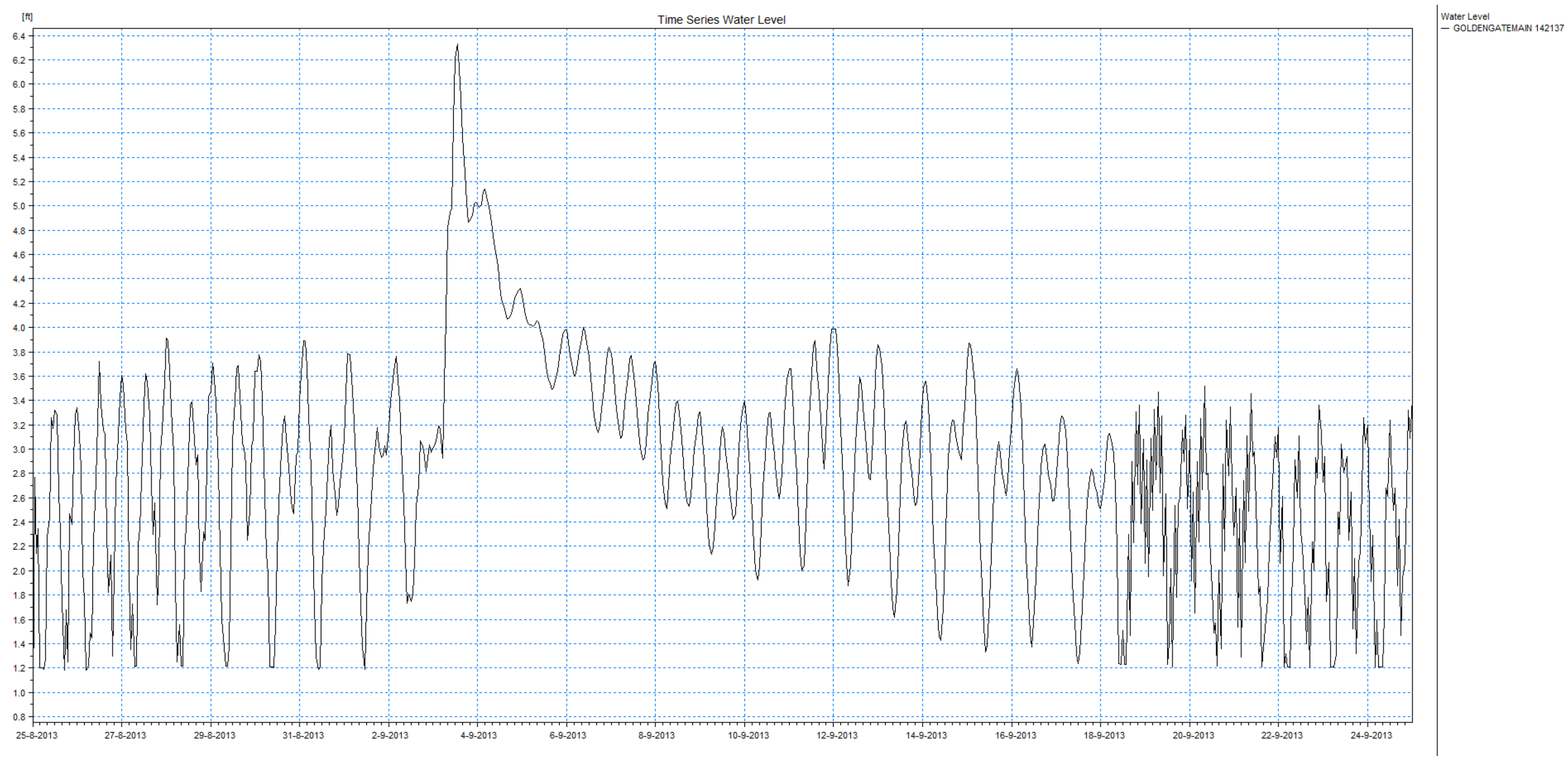


Figure 21. GG1 Headwater Stage (ft)



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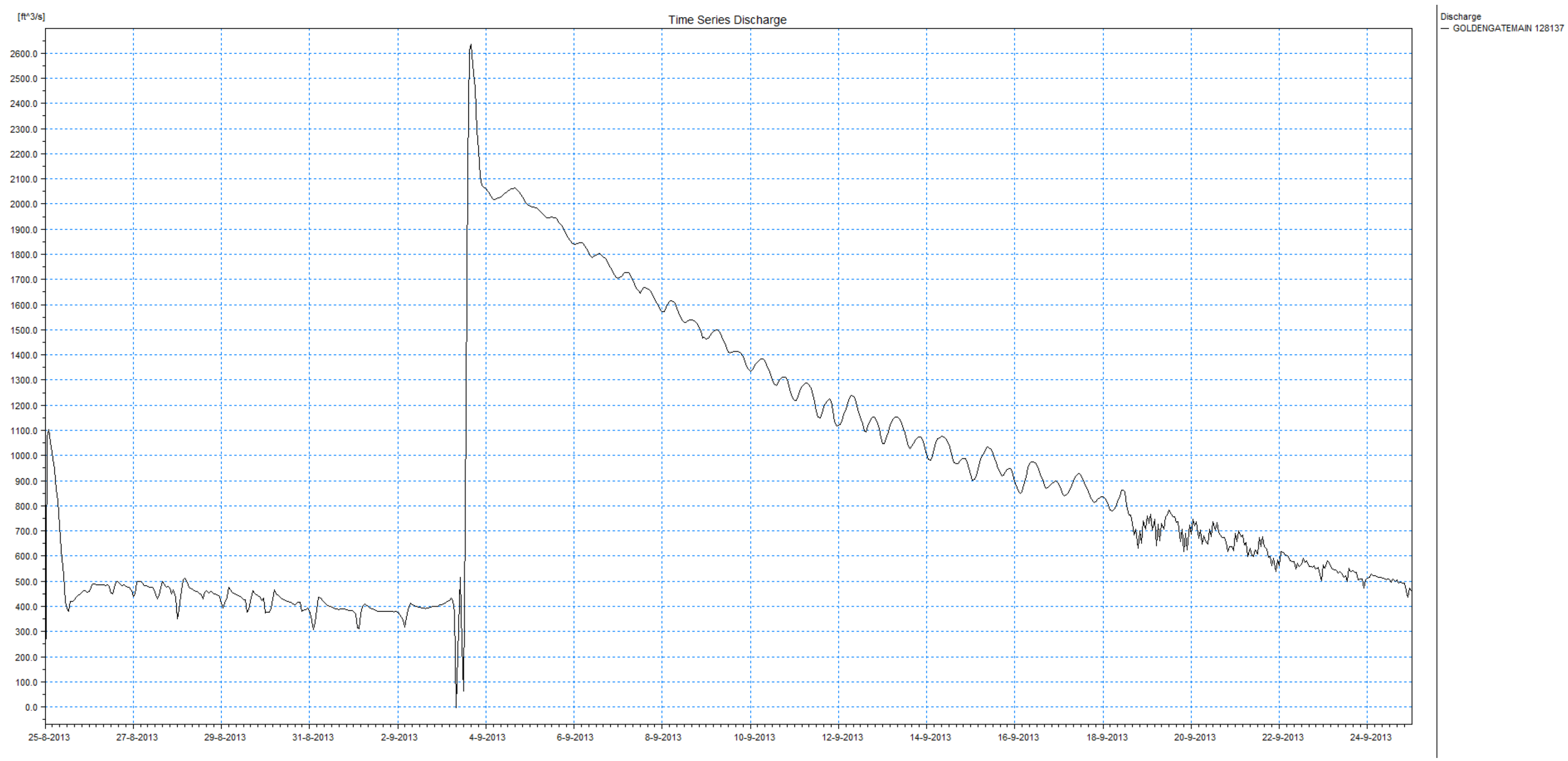

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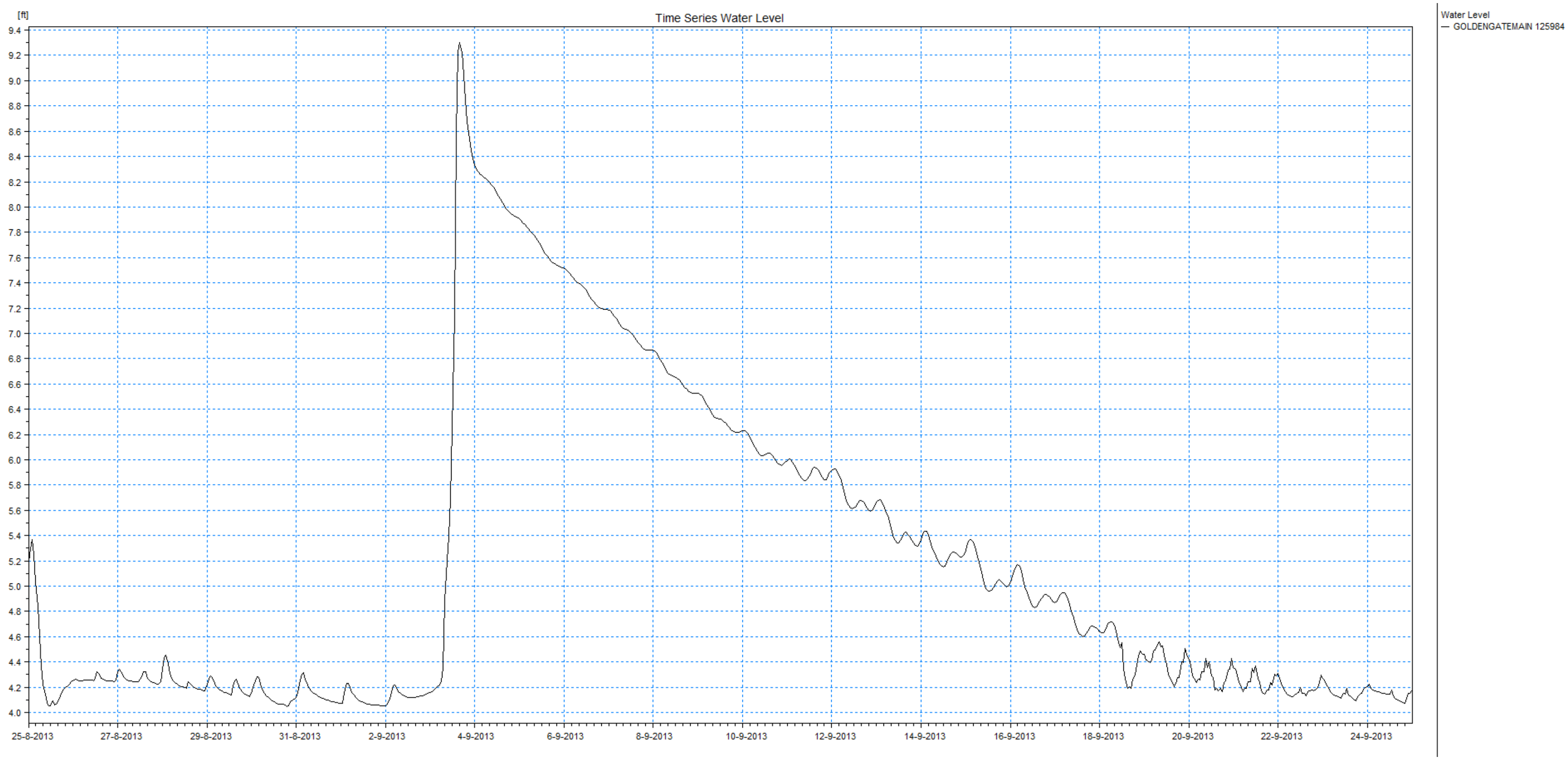

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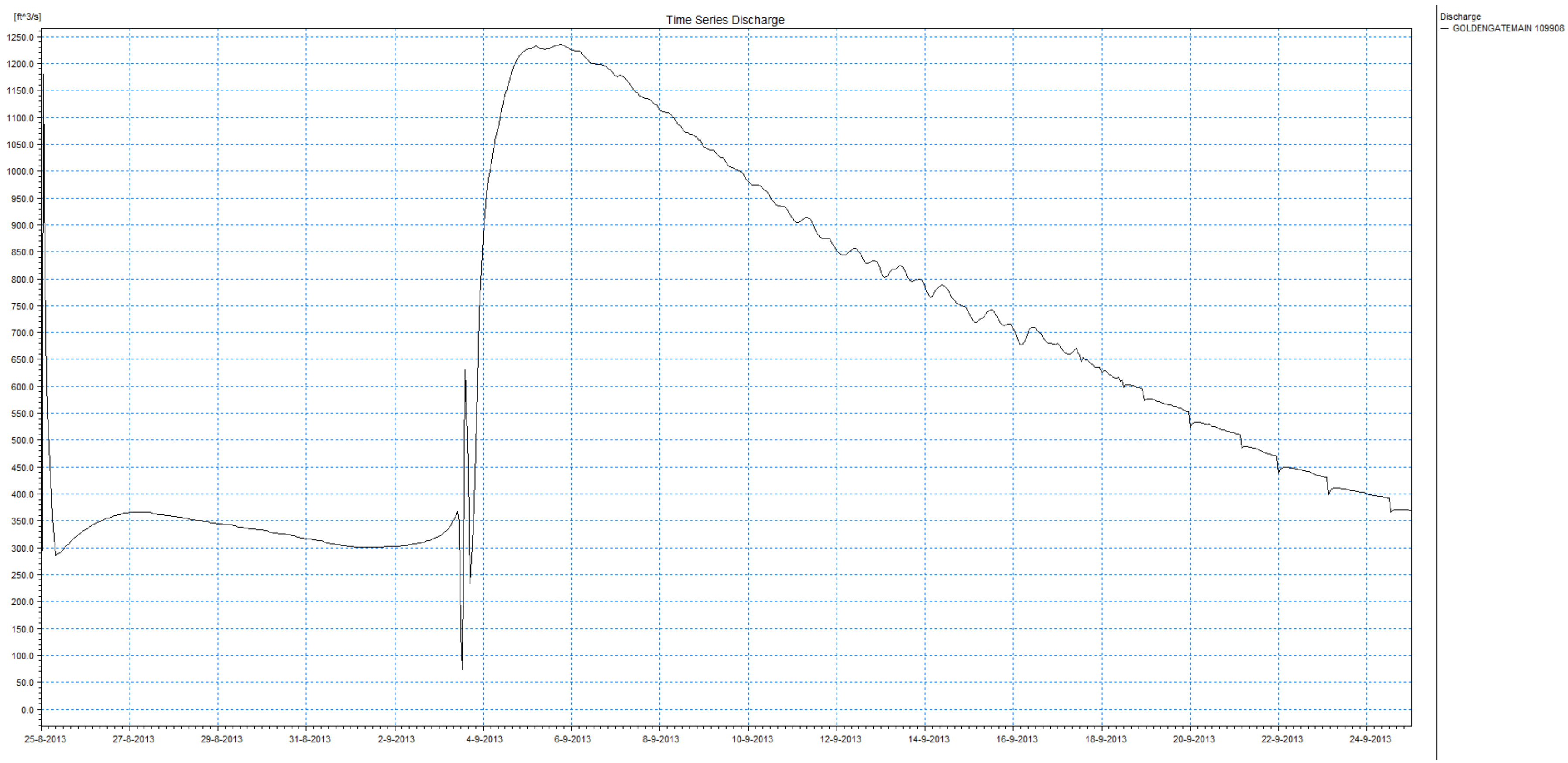

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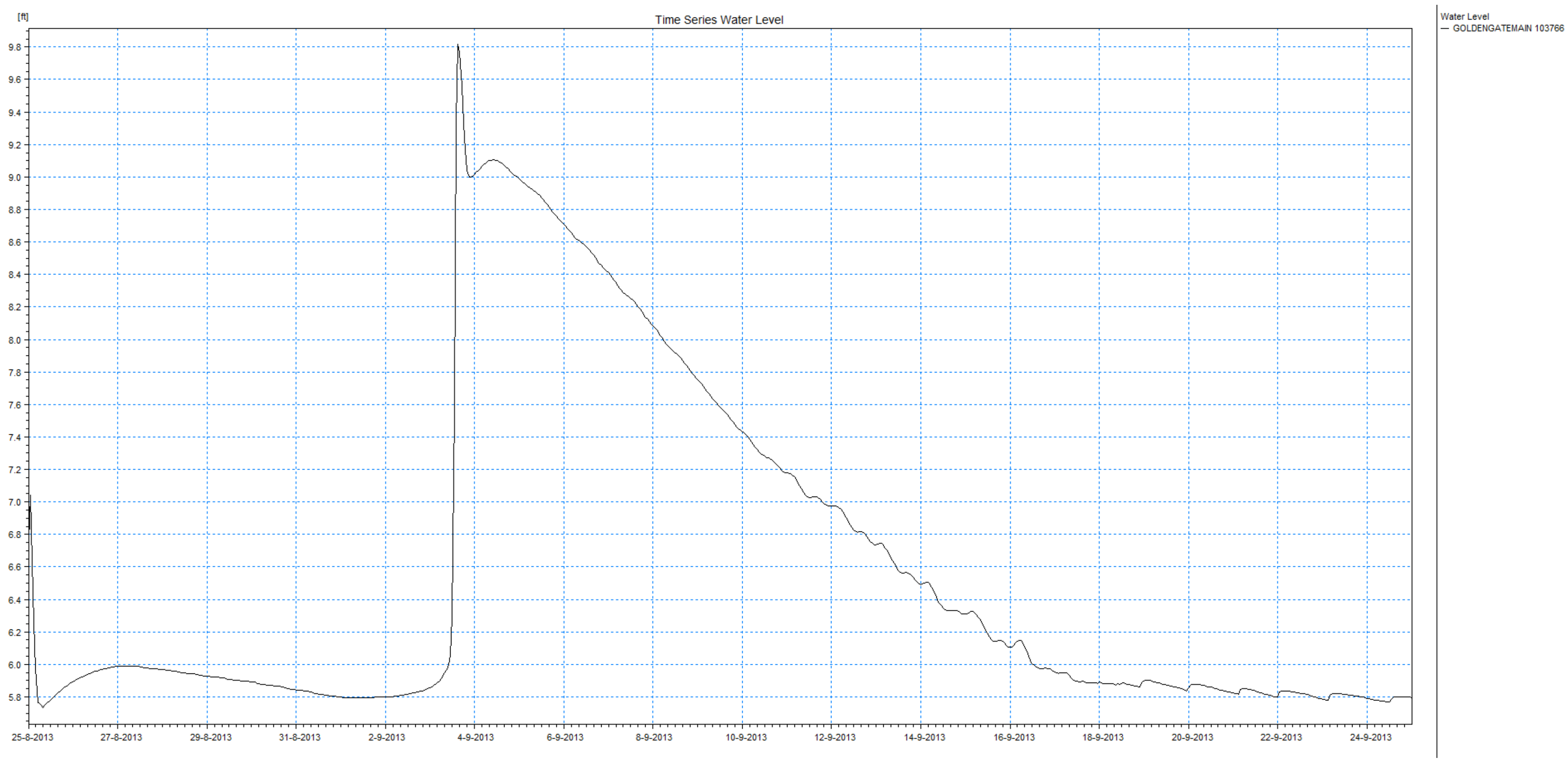
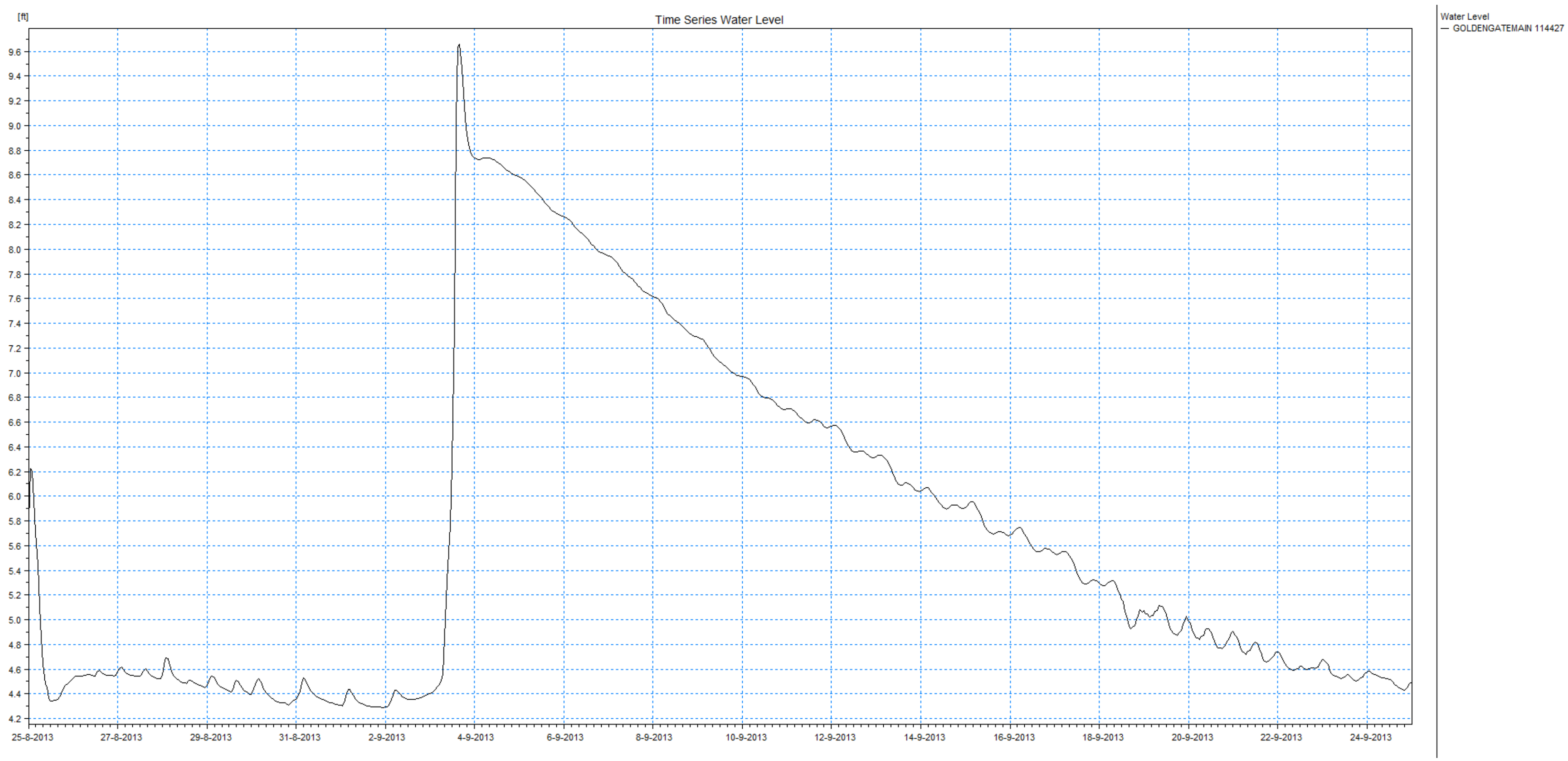


Figure 27. GG3 Headwater Stage (ft)



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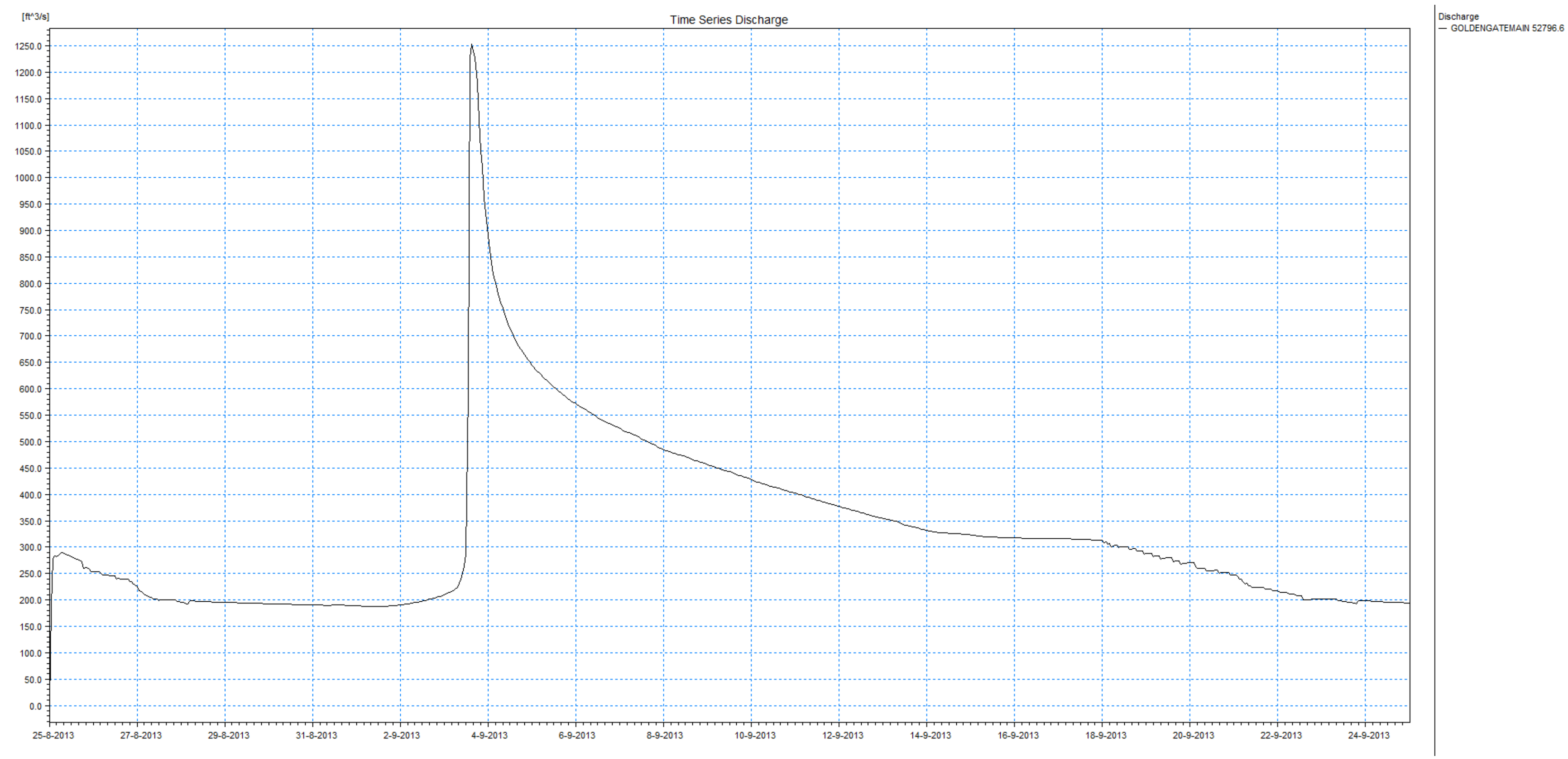

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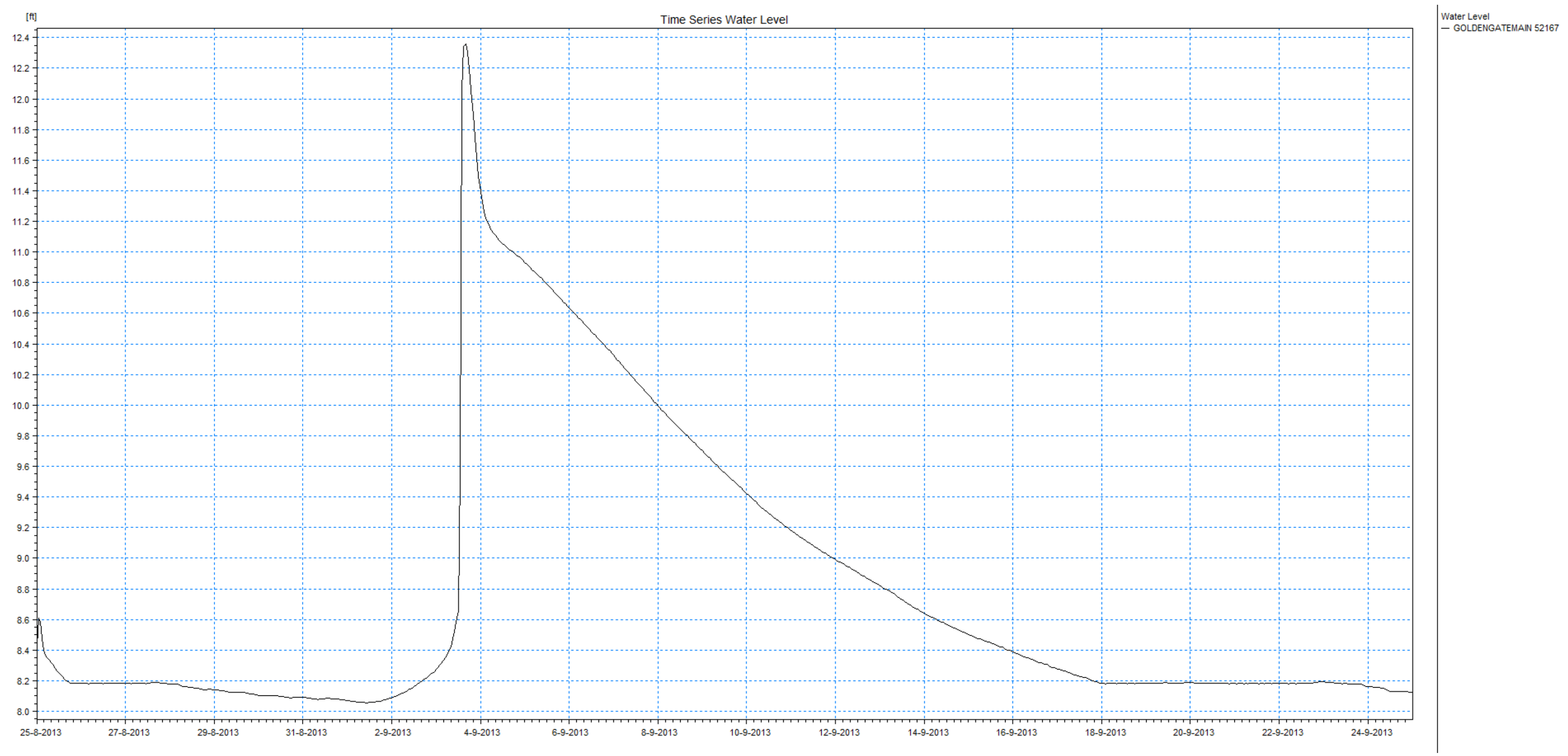

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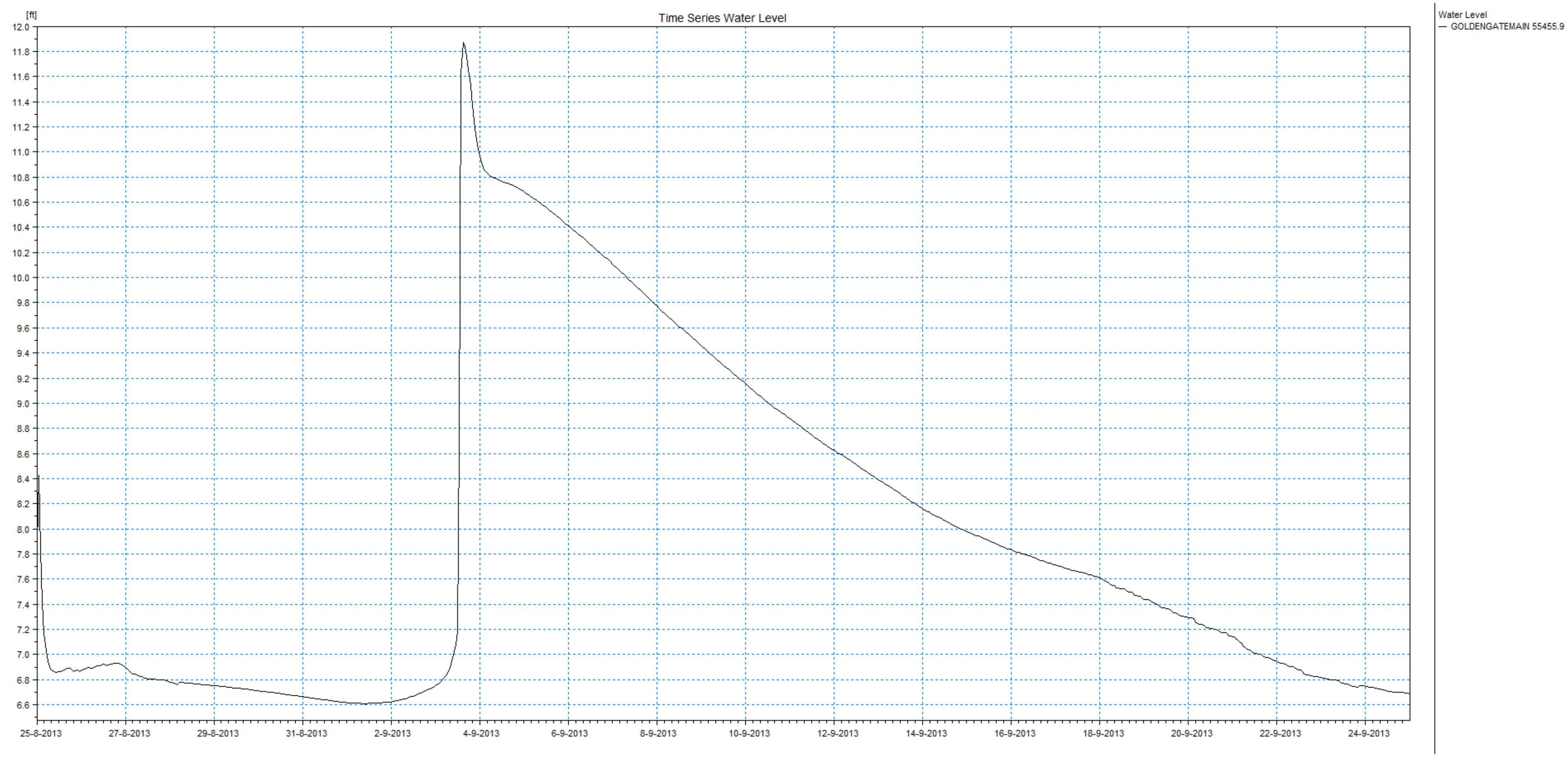

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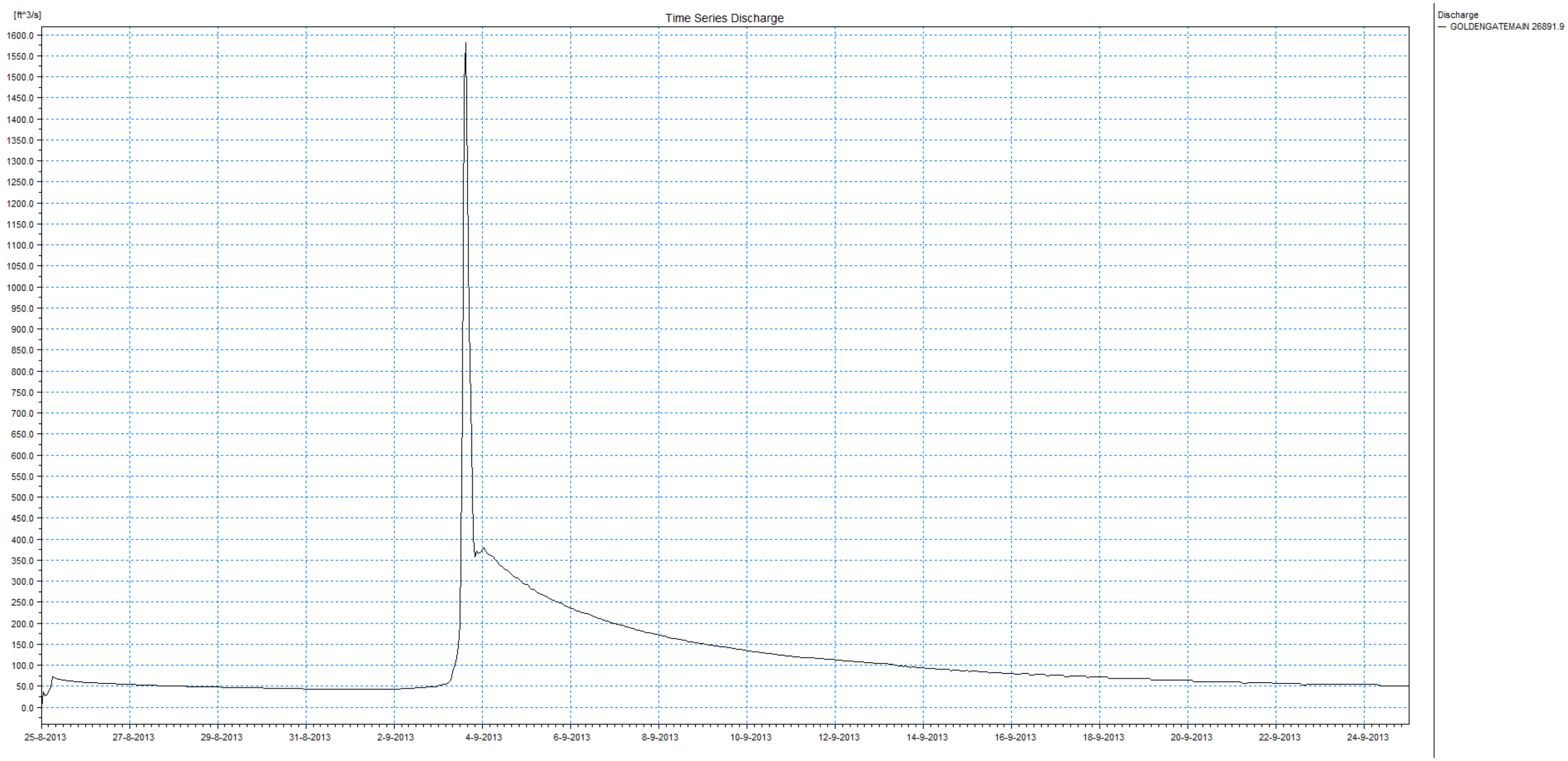
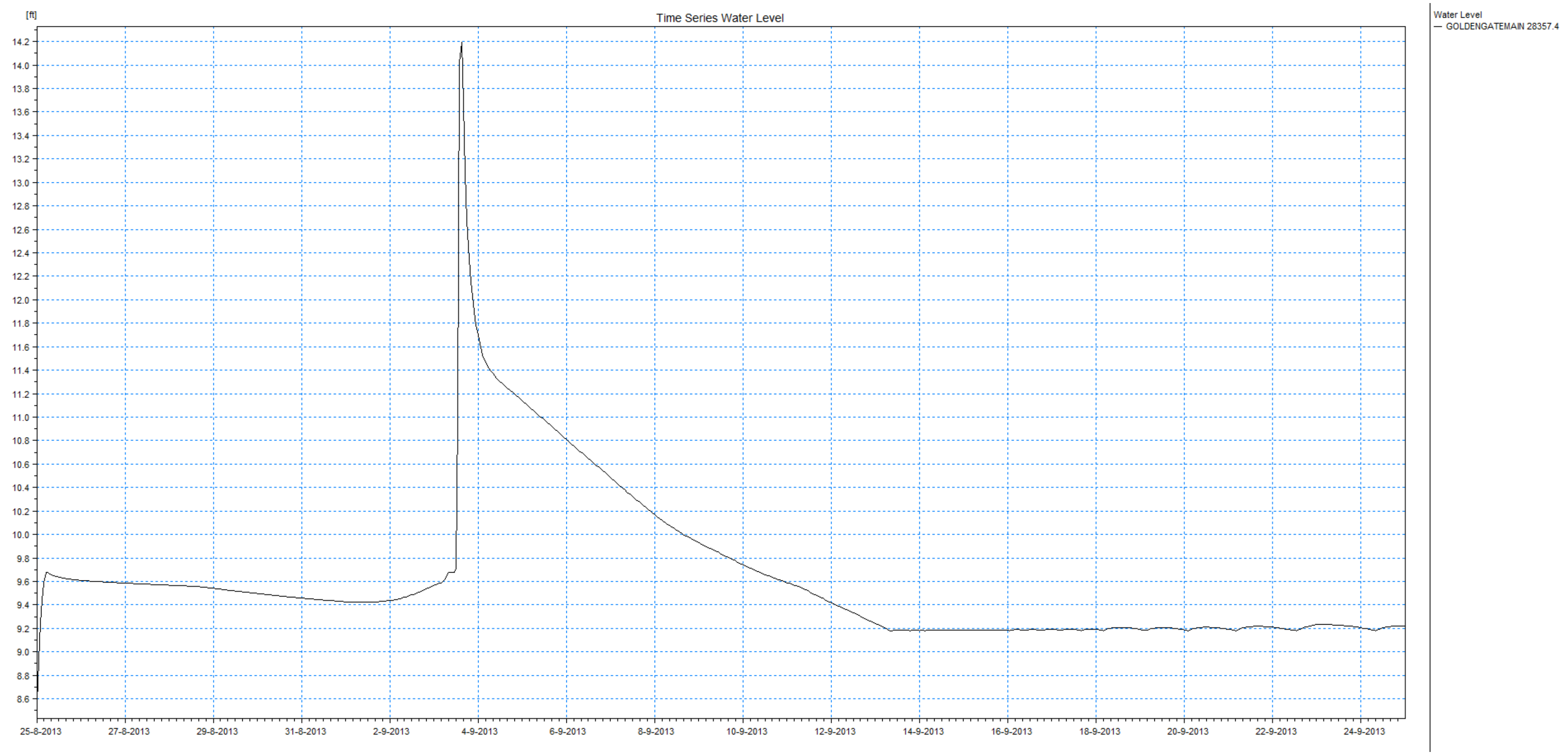


Figure 32. GG5 Discharge (cfs)



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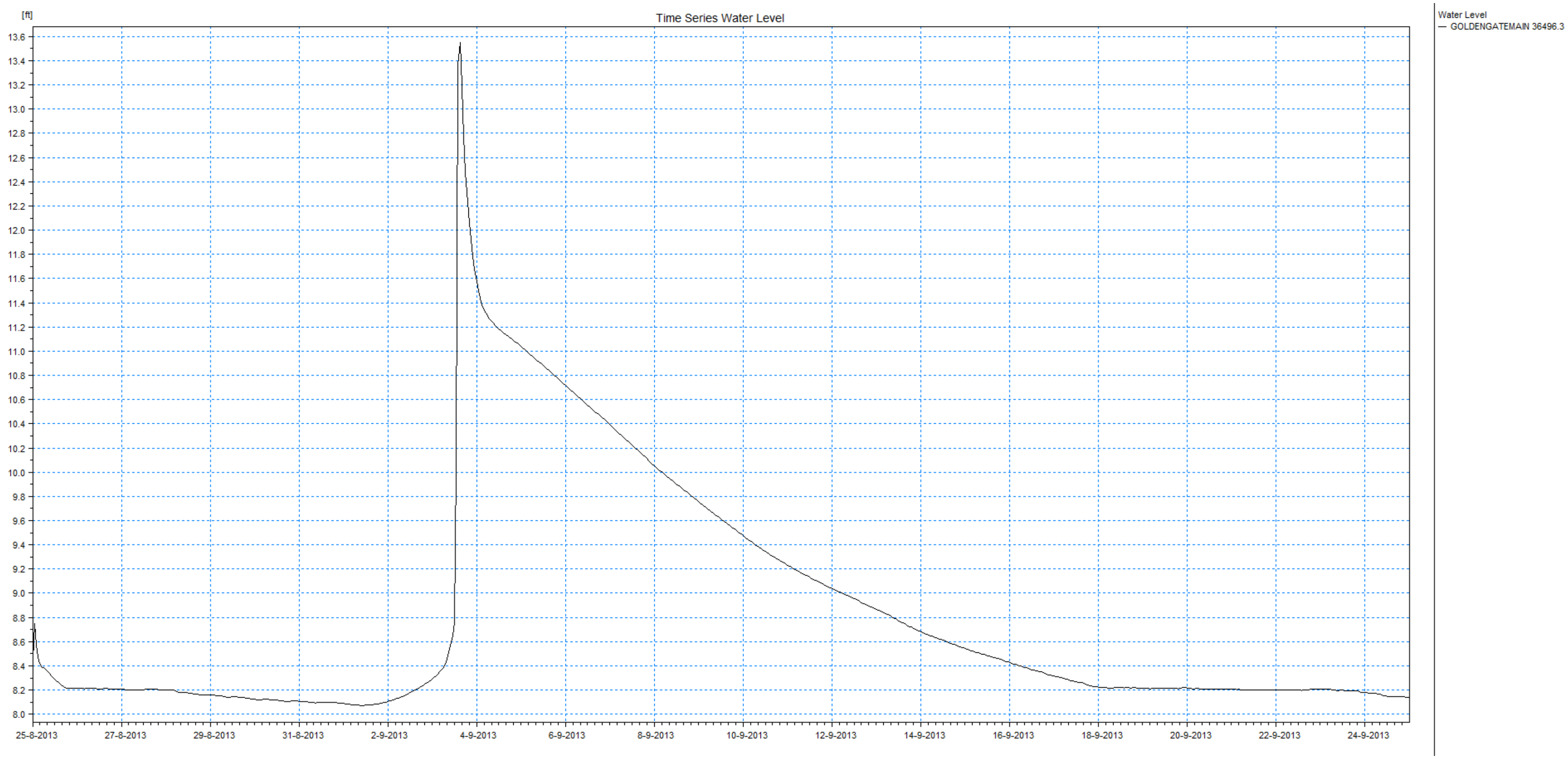

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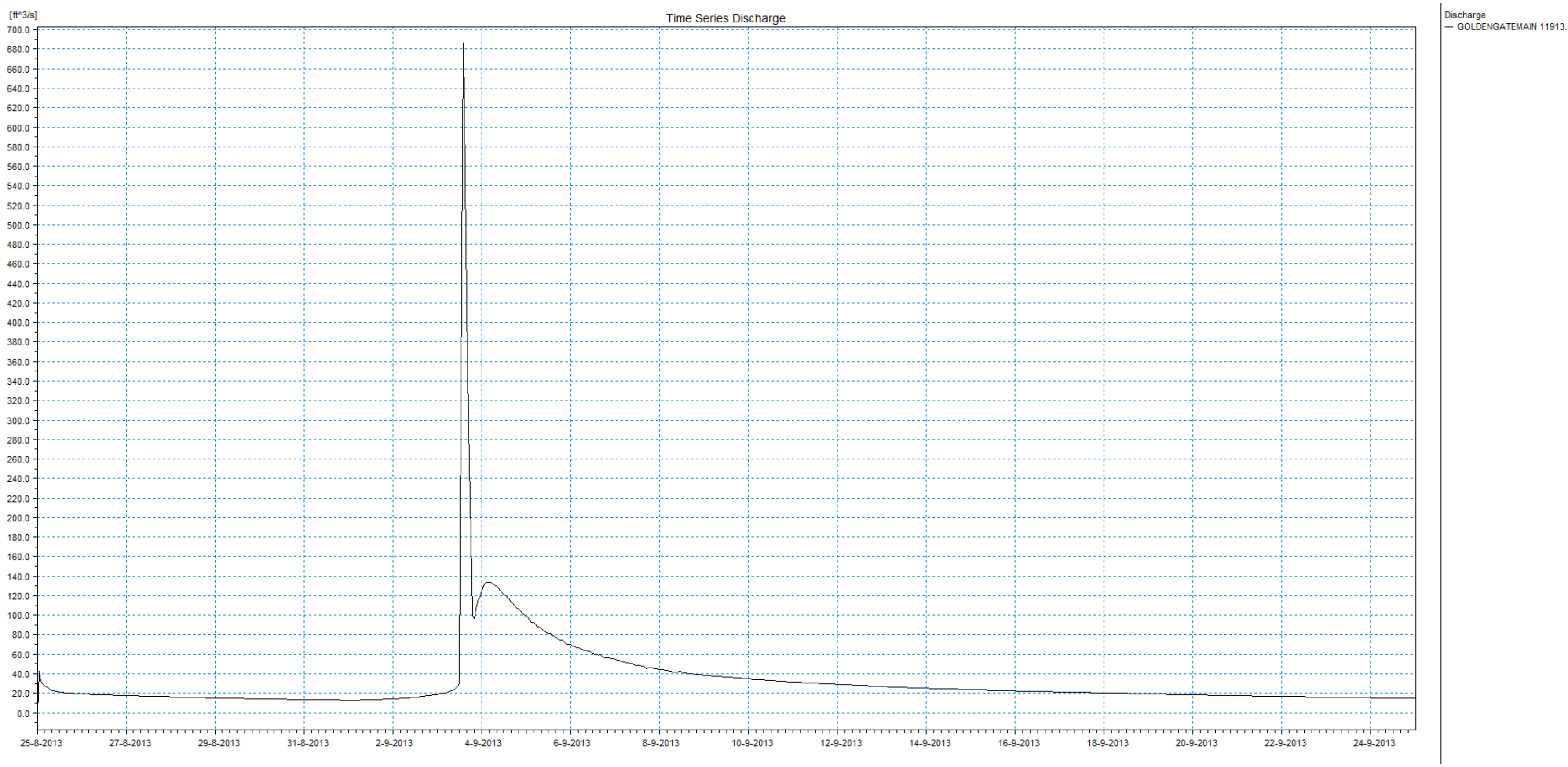


Figure 35. GG6 Discharge (cfs)

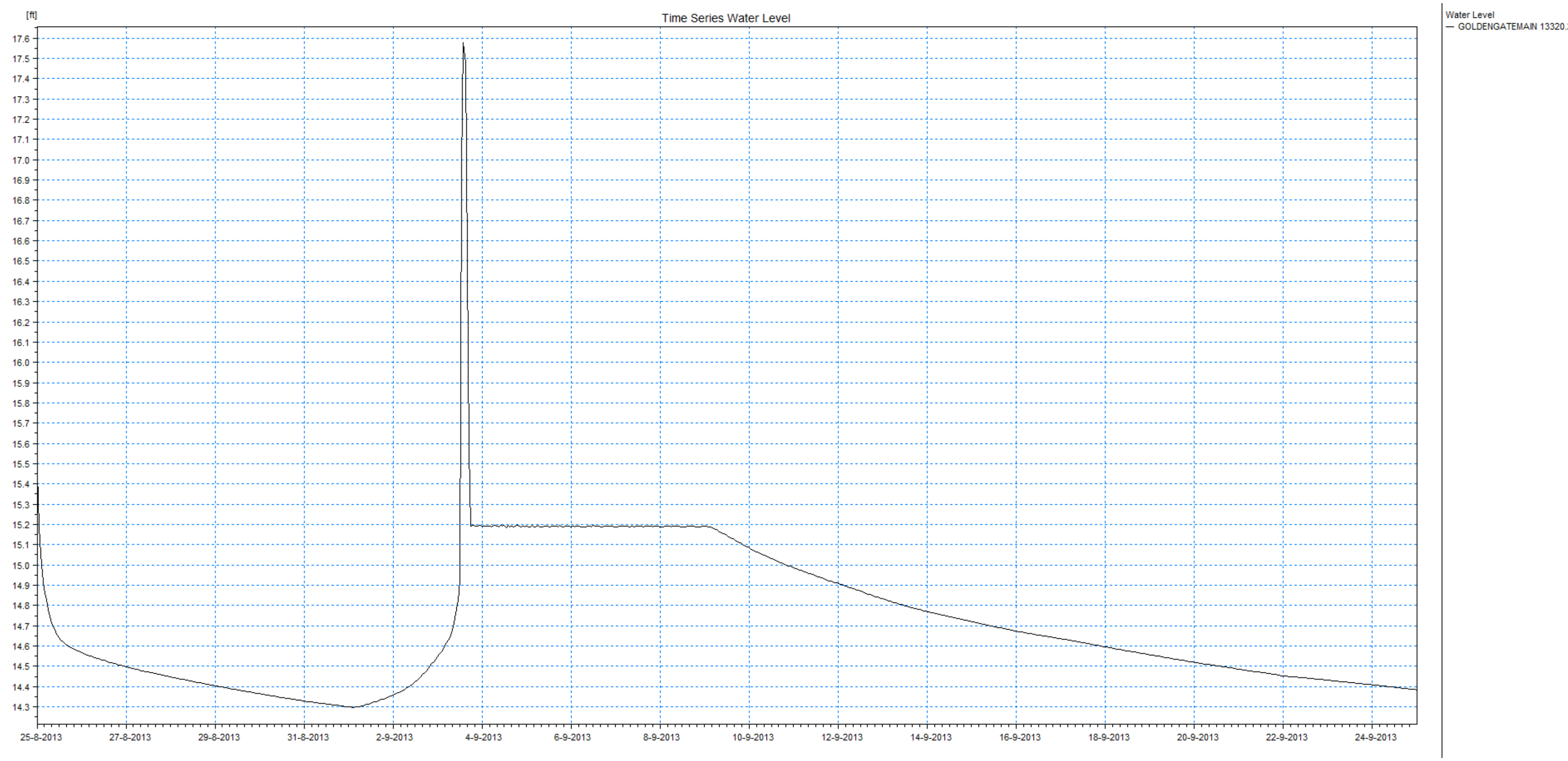
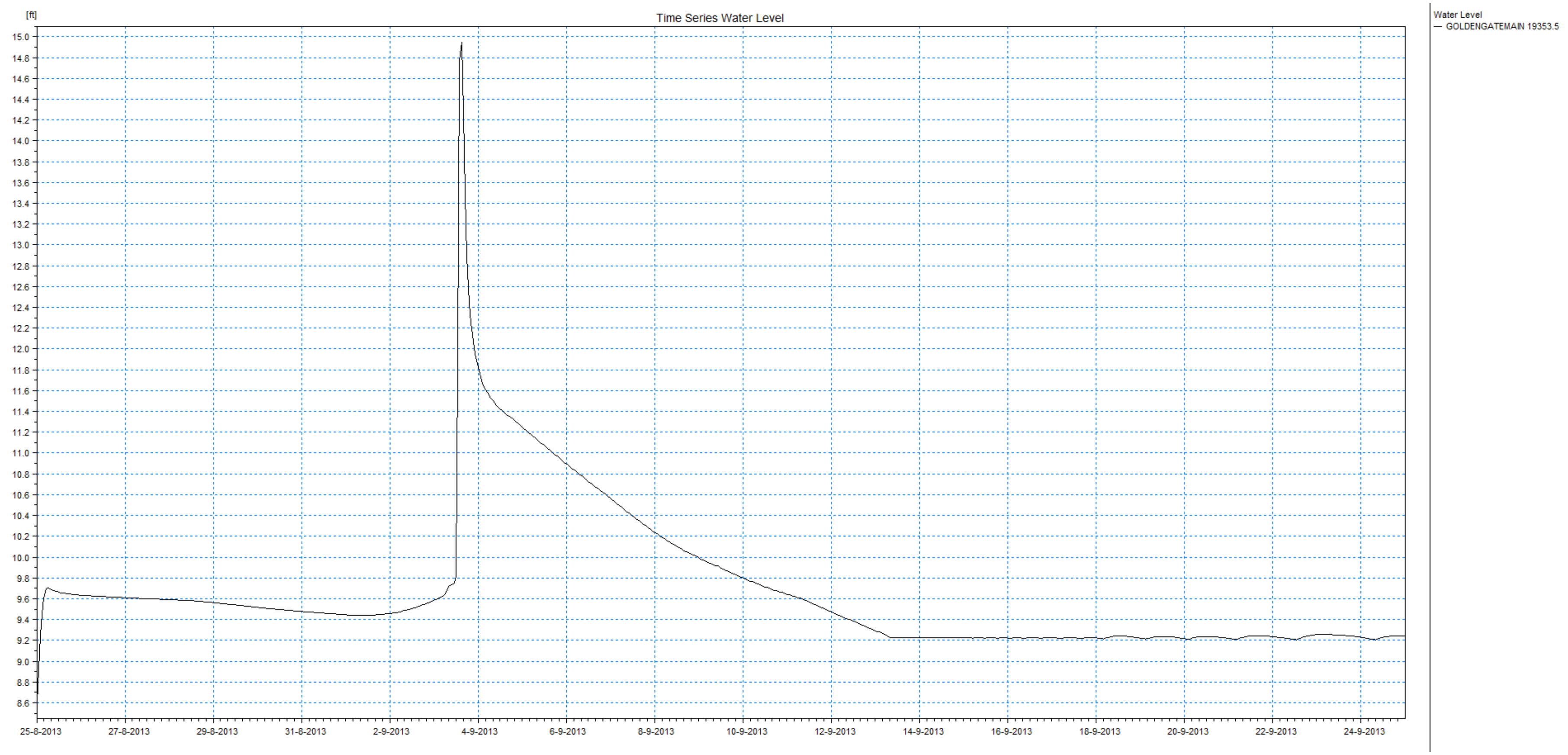


Figure 36. GG6 Headwater Stage (ft)



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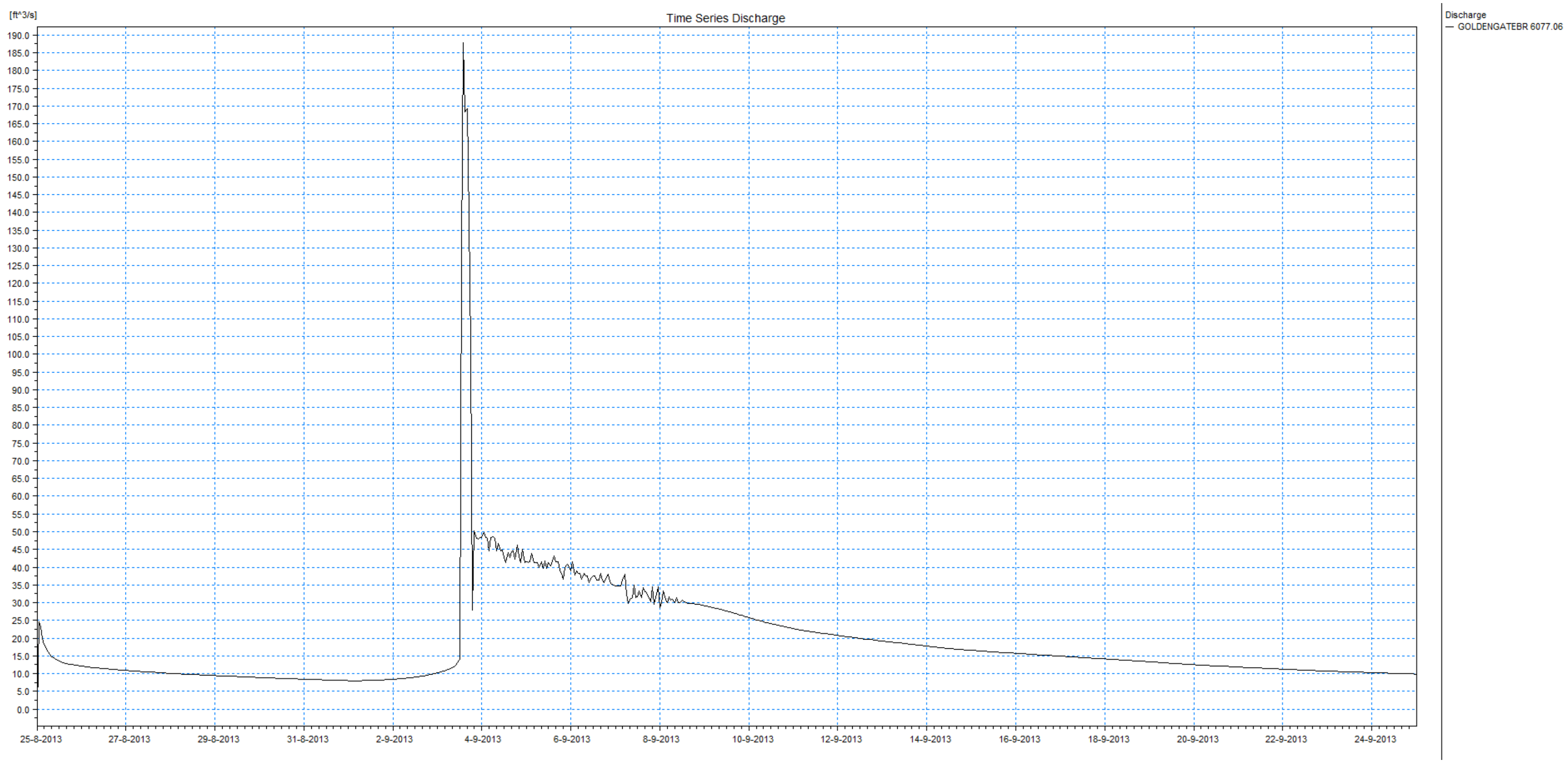

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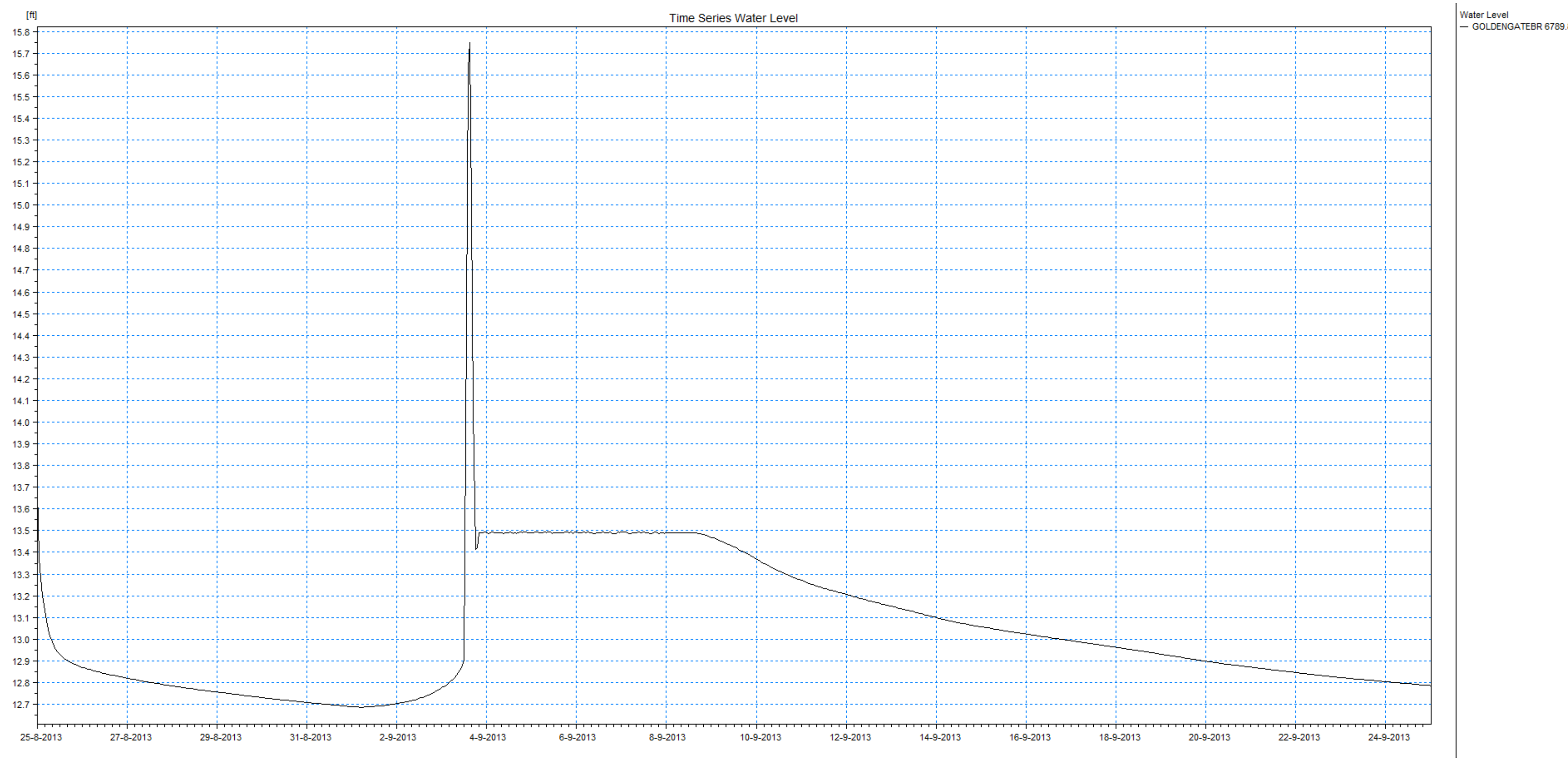
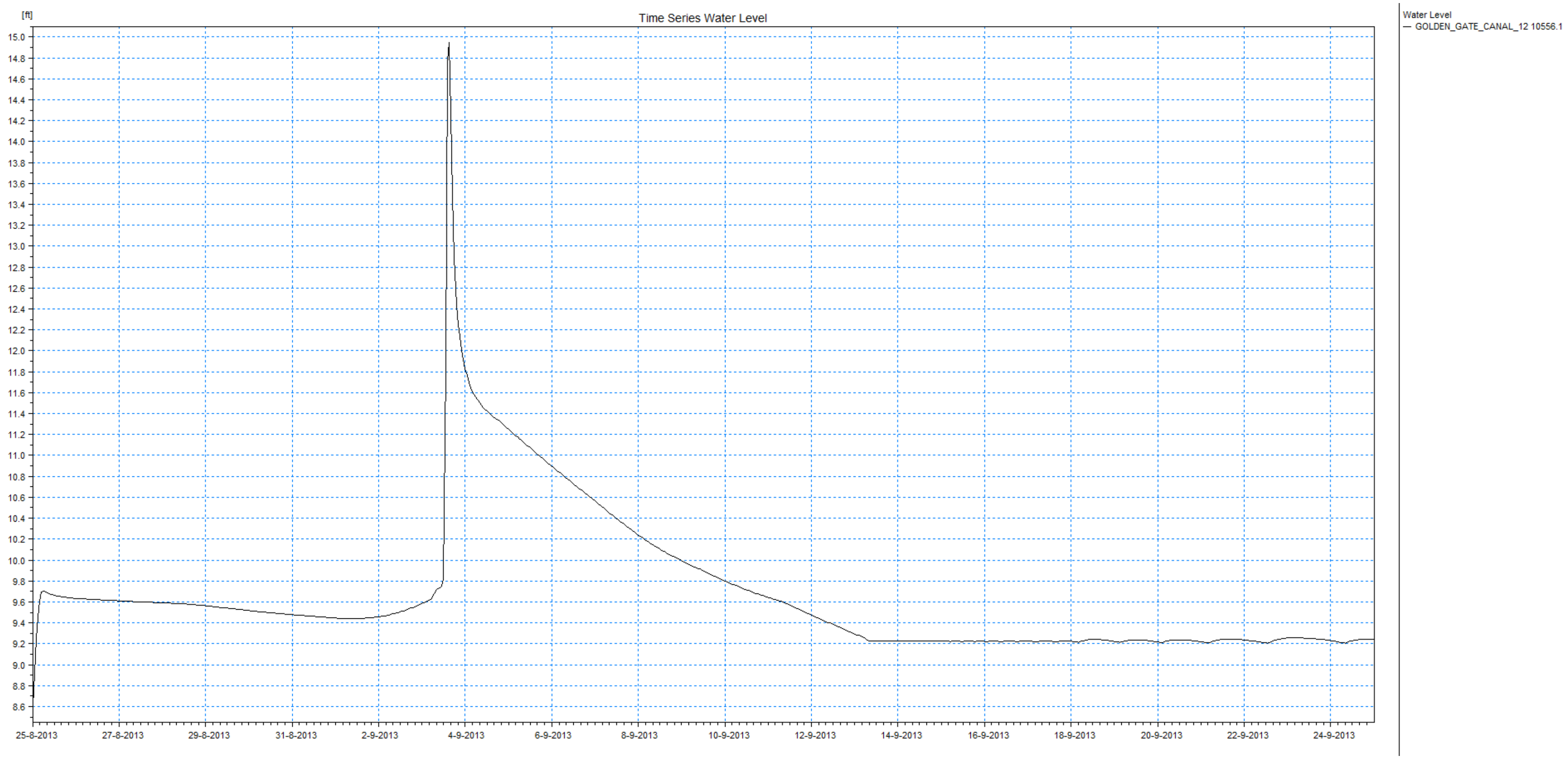


Figure 39. GG7 Headwater Stage (ft)



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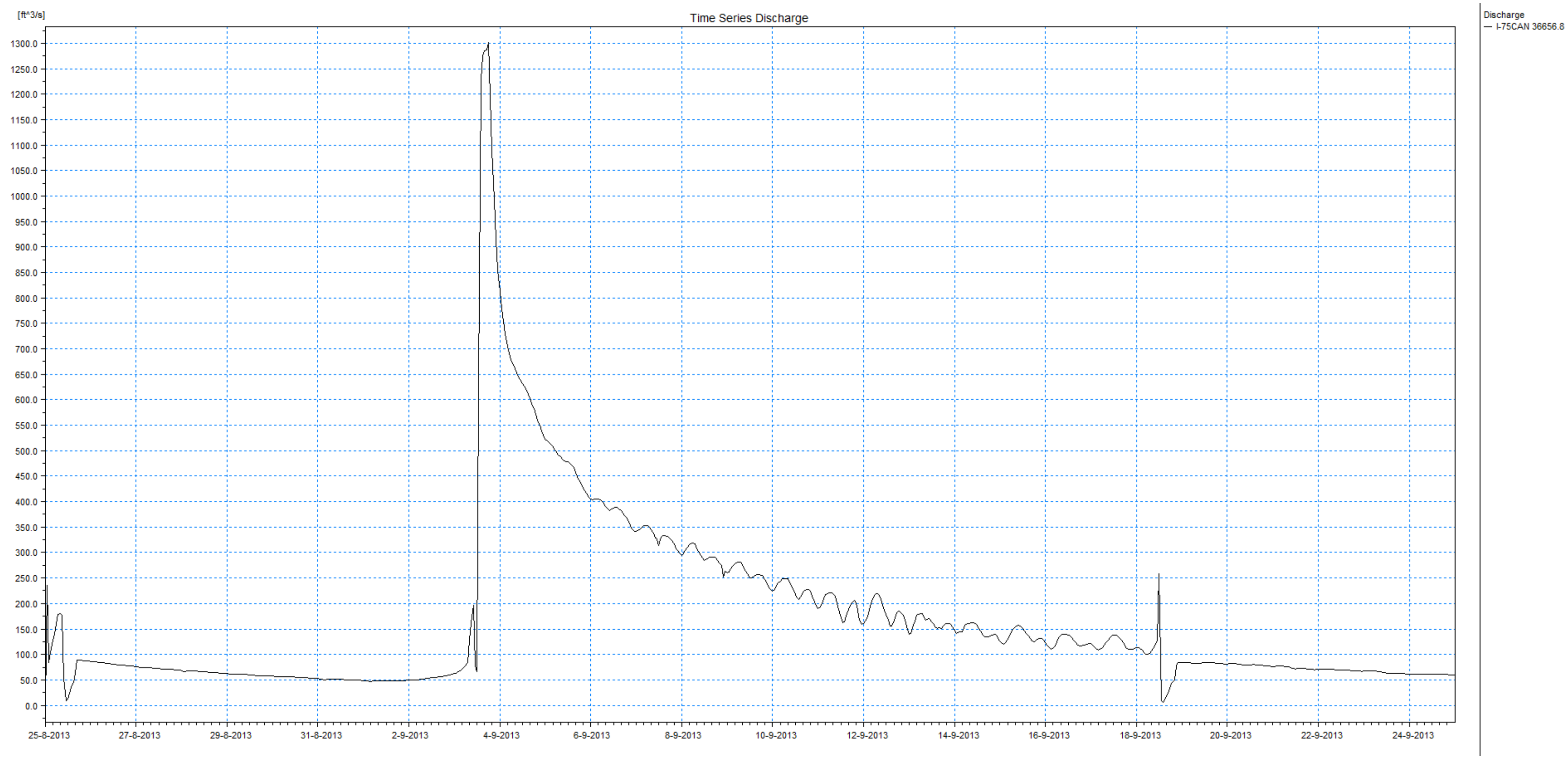

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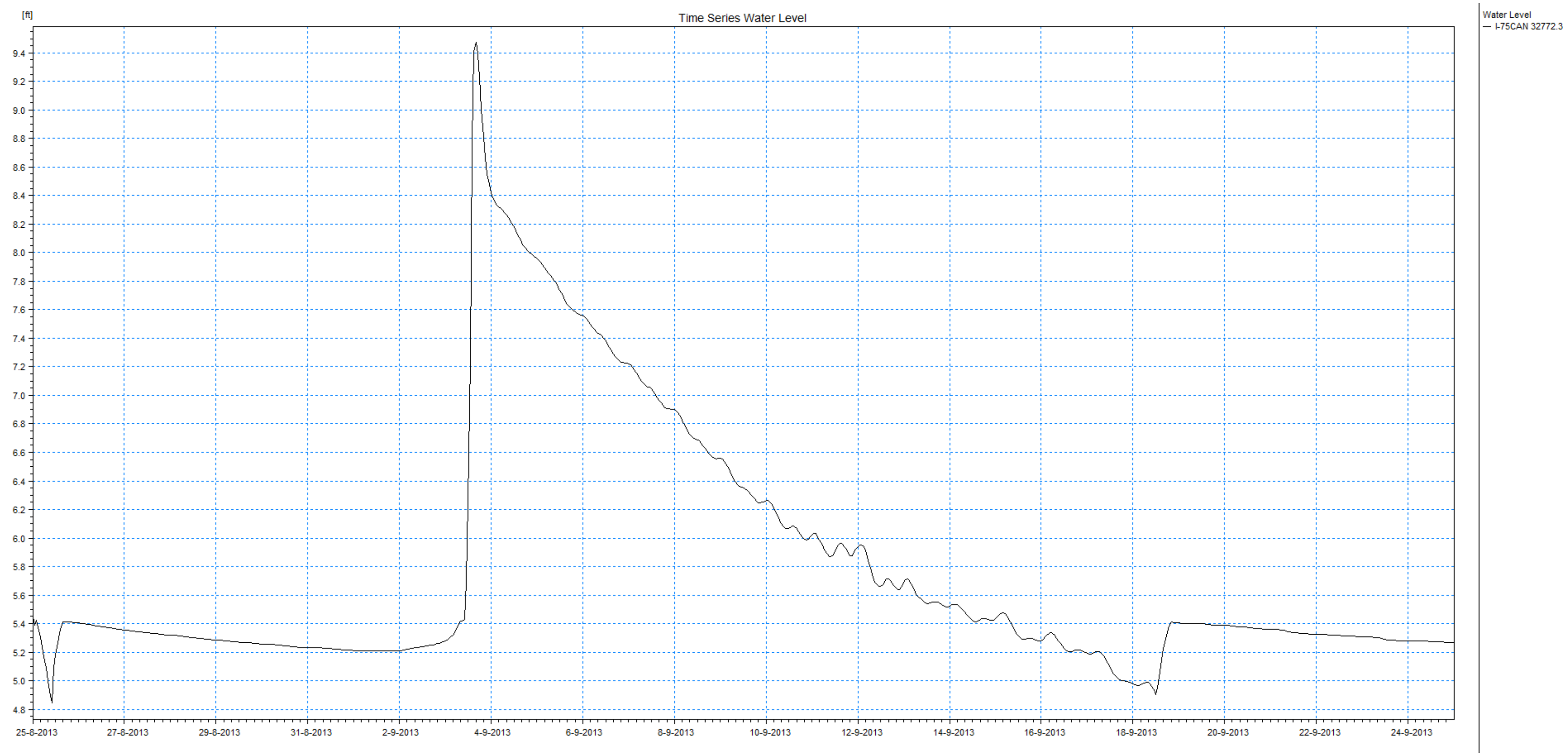

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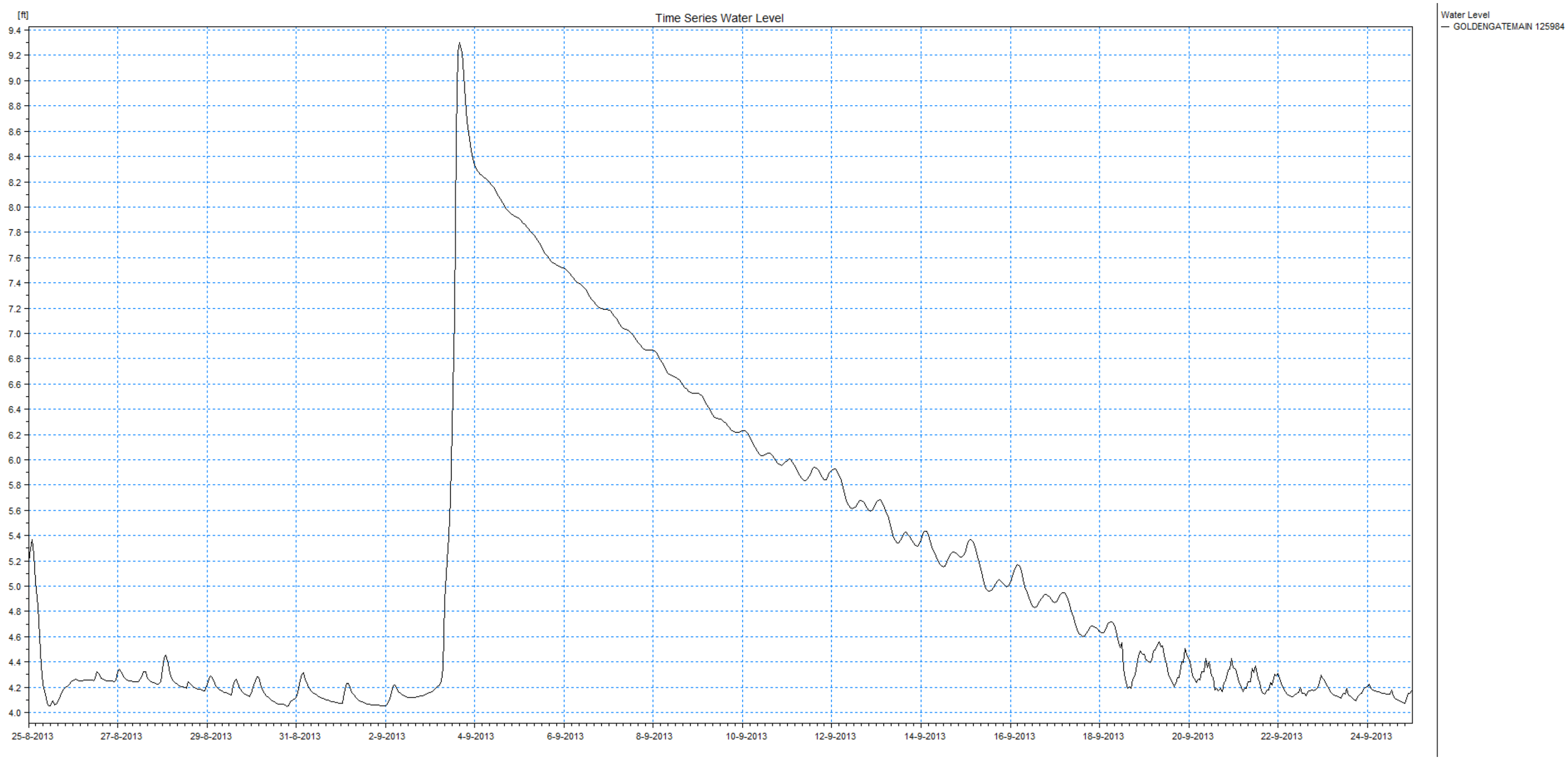

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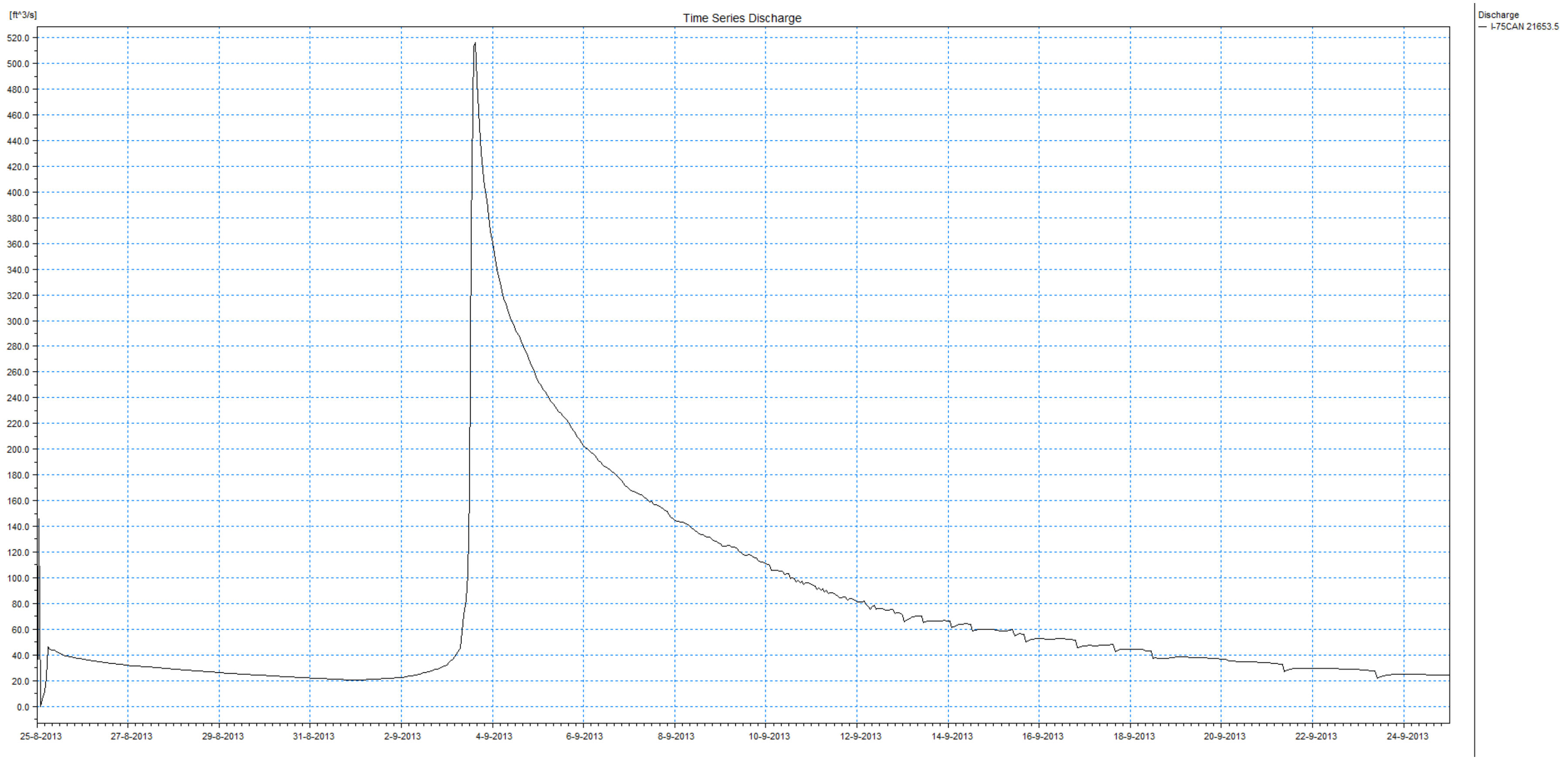

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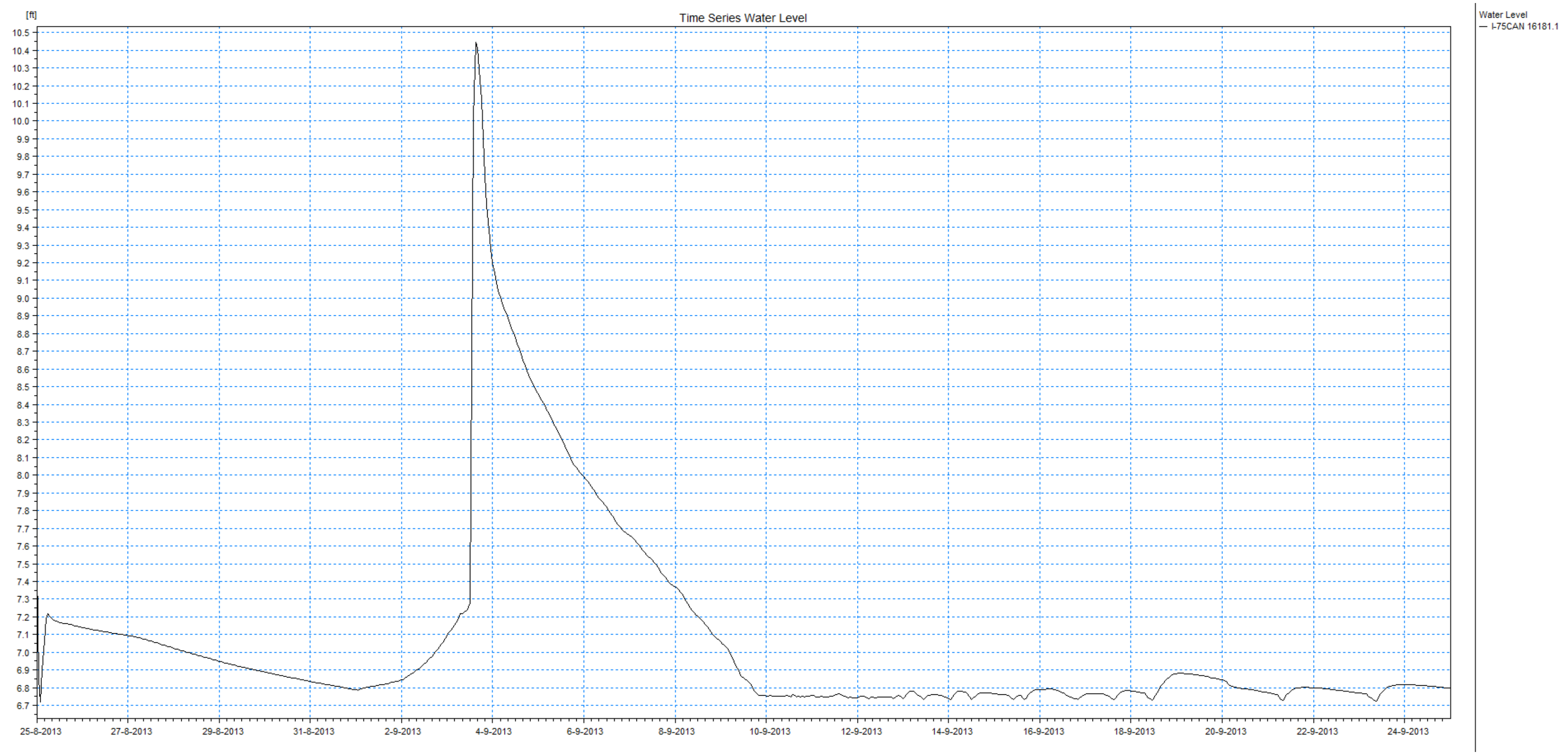

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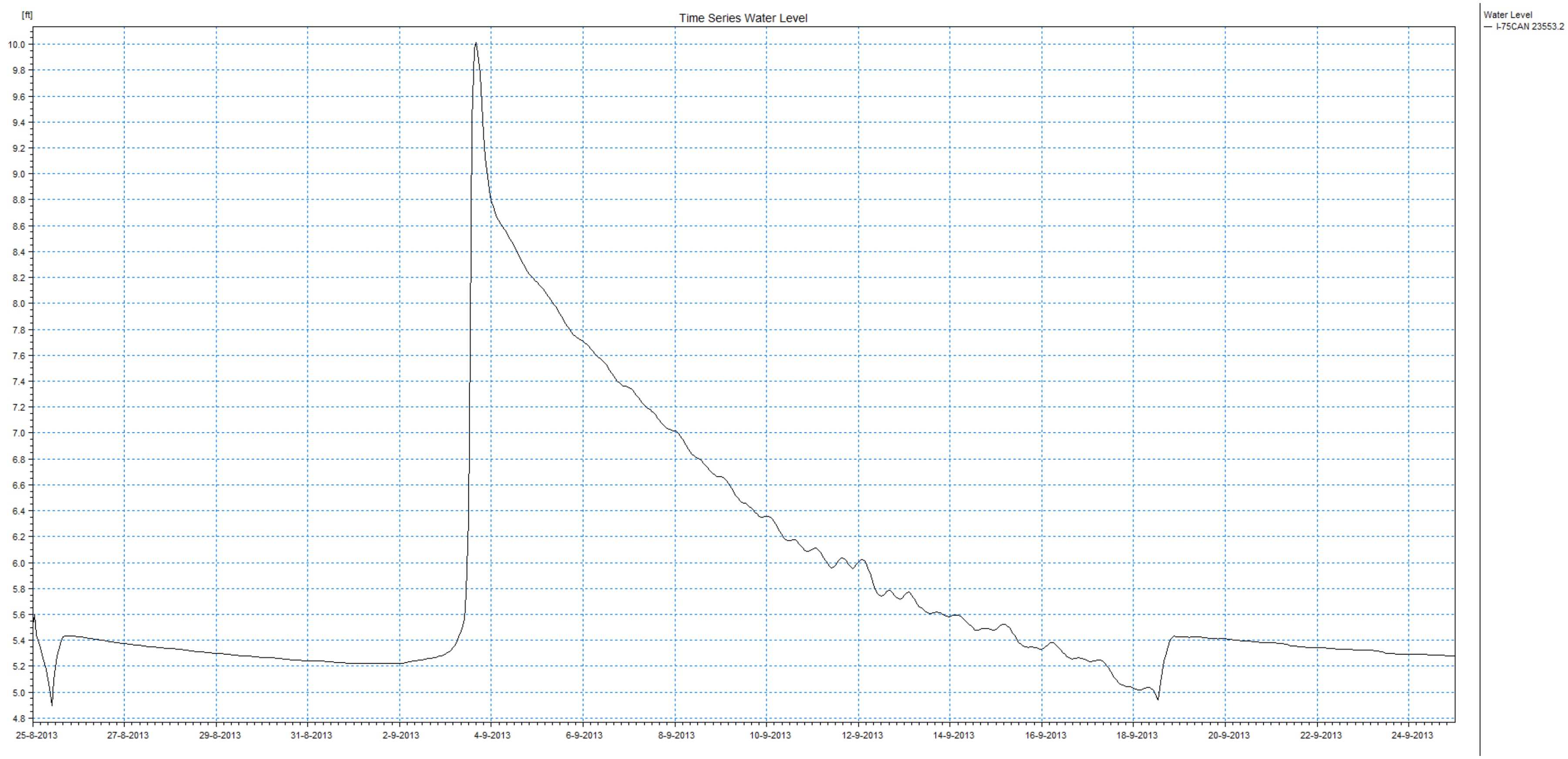

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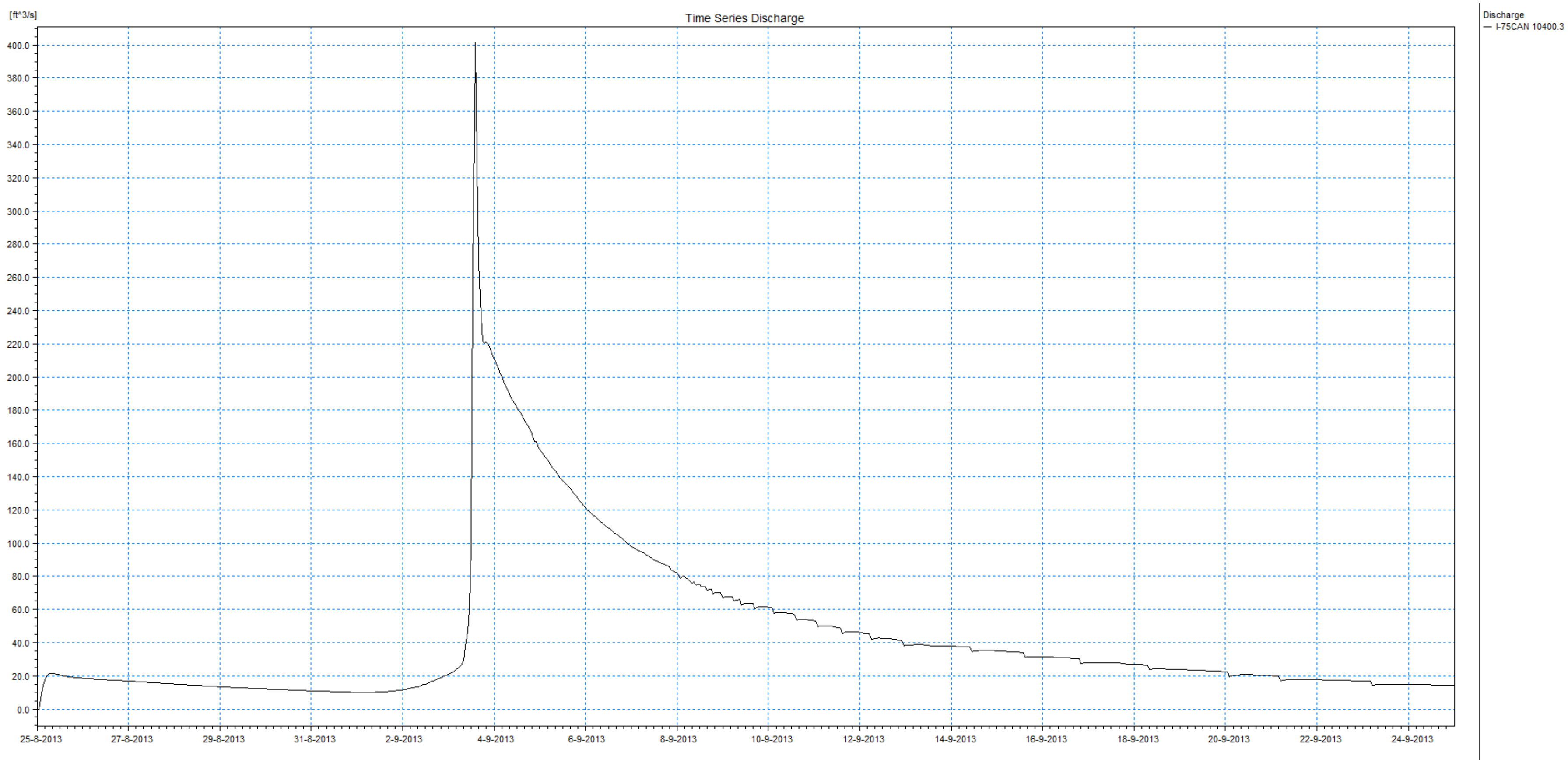

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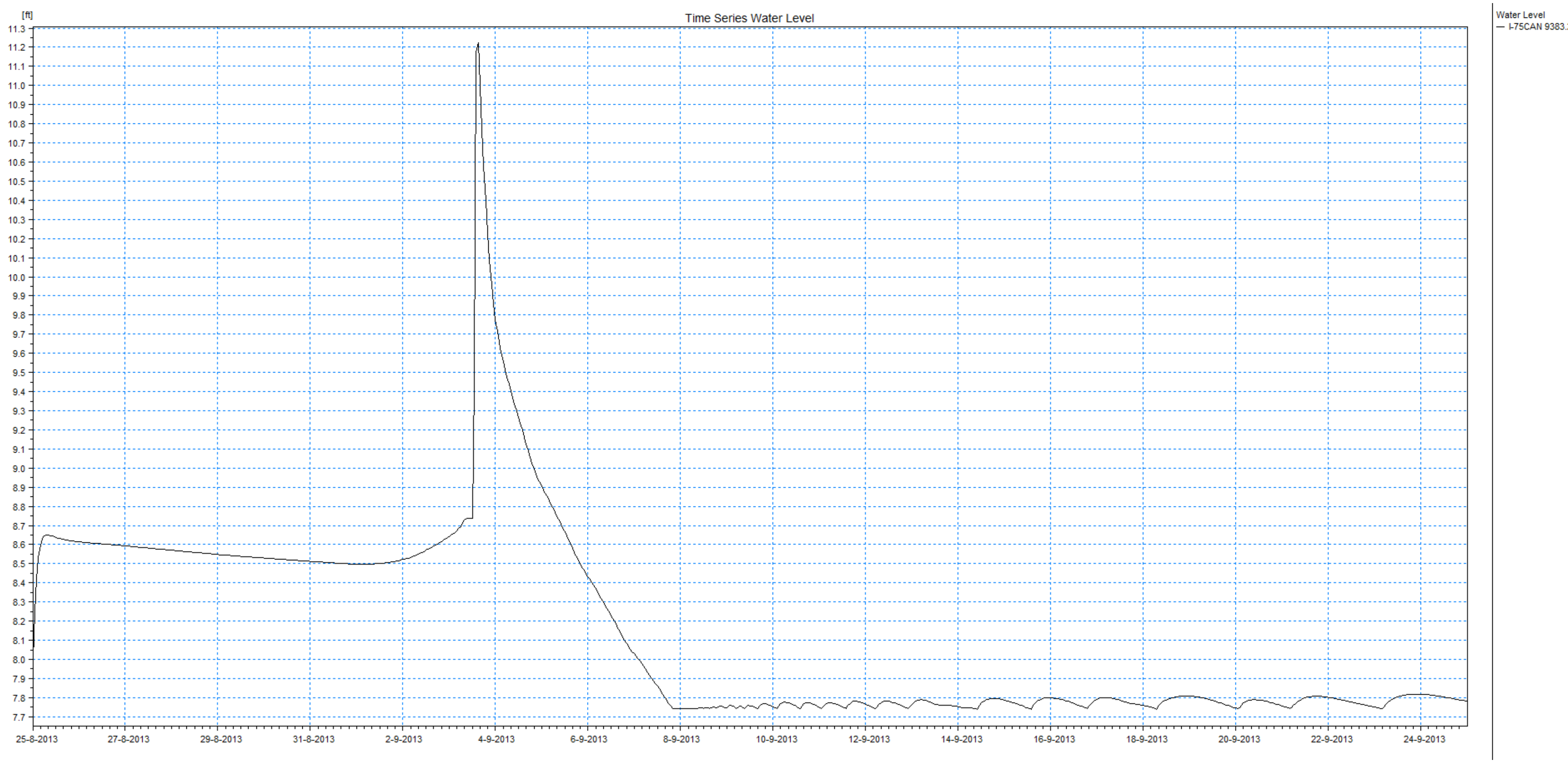

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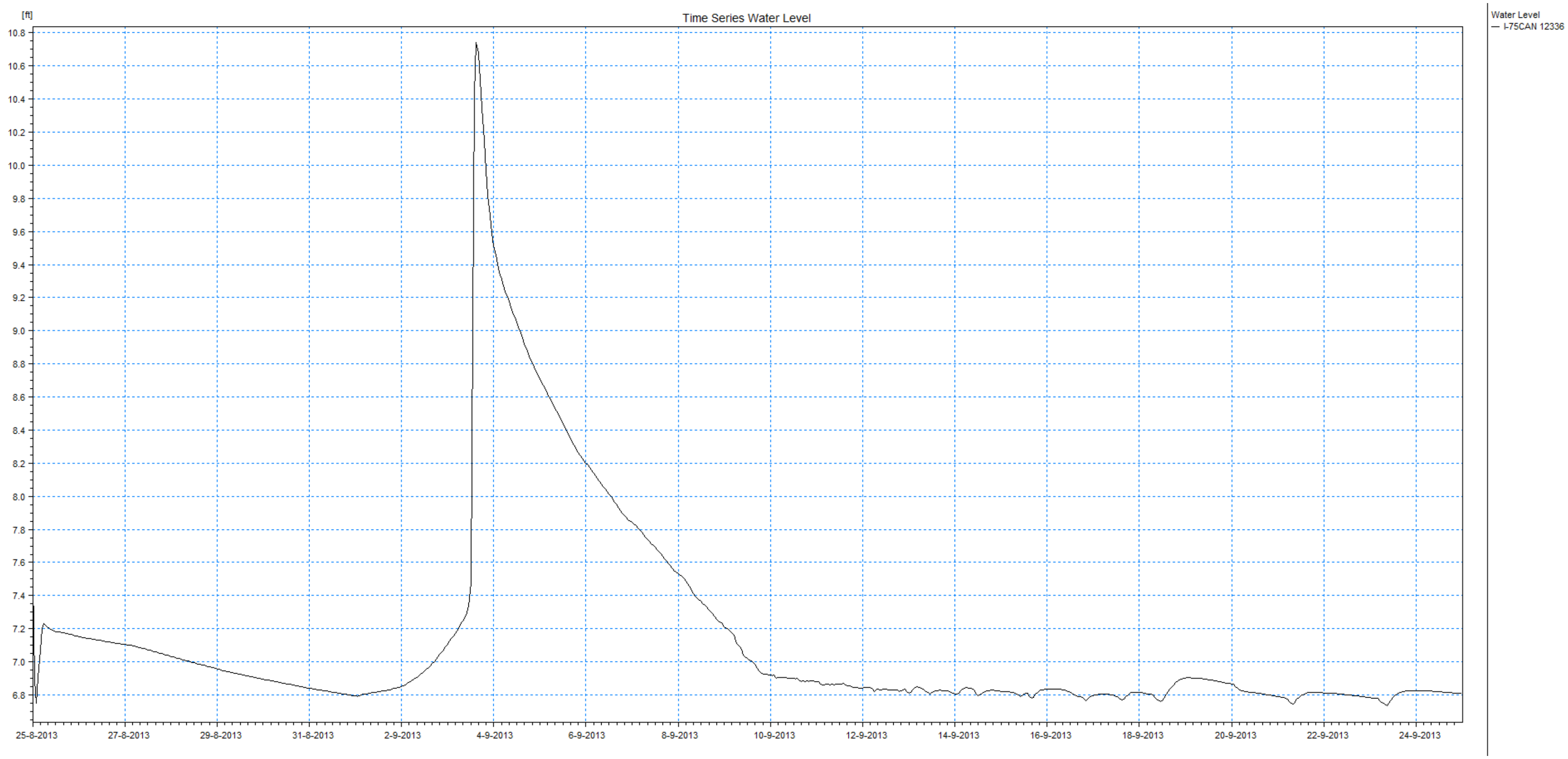

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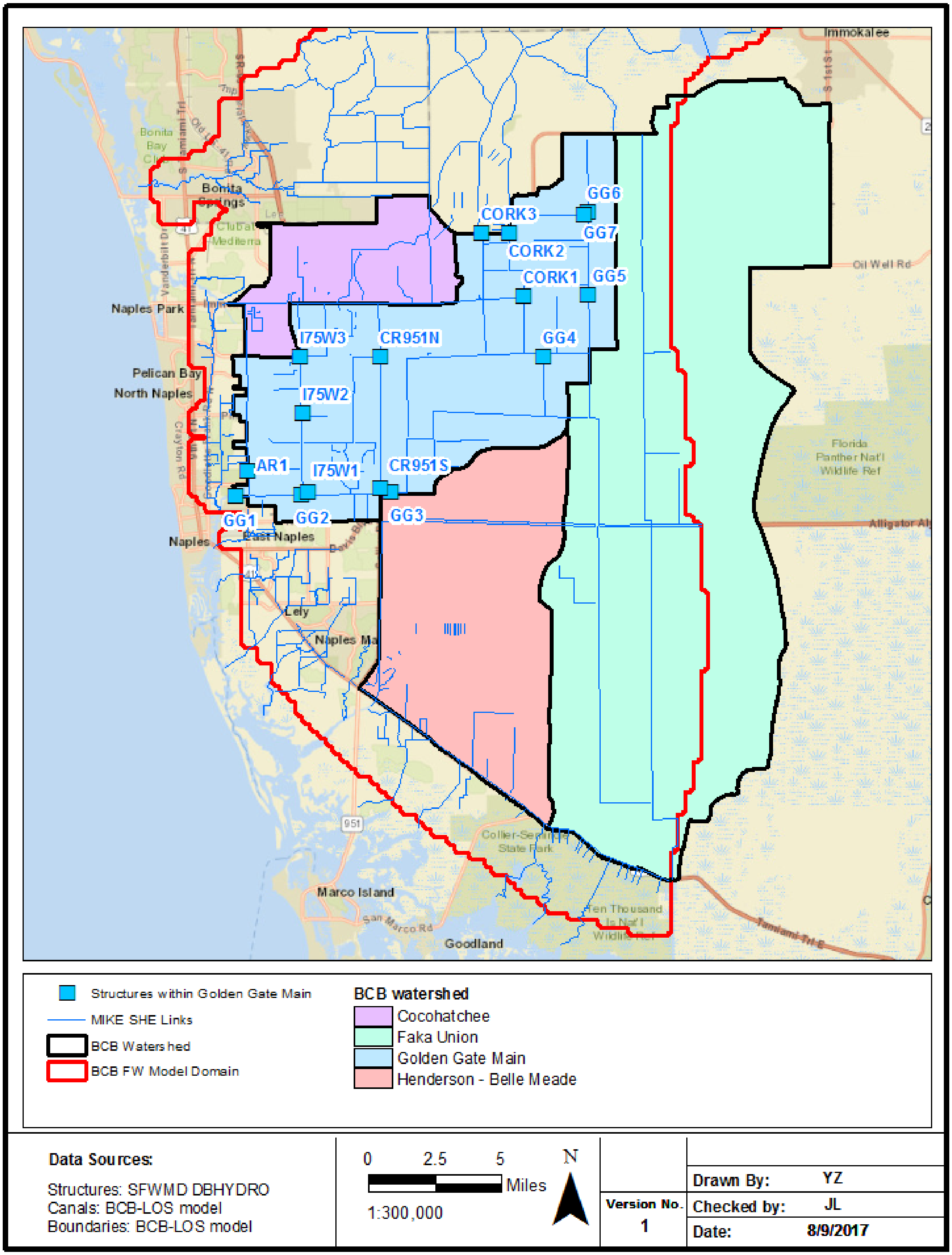

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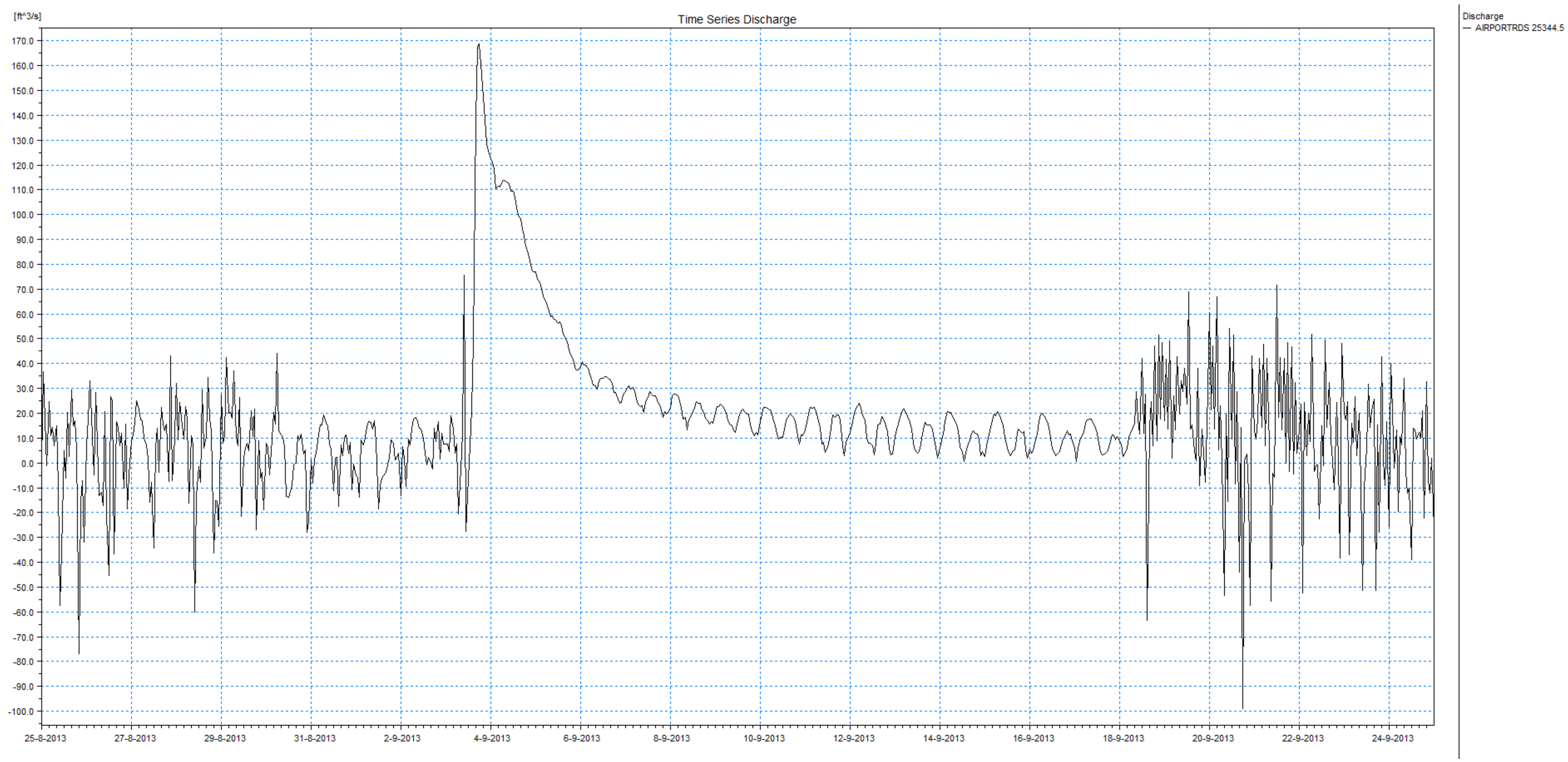

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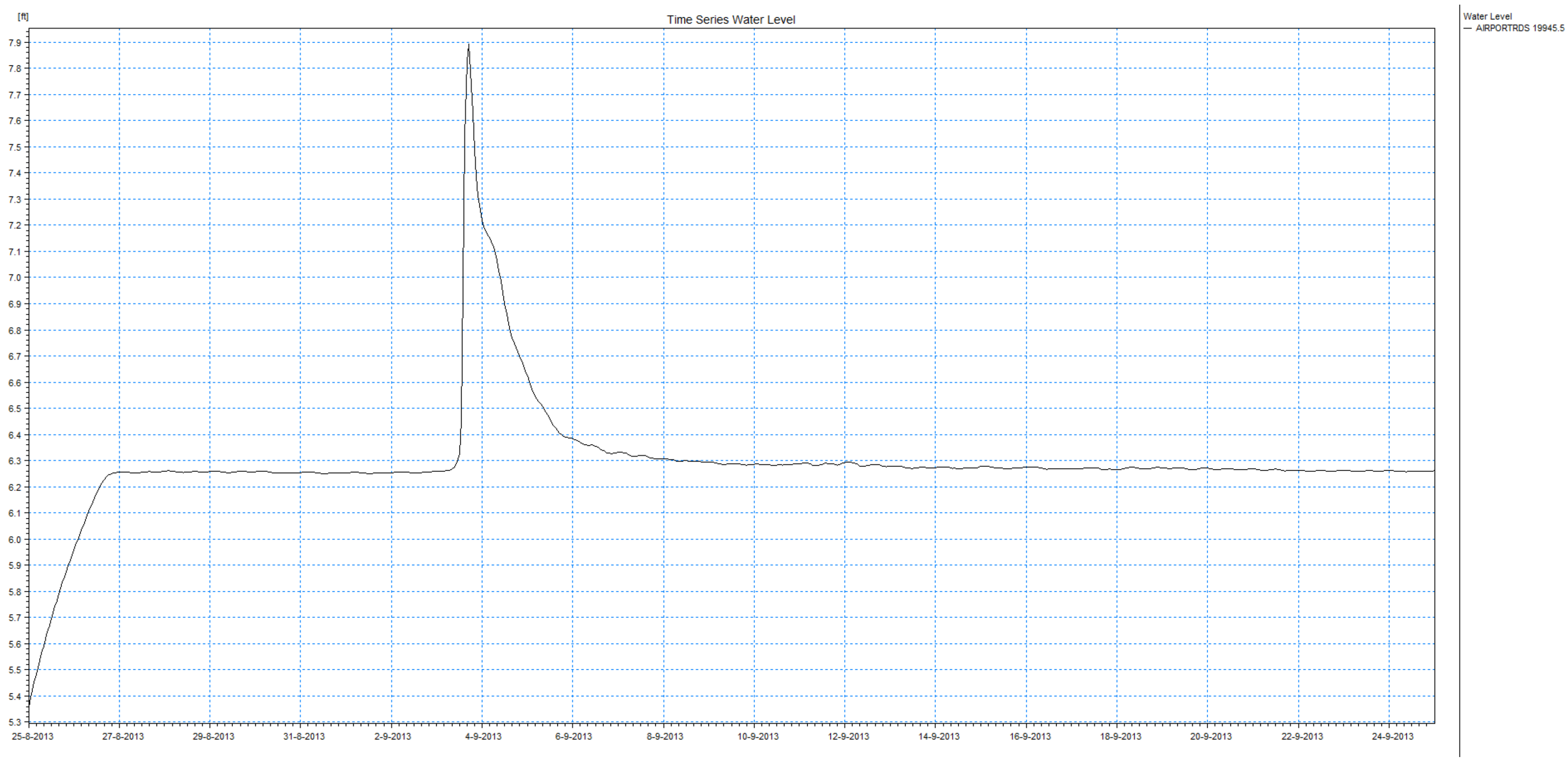

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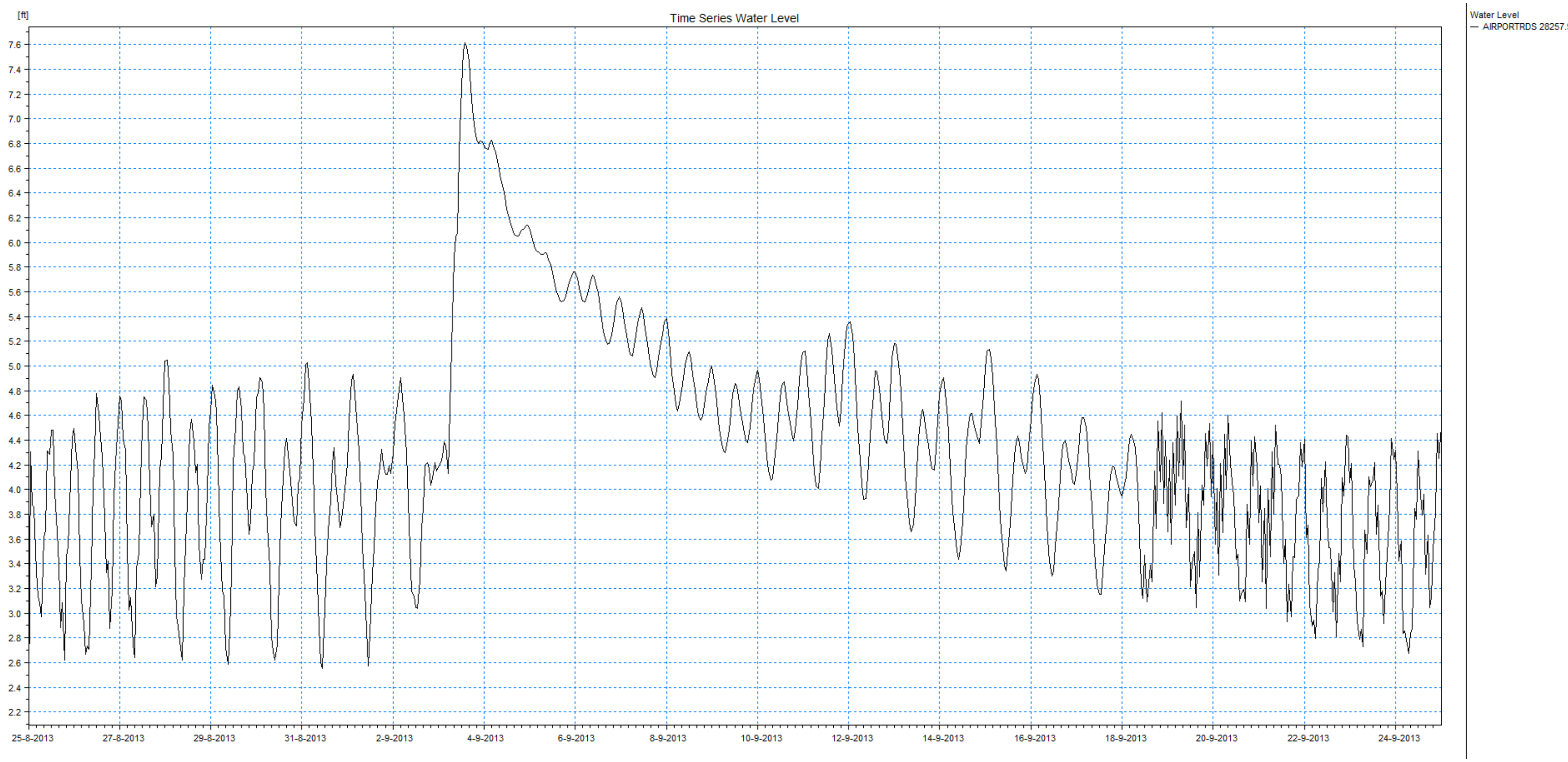

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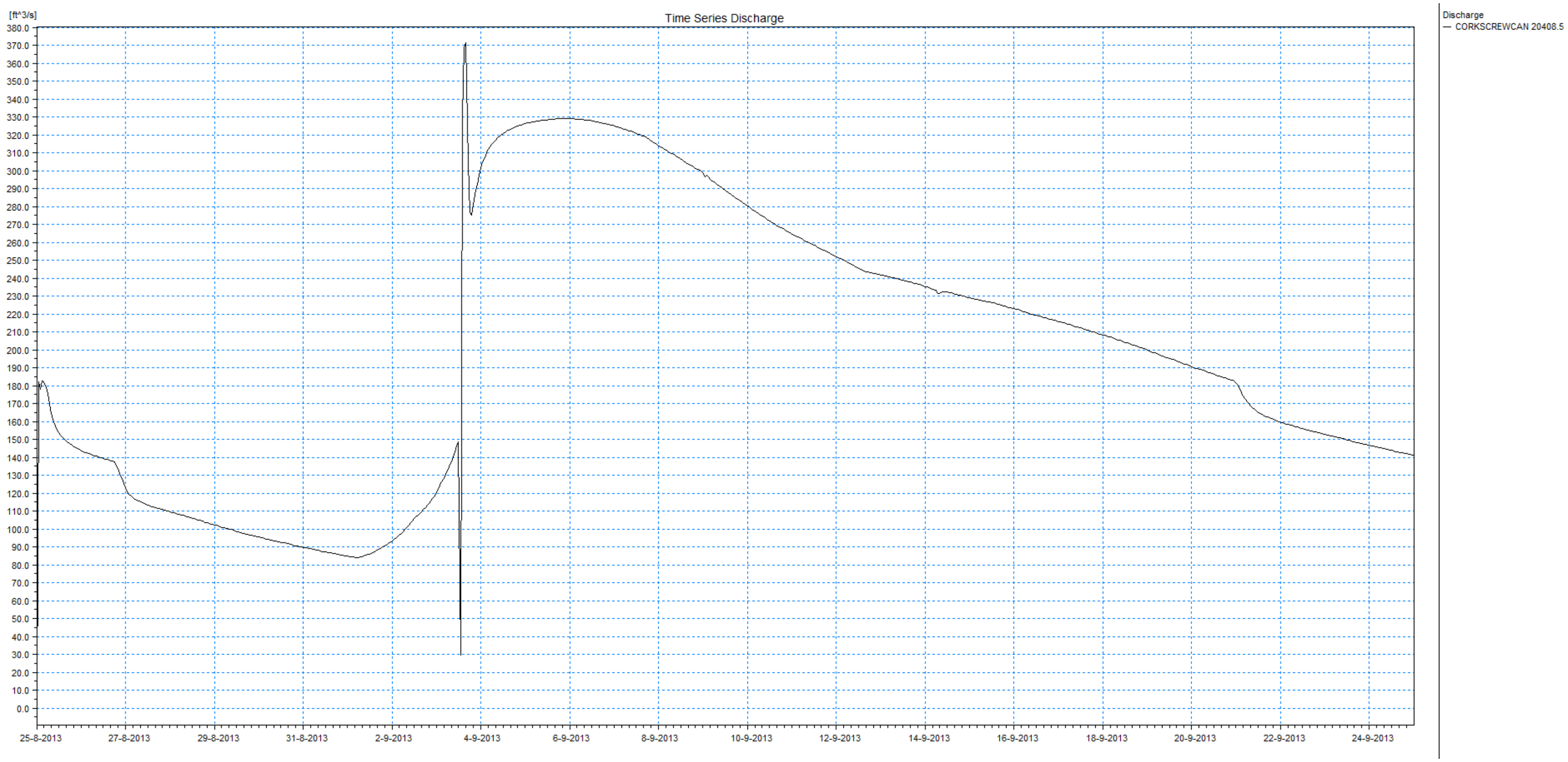

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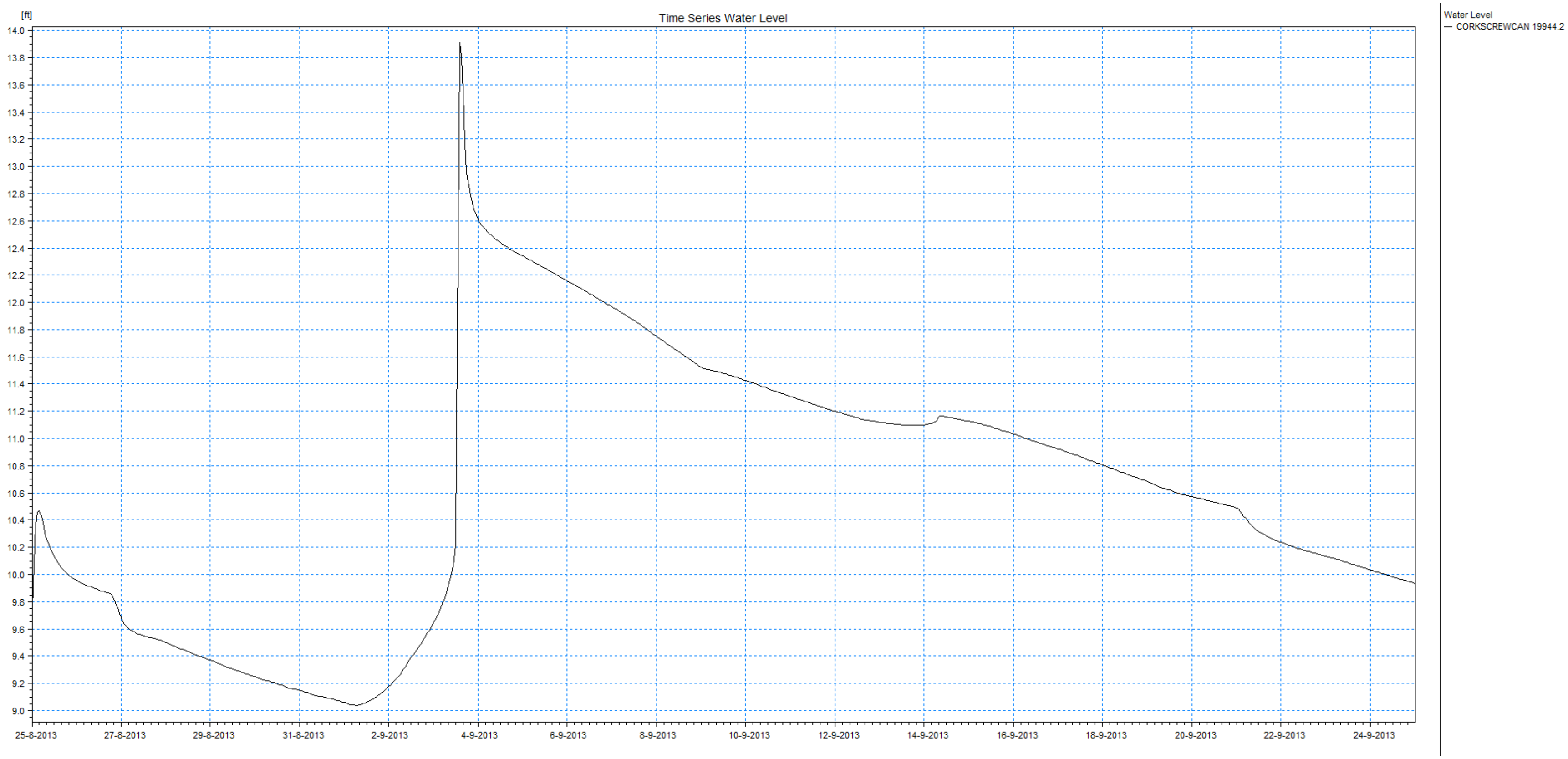

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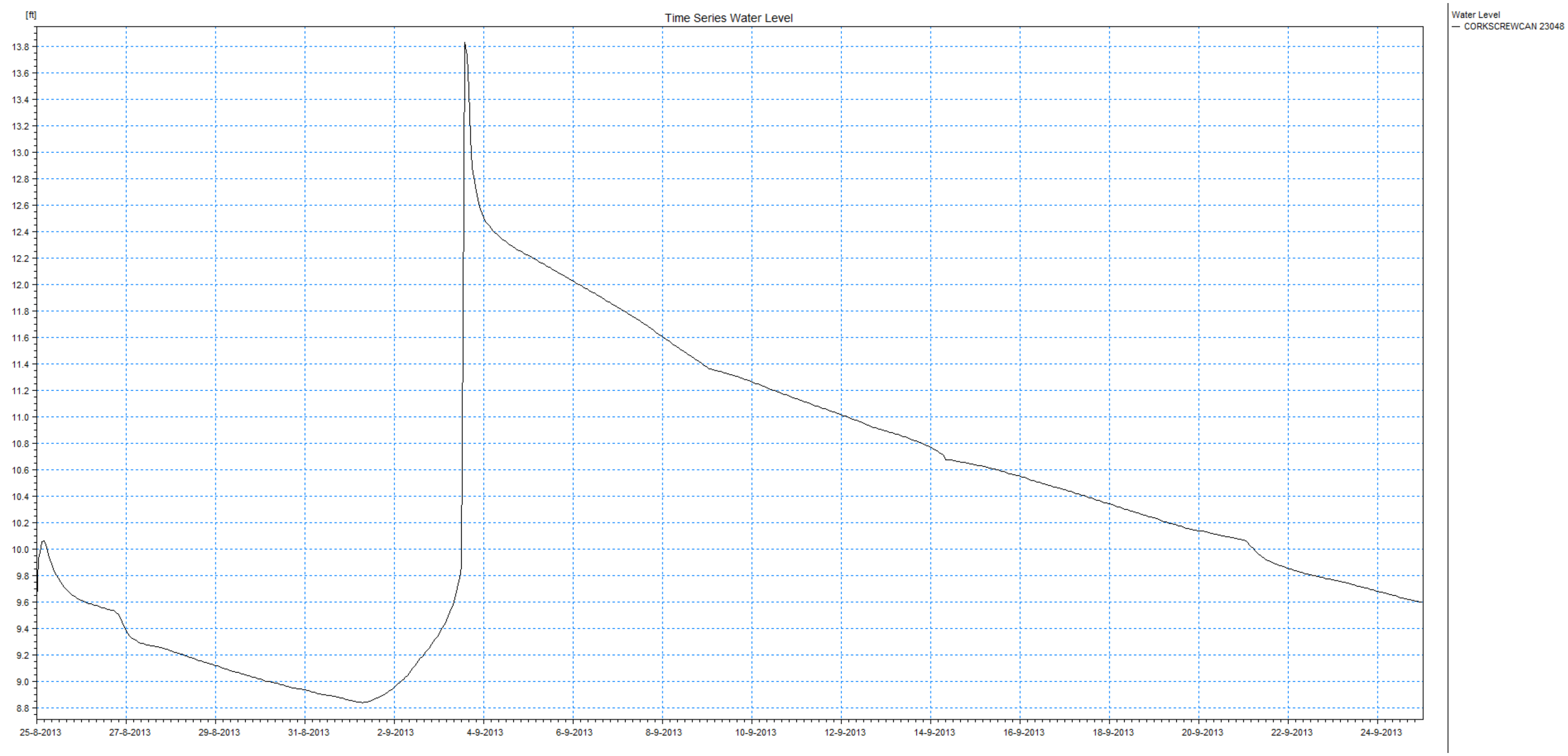

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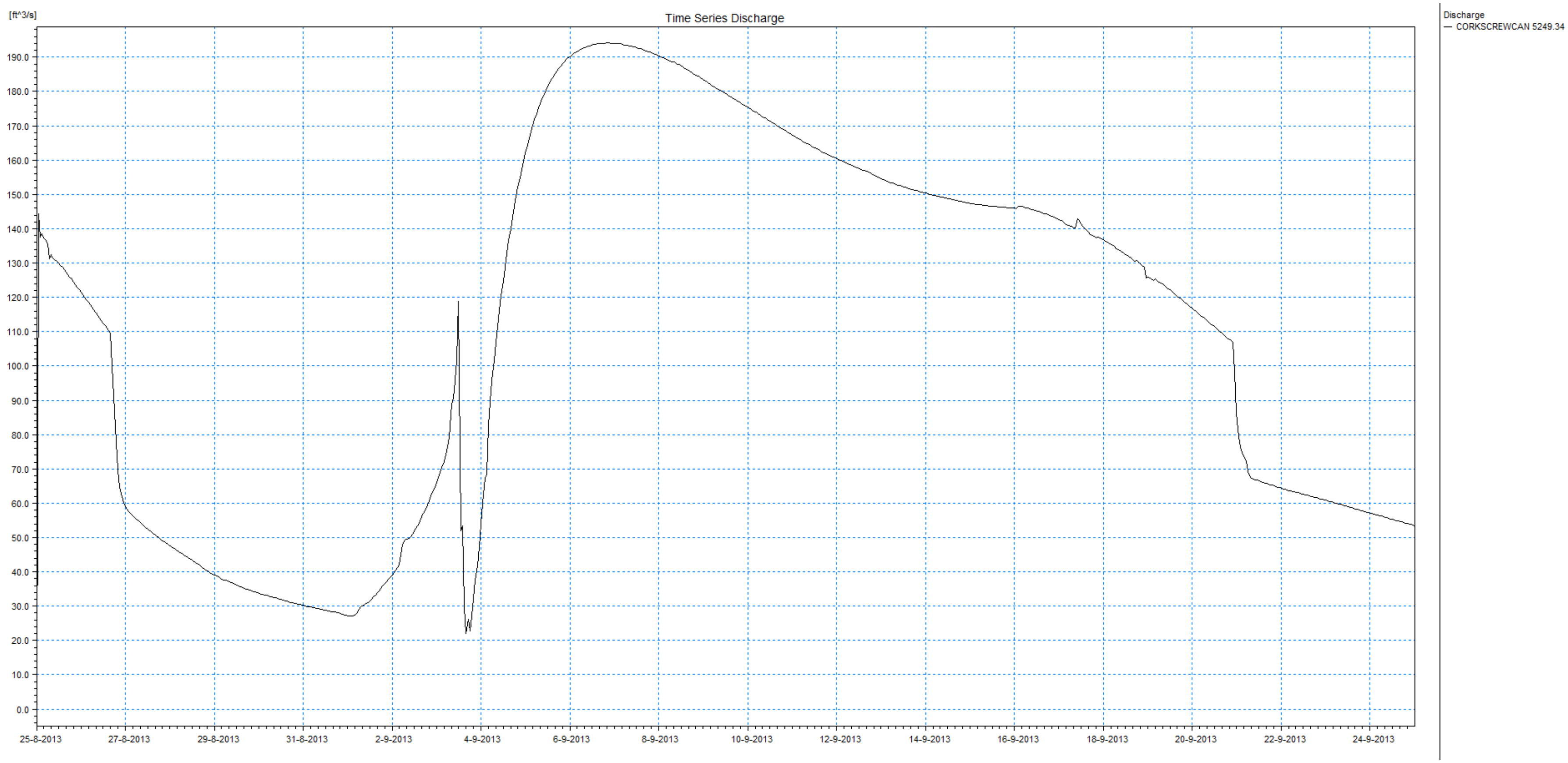

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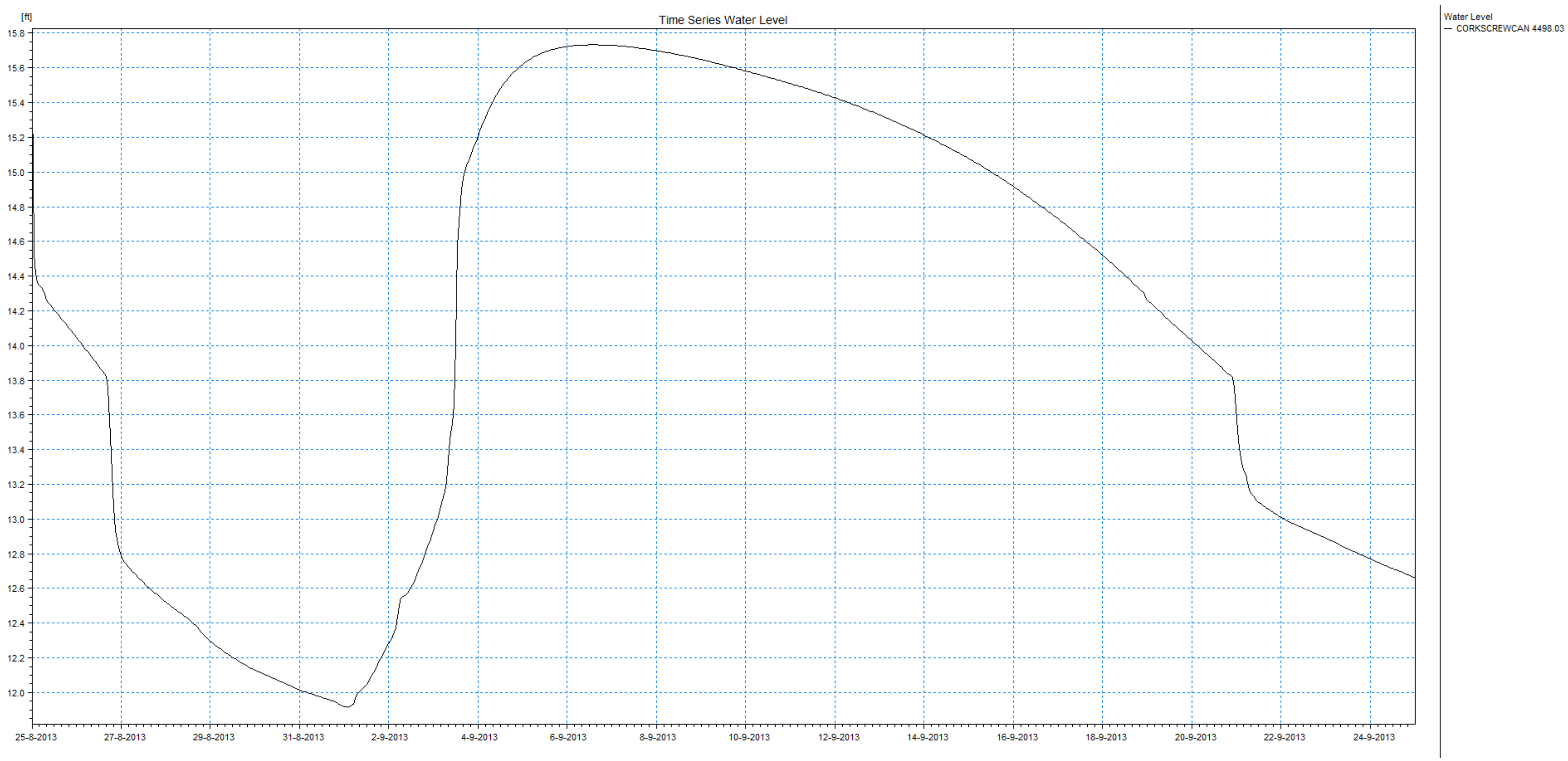

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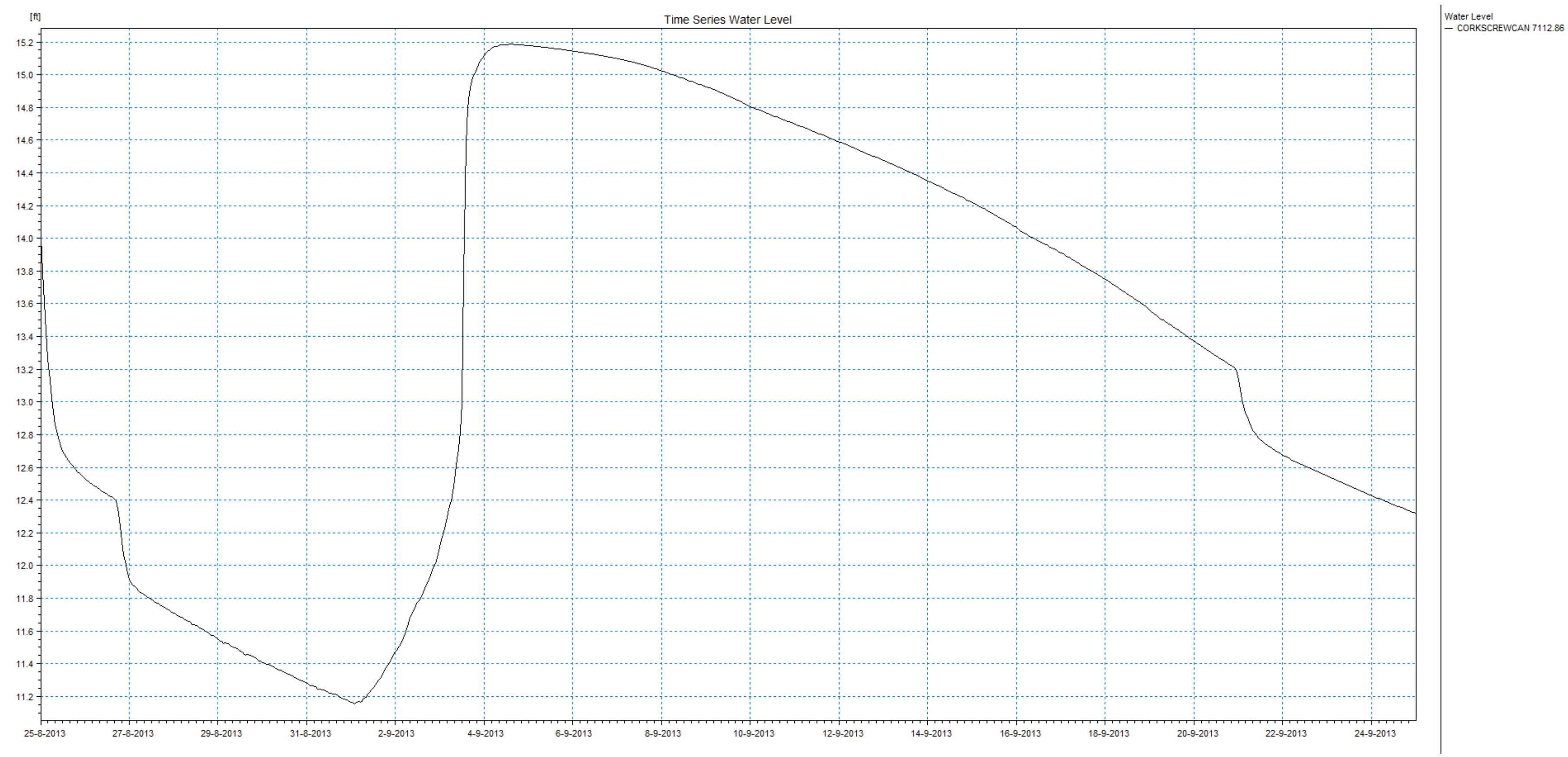

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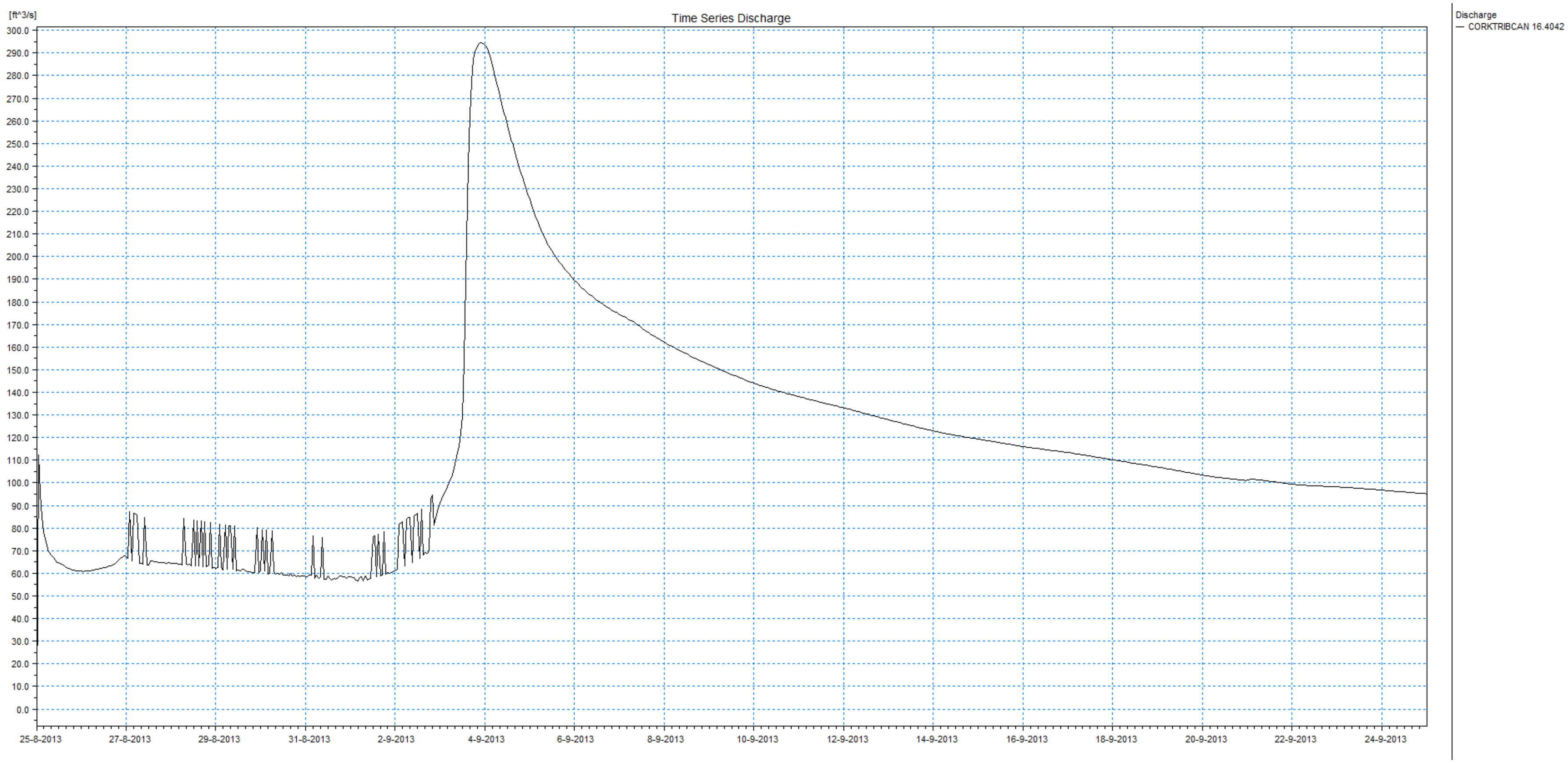


Figure 11. CORK 3 Discharge (cfs)

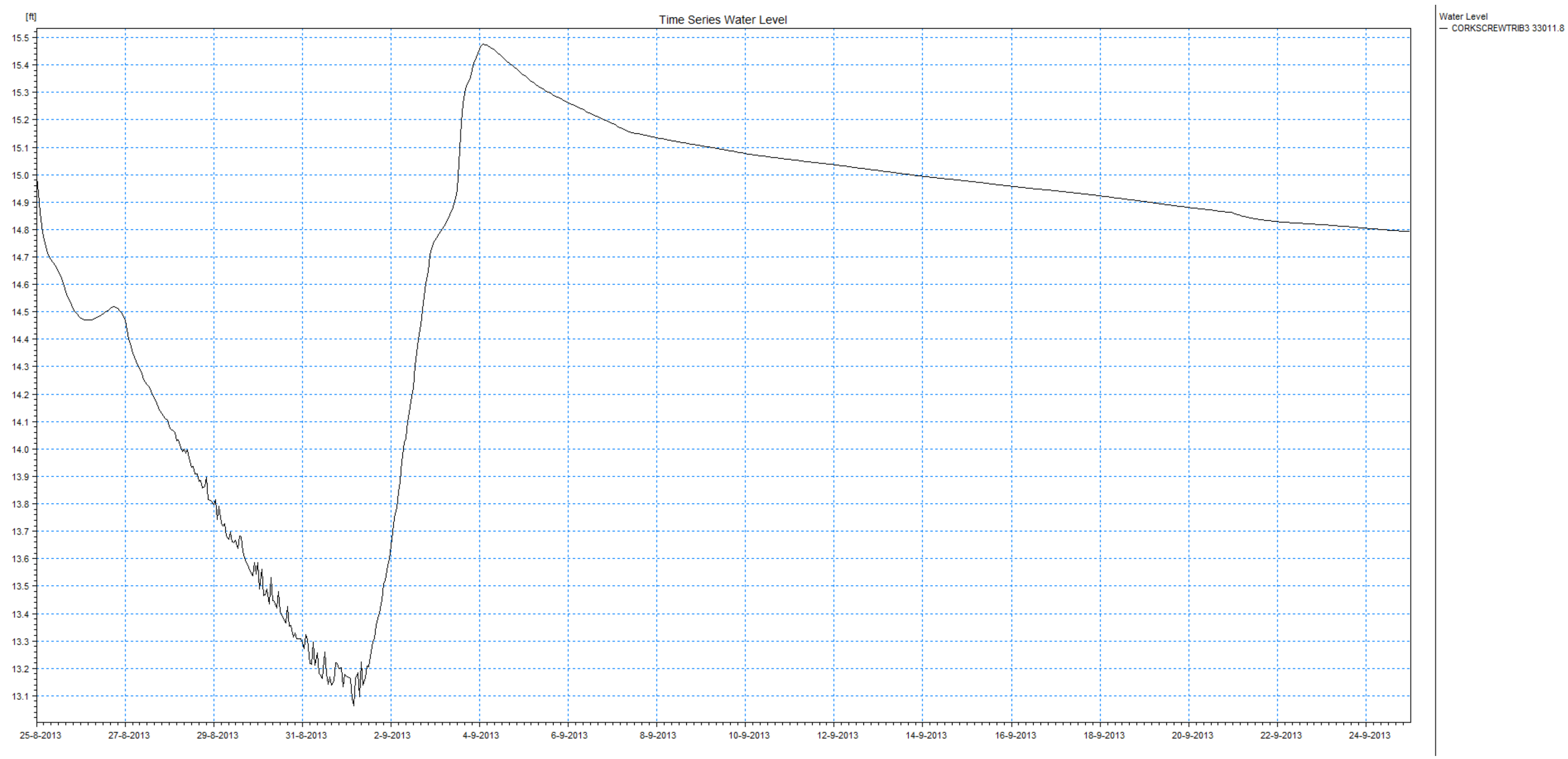


Figure 12. CORK 3 Headwater Stage (ft)

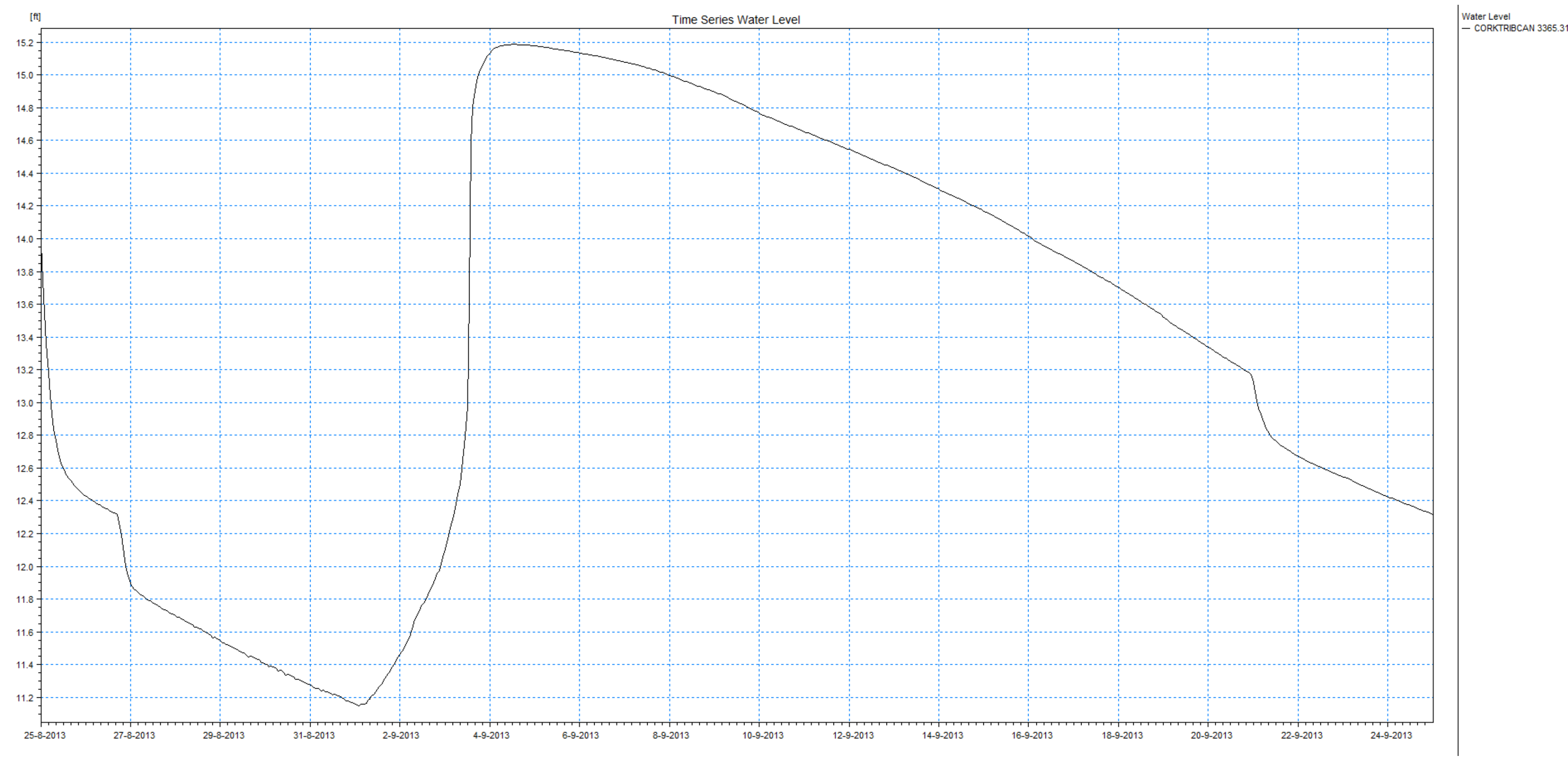


Figure 13. CORK 3 Tailwater Stage (ft)

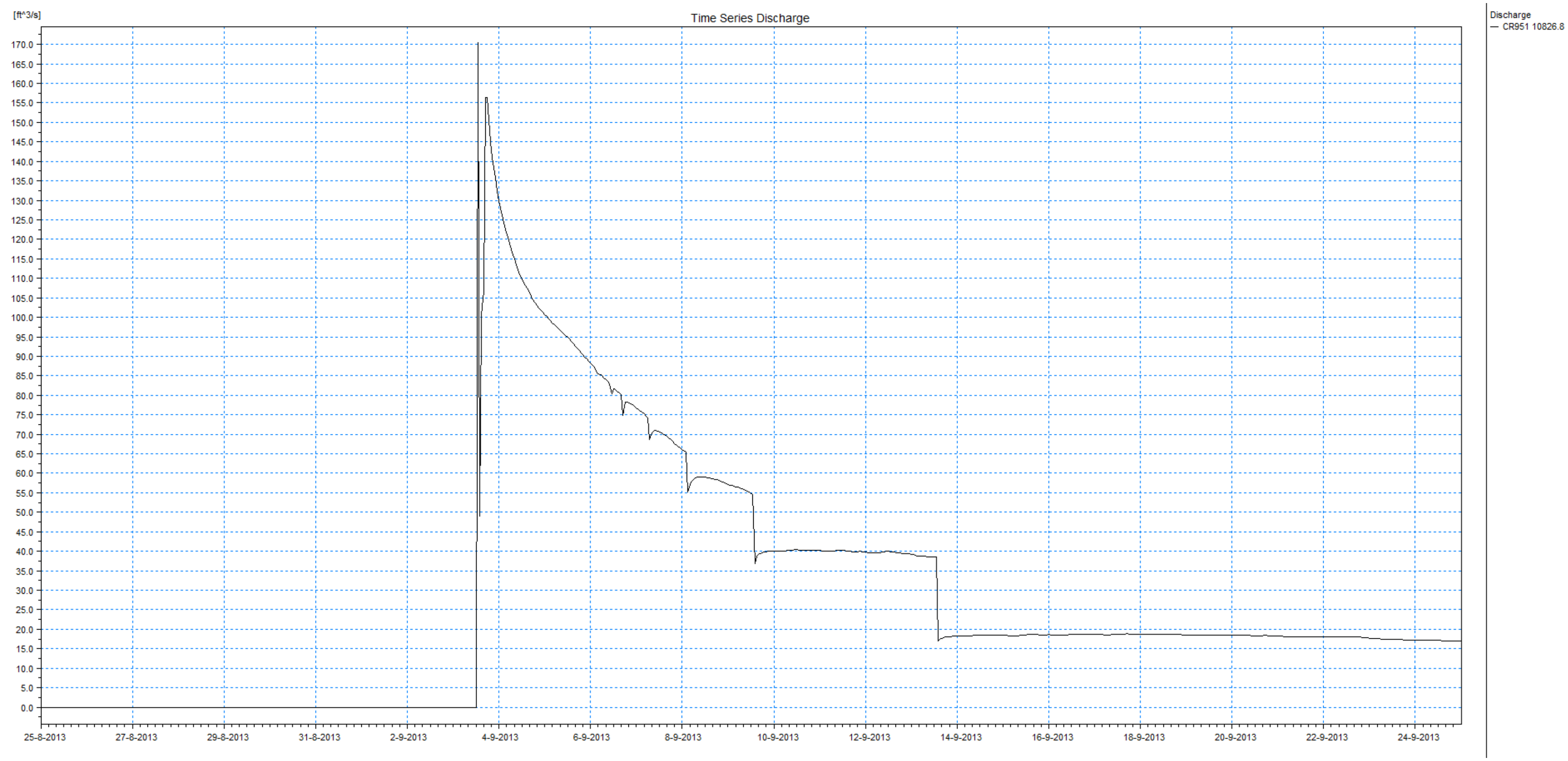


Figure 14. CR951N Discharge (cfs)

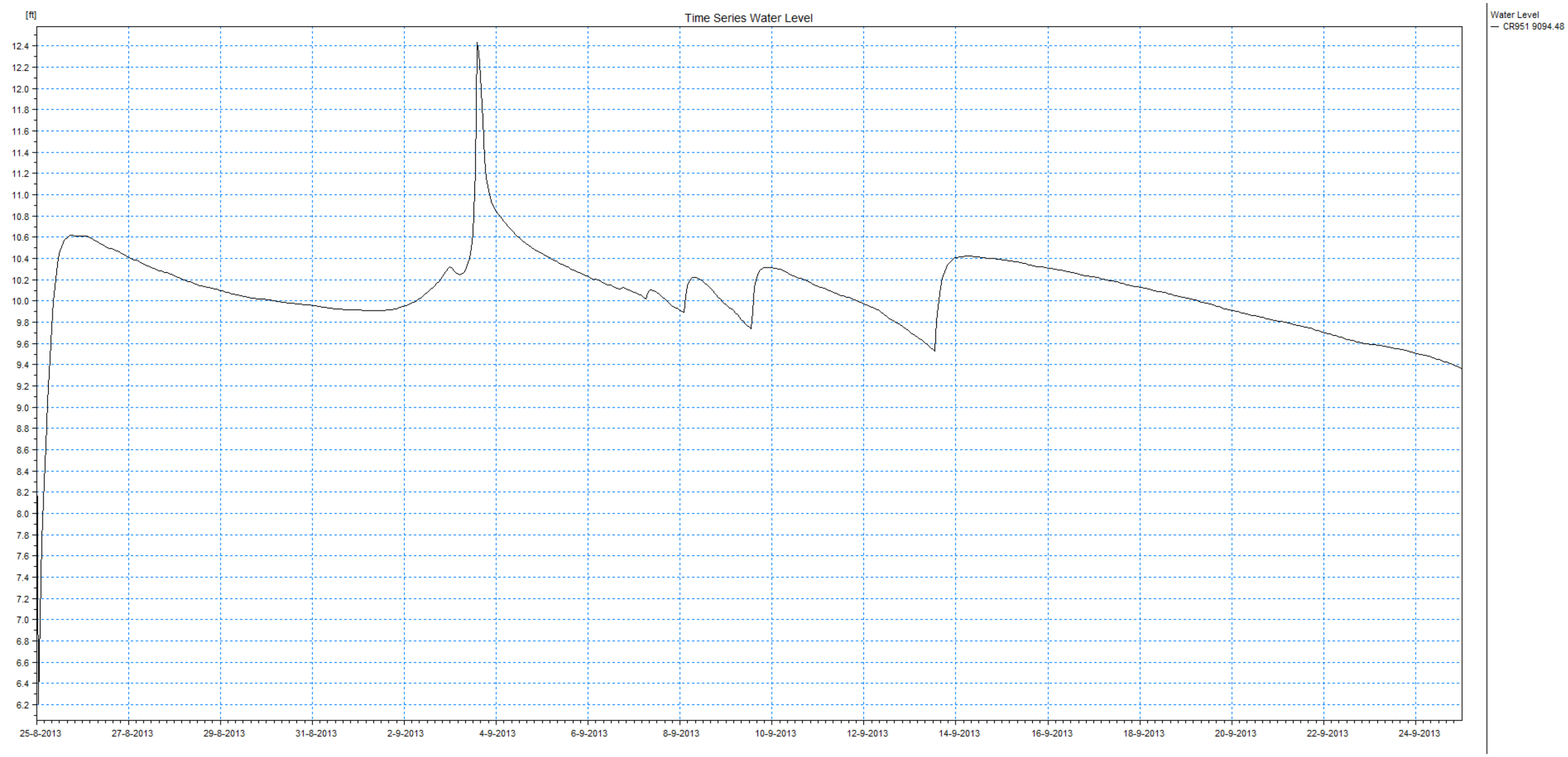


Figure 15. CR951N Headwater Stage (ft)

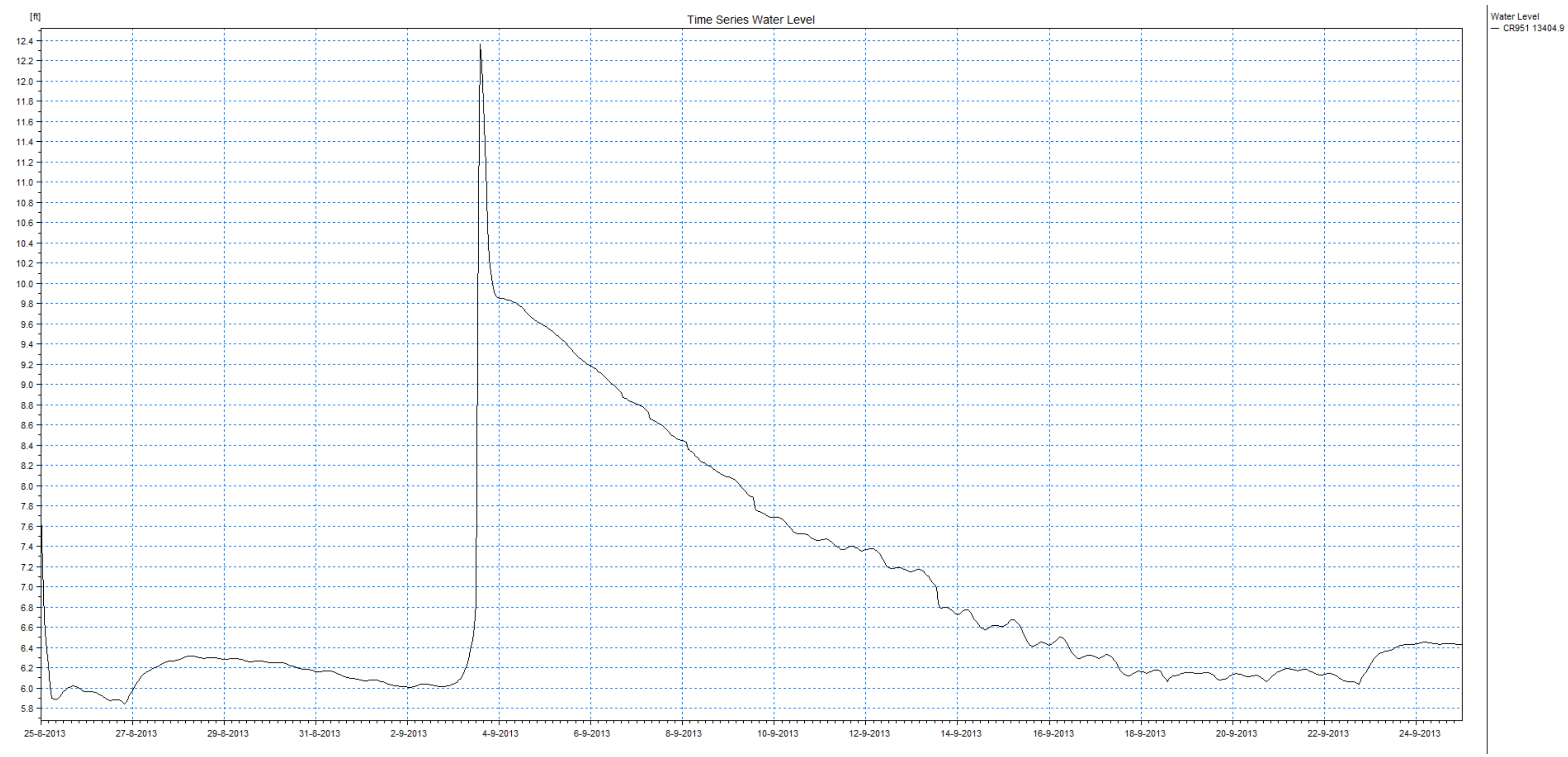


Figure 16. CR951N Tailwater Stage (ft)

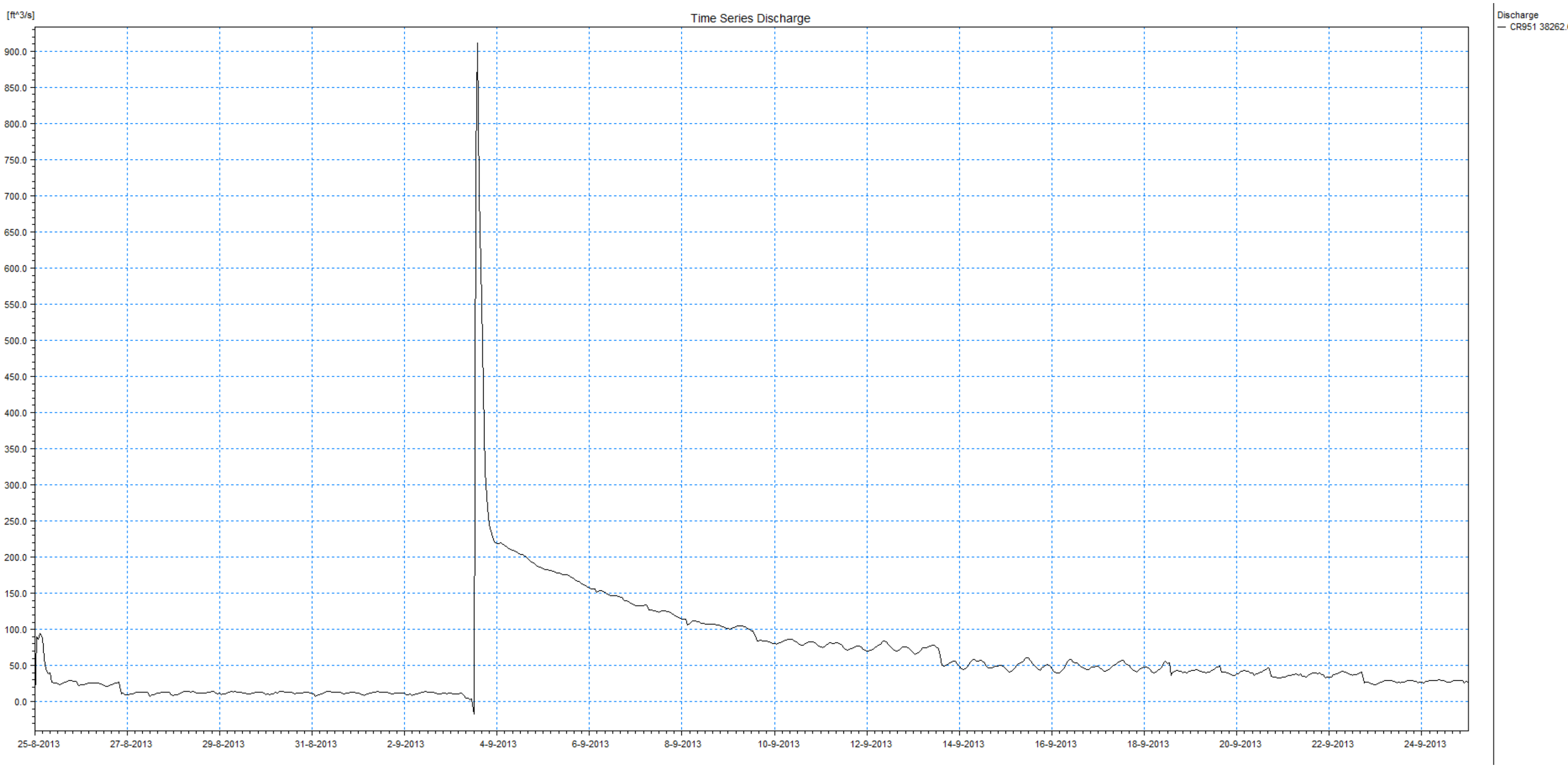


Figure 17. CR951S Discharge (cfs)

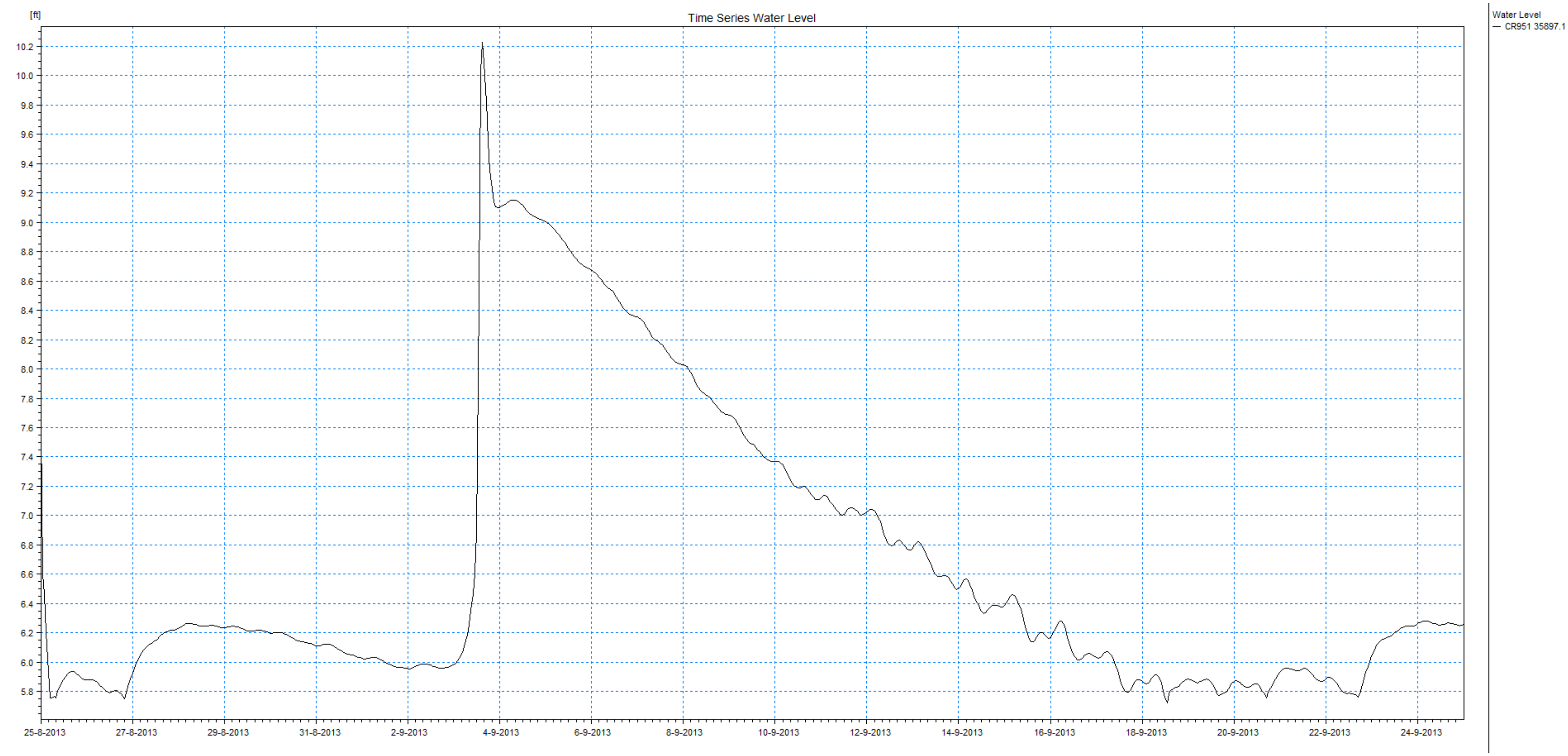


Figure 18. CR951S Headwater Stage (ft)

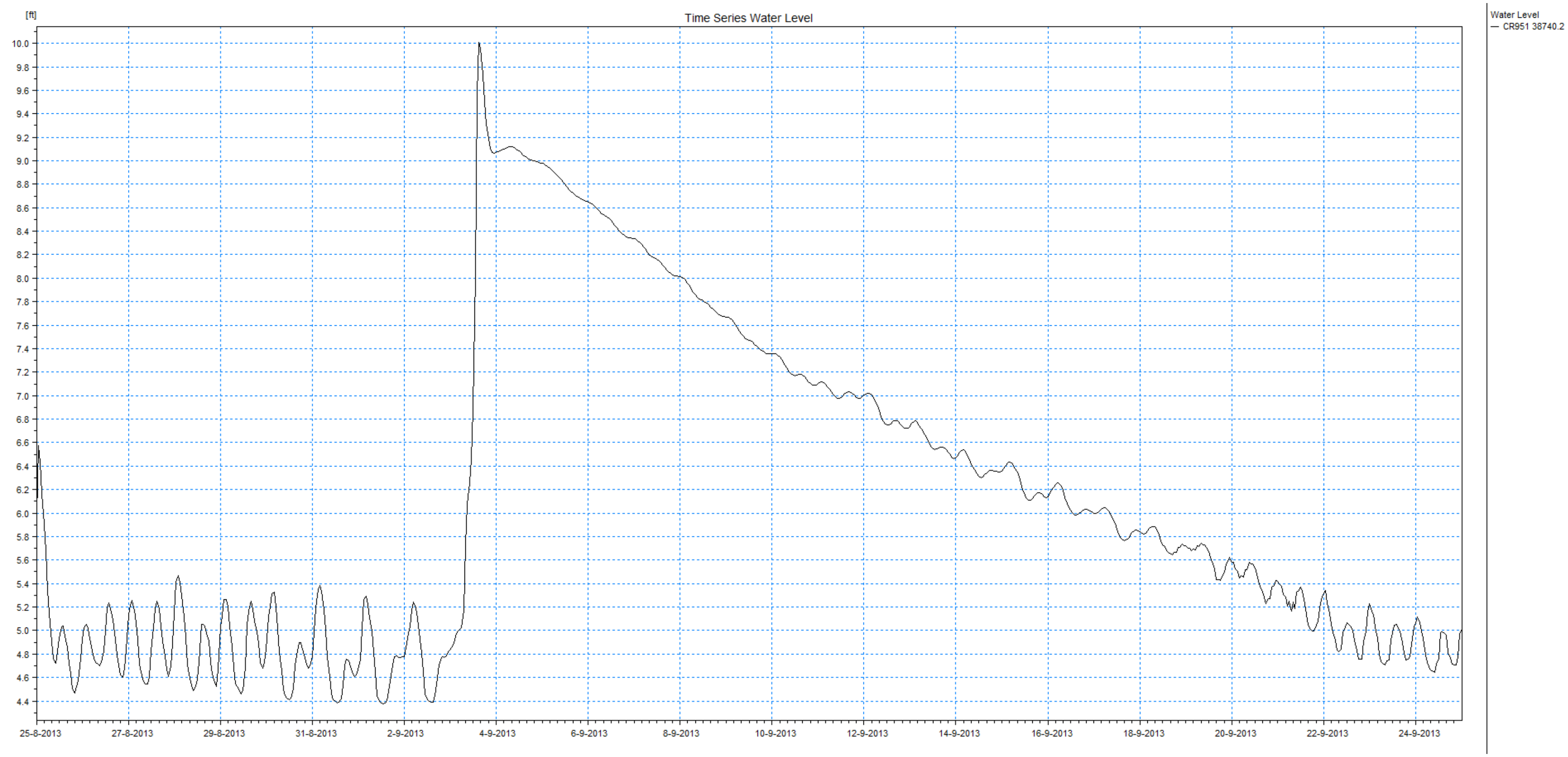


Figure 19. CR951S Tailwater Stage (ft)

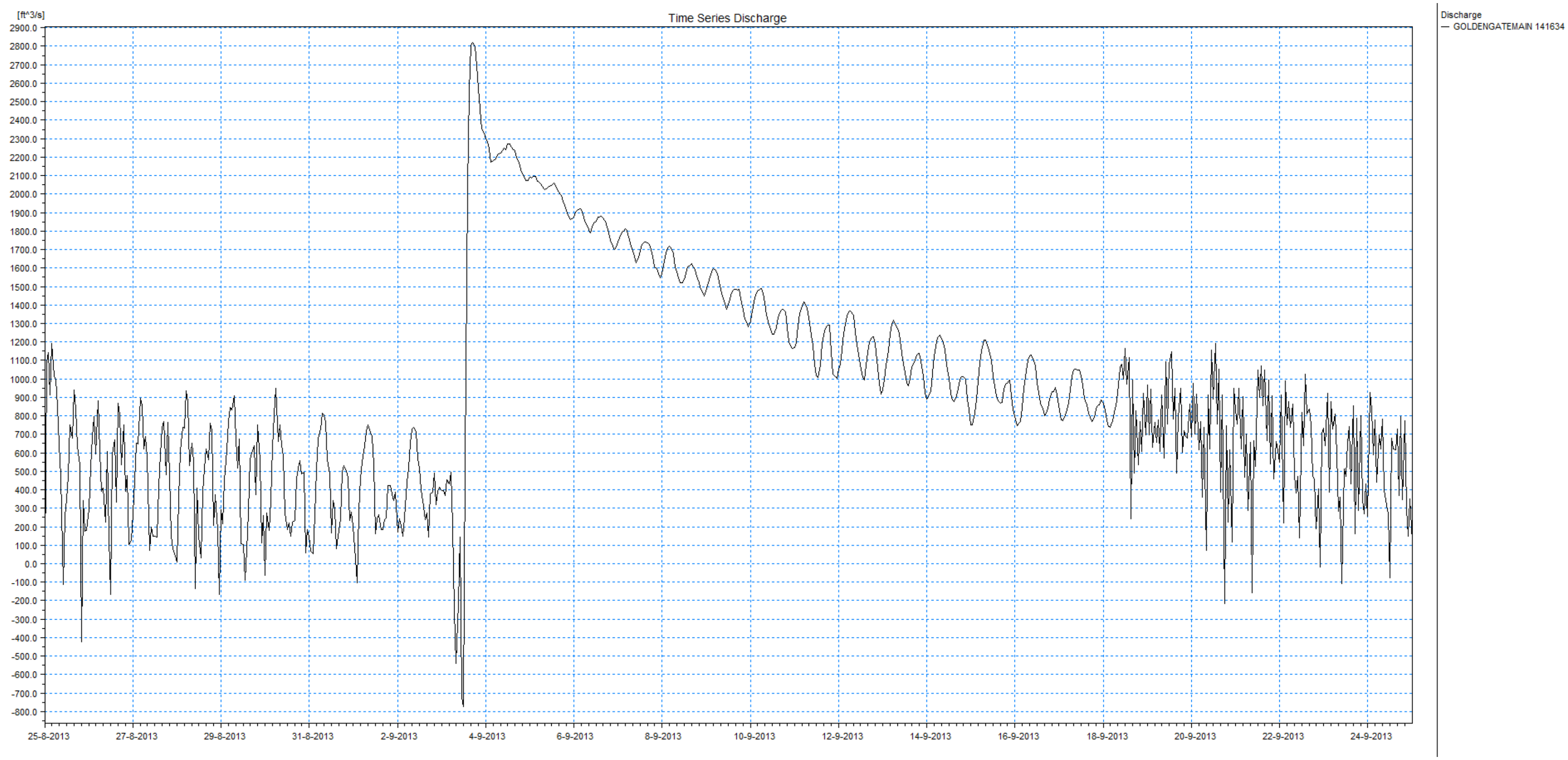


Figure 20. GG1 Discharge (cfs)

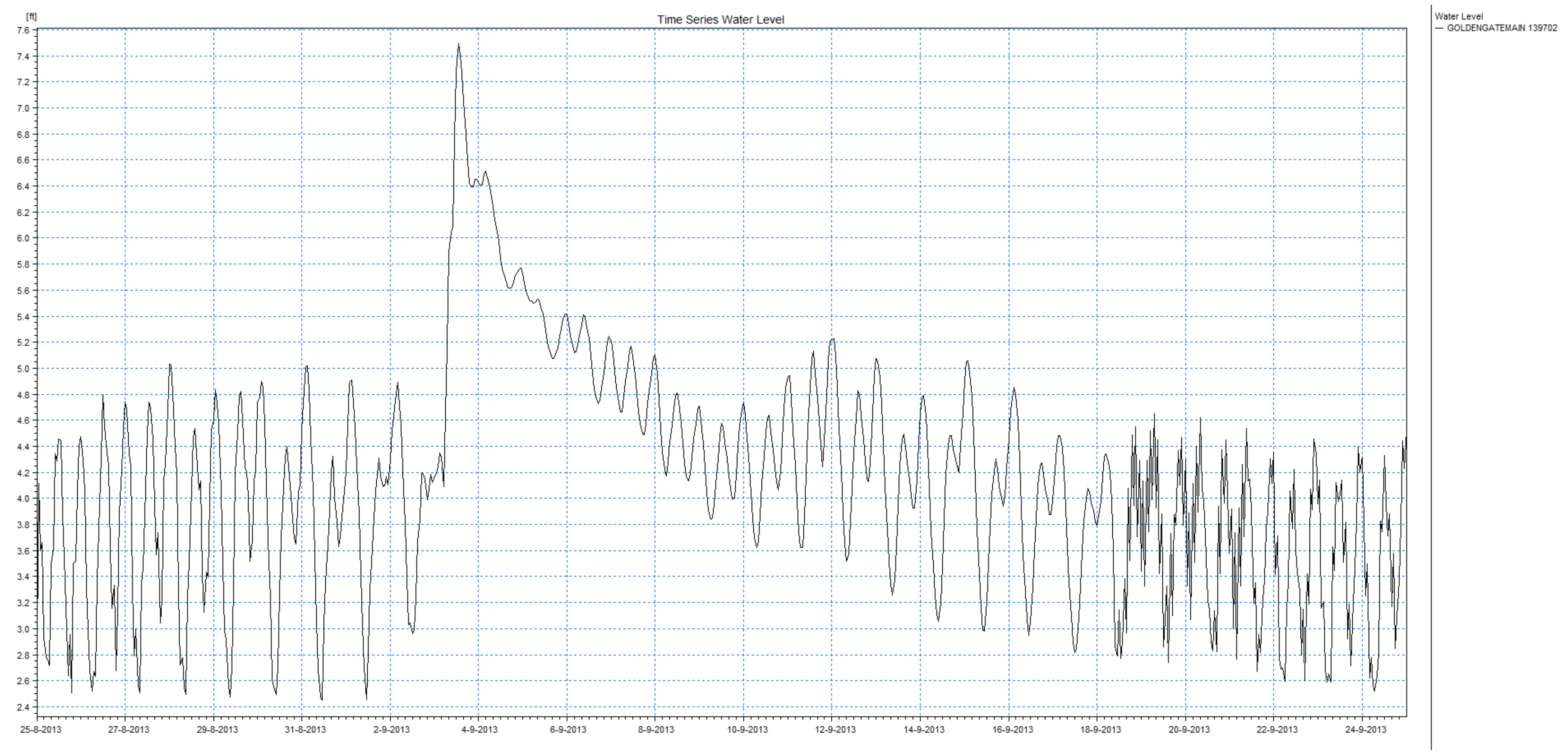


Figure 21. GG1 Headwater Stage (ft)

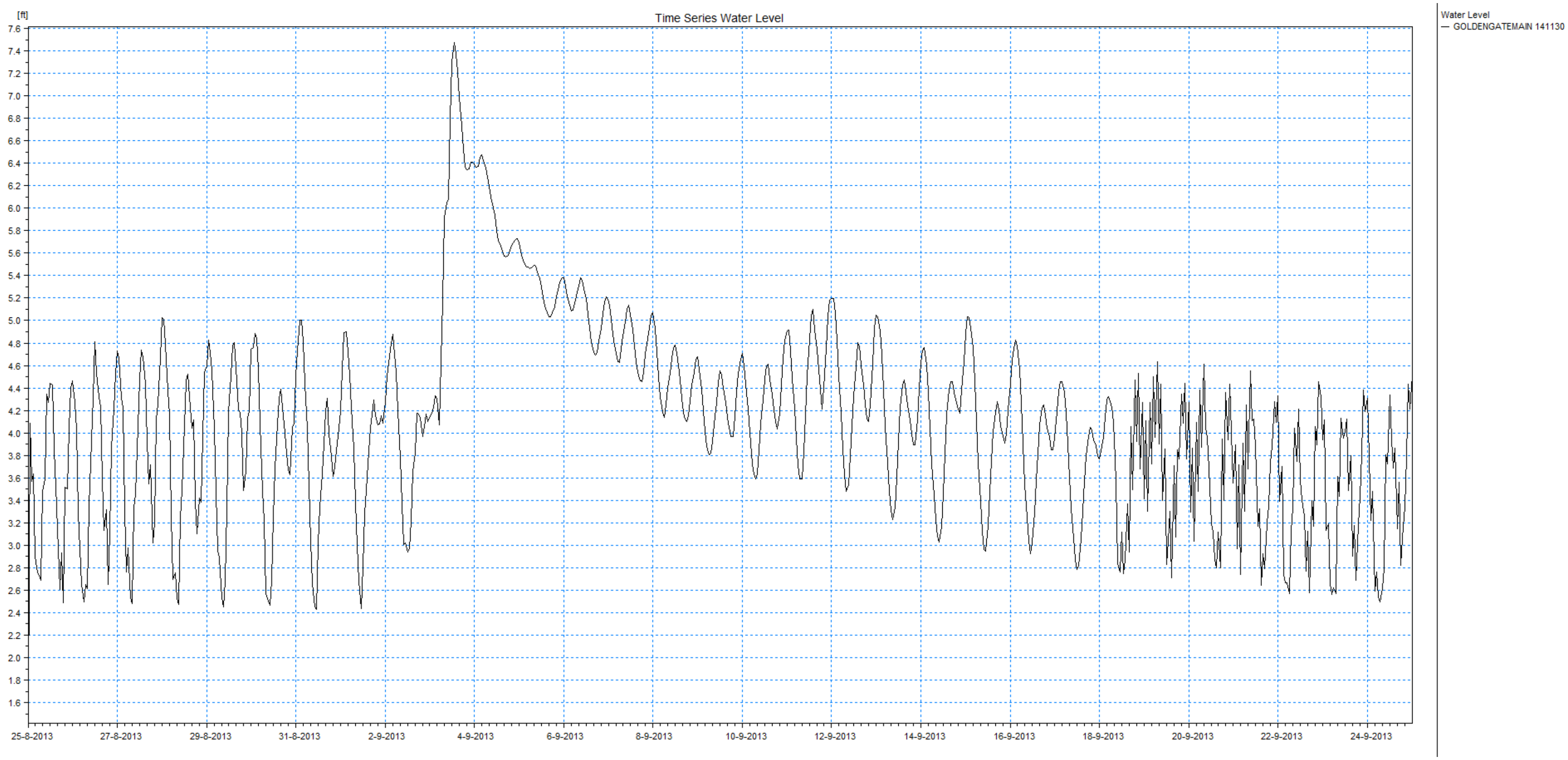


Figure 22. GG1 Tailwater Stage (ft)

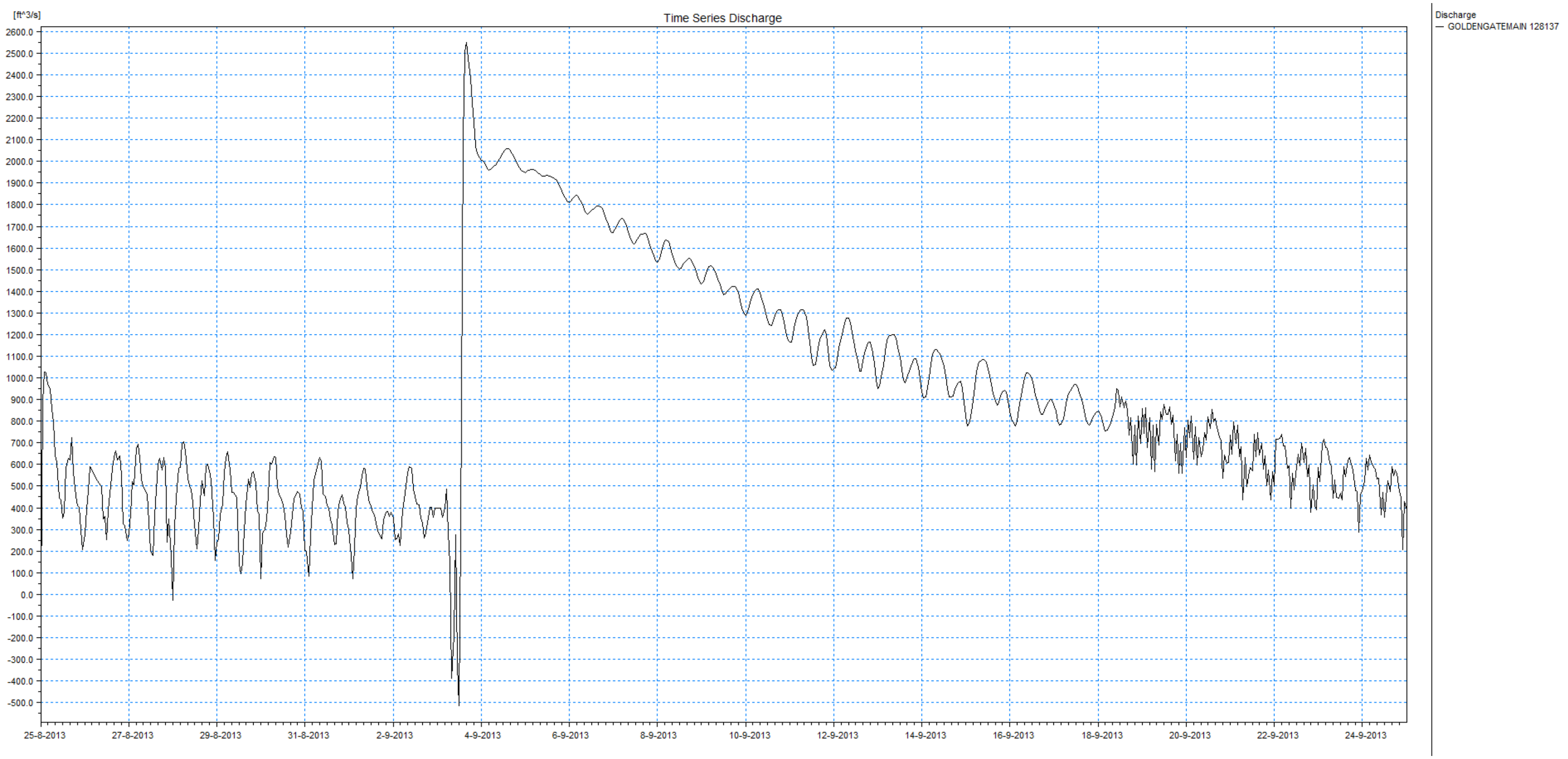


Figure 23. GG2 Discharge (cfs)

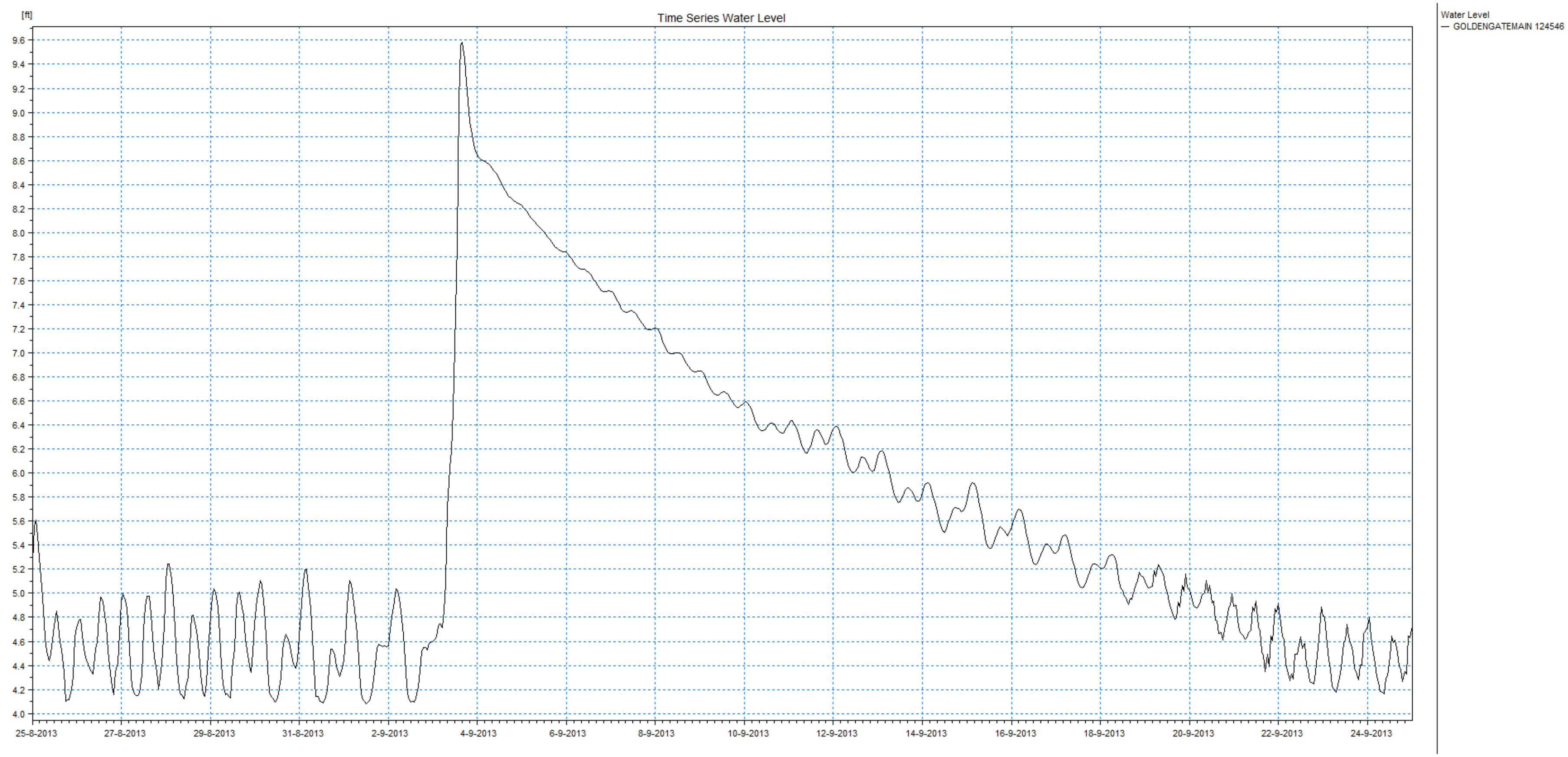


Figure 24. GG2 Headwater Stage (ft)

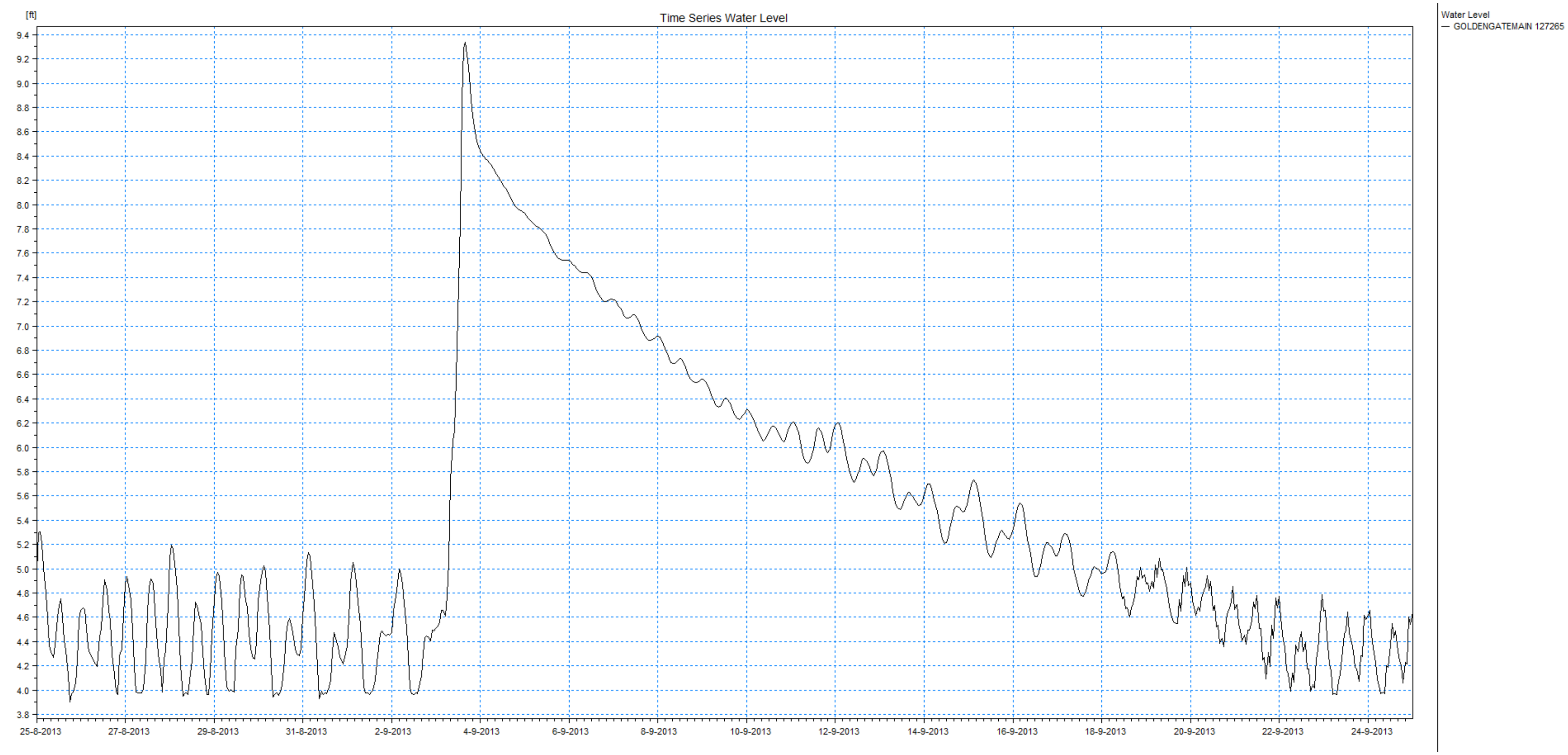


Figure 25. GG2 Tailwater Stage (ft)

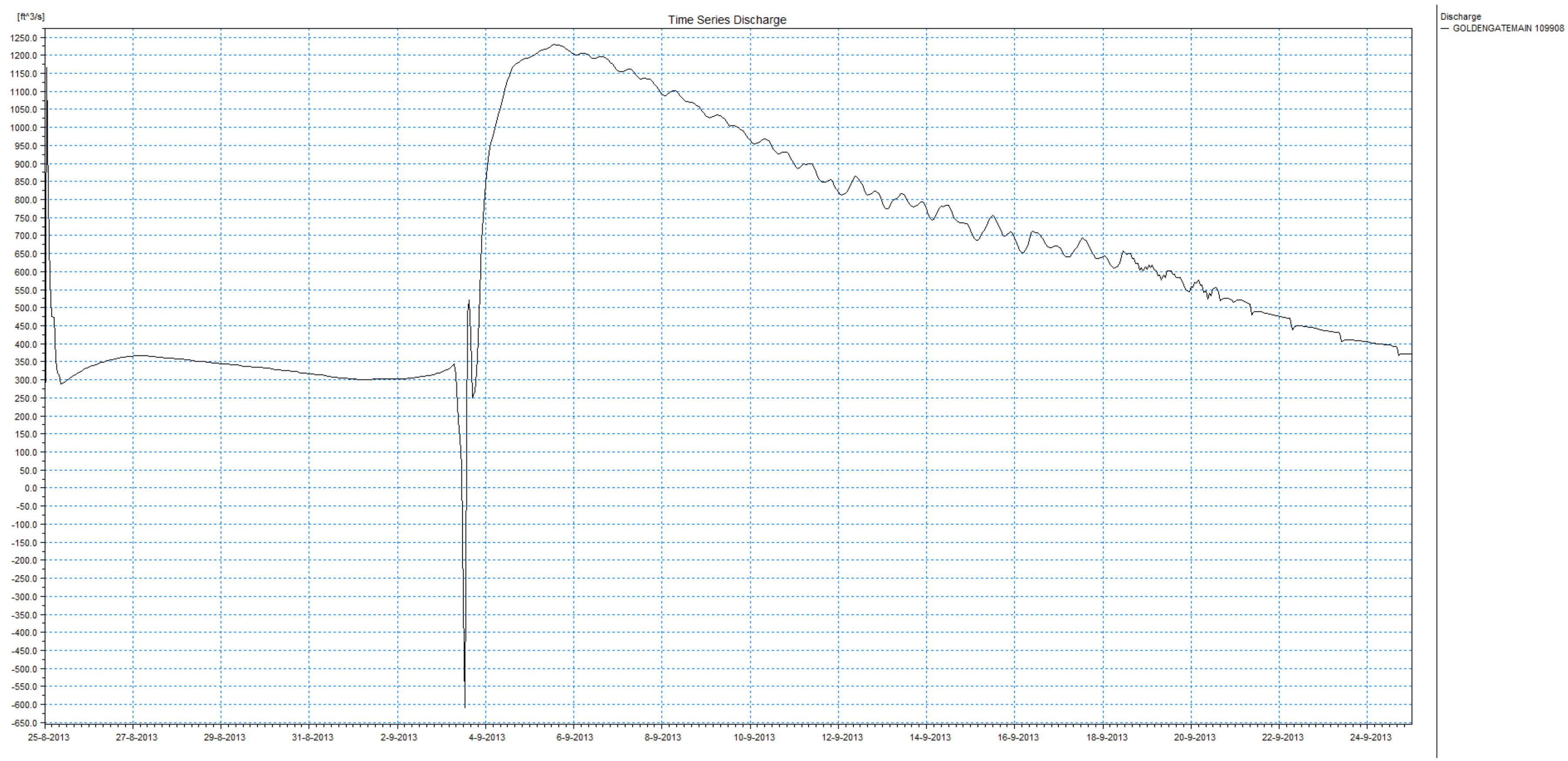


Figure 26. GG3 Discharge (cfs)

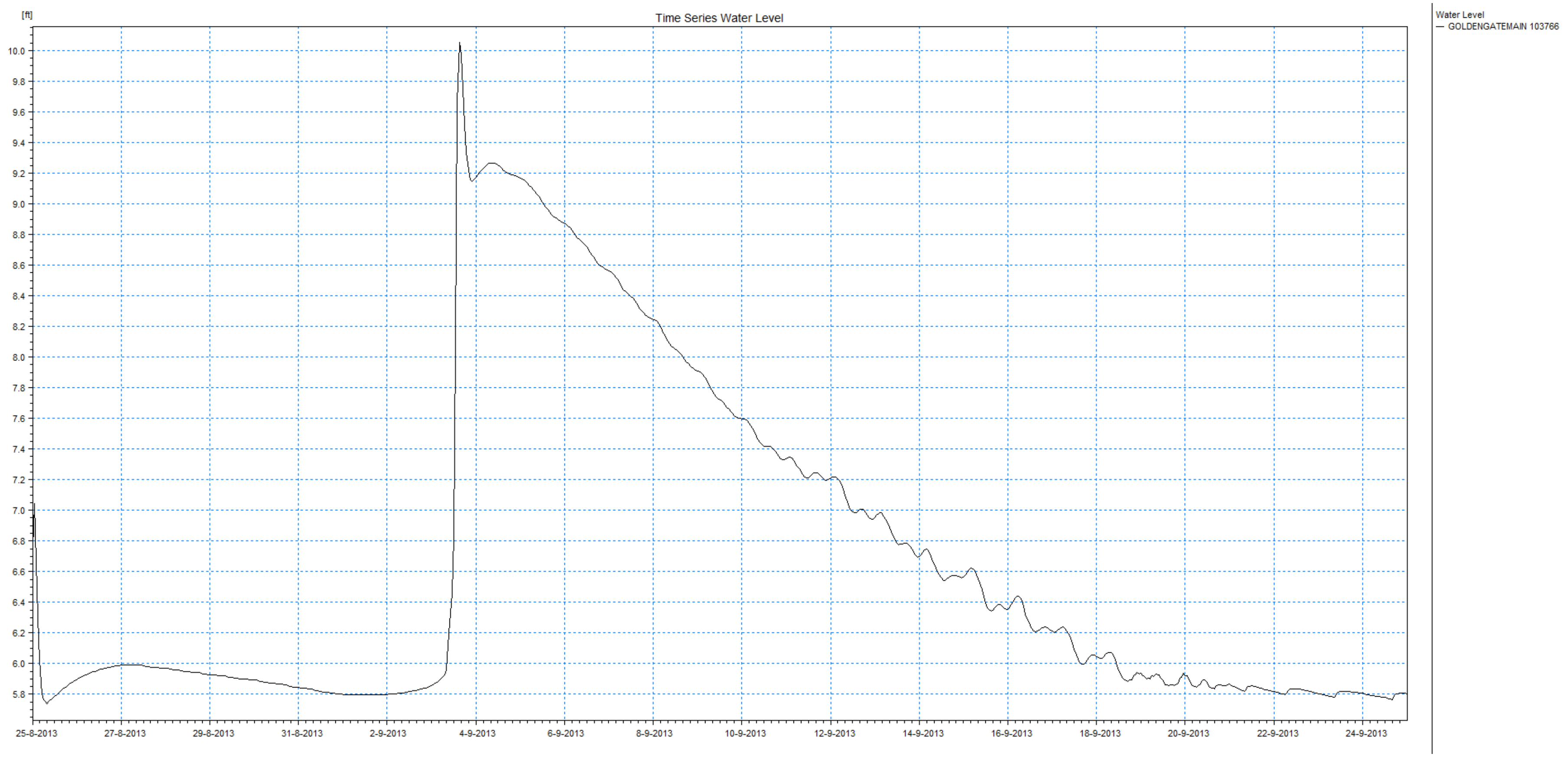


Figure 27. GG3 Headwater Stage (ft)

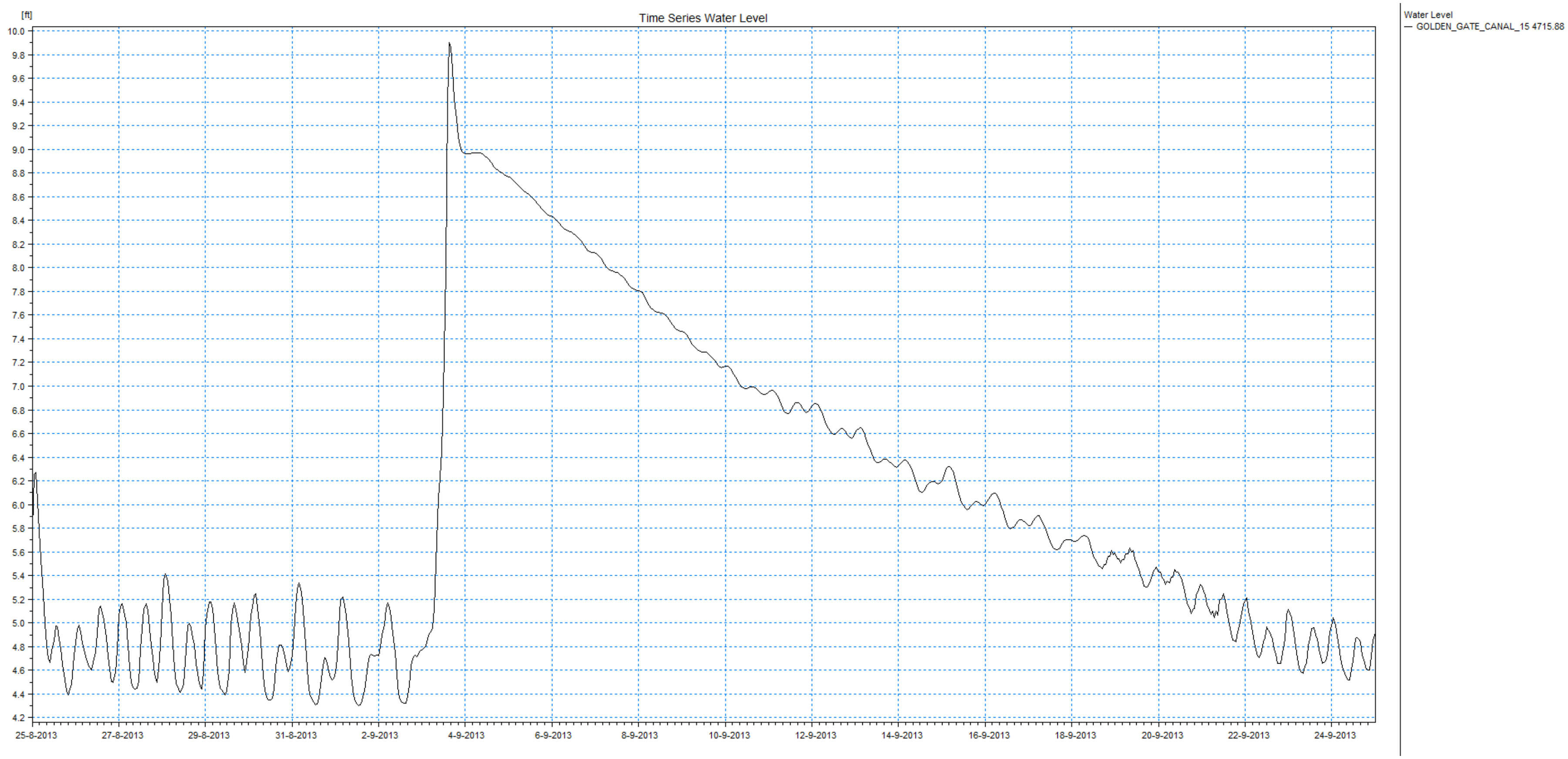


Figure 28. GG3 Tailwater Stage (ft)

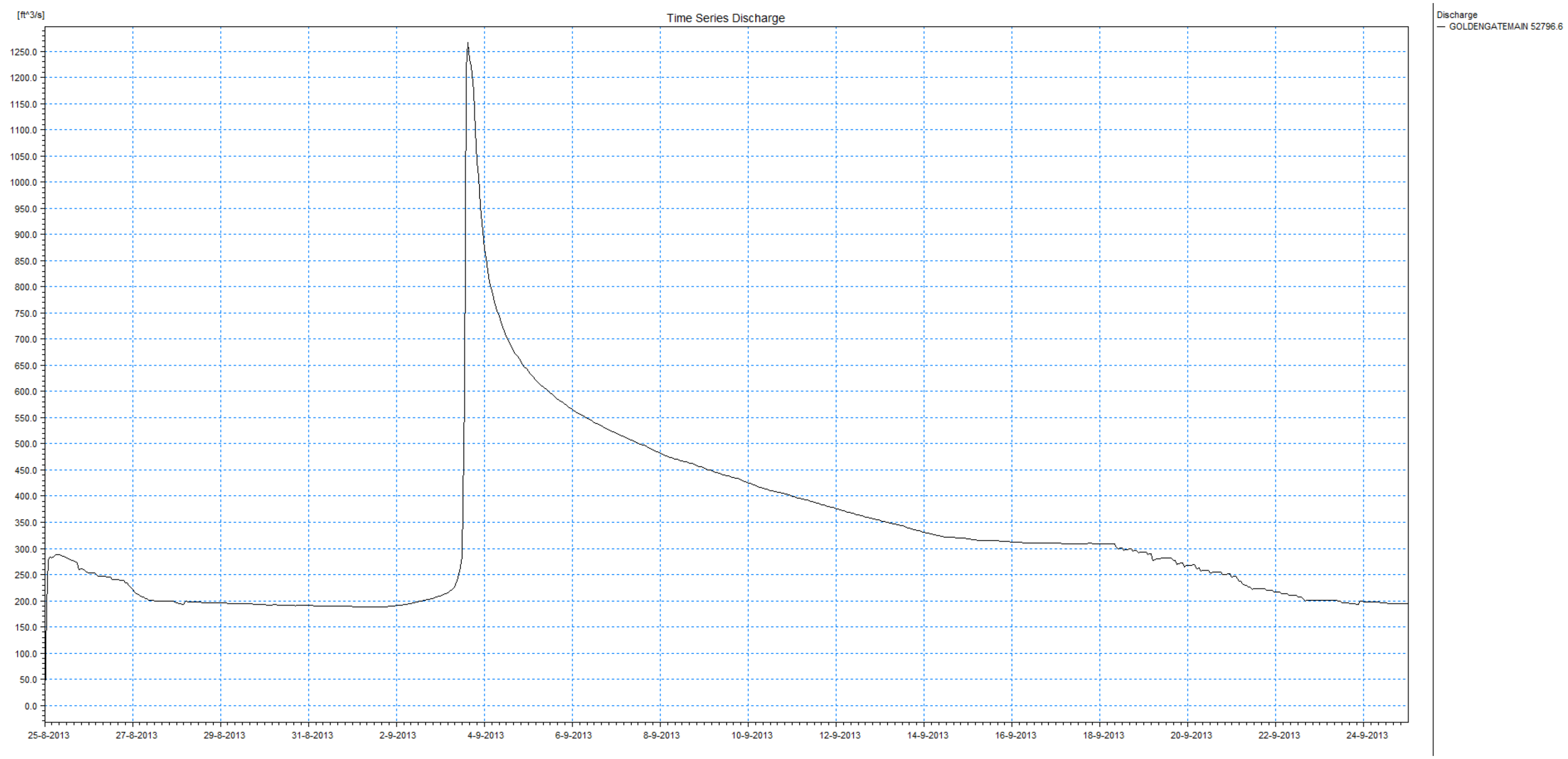


Figure 29. GG4 Discharge (cfs)

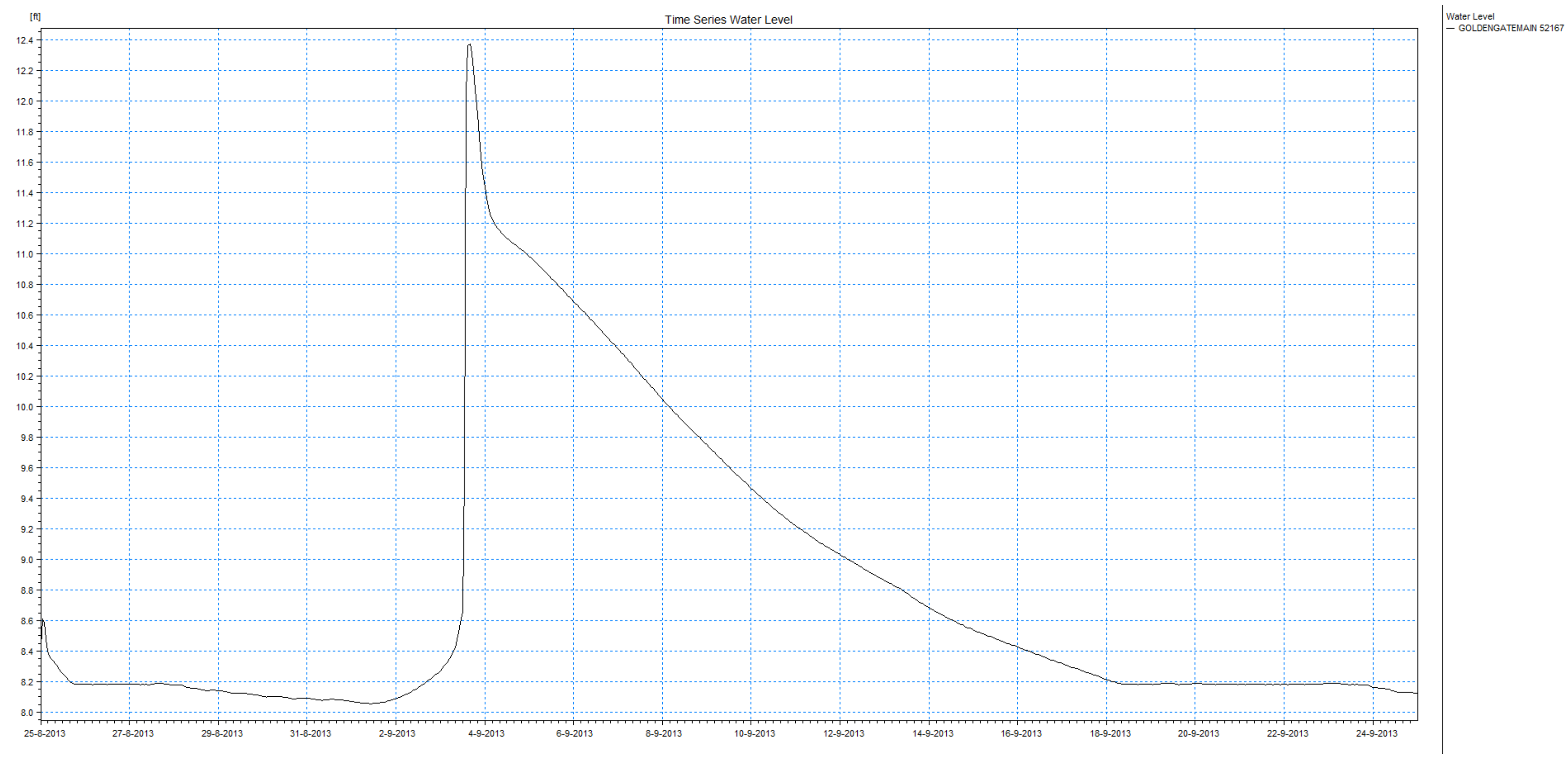


Figure 30. GG4 Headwater Stage (ft)

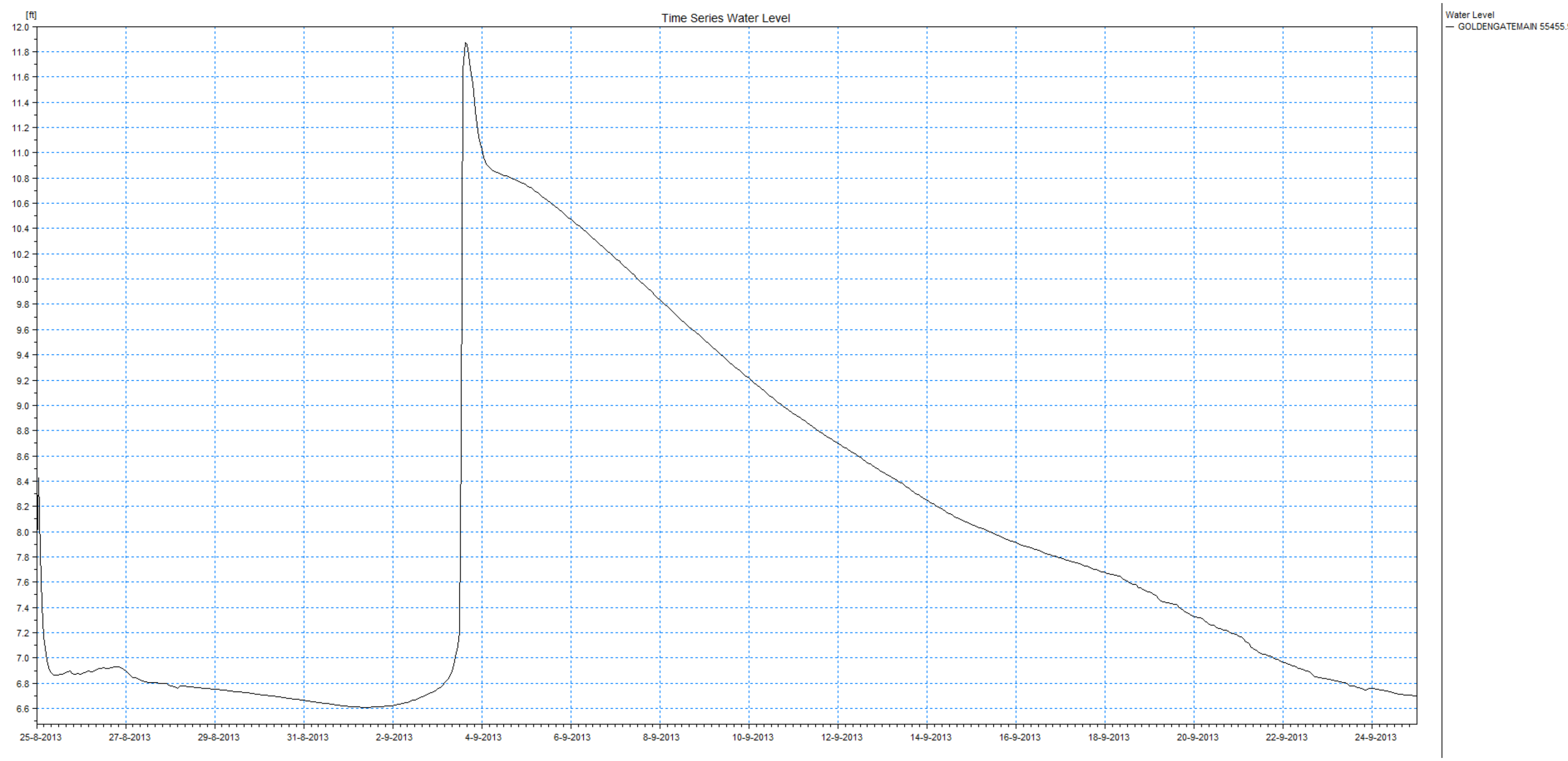


Figure 31. GG4 Tailwater Stage (ft)

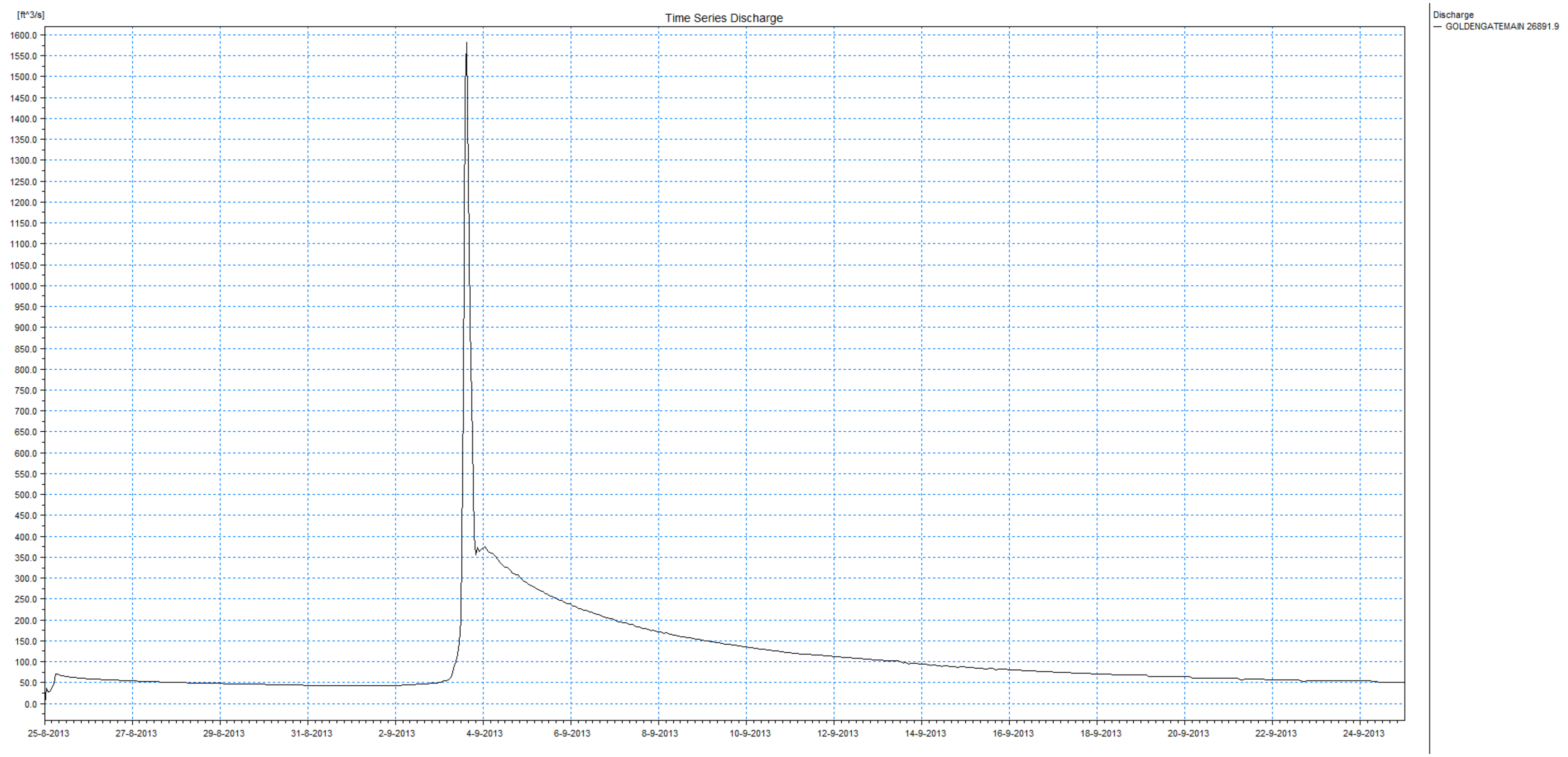


Figure 32. GG5 Discharge (cfs)

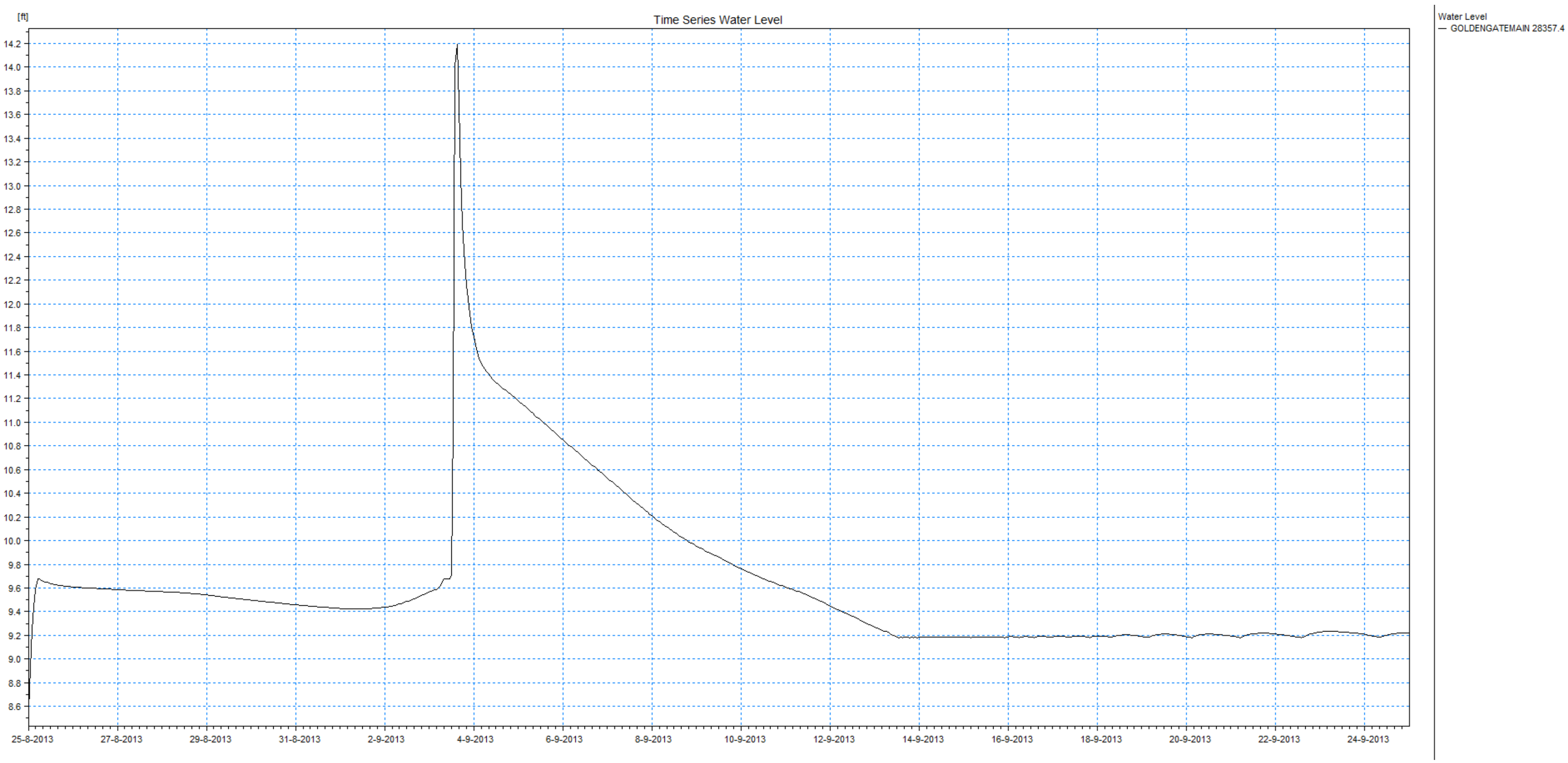


Figure 33. GG5 Headwater Stage (ft)

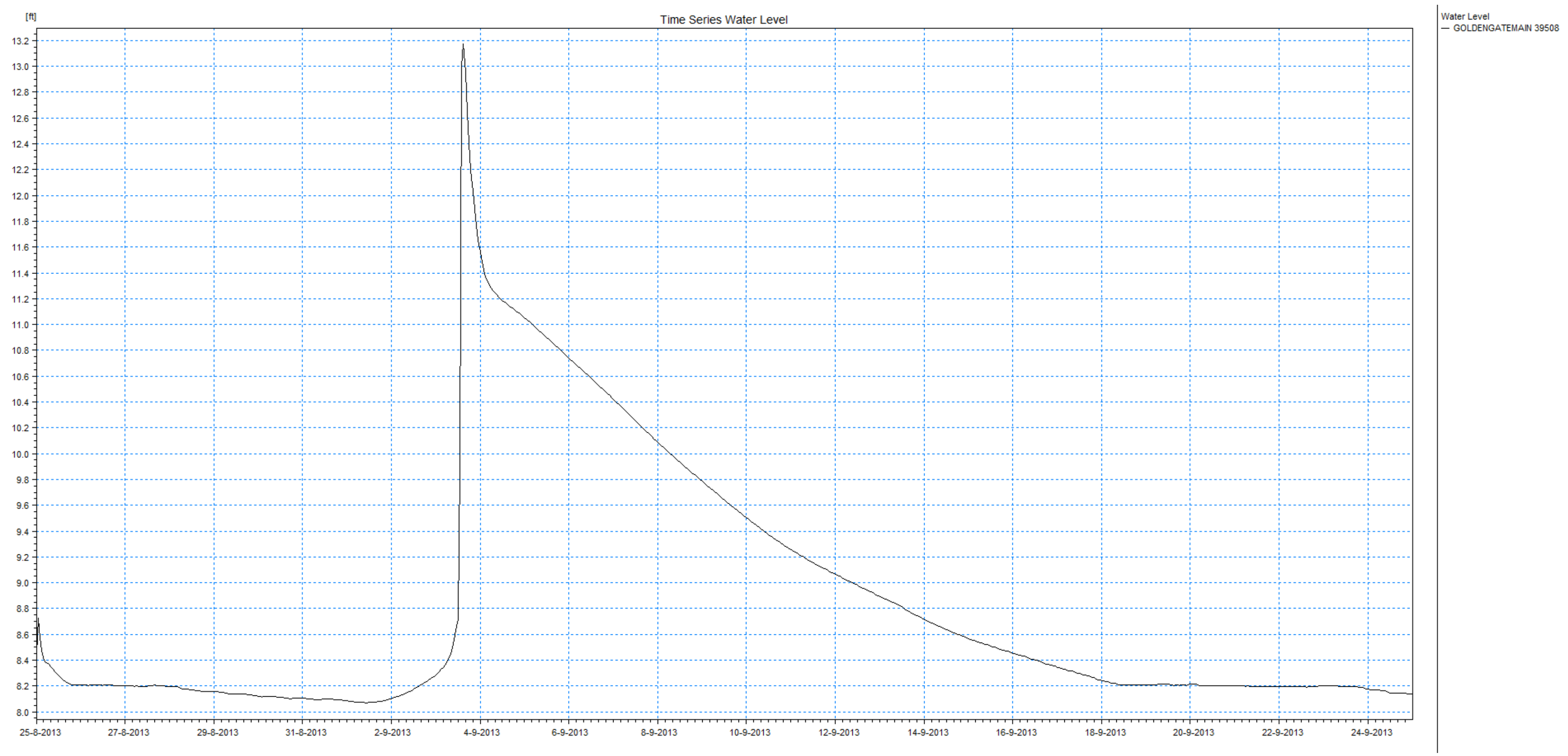


Figure 34. GG5 Tailwater Stage (ft)

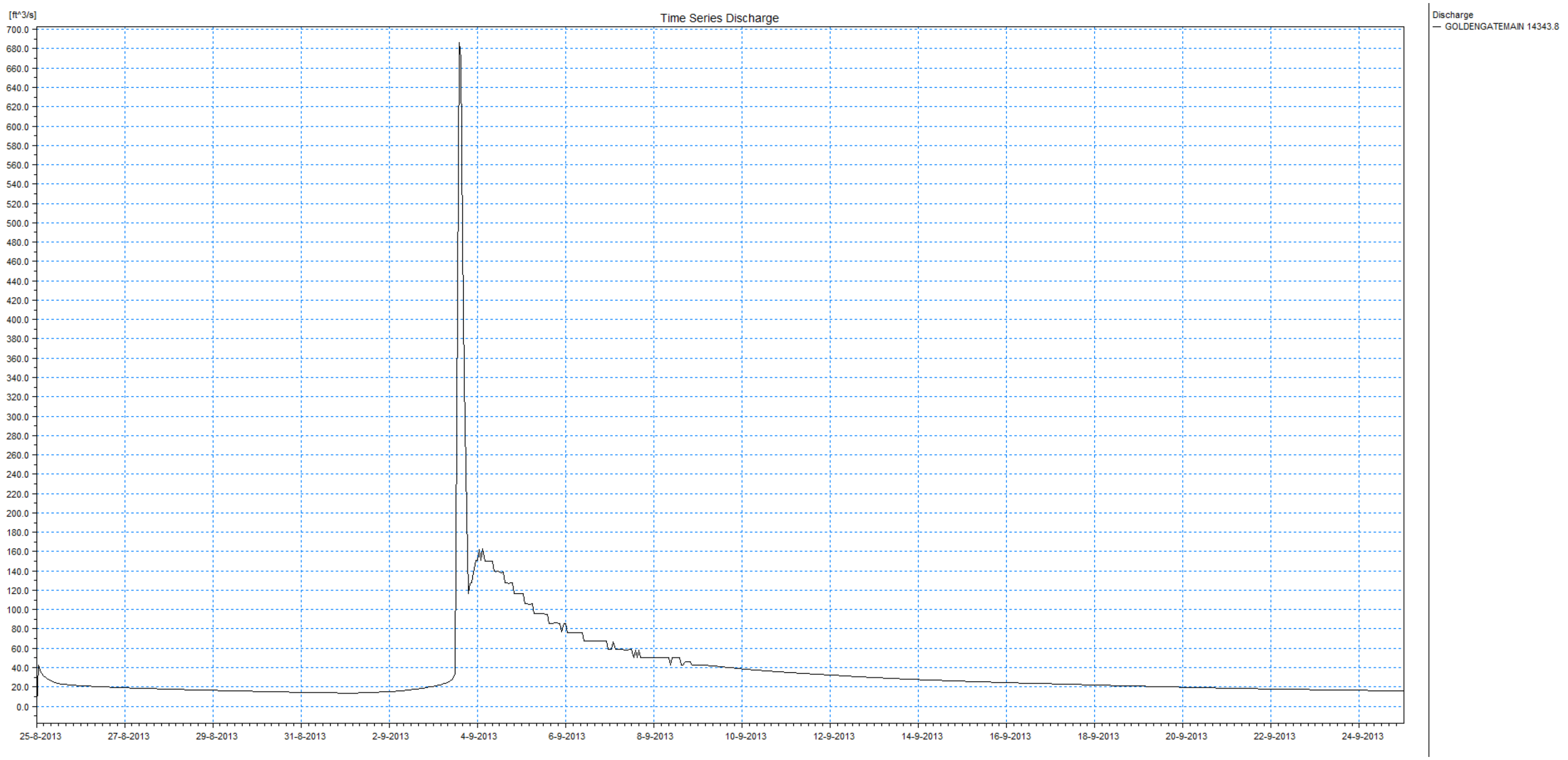


Figure 35. GG6 Discharge (cfs)

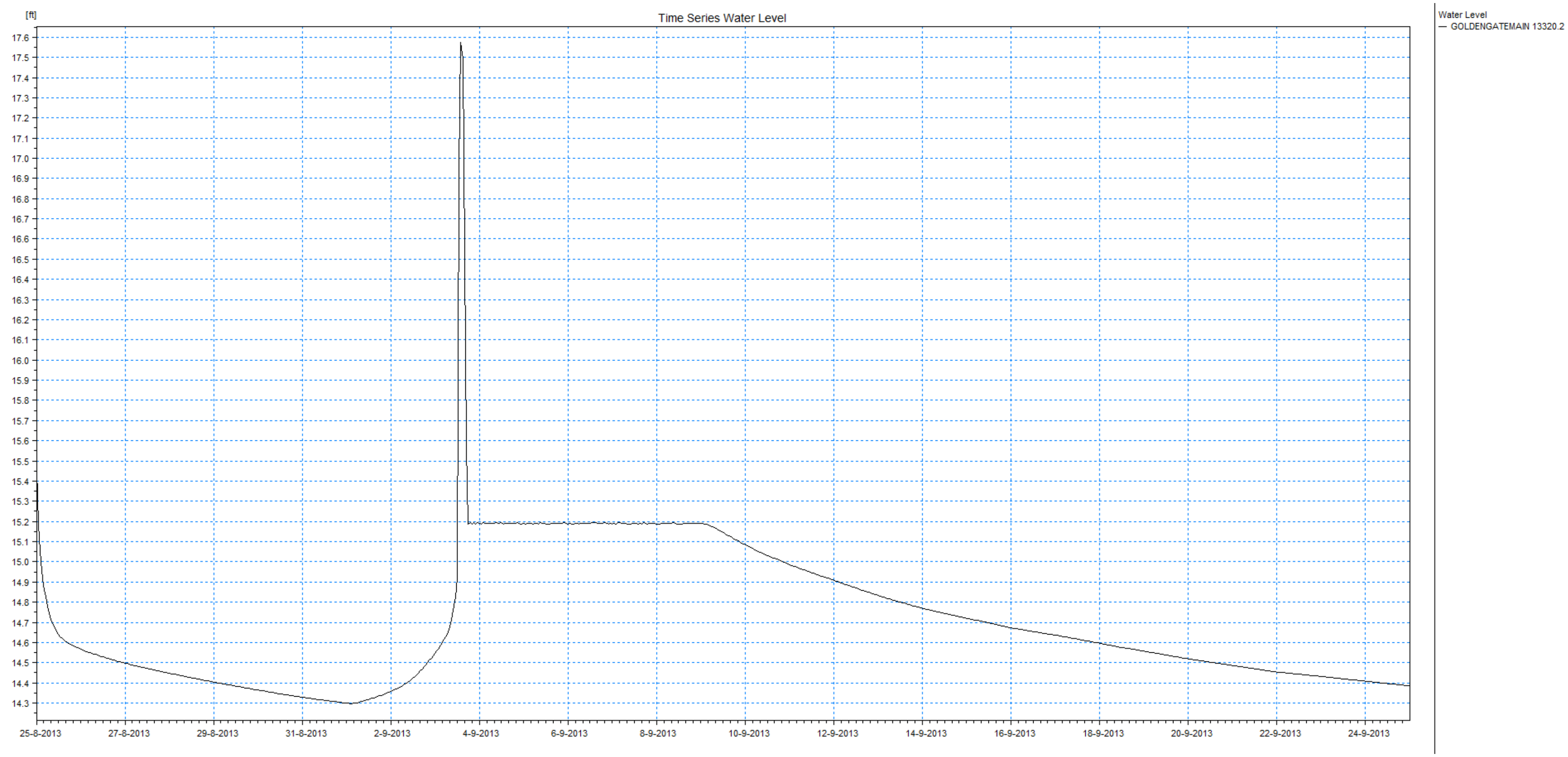


Figure 36. GG6 Headwater Stage (ft)

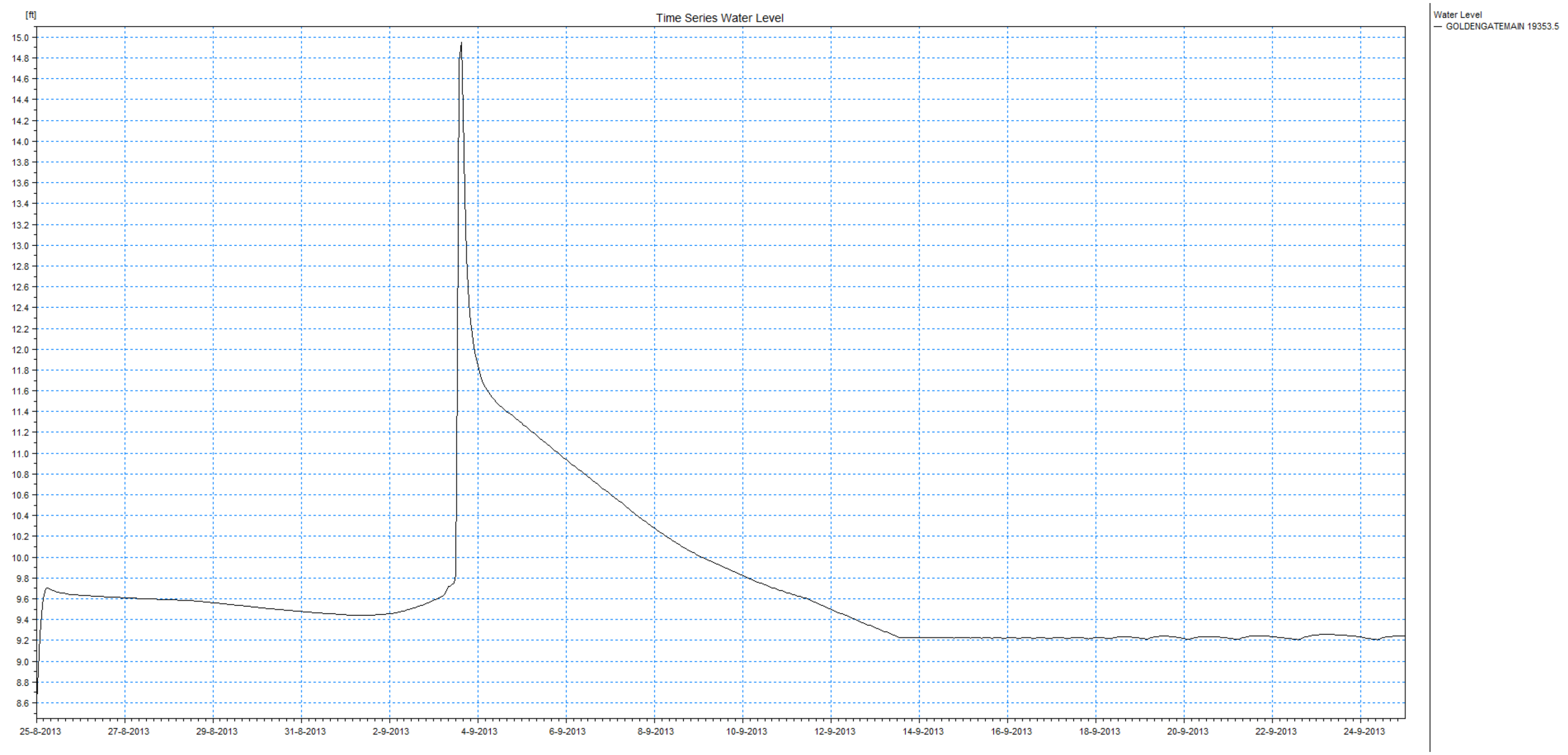


Figure 37. GG6 Tailwater Stage (ft)

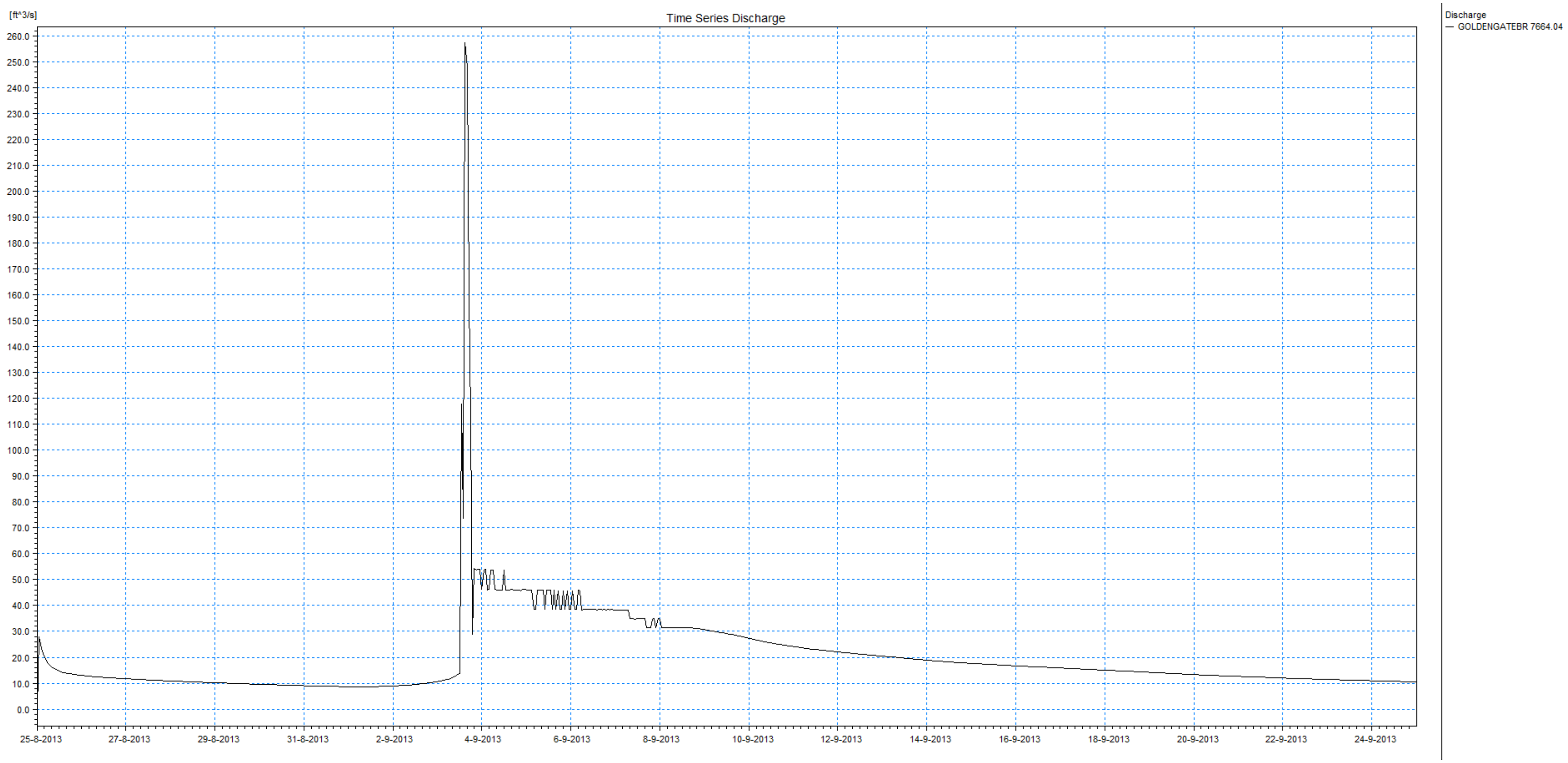


Figure 38. GG7 Discharge (cfs)

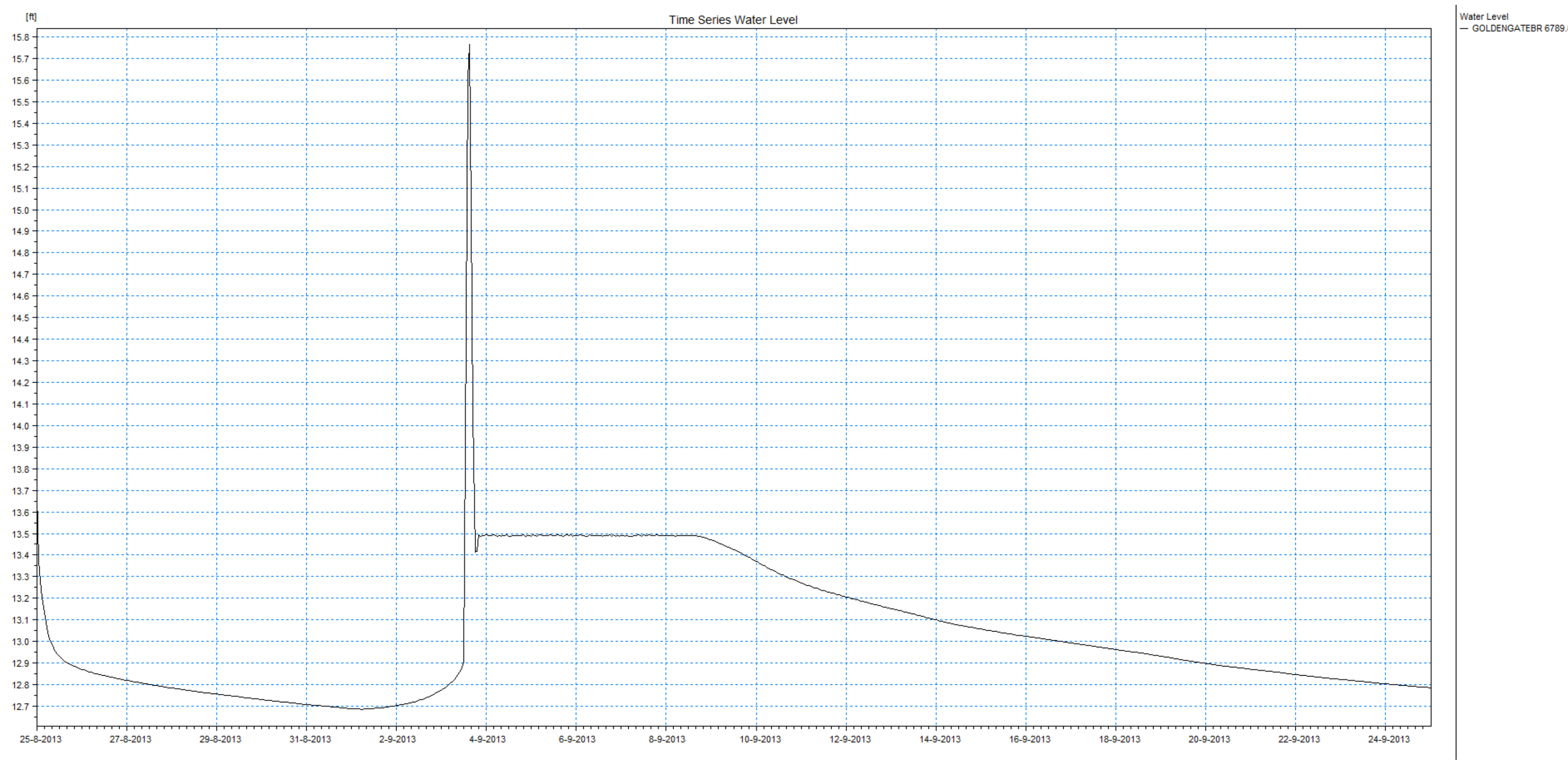


Figure 39. GG7 Headwater Stage (ft)

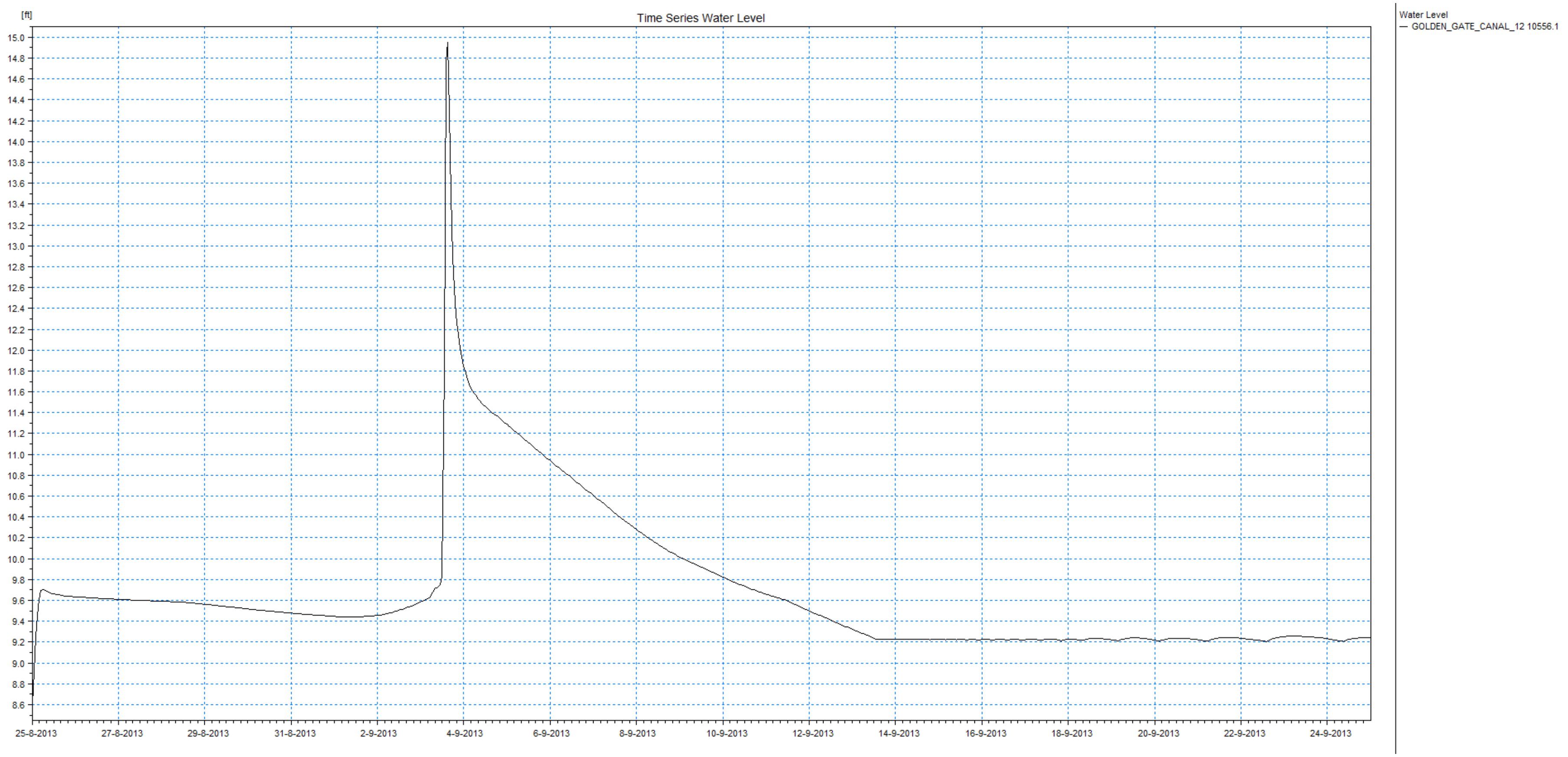


Figure 40. GG7 Tailwater Stage (ft)

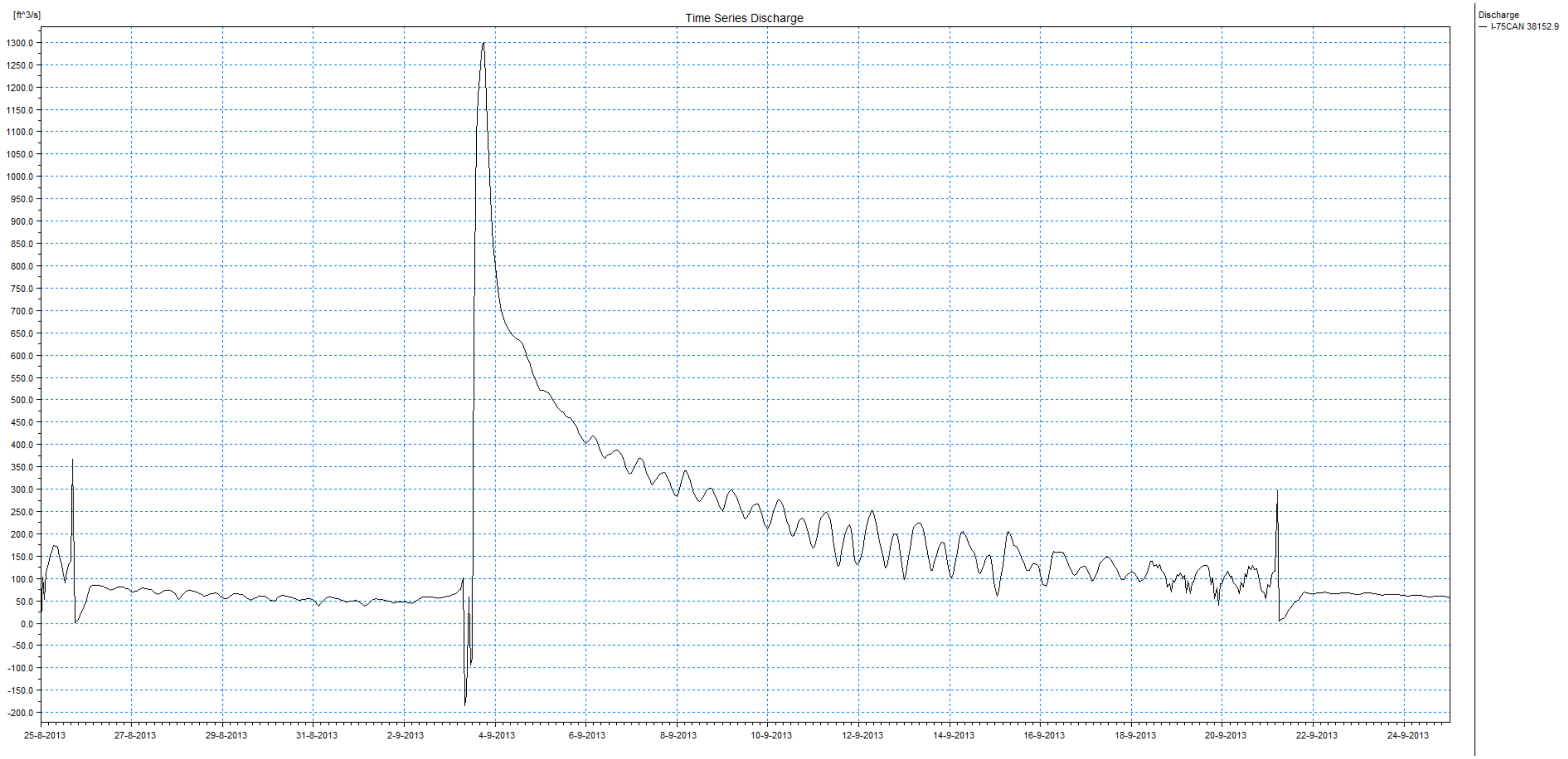


Figure 41. I75W1 Discharge (cfs)

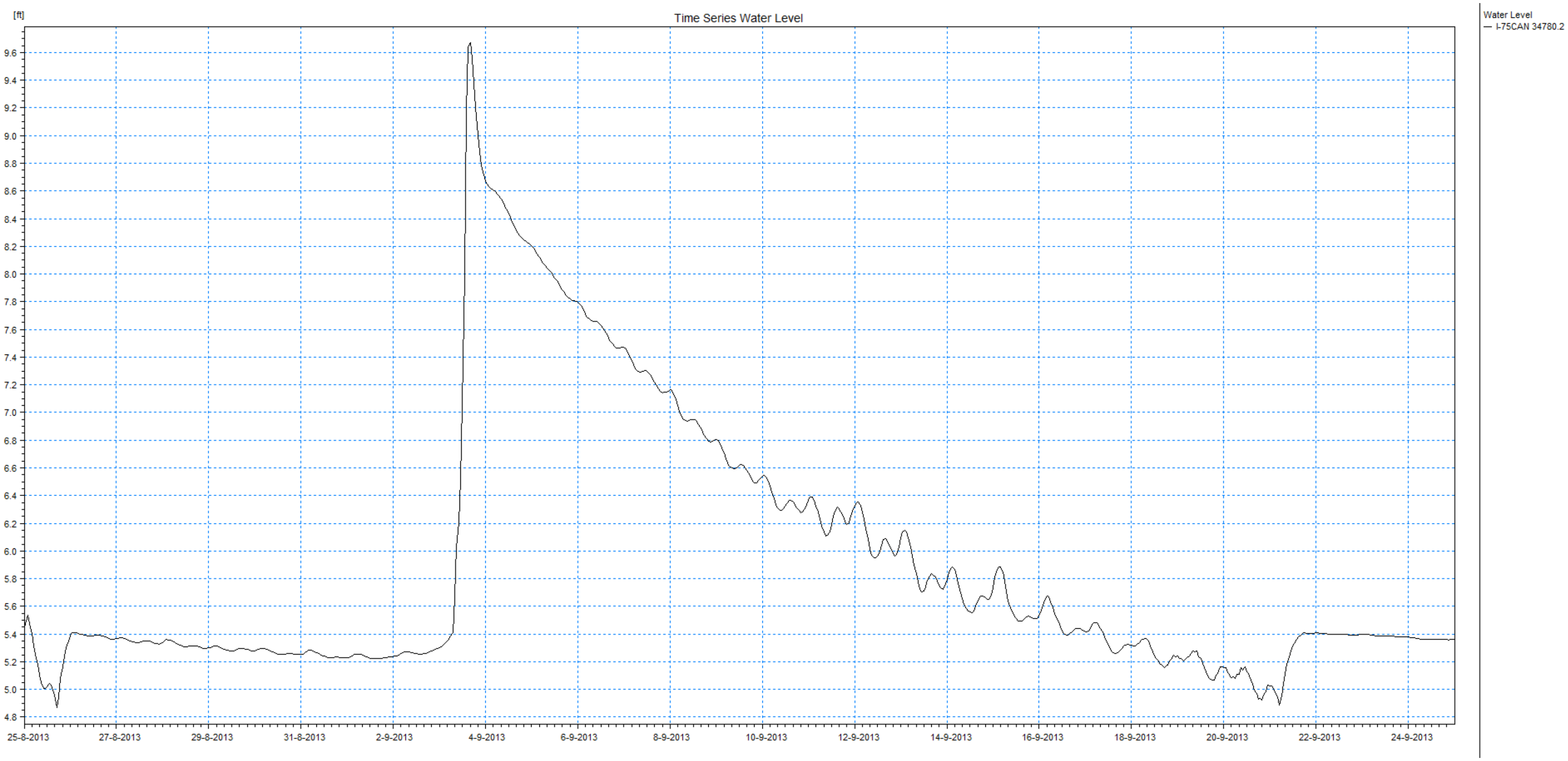


Figure 42. I75W1 Headwater Stage (ft)

Figure 43. I75W1 Tailwater Stage (ft)

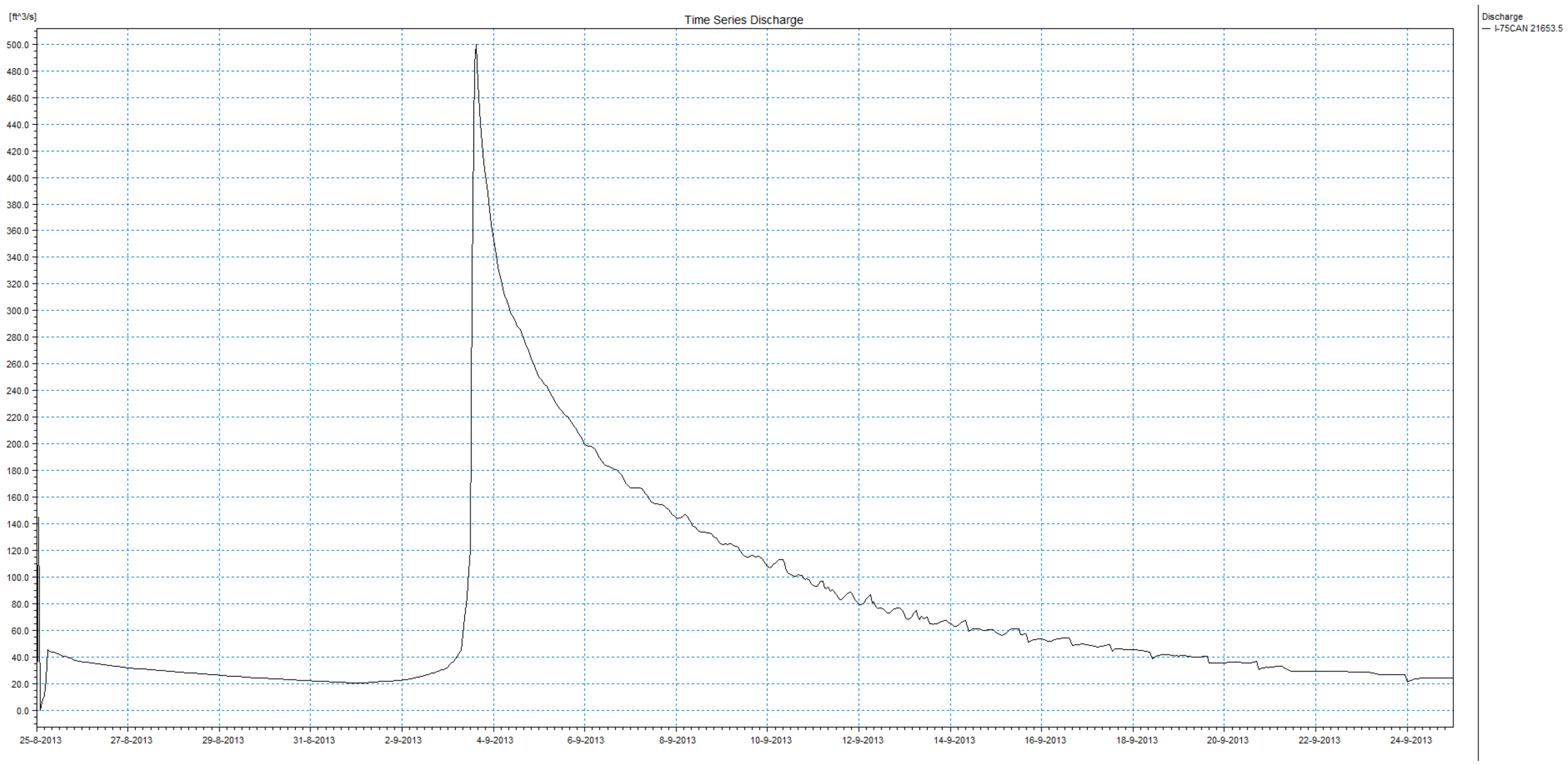


Figure 44. I75W2 Discharge (cfs)

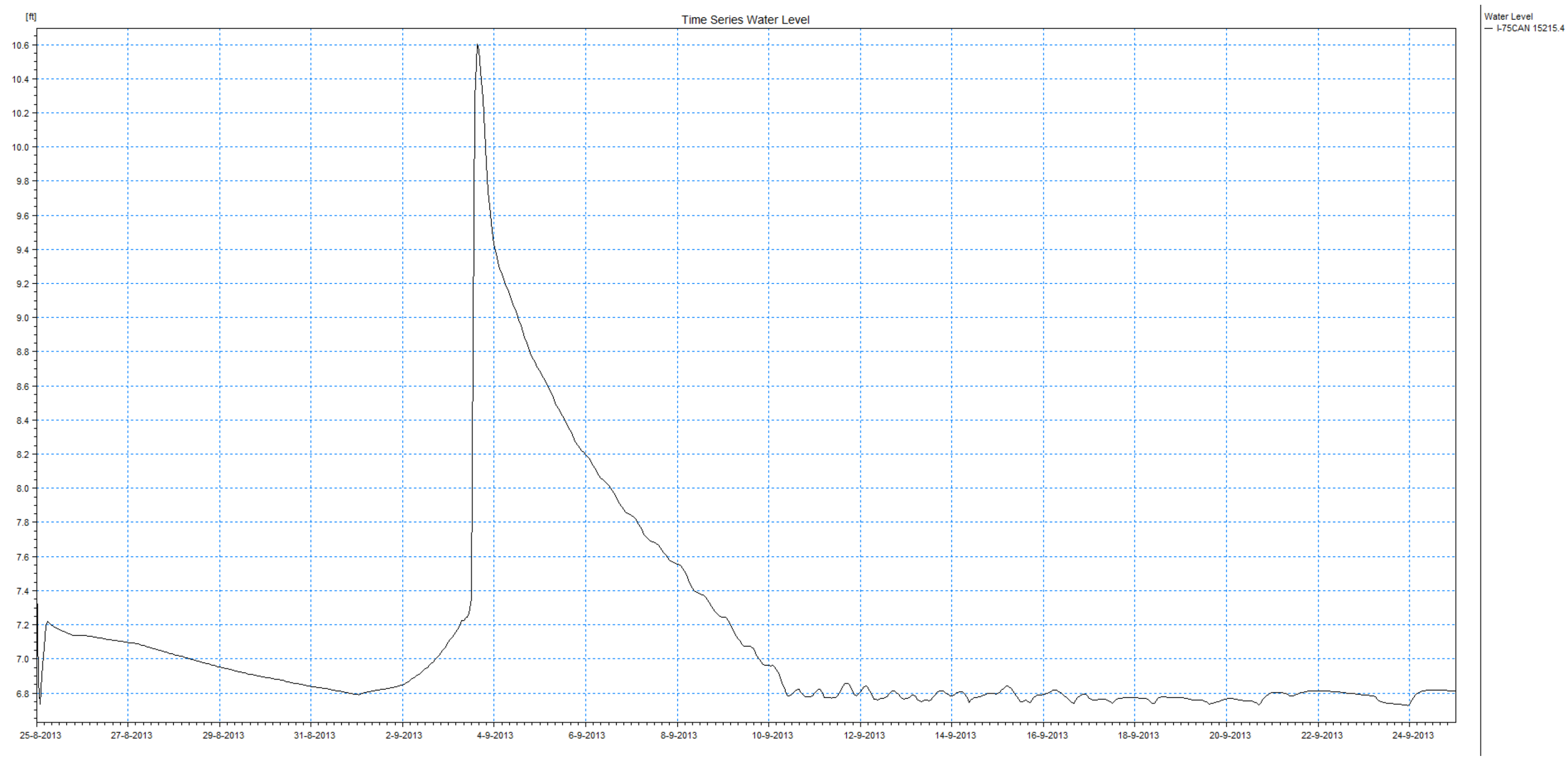


Figure 45. I75W2 Headwater Stage (ft)

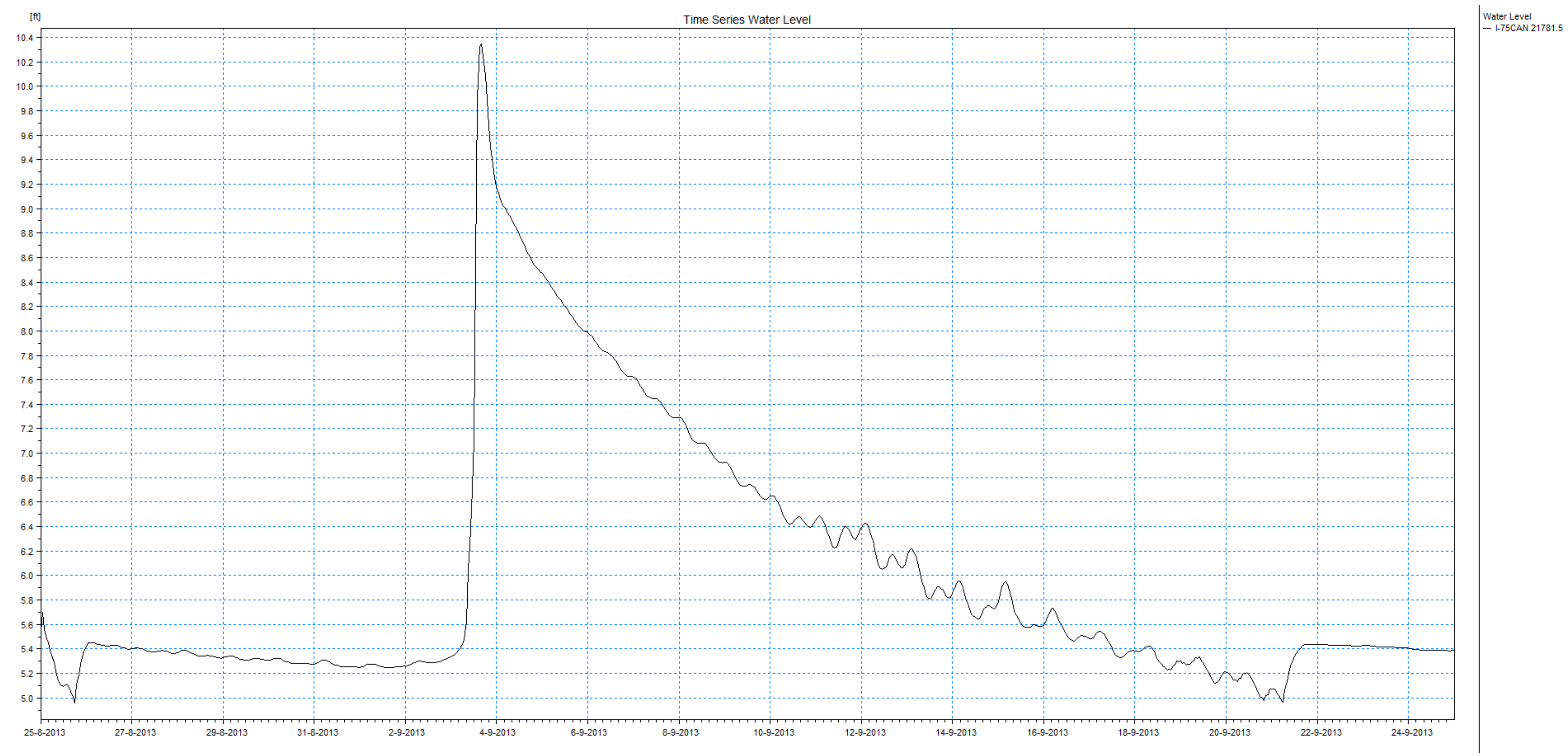


Figure 46. I75W2 Tailwater Stage (ft)

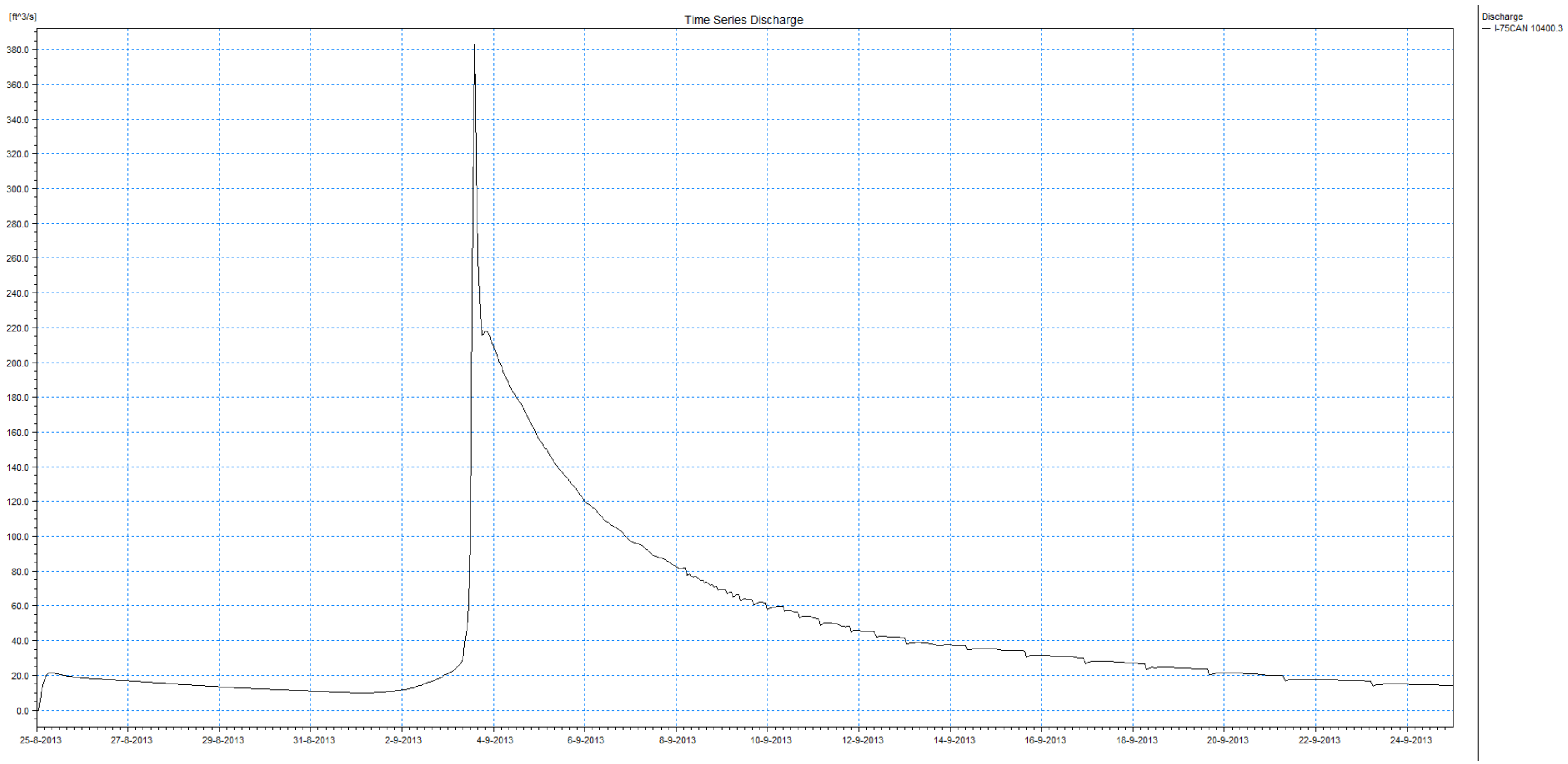


Figure 47. I75W3 Discharge (cfs)

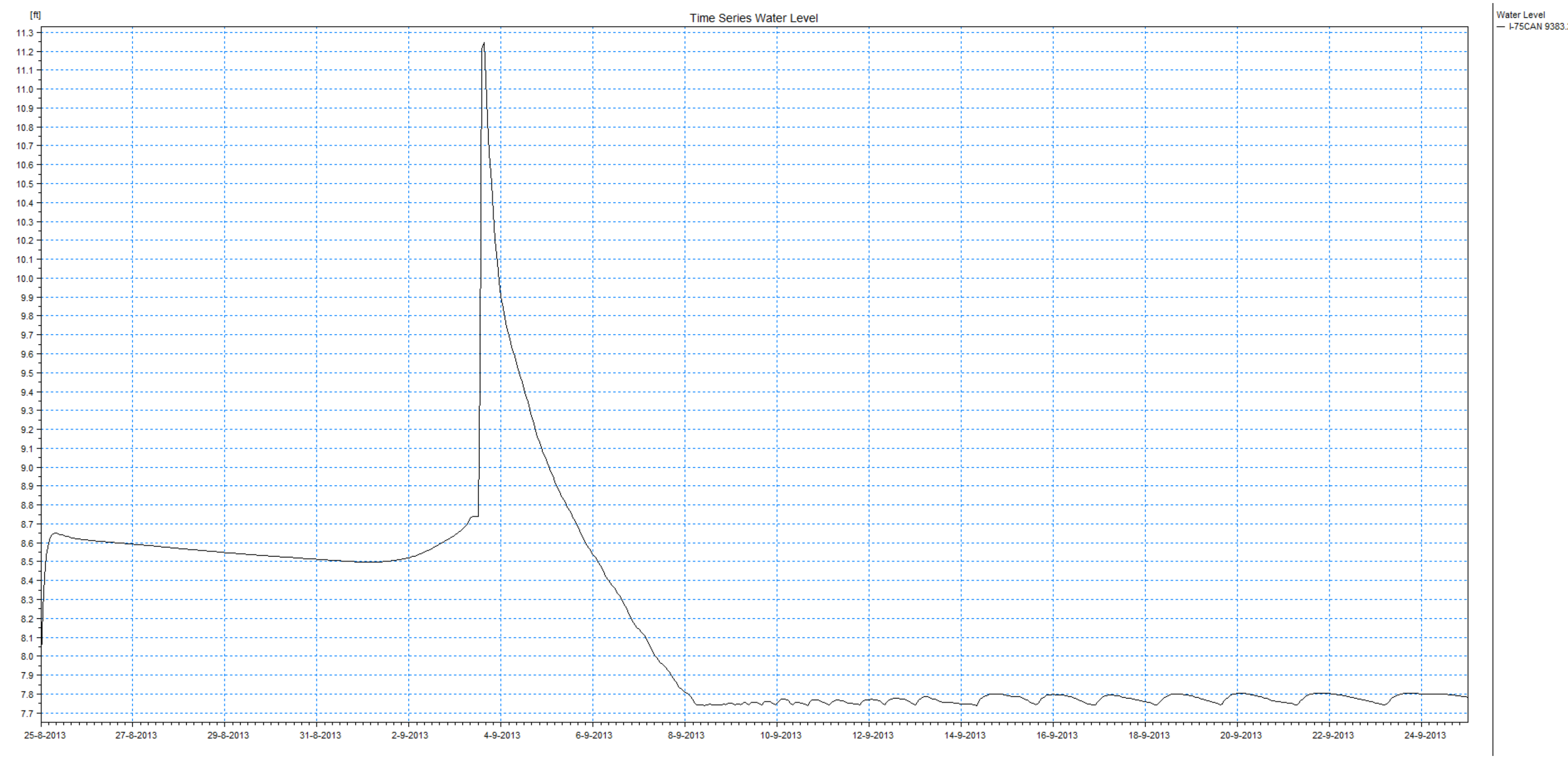


Figure 48. I75W3 Headwater Stage (ft)

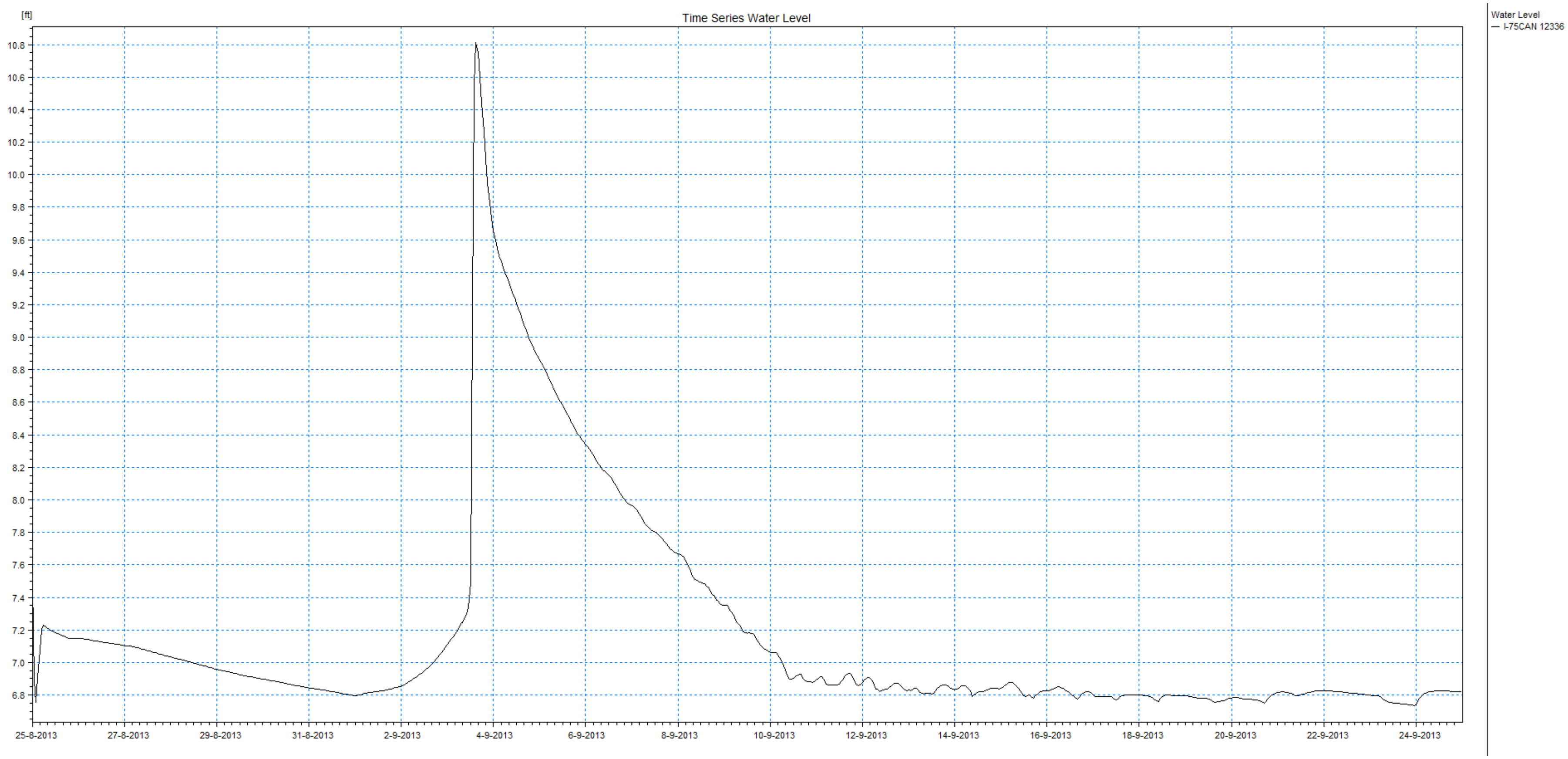
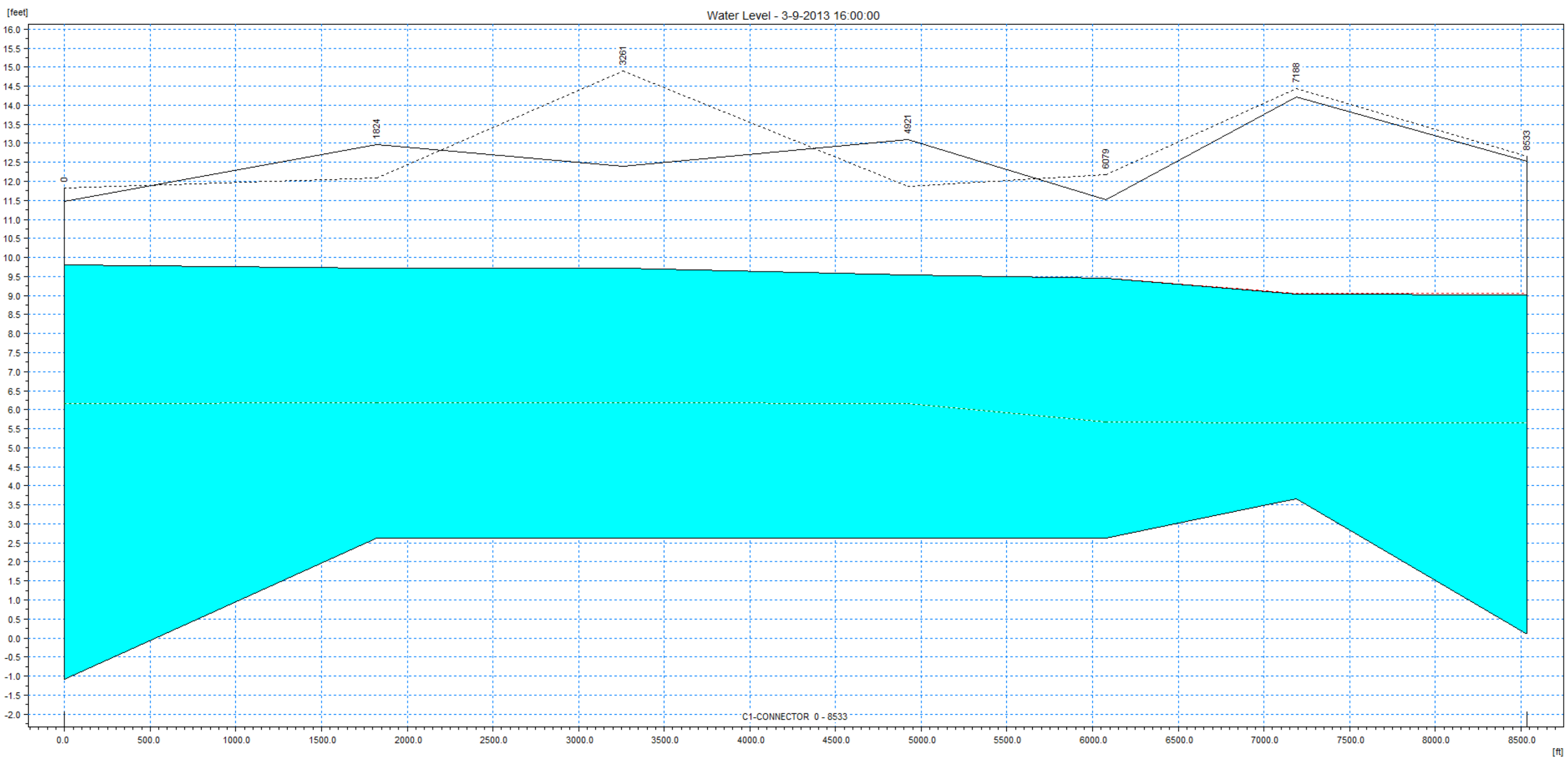


Figure 49. I75W3 Tailwater Stage (ft)

M – Supplemental Maximum Stage Profiles, Current Conditions

Part 1 5-year Current Conditions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[ft]
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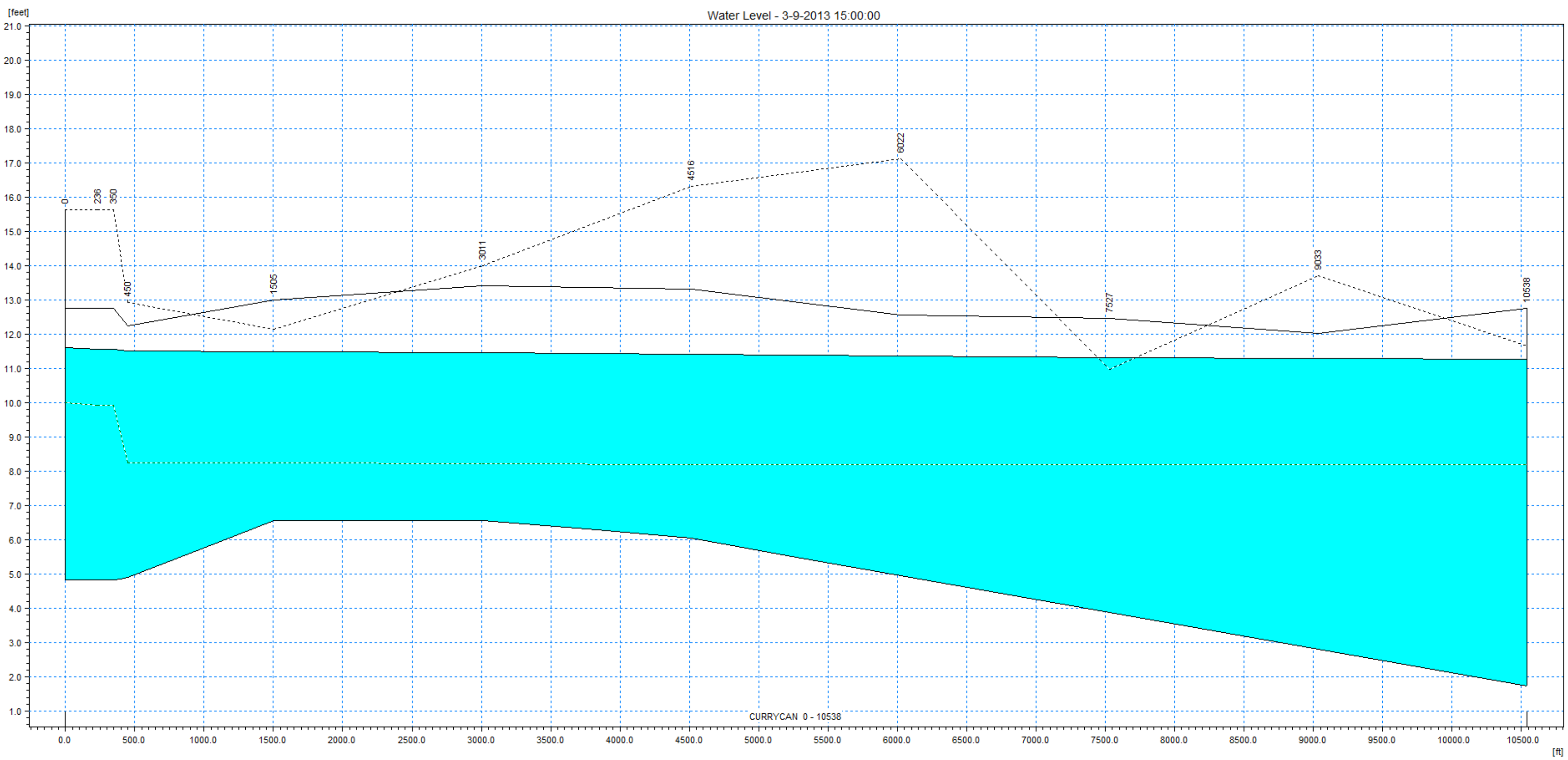
3261
4921
6079
7188
8533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3011
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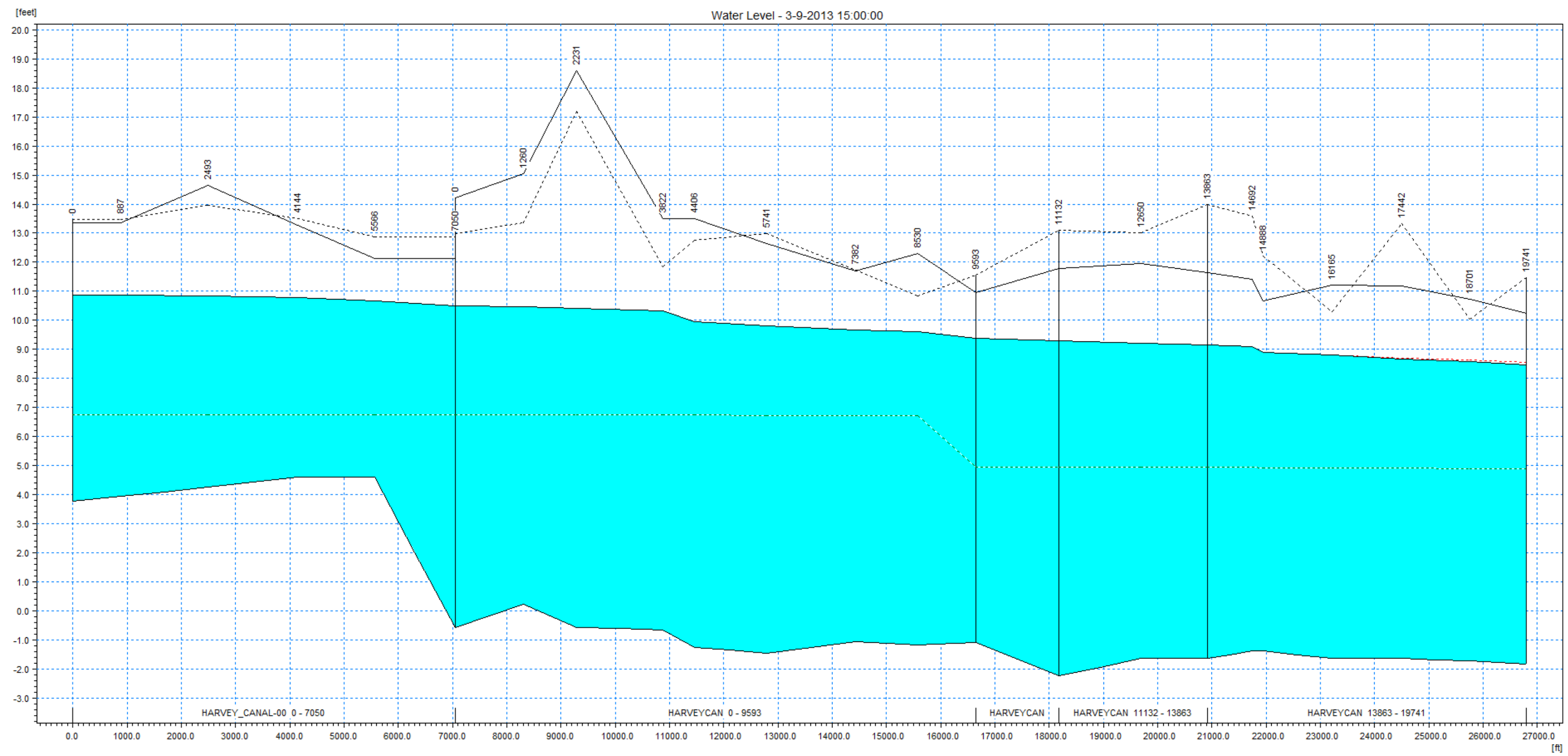
7527
9033
10538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[ft]
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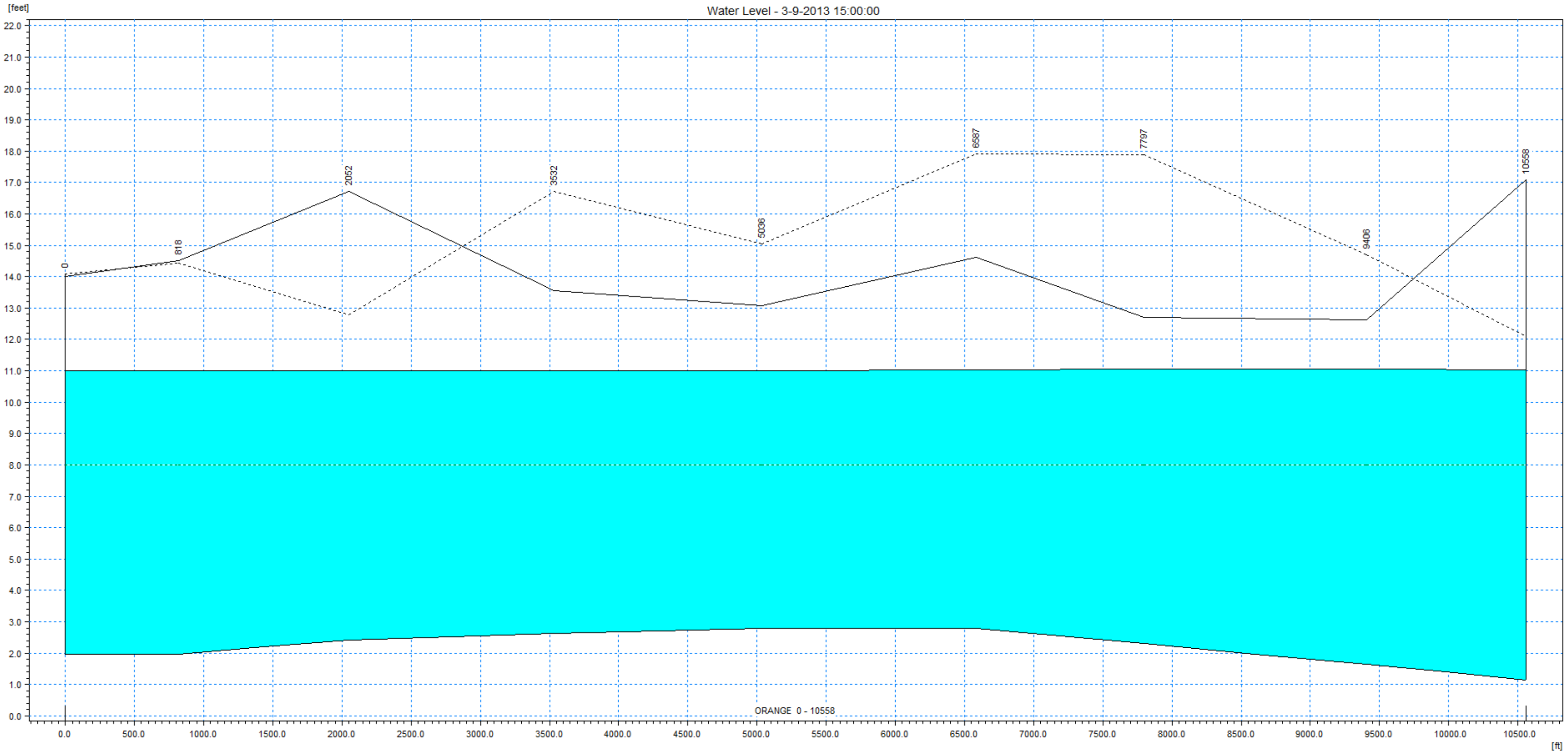
HARVEYCAN 0 - 9593
HARVEYCAN
HARVEYCAN 11132 - 13863
HARVEYCAN 13863 - 19741
0
887
2493
4144
5566
7050
0
1260
2231
3822
4406
5741
7382
8530
9593
11132
12650
13863
14692
14888
16165
17442
18701
19741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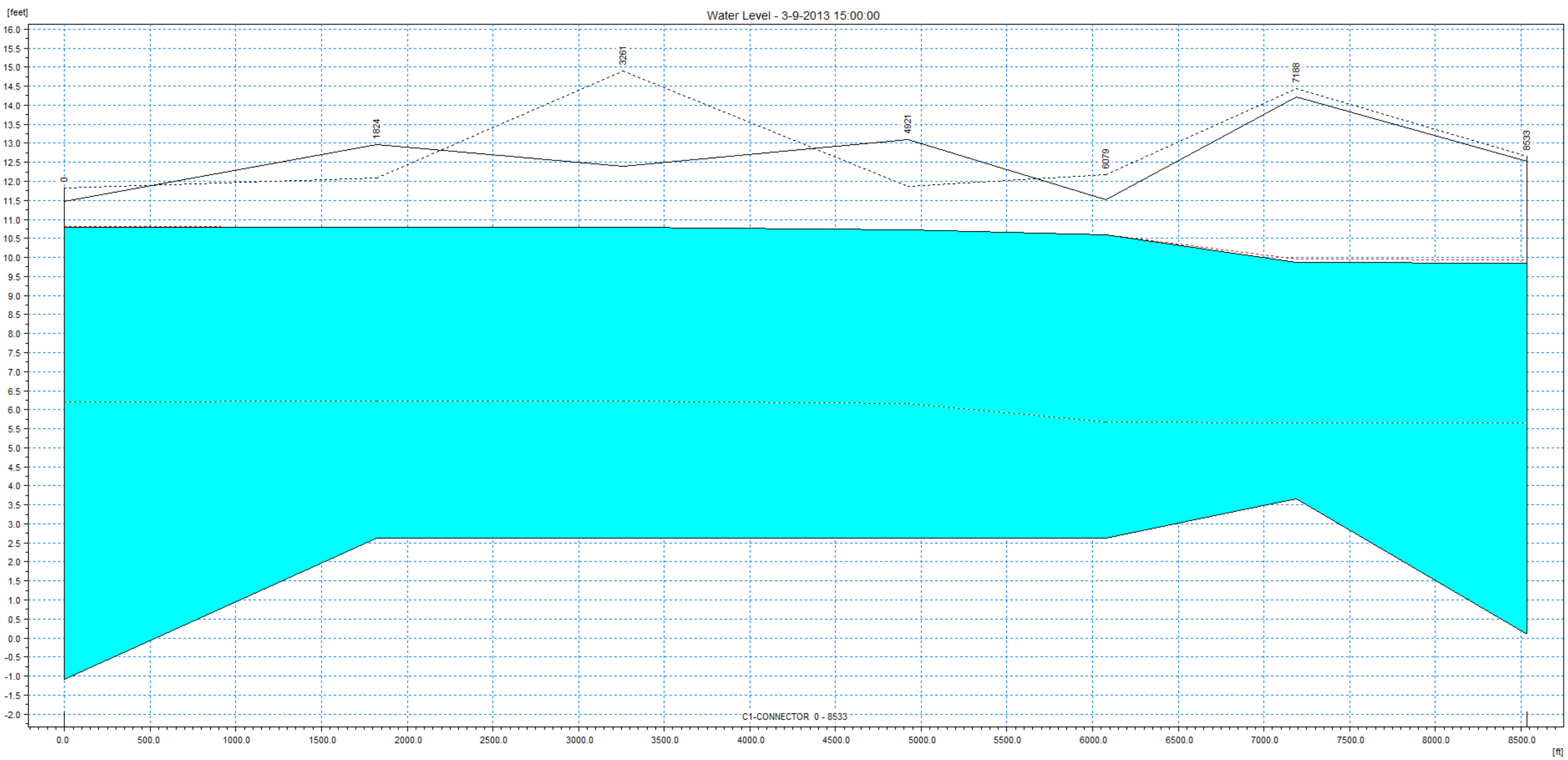
6587
7797
9406
10558

Part 2 10-year Current Conditions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3261
4921
6079
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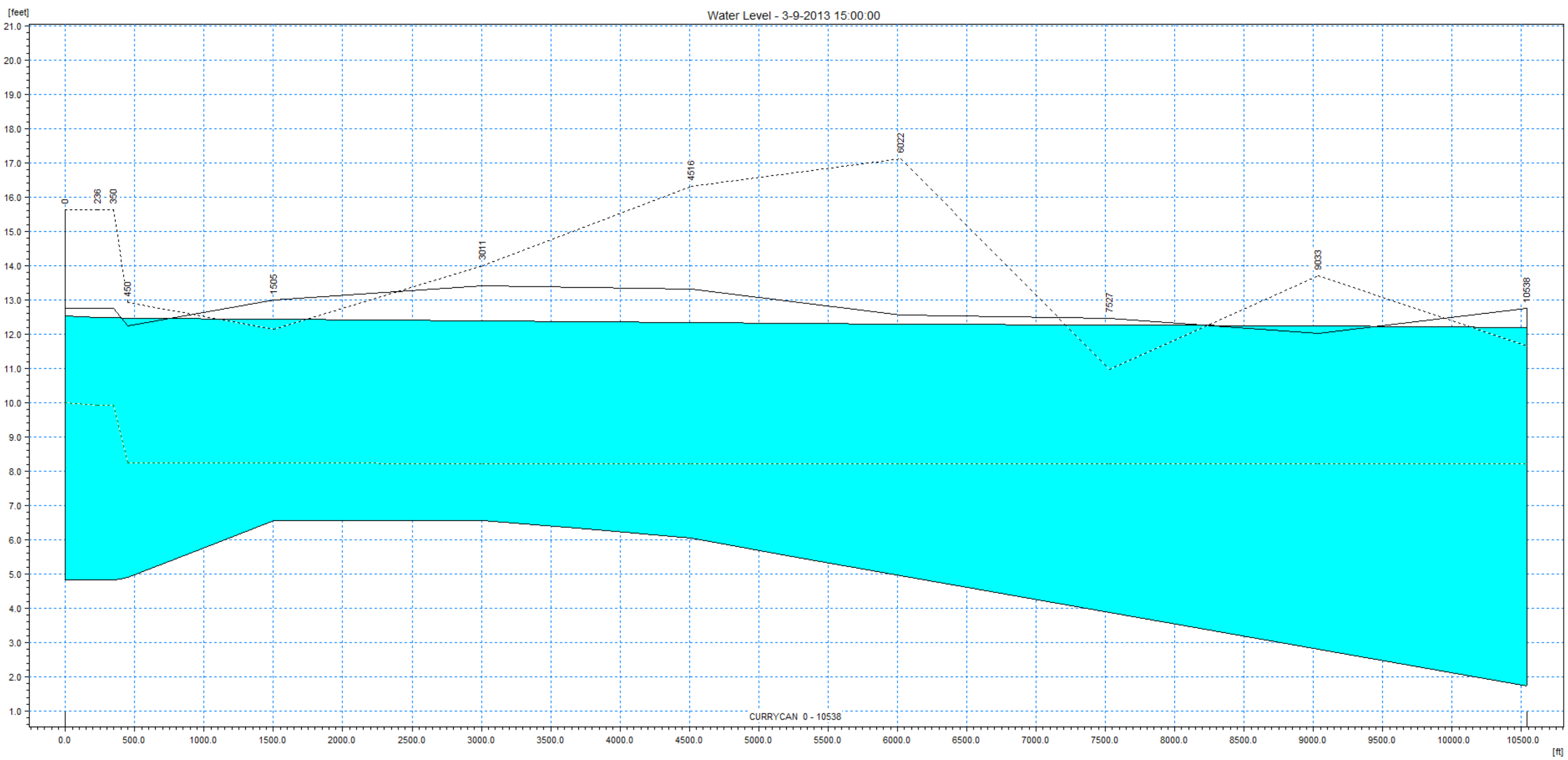
8533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18.0
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15.0
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3.0
2.0
1.0
0
236
350
450
1505
3011
4516
6022
7527
9033
10538
CURRYCAN 0 - 10538
0.0
500.0
1000.0
1500.0
2000.0
2500.0
3000.0
3500.0
4000.0
4500.0
5000.0
5500.0
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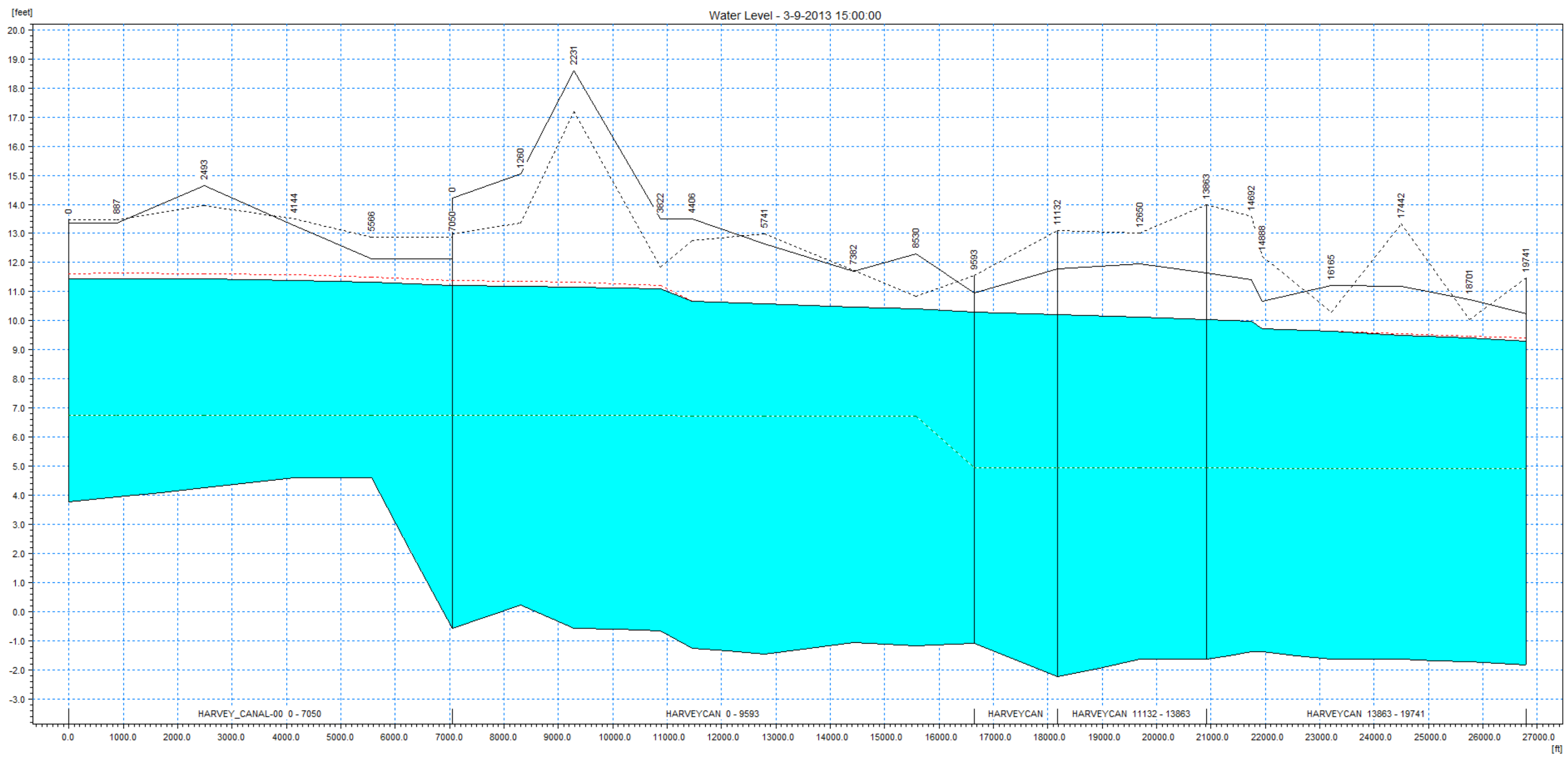
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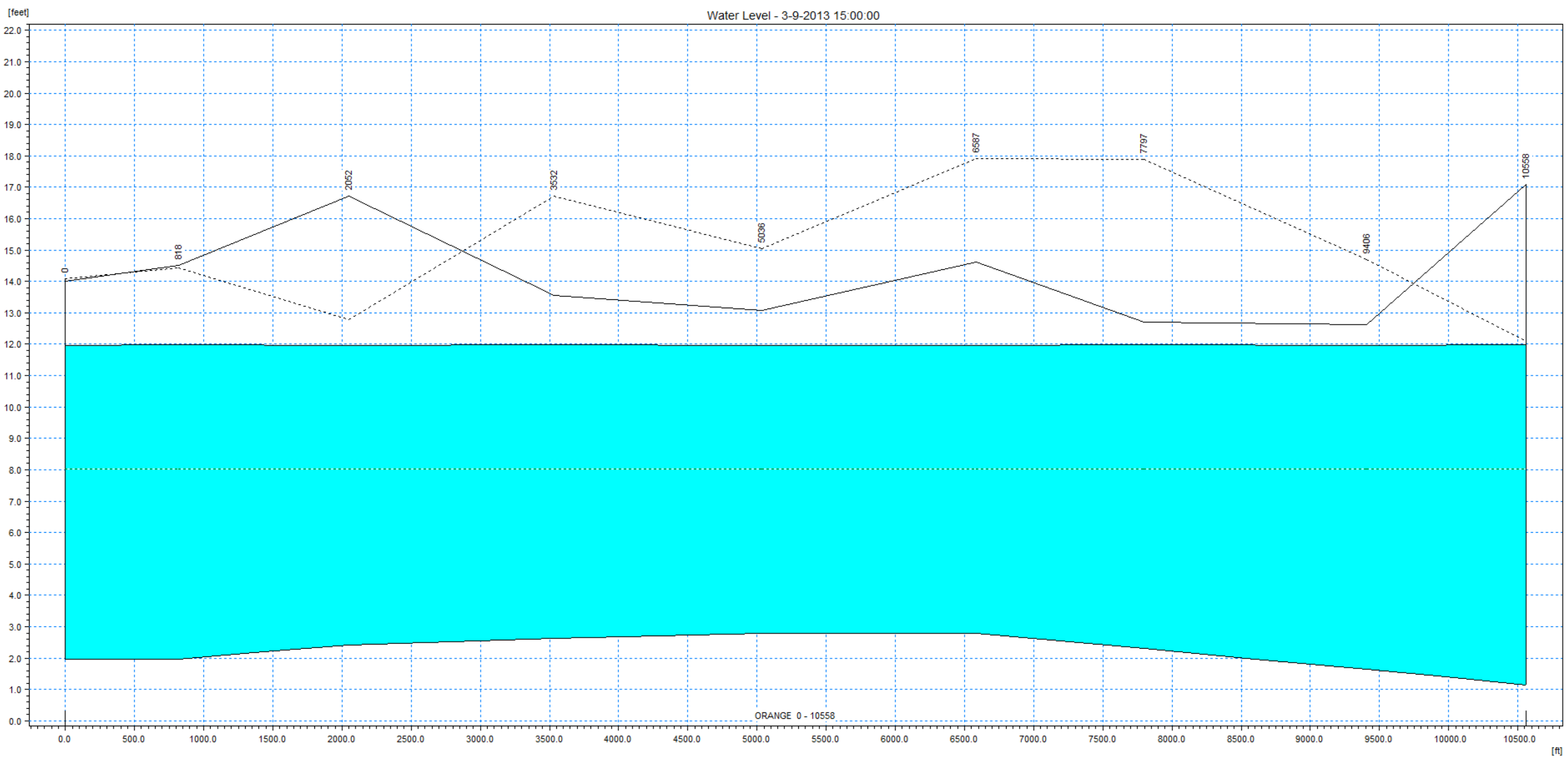
HARVEYCAN 0 - 9593
HARVEYCAN
HARVEYCAN 11132 - 13863
HARVEYCAN 13863 - 19741
[ft]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8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
0
818
2052
3532
5036
6587
7797
9406
10558
ORANGE 0 - 10558
0.0
500.0
1000.0
1500.0
2000.0
2500.0
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3500.0
4000.0
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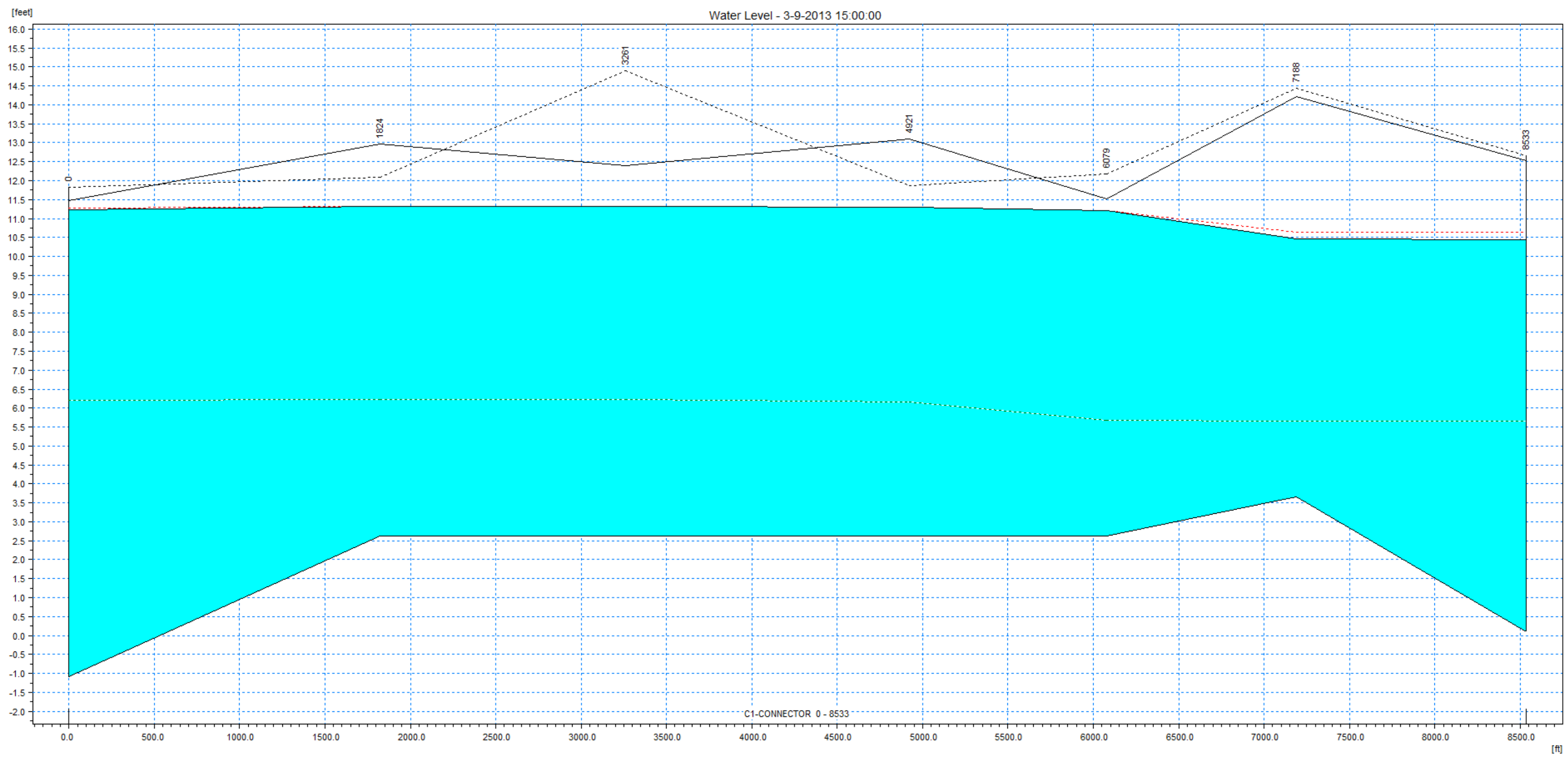
Part 3 25-year Current Conditions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6.5
6.0
5.5
5.0
4.5
4.0
3.5
3.0
2.5
2.0
1.5
1.0
0.5
0.0
-0.5
-1.0
-1.5
-2.0
0
1824
3261
4921
6079
7188
8533
C1-CONNECTOR 0 - 8533
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1500.0
2000.0
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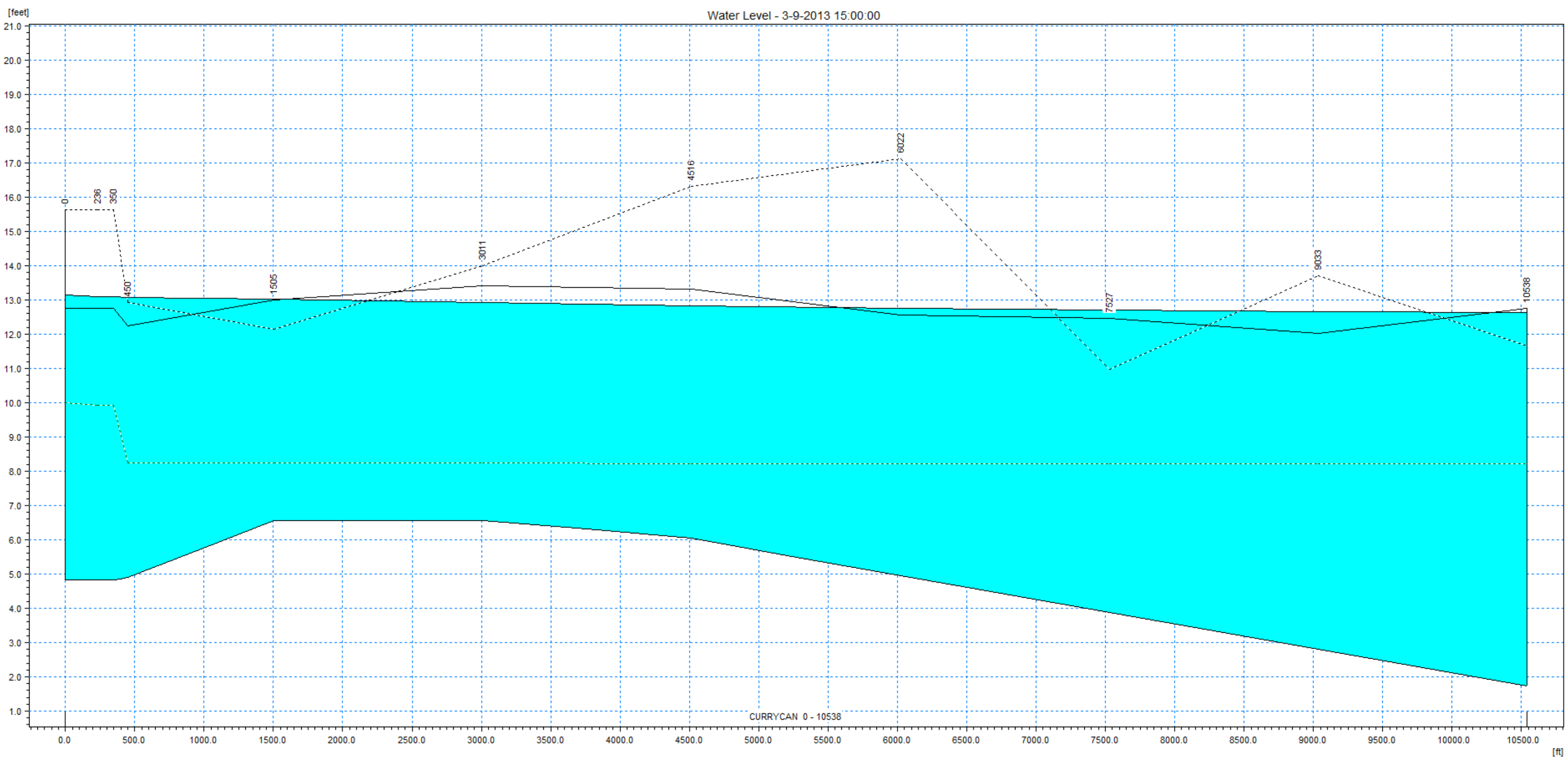
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350
450
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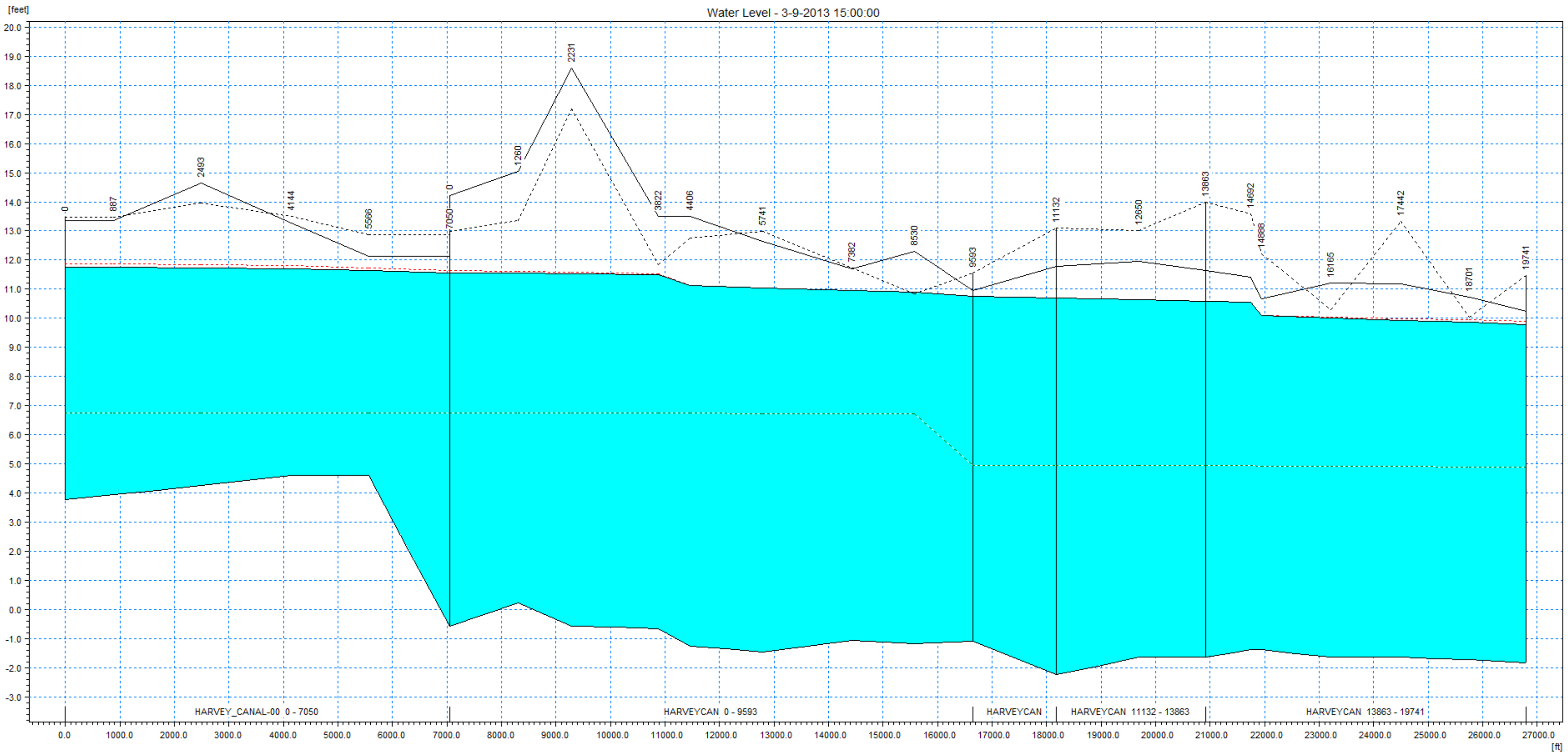
10538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[ft]
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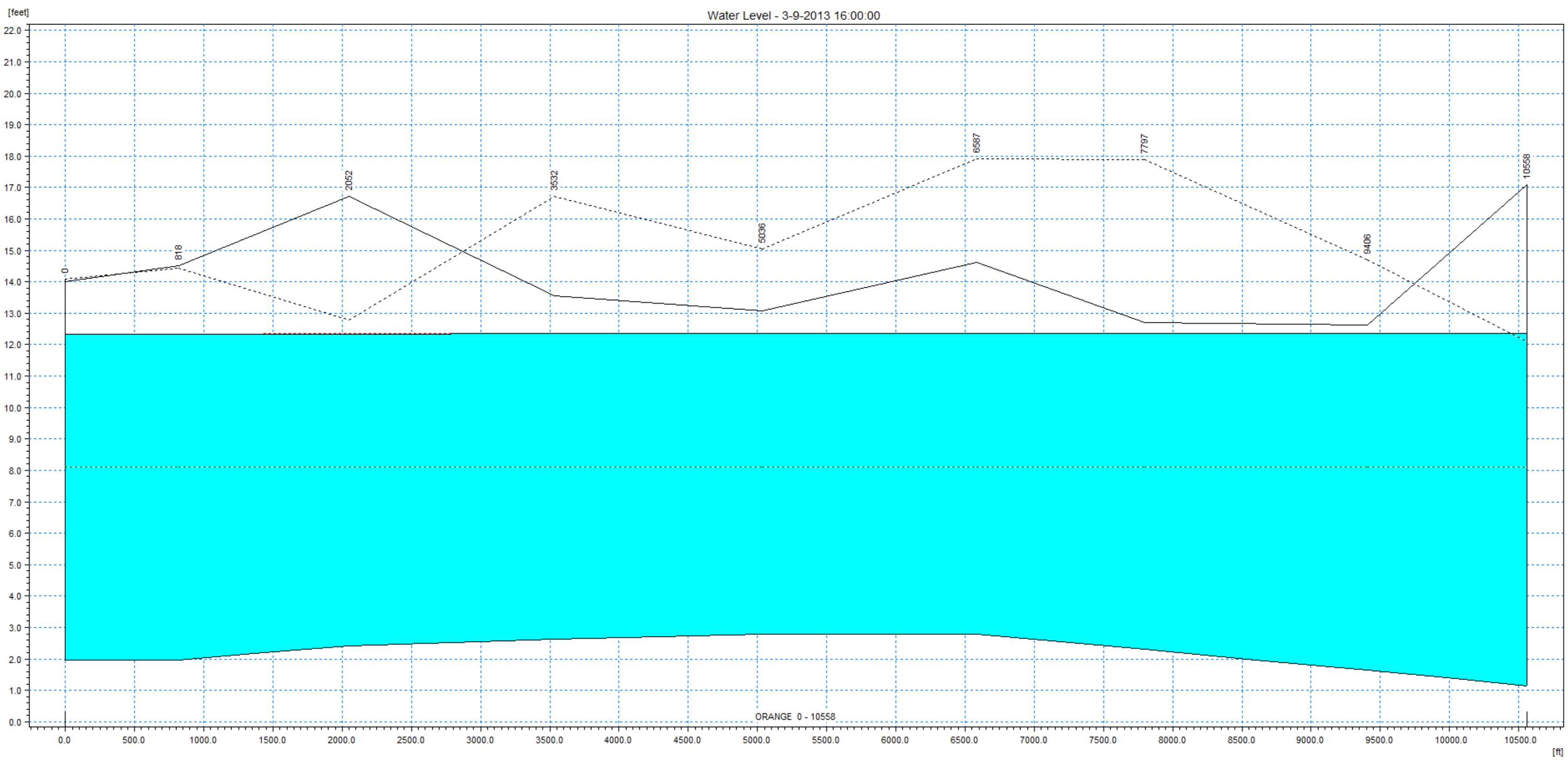
HARVEYCAN
HARVEYCAN 11132 - 13863
HARVEYCAN 13863 - 19741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11.0
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5.0
4.0
3.0
2.0
1.0
0.0
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818
2052
3532
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10558
ORANGE 0 - 10558
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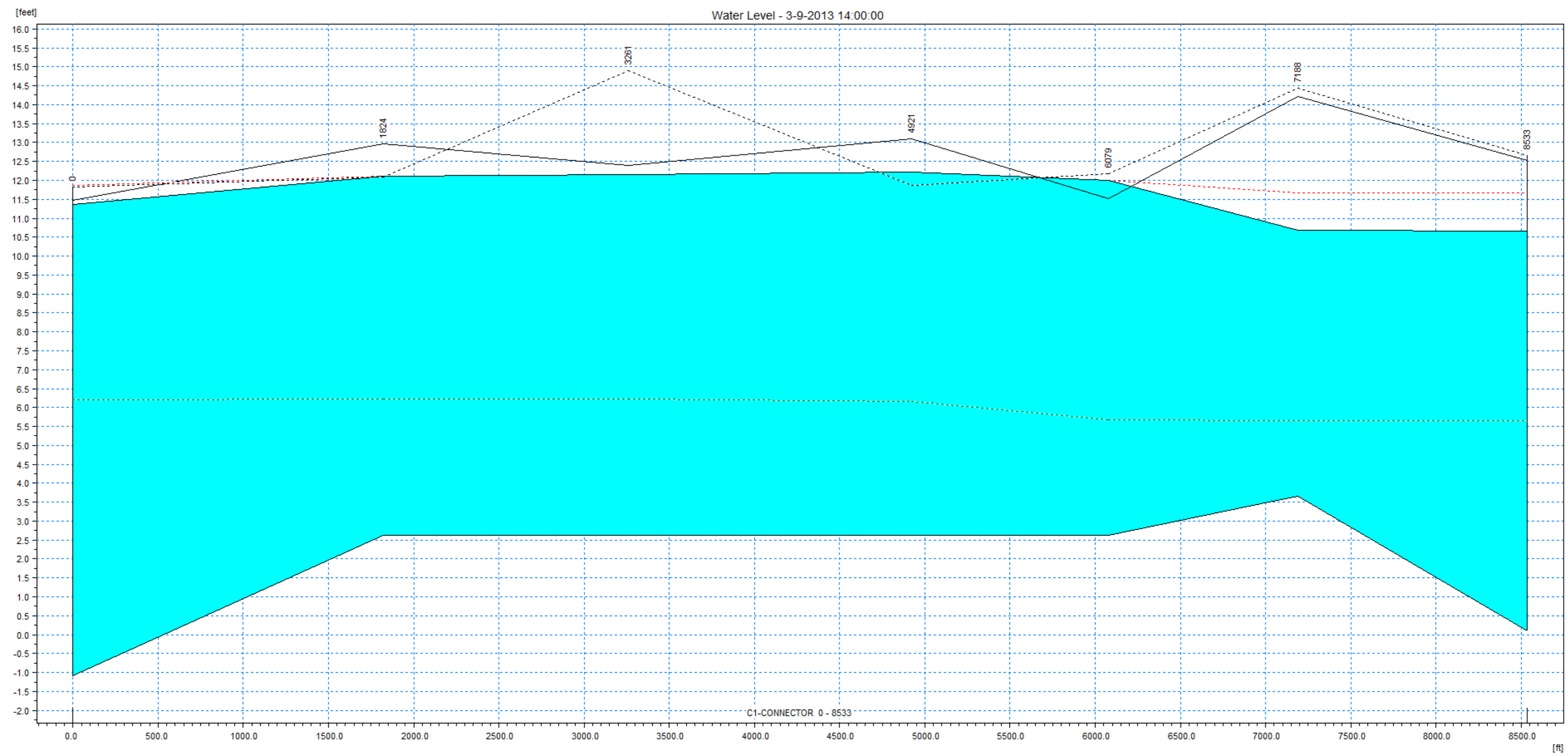
Part 4 100-year Current Conditions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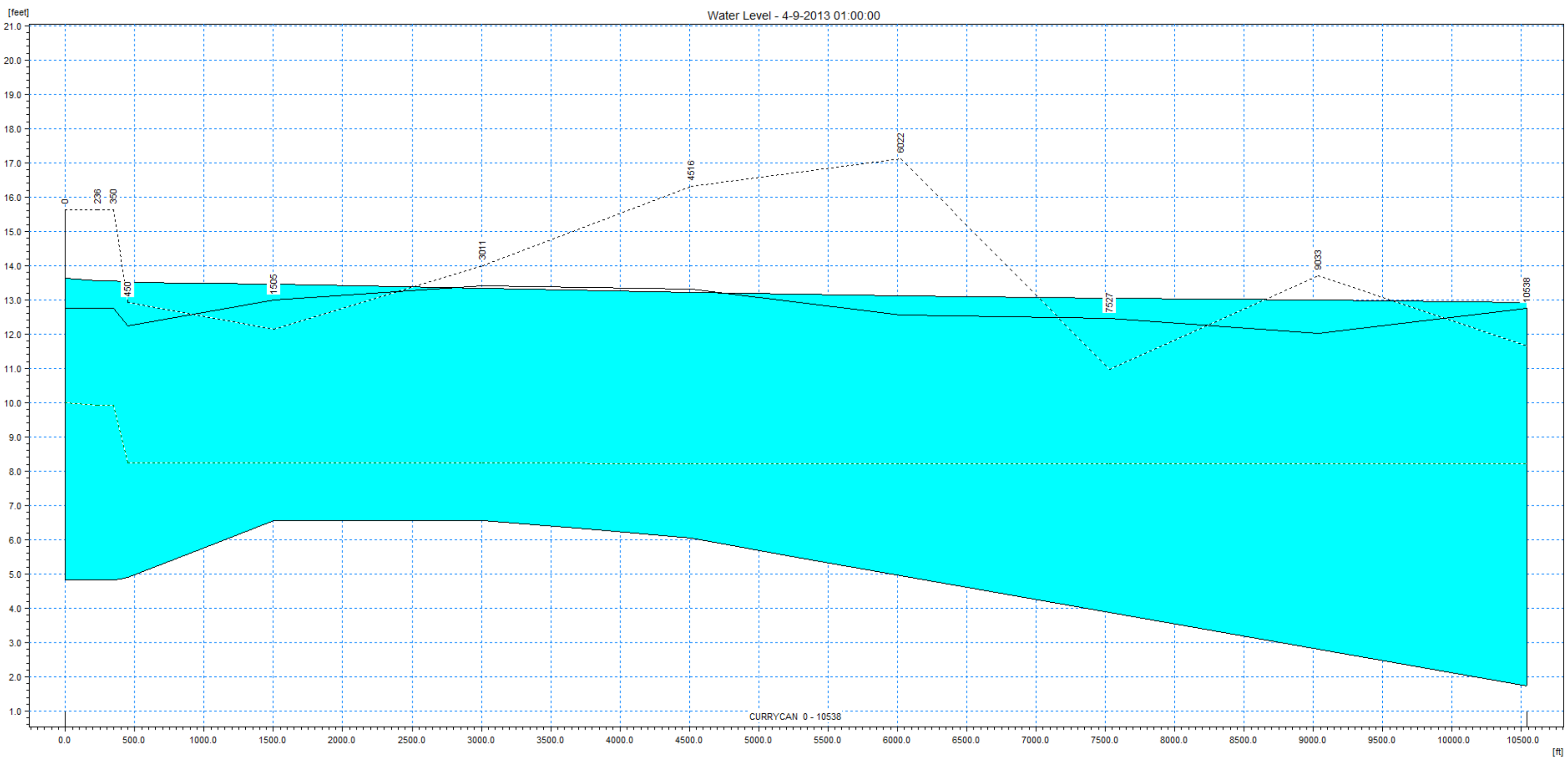
4921
6079
7188
8533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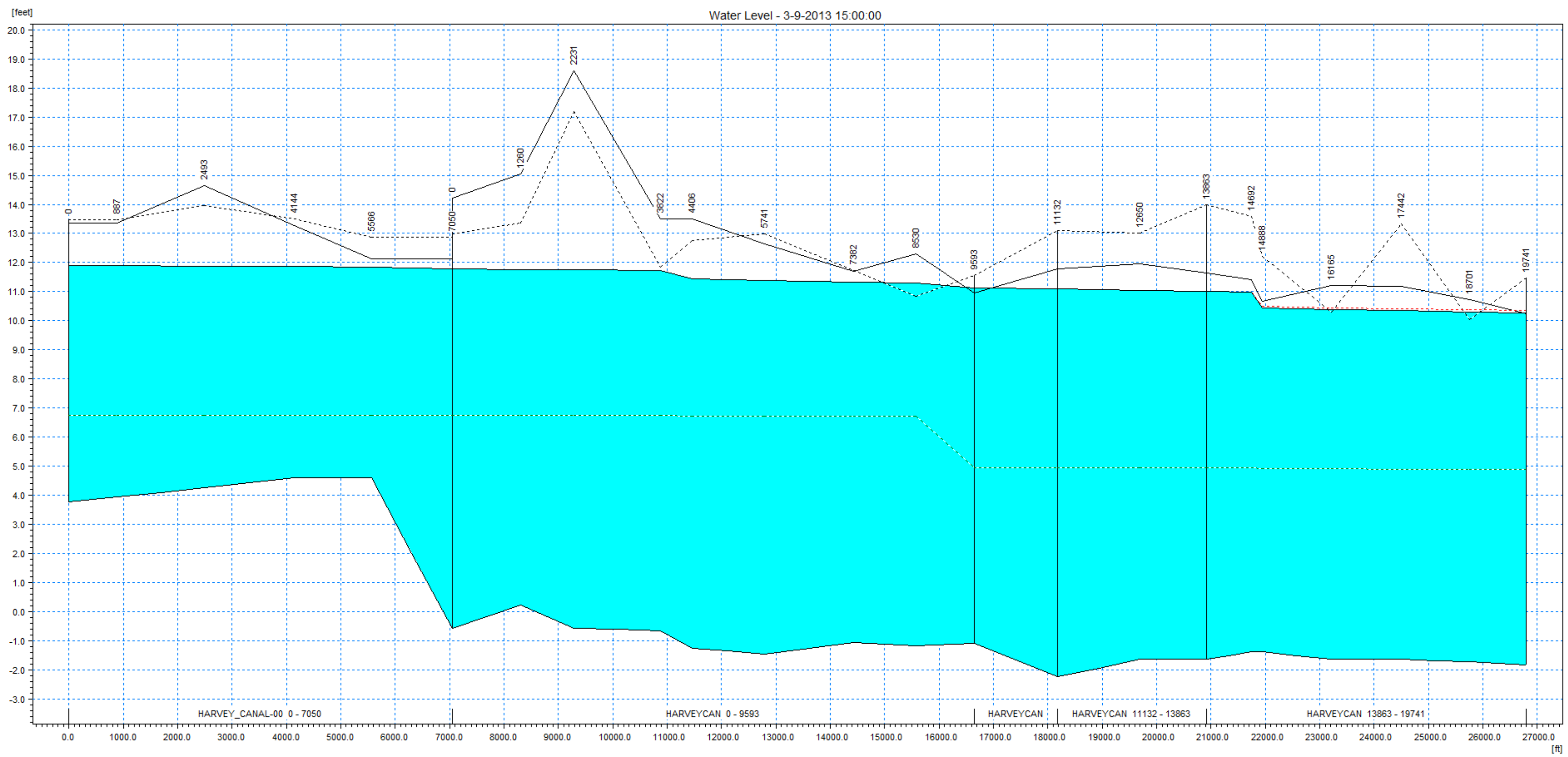
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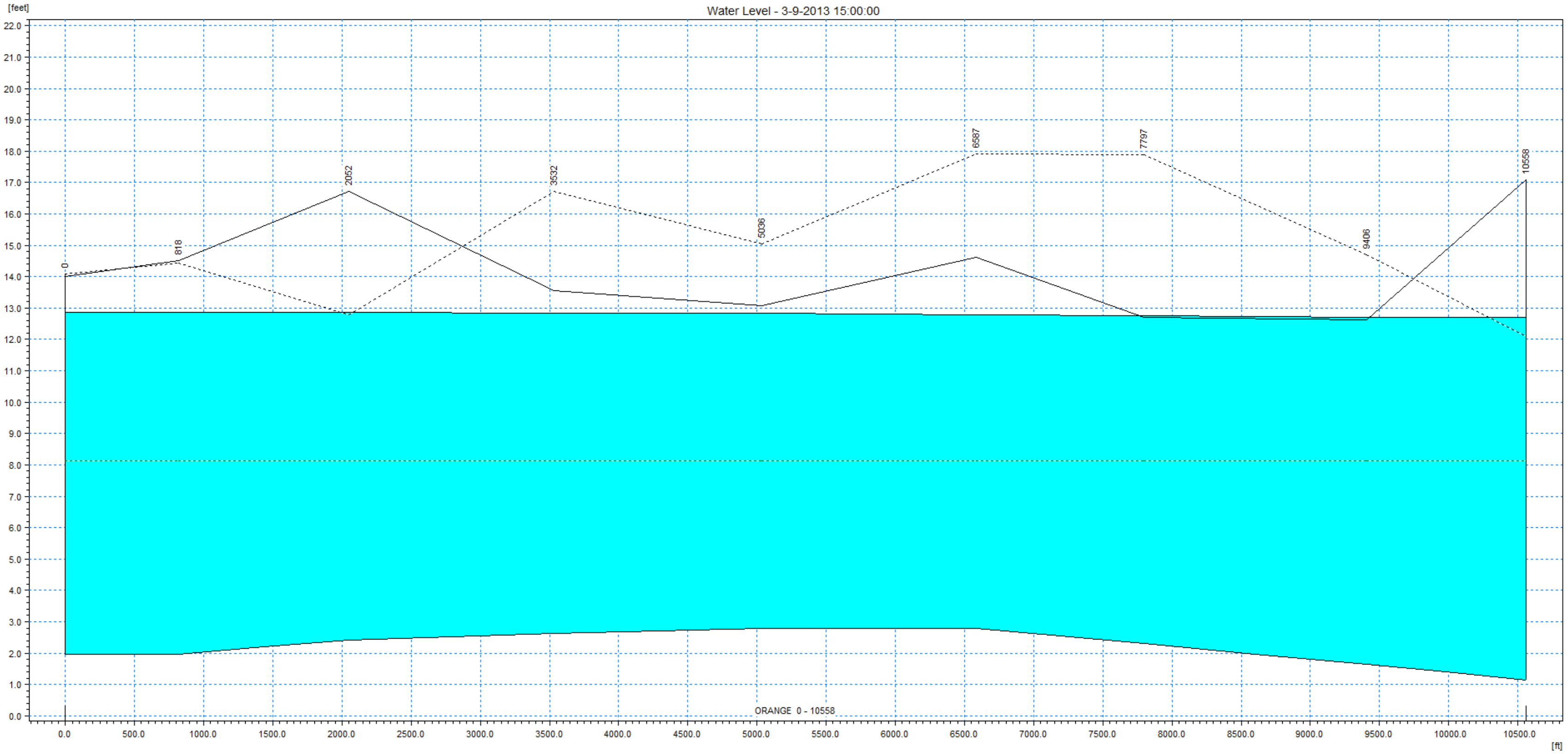
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N – Supplemental Maximum Stage Profiles, Future Conditions

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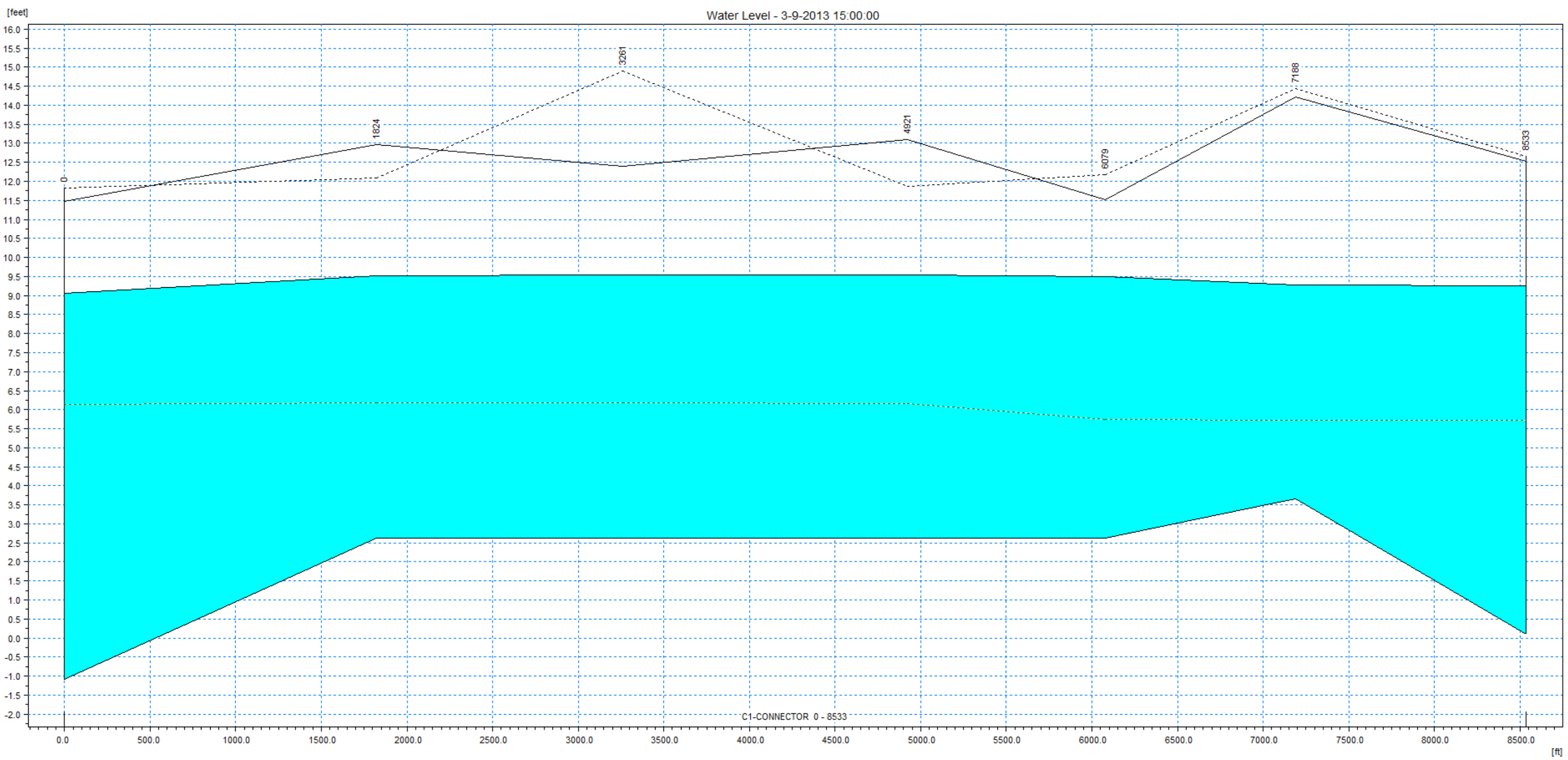

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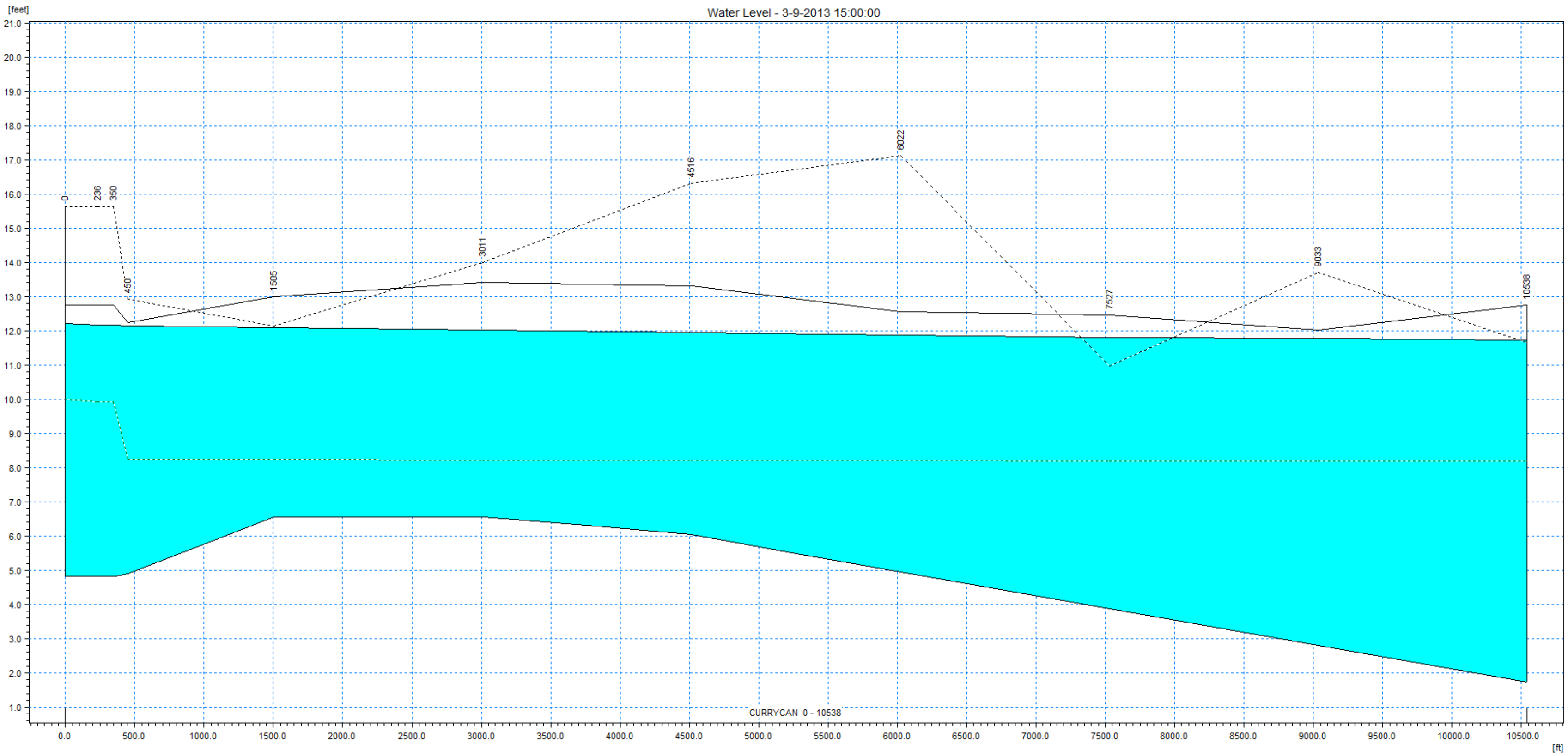
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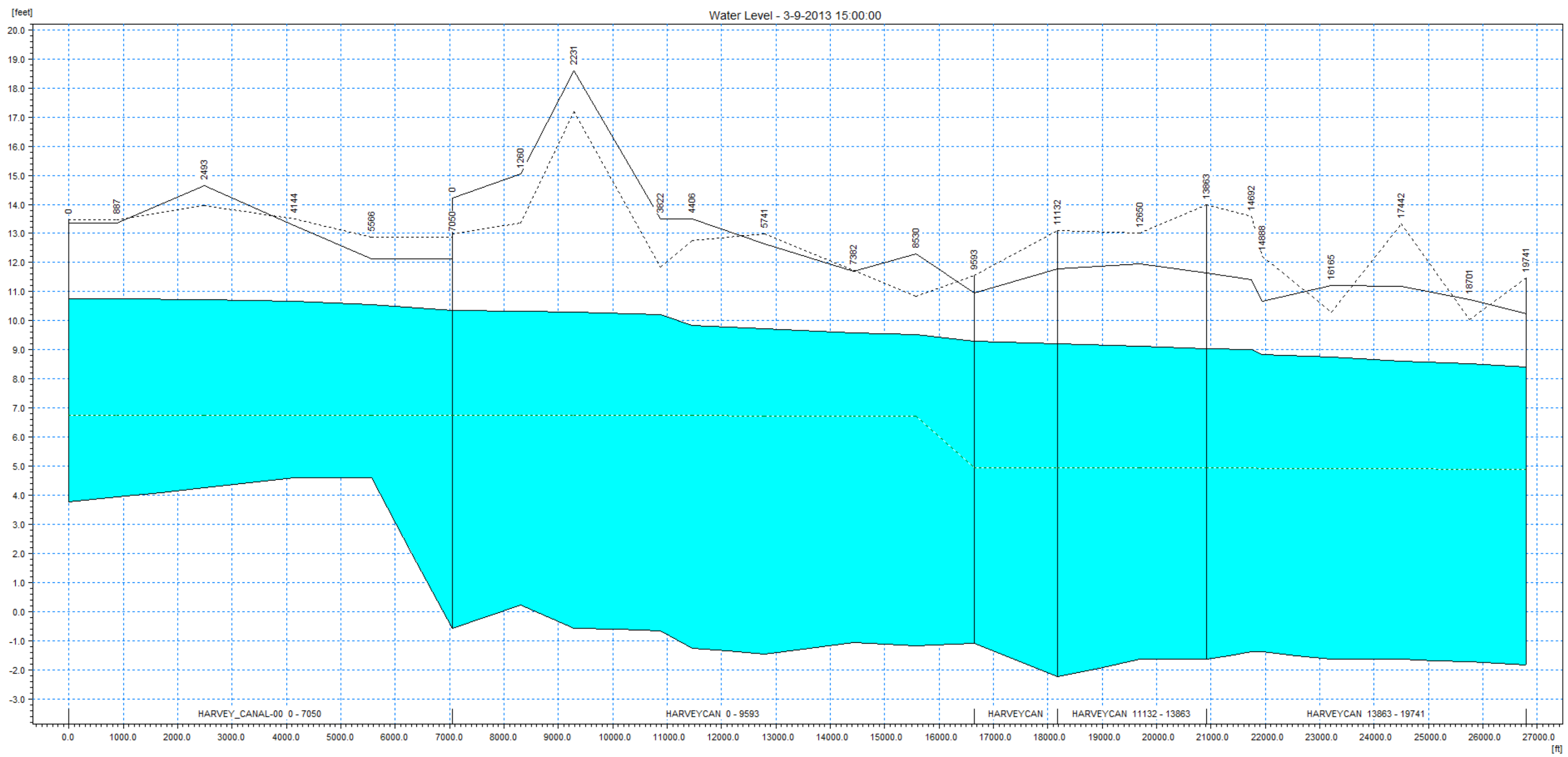
Curry Canal



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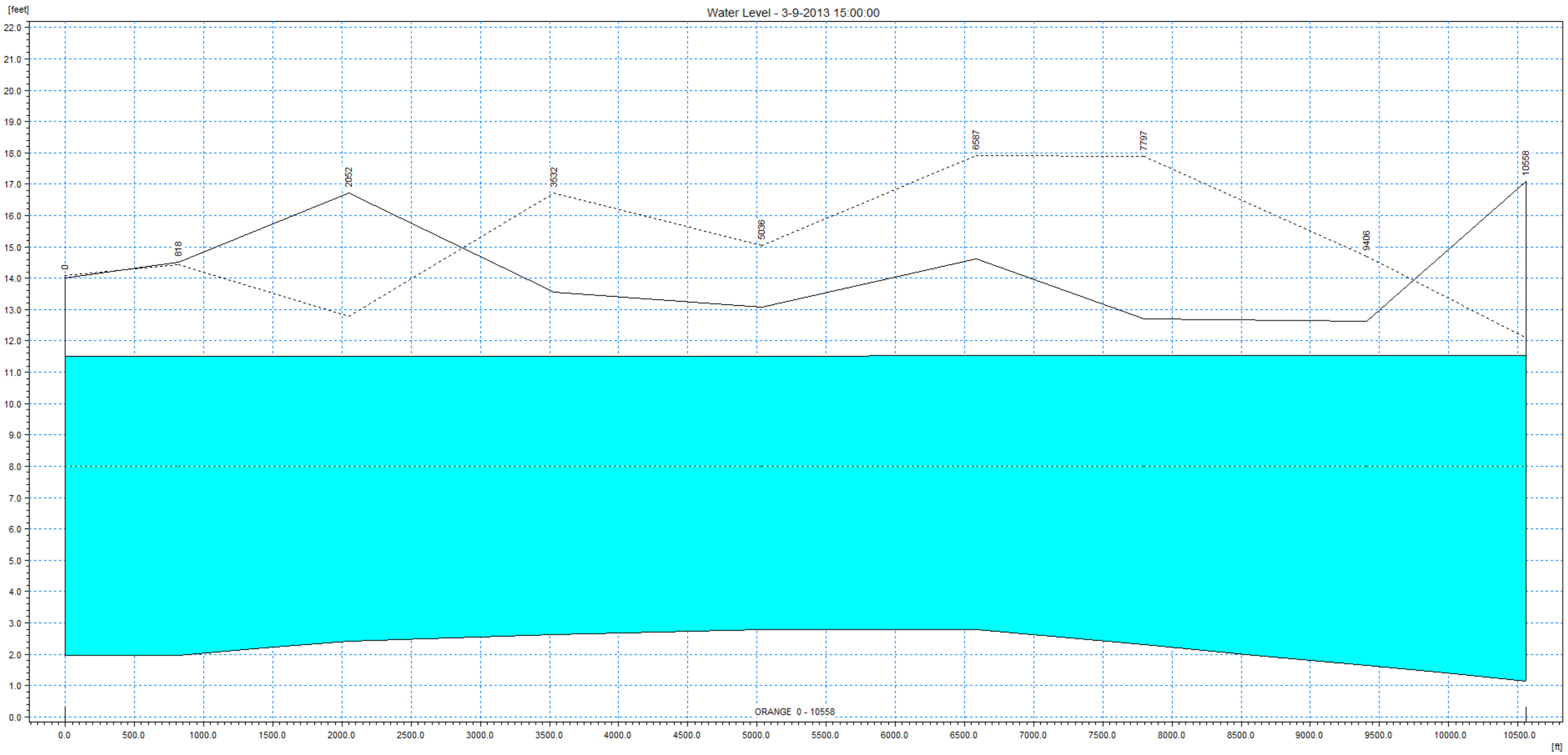
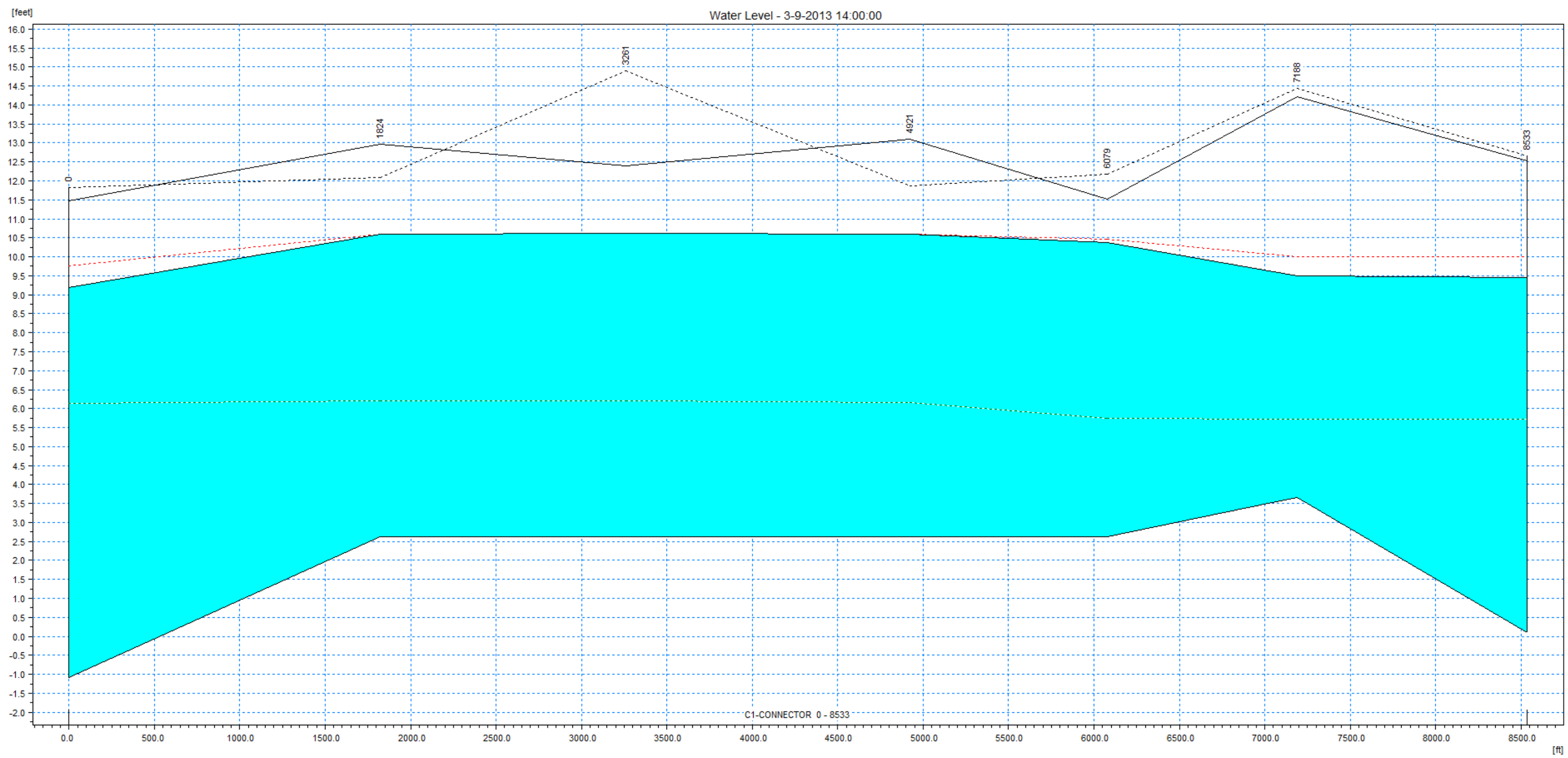


Orange Canal

Part 2 10-year Future Conditions



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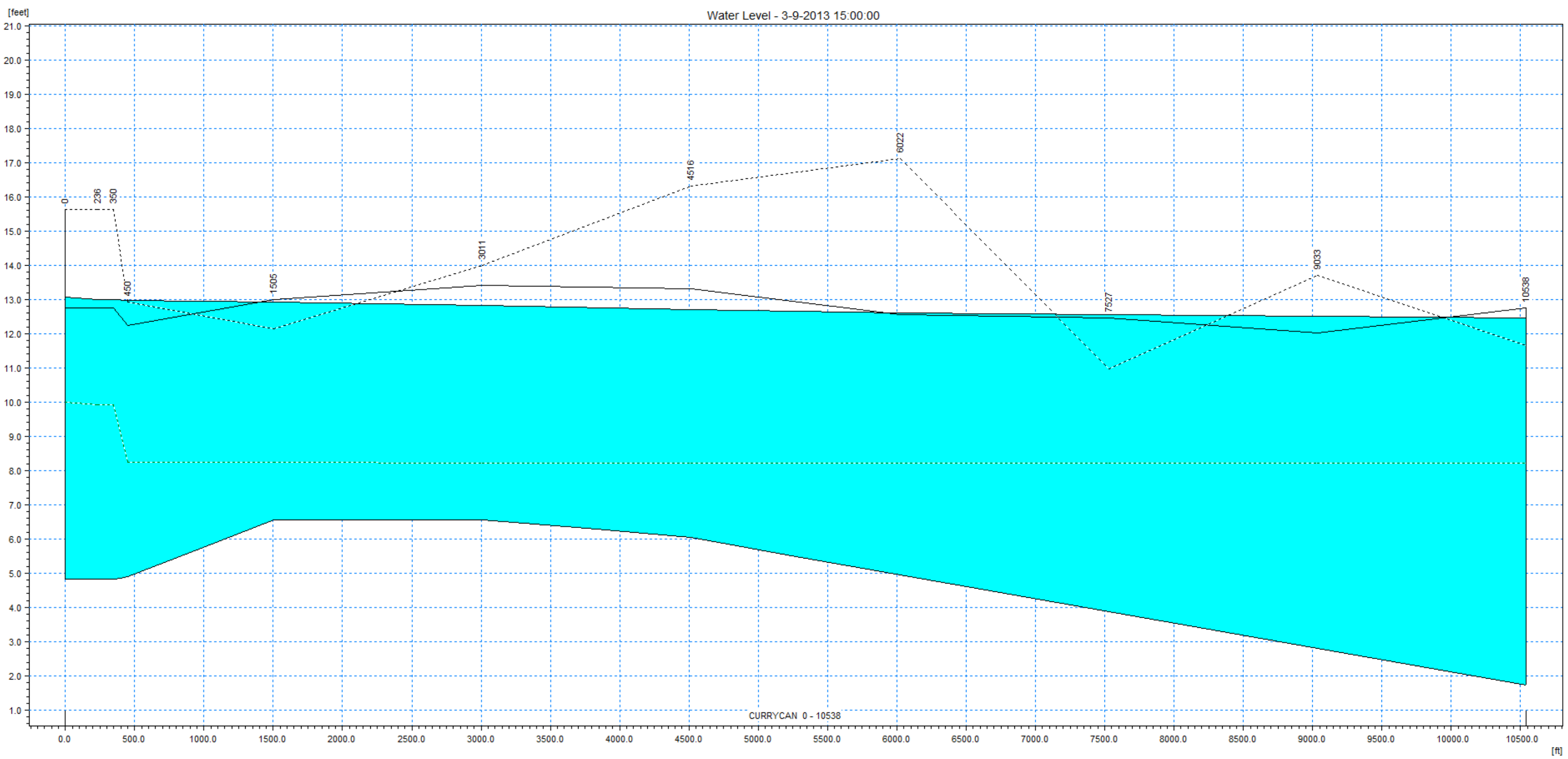

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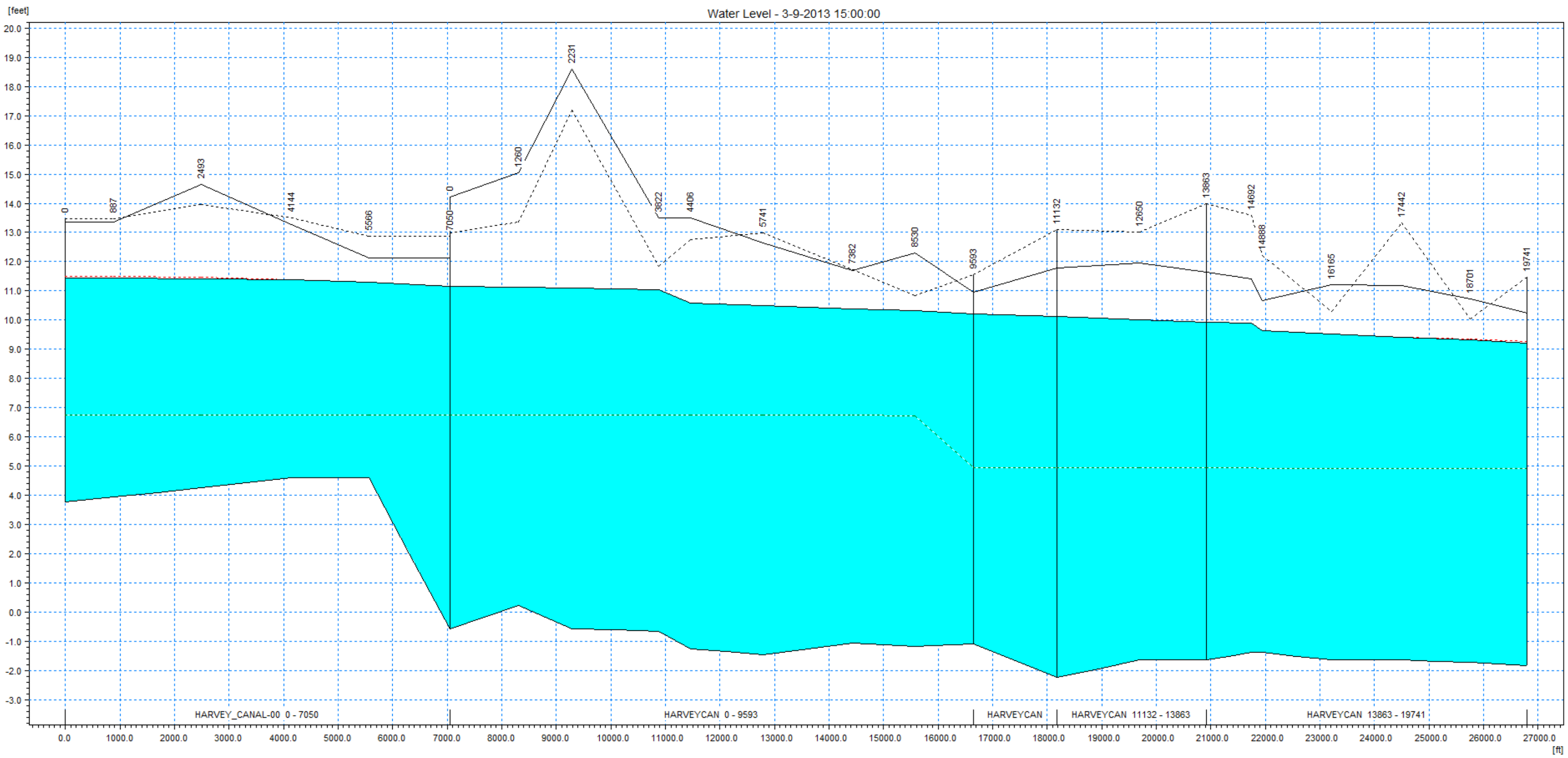

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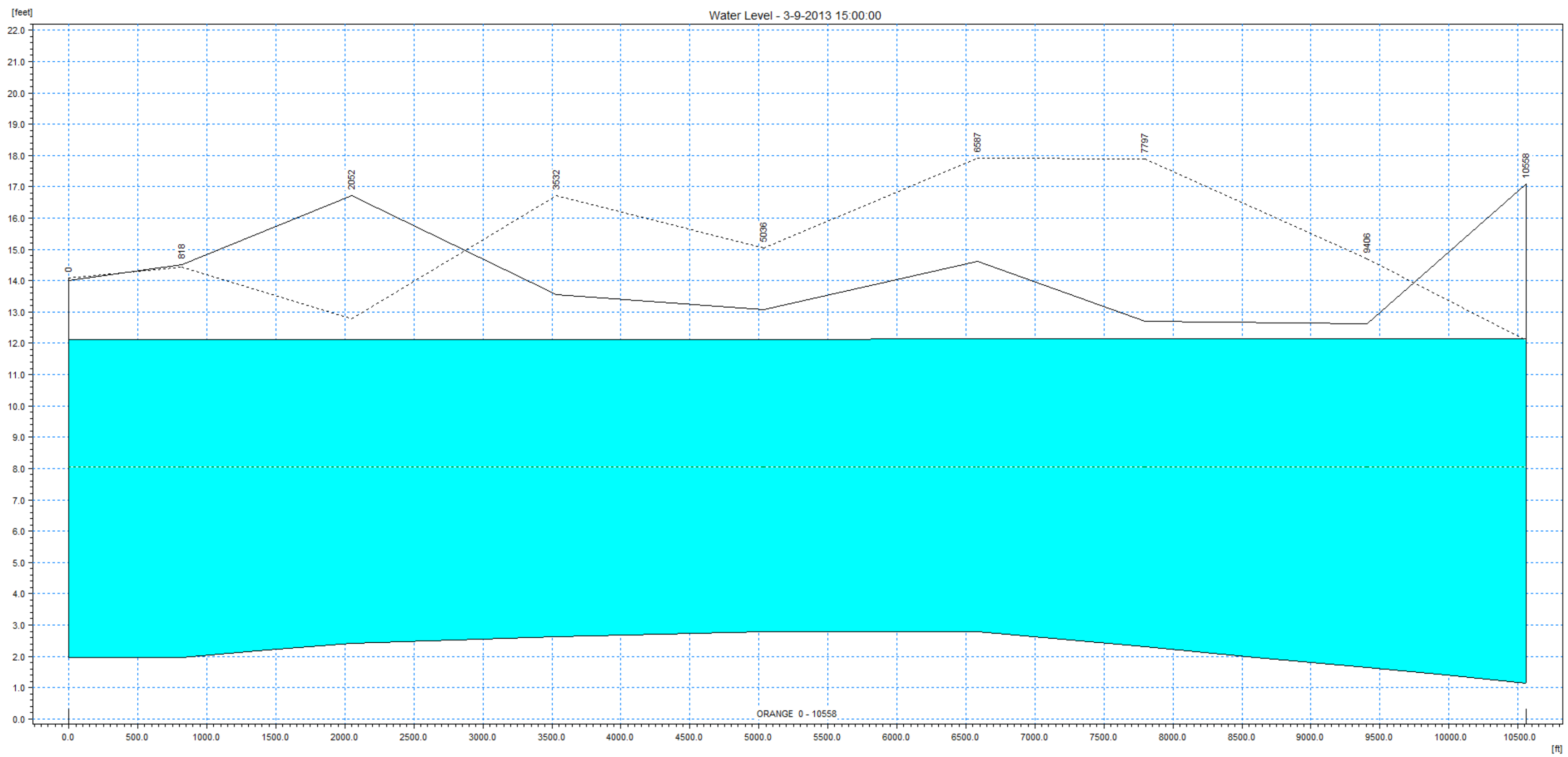

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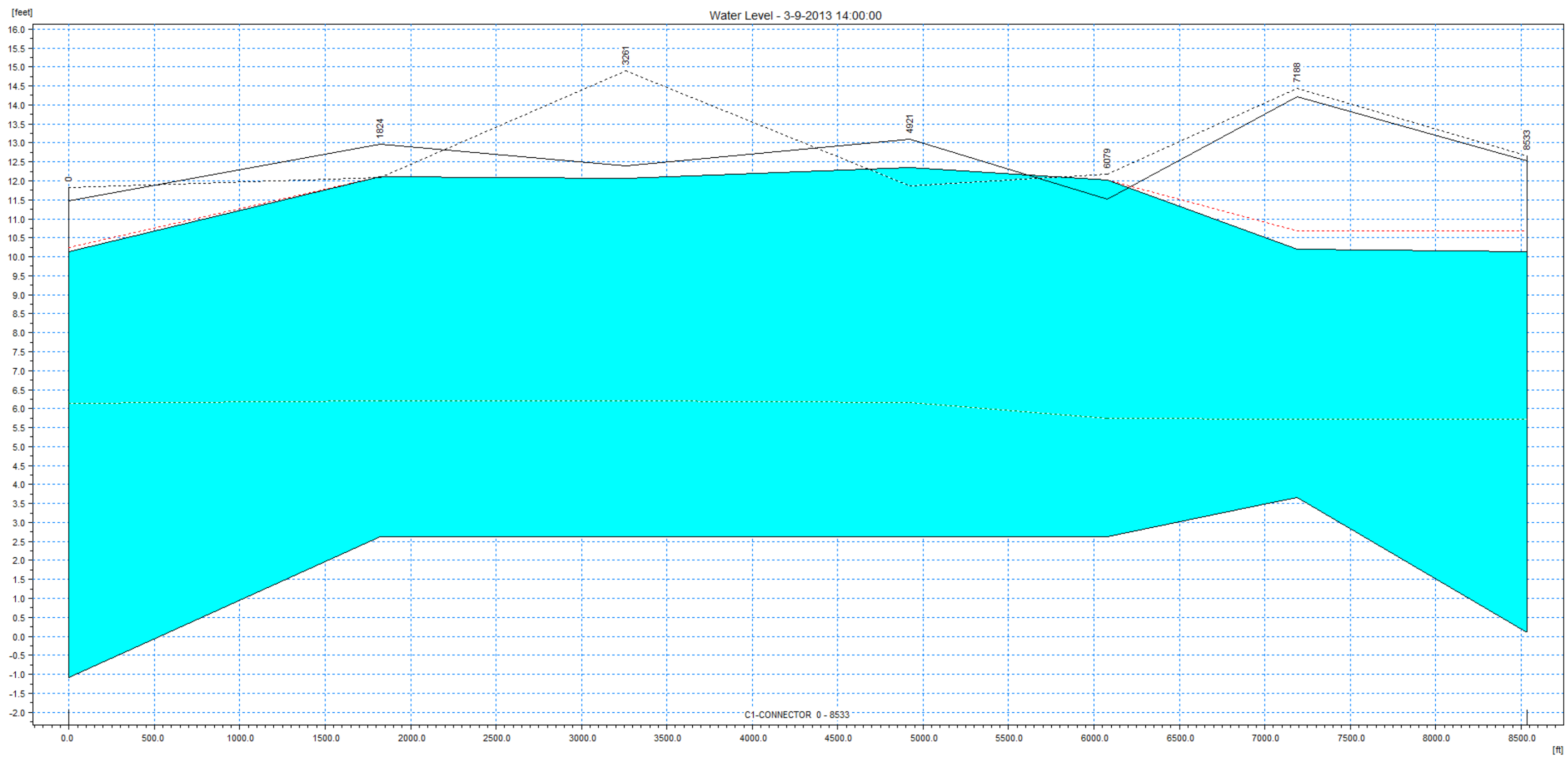

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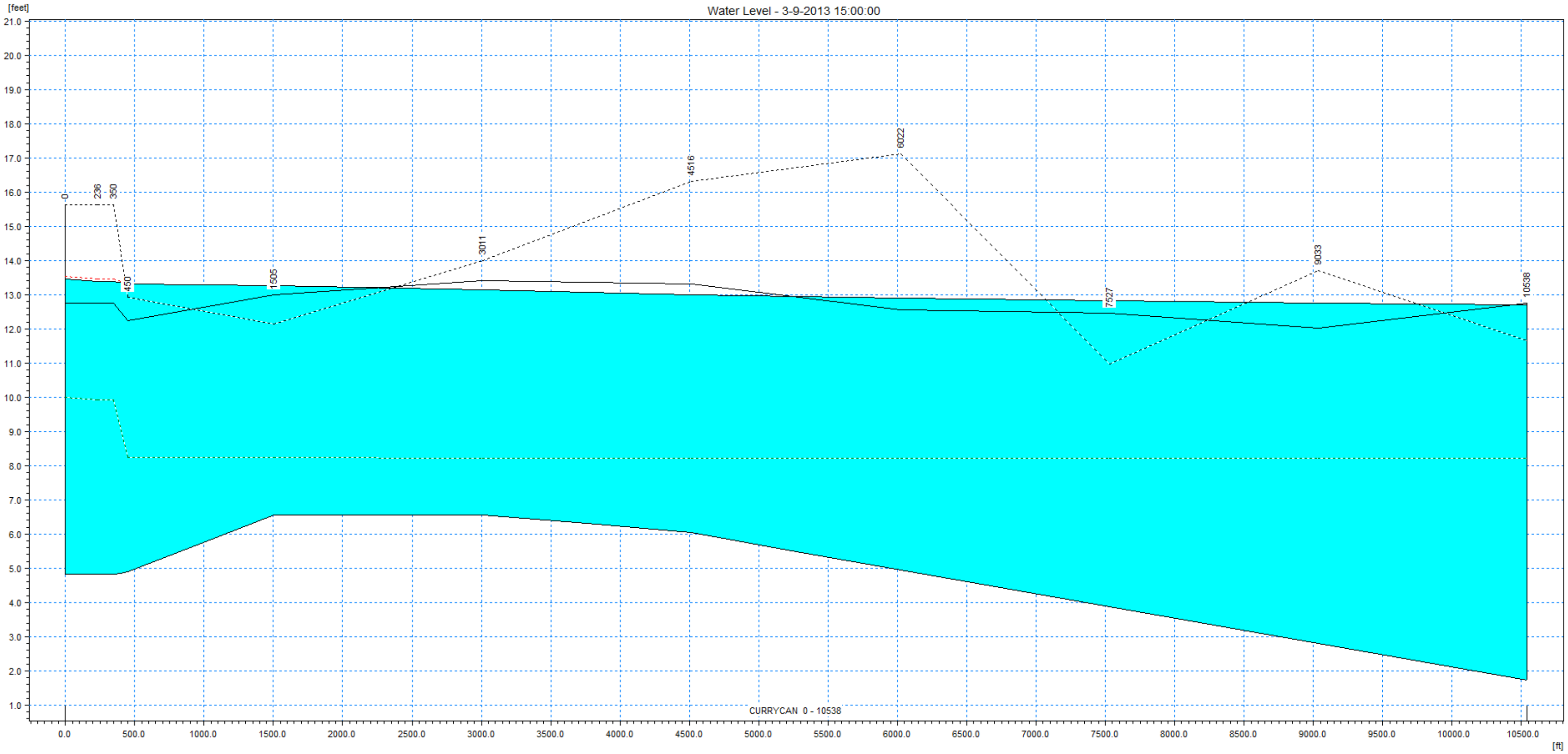
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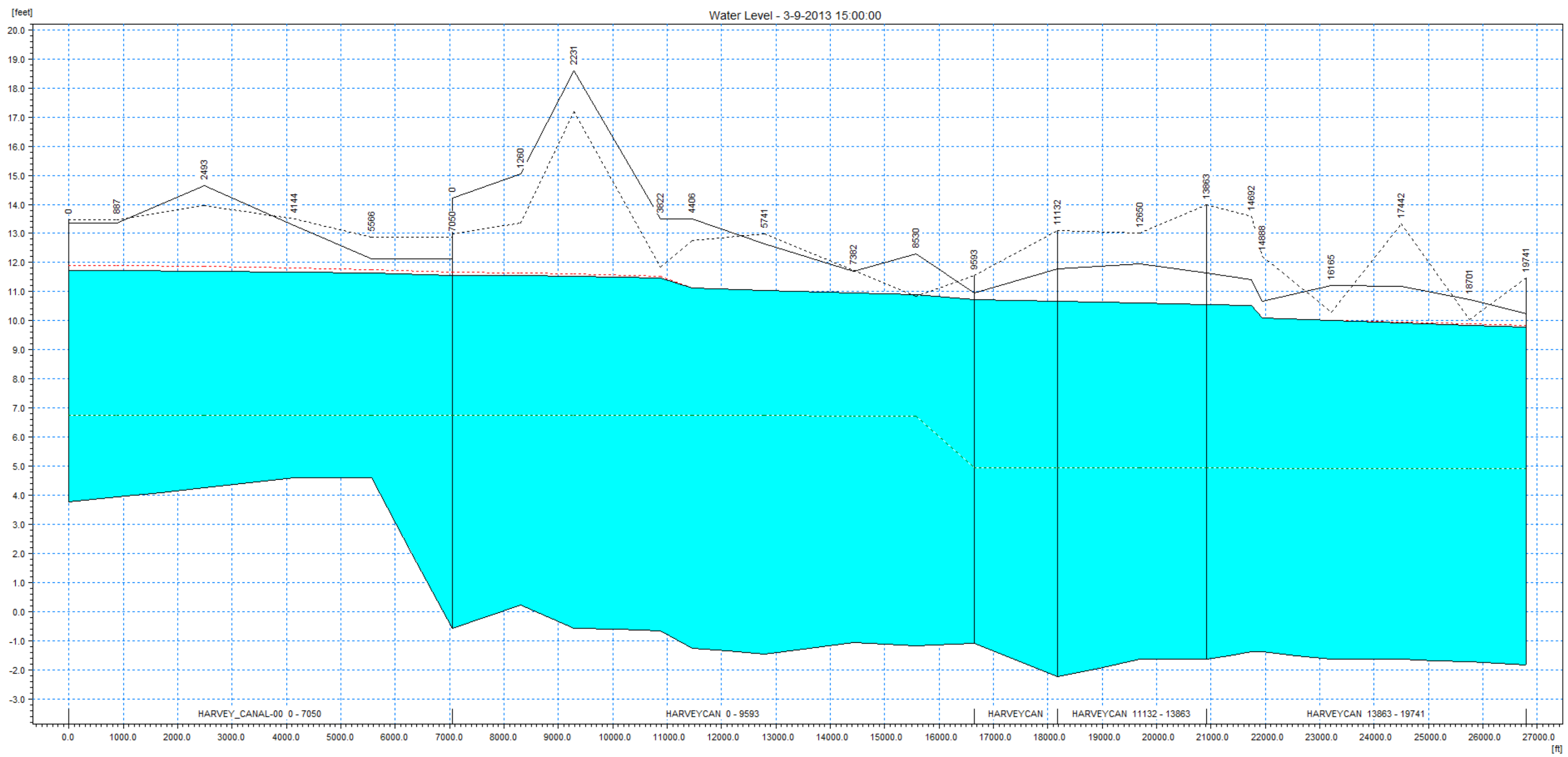

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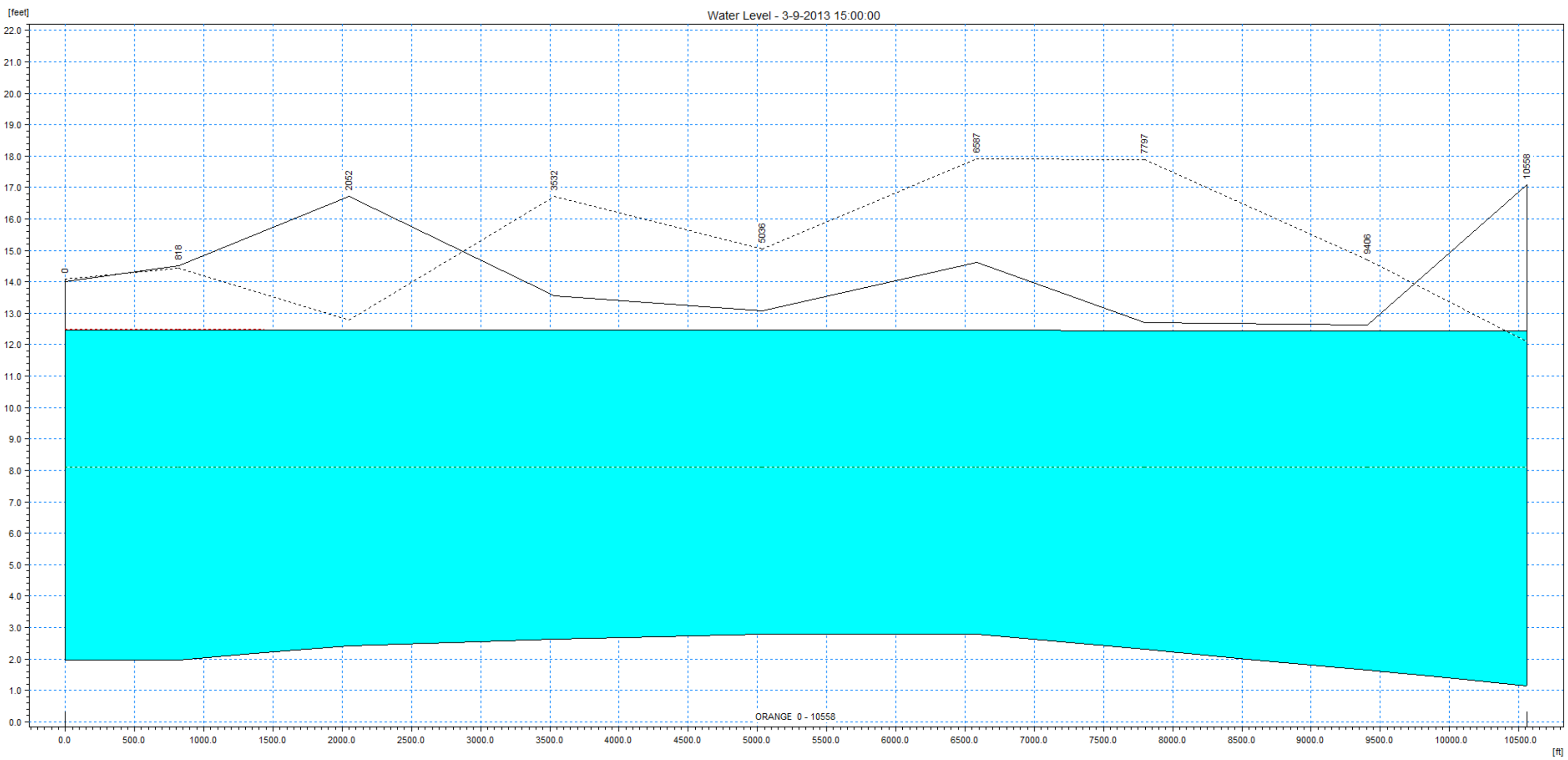

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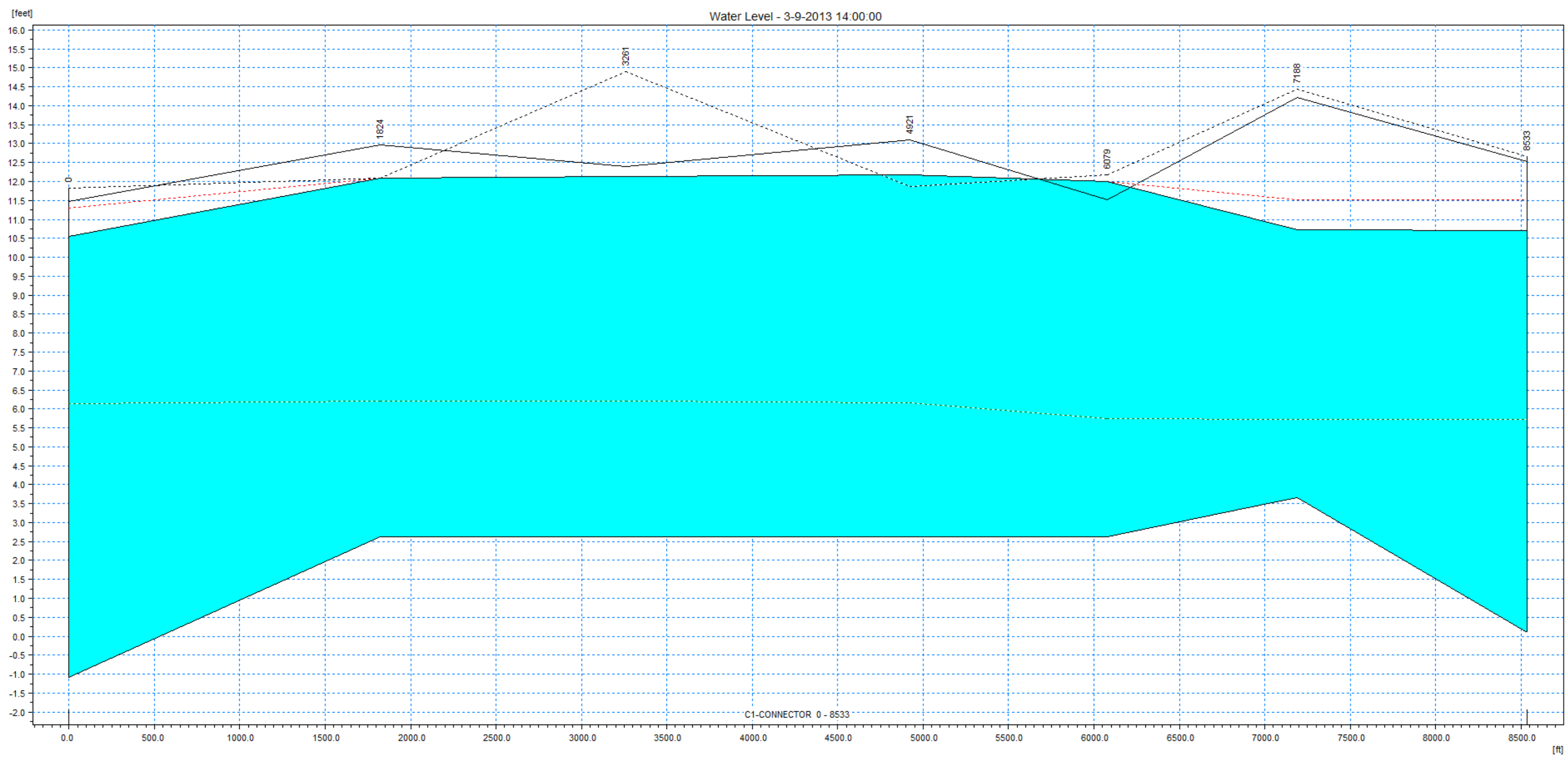

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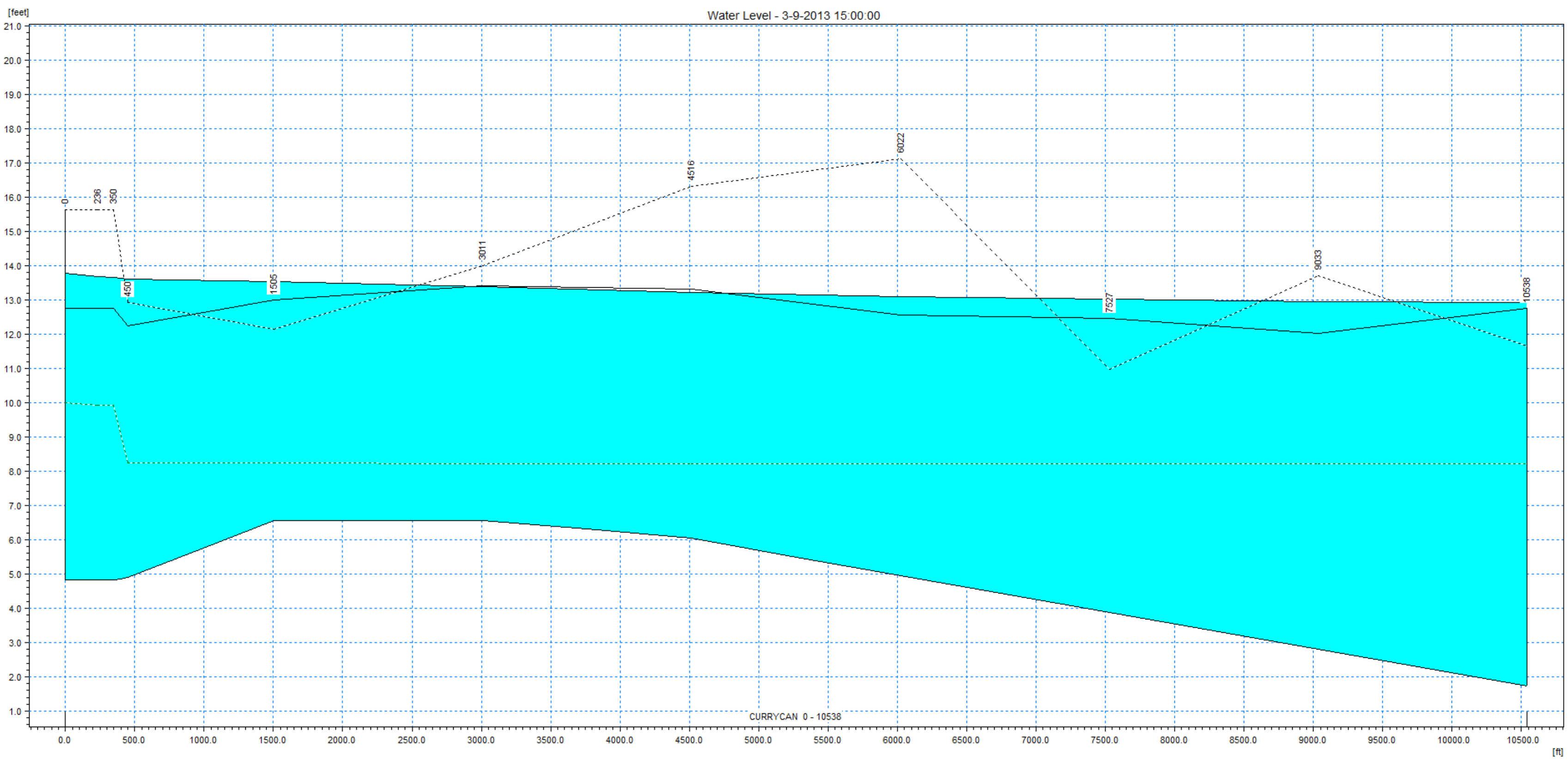
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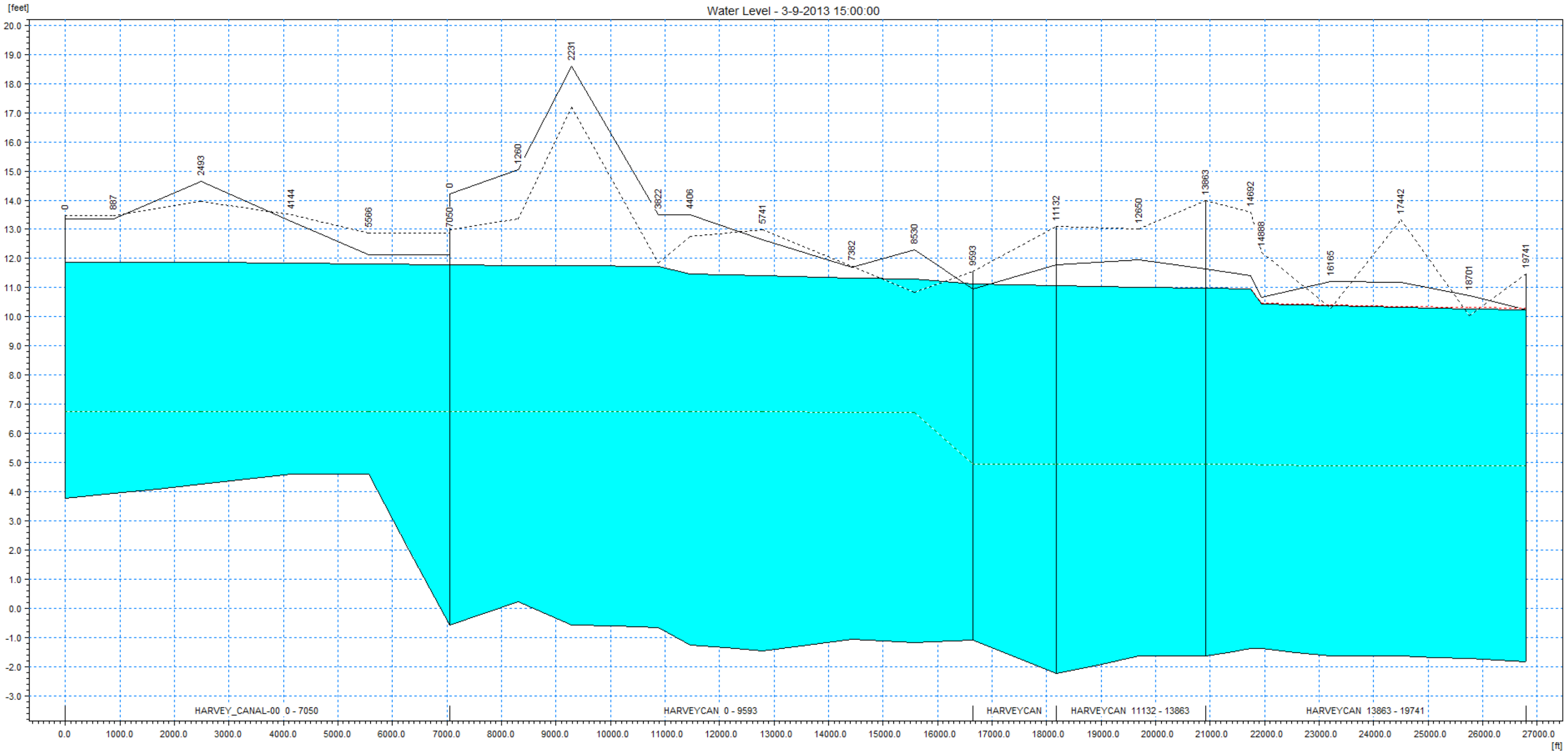

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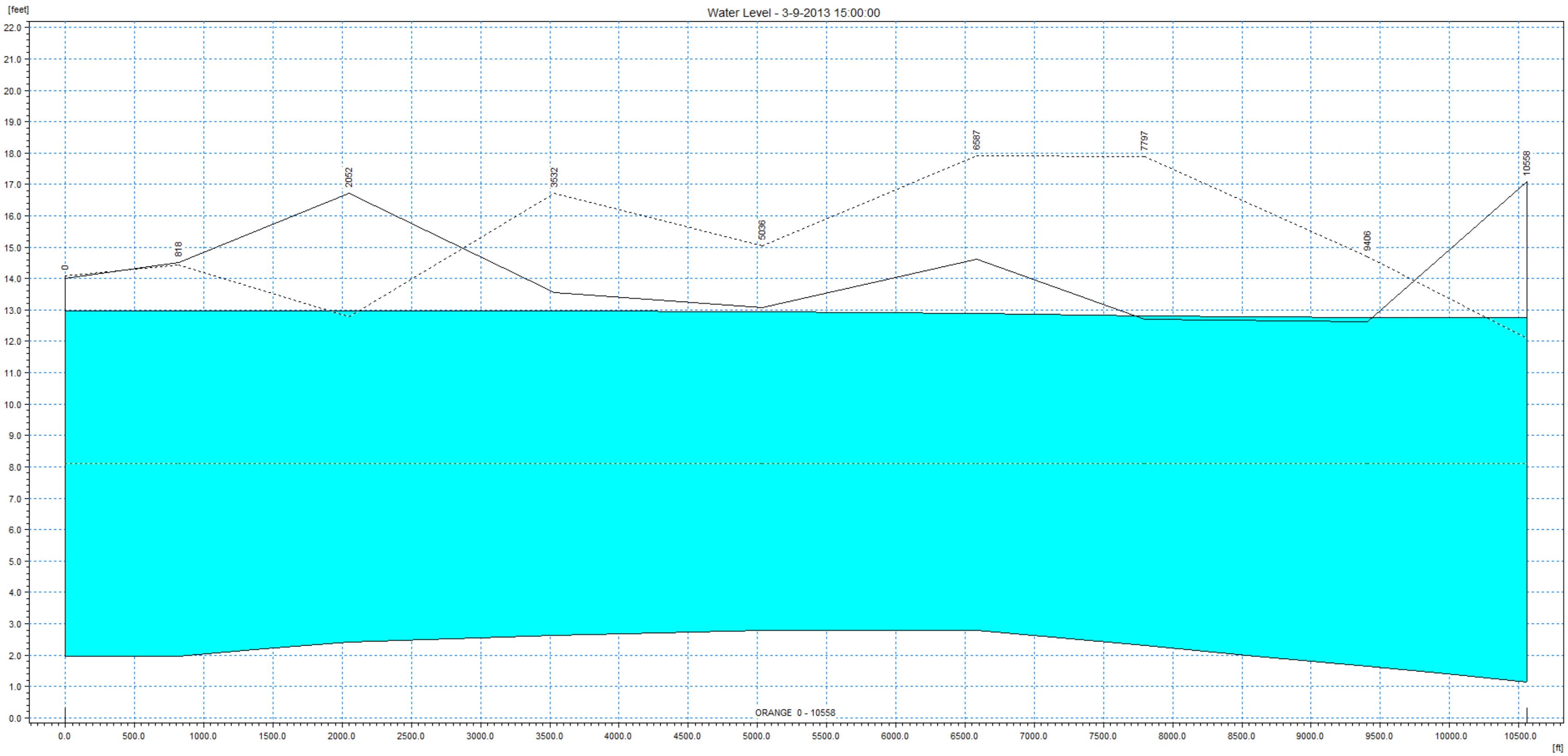
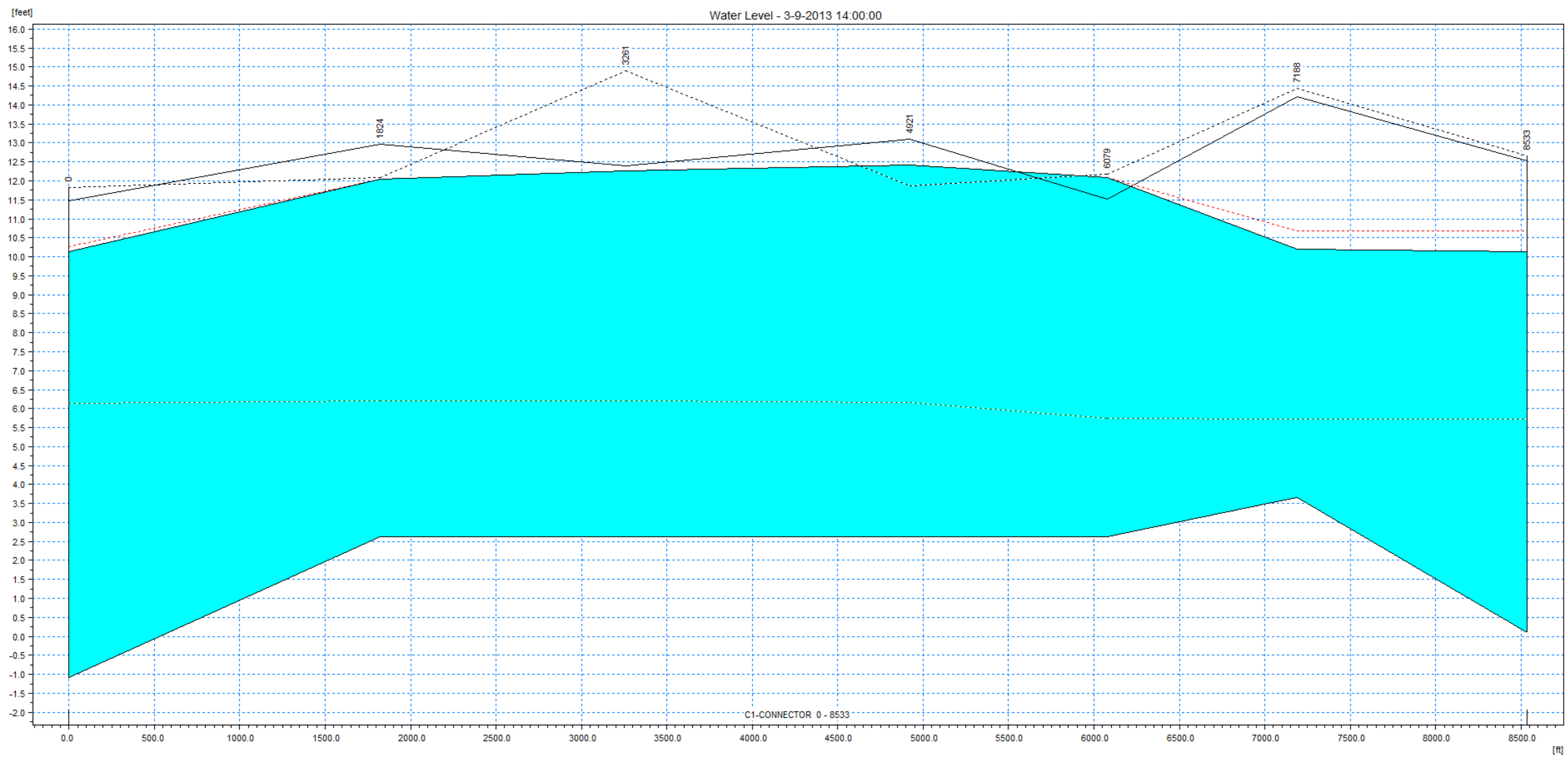


Orange Canal

Part 5 25-year Future Conditions SLR1



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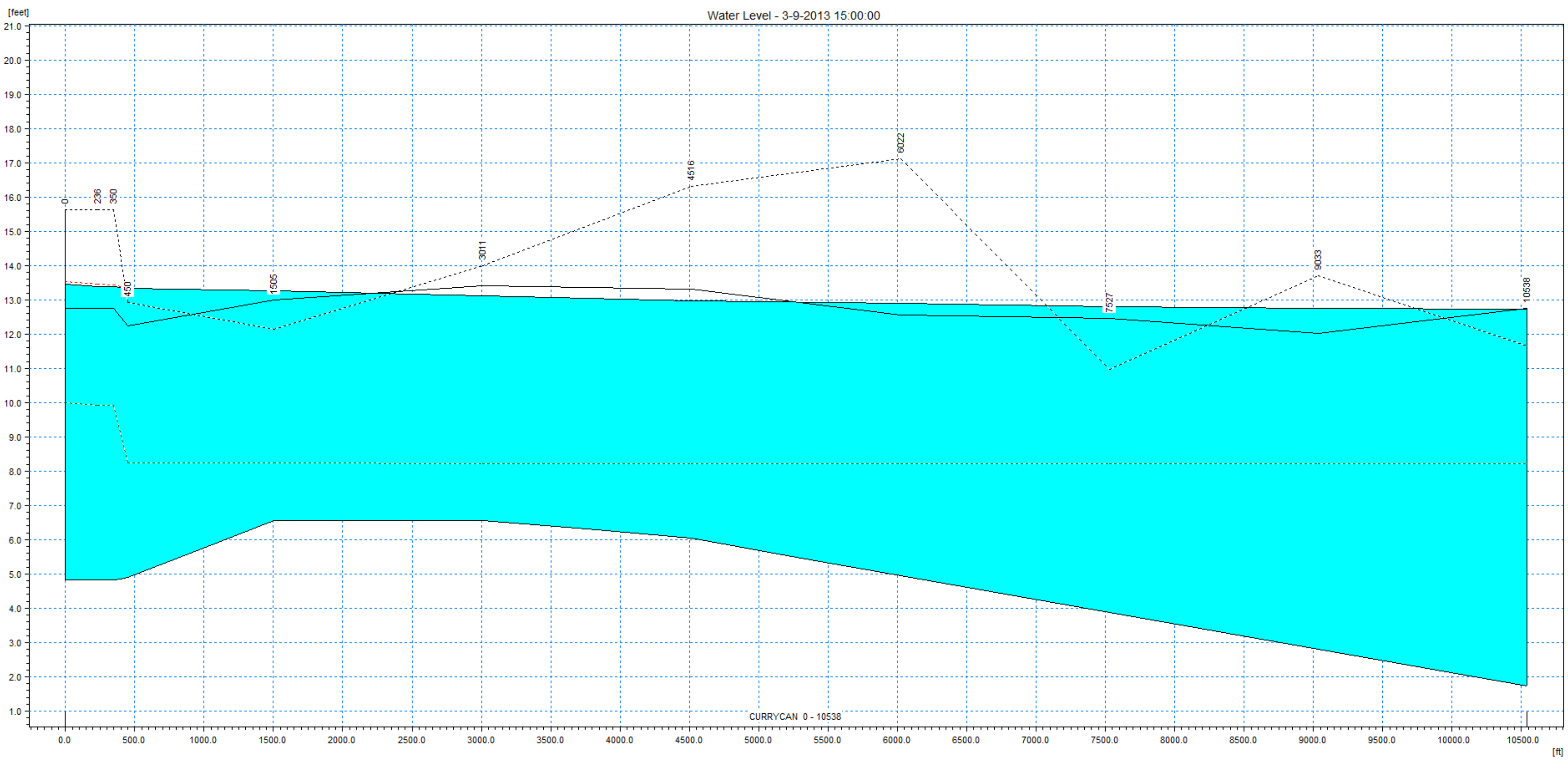

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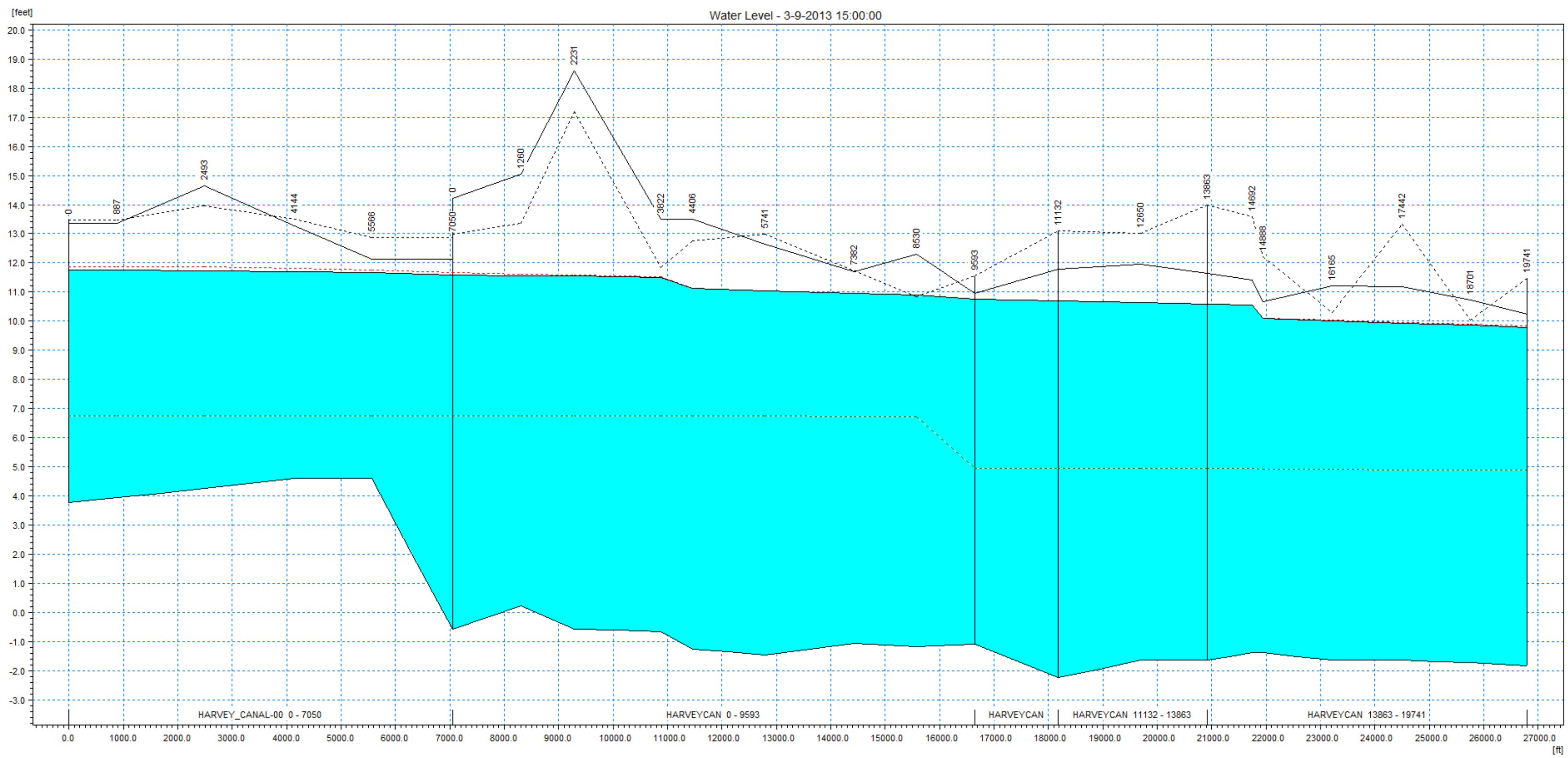

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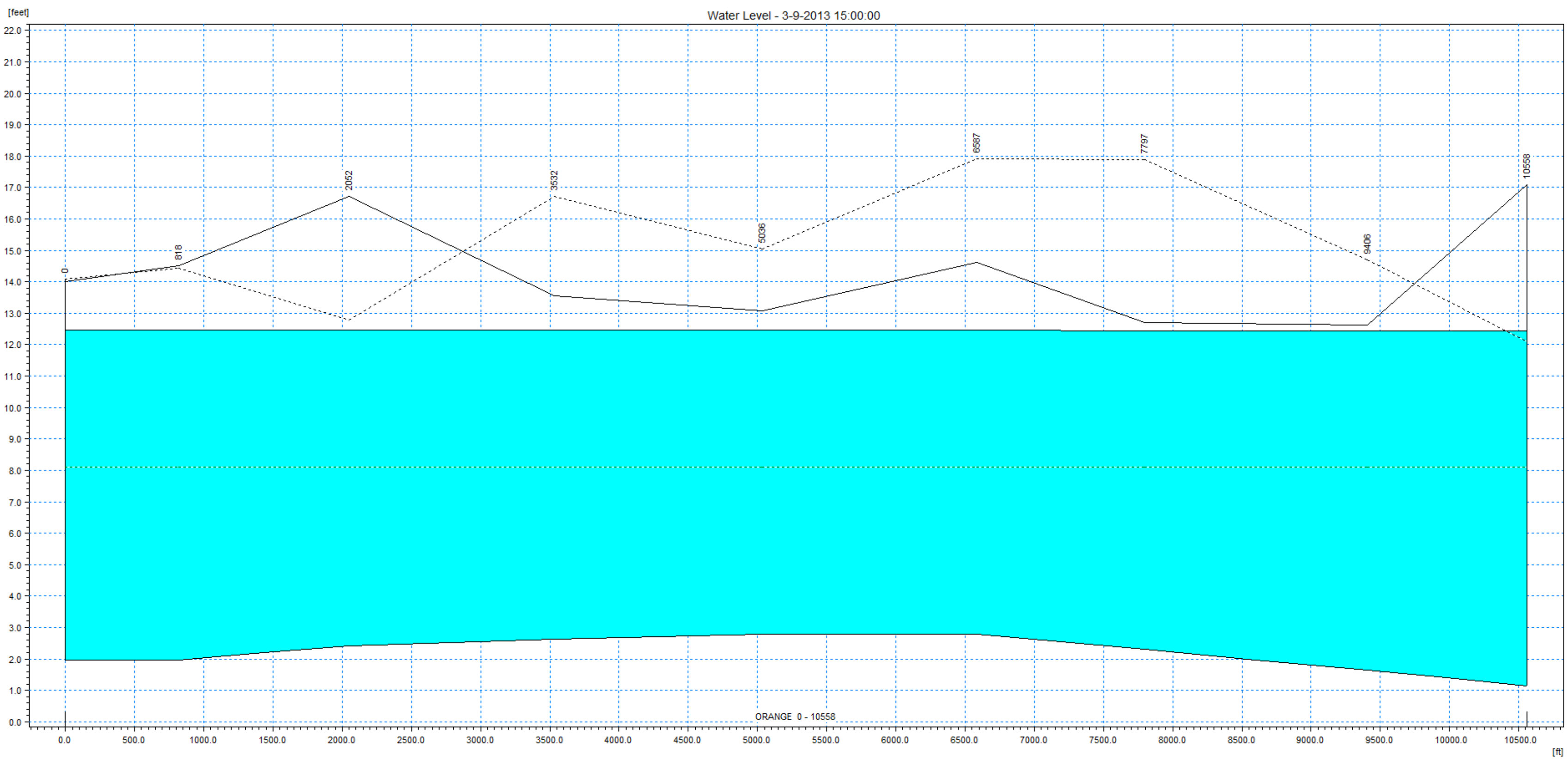

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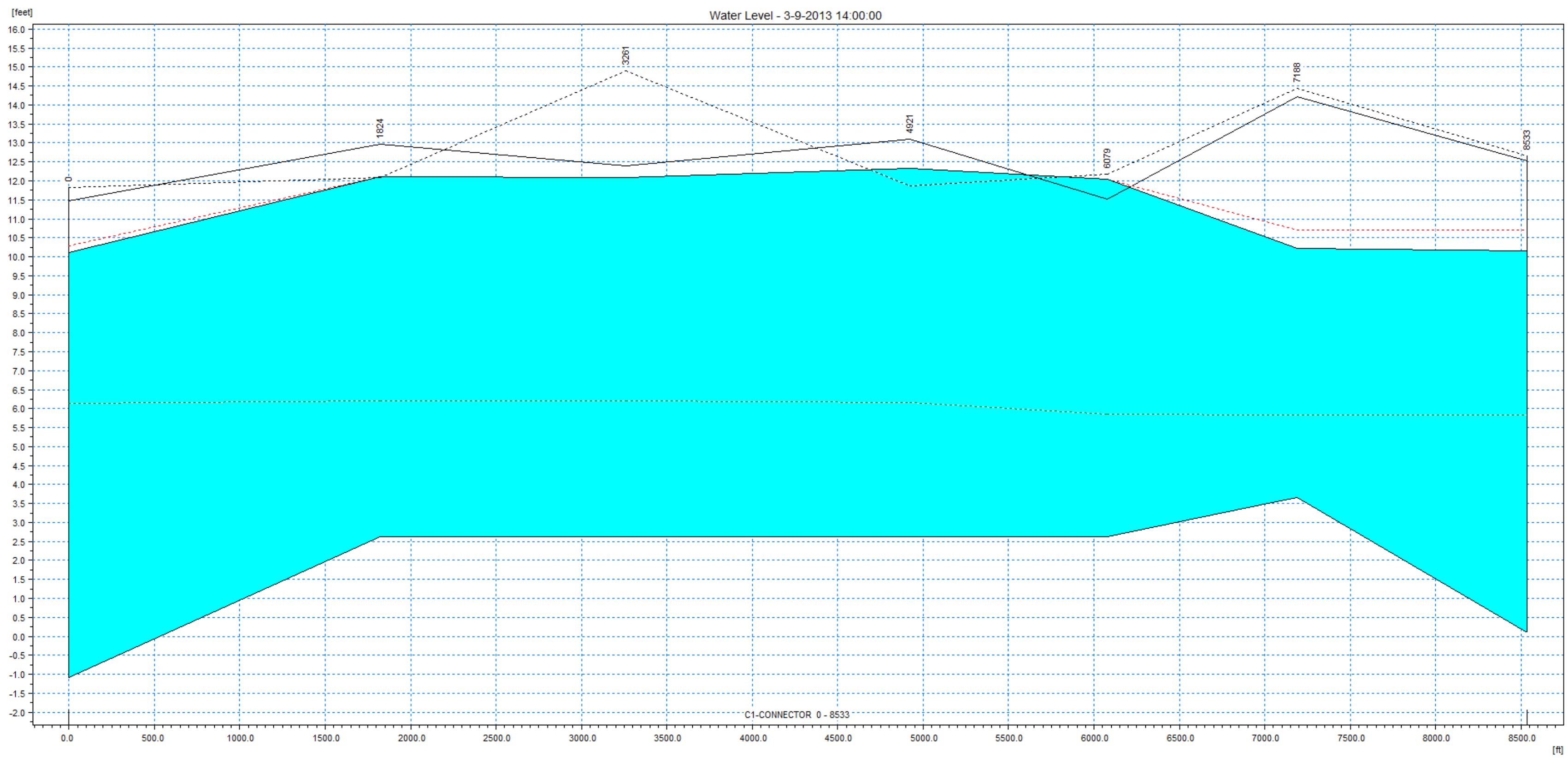

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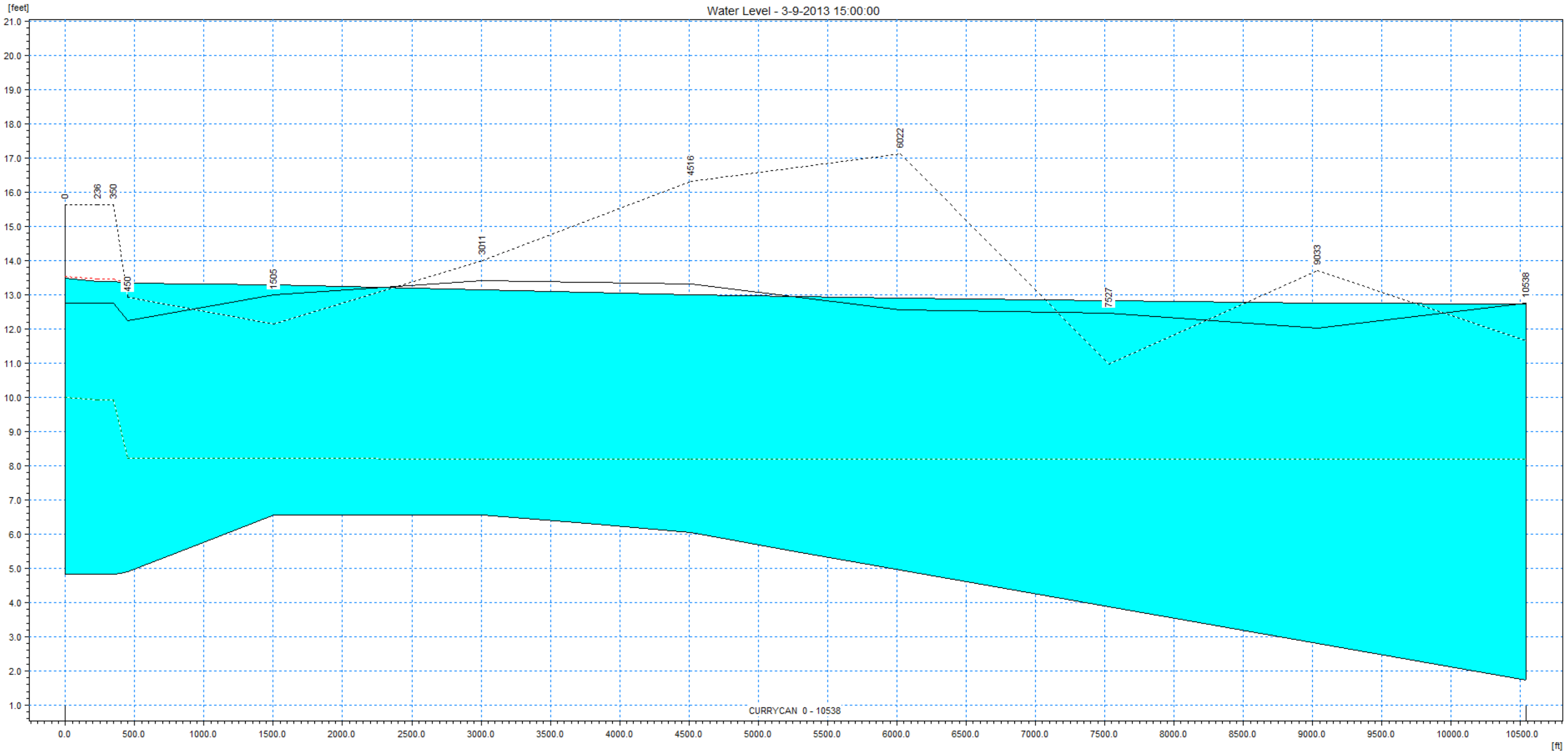
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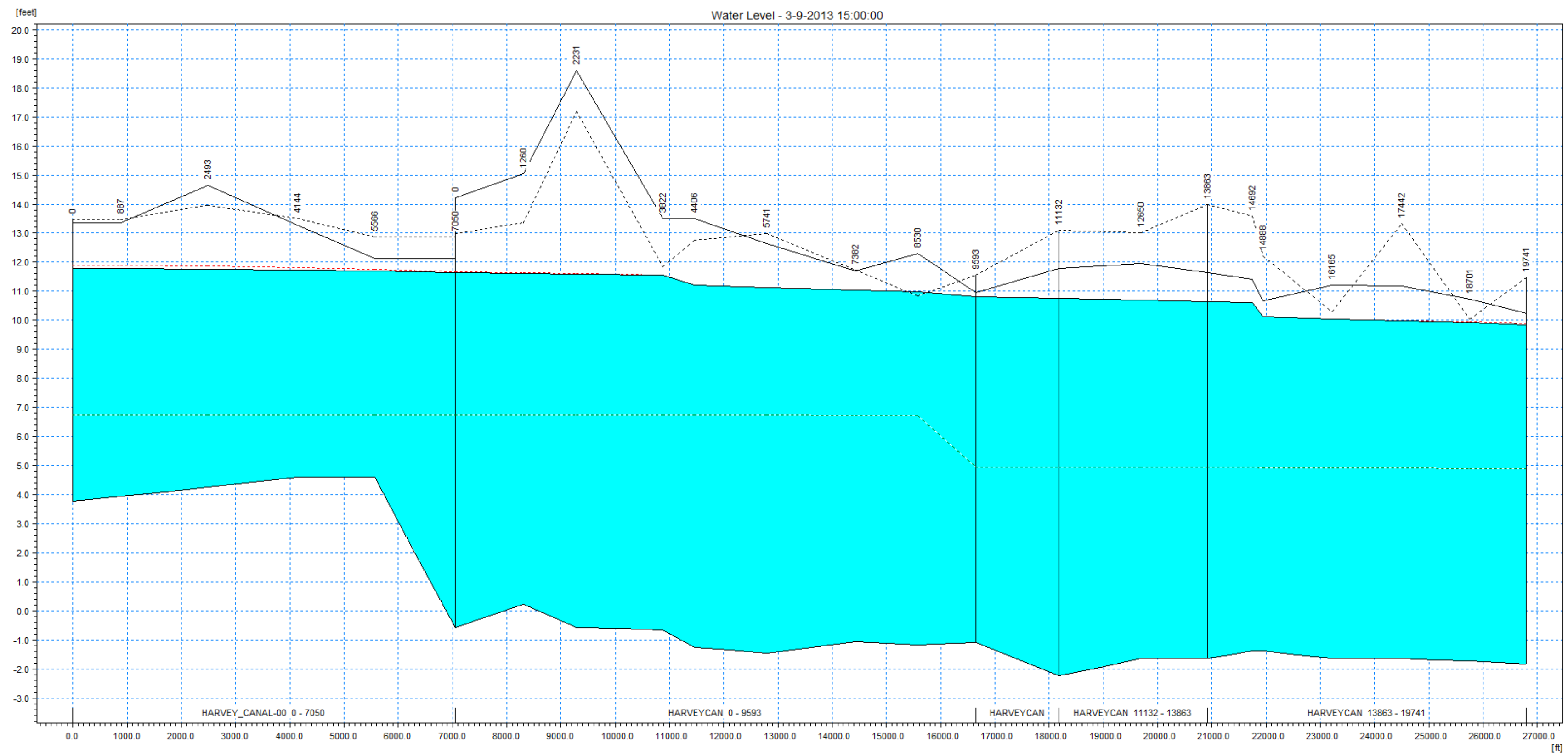

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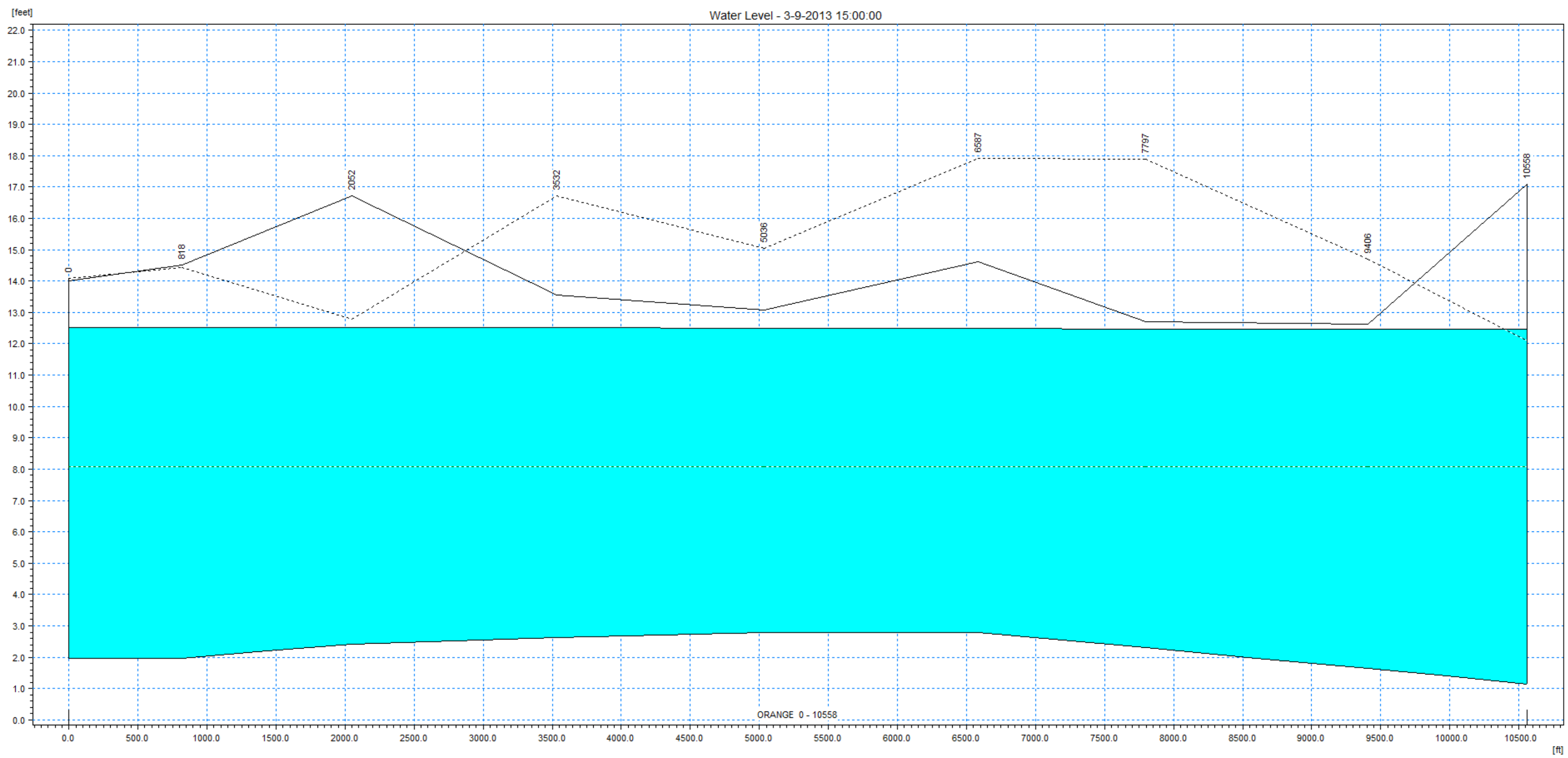
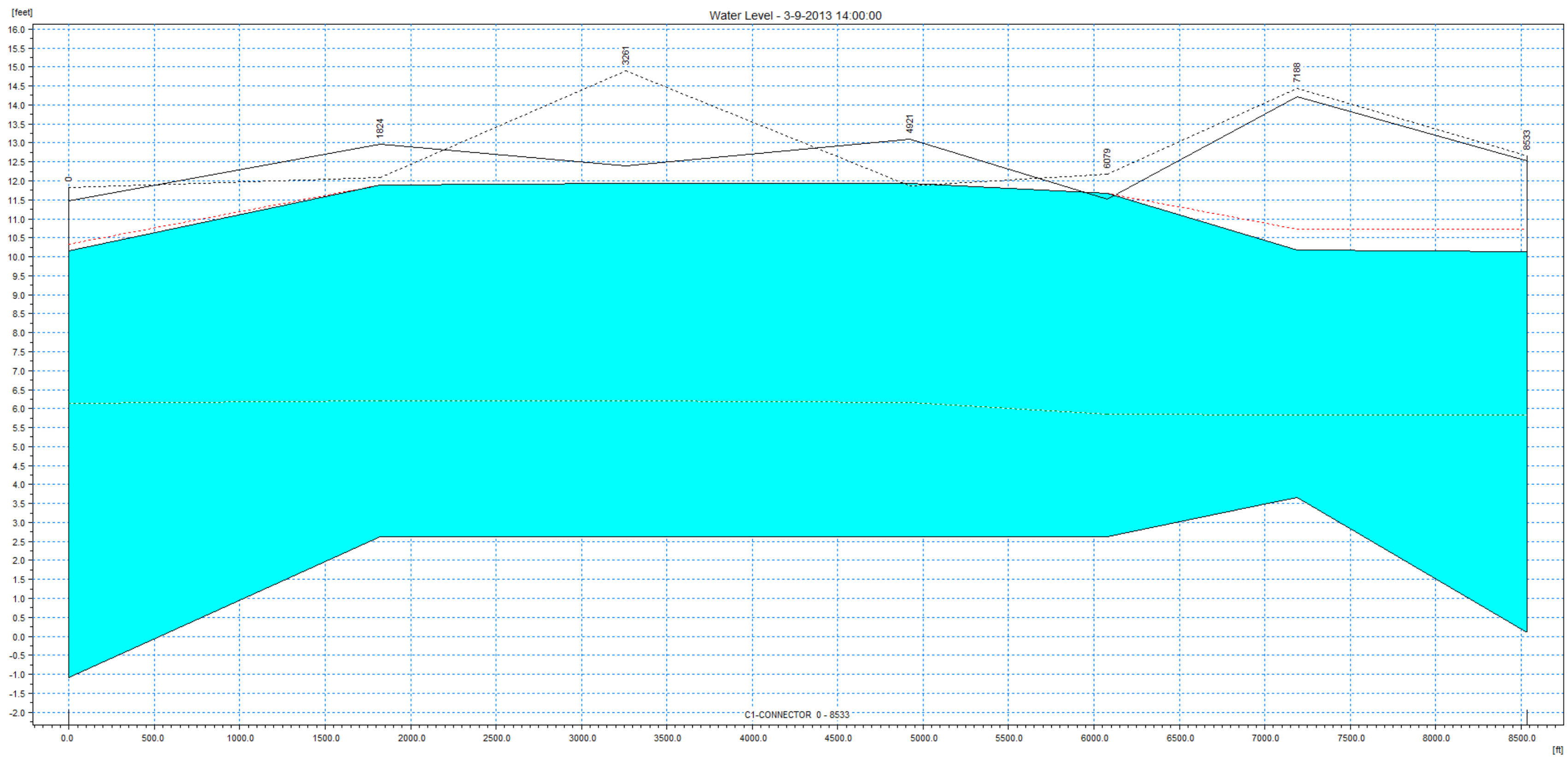


Orange Canal

Part 7 25-year Future Conditions SLR3



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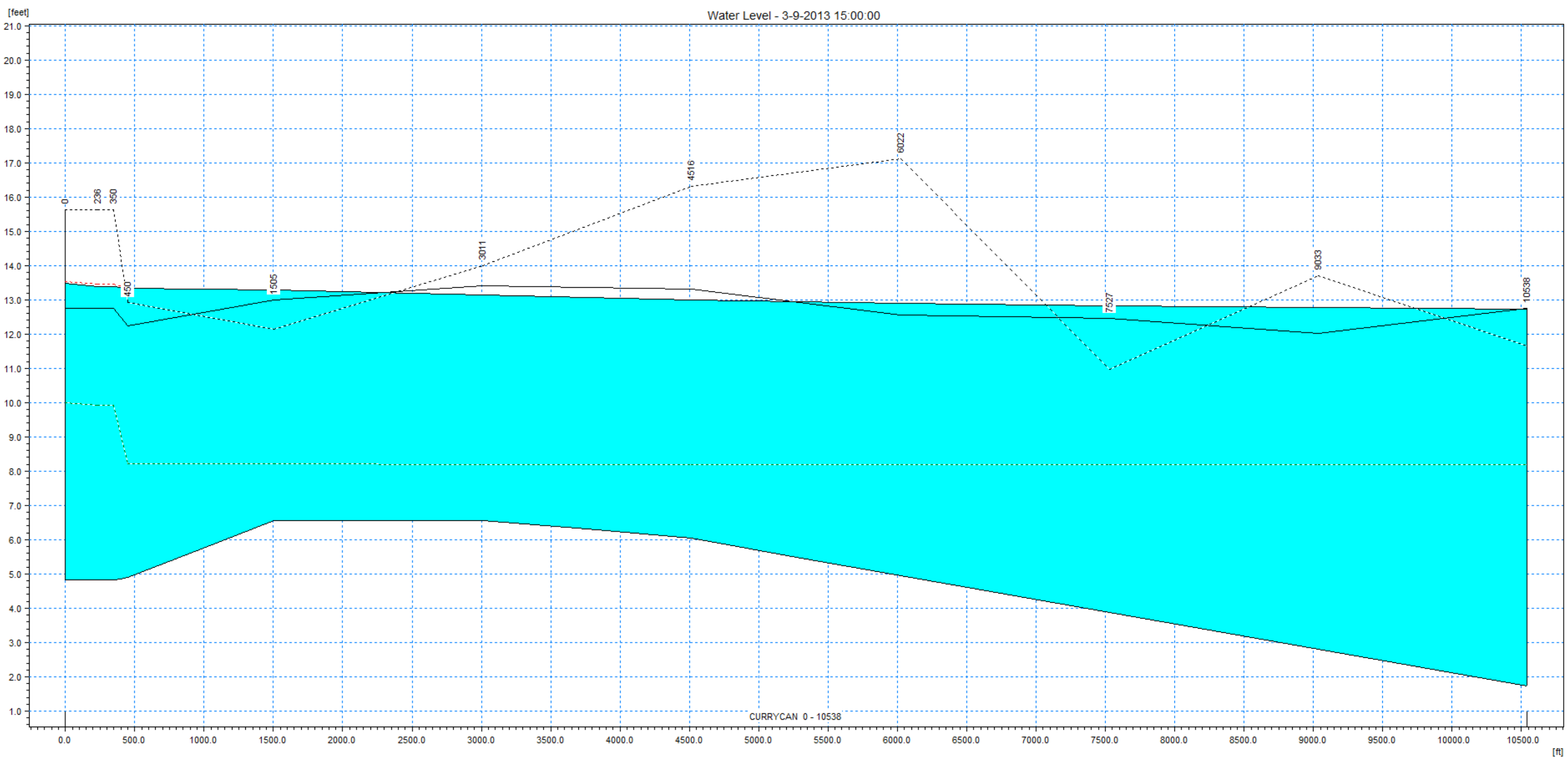

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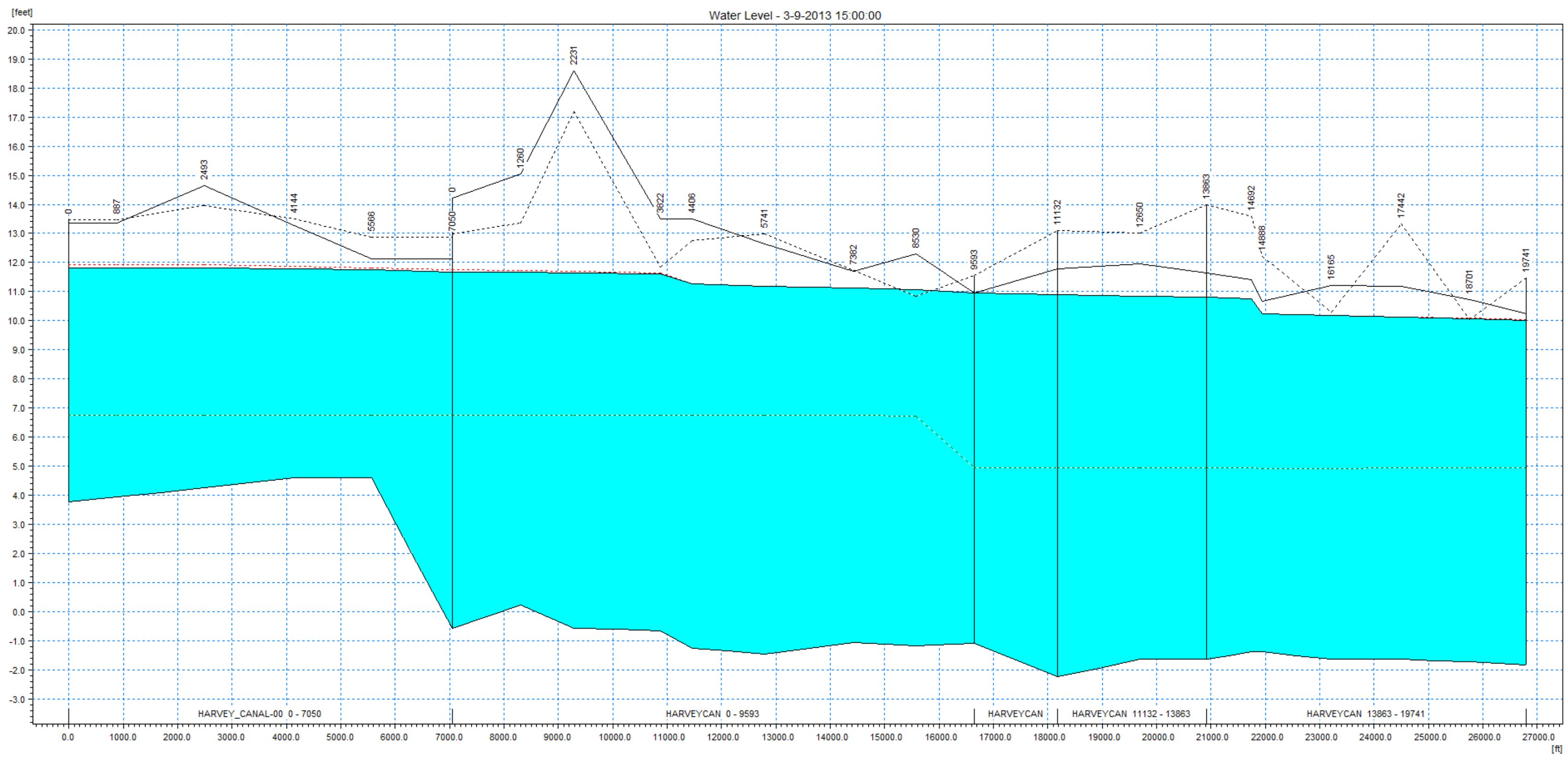

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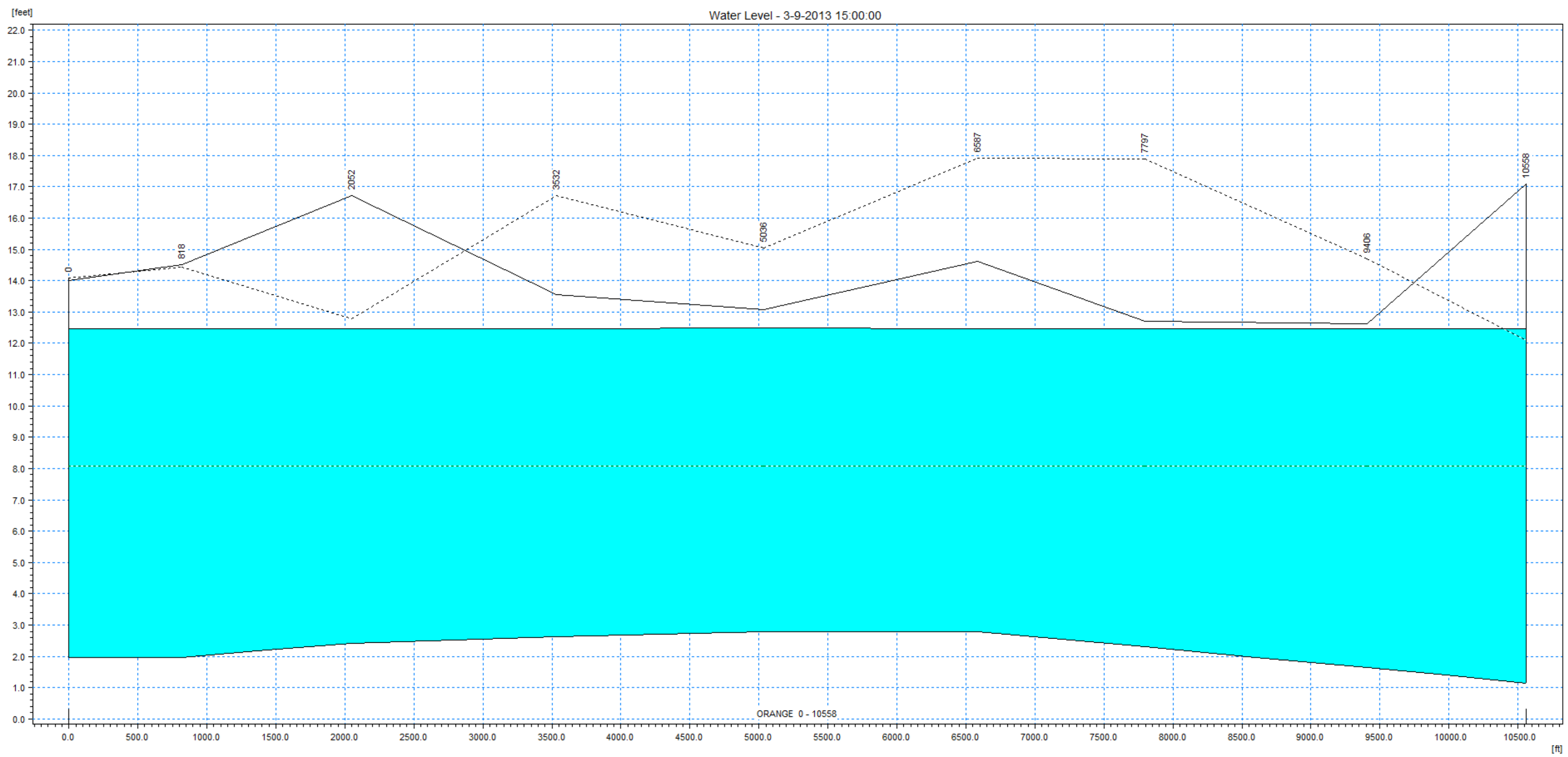

Orange Canal

Flood Protection Level of Service Provided by Existing District Infrastructure for Current (2015) Sea Level Conditions and Future (2065) Scenarios for Cocohatchee, Henderson/Belle Meade, and Faka Union Watersheds – Final Report

FLOOD PROTECTION LEVEL OF SERVICE FOR BIG CYPRESS BASIN: CURRENT AND FUTURE SERVICE IN GOLDEN GATE, COCOHATCHEE, HENDERSON-BELLE MEADE, AND FAKA UNION WATERSHEDS

CONTRACT 4600003096
Work Order 06 – Deliverable 4.4.2

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September, 2018

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8 September 2018

9/8/18

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Annex A

Performance Measure #3 Evaluations for COCO1, GG1, and HC1, prepared by Lichun Zhang, SFWMD:

- THE EFFECTS OF SEA LEVEL RISE ON COCO1 PERFORMANCE
- THE EFFECTS OF SEA LEVEL RISE ON GG1 PERFORMANCE
- THE EFFECTS OF SEA LEVEL RISE ON HC1 PERFORMANCE

1.0 Introduction

The South Florida Water Management District (District) is conducting a system-wide review of its regional water management infrastructure to determine the flood protection level of service (FPLOS) currently provided. The FPLOS describes the level of protection provided by the water management facilities within a watershed under both current and future conditions, where future conditions FPLOS considers sea level rise and future development. This information can be used by local governments, the SFWMD, and other state and federal agencies to identify areas where improvements or upgrades of water management facilities are required, the appropriate entity or entities responsible for making improvements, and funding and technical resources available to support these efforts.

This Final Report is the last in a series of documents describing the flood protection LOS within the Big Cypress Basin (BCB). The four watersheds within the BCB Study Area, along with the primary canal network, are depicted on **Figure 1.1.** The first and second draft reports described the current and future conditions FPLOS, respectively, within the Golden Gate Watershed. These two reports were combined into a Final Report for the Golden Gate Watershed in October 2017. The third and fourth draft reports described the current and future conditions FPLOS within the Cocohatchee, Henderson/Belle Meade, and Faka Union Watersheds. This Final Report combines the information in the third and fourth draft reports and addresses the District's comments on those two reports. As such, this report is limited to a description of FPLOS within the Cocohatchee, Henderson/Belle Meade, and Faka Union Watersheds.

Portions of the Faka Union Watershed and the Henderson/Belle Meade Watershed were not evaluated as part of this effort, due to the on-going implementation of the Picayune Strand Restoration Project (PSRP). The project area of the PSRP covers the southern portion of the Faka Union Watershed. Within the project area, the majority of the District's flood control infrastructure will be abandoned via the plugging of the Faka Union, Miller, and Merritt Canals downstream of the pump stations currently under construction. Also associated with the PSRP is a feature designed to hydraulically isolate the 6Ls Agricultural Area from the effects of the higher stages expected within the PSRP area, known as the Southwest Protection Feature. The 6Ls Agricultural Area was therefore also excluded from the FPLOS evaluation.

The flood protection LOS is determined through several metrics, the majority of which are derived from the outputs of watershed-scale flood event modeling. The flood protection metrics are defined in **Sec. 2**.

Previous tasks completed as part of this study effort include model conceptualization, model calibration, and model verification. The model tool used for this study is a MIKE SHE / MIKE-11 model originally developed by others for long-term simulations. Because the flood protection LOS performance metrics rely on model outputs resulting from synthetic short-term high intensity rainfall events, it was necessary to re-conceptualize certain aspects of the model, and to then re-calibrate the model using data from short-term high intensity rainfall events of record. The model calibration/verification effort is documented in the April 2017 report titled *Deliverable 2.5 Model Recalibration*. The re-calibrated version of the BCB MIKE SHE / MIKE-11 model is referred to herein as Version EC-Cal, dated March 2017, was used for the subsequent design storm simulations described in **Section 3**. The EC-Cal model was subsequently updated to improve representation of the Henderson Creek Canal (under Task 2.6) in May 2018, in response to observations made during Hurricane Irma (ref: K. Feng and J. Nageon de Lestang, "*Rainfall Characteristics and Peak Water Levels Along Primary Canals in BCB During Hurricane IRMA",* SFWMD, July 2018).

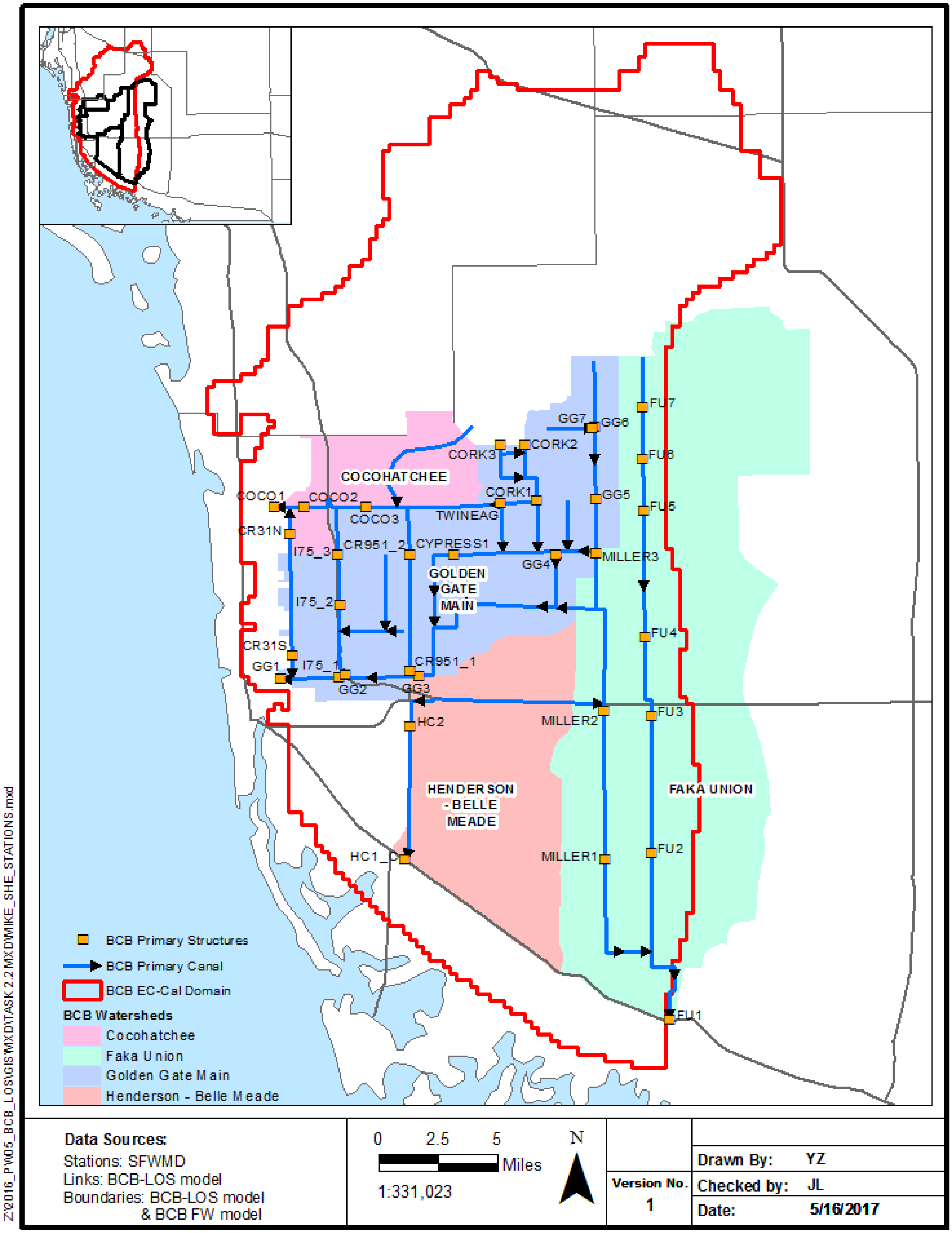

Figure 1.1 – BCB Watersheds, Primary Canals, and Primary Structures

2.0 Flood Protection LOS Performance Metrics

The District relies on six (6) formal performance metrics (PMs) to evaluate the flood protection LOS provided by the primary water management infrastructure. These metrics, defined briefly in this section, were derived from the District publication *Flood Protection LOS Analysis for the C-4 Watershed, Appendix A: LOS Basic Concepts* (SFWMD H&H Bureau, December 29, 2015). **Section 4.0** provides the results of the FPLOS evaluation for current conditions, and **Sections 6.0** and **7.0** provide the results of the FPLOS evaluation for future conditions within the Cocohatchee, Henderson/Belle Meade, and Faka Union Watersheds.

<u>PM #1 Maximum Stage in Primary Canals</u> – This is the peak stage profile in the primary canal system. The profile is developed for a range of design storms (5-year, 10-year, 25-year, and 100-year). The largest design storm that stays within the canal banks establishes the FPLOS of the primary canal system.

<u>PM #2 Maximum Daily Discharge Capacity through the Primary Canals</u> – PM #2 is the maximum discharge capacity throughout the primary canal network. Discharge is calculated as areally weighted flow, in units of cubic feet per second per square mile of contributing area for the 25-year design event. Tidal effects are filtered by using a 12-hour moving average of discharge. Although the peak of the 25-year net discharge hydrographs are referred to in this report as the calculated discharge capacity, the true capacity of the canal segment is the net discharge corresponding to the largest design flood event that remains within the banks of the canal using the results of the 5-year, 10-year, and 100-year events.

<u>PM #3 – Structure Performance – Effects of Sea Level Rise</u> – This metric shows the effective capacity of a tidal structure. It is comparable to the static design condition assumed in the original design but compares structure flow over a range of storm surge events and a range of sea level rise scenarios. For the BCB FPLOS evaluation, this metric was evaluated internally by District staff and documented separately in three reports attached as **Annex A**.

<u>PM #4 Peak Storm Runoff – Effects of Sea Level Rise</u> – PM #4 is the maximum conveyance capacity of a watershed at the tidal structure for a range of design storms. It shows the maximum conveyance (moving 12-hour average) for a specific design storm and a specific tidal boundary condition. This metric examines the behavior of the system under severe stress and can be used to check if conditions exceed design limits. In evaluating this PM, it is assumed that design rainfall and design storm surge occur simultaneously or with a temporal offset that maximizes stress on the structure.

<u>PM #5 Frequency of Flooding – Stage-based FPLOS for Subwatersheds</u> – In this PM, the flood elevations or depths of overland flooding are evaluated for a range of design storms (5-year, 10-year, 25-year, and 100-year). These flood depths/elevations can then be compared with elevations of build features such as buildings and roadways, where such information exists. For the purposes of this BCB FPLOS evaluation, flood inundation maps were developed from the model output for each storm event.

<u>PM #6 Duration of Flooding</u> – PM #6 quantifies the duration of flooding at specific locations of interest within a watershed. For this Study, the length of time the flood elevation is projected to be above a threshold depth was mapped over the entire study area using the gridded model output files for the 2-D overland flow component.

3.0 Current Conditions Application Model Set-Up

The re-calibrated BCB MIKE SHE / MIKE-11 model, described in the April 2017 report titled *Deliverable 2.5 Model Recalibration* (as updated in May 2018) and referred to herein as Version EC-Cal, was used as the basis for the design storm simulations described in this section. Several changes to the EC-Cal model setup were required to simulate the synthetic design storm events. These changes included structure operations, tailwater boundary conditions, rainfall, and initial conditions.

Model Parameters and Structure Operations: The current conditions model application incorporated all applicable changes to the model setup and parameterization documented in the Model Recalibration Report. These include hydrologic parameter adjustments (i.e., drain codes, drain levels, drain time constants, detention storage, and paved runoff coefficients), hydraulic parameter adjustments (manning's roughness coefficients), and addition of MIKE-11 branches in the Cocohatchee Canal subwatershed. All recorded structure operations from the calibration model have been replaced with rule-based operations, based on the structure descriptions in the District's *Water Control Operations Atlas: Big Cypress Basin System* dated September 21, 2016 and updated March 31, 2017. The Miller 3 structure was updated to represent the post-2015 configuration (with 3 4x8 dual-leaf gates), as described in the most recent Water Control Operations Atlas. The pump stations associated with the ongoing Picayune Strand Restoration project (on the Miller and Faka Union Canals) are not included in the current conditions model but are included in the future conditions modeling of the Faka Union system.

Within the Henderson Creek Canal, several changes were made to the MIKE-11 hydraulic model input for Henderson Creek, including improved culvert data and channel geometry. The changes are documented in Task 2.6 – *Model Adjustments and Validation in Henderson Creek Watershed Technical Memorandum*, Taylor Engineering, Inc., May 2018.

Tailwater Boundaries at Coastal Outfall Structure GG1: Tailwater hydrographs for use in the design storm simulations are documented in a report prepared by the District (ref: South Florida Water Management District, 2017. Flood Protection Level of Service Analysis for the Big Cypress Basin. Appendix C: Preparation of Boundary Conditions at the Tidal Structures. H&H Bureau, SFWMD, West Palm Beach, FL. 34 pp. June 21, 2017). The accompanying spreadsheets included existing conditions storm surge hydrographs for the 5-year, 10-year, 25-year, and 100-year return intervals. Also included were storm surge hydrographs for future sea level rise. for the current conditions model, the columns in the spreadsheet designated "YEAR2015/IPCCAR-MEDIAN", were used, which represent current sea level. **Table 3.1** and **Figures 3.1** and **3.2** show the peak tailwater levels and water level time series, respectively, for each return interval for the COCO1 and HC1 structures.

Tailwater levels at the FU-1 structure were left unchanged from the calibration model. As discussed in meetings with District Staff, the FPLOS evaluation will not include the southern portion of the Faka Union Watershed (i.e., areas downstream of the MILLER2 and FU3 structures), as this area is part of the Picayune Strand restoration project. District infrastructure within the restoration project area no longer serves a flood protection purpose.

Table 3.1 – Peak Tailwater Stages for Current Sea Level at COCO1 and HC1

Return Period	Current Sea Level Conditions Peak Tailwater Stage at COCO1 (ft. NAVD88)	Current Sea Level Conditions Peak Tailwater Stage at HC1 (ft. NAVD88)
5-year	4.93	4.89
10-year	5.33	5.32
25-year	5.89	5.88
100-year	6.81	6.79

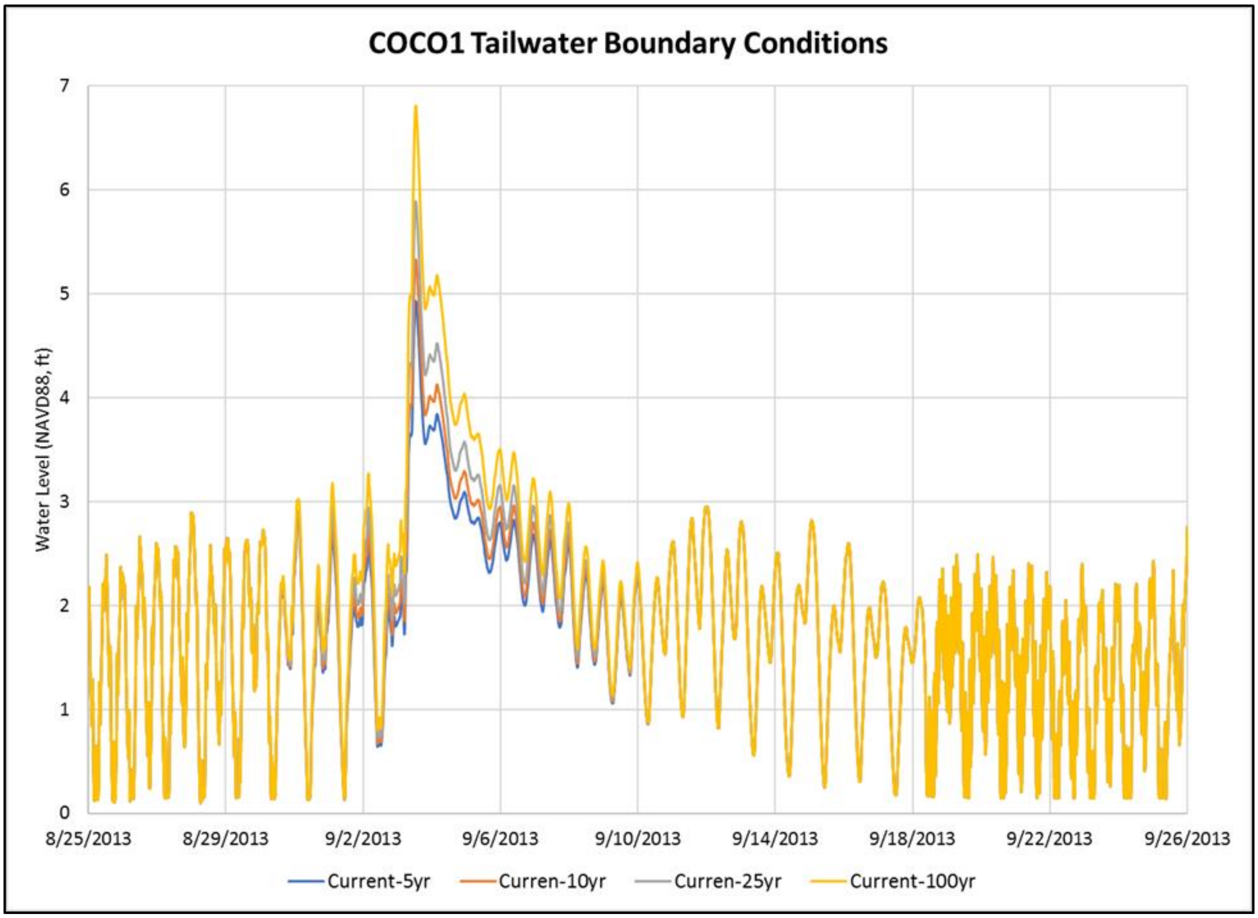

Figure 3.1 – COCO1 Tailwater Boundary Conditions for Current Sea Level

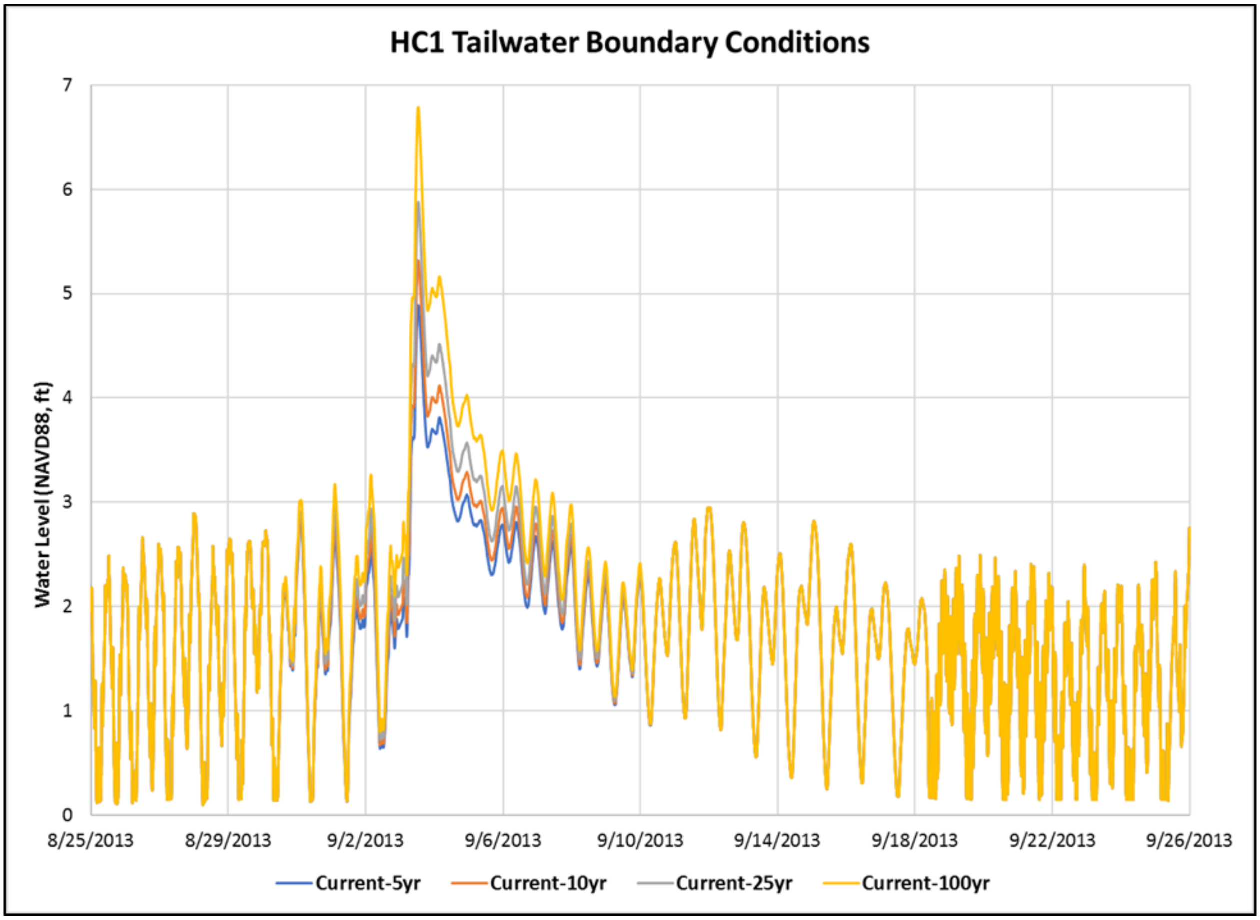

Figure 3.4 - HC1 Tailwater Boundary Conditions for Current Sea Level

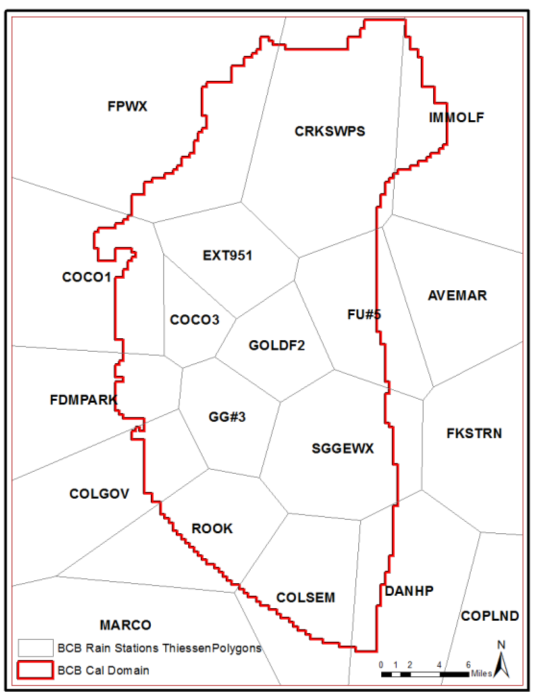

Figure 3.3 – Rainfall Polygons

Rainfall: In the calibration models, the rainfall was distributed on the 2 km x 2 km NEXRAD radar rainfall grid. For the design storm simulations, this was replaced with a Thiessen-polygon approach, identical to the approach used for the design storm runs in the BCB-FW model developed by Lago Consulting in 2015. The centroid of each polygon corresponds to a rainfall gage location. Rainfall 3-day distribution and totals for each return period were based on the SFWMD Environmental Resource Permit Information Manual Volume IV, Water Resource Regulation Department (July 2010 version). Rainfall totals varied from polygon to polygon based on the position of each centroid relative to the isohyets published in the Manual. Rainfall depths for each Thiessen polygon are listed in **Table 3.2**.

Table 3.2 – Total Design Storm Rainfall Depths

Rainfall Gage	5-year	10-year	25-year	100-year
COCO1	7.69	9.24	11.33	13.98
COCO3	7.64	9.14	11.09	13.69
COLGOV	7.75	9.63	11.81	14.75
COLSEM	7.68	10.10	12.51	15.91
CRKSWPS	7.37	8.55	9.74	11.84
EXT951	7.60	9.04	10.93	13.28
FKSTRN	6.76	8.47	9.92	13.37
DANHP	7.52	10.03	12.23	15.86
GOLDF2	7.57	9.02	10.94	13.68
IMMOLF	6.65	7.76	8.70	10.71
MARCO	7.81	10.20	12.85	16.10
ROOK	7.75	9.94	12.08	15.37
SGGEWX	7.55	9.13	11.05	14.28
AVEMAR	6.87	8.08	9.29	11.70
FPWX	7.56	9.03	10.73	13.25
GG#3	7.66	9.32	11.33	14.27
COPLND	7.18	9.90	11.97	15.68
FU#5	7.32	8.58	10.02	12.30
FDMPARK	7.74	9.48	11.65	14.43
Area-Weighted 3-Day Rainfall Depth (inches)	7.50	9.10	10.90	13.60

The dates for the rainfall distributions were adjusted so that the peak rainfall intensity lined up with the peak tailwater stage, per the Scope of Work. This resulted in a spin-up period of approximately one week with normal tidal boundaries and no rainfall, and with the peak rainfall intensity and peak tailwater stage occurring on September 3, 2013 between noon and 1 PM.

Initial Groundwater Levels and Surface Water Depths: As described previously for the Golden Gate Watershed setup, the results of the BCB-FW 7-year simulation (2008 through 2014) in conjunction with the stand-alone “DFSpercentiles” tool were used to calculate the 80th percentile water level in each model grid cell and within each groundwater layer. The resulting grid files were then used as the starting groundwater elevations. The 80th percentile overland water depths were also computed and used as the starting overland water depth.

4.0 Flood Protection Level of Service – Current Conditions

Once all model setup changes were completed to represent design storm conditions, the model was executed for the 5-year, 10-year, 25-year, and 100-year 3-day storm events. Model results were evaluated for stability and reasonableness prior to proceeding with the FPLOS evaluation. **Appendix A** provides summary model results at primary control structures, while **Appendices B** through **E** provide the complete flow and stage hydrographs for each of the four design storm events. The remainder of this section describes the results of the FPLOS evaluations for all relevant performance metrics, which for current conditions include PM #1, PM #2, PM #5, and PM #6.

4.1 PM #1 – Maximum Stage in Primary Canals

This is the peak stage profile in the primary canal system. The profile is developed for a range of design storms (5-year, 10-year, 25-year, and 100-year). The largest design storm that stays within the canal banks establishes the FPLOS of the primary canal system.

To evaluate this PM under current conditions with the three Watersheds, instantaneous peak stage profiles were prepared for all primary canals within or bordering the three watersheds. Bank elevations on the profile figures are generally based on the MIKE-11 cross section data. However, in several cases the bank elevations appeared suspect and were later modified (based on the current LiDAR data) for use in preparing the profile plots. Also shown in the Figures are major roadway landmarks, control structures, and primary canal junctions.

Table 4.1 summarizes the PM #1 Results shown graphically on **Figures 4.1.1** through **4.1.4**, listing the maximum return period profile that is contained within the canal banks. Although all the canals contained the 10-year and 25-year profiles along the majority of the bank lengths, the bank elevation was exceeded for the 10-year event over short segments in multiple locations for all canals. Therefore, none of the canal segments evaluated under this PM technically provide greater than a 5-year FPLOS, using a strict interpretation of this criteria and on a localized basis. Overall, however, the canals all provided a 10- to 25-year FPLOS when considering the other PMs discussed later in this report, as summarized in the Conclusions Section (**Section 8**).

Table 4.1.1 – PM #1 Summary Results

Canal Segment	Figure Number	FPLOS - Localized	FPLOS - Overall	Comment
Cocohatchee	**4.1.1**	5-year	10- to 25- year	Overall FPLOS from Section 8.1.1
Henderson	**4.1.2**	5-year	25-year	Overall FPLOS from Section 8.1.2
Faka Union	**4.1.3**	<5-year	10-year	Overall FPLOS from Section 8.1.3
Miller	**4.1.4**	5-year	10-year	Overall FPLOS from Section 8.1.3

During review of the draft report, District staff pointed out that **Figure 4.1.1** shows the highest stage upstream of the COCO4 structure, although the watershed divide is considered to be a mile or two west of the COCO4 structure. Examination of the model output revealed that at COCO4, flow direction is to the east for the vast majority of the simulation. During the peak of the 25-year and 100-year events, there is a brief flow reversal, when flow is to the west for a brief period**,** as discussed in the Interbasin Flows discussion in **Section 4.2**. During this brief period, the divide shifts to the east of COCO4.

The majority of the Henderson Creek Canal provides a 25-year level of service with respect to PM #2, with the exception being a low-lying area along the west bank of the canal just upstream of US 41. It is noted that at the downstream end of the Henderson Creek Canal, the peak tailwater elevation at HC1 is slightly higher than the peak headwater elevation (**Figure 4.1.2**). This is due to the time lag of the peak flow from rainfall-generated runoff, which comes after the peak of the storm surge and the associated reverse flow. Instantaneous peak water levels were used to develop the flood profiles (as opposed to concurrent), and when the simulated peak rainfall-generated flood wave reaches the structure, the imposed boundary tailwater level has fallen and thus and the peak stage from the second (runoff) flood wave is lower than the initial surge.

In the Faka Union Canal, significant flooding is predicted to occur along the canal upstream (and over) Randall Boulevard upstream of FU5 (**Figure 4.1.3**). According to the model results, the Randall Boulevard bridge itself constitutes a significant restriction to flow. Water levels for all events, including the 5-year, are predicted to exceed the low chord elevation of the bridge. Overtopping is predicted for the 10-year, 25-year, and 100-year events. According to District staff, this is an area of known flooding.

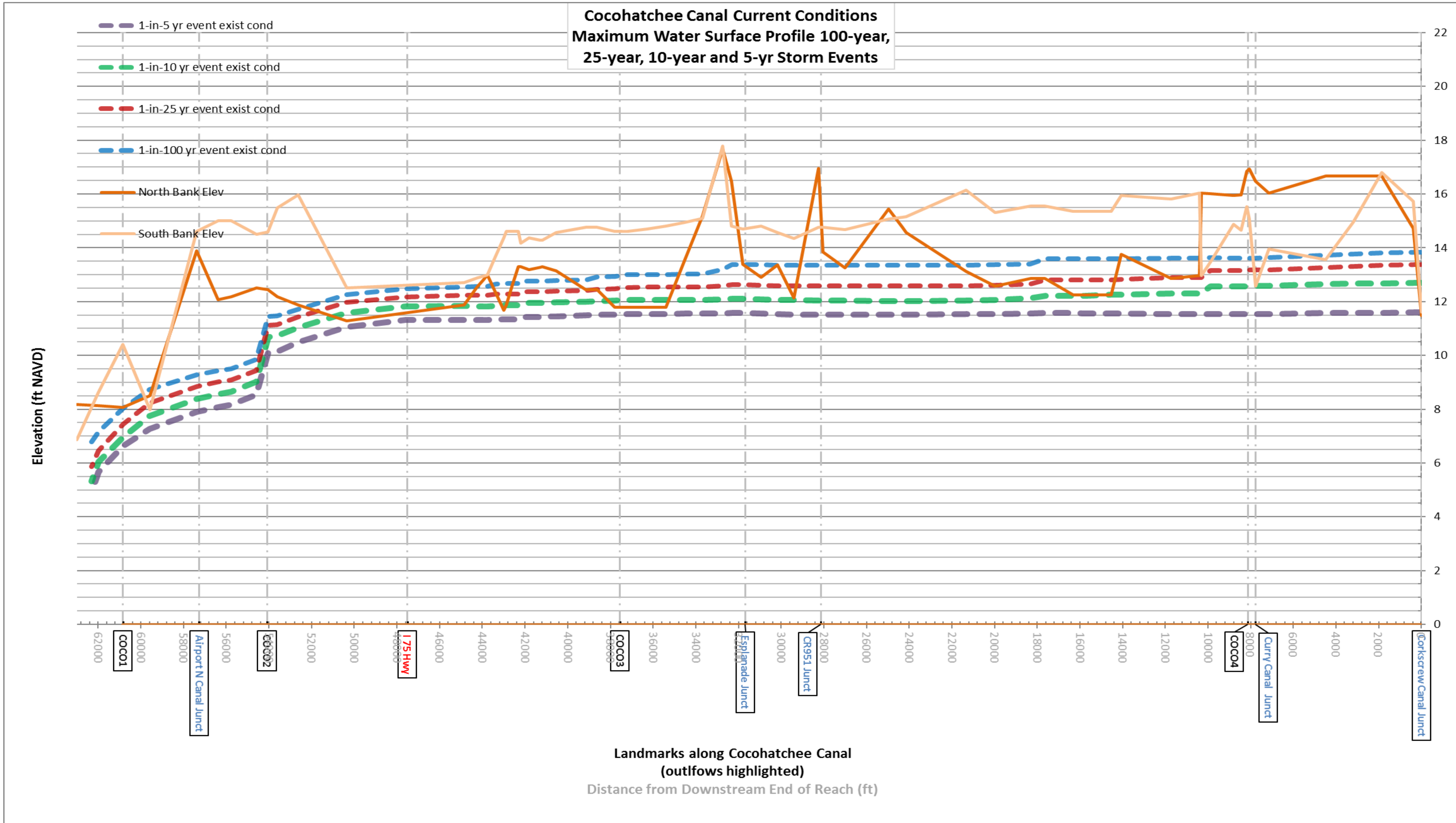

Figure 4.1.1 – Cocohatchee Main Peak Stage Profiles

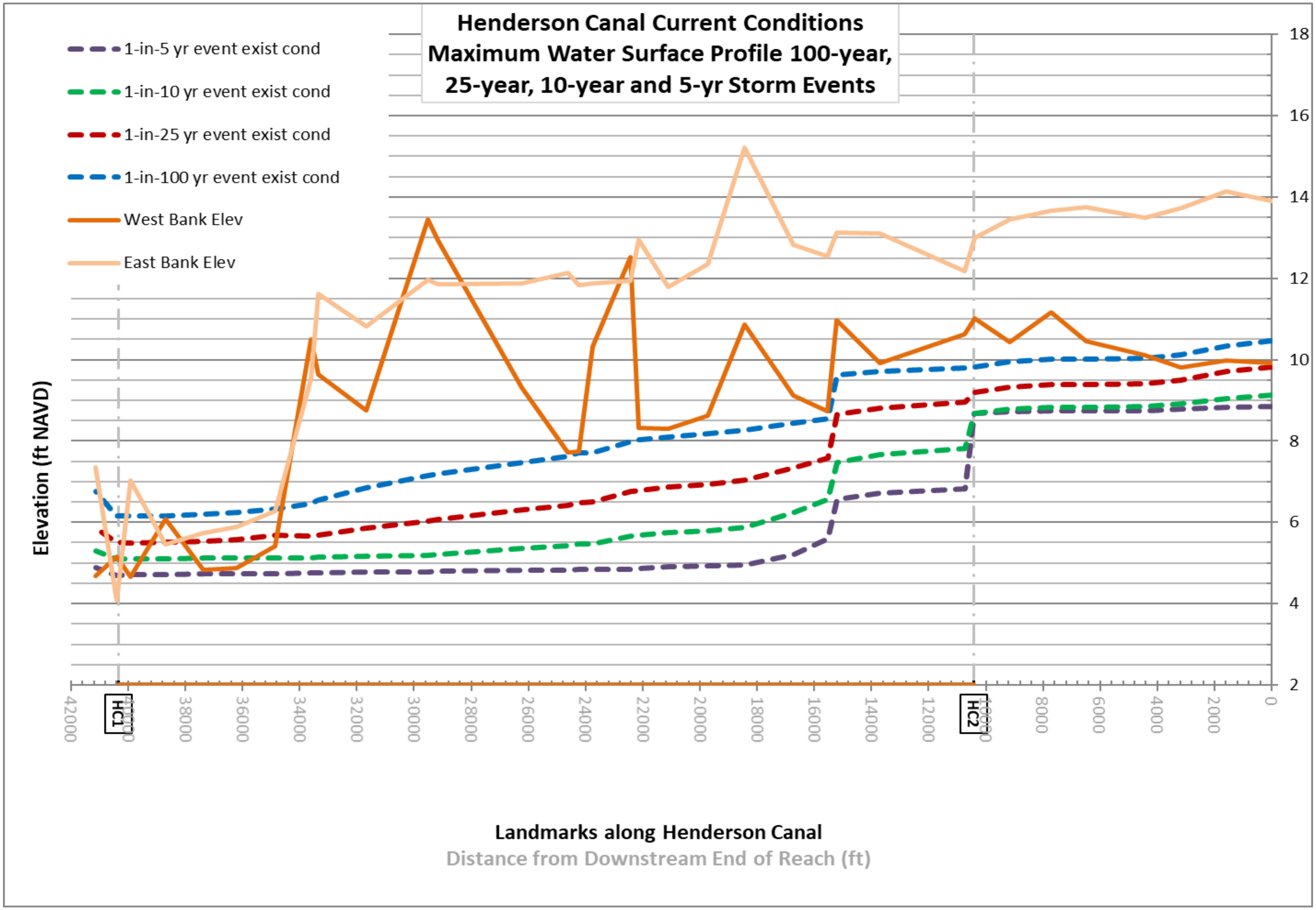

Figure 4.1.2 – Henderson/Belle Meade Canal Peak Stage Profiles

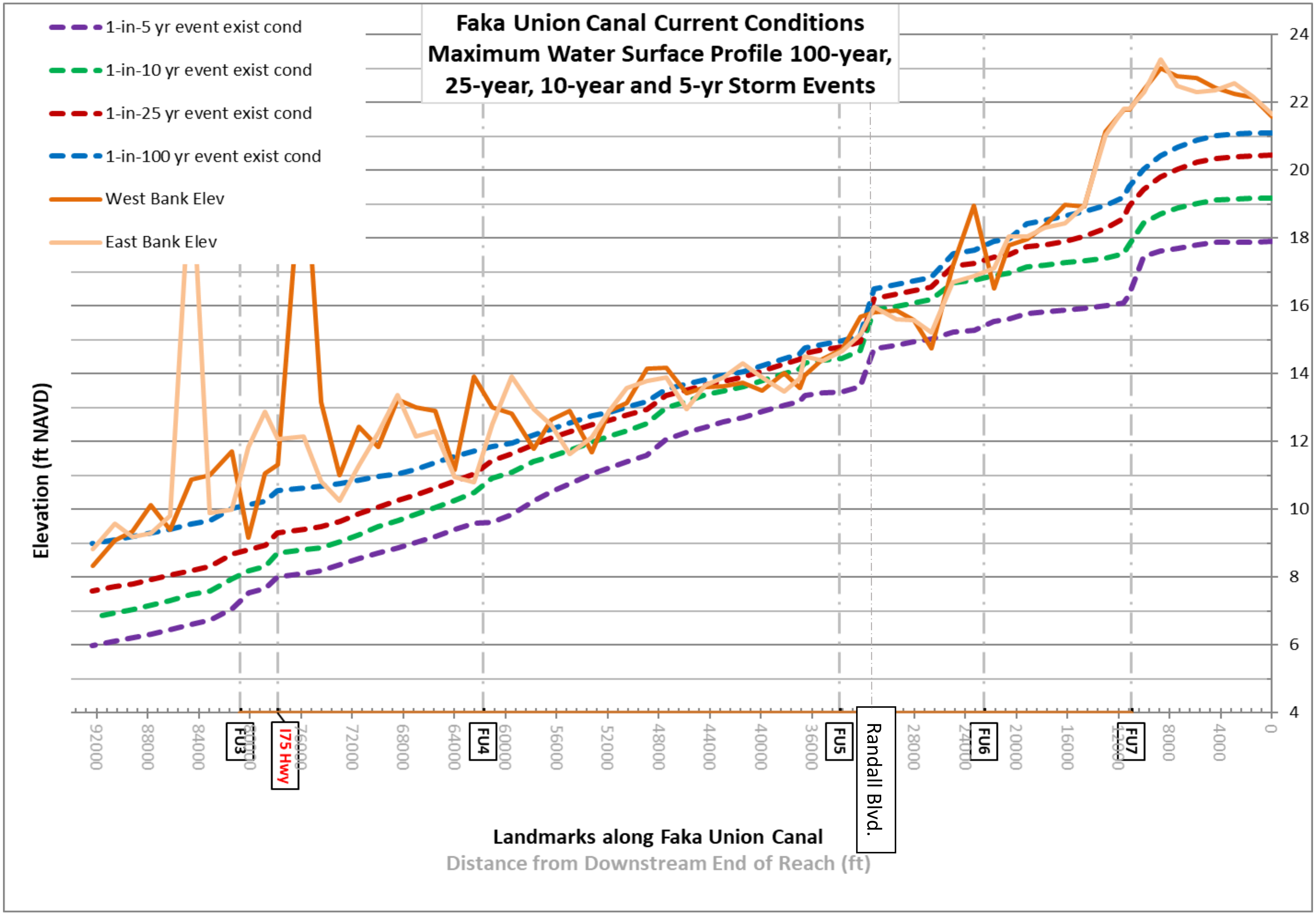

Figure 4.1.3 – Faka Union Canal Peak Stage Profiles

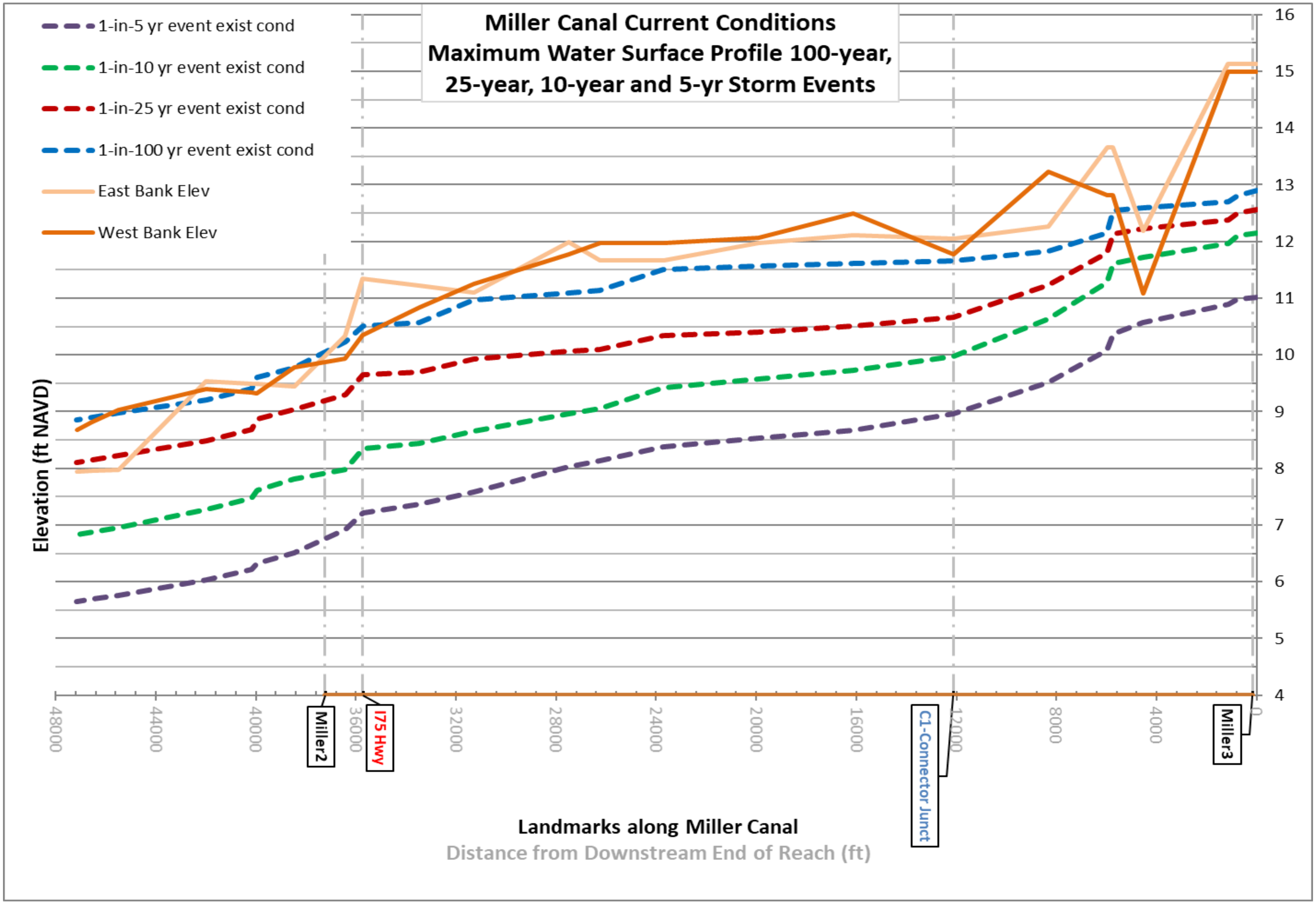

Figure 4.1.4 – Miller Canal Peak Stage Profiles

4.2 PM #2 – Maximum Daily Discharge Capacity through the Primary Canals

PM #2 is the maximum discharge capacity throughout the primary canal network. Discharge is calculated for defined canal segments as areally weighted flow, in units of cubic feet per second per square mile of contributing area. Canal segments are generally those segments between water control structures. for example, the segment associated with structure HC1 is the Henderson/Belle Meade Canal between structures HC1 and HC2, and the contributing area is defined as only the area contributing runoff to that segment. **Table 4.2.1** lists the canal segments identified for this analysis. The table also identifies the contributing area for each canal segment, and the discharge capacity calculated for each segment associated with the 25-year, 3-day design storm event.

Discharge capacity was calculated by subtracting the hydrographs at all inflow points to each segment from the segment's outflow hydrograph, and then dividing the peak of the resulting net discharge hydrograph by the segment's contributing area. Tidal effects were filtered by using a 12-hour moving average of discharge.

Table 4.2.1 – Water Control Catchment Inflow and Outflow Points and 25-year Discharge Capacity

Structure/Segment	Inflow	Outflow	Water Control Catchment Area (sq.mi)	25-year Peak Discharge Capacity (cfs/sq.mi)
HC1	HC2	HC1	4.81	86.67
HC2	BEGINNING OF HENDERSON	HC2	3.07	14.29
COCO1	Airport N, COCO2	COCO1	3.37	99.83
COCO2	COCO3	COCO2	6.08	79.62
COCO3	BEGINNING OF COCOHATCHEE, BEGINNING OF CR951	COCO3	17.03	25.32
FU7		FU7	4.09	48.35
FU6	FU7	FU6	3.90	53.67
FU5	FU6	FU5	4.81	39.18
FU4	FU5	FU4	11.66	46.80
FU3	FU4	FU3	8.39	41.99
Miller2	Miller3, C-1 Connector	Miller2	11.93	50.11

Figure 4.2.1 shows the contributing areas ("water control catchments") draining to each canal segment. For completeness, the figure also includes the Golden Gate Watershed segments previously reported in the Task 4.4.1 Deliverable. The catchment polygons were taken from the District's Arc Hydro Enhanced Database (AHED), and were used to determine the contributing areas listed in **Table 4.2.1.** One exception to this is the contributing area for HC1, which was found to include areas draining under US41 via other structures including the TAMIHEND structure. The catchment area was revised to include only the areas draining to HC1.

The numerical values on **Figure 4.2.1** are the discharge characteristics (D.C.) for each subwatershed in the BCB. It is important to note that the D.C. values shown do not reflect the conveyance of a full canal, rather they show the subwatershed's ability to drain. Low drainage can have several causes. For example, the low D.C. values in subwatersheds adjoining Corkscrew Swamp are caused by uncontrolled inflows from the swamp. In these subwatersheds the low values reflect extensive flooding, as seen in other performance metrics #1 (canal profile) and #5 (flood depths). In contrast, the low D.C. value in the Airport Road system is caused by a low water table that allows for greater infiltration. This low value is not a source of concern, as seen in PM#1 and PM#5. The low values in HC2 and GG3 subwatersheds are a result of low discharge rates. The low discharge in HC2 is caused by an undersized culvert downstream of the HC2 structure. The low discharge in GG2 is an undersized canal system that could be improved by expanding the C1 connector canal.

Figure 4.2.2 through **Figure 4.2.12** graphically depict the net area-weighted discharge hydrographs for each canal segment, and for each design storm event (5-year, 10-year, 25-year, and 100-year). Negative values on the graphs correspond to times where total inflows exceeded total outflows for each segment. The negative net discharge of segments associated with FU3, HC1 and HC2 result from a difference in timing of peak flows as the main flood wave moves downstream through the canal network.

Of all the control structures in the Cocohatchee, Henderson, and Faka Union watersheds, HC2 is shown to have the smallest discharge capacity of just over 14 CFS per square mile. The structure capacity is limited in part by high tailwater conditions resulting from undersized culverts in the canal approximately one mile downstream of the structure.

Although the peak of the net discharge hydrographs for each design storm event are referred to in this section as the calculated discharge capacity for each event, the true capacity of the canal segment is the net discharge corresponding to the largest design flood event that remains within the banks of the canal. for this reason, the results of PM#2 must be evaluated in conjunction with the results of PM#1 (Maximum Stage in Primary Canals) and PM#5 (Frequency of Flooding). From the PM#1 results presented in the previous section, peak stages in all canals exceed the canal banks for the 100-year event. In many cases, a 10-year event is sufficient to cause water levels to exceed the canal banks (Refer to PM#1 **Figures 4.1.1, 4.1.2,** and **4.1.4**), but these generally appear to be localized flooding instances that do not extend far from the canal banks. This is based on an examination of the PM#5 results, which are discussed in the next section.

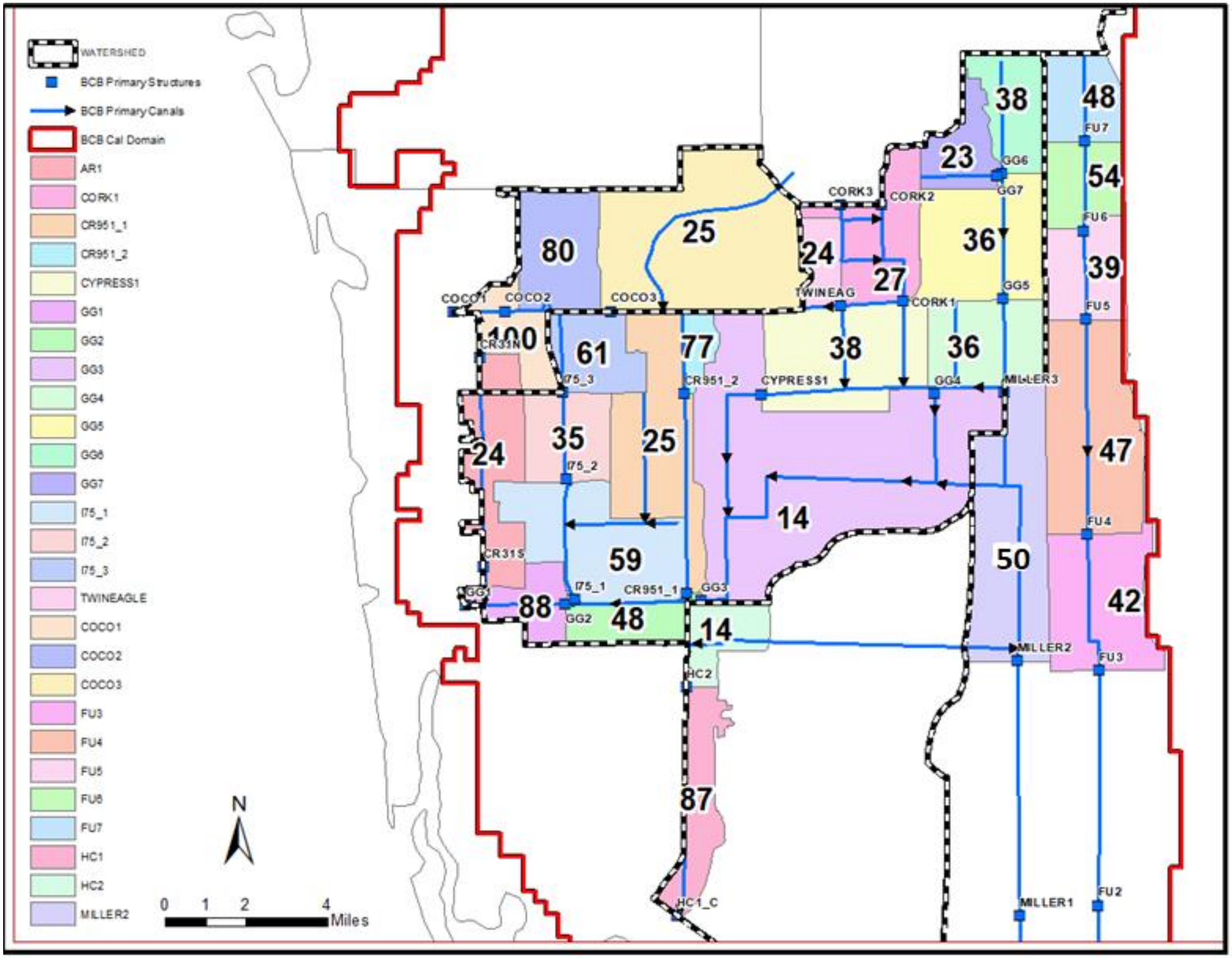

Figure 4.2.1 – Control Structure Catchments for Calculating PM#2

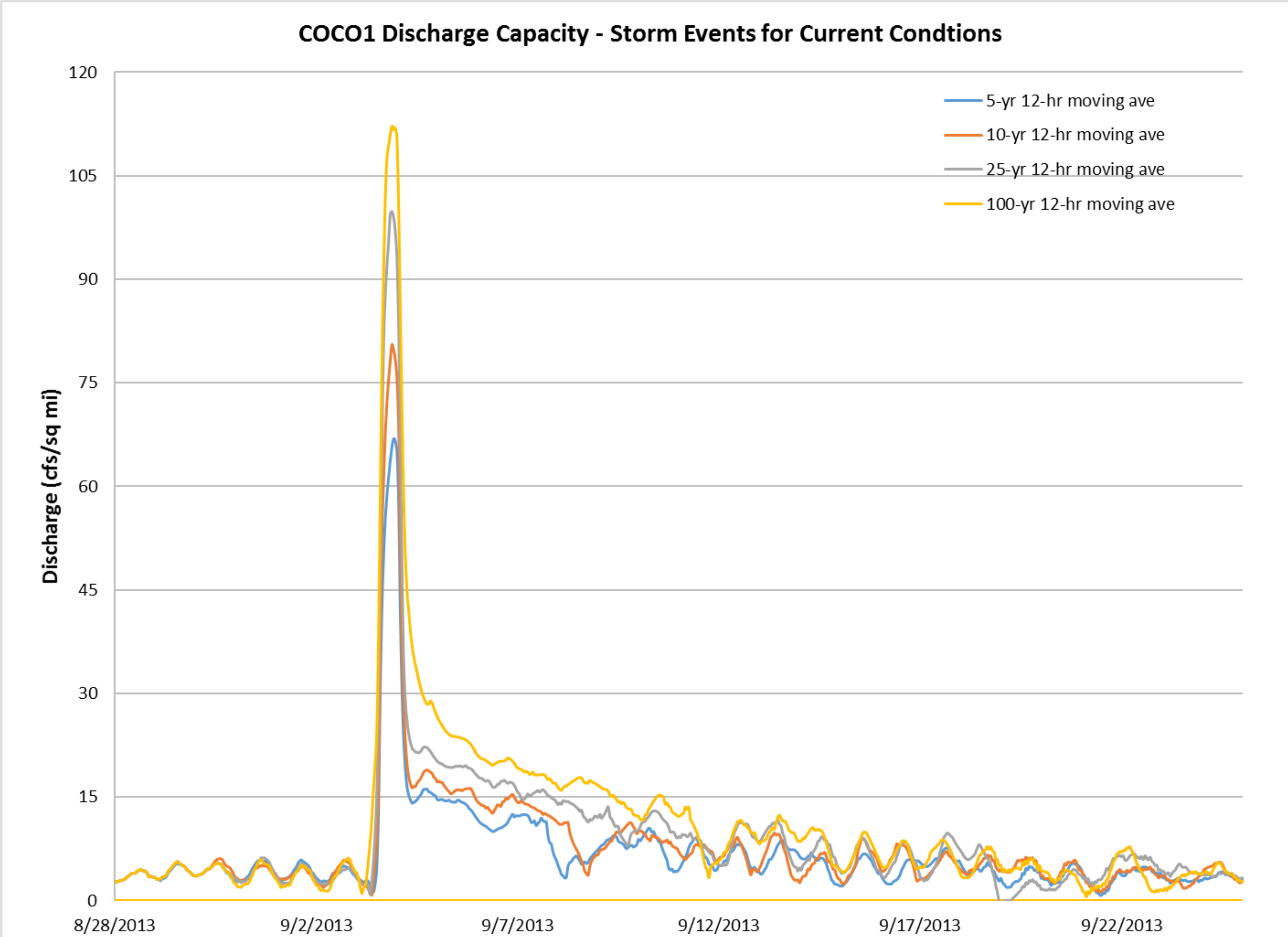

Figure 4.2.2 – Net Area-Weighted Discharge Hydrograph for COCO1

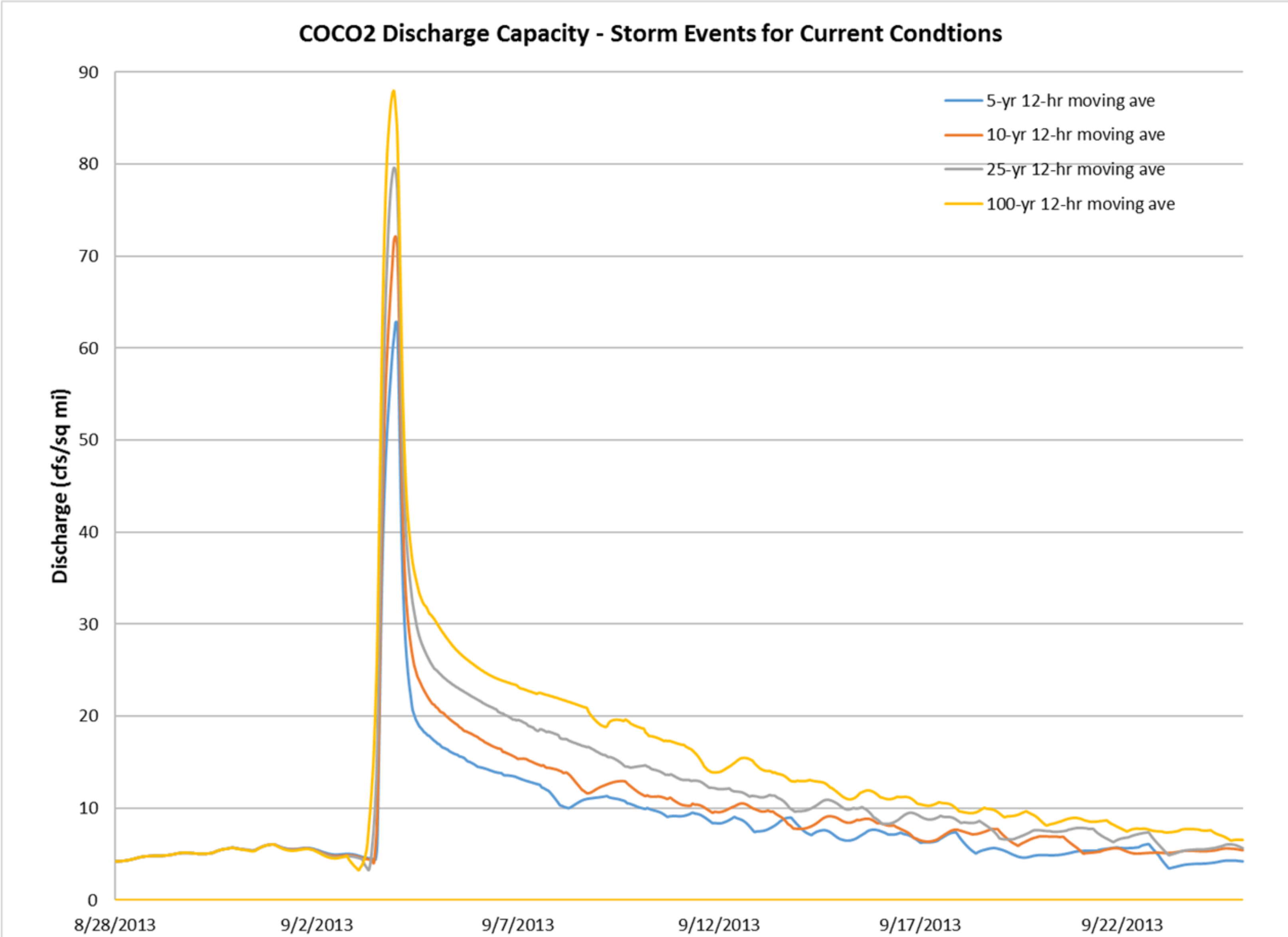

Figure 4.2.3 – Net Area-Weighted Discharge Hydrograph for COCO2

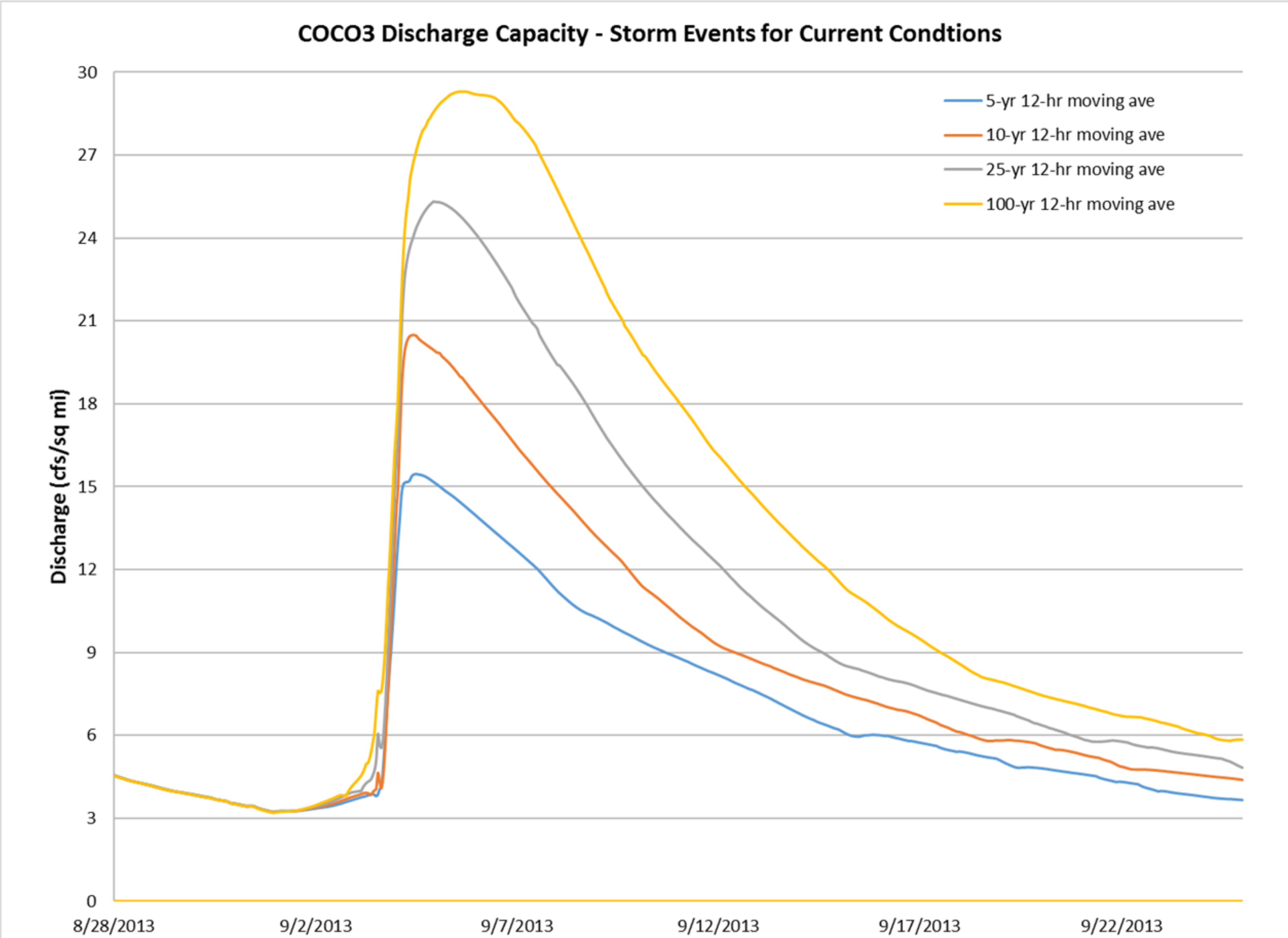

Figure 4.2.4 – Net Area-Weighted Discharge Hydrograph for COCO3

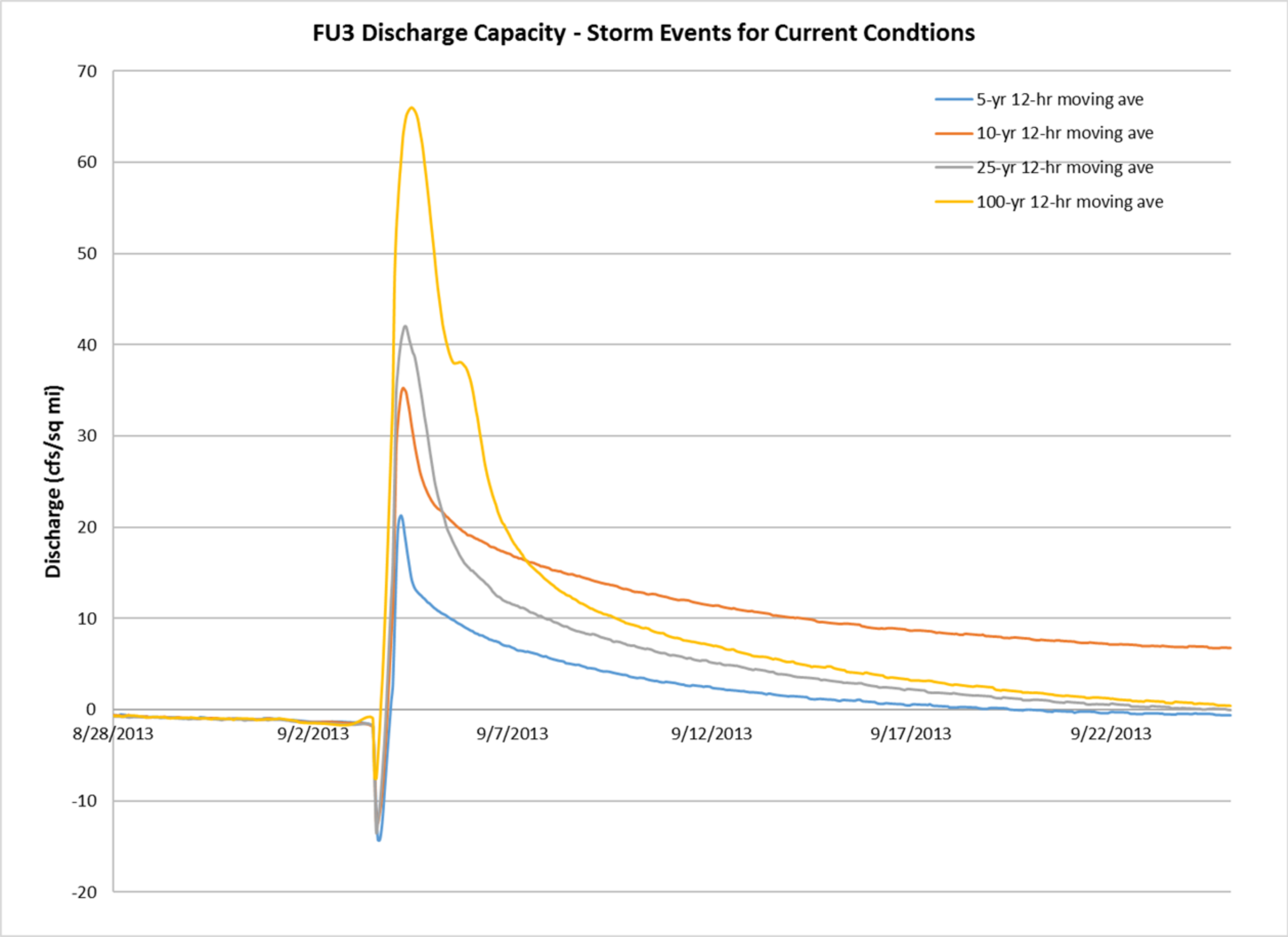

Figure 4.2.5 – Net Area-Weighted Discharge Hydrograph for FU3

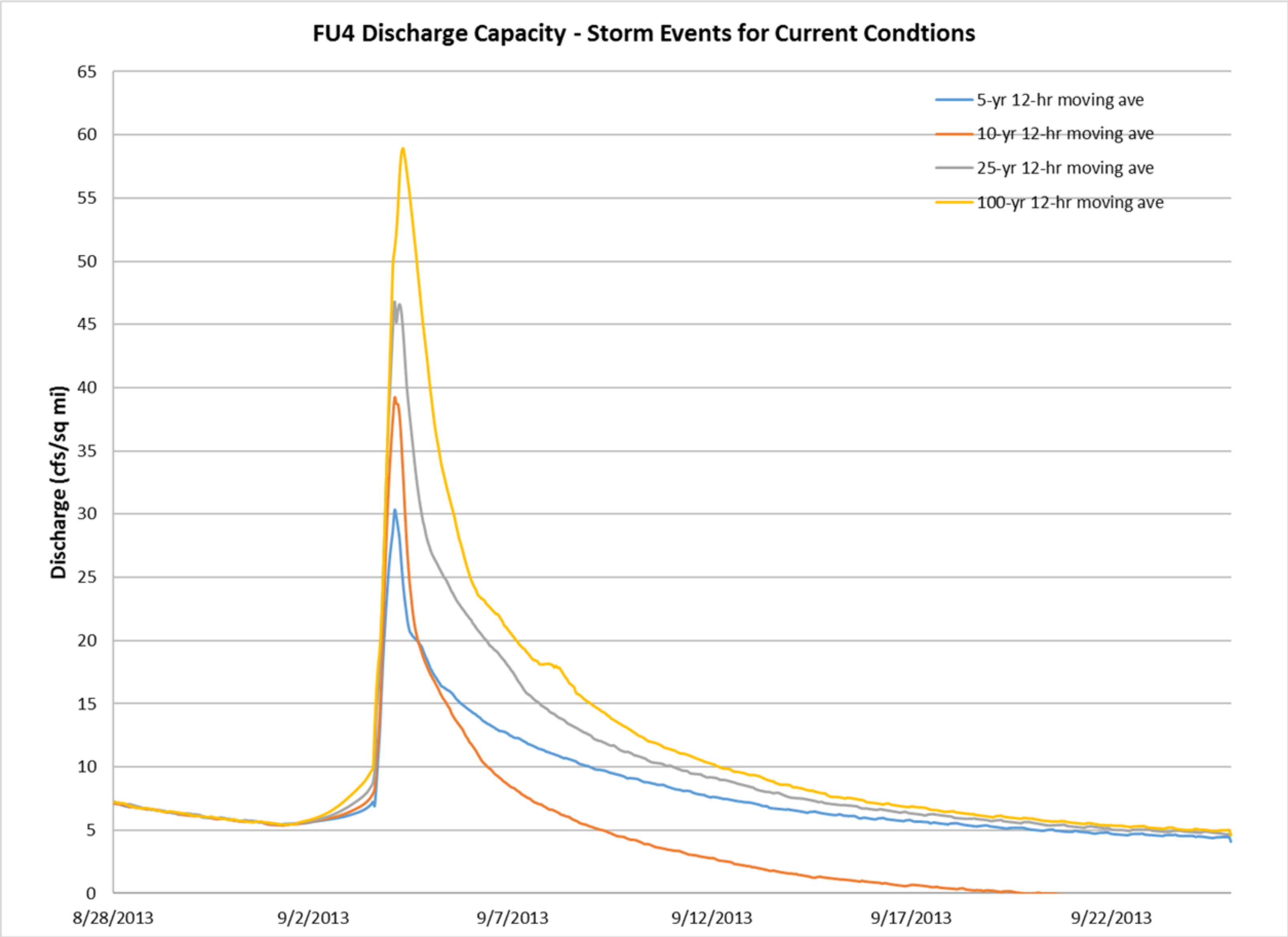

Figure 4.2.6 – Net Area-Weighted Discharge Hydrograph for FU4

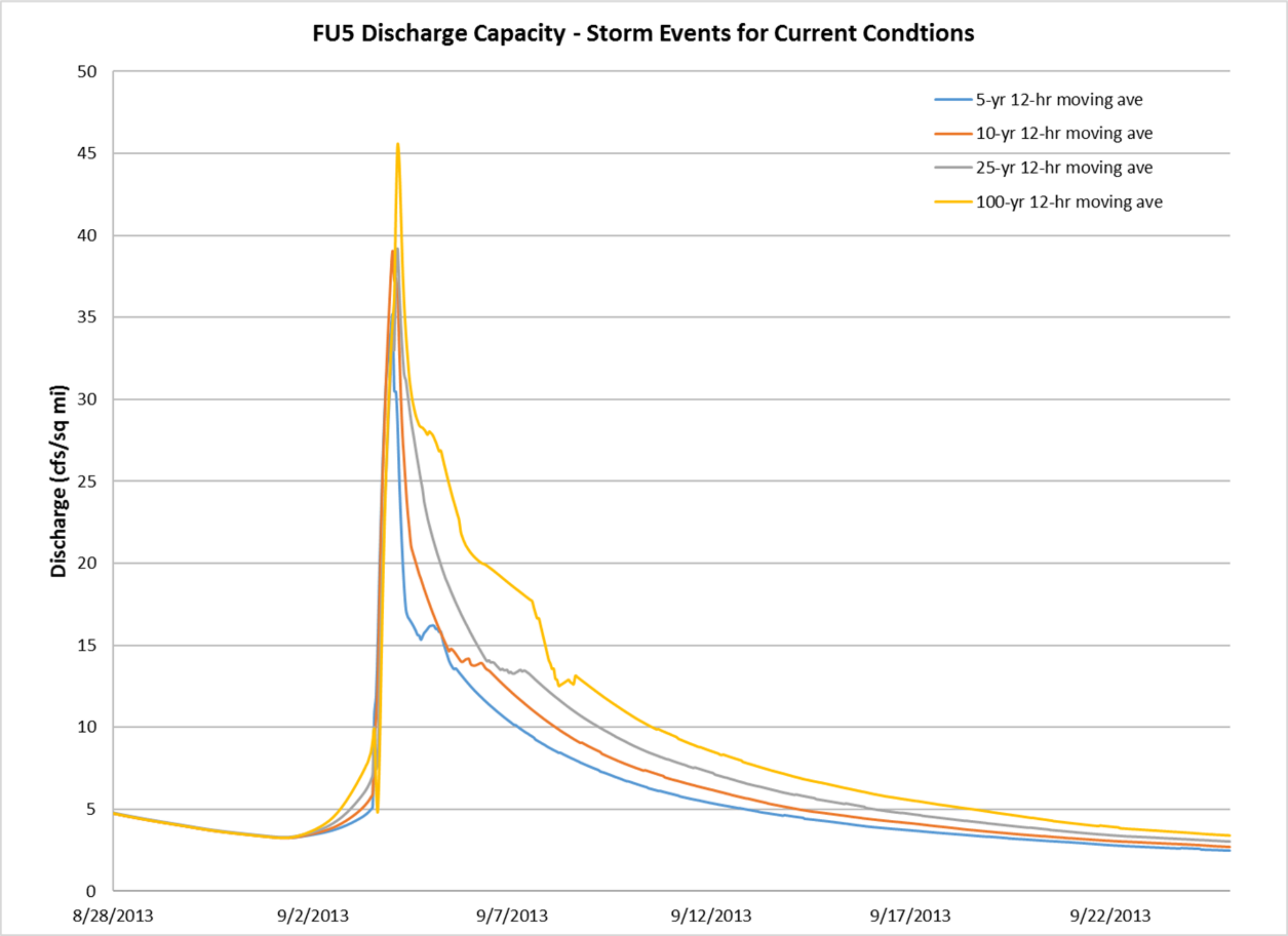

Figure 4.2.7 – Net Area-Weighted Discharge Hydrograph for FU5

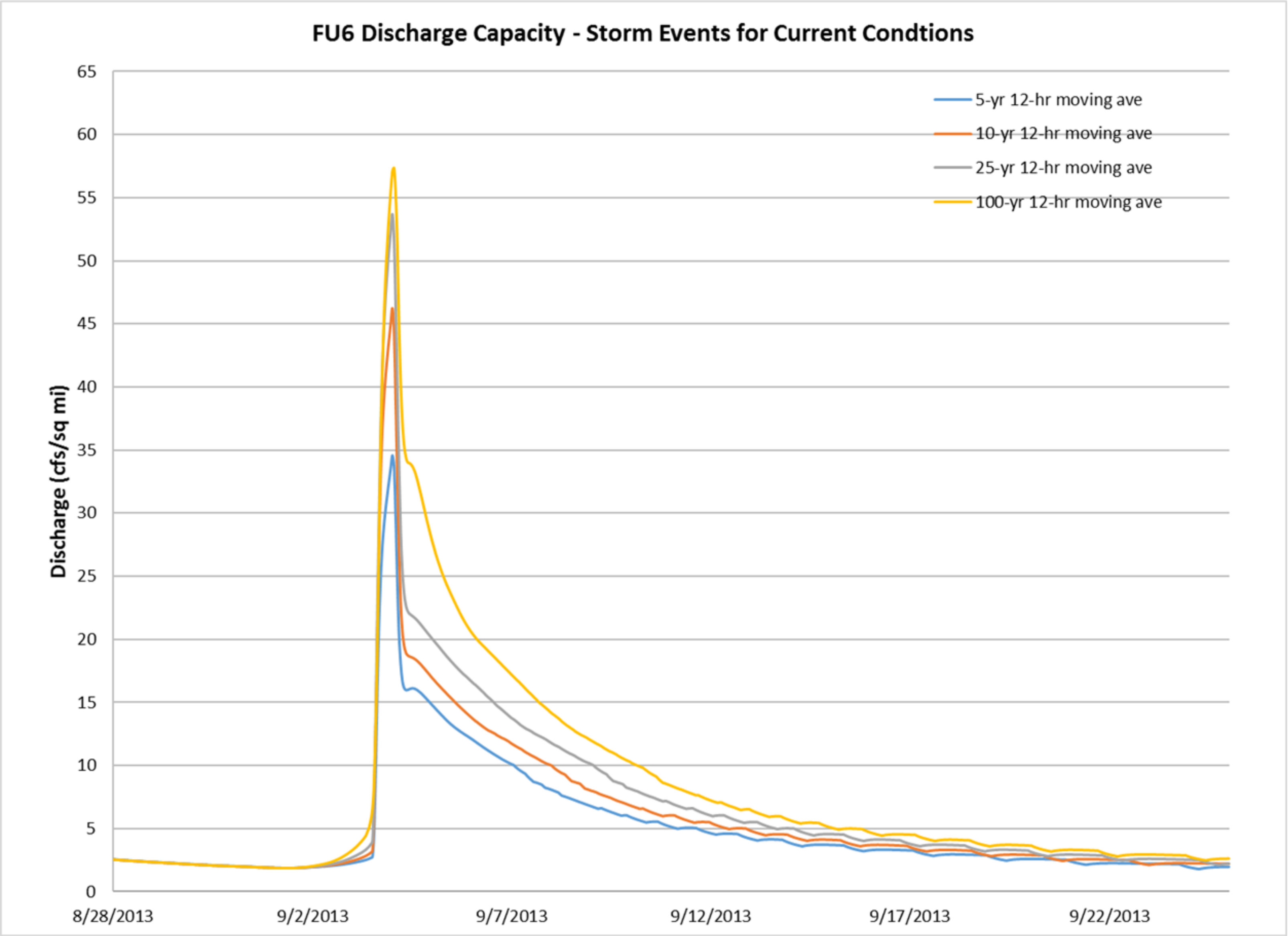

Figure 4.2.8 – Net Area-Weighted Discharge Hydrograph for FU6

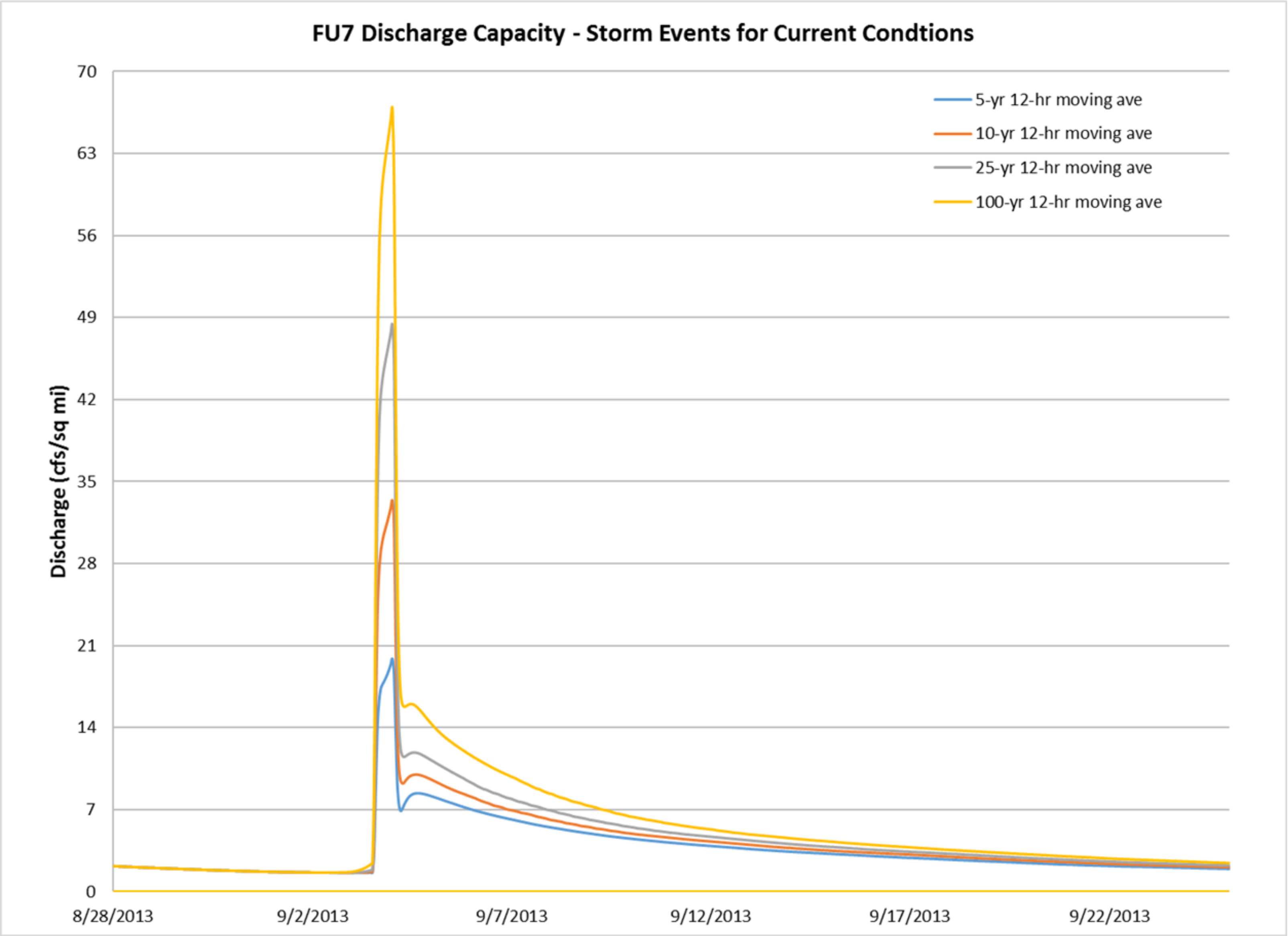

Figure 4.2.9 – Net Area-Weighted Discharge Hydrograph for FU7

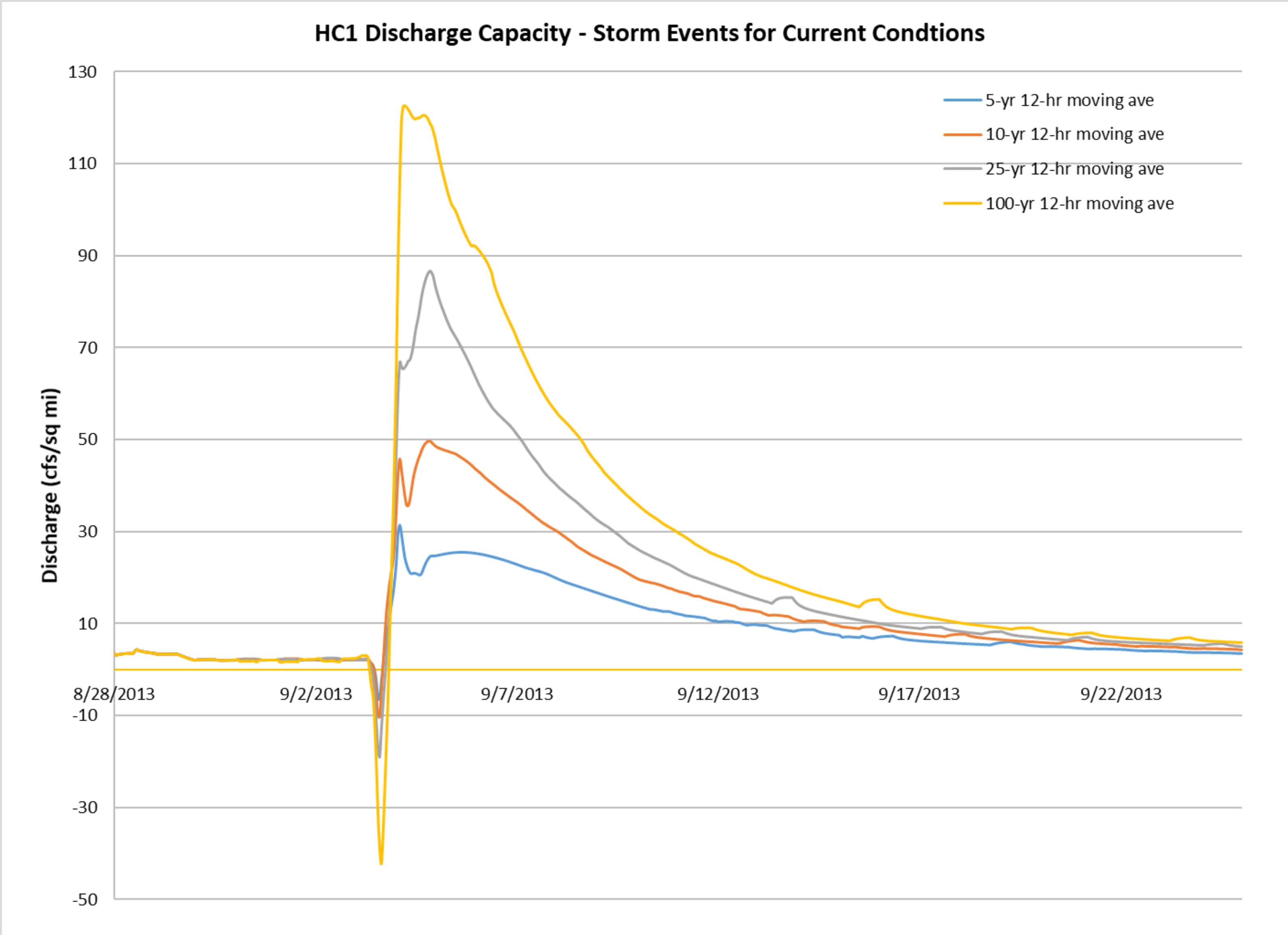

Figure 4.2.10 – Net Area-Weighted Discharge Hydrograph for HC1

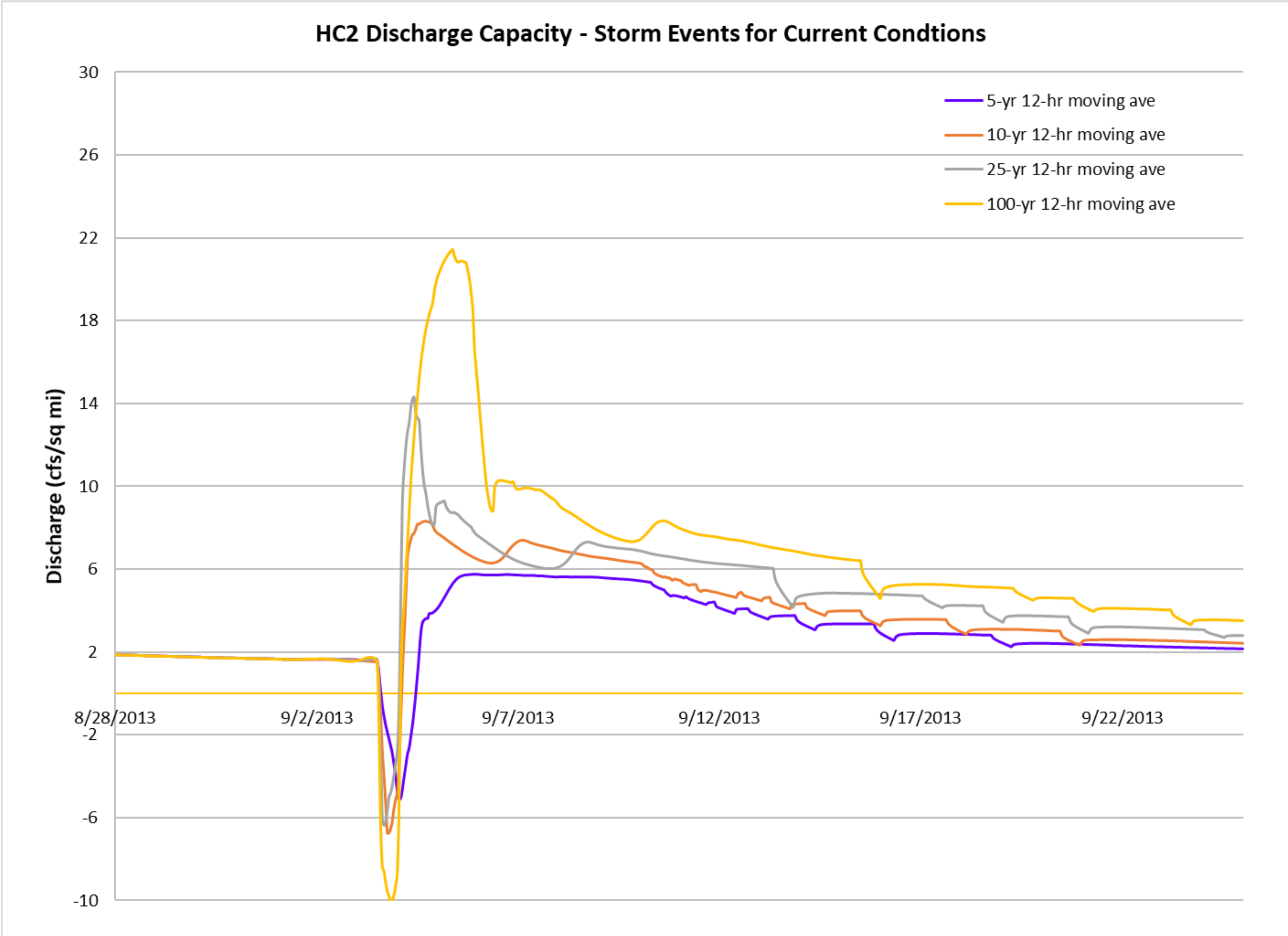

Figure 4.2.11 – Net Area-Weighted Discharge Hydrograph for HC2

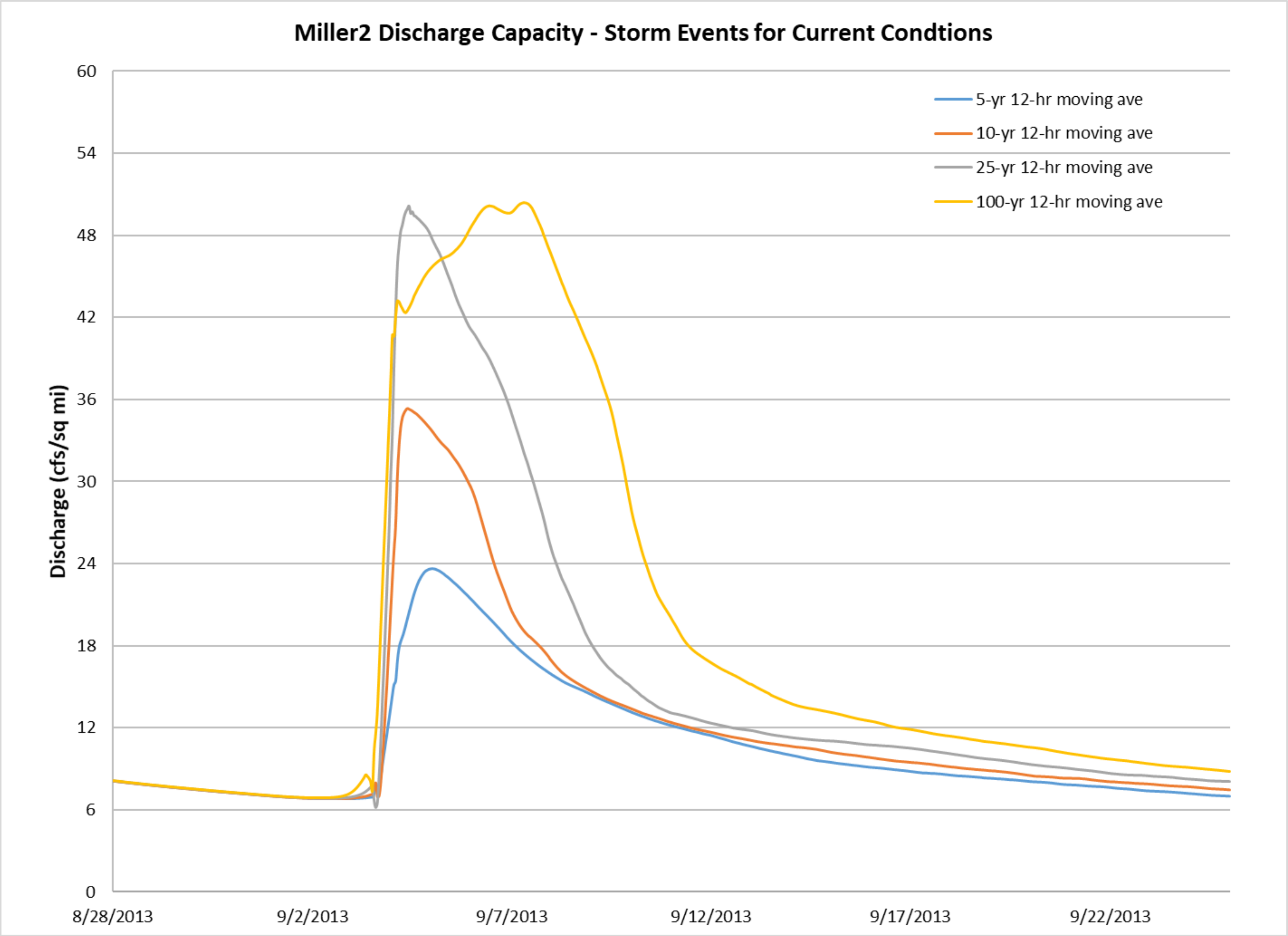

Figure 4.2.12 – Net Area-Weighted Discharge Hydrograph for Miller2

Interbasin Flows, Corkscrew Swamp

Although not shown as part of the Cocohatchee or Golden Gate Watersheds, significant additional inflows from the Corkscrew Swamp contribute to the Cocohatchee Canal Watershed across it's northeastern boundary, and through structures CORK2 and CORK3 into the Golden Gate Watershed. These inter-basin flows from the Corkscrew Swamp are from an indeterminate area; flows originating in the Corkscrew Swamp can enter the Imperial River, Golden Gate, and Cocohatchee Watersheds.

The discharge capacity at structure CORK1 (**Figure 4.2.1**) is partly a function of inter-basin flows entering from the Corkscrew Swamp. These flows enter the CORK system through structures CORK2 and CORK3, located along the northern boundary of the Golden Gate Watershed. Under the design storm conditions these inter-basin flows, totaling several hundred cubic feet per second, are subtracted (along with the Twin Eagles structure discharge) from the CORK1 catchment outflows, to compute the discharge capacity in the CORK1 water control catchment.

Significant additional inflows from the Corkscrew Swamp contribute to the Cocohatchee Canal Watershed across it's northeastern boundary. In the MIKE SHE / MIKE-11 model setup, the inflows are concentrated at two major inflow points shown on **Figure 4.2.13**.

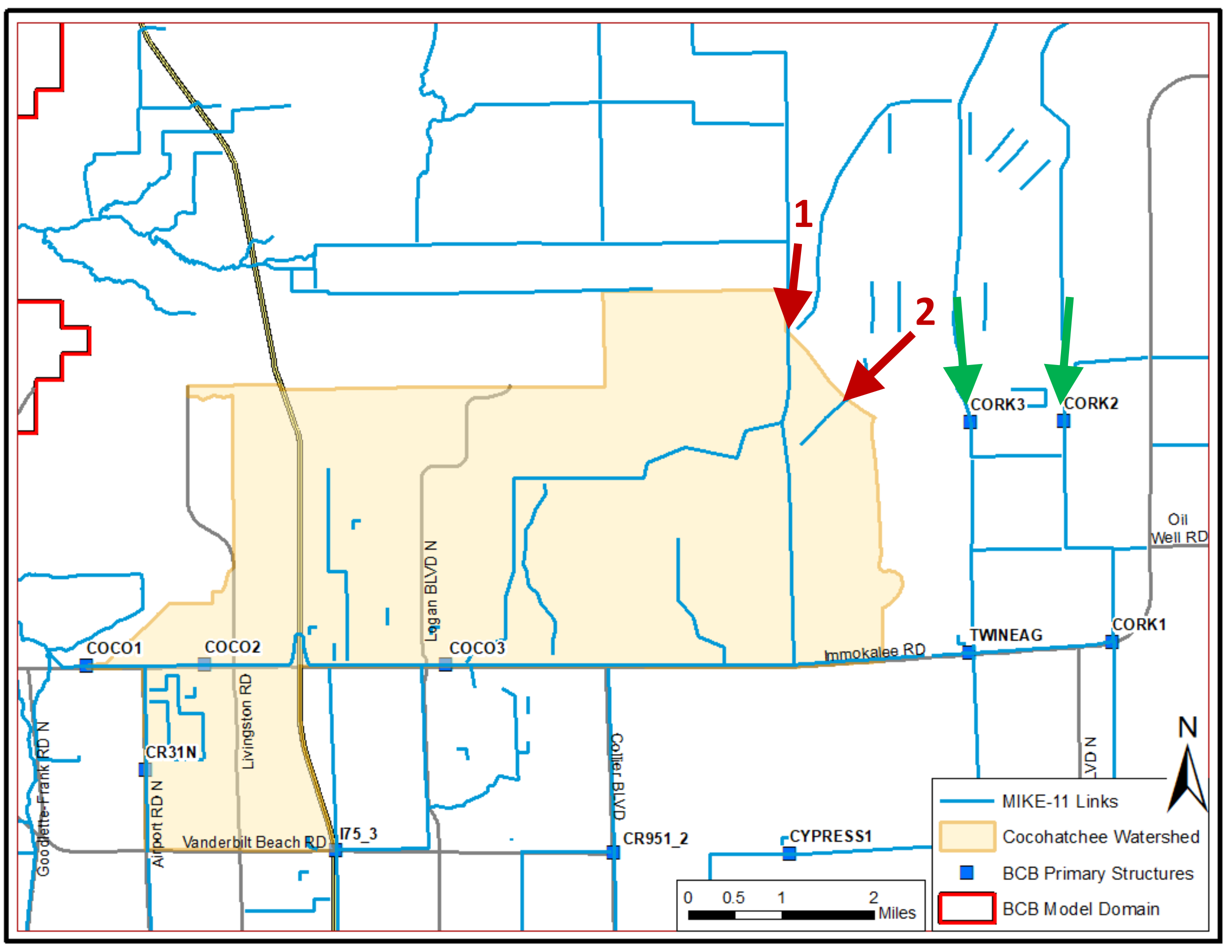

Figure 4.2.13 – Locations of Inflow Points from Bird Rookery/Corkscrew Swamp into Cocohatchee Watershed (red arrows) and CORK2/CORK3 into Golden Gate (green arrows)

Table 4.2.2 lists the peak inflows entering the Cocohatchee Watershed from the Bird Rookery area and from the Corkscrew Swamp into the Golden Gate Watershed. **Figure 4.2.14** shows discharge hydrographs into the Cocohatchee Watershed at the two locations for the 25-year design storm event. Discharge hydrographs for the CORK2 and CORK3 structures can be found in **Appendices B** through **E**.

Table 4.2.2 – Maximum Inflows from Bird Rookery/Corkscrew Swamp in to Cocohatchee and Golden Gate Watersheds

Corkscrew Swamp Inflow Point	5-Year (cfs)	10-Year (cfs)	25-Year (cfs)	100-Year (cfs)
Location 1 (into Cocohatchee)	82	96	117	146
Location 2 (into Cocohatchee)	59	71	87	107
CORK2 (into Golden Gate)	143	160	195	256
CORK3 (into Golden Gate)	205	246	296	345

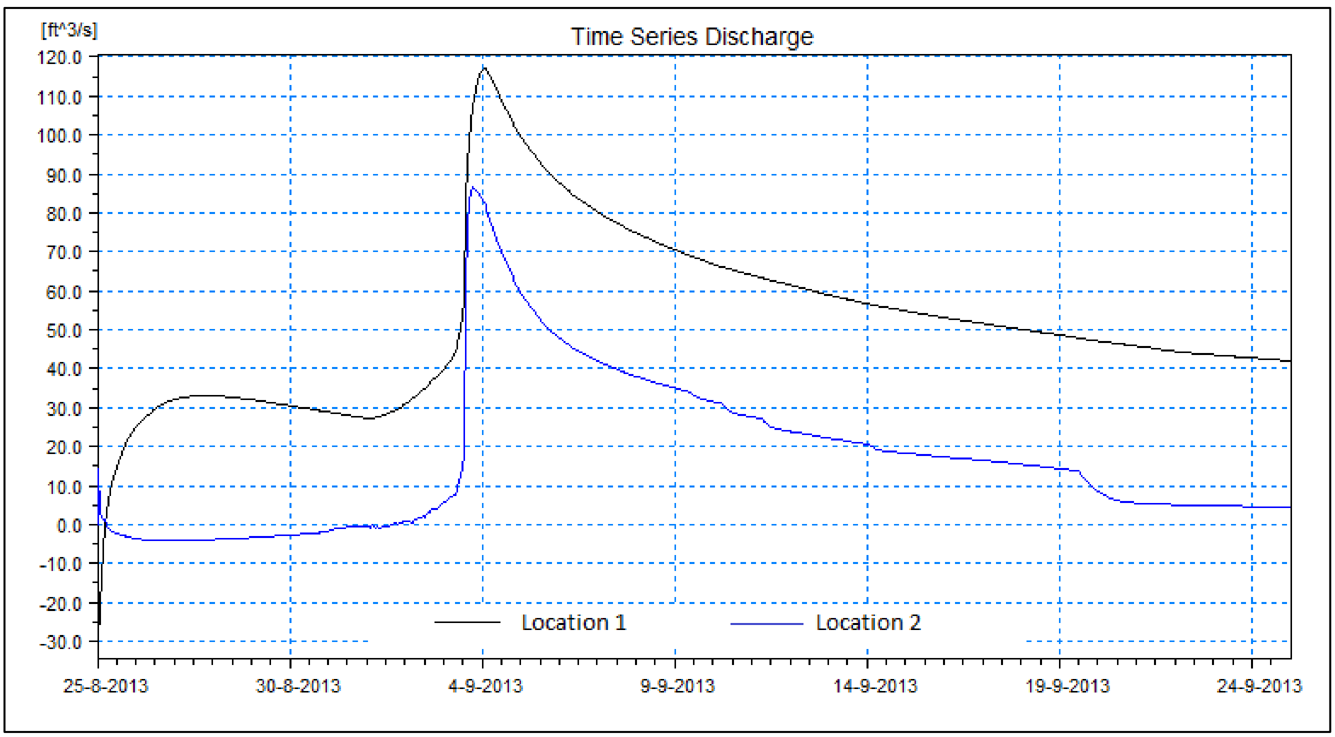

Figure 4.2.14 – Inter-basin Discharge into Cocohatchee 25-year Design Storm Event

Interbasin Flows, Golden Gate Watershed

The watershed divide along the Cocohatchee Canal is generally considered to be a mile or so west of the Twin Eagles (a.k.a. COCO4) structure. West of the divide the Canal flows towards COCO3 and the coast, and East of the divide flows are towards the upper portion of the Golden Gate Watershed. Examination of the model output revealed that at COCO4, flow direction is to the east for most of the simulation as expected. However, during the peak of the 25-year and 100-year events, there is a brief flow reversal, when flow is to the west for a brief period (2-3 hours). During this brief period, the divide (as defined by the hydraulic gradient) shifts to the east of COCO4. Flows from the Golden Gate Watershed enter the Cocohatchee Watershed briefly at the Twin Eagles Structure on the Cocohatchee East Canal. **Figure 4.2.15** shows the 25-year discharge hydrograph at the Twin Eagles structure, where positive values represent eastward flow from the direction of the Cocohatchee Watershed to the Golden Gate Watershed. The brief flow reversals occur during all design storm events simulated, but are most pronounced for the 10-year, 25-year, and 100-year events.

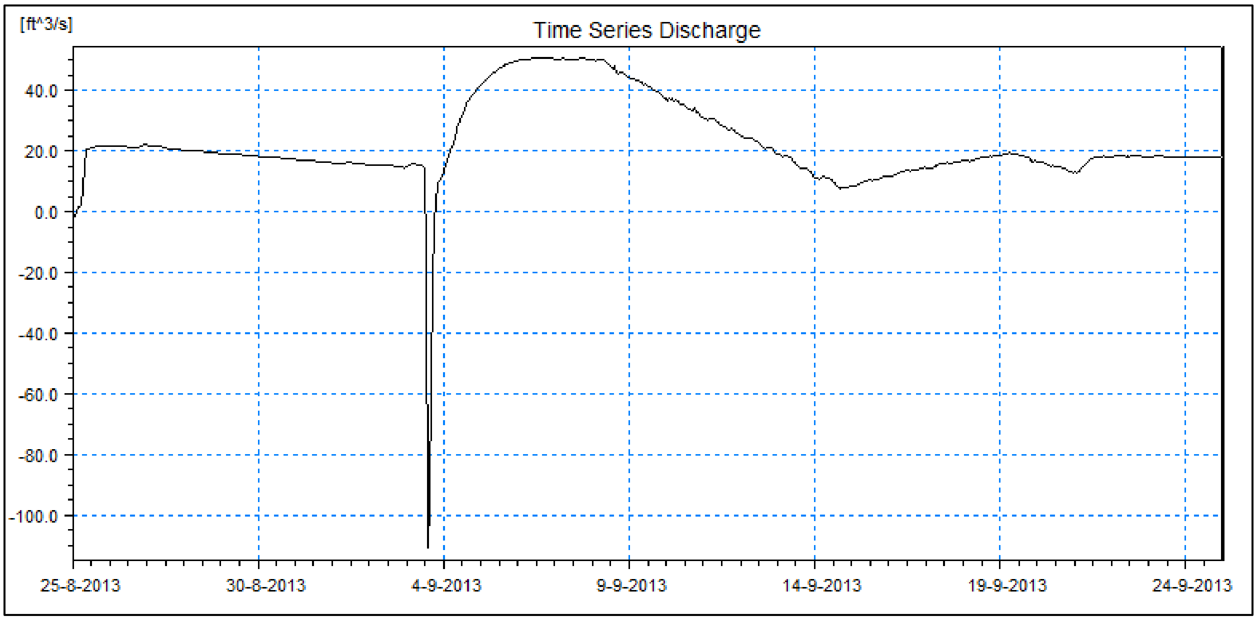

Figure 4.2.15 – 25-year Discharge at Twin Eagles (COCO4) Structure, Current Conditions

Interbasin flows from the Golden Gate Main Canal enter the Faka Union Watershed at two locations; both discharge into the Miller Canal. These locations are the Miller 3 control structure and the C-1 Connector Canal. **Table 4.2.3** summarizes the peak discharge rates (12-hour running average) at interbasin flow locations from the Golden Gate Watershed into the Cocohatchee and Miller/Faka Union Watersheds.

Table 4.2.3 – Maximum Interbasin Outflows from Golden Gate Watershed (cfs, 12-hr running average)

	5-Year	10-Year	25-Year	100-Year
COCO4 Positive (into Golden Gate Watershed)	26	45	51	54
COCO4 Negative (into Cocohatchee Watershed)	0	-10	-21	-34
Miller3 (into Miller/Faka Union Watershed)	218	311	436	503
C-1 Connector (into Miller/Faka Union Watershed)	45	45	38	33

4.3 PM #5 – Frequency of Flooding

In this PM, the flood elevations or depths of overland flooding are evaluated for a range of design storms (5-year, 10-year, 25-year, and 100-year). These flood depths/elevations can then be compared with elevations of features such as buildings and roadways, where such information exists. For the purposes of this BCB FPLOS evaluation, flood inundation maps were developed from the gridded MIKE SHE model output for each storm event, in the form of depth of overland water. These overland water depths, with the 500-foot model grid spacing, were converted into a TIN for presentation purposes. The resulting flood inundation maps are presented on **Figures 4.3.1 through 4.3.4** for each of the four design storm events.

Large areas of the Cocohatchee and Henderson/Belle Meade Watersheds are undeveloped (as of the date of current condition model development, or year 2015), and thus were not served by stormwater collection and conveyance facilities. These natural areas show the greatest extents and depths of flooding for the design storm events.

Most of the newer developments (i.e., those with modern stormwater management systems) within the Cocohatchee, Henderson/Belle Meade, and Faka Union Watersheds were predicted to remain relatively flood-free for all storm events. Conversely, Golden Gate Estates was predicted to be mostly inundated for the 100-year storm event with flood depths ranging from 0.25 feet to 2.0 feet. The most severely flooded areas within Golden Gate Estates were predicted to be in those areas furthest from the Faka Union and Miller canals (**Figure 4.3.4**); depths of flooding generally increase with distance from the canals.

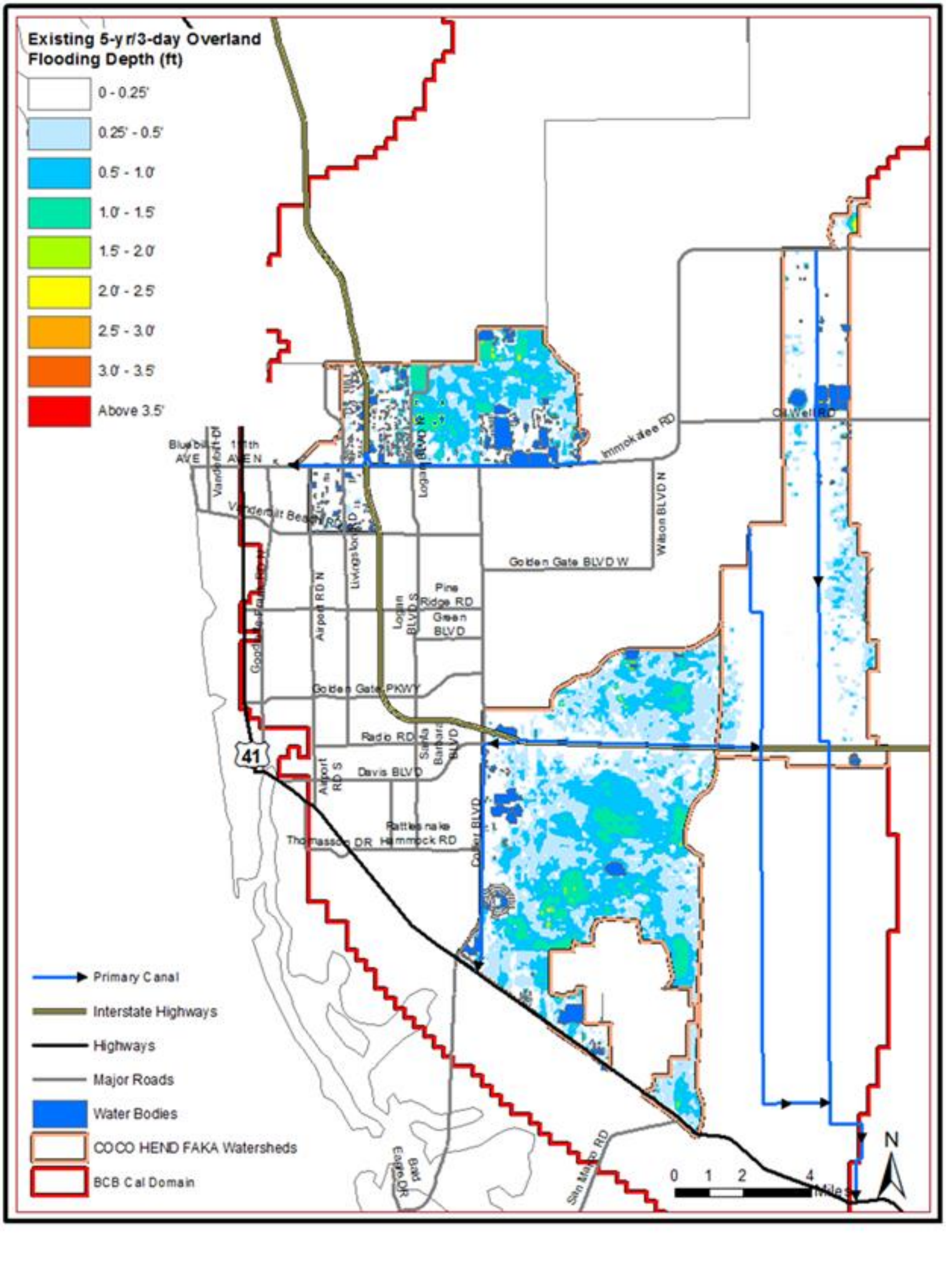

Figure 4.3.1 – Inundation Map for 5-year Design Storm Event, Cocohatchee, Faka Union and Henderson

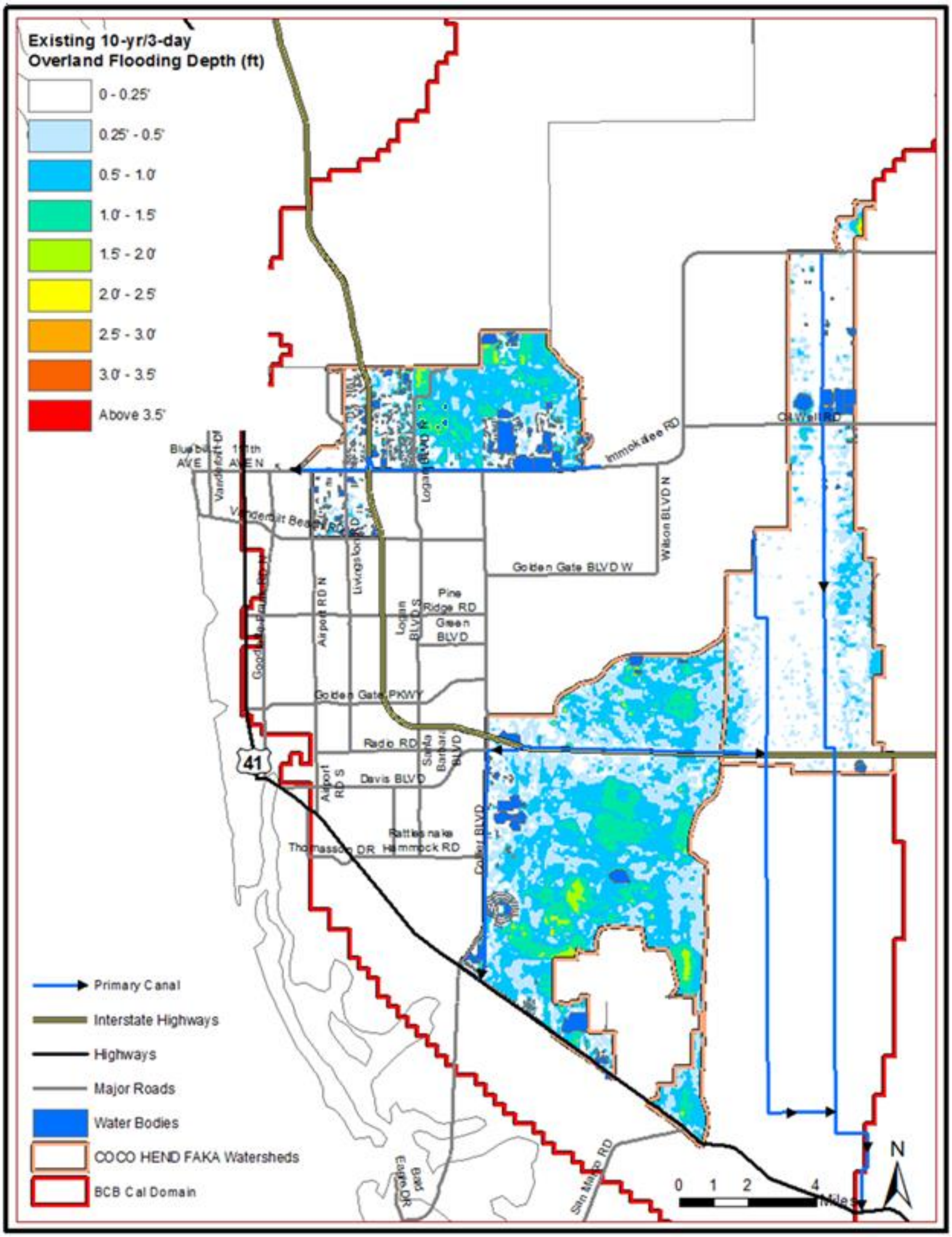

Figure 4.3.2 – Inundation Map for 10-year Design Storm Event, Cocohatchee, Faka Union and Henderson

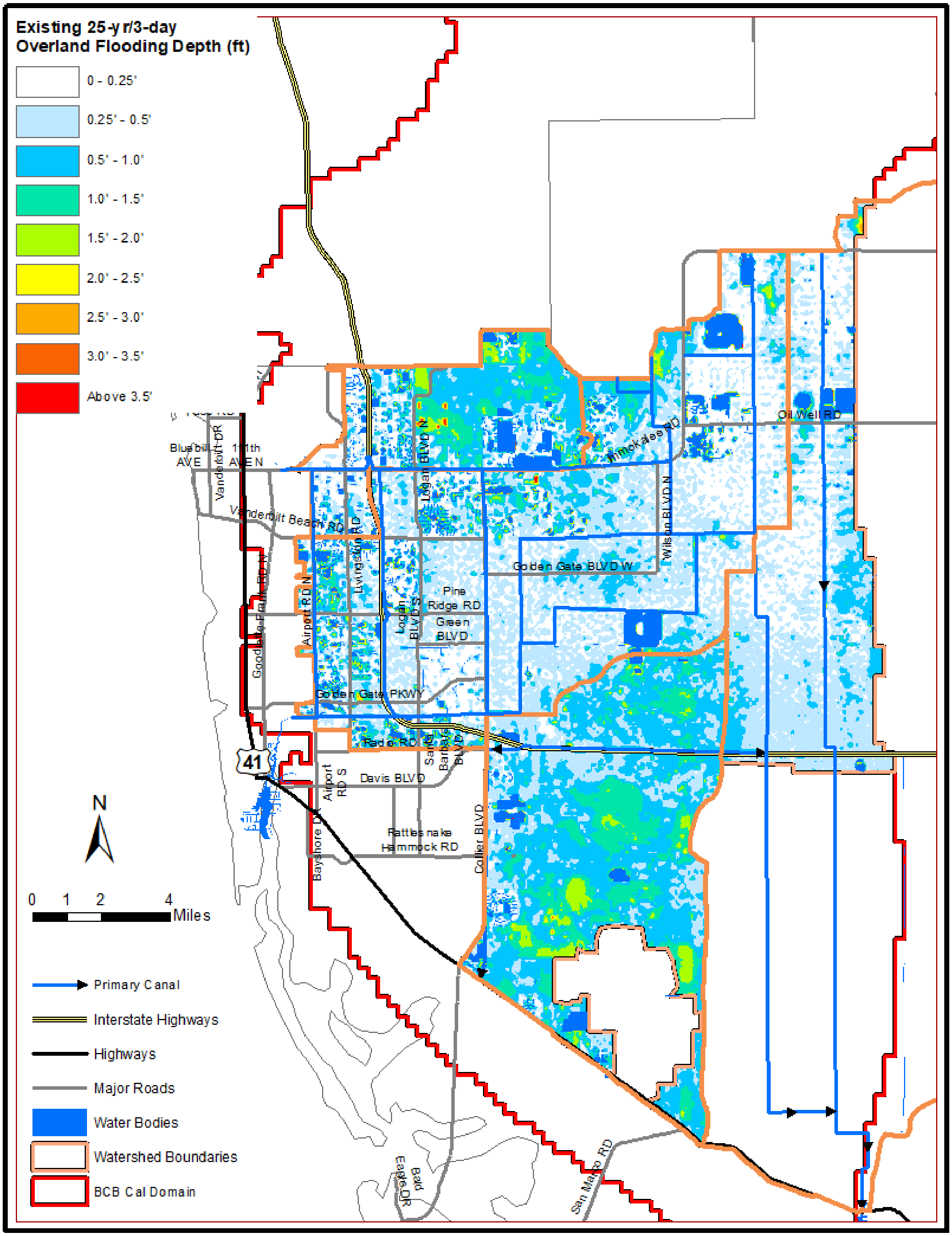

Figure 4.3.3 – Inundation Map for 25-year Design Storm Event, Golden Gate, Cocohatchee, Faka Union and Henderson

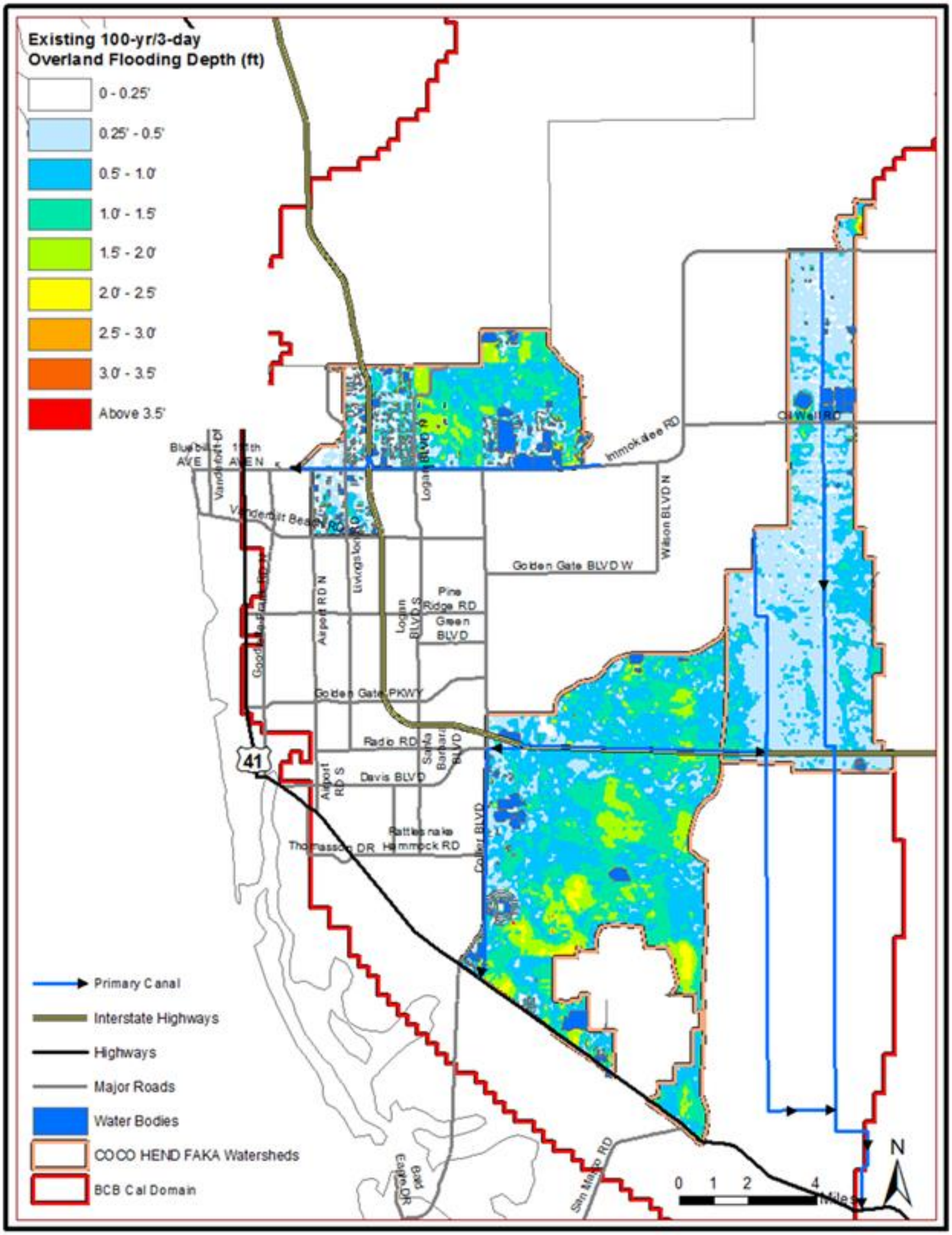

Figure 4.3.4 – Inundation Map for 100-year Design Storm Event, Cocohatchee, Faka Union and Henderson

4.4 PM #6 – Duration of Flooding

PM #6 quantifies the duration of flooding at specific locations of interest within a watershed. For this Study, the length of time the flood elevation is projected to be above a threshold depth was mapped over the entire study area using the gridded model output files for the 2-D overland flow component.

Using a threshold depth of 0.25 feet, the length of time the overland flood depth was predicted to exceed the threshold within each MIKE SHE grid cell was calculated using the statistics tool in MIKE ZERO. The results of the flood duration calculations for all grid cells within all four study watersheds (including the Golden Gate Watershed) are shown on **Figures 4.4.1** through **4.4.4** for the 5-year, 10-year, 25-year, and 100-year storm events, respectively.

From **Figure 4.4.1,** it is evident that large areas are predicted to be inundated for more than 72 hours during the 31-day simulation period, even for the 5-year design storm event. These areas are comprised primarily of lakes and wetlands and other low-lying undeveloped areas. For the 10-year through 100-year storm events, greater proportions of the study watersheds are shown to be inundated for significant periods of time, with **Figure 4.4.4** showing the vast majority of the watershed to be inundated for at least a small duration. Like the maps of inundation depth, the flood durations generally increase with distance from the major canals. This is particularly evident along the I-75 Canal in the Henderson Creek/Belle Meade Watershed for the 5-year and 10-year events, and along the North-South canals within Golden Gate Estates for the 25-year and 100-year events.

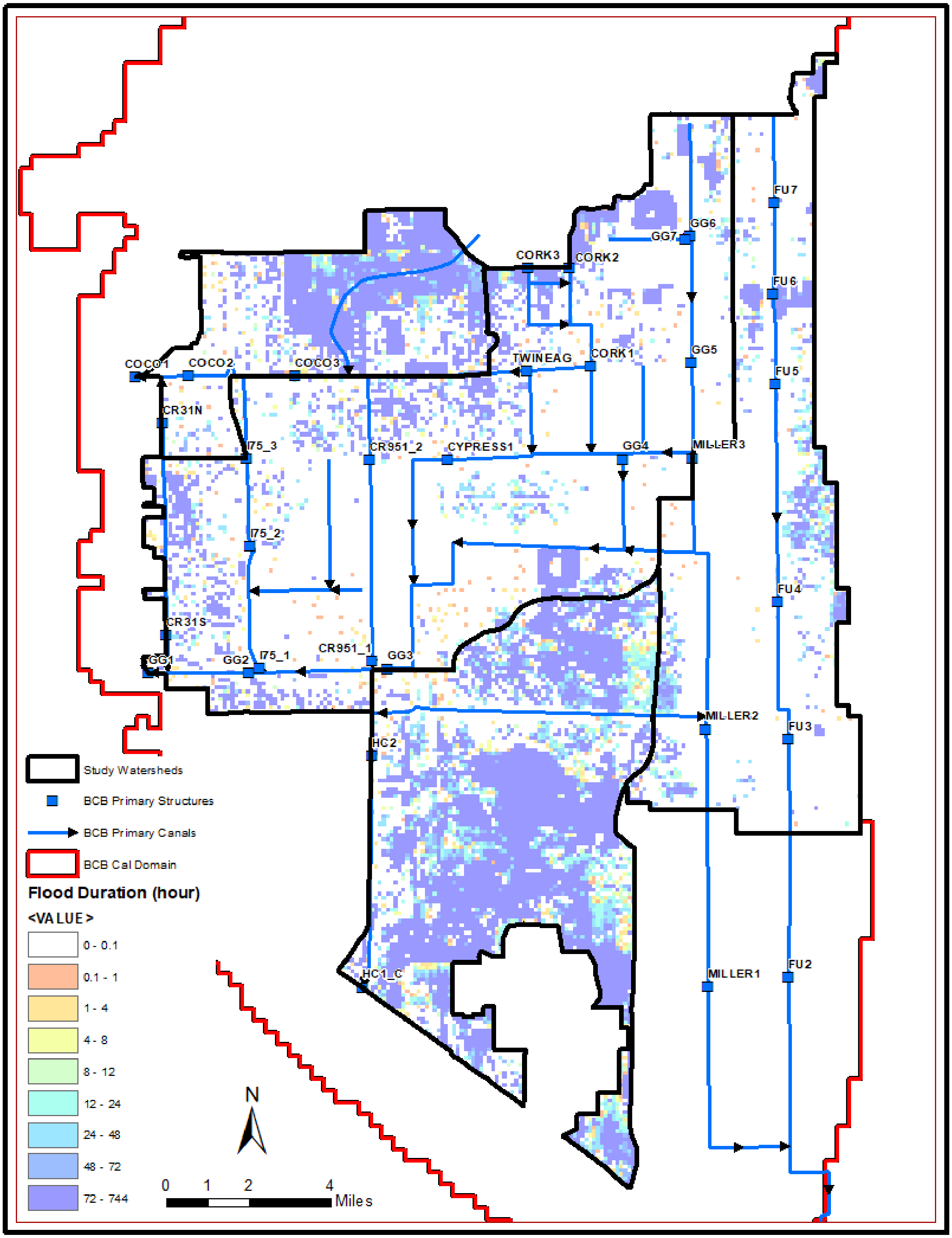

Figure 4.4.1– Flood Duration Map for 5-year Design Storm Event, Current Condition

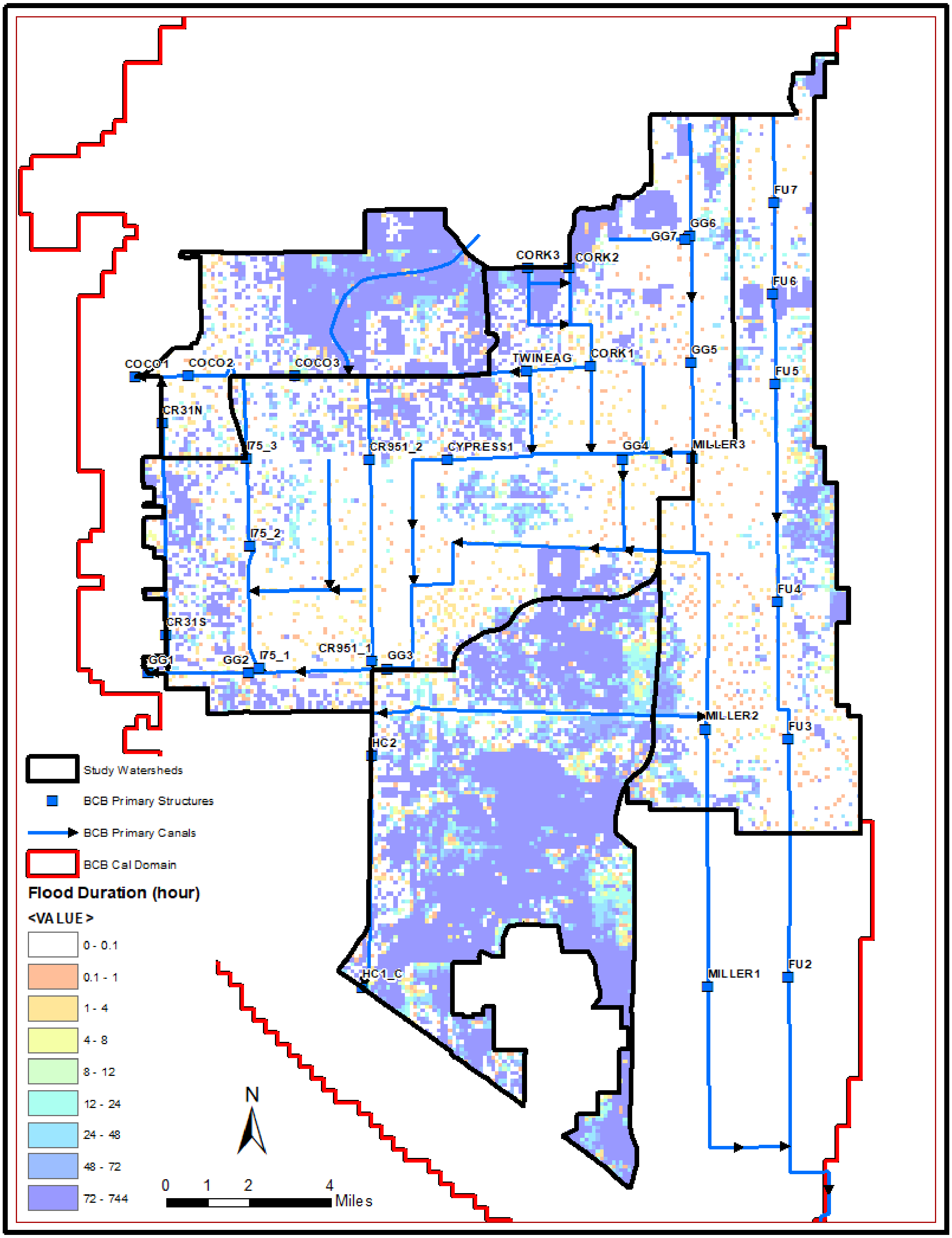

Figure 4.4.2– Flood Duration Map for 10-year Design Storm Event, Current Condition

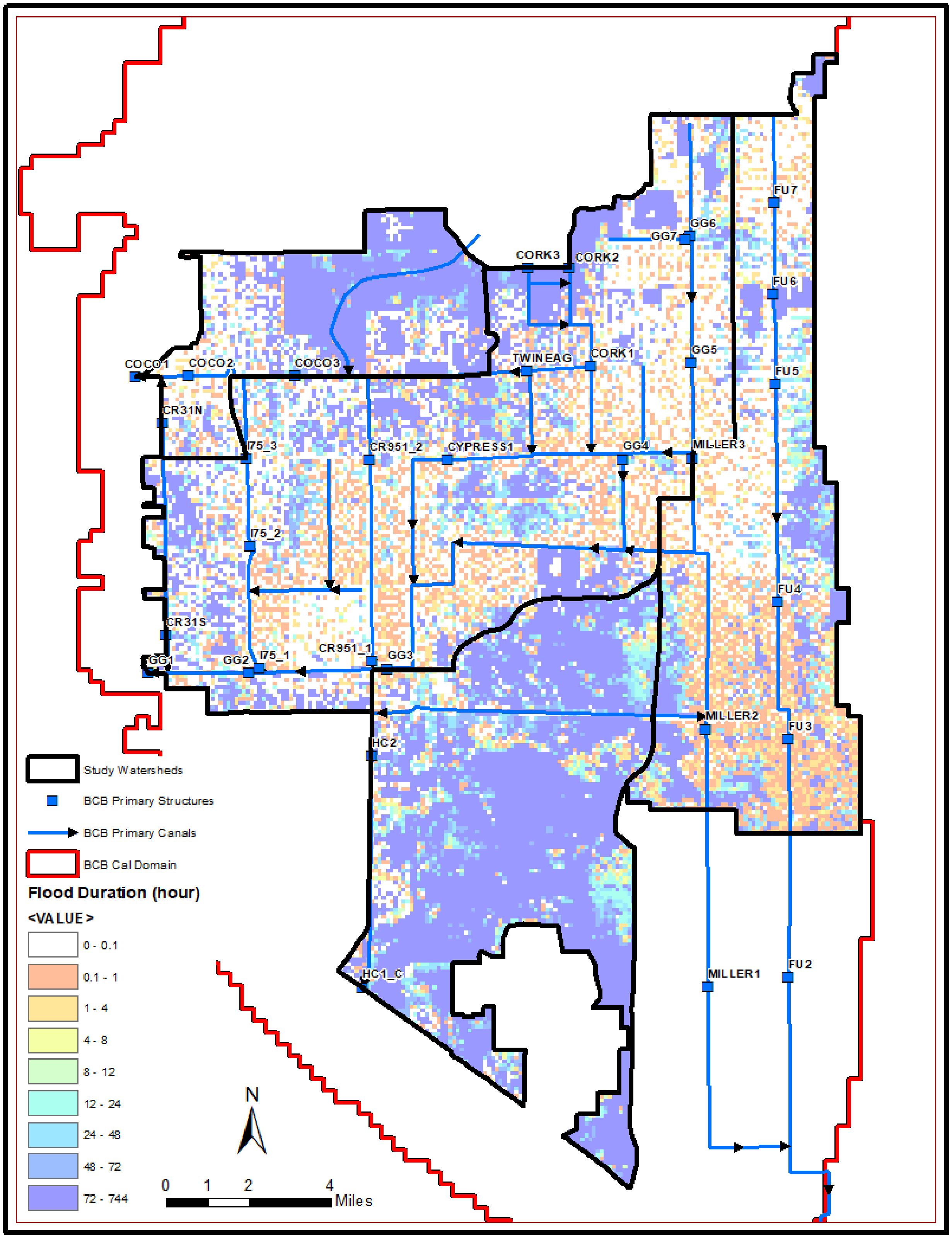

Figure 4.4.3– Flood Duration Map for 25-year Design Storm Event, Current Condition

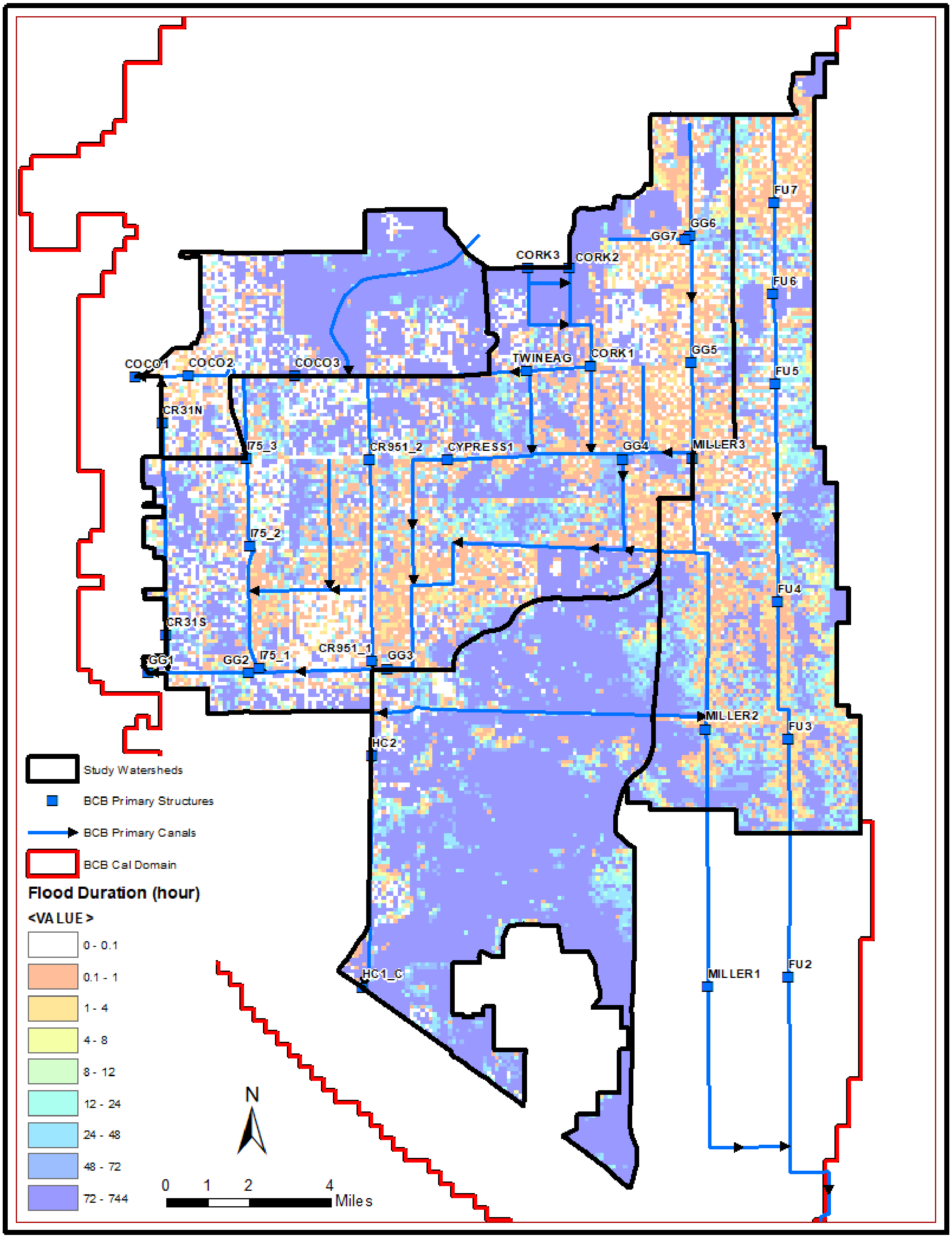

Figure 4.4.4– Flood Duration Map for 100-year Design Storm Event, Current Condition

5.0 Future Conditions Application Model Set-Up

For the Future Conditions model, the "current conditions" model infrastructure and structure operations were carried over, unchanged, with three exceptions. In addition to the current infrastructure, three structures that are either planned or under construction were added into model: The Curry Canal gated structure, The Miller Canal pump station and the Faka Union Canal pump station. The two pump stations are being constructed as part of the Picayune Strand Restoration Project. The relevant information needed to add these structures (construction drawings and operating protocols) were provided by the District through emails (12/4/2017, 12/12/2017, and 3/2/18) and SFWMD Environmental resource permit No. 0288313-008. The performance of the existing and near-future infrastructure was tested under several future scenarios and design storm events.

5.1 Future Land Use

The District provided shape files of future (year 2065) land use projections, which are documented in a December 2016 report prepared by Tim Lieberman of the District titled *Collier County 2065 Landuse for MIKESHE Modeling*. In one of the shape files, the land use was categorized in a fashion identical the MIKE SHE land use classification. Interflow analyzed the shape file to identify areas where urbanization is expected to expand or intensify. **Figure 5.1.1** shows all such areas within the Cocohatchee, Henderson/Belle Meade, and Faka Union Watersheds. This figure also shows the land use changes within the Golden Gate Watershed, which are described in the Golden Gate Future Conditions FPLOS Report (Deliverable 4.2.1) and included in the updated Future Conditions model described herein.

In most cases, undeveloped lands are projected to become developed into one of three urban land use categories associated with dwelling unit density (high, medium, and low). In a few cases, existing low-density urban is expected to increase to medium or high density urban. As evident on **Figure 5.1.1**, most of future development in the general area is expected to occur to the east of the Northern Golden Gate Estates, which is outside of the study area. Land use changes within the 6Ls Agricultural Area were not evaluated, as this area is currently the subject of a project underway designed to protect this area from potential impacts from the Picayune Strand Restoration Project (ref: NorthStar Contracting Group, *PSRP Southwest Protection Feature Preliminary DDR*, December, 2017).

The polygon-based land use changes were mapped onto the MIKE SHE Grid. **Figure 5.1.2** shows the grid cells that were changed under the current task to represent 2065 Land Use. All land-use based model parameters were changed accordingly for these cells, to appropriately represent the potential urbanization. These include the following:

- Paved Area Runoff Coefficient
- Drain Code
- Drain Level
- Drain Time Constant
- Topography (if cell current topo was below FEMA BFE)
- Detention Storage

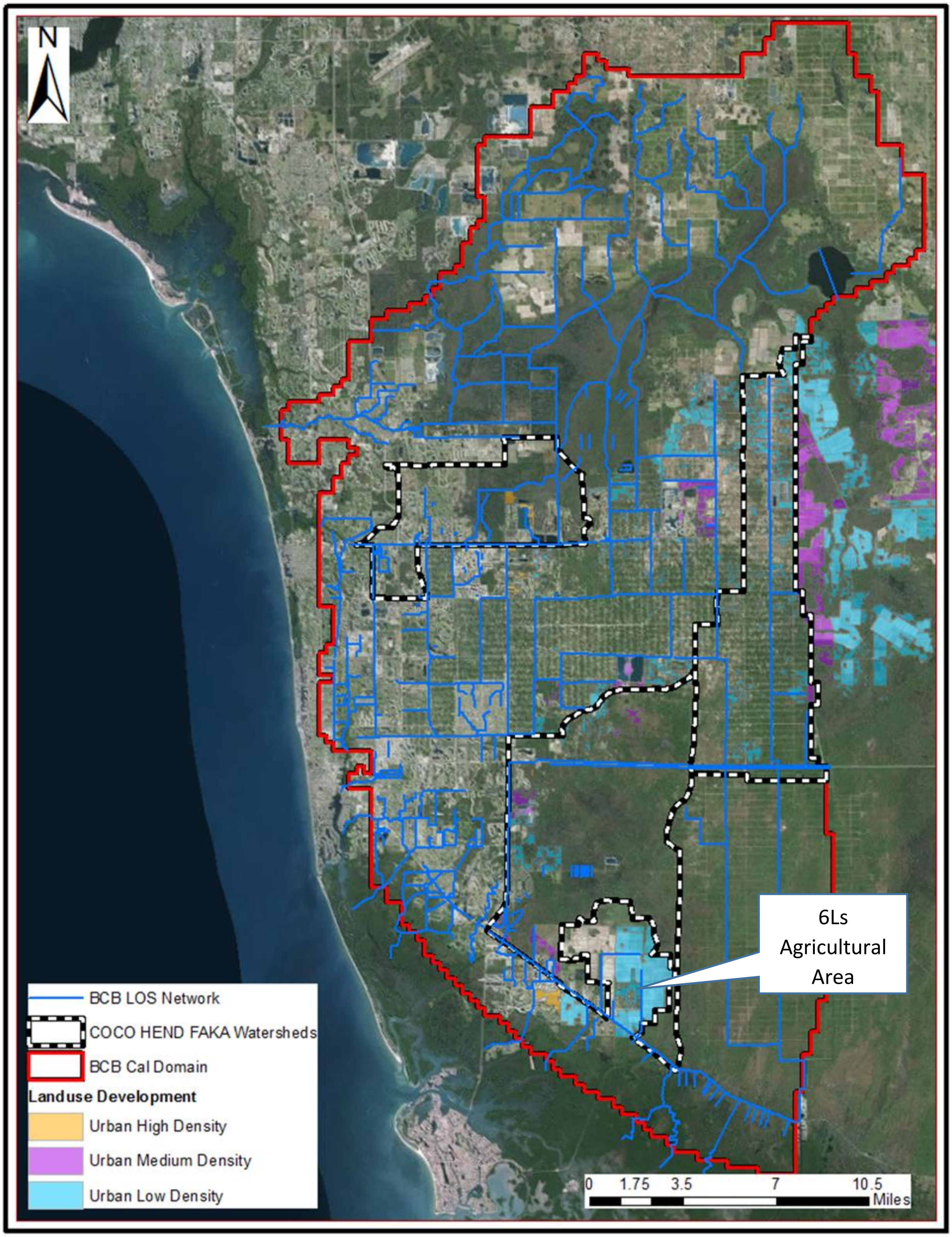

Figure 5.1.1 – Projected Changes in Land Use, 2015 to 2065 (Source: SFWMD)

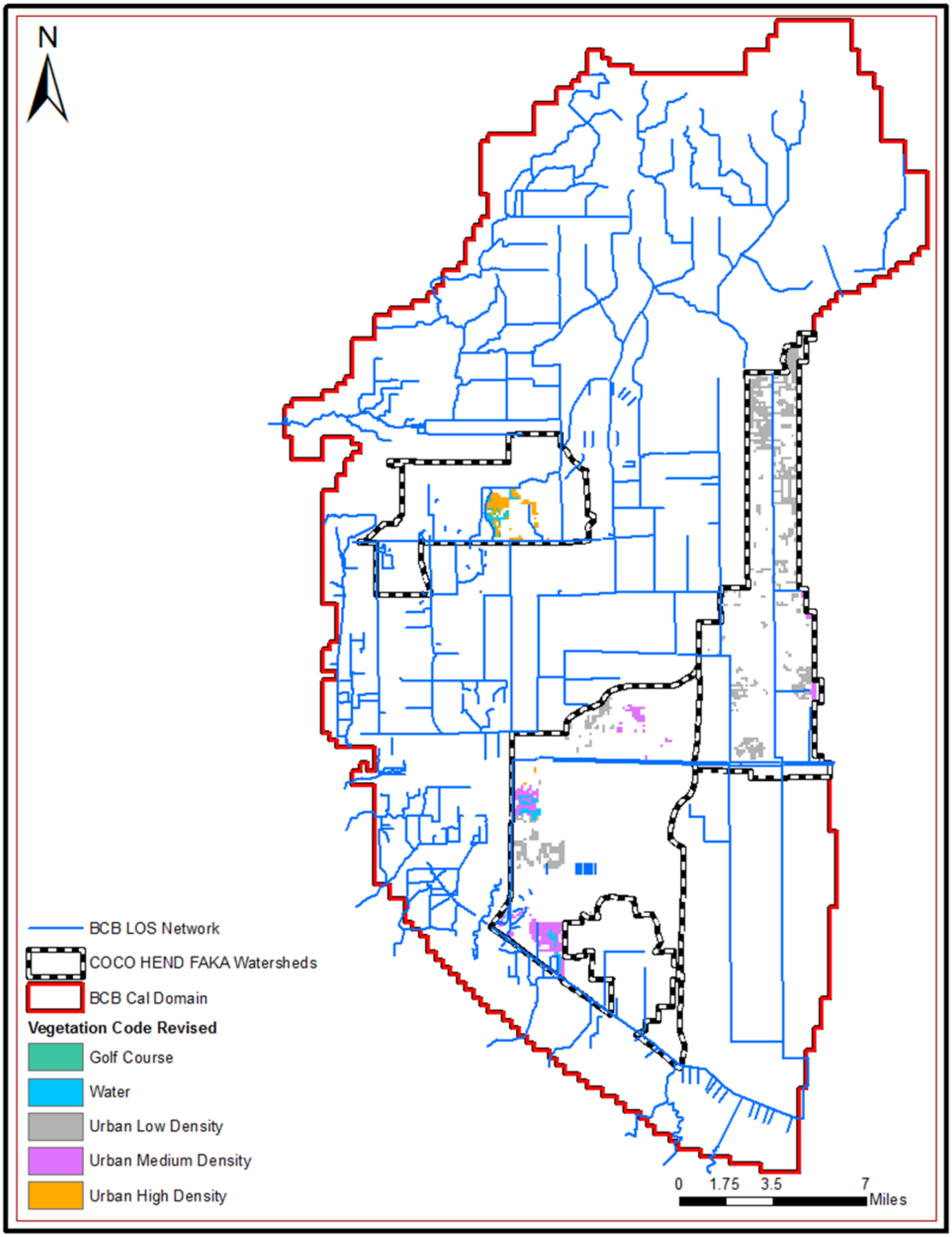

Figure 5.1.2 – Changes in MIKE SHE Vegetation Codes

For larger areas projected to be developed, a procedure like that proposed in the Task 1.2 (Test Bed Model) Report was applied. In this approach, MIKE-11 branches were added to represent stormwater detention ponds and their associated outfall structures. An iterative process was employed to size the outfall structure to pass the permittable peak discharge rate, based on Collier County's current permitting criteria (as amended in 2017). The allowable discharge rates varied from 0.04 cfs per acre in the Cocohatchee River Canal and Henderson Creek / Belle Meade South Basin up to 0.09 cfs per acre in the Faka Union North Basin.

Figure 5.1.3 shows an example of one of the areas where this approach was employed within the Cocohatchee Watershed. It was noted that the west development (Esplanade area) in the map was not shown in the future land use map as developed. But, it is currently under development and therefore is represented as fully developed in the future conditions model setup. Corresponding vegetation code and hydrologic parameters were changed to represent this residential and golf course development. A MIKE 11 link with outfall weir was added to the model to represent the combined stormwater storage and attenuation associated with the ponds in the development. In this instance, instead of sizing a conceptual weir to meet the allowable peak discharge, the width of the simulated outfall weir was input as the total width of three outfall weirs shown on the Esplanade Construction Plans.

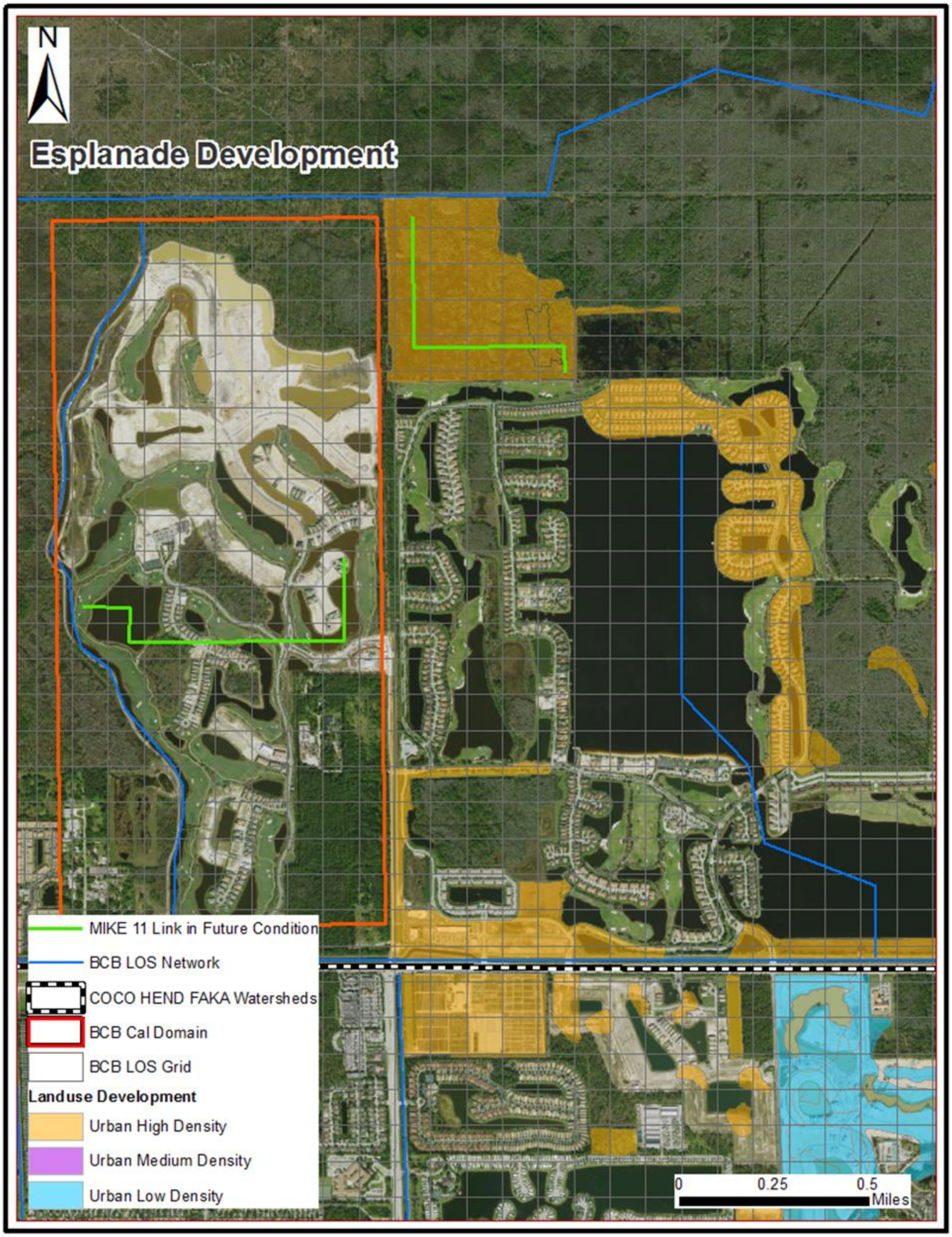

Figure 5.1.3 – Potential Urbanization – Location #1

Figure 5.1.4 shows a potential development in the northern part of the Henderson Creek / Belle Meade watershed. Because of its relatively remote location relative to the existing canal network, a different method was used in which a larger detention storage value and no MIKE 11 link added into model. A similar approach was used in smaller isolated land use change areas. In areas where the test-bed approach is used (where ponds and weirs are simulated in MIKE-11) a detention storage of 0.2 inches was used. In the other future urbanized areas, a value of 1.0 inch was used to represent stormwater retention/detention. No separated overland flow area covers this area, which allows overland flow to pass from the potential development to the surrounding grid cells.

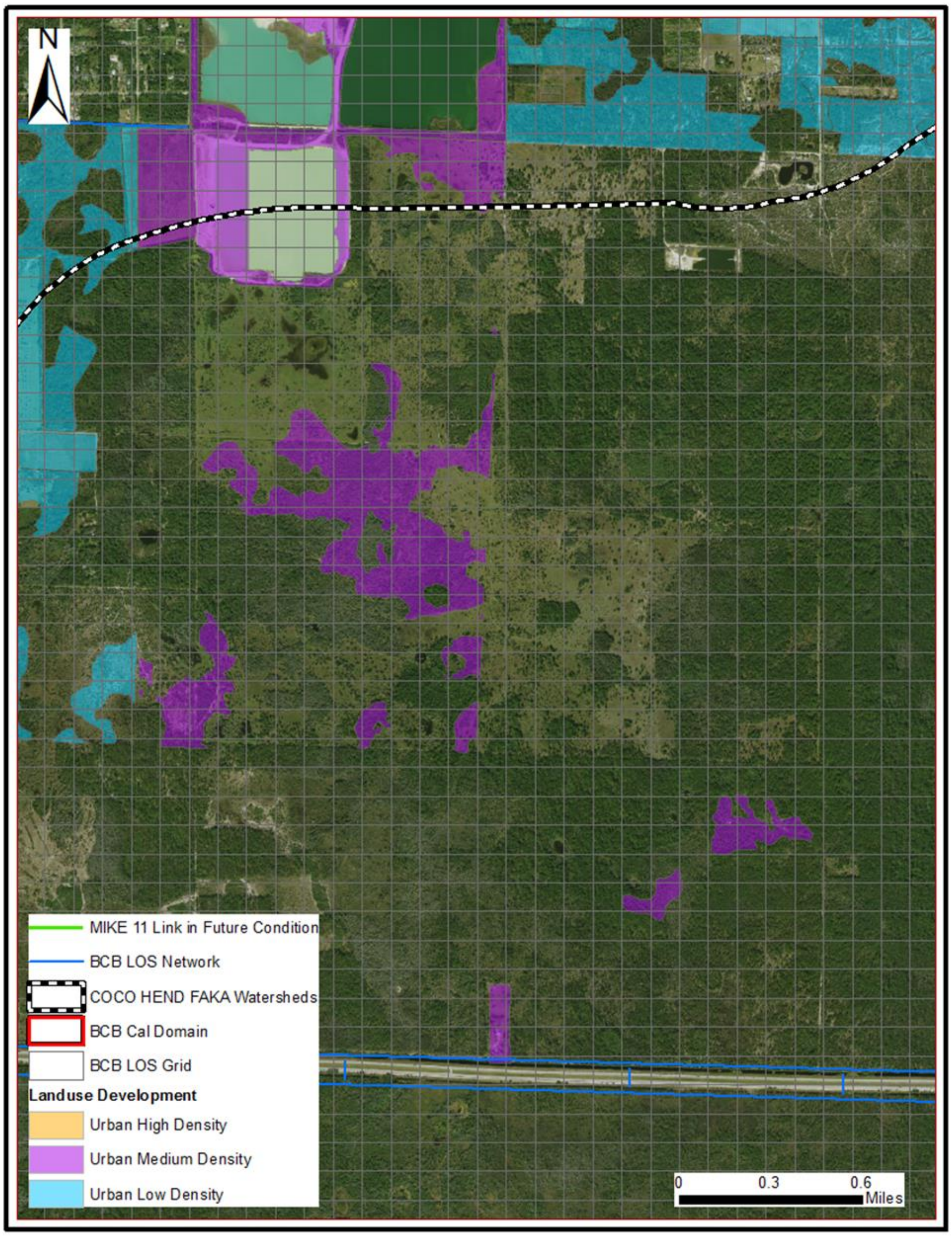

Figure 5.1.4 – Potential Urbanization – Location #2

Figure 5.1.5 shows an area of existing low-density residential, agriculture, and forested lands adjacent to the Henderson Creek Canal projected to become medium density urban. Like the Esplanade development, some of the areas currently under development were not projected to be developed in the future conditions land use mapping. For the areas shown in the map figure and outlined in red, the vegetation code was changed to urban medium density, and the related parameters in model were modified to represent the full build-out of those areas.

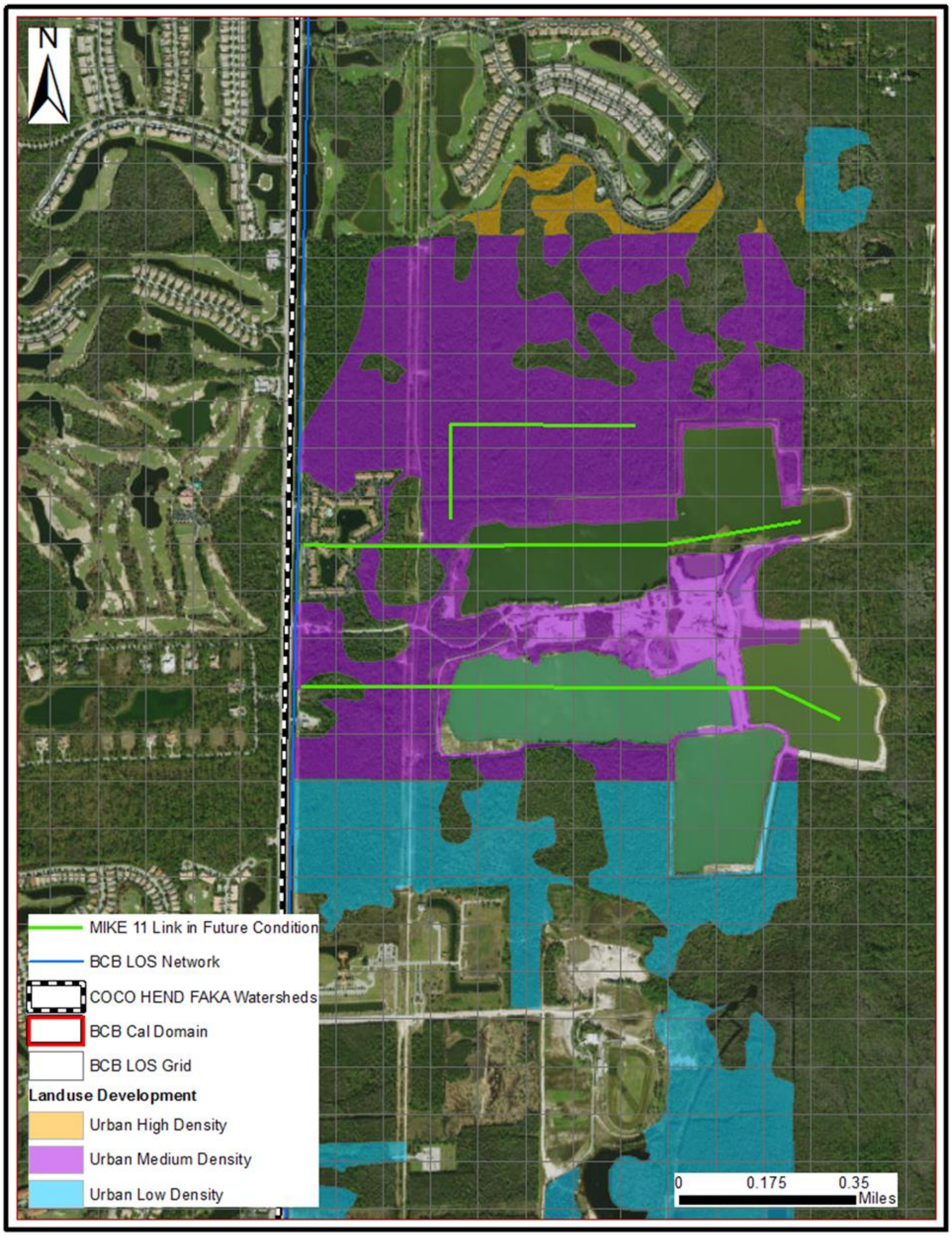

Figure 5.1.5 – Potential Urbanization – Location #3

Location #4 (**Figure 5.1.6**) shows an area of existing low-density residential, agriculture, and forested lands projected to become medium density urban (the Naples Reserve development). Like the Esplanade development, some of the areas currently under development were not projected to be developed in the future conditions land use mapping. For the areas shown in the map figure and outlined in red, the vegetation code was changed to urban medium density, and the related parameters in model were revised to represent full build-out.

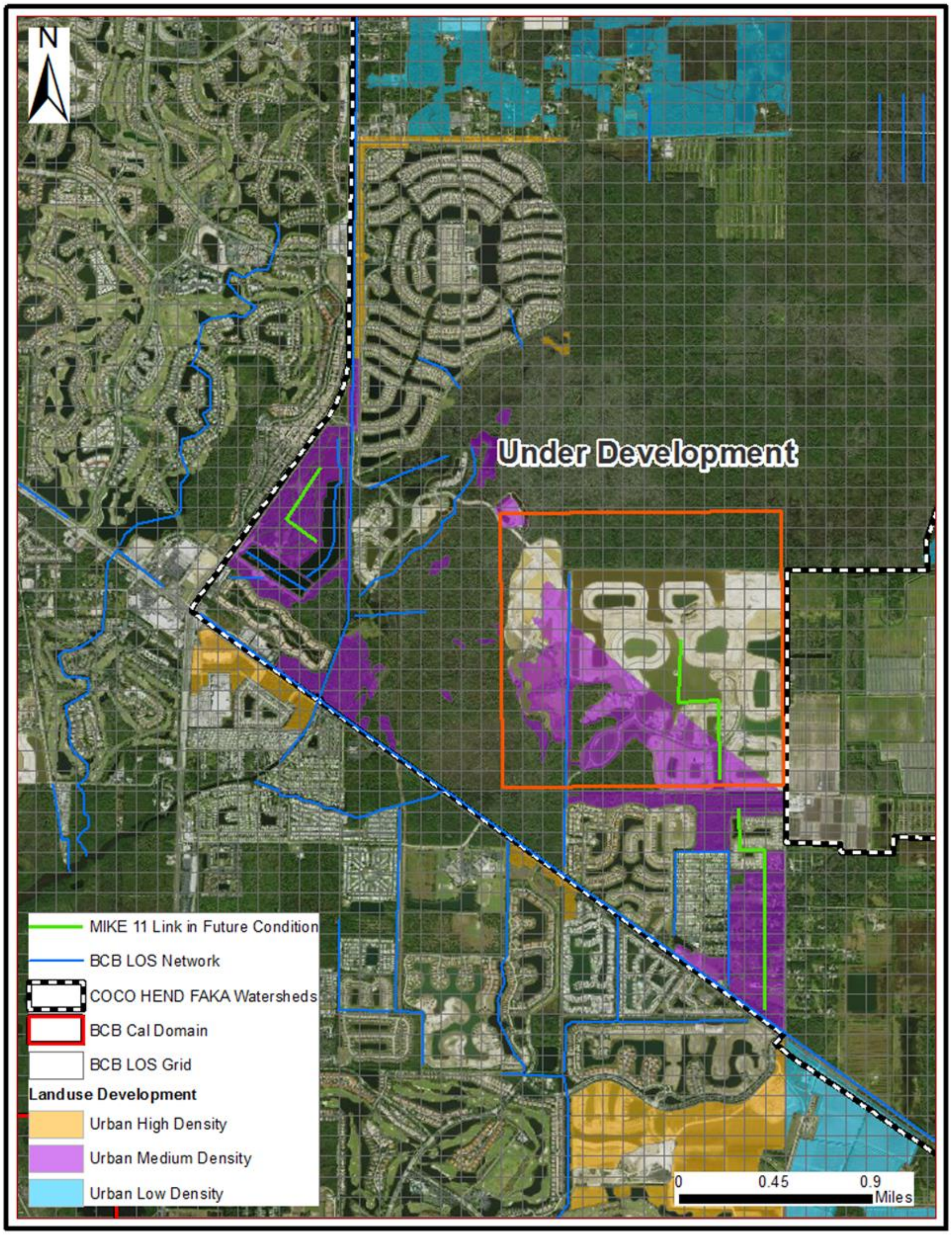

Figure 5.1.6 – Potential Urbanization – Location #4

Topography was changed in selected grid cells, based on a comparison of current topography and the current FEMA Flood Insurance Rate Maps in the areas identified for future development. If the grid cell topography was lower than the FEMA Base Flood Elevation (BFE), then the topographic elevation of the grid cell was raised to be near the FEMA BFE. It was assumed that the ground elevations would be a few tenths of a foot below the BFE, with the understanding that the floor slabs would typically be about one-half of a foot or so above the adjacent grade. **Figure 5.1.7** shows the changes in topography (Future Conditions minus Current Conditions).

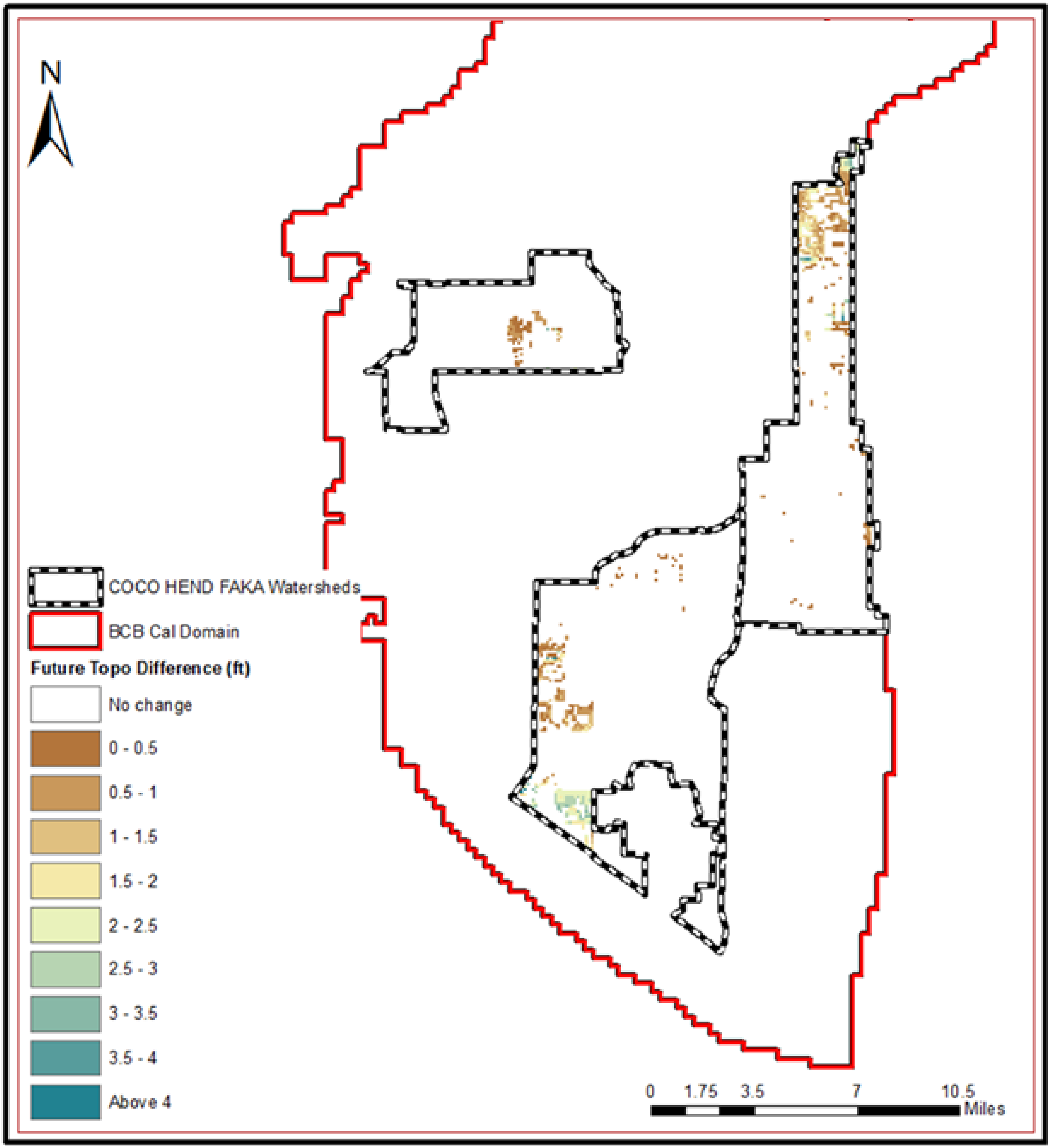

Figure 5.1.7 – Changes in Topography (Future Conditions minus Current Conditions)

Drain codes were set up for the areas identified for urbanization. Some of the drain codes in the current conditions model in this area were negative, which allowed the model to route drainage to local depressions. All new drain codes are positive values, which means that all drainage in the areas of future development are routed to either the nearest MIKE-11 h-point or a specified MIKE-11 h-point. This is based on the expectation that drainage will be improved in these areas to include positive outfalls for new development. **Figure 5.1.8** shows the drain code areas that were added or revised.

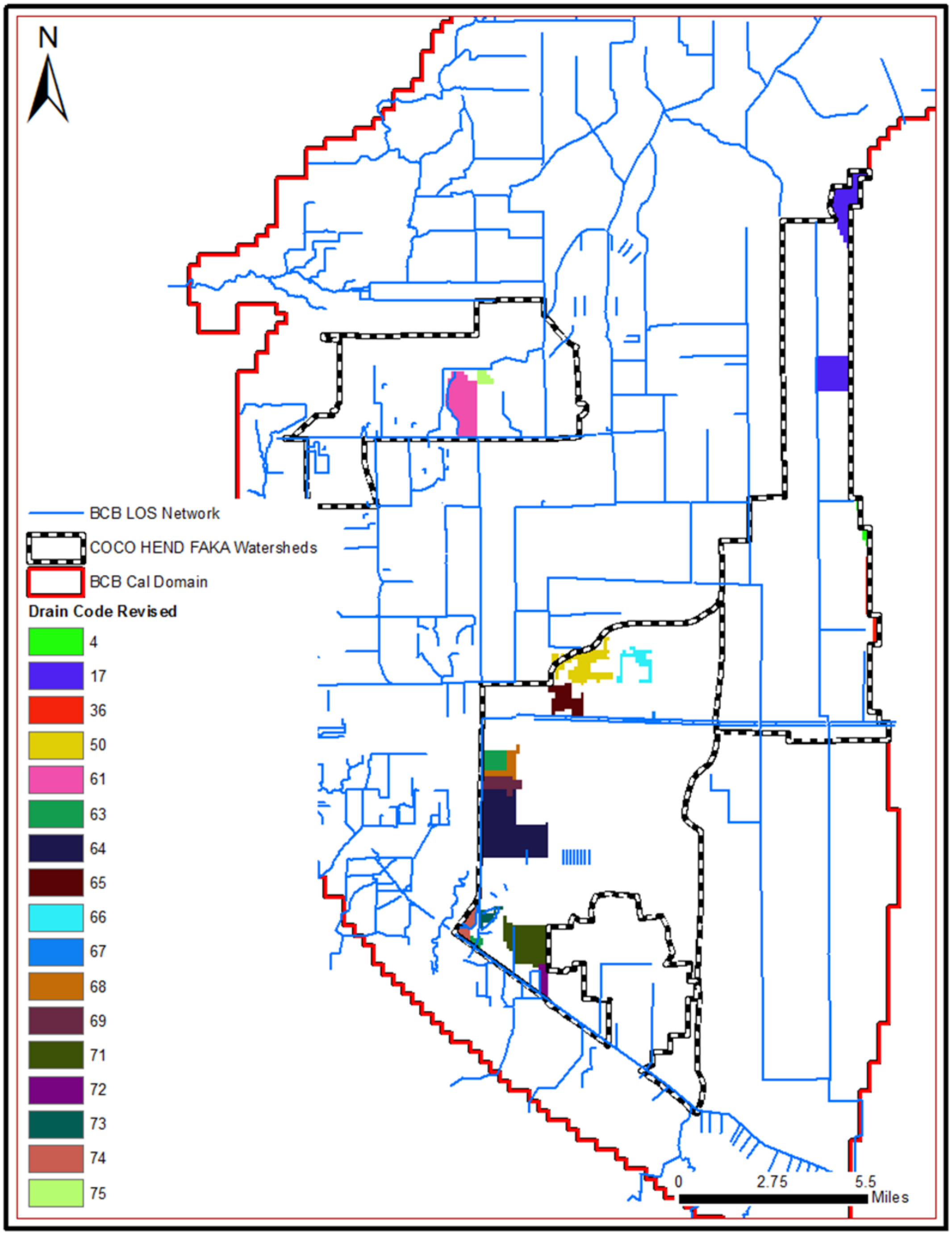

Figure 5.1.8 – Revised Drain Codes

Similarly, separated overland flow areas (SOLFAs) were adjusted to represent the changes in urban land use. **Figure 5.1.9** shows the SOLFAs that were added or modified. For the low-density developments it was assumed that offsite runoff would be accommodated rather than re-routed as is often the case with medium and high-density developments. SOLFA 155 (in dark green) represents the tie-back levee associated with the PSRP.

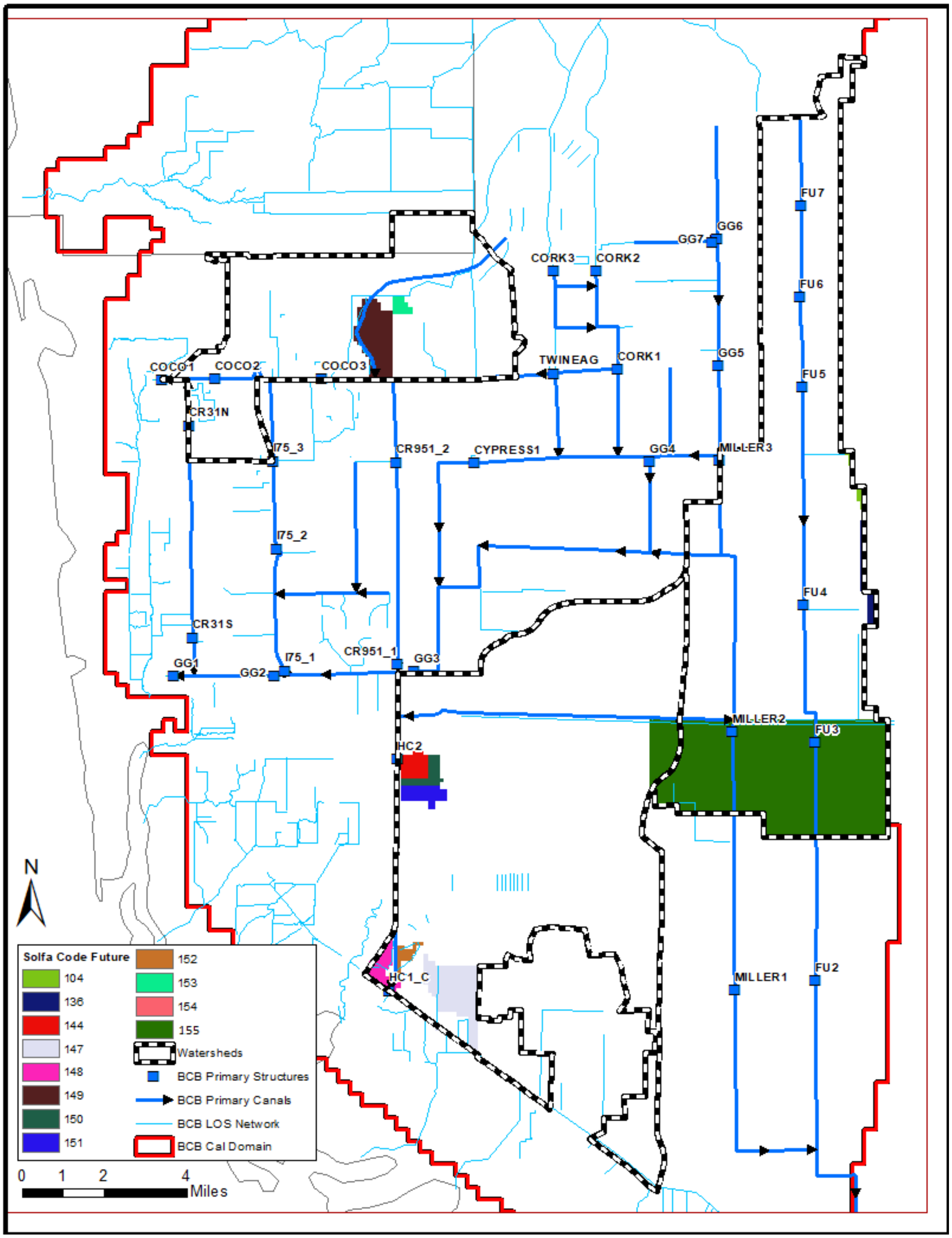

Figure 5.1.9 – Separated Overland Flow Areas (SOLFAs) Added or Revised

5.2 Future Sea Level

Future sea level is represented in the model using a combined approach involving three components of the model setup:

- Surface water (MIKE-11) tailwater conditions
- Groundwater boundary conditions
- Initial water table elevations

<u>MIKE-11 Tailwater Boundaries at Coastal Outfall Structures COCO1 and HC1:</u> Tailwater hydrographs for use in the future conditions design storm simulations were provided by the District in a spreadsheet and are documented in the report: *South Florida Water Management District, 2017. Flood Protection Level of Service Analysis for the Big Cypress Basin. Appendix C: Preparation of Boundary Conditions at the Tidal Structures. H&H Bureau, SFWMD, West Palm Beach, FL. 34 pp. June 21, 2017.* Tailwater hydrographs for three year-2065 sea level scenarios were included; low (SLR1), medium (SLR2), and high (SLR3). Tailwater hydrographs were provided for all four design storm events (5-year, 10-year, 25-year, and 100-year), but it was decided to run all three sea level rise scenarios only for the 25-year event. Separate MIKE-11 boundary files were created for each scenario. **Figures 5.2.1** and **5.2.2** show the 25-year tailwater hydrographs for the three future conditions scenarios, with current conditions shown for comparison, for structures COCO1 and HC1, respectively. **Table 5.2.1** lists the peak tailwater stages at COCO1 and HC1 for current conditions (SLR0) and the three future sea level scenarios.

Like current conditions, the dates for the rainfall distributions were adjusted so that the peak rainfall intensity lined up with the peak tailwater stage. This resulted in a spin-up period of approximately one week with normal tidal boundaries and no rainfall, and with the peak rainfall intensity and peak tailwater stage occurring on September 3, 2013 between noon and 1 PM.

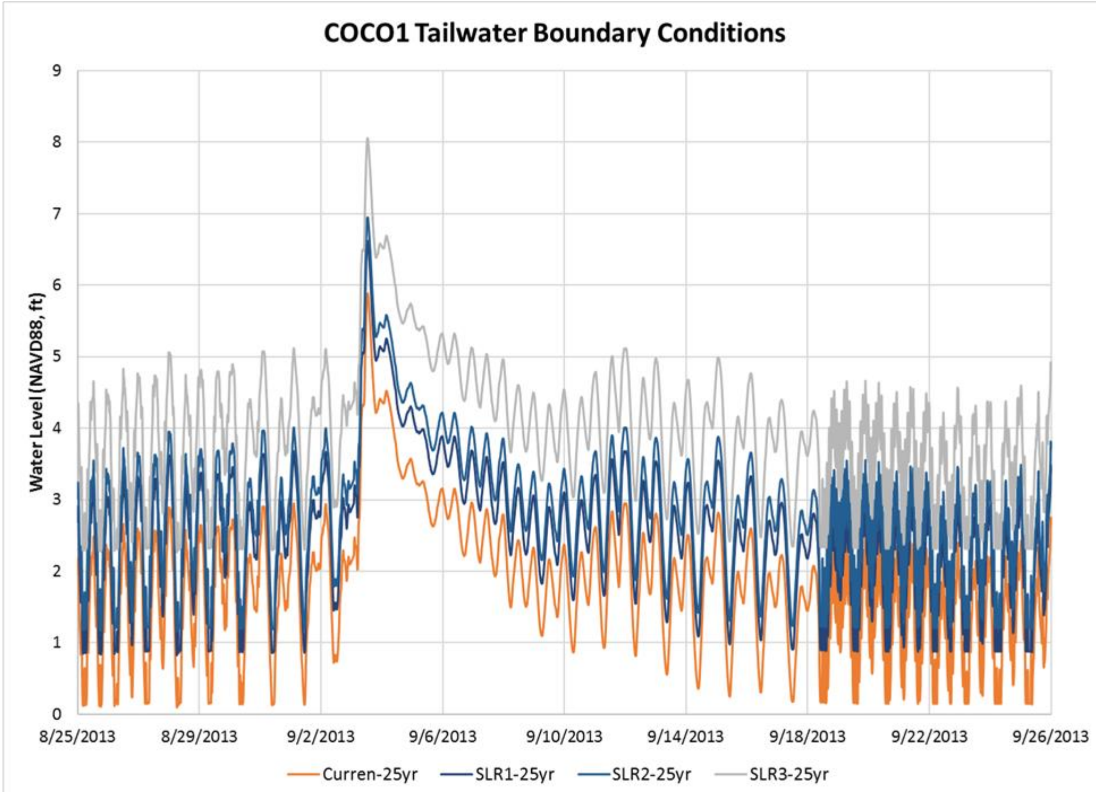

Figure 5.2.1 – COCO1 Tailwater Boundary Conditions for Future Sea Level (25-year event)

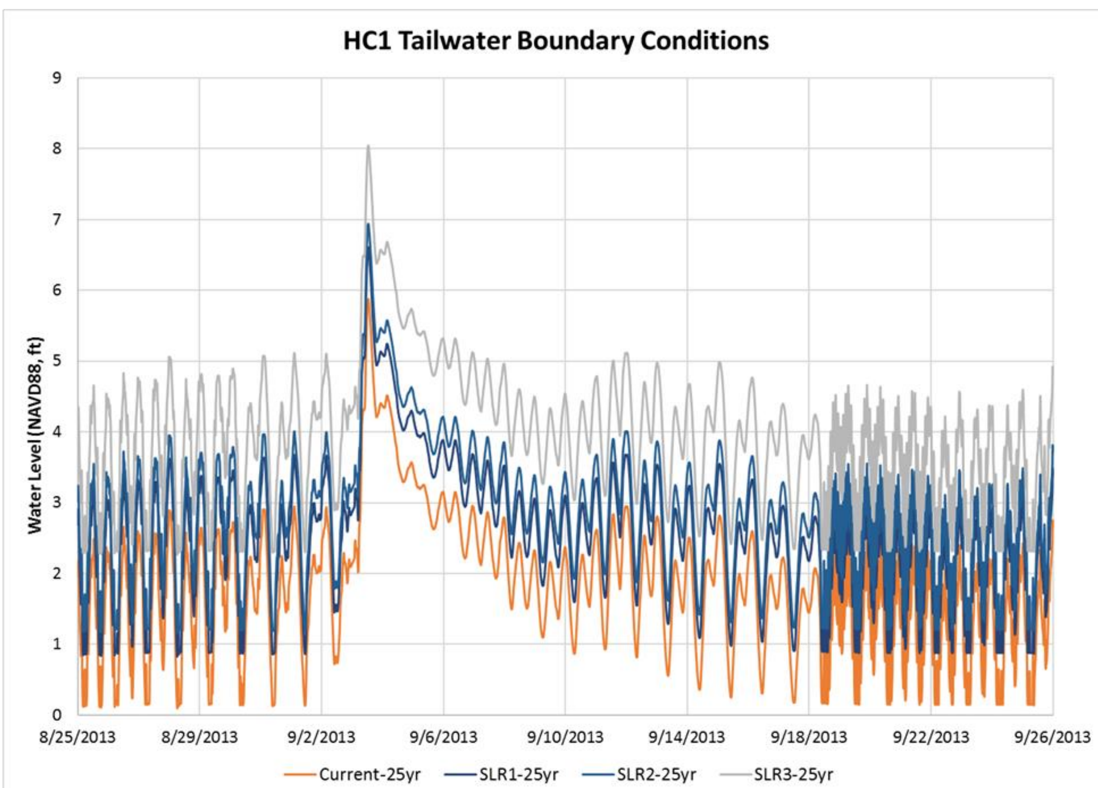

Figure 5.2.2 – HC1 Tailwater Boundary Conditions for Future Sea Level (25-year event)

Table 5.2.1 – Peak Tailwater Stages for Future Sea Level at COCO1 and HC1

Return Period	Future Sea Level Conditions Peak Tailwater Stage at COCO1 (ft. NAVD88)			Future Sea Level Conditions Peak Tailwater Stage at HC1 (ft. NAVD88)		
	SLR1	SLR2	SLR3	SLR1	SLR2	SLR3
5	5.66	5.99	7.10	5.62	5.95	7.06
10	6.06	6.39	7.50	6.05	6.38	7.49
25	6.62	6.95	8.06	6.61	6.94	8.05
100	7.54	7.87	8.98	7.52	7.85	8.96

Groundwater Boundary Conditions: The time-varying Groundwater boundary conditions along the coast for Layer 1 (surficial aquifer) of the MIKE SHE groundwater model were increased for each sea level rise scenario. The increase was equal to the projected sea level rise, relative to current conditions, associated with each scenario. The increase is 0.73 feet, 1.06 feet, and 2.17 feet respectively for SLR1, SLR2, and SLR3.

Initial Groundwater Levels: Initial groundwater levels were increased to account for higher tidal boundaries along the coast. Average tide levels are projected to increase, compared to current (year 2015) conditions, by 0.73 feet, 1.06 feet, and 2.17 feet for SLR1, SLR2, and SLR3, respectively. This increase will be manifested some distance inland in the form of elevated water tables.

Adjustments were made to the current conditions initial water table levels by first determining the average water table level at the cells along the coastal model boundary under current conditions. The model domain was divided into three parts based on watershed boundaries (Cocohatchee, Golden Gate and Henderson Belle Meade). For each part, 5 to 10 points along the model boundary were chosen. The corresponding initial water table level value for those points were extracted and averaged (3.38 ft for Cocohatchee, 0.24 for Golden Gate and -0.01 for Henderson). To adjust the initial water table level, for example, the average value of -0.01 feet for Henderson Belle Meade watershed was added the increase in sea level to determine the minimum coastal water table elevation for each of the three future conditions scenarios. The minimum coastal water table level for SLR3 was set to -0.01 + 2.17 = 2.16 feet NAVD88. To create the initial water table grid (dfs2) file for SLR3, all cells in the initial water table grid within Henderson Belle Meade part with values less than 2.16 were selected, and then those cells were set equal to 2.16. The same procedure was applied for the other two parts. **Figure 5.2.3** shows the increases in initial water table elevations for SLR3, as compared to current conditions. From the Figure, it is evident that the water table impacts would propagate several miles inland in some areas, particularly in the southern portions of the model domain.

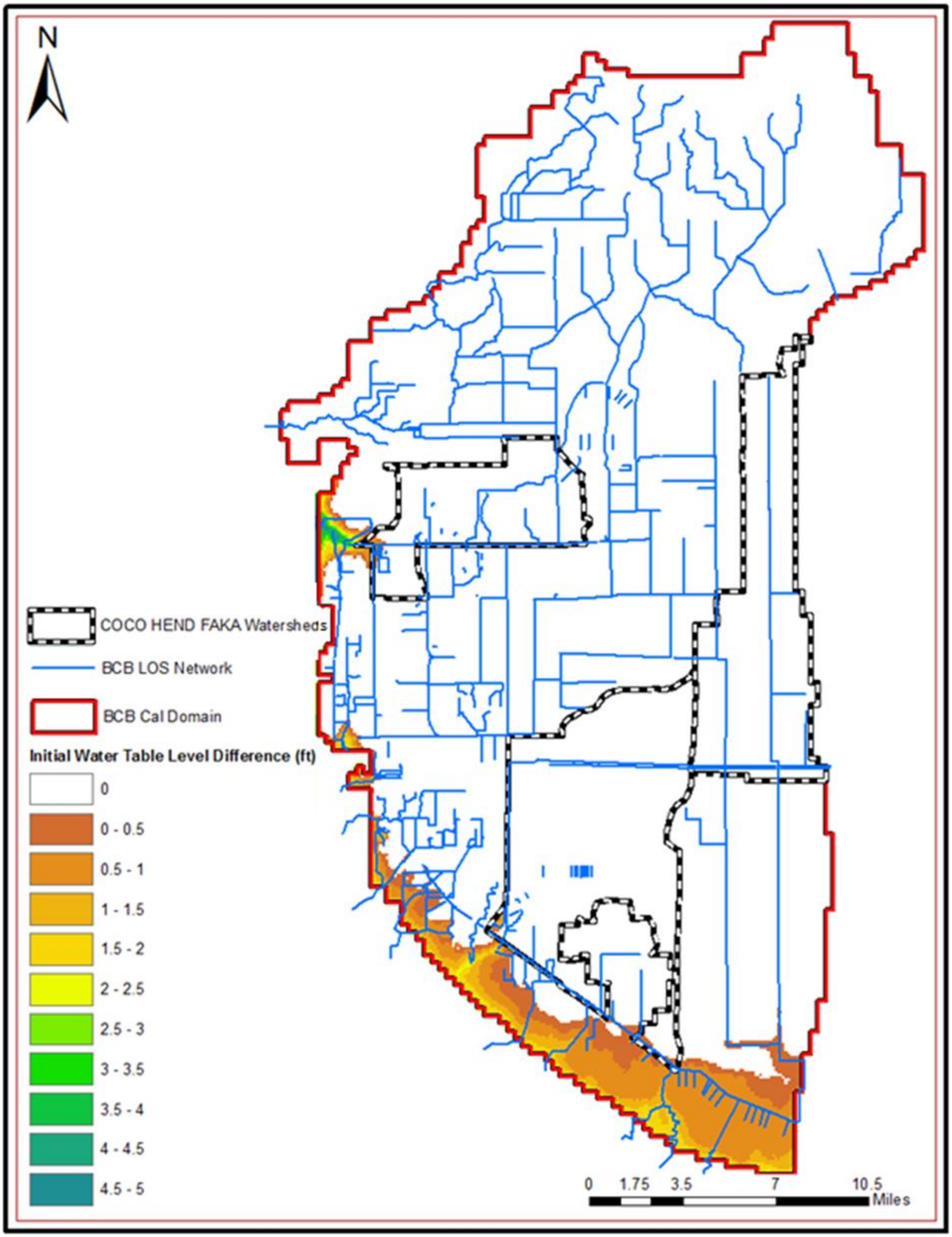

Figure 5.2.3 – Change in Initial Water Table Stages for SLR 3

6.0 Flood Protection Level of Service – Future Land Use

Initial model simulations of the 5-year, 10-year, 25-year, and 100-year, 3-day design storm were conducted with current sea level conditions and the future land use changes described in **Section 5.1**. **Appendix A** provides summary model results at primary control structures, while **Appendices F** through **I** provide the complete flow and stage hydrographs for each of the four design storm events. The performance of the existing infrastructure with future land use changes was evaluated with respect to the five performance metrics described in the remainder of this Section.

6.1 PM #1 – Maximum Stage in Primary Canals, Future Land Use

This is the peak stage profile in the primary canal system. The profile is developed for a range of design storms (5-year, 10-year, 25-year, and 100-year). The largest design storm that stays within the canal banks establishes the LOS of the primary canal system.

To evaluate this PM under future land use conditions within the Cocohatchee, Henderson/Belle Meade, and Faka Union Watersheds, peak stage profiles were prepared for all primary canals within the Cocohatchee, Henderson/Belle Meade, and Faka Union Watersheds. For reference, peak stage profiles are also shown for current conditions. Figures on the following pages depict the peak stage profiles for the following primary canals:

- Cocohatchee Canal (**Figure 6.1.1**)
- Henderson Creek Canal (**Figure 6.1.2**)
- Faka Union Canal (**Figure 6.1.3**)
- Miller Canal (**Figure 6.1.4**)

Bank elevations on the profile figures are generally based on the MIKE-11 cross section data. However, in several cases the bank elevations appeared suspect and were later modified by Taylor (based on the current LiDAR data) for use in preparing the profile plots. Also shown in the Figures are major roadway landmarks, control structures, and primary canal junctions.

Figure 6.1.1 indicates that the maximum stages for all storm events within the Cocohatchee Canal could be expected to increase because of the simulated land use changes. The increases are most pronounced upstream of COCO3. At the eastern terminus of the canal, the increases are a result of the land use changes within the Golden Gate Watershed; specifically, within the Corkscrew Canal as discussed in the Golden Gate Future Conditions FPLOS Report (Deliverable 4.2.1). A portion of the increase is associated with higher runoff volumes and displacement of floodwaters associated with the Esplanade development and other future development to the east of the Esplanade (refer to **Figure 5.1.3**). Although this development was designed to limit peak flows to the maximum allowed rate, the County's criteria does not address changes in overall runoff volume.

Figure 6.1.2 indicates that maximum stages for the 5-year event is projected to increase in the canal segment downstream of HC2. Small increases in the same segment are also projected for the 10-year event. This is a result of the potential future development along the Henderson Creek Canal. Because the medium-density developments were assumed to have adequate storage and attenuation to meet the current Collier County peak discharge requirements, the simulated increase in flow is most likely due to the low-density residential development and associated drainage parameterization. It is interesting to note that the same canal segment is projected to experience decreases in maximum stages for the 25-year and 100-year events, due to the additional storage provided by the connection of the existing mine pits east of the Henderson Canal (refer to **Figure 5.1.5**). The additional storage would more than offset any additional flow from the low-density areas during the 25-year and 100-year events but would apparently not be enough of a mitigating factor for the 5-year and 10-year storm events. Immediately upstream of HC1, slightly higher maximum stages are predicted to occur because of elevated topography, separated overland flow areas, and truncated canal cross-sections associated with the potential future development in the area.

The Faka Union and Miller Canals are both projected to experience increases in maximum stages for the 5-year and 10-year design storm events at the downstream ends of the evaluated segments (**Figures 6.1.3** and **6.1.4**). The downstream ends of these profiles coincide with the locations of the Faka Union and Miller Pump stations currently under construction as part of the PSRP. For these smaller events, the future conditions flood profiles are generally contained within the banks, although the 10-year profile on the Miller Canal is right at or just over the bank for a short segment immediately upstream of the pump.

For the 25-year and 100-year storm events, the maximum stages in the Miller Canal are projected to significantly increase and exceed the bank elevations immediately upstream of the pump station, while in the Faka Union Canal, maximum stages are projected to decrease for the 25-year and 100-year events upstream of the pump station. These results suggest that the Miller Pump Station would strain to handle the additional future flows in the Miller Canal under the assumed Future Conditions scenario. The additional future flows would result both from land use changes within the Miller Canal Watershed and from increased interbasin flows from the Golden Gate Watershed. It is important to note, however, that other performance measures evaluated (depth and duration of flooding) suggest the impacts of this would be localized to areas immediately adjacent to the Miller Canal and would likely not affect I-75.

The model results at the Miller and Faka Union pump locations prompted an examination of the assumed operating criteria. **Table 6.1.1** provides the assumed operating ranges and criteria for the Faka Union and Miller Pump Stations, as provided by the District. The two pump stations have operating rules whereby five high-flow pumps begin to be activated with the first high-flow pump turning on when the upstream water elevation reaches 6.2 and 6.45 feet NAVD88 for the Faka union and Miller Canals, respectively. The remaining four pumps turn on sequentially as water levels rise in 0.5-foot increments.

At the Faka Union Pump, initial model results showed only three of the five pump stations active during the peak of the 25-year event, while all five pump stations were activated in the Miller Pump Stations during both the 25-year and 100-year events. Because an actual 25-year flood would likely trigger activation of all five pumps, the operating ranges for the Faka Union high-flow pumps were reduced by 0.4 feet. However, this change did not result in additional pumps turning on during the 25-year event. Because the flows at this location are likely under-represented due to a portion of the Faka Union Watershed lying east of the model boundary (not included in the model domain), no further adjustments were made to the pump operating rules.

Table 6.1.1 – Faka Union and Miller Pump Stations Operating Criteria from SFWMD (Ref: Jocelyn Nageon De Lestang, SWFWMD, email communication 3/2/18)

STAGE	STAGE	Discharge Capacity FAKA Union Pumps		Discharge Capacity Miller Pumps	
[ft-NGVD]	ft-NAVD88	[cfs]		[cfs]	
6.25	4.95	100	Low -Flow (100cfs)	75	Low -Flow (75 cfs)
6.50	5.20	100		75	
6.75	5.45	200		75	
7.00	5.70	200		75	
7.25	5.95	300		75	
7.50	6.20	300		310	High Flows Kick In (+235 cfs)
7.75	6.45	770	High Flows Kick In (+470 cfs)	310	
8.00	6.70	770		310	
8.25	6.95	1240		545	
8.50	7.20	1240		545	
8.75	7.45	1710		780	
9.00	7.70	1710		780	
9.25	7.95	2180		1015	
9.50	8.20	2180		1015	
9.75	8.45	2650		1250	
10.00	8.70	2650		1250	
10.25	8.95	2650		1250	
10.50	9.20	2650		1250	
10.75	9.45	2650		1250	
11.00	9.70	2650		1250	
11.25	9.95	2650		1250	
11.50	10.20	2650		1250	
11.75	10.45	2650		1250	
12.00	10.70	2650		1250	

In the upper reaches of the Miller Canal, increased flow from the Golden Gate Canal through the Miller3 structure is projected to cause higher maximum stages in the upper reaches of the Miller Canal. This is a result of land use changes within the Golden Gate watershed discussed in the Golden Gate Future Conditions FPLOS Report (Deliverable 4.2.1). These increased flows through Miller3 resulting from this scenario would contribute to the aforementioned flow capacity issue at the PSRP Miller Pump Station.

In the upper reaches of the Faka Union Canal, the conversion of native lands to low-density urban is projected to result in higher maximum stages, approaching or exceeding a two-foot rise at the upstream end for all design storm events. The land use changes in this area are primarily assumed to be ultimate build-out of previously platted single family lots in the low-density Golden Gate Estates.

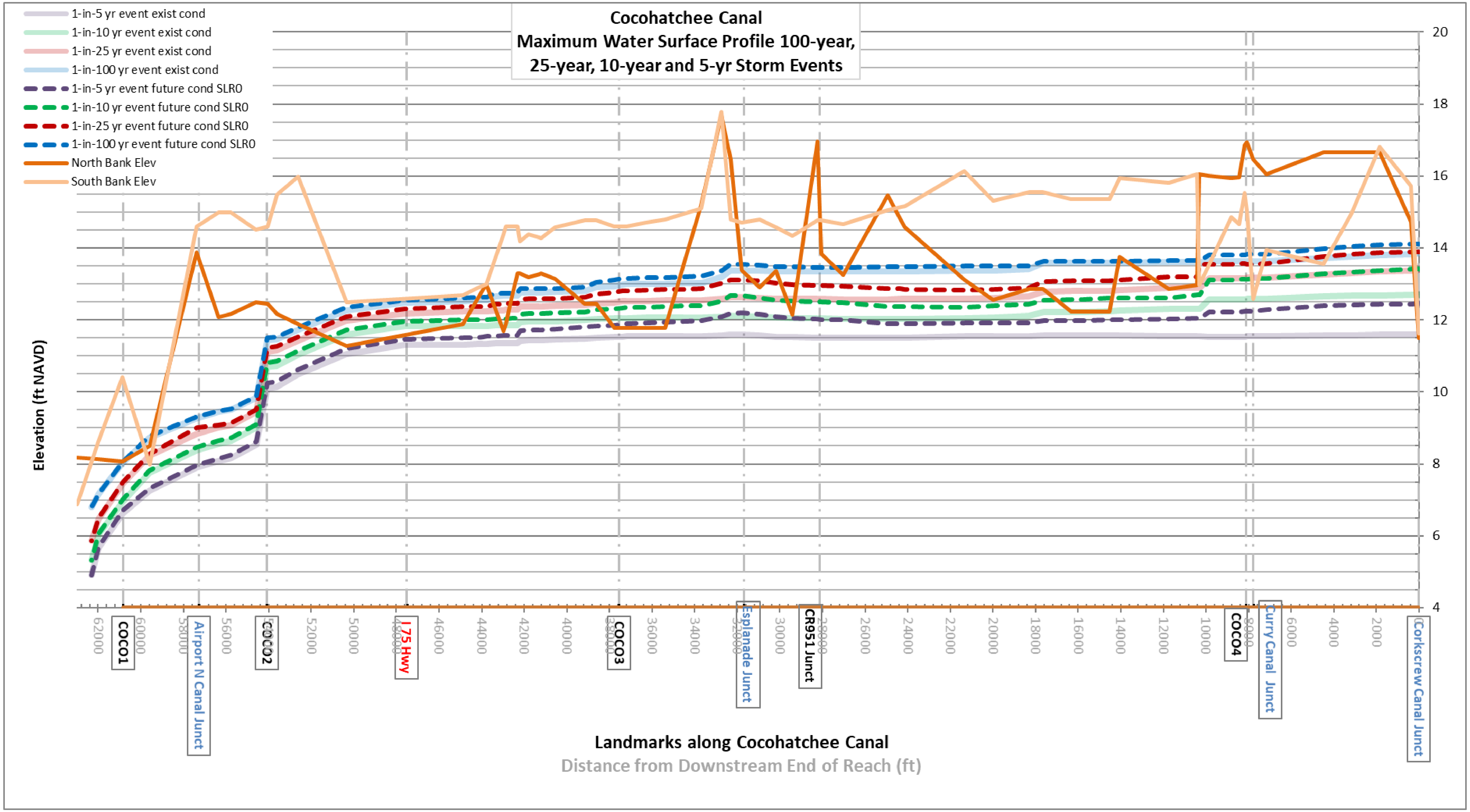

Figure 6.1.1 – Cocohatchee Main Canal Peak Stage Profiles, Current and Future Land Use

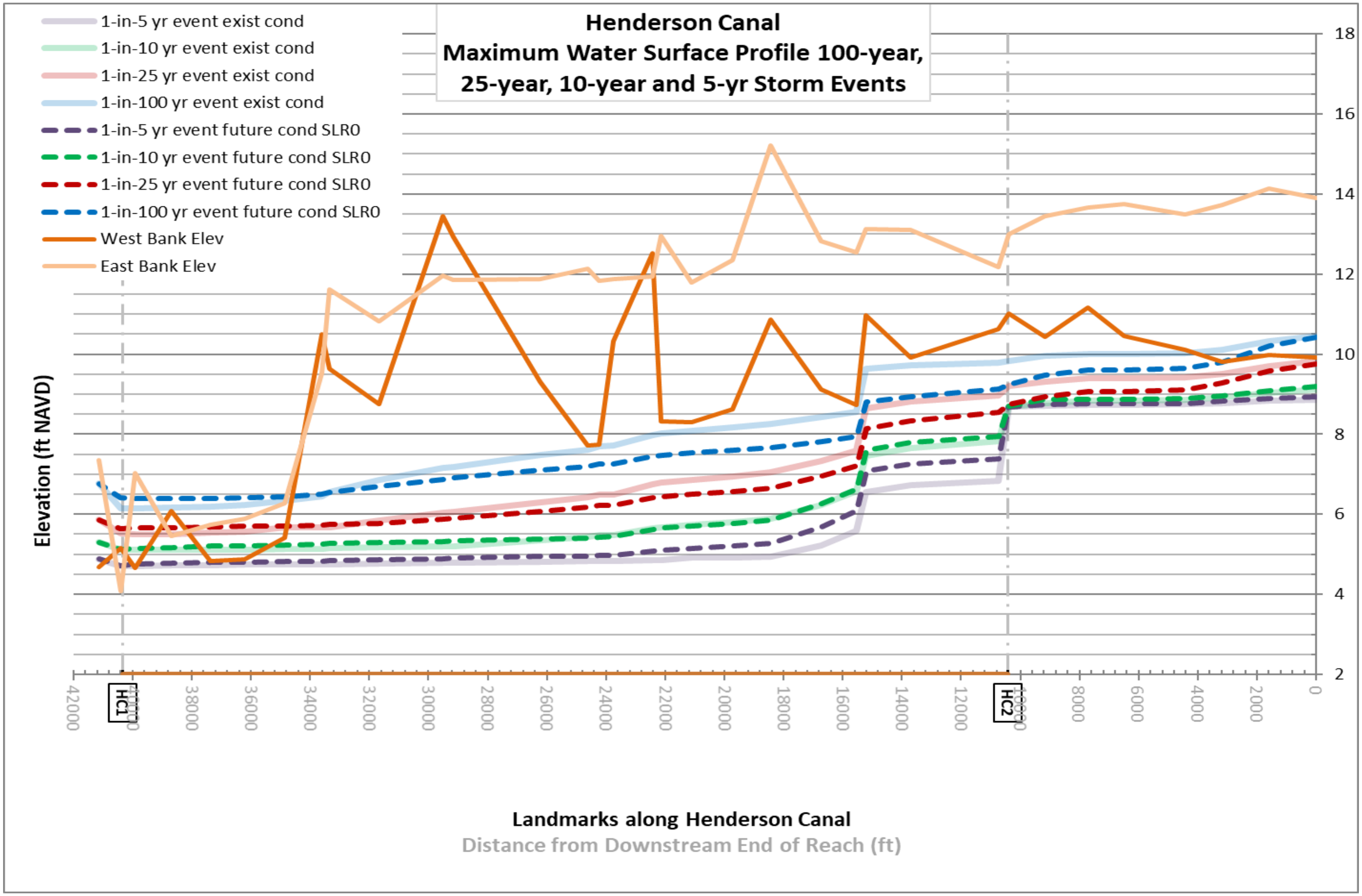

Figure 6.1.2 – Henderson Creek Canal Peak Stage Profiles, Current and Future Land Use

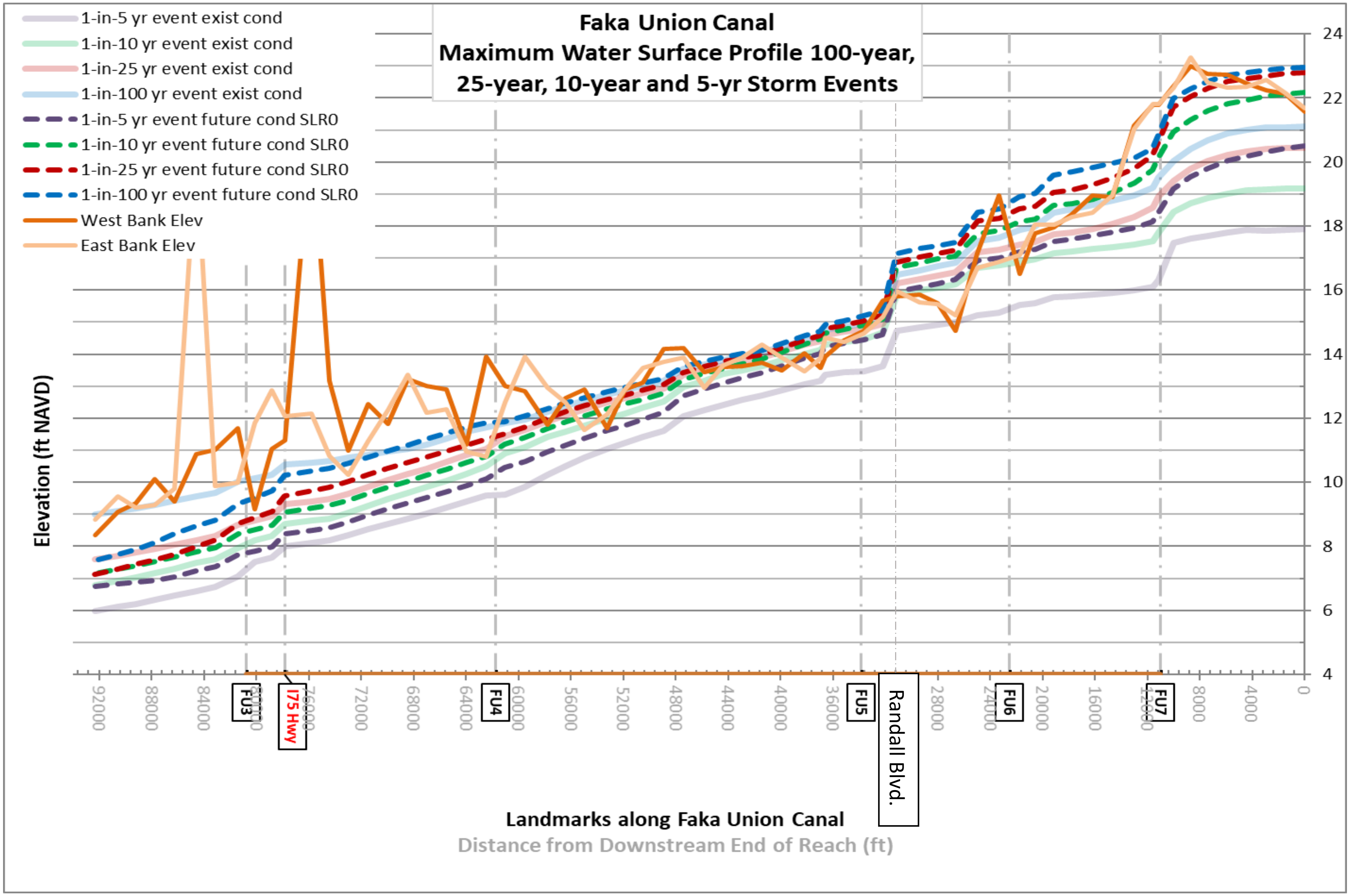

Figure 6.1.3 – Faka Union Canal Peak Stage Profiles, Current and Future Land Use

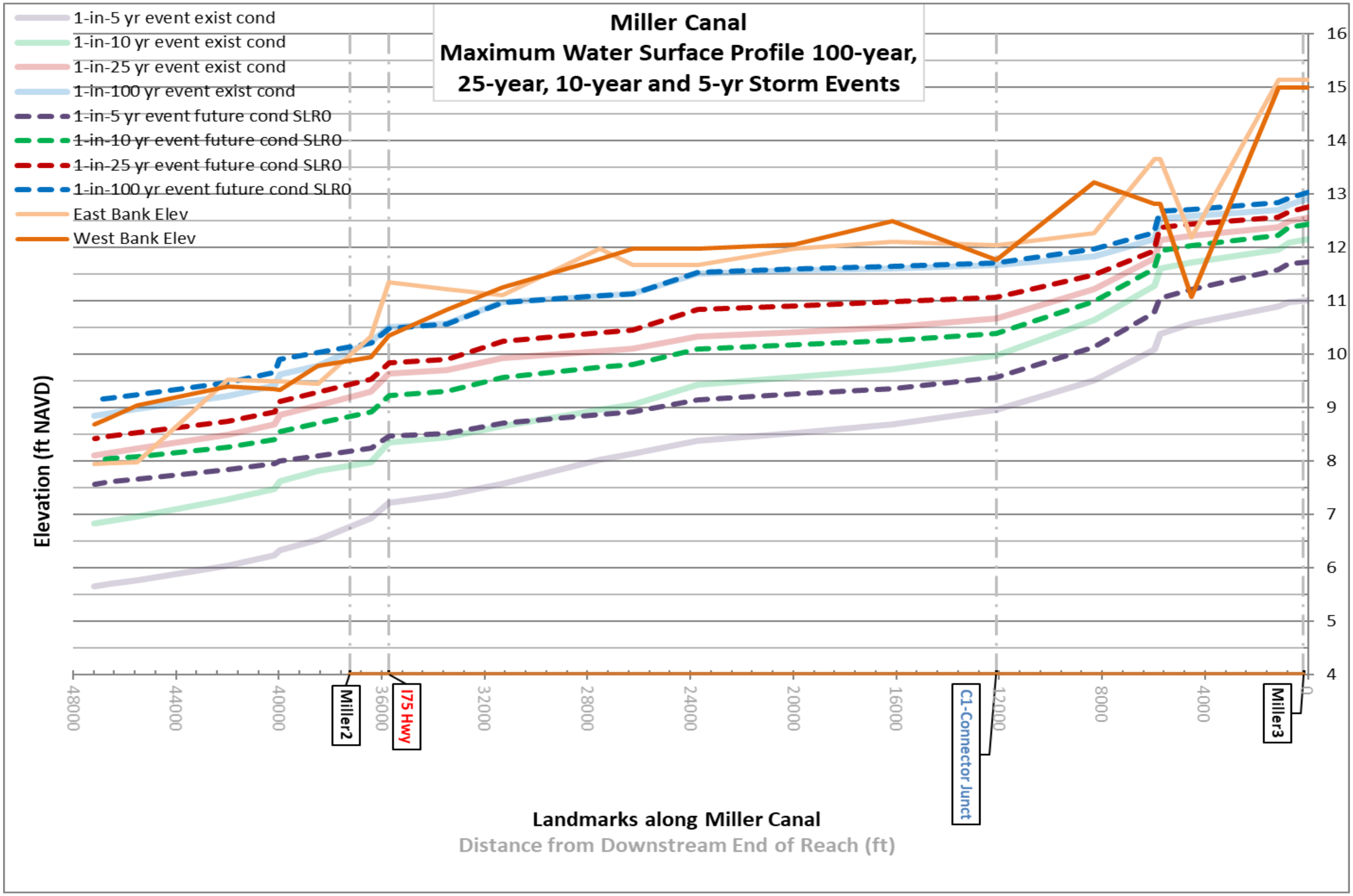

Figure 6.1.4 – Miller Canal Peak Stage Profiles, Current and Future Land Use

6.2 PM #2 – Maximum Daily Discharge Capacity through the Primary Canals, Future Land Use

PM #2 is the maximum discharge capacity throughout the primary canal network. Discharge is calculated for defined canal segments as areally weighted flow, in units of cubic feet per second per square mile of contributing area. Canal segments are generally those segments between water control structures. For example, the segment associated with structure COCO1 is the Cocohatchee Canal between structures COCO1 and COCO2, and the contributing area is defined as only the area contributing runoff to that segment. **Table 6.2.1** lists the canal segments identified for this analysis, where each segment is identified by the downstream structure. The table also identifies the contributing area for each canal segment, and the discharge capacity calculated for each segment associated with the 25-year, 3-day design storm event for both current conditions and future land use conditions.

Discharge capacity under current and future land use conditions was calculated by subtracting the hydrographs at all inflow points to each segment from the segment's outflow hydrograph(s), and then dividing the peak of the resulting net discharge hydrograph by the segment's contributing area. Tidal effects were filtered by using a 12-hour moving average of discharge.

Table 6.2.1 – Water Control Catchment Inflow and Outflow Points and 25-year Discharge Capacity for Current and Future Land Use Conditions

Structure\ Segment	Inflow Point(s)	Outflow Point(s)	Water Control Catchment Area (sq.mi)	25-year Peak Discharge Capacity (cfs/sq.mi)	
				Current	Future
HC2	BEGINNING OF HENDERSON	HC2	3.07	14	15
HC1	HC2	HC1	71.08	87	82
COCO3	BEGINNING OF COCOHATCHEE, BEGINNING OF CR951	COCO3	17.03	25	30
COCO2	COCO3	COCO2	6.08	80	75
COCO1	AirportN, COCO2	COCO1	3.37	100	98
FU7		FU7	4.09	48	83
FU6	FU7	FU6	3.90	54	55
FU5	FU6	FU5	4.81	39	46
FU4	FU5	FU4	11.66	47	49
FU3	FU4	FU3	8.39	42	
Miller2	Miller3, C-1 Connector	Miller2	11.93	50	
FU Pump	FU4	FU Pump	14.88		33
Miller Pump	Miller3, C-1 Connector	Miller Pump	15.56		67

Figure 6.2.1 shows the contributing areas ("water control catchments") draining to each canal segment. The catchment polygons were taken from the District's Arc Hydro Enhanced Database (AHED), and were used to determine the contributing areas listed in **Table 6.2.1.** For completeness, the water control catchments and associated discharge capacities are also shown for the Golden Gate watershed.

Under future land use conditions, the discharge capacity is projected to increase from 25 to 30 cfs per square mile for canal segment COCO3. This is partly due to increased flows from the Corkscrew Canal within the Golden Gate Watershed (discussed later in this Section), and partly due to increased flow volumes from the Esplanade and other conceptual future simulated development east of the Esplanade. This also resulted in the segment associated with COCO2 having higher inflows. With the outflows from COCO2 relatively unchanged, the net result would be a decrease in the computed discharge capacity for segment COCO2, from 80 to 75 cfs per square mile.

Under future land use conditions, the discharge capacity is projected to decrease slightly in the canal segments associated with HC1. This is likely due to the projected decrease in flow and stage for the 25-year and 100-year events, due to the additional storage provided by the connection of the existing mine pits east of the Henderson Canal (refer to Figure 5.1.5). The additional storage would more than offset any additional flow from the future low-density development areas along the canal during the 25-year and 100-year design storm events, but the connected mine pits would apparently not be a factor for the 5-year and 10-year storm events.

Canal segment FU7 has no inflows, and the outflows would increase due to the potential future land use changes from undeveloped to low-density residential. Thus, the computed discharge capacity for this canal segment would be higher under future land use conditions. Canal Segments FU5 and FU6 also have similar projected future land use changes (albeit to a lesser degree) that would further increase discharge rates in the canal, also resulting in higher computed discharge capacities for the 25-year event. However, it is important to note that the 10-year, 25-year, and 100-year peak stage profiles between structures FU5 and FU7 are out-of-bank, and therefore the true discharge capacity at this structure would be less than the computed 25-year discharge capacity.

Structure FU3 is proposed to be removed as part of the PSRP. For existing Canal segment FU3, the area was replaced by the area between FU4 and the proposed pump station. The area of this water control catchment was increased from 8.39 square miles to 14.88 square miles. Similarly, Structure Miller2 is also proposed to be removed, and for that canal segment the water control catchment area was replaced by the area between Miller3 and the Miller Pump Station, incorporating an additional 3.6 square miles of relatively undeveloped land. Because of the changes in downstream structure type and location, direct comparisons of the respective discharge capacities of these canal segments between current and future condition scenarios is not possible.

Overall, discharge capacity was found to be minimally impacted or slightly improved as a result of the simulated future development.

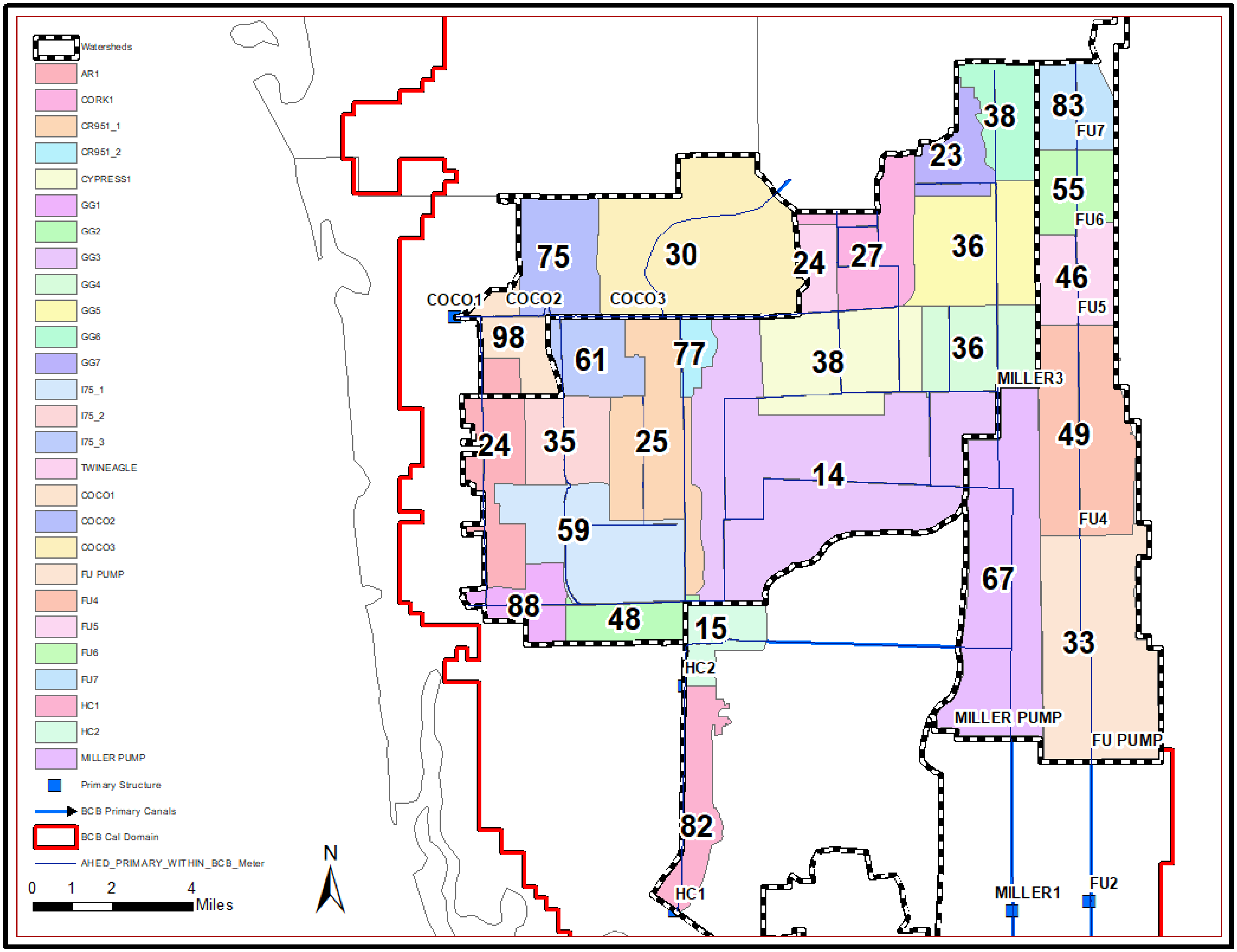

Figure 6.2.1 - Control Structure Catchments for Calculating PM#2, with 25-year Discharge Capacities for Future Land Use Conditions

Interbasin Flows

The future condition interbasin flows from the Corkscrew Swamp into the Cocohatchee and Golden Gate Watersheds previously discussed in **Section 4.2** were examined and compared with the current conditions inter-basin flows. No significant changes in these inflows were predicted to result from the future land use changes.

Table 6.2.2 provides a comparison of simulated future conditions interbasin flows from the Golden Gate Watershed. In the table, flows are presented as maximum 12-hour running averages. Interbasin flows from the Golden Gate Watershed to the Cocohatchee Canal are predicted to increase significantly under future conditions, while 12-hour average interbasin flows from the Golden Gate Watershed to the Faka Union Watershed (via the Miller Canal) are predicted to remain about the same or decrease slightly overall, for the 25-year and 100-year events.

Table 6.2.2 – Comparison of Maximum Interbasin flows from Golden Gate Watershed (12-hr average, cfs)

	5-Year		10-Year		25-Year		100-Year	
	Current	Future	Current	Future	Current	Future	Current	Future
COCO4 Positive (into Golden Gate)	26	48	45	58	51	61	54	58
COCO4 Negative (into Cocohatchee)	0	0	-10	-32	-21	-23	-34	-38
Miller3 (into Miller Canal)	218	240	311	326	436	409	503	467
C-1 Connector (into Miller Canal)	45	20	45	32	38	38	33	43

It should be noted that although not evident in the 12-hour average flows reported above, the peak 1-hour flows are predicted to increase significantly under future conditions through the Miller3 structure (e.g., an increase of 77 cfs for the 25-year event; refer to Table 2 in Appendix A). These increased peak inflows, combined with additional runoff associated with the ultimate future build-out of previously platted residential lots in the Northern Golden Gate Estates, are projected to result in peak flows that exceed the discharge capacity of the Miller Pump Station for a short duration. The increased peak flows are evident in the PM#1 flood profiles reported in the preceding section. It is important to note, however, that other performance measures evaluated (depth and duration of flooding) suggest the impacts of this would be localized to areas immediately adjacent to the Miller Canal and would likely not affect critical infrastructure such as I-75.

6.3 PM #5 – Frequency of Flooding, Future Land Use

In this PM, the flood elevations or depths of overland flooding are evaluated for a range of design storms (5-year, 10-year, 25-year, and 100-year). These flood depths/elevations can then be compared with elevations of features such as buildings and roadways, where such information exists. For the purposes of this BCB LOS evaluation, flood inundation maps were developed from the gridded MIKE SHE model output for each storm event, in the form of maximum depths of depth of overland water. These overland water depths, with the 500-foot model grid spacing, were used to develop a TIN for presentation purposes. The resulting flood inundation maps are presented on **Figures 6.3.1 through 6.3.4** for each of the four design storm events.

To provide a comparison of the future land use results with the current conditions model results, inundation difference maps were created for the 10-year, 25-year, and 100-year design storm events (**Figures 6.3.5** through 6**.3.7**). The current conditions overland flood depths were subtracted from the future land use overland flood depths; positive values mean flood depths would increase and negative values indicate flood depths would decrease under future land use conditions.

In many areas the flood depths are projected to decrease under future land use conditions. This results from a combination of higher topography and the resulting displacement of floodwaters within the footprint of the potential developments (refer to **Figure 5.1.7**) and simulated drainage improvements associated with the future land use conversions. However, there are some areas, outside the footprints of the potential developed areas, where flood levels are projected to increase due to the simulated future development. The most significant example of this is the area north of the Naples Reserve development which is shown on **Figure 5.1.6**. Maximum flood depths are projected to increase by up to 0.3 feet for the 10-year and 25-year events, and to a lesser depth/extent for the 100-year event. Further north in the Henderson Creek / Belle Meade Watershed, areas adjacent to a potential low-density residential development east of the Collier Blvd./Hammock Road intersection are projected to experience slight increases in maximum water level.

Small increases (on the order of 0.1 to 0.2 feet) are projected in a low-lying area north and east of the Esplanade development north of the Cocohatchee canal. In the Faka Union Watershed, maximum water depths along the Faka Union Canal upstream of FU4 are projected to increase along a narrow corridor adjacent to the canal. This is consistent with the PM#1 results which show higher flood profiles within the canal, downstream of areas in the northern Faka Union Watershed which are projected to convert to low-density urban.

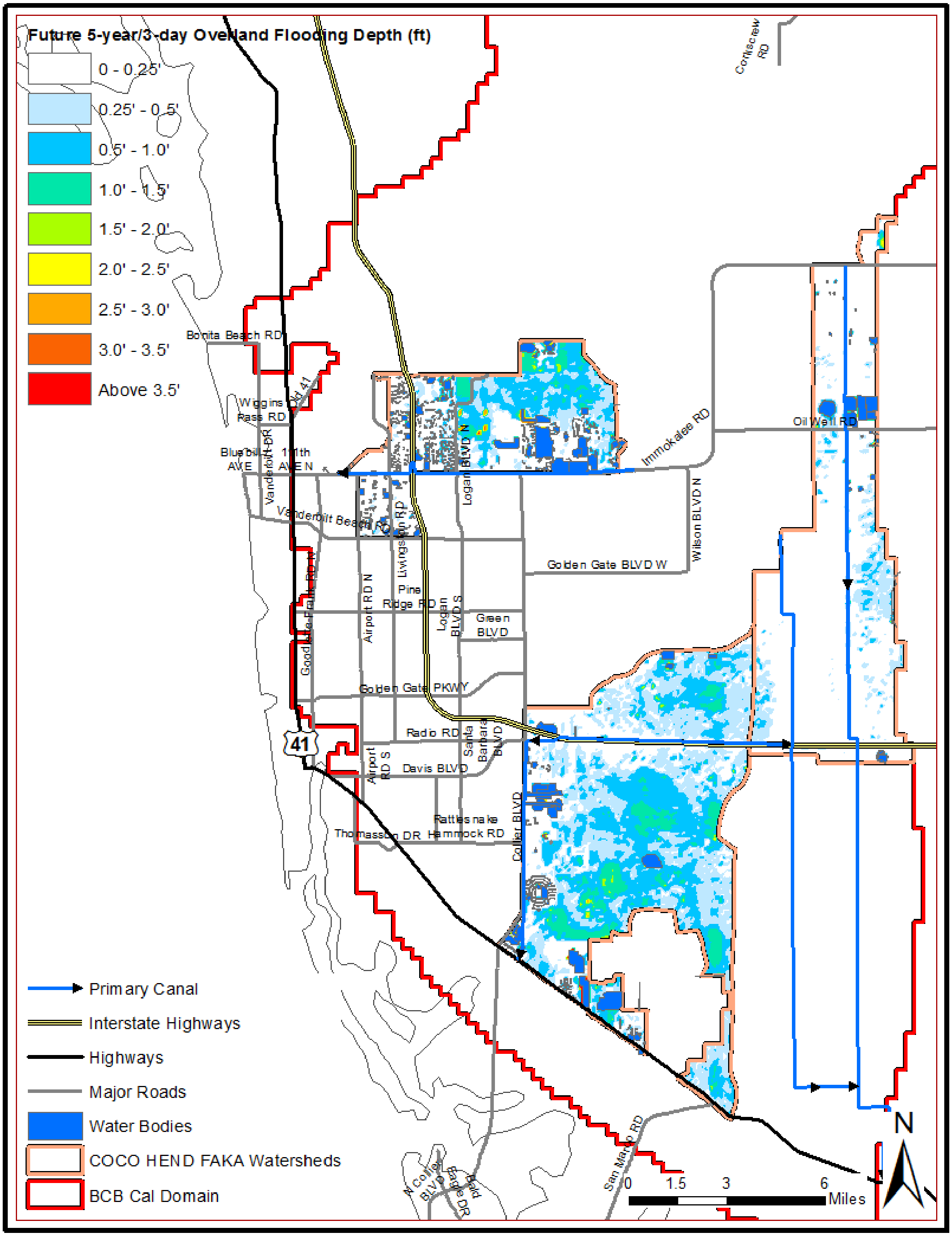

Figure 6.3.1 – Future Land Use Conditions Inundation Map for 5-year Design Storm Event

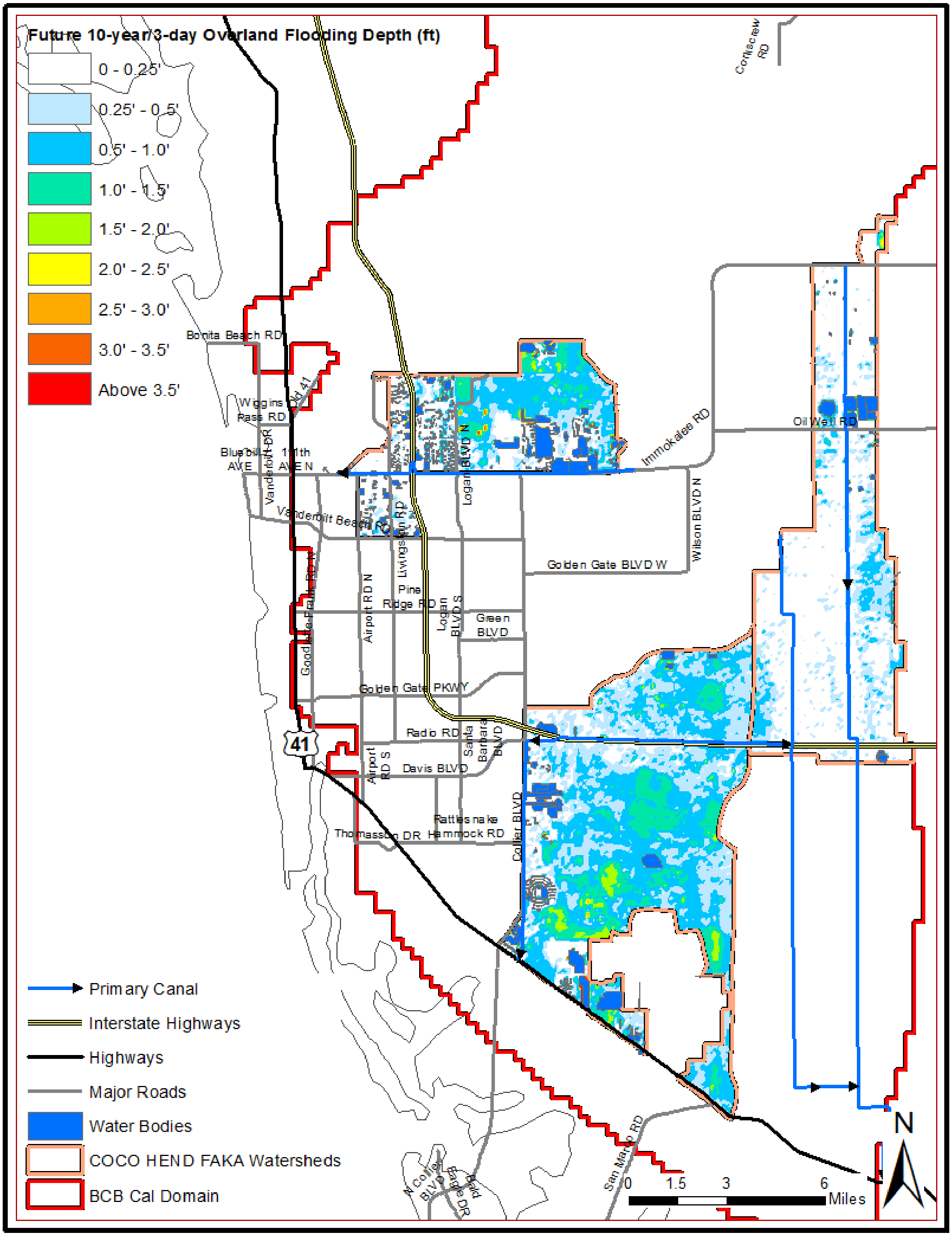

Figure 6.3.2 – Future Land Use Conditions Inundation Map for 10-year Design Storm Event

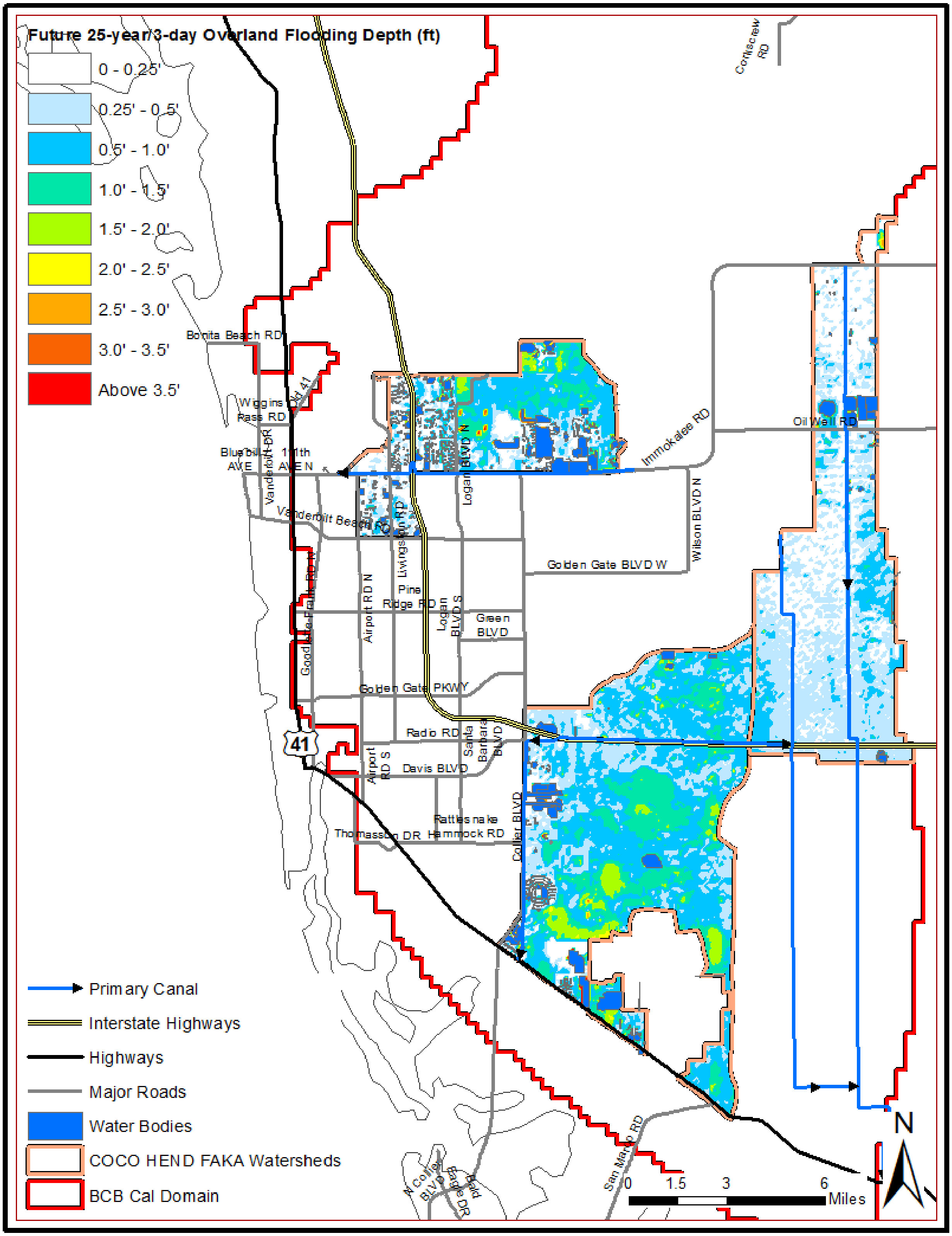

Figure 6.3.3 – Future Land Use Conditions Inundation Map for 25-year Design Storm Event

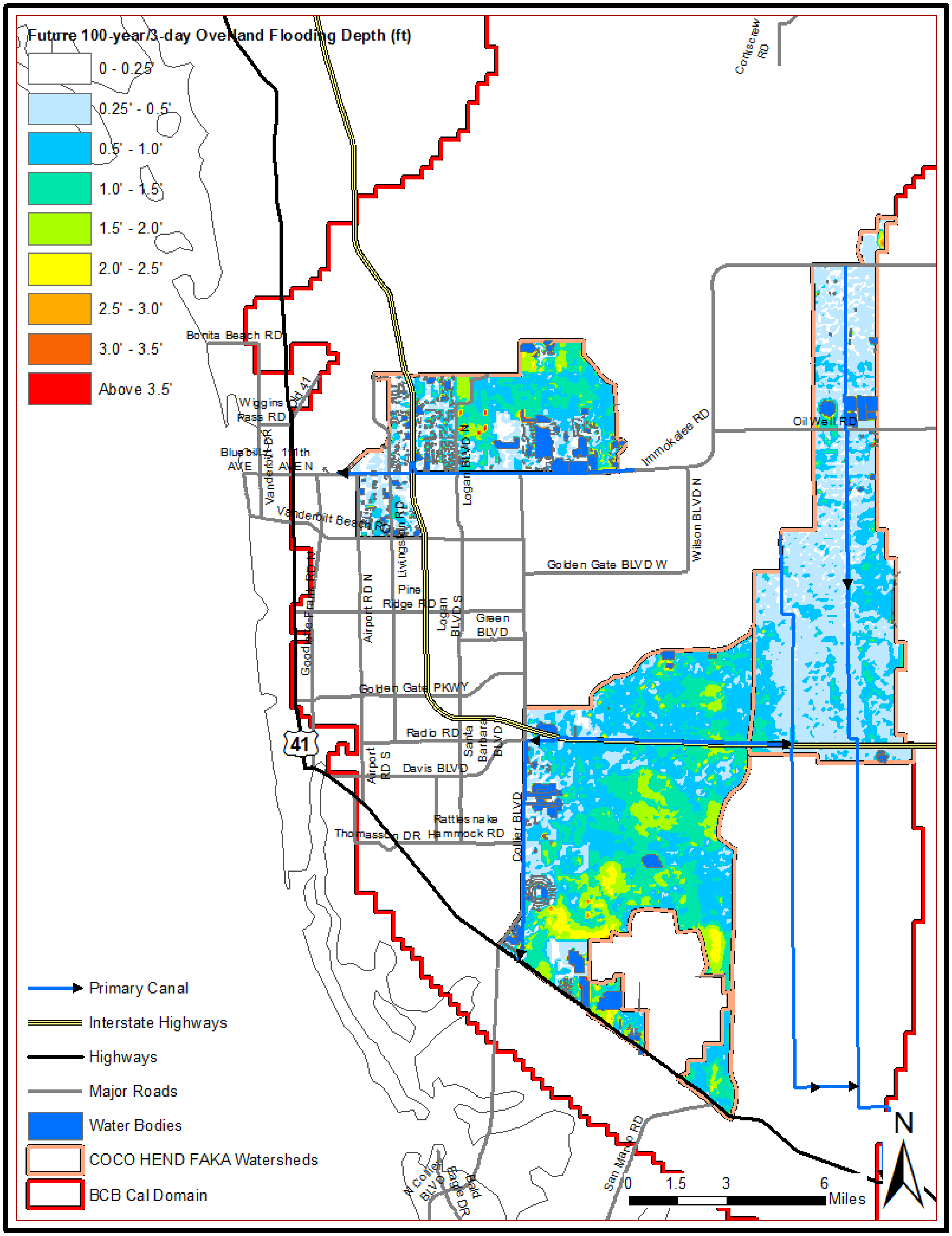

Figure 6.3.4 – Future Land Use Conditions Inundation Map for 100-year Design Storm Event

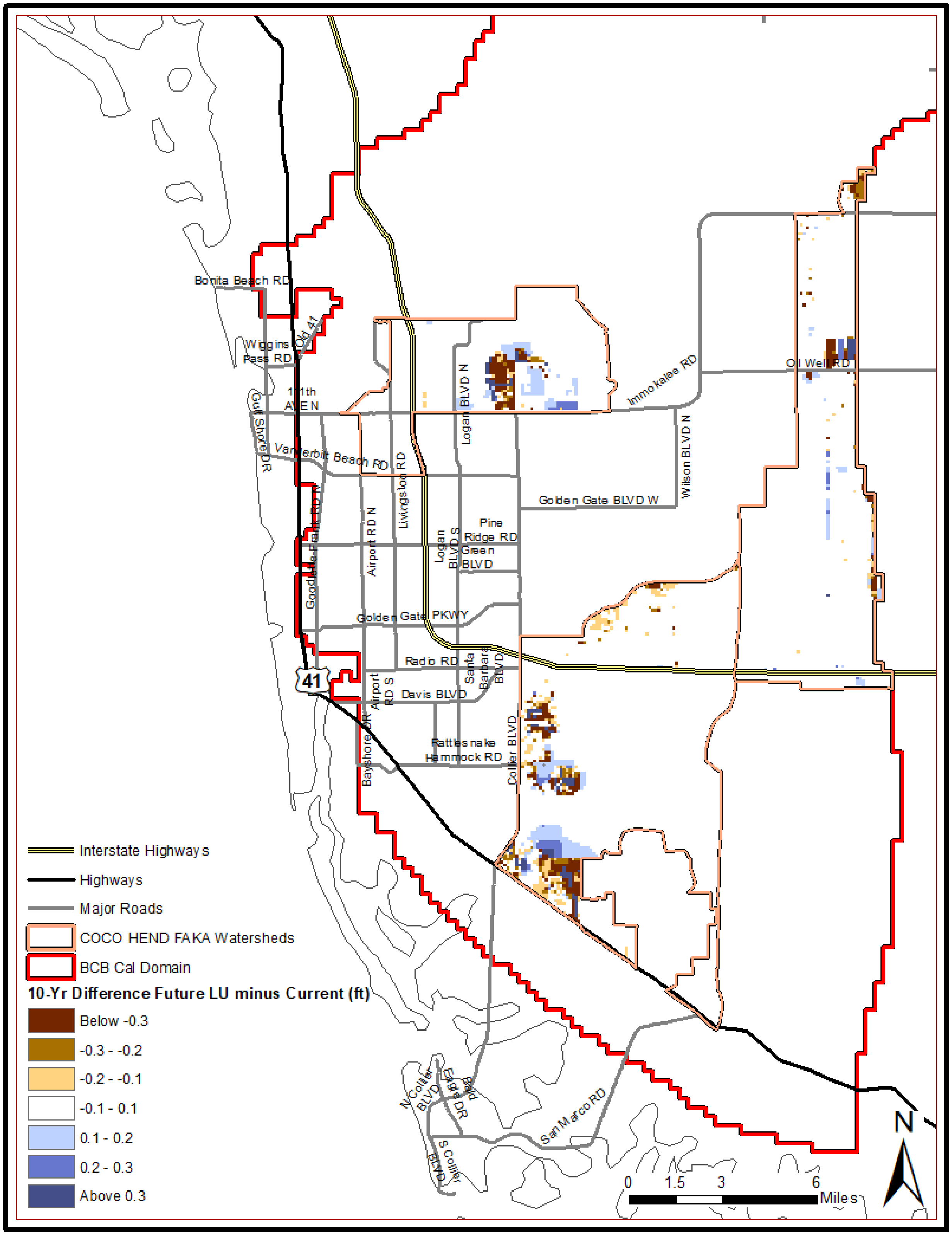

Figure 6.3.5 – 10-Year Inundation Difference Map – Future LU minus Current LU

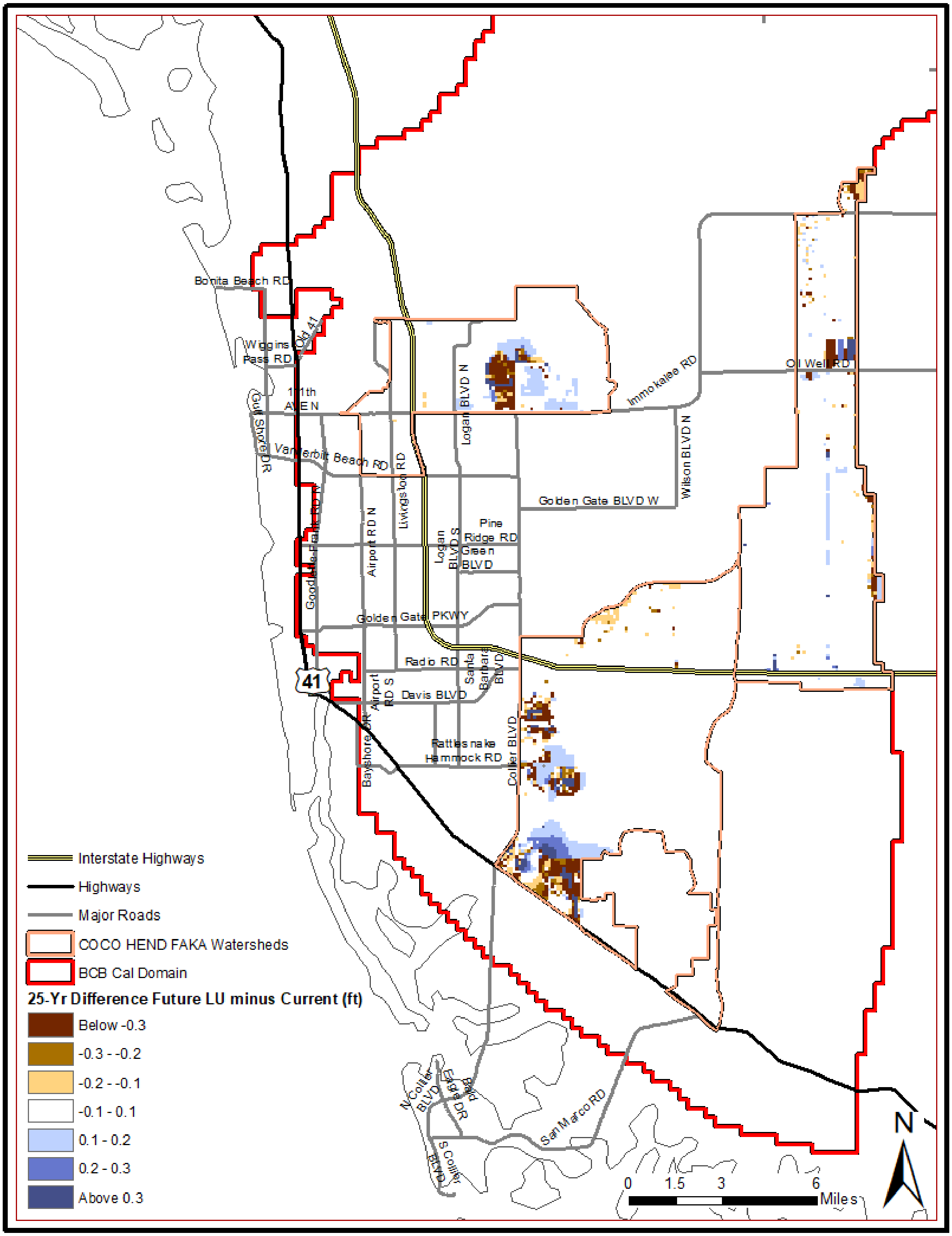

Figure 6.3.6 – 25-Year Inundation Difference Map – Future LU minus Current LU

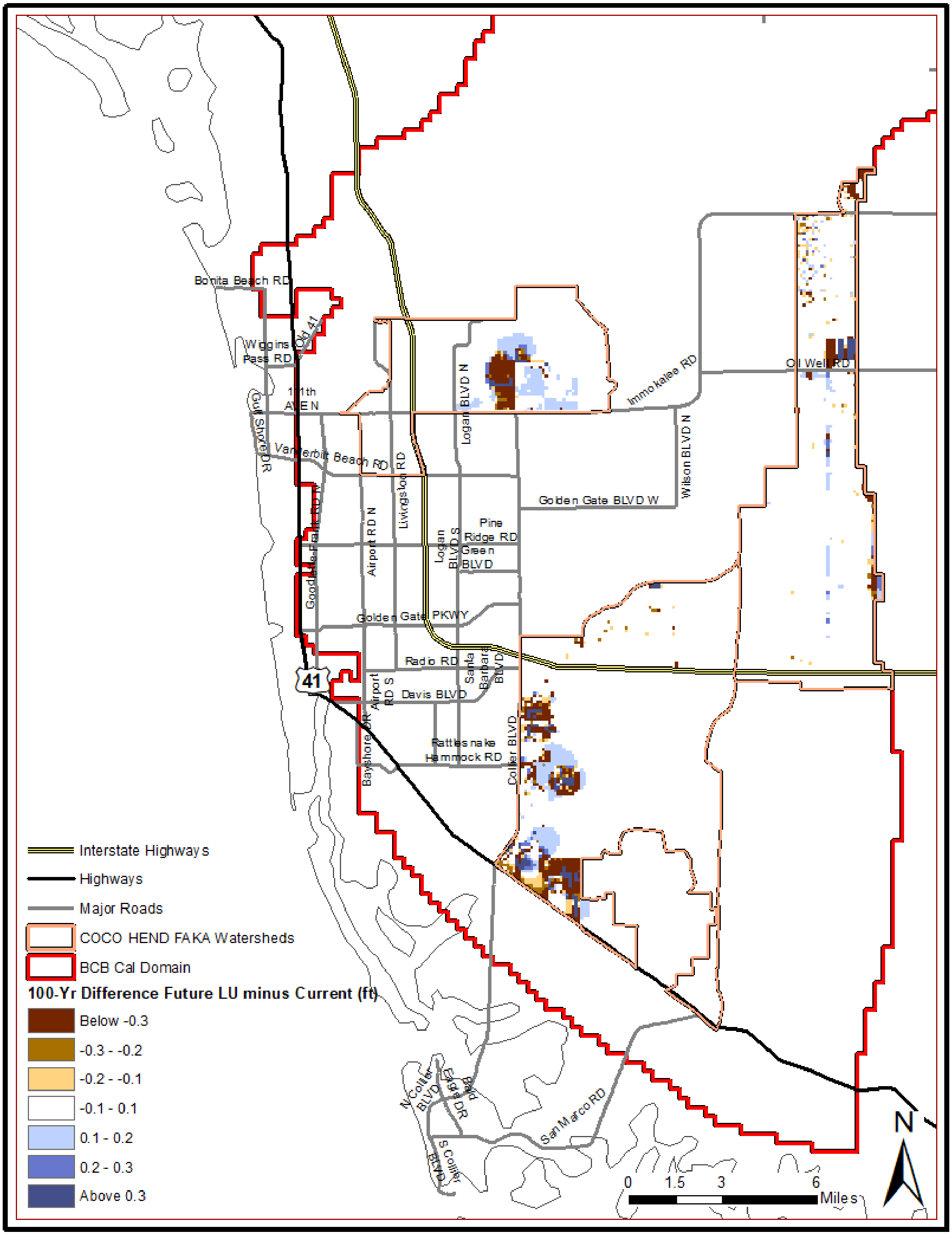

Figure 6.3.7 – 100-Year Inundation Difference Map – Future LU minus Current LU

6.4 PM #6 – Duration of Flooding, Future Land Use

PM #6 quantifies the duration of flooding at specific locations of interest within a watershed. For this Study, the length of time the flood elevation is projected to be above a threshold depth was mapped over the entire study area using the gridded model output files for the 2-D overland flow component.

Using a threshold depth of 0.25 feet, the length of time the overland flood depth was predicted to exceed the threshold within each MIKE SHE grid cell was calculated using the statistics tool in MIKE ZERO. The results of the flood duration calculations for all grid cells within all four study watersheds (including the Golden Gate Watershed) are shown on **Figures 6.4.1** through **6.4.4** for the 5-year, 10-year, 25-year, and 100-year storm events, respectively.

From **Figure 6.4.1,** it is evident that large areas are predicted to be inundated for more than 72 hours during the 31-day simulation period, even for the 5-year design storm event. These areas are comprised primarily of lakes and wetlands and other low-lying undeveloped areas. For the 10-year through 100-year storm events, greater proportions of the study watersheds are shown to be inundated for significant periods of time, with **Figure 6.4.4** showing the vast majority of the watershed to be inundated for at least a small duration.

Areas projected to experience increases in flood depth (PM#5) as a result of the future land use changes generally fall within those areas predicted to be inundated for more than 72 hours during the simulation period under current conditions. Because 72 hours defines the bottom of the highest duration interval selected for mapping, these areas to not show up as a change in the future flood duration maps.

In comparing these figures with the current conditions flood duration, areas simulated as future development show a marked decrease in the simulated duration of flooding. For example, the Esplanade development in the Cocohatchee Watershed and the Naples Reserve area in the Henderson/Belle Meade area both show extended durations of flooding in current conditions and much shorter flood durations in the future conditions. However, both areas do show significant durations of flooding during the 100-year design event.

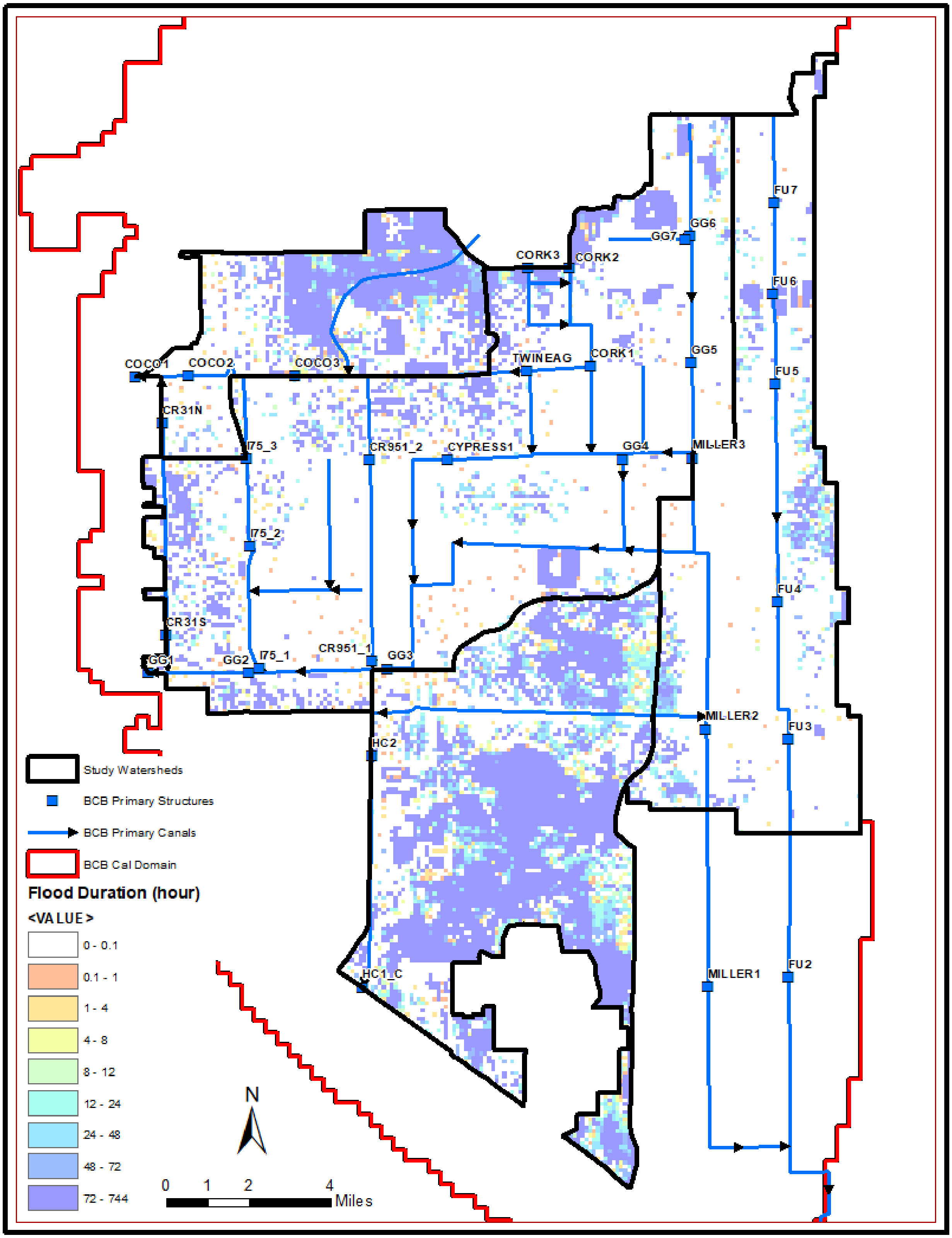

Figure 6.4.1 – Flood Duration Map for 5-year Design Storm Event, Future Conditions

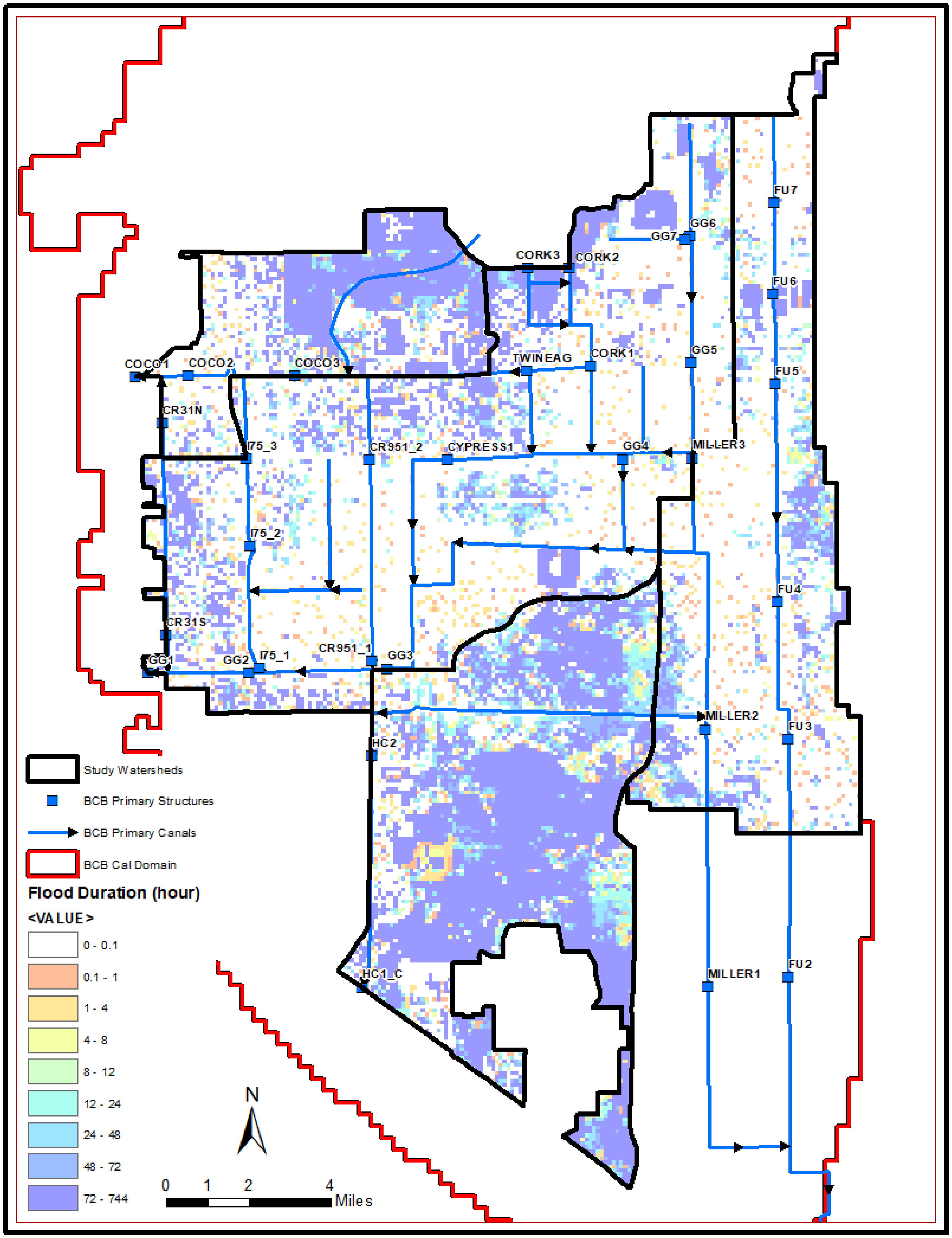

Figure 6.4.2 – Flood Duration Map for 10-year Design Storm Event, Future Conditions

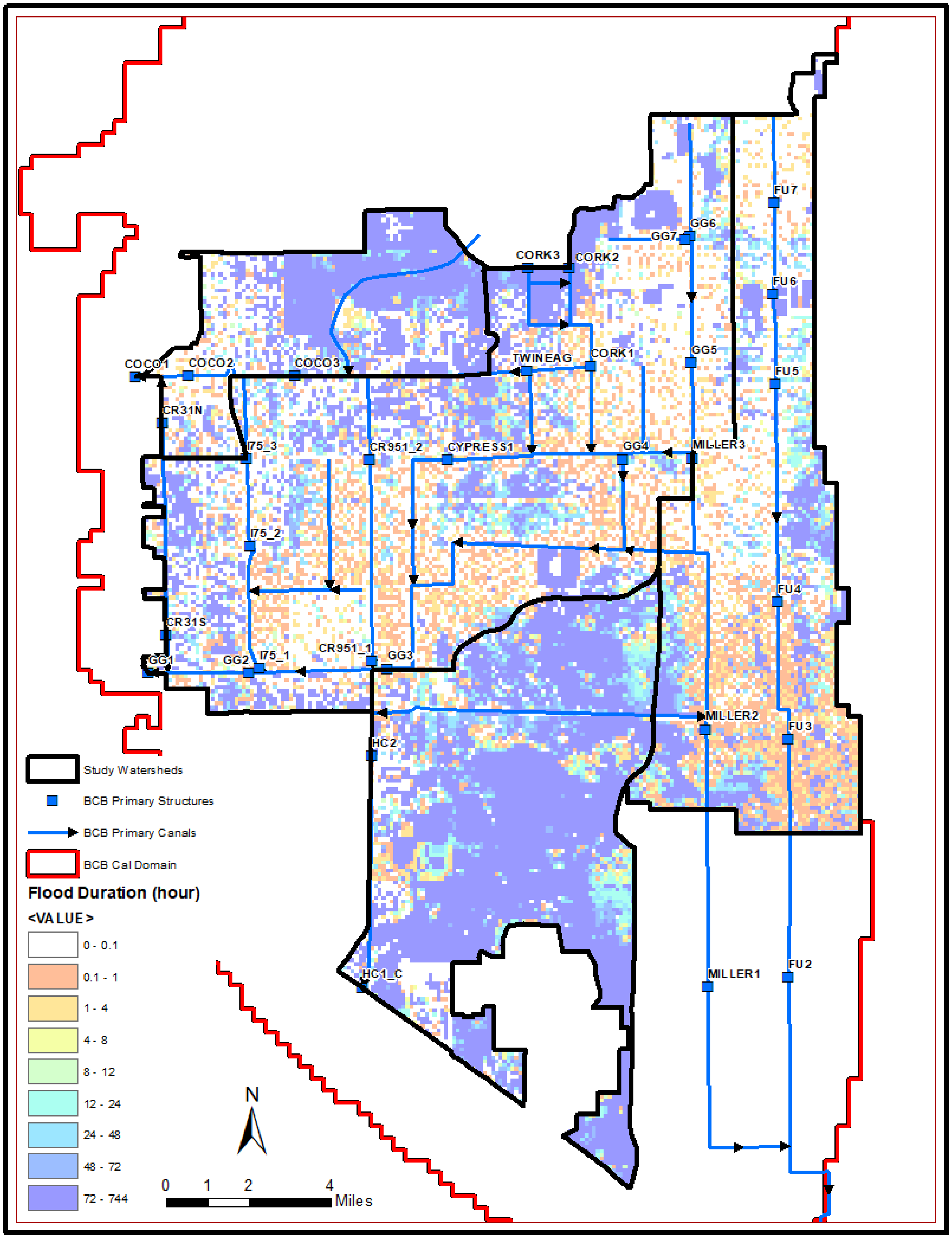

Figure 6.4.3 – Flood Duration Map for 25-year Design Storm Event, Future Conditions

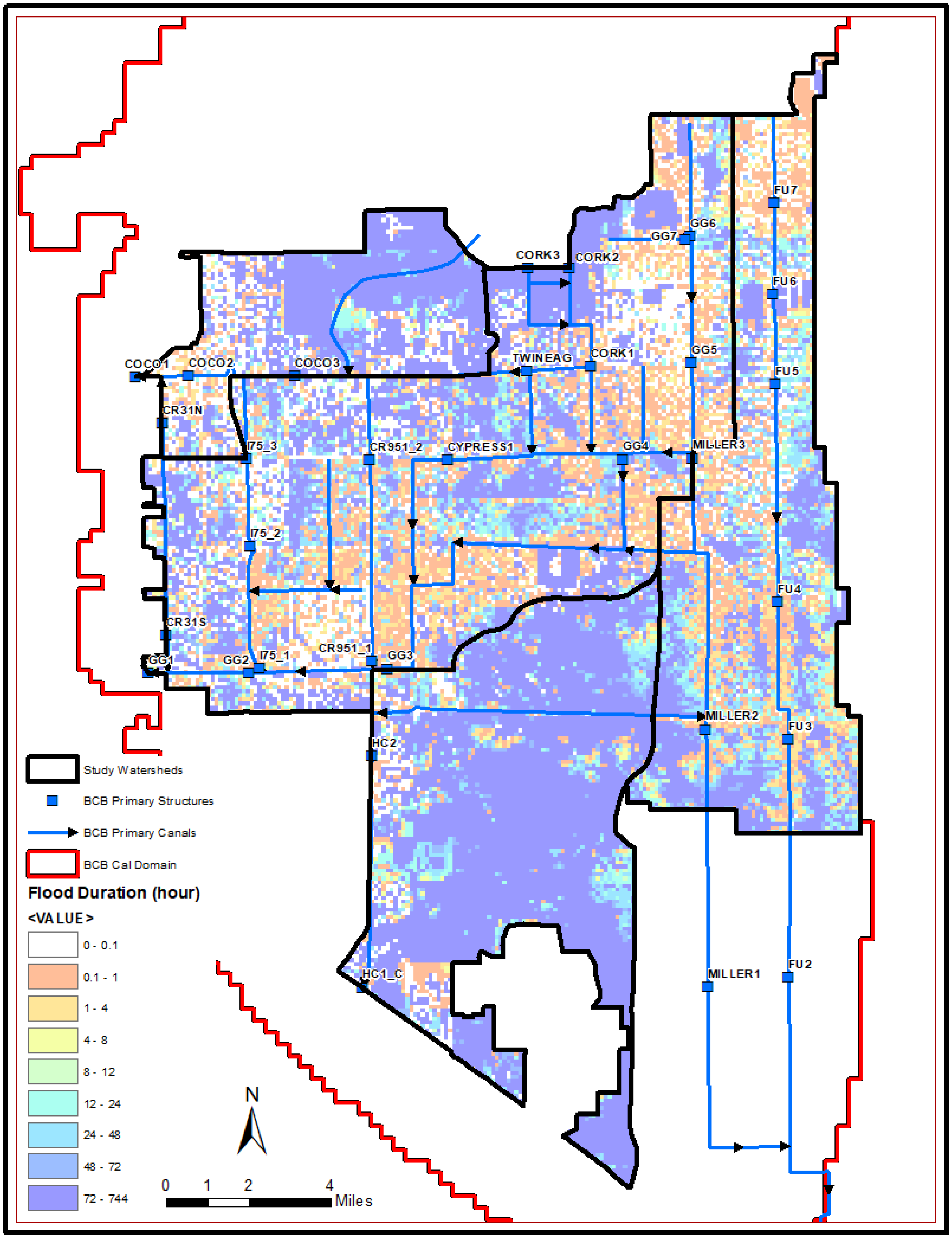

Figure 6.4.4 – Flood Duration Map for 100-year Design Storm Event, Future Conditions

7.0 Flood Protection Level of Service – Future Sea Level

Future sea level rise was simulated for the 25-year, 3-day design storm event only. Using the model developed for future land use conditions, three future sea level rise scenarios were simulated (SLR1, SLR2, and SLR3). The model setup for these scenarios was previously described in **Section 5.2**.

Appendix A provides summary model results at primary control structures, while **Appendices J** through **L** provide the complete flow and stage hydrographs for each of the three sea level rise scenarios. The performance of the existing infrastructure with future land use changes and future sea level rise was evaluated with respect to the five performance metrics described in the remainder of this section.

7.1 PM #1 – Maximum Stage in Primary Canals, Future Sea Level

Figures 7.1.1 through **7.1.4** depict the maximum 25-year stage profiles in the primary canals under future land use and future sea level rise conditions. The future land use with current sea level (SLR0) profiles are also provided on the plots for reference. As expected, the largest increases in maximum stages would be under SLR3 conditions at the downstream ends of the Cocohatchee and Henderson Creek Canals. The increases are on the order of 1.3 feet and 1.7 feet, respectively, just upstream of coastal outfall structures COCO1 and HC1. The magnitude of the increase diminishes with distance upstream. Just west of COCO2 the projected rise is about 0.5 feet, and just east of COCO2, the projected rise under SLR3 conditions is less than 0.2 feet.

In Henderson Creek, significant stage increases are associated with all sea level rise scenarios over essentially the entire 8-mile length of the canal. Upstream of HC2, significant increases (approximately 0.5 feet) are projected for the 25-year event under SLR1, SLR2, and SLR3 conditions; all associated with the gated structure becoming submerged. Under SLR0, the gated HC2 structure is projected to be just on the verge of submergence, which occurs at an elevation of 8.7 feet NAVD88. Under SLR1, SLR2, and SLR3, the tailwater increases just enough to force a transition of the flow through the open gates from weir flow to orifice flow, resulting in additional head loss through the structure. The likelihood of this occurring could be reduced by removing the channel restriction downstream of HC2. The restriction is a culvert crossing consisting of 4-42” diameter metal pipes located about 5,000 feet downstream of HC2, which should be replaced with an appropriately sized bridge.

In the Cocohatchee Canal under SLR2, the maximum increase is projected to be approximately 1.0 foot upstream of COCO1; this increase rapidly diminishes to approximately 0.25 feet just west of COCO2. Stage increases associated with the SLR1 scenario are projected to be minimal within the Cocohatchee watershed, as there is almost no difference between the SLR1 and SLR0 profiles.

Within the portions of the Faka Union and Miller Canals evaluated under this task, no increases were projected under the SLR1, SLR2, and SLR3 scenarios. This is because the profiles only include the canal sections to remain under future conditions, which are the segments upstream of the Miller and Faka Union pump stations now under construction as part of the PSRP.

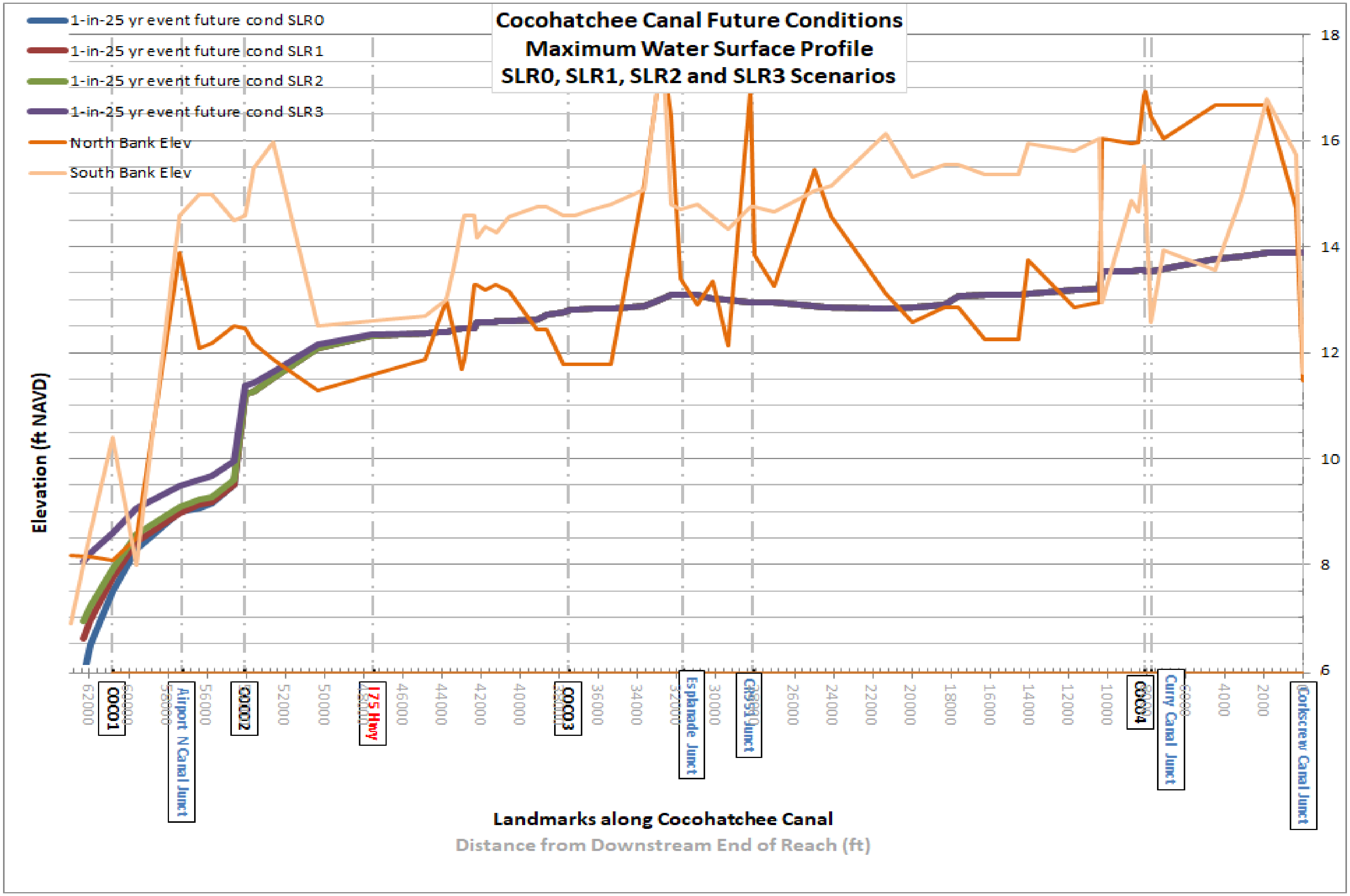

Figure 7.1.1 – Cocohatchee Main Canal Peak Stage Profiles – Current and Future Sea Level Scenarios

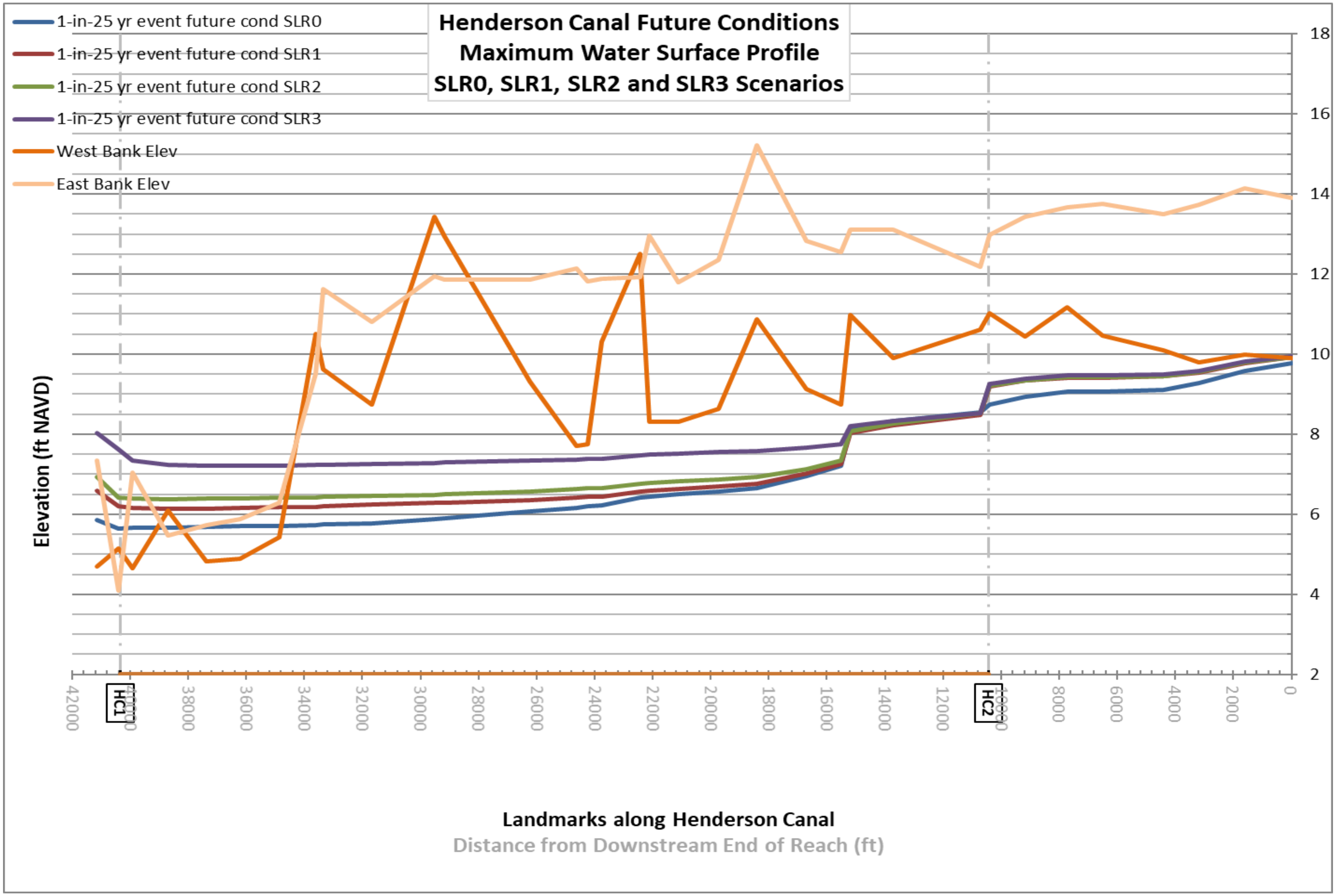

Figure 7.1.2 – Henderson Creek Canal Peak Stage Profiles – Current and Future Sea Level Scenarios

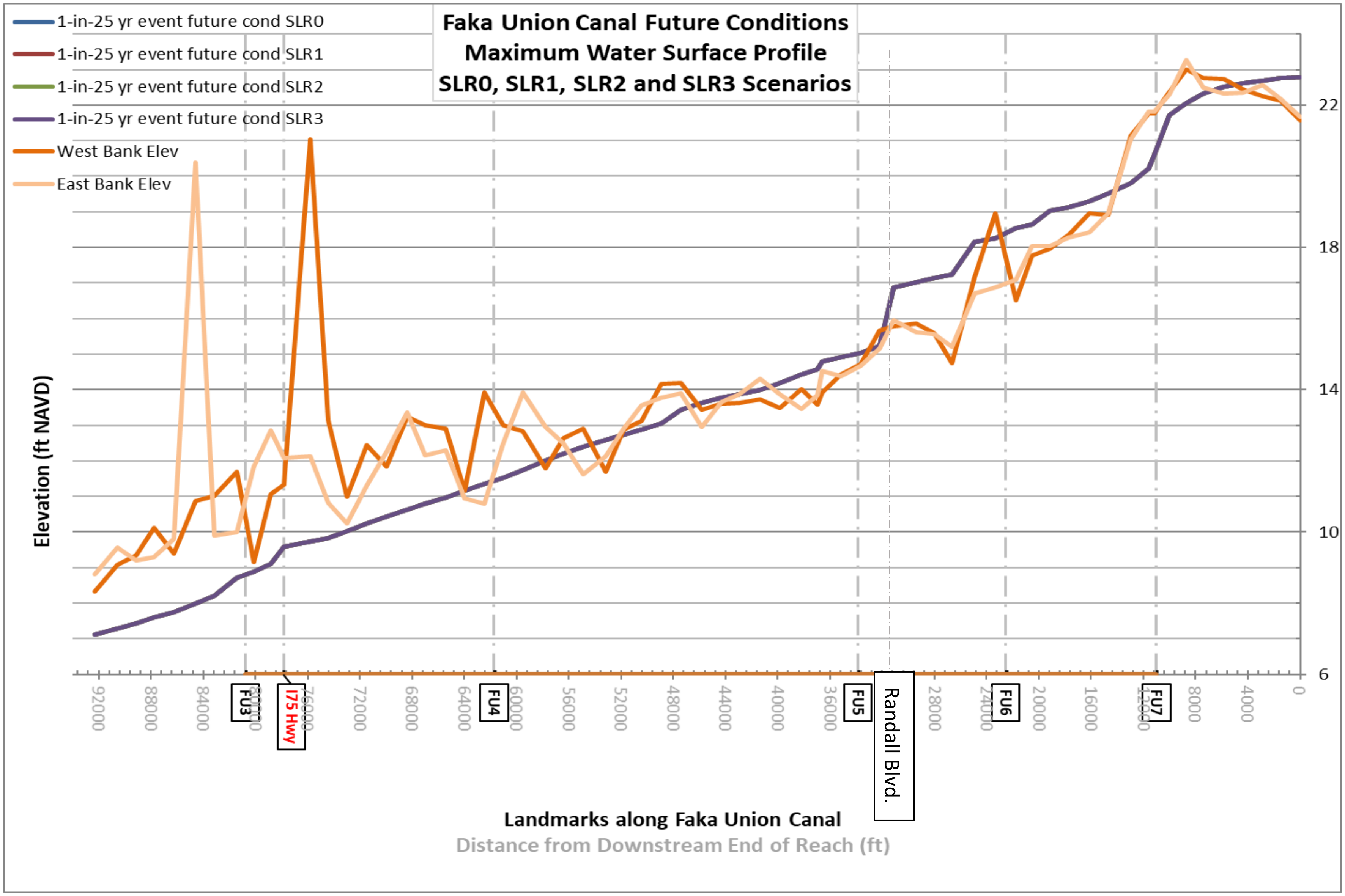

Figure 7.1.3 – Faka Union Canal Peak Stage Profiles – Current and Future Sea Level Scenarios

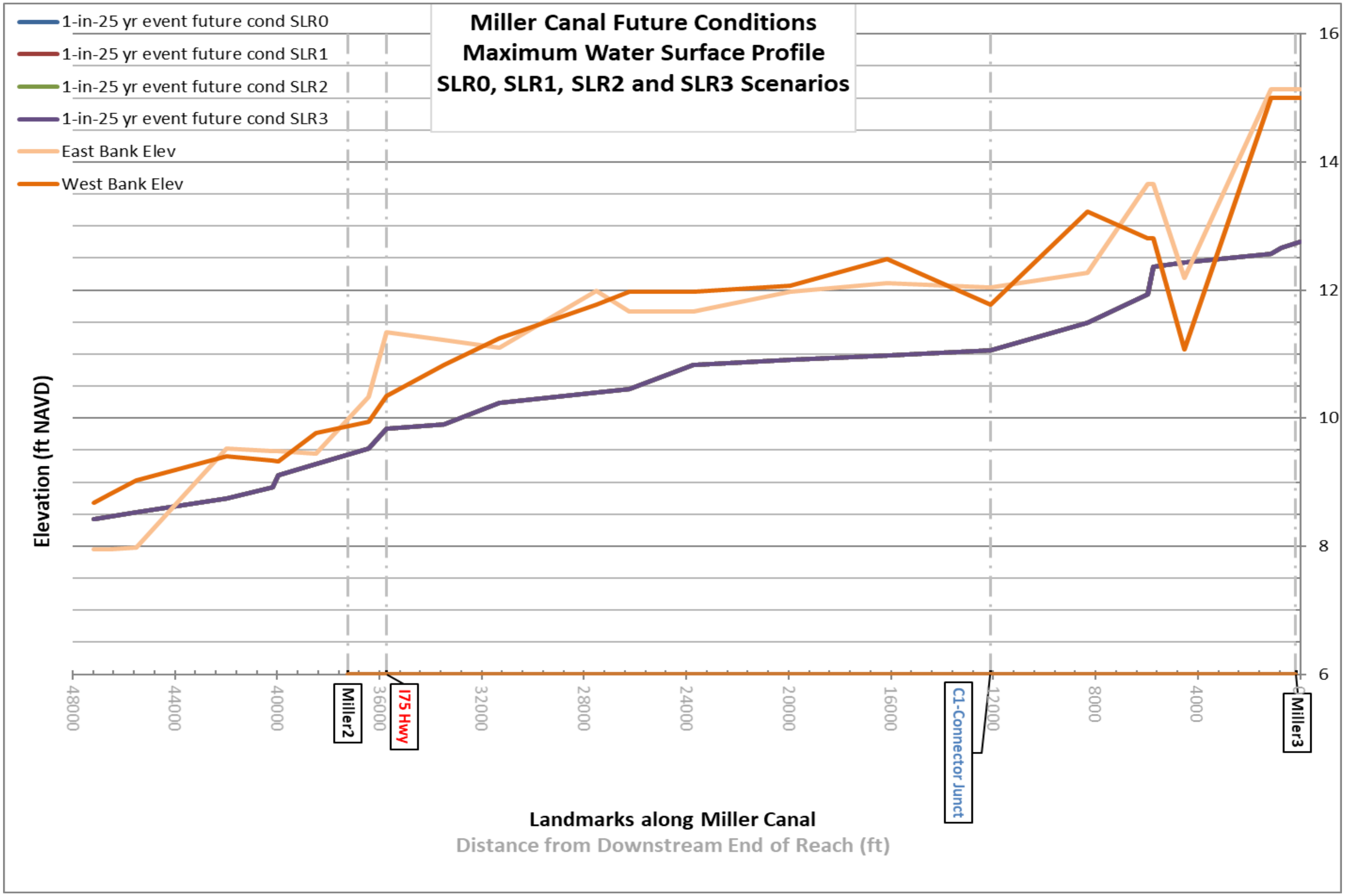

Figure 7.1.4 – Miller Canal Peak Stage Profiles – Current and Future Sea Level Scenarios

7.2 PM #2 – Maximum Daily Discharge Capacity through the Primary Canals, Future Sea Level

Maximum discharge capacity through the primary canal segments for the 25-year design storm event was calculated for each of the three future sea level scenarios (SLR1, SLR2, and SLR3) and compared to the future land use conditions model results for SLR0 (refer to the "Future" column in **Table 6.2.1**).

The discharge capacities for the inland canal segments evaluated were essentially unaffected by the sea level rise scenarios. However, the downstream-most segments of the Cocohatchee and Henderson Creek Canals would be significantly affected under the SLR3 scenario. The most significant changes in canal discharge capacities would occur in the following locations and conditions:

- In canal segment COCO1 under SLR3, the discharge capacity would be reduced to 90.5 cfs per square mile from 98.0 cfs per square mile for the SLR0 scenario (a 7.6% decrease).
- In canal segment HC1 under SLR3, the discharge capacity is projected to *increase* to 95.5 cfs per square mile from 82.0 cfs per square mile for the SLR0 scenario (a 16% increase). However, this "increase" in discharge capacity is an artifact of the timing of the tidal boundary condition as illustrated in **Figure 7.2.1**. The delayed timing of the peak flow from rainfall-generated runoff allows the SLR3 storm surge peak to rush into the empty canal. This surge volume combines with the main flood wave from the upstream watershed to result in a higher peak outflow.

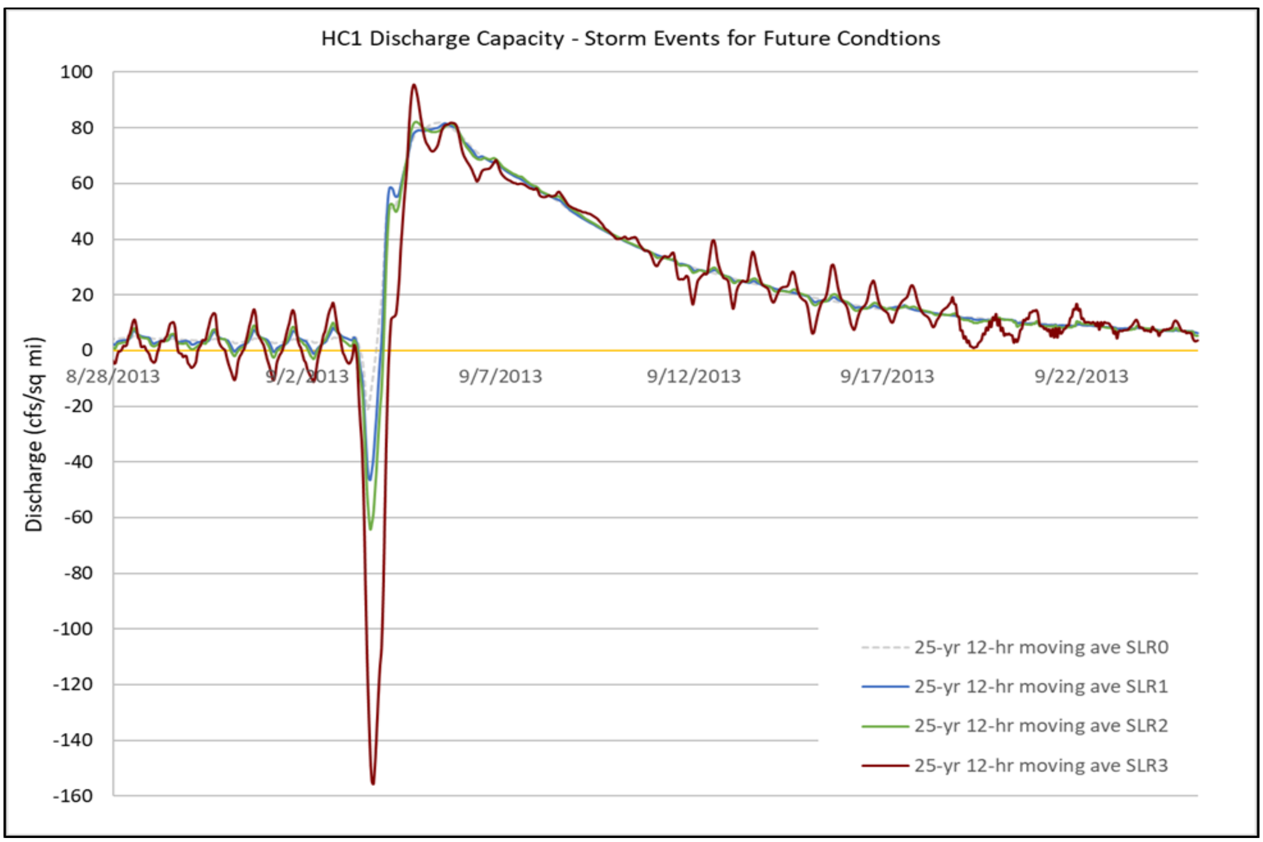

Figure 7.2.1 – Discharge Capacity at HC1 for SLR1, SLR2, and SLR3

7.3 PM #4 – Peak Storm Runoff – Effects of Sea Level Rise

PM #4 is the maximum conveyance capacity of a watershed at the tidal structure for a range of design storms. It shows the maximum conveyance (moving 12-hour average) for a specific design storm and a specific tidal boundary condition. This metric examines the behavior of the system under severe stress and can be used to check if conditions exceed design limits. In evaluating this PM, it is assumed that design rainfall and design storm surge occur simultaneously.

Figure 7.3.1 provides a graph of the flow vs. return period for current sea level conditions (SLR0) at COCO1. The 25-year results for SLR1, SLR2, and SLR3 are also shown on the graph. The higher tailwater elevations associated with SLR1 and SLR2 would result in negligible changes in discharge capacity, while SLR3 would result in a reduction in discharge capacity of approximately 26 cfs, about a 2.3% reduction.

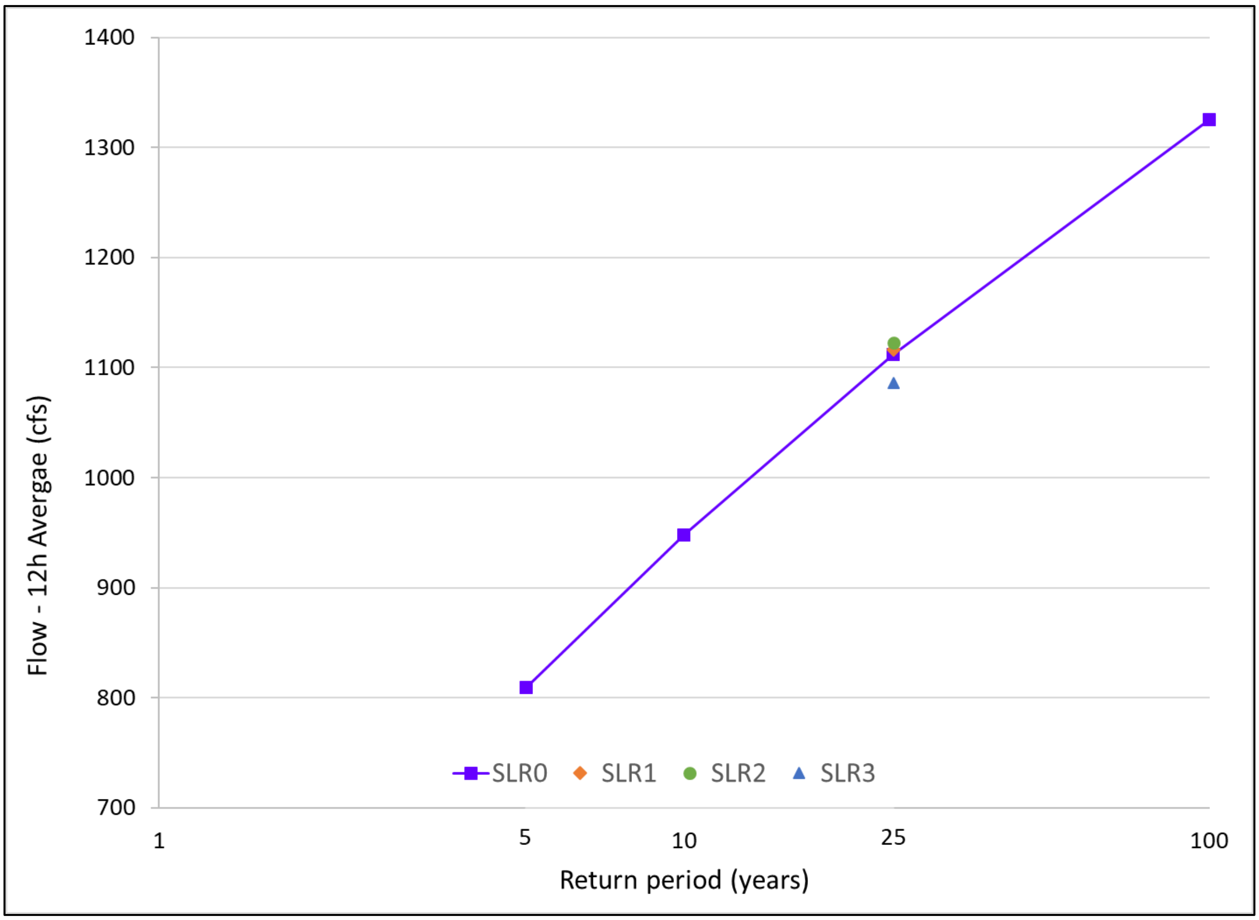

Figure 7.3.1 – Flow at Tidal Structure COCO1

Due to the low topography and relatively flat water surface profile of the Henderson Creek Canal, the HC1 structure is particularly susceptible to backwater effects associated with storm surge and sea level rise as illustrated in the PM#1 results. However, this susceptibility does not manifest itself in the PM#4 results. **Figure 7.3.2** provides a graph of the flow vs. return period for current sea level conditions (SLR0) at HC1. The 25-year results for SLR1, SLR2, and SLR3 are also shown on the graph. While the SLR1 and SLR2 flows are similar to the SLR0 flows, there is an apparent increase in the peak discharge for the SLR3 scenario. is "increase" in discharge capacity is an artifact of the timing of the tidal boundary condition as illustrated in **Figure 7.2.1**. The delayed timing of the peak flow from rainfall-generated runoff allows the SLR3 storm surge to rush into the empty canal. This surge volume combines with the main flood wave from the upstream watershed to result in a higher peak outflow, compared to SLR0, SLR1, and SLR2.

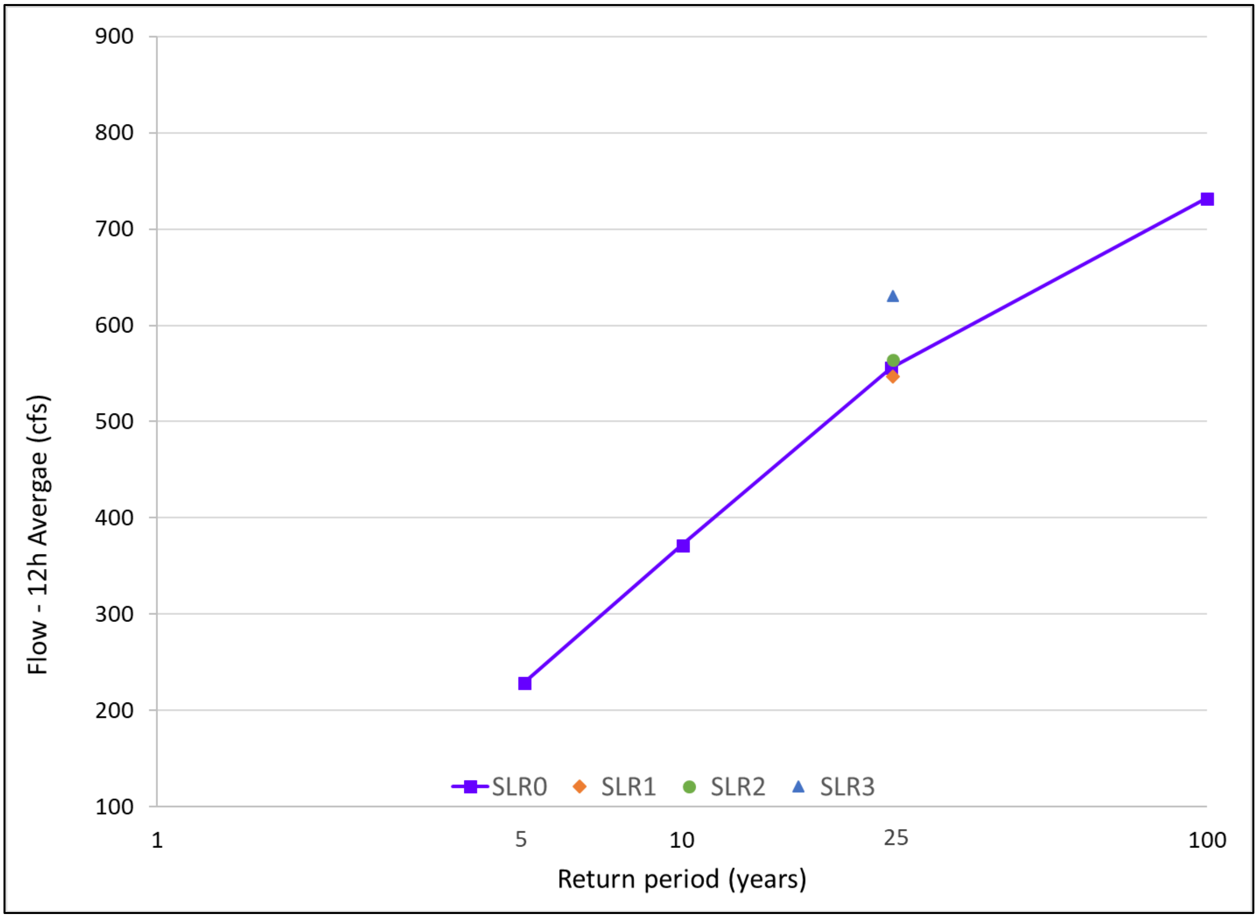

Figure 7.3.2 – Flow at Tidal Structure HC1

7.4 PM #5 – Frequency of Flooding, Future Sea Level

The three future sea level rise scenarios evaluated in this modeling effort would result in little or no change to the frequency or depths of flooding throughout the majority of the Cocohatchee and Faka Union Watersheds for the 25-year design storm event. Part of the reason for this is that the boundaries of the watersheds are two or more miles inland from the coast, where the largest impacts are likely to occur. The inundation maps prepared for SLR1, SRL2, and SLR3 are therefore very similar to the one prepared for SLR0 (refer to **Figure 6.3.3**, Future Land Use Conditions Inundation Map for 25-year Design Storm Event).

Within the Henderson Creek / Belle Meade Watershed, some significant increases in the 25-year inundation depths would occur, particularly under the SLR3 scenario (**Figure 7.4.1**). The largest increases (more than 1.5 feet) within the Henderson Creek / Belle Meade Watershed would occur adjacent to the downstream portion of the Henderson Creek Canal, between HC1 and HC2. Under the SLR2 scenario, the differences in inundation depths for the 25-year event would still be significant (up to 0.8 feet) but would be confined to a smaller region immediately upstream of HC1 (refer to **Figure 7.4.2**). Under the SLR1 scenario, the differences in inundation depths for the 25-year event would be negligible.

Another way to measure inundation impacts due to sea level rise is to consider maximum water table stages. Higher water table stages can affect infrastructure such as septic tanks, older, unlined sanitary sewers (through higher infiltration), and roadway base material. Pronounced differences in maximum water table stages are evident in coastal areas west and south of the Cocohatchee and Henderson Creek / Belle Meade Watersheds. **Figures 7.4.3** through **7.4.5** show differences in peak water table elevations throughout the model domain for sea level rise scenarios SLR1, SLR2, and SLR3 respectively, relative to SLR0.

Within the Henderson Creek / Belle Meade Watershed, the largest and most widespread increases in maximum water table stage would occur under the SLR3 scenario, in an area north of U.S. 41.

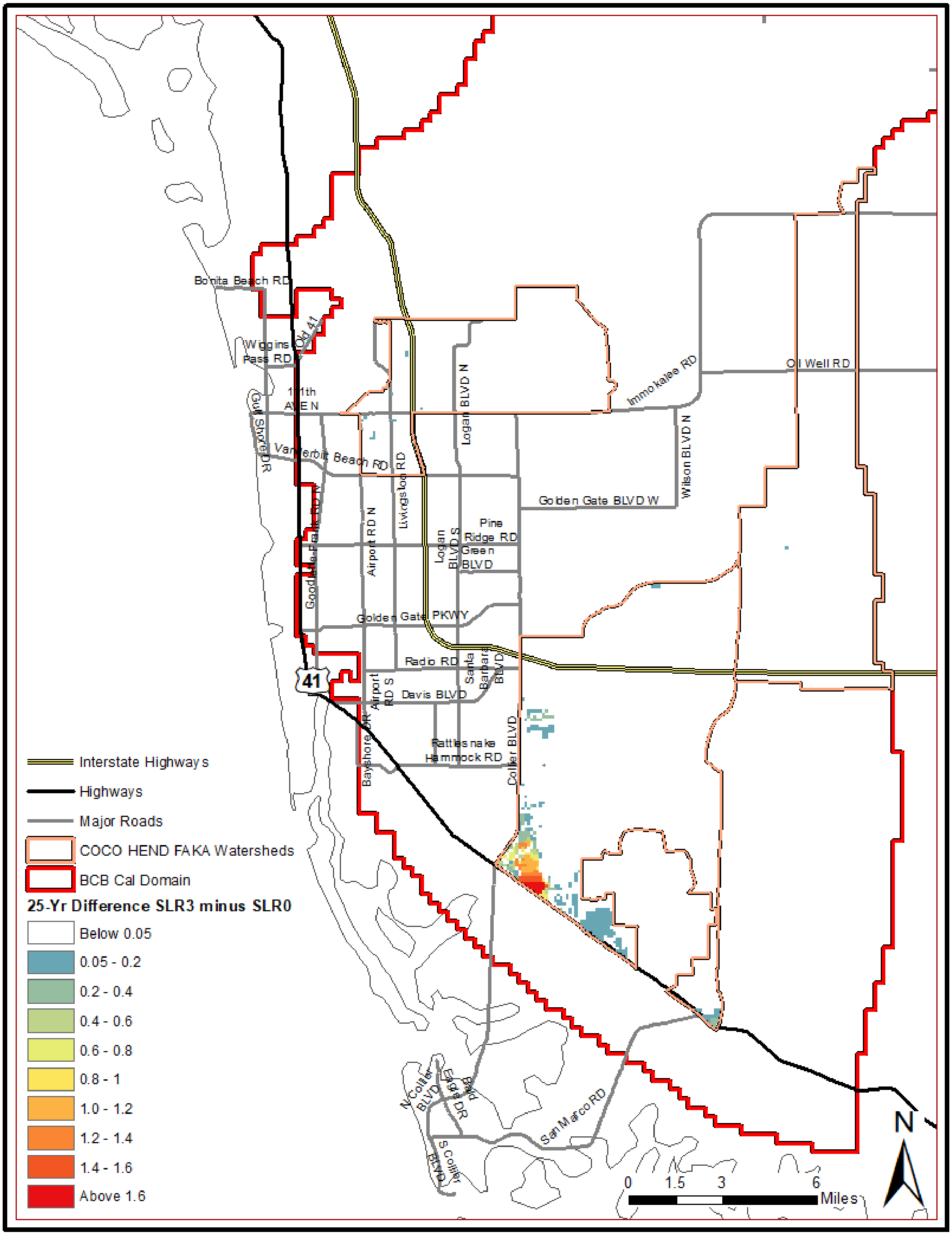

Figure 7.4.1 – Cocohatchee, Henderson/Belle Meade, and Faka Union 25-Year Inundation Difference Map – Future SLR3 minus Current Sea Level (SLR0)

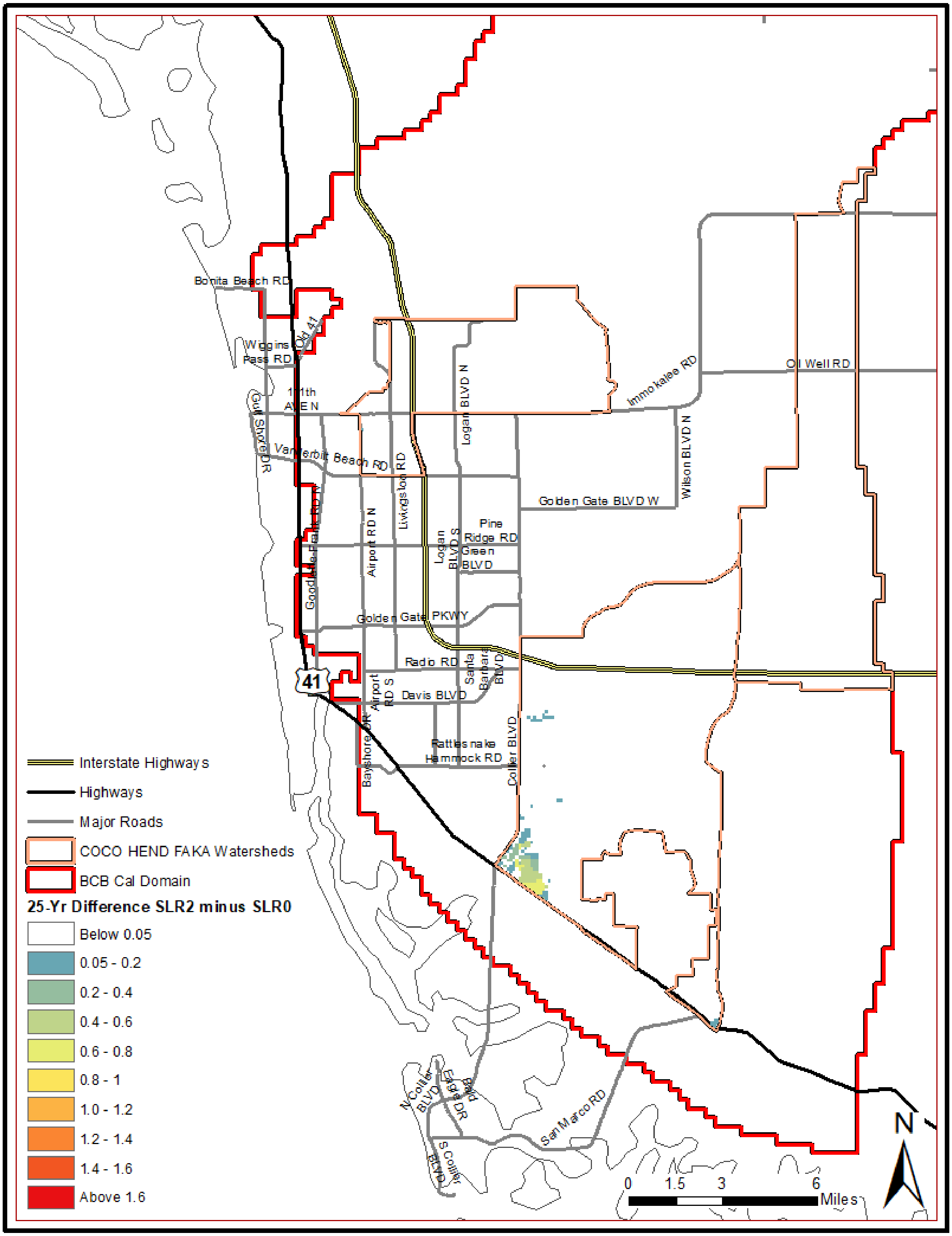

Figure 7.4.2 – Cocohatchee, Henderson/Belle Meade, and Faka Union 25-Year Inundation Difference Map – Future SLR2 minus Current Sea Level (SLR0)

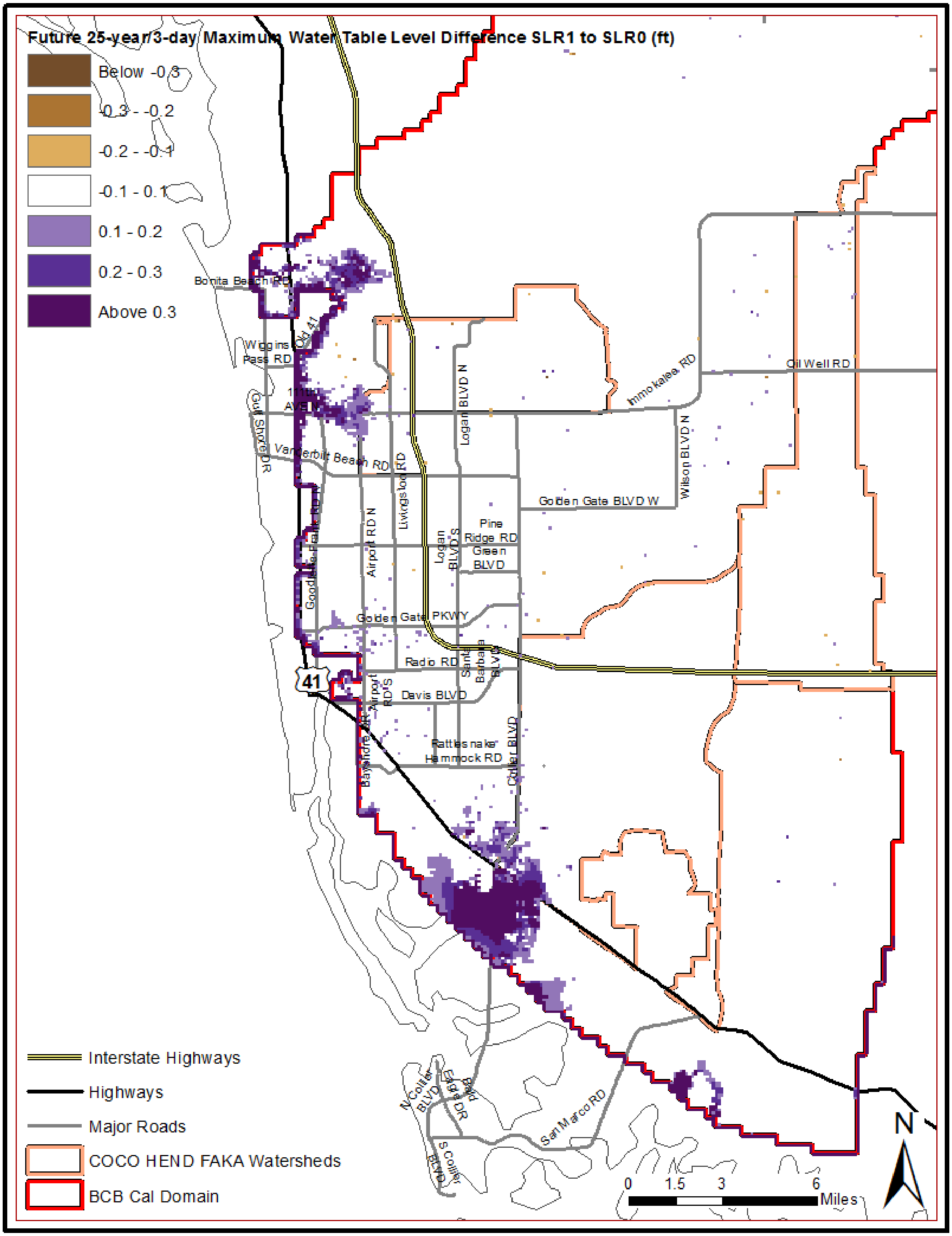

Figure 7.4.3 – Difference Map of Maximum 25-Year Water Table Elevation, SLR1 vs. SLR0

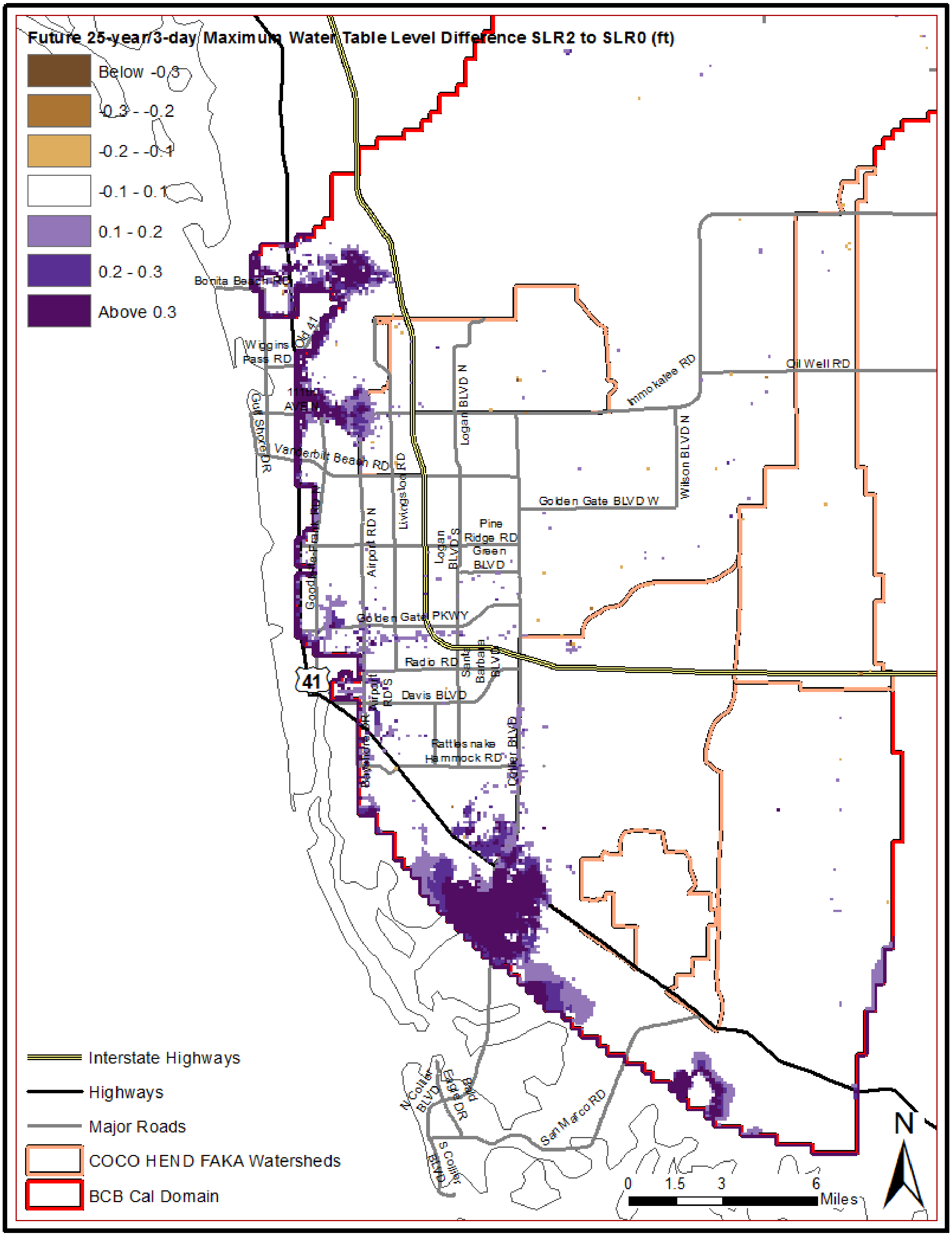

Figure 7.4.4 – Difference Map of Maximum 25-Year Water Table Elevation, SLR2 vs. SLR0

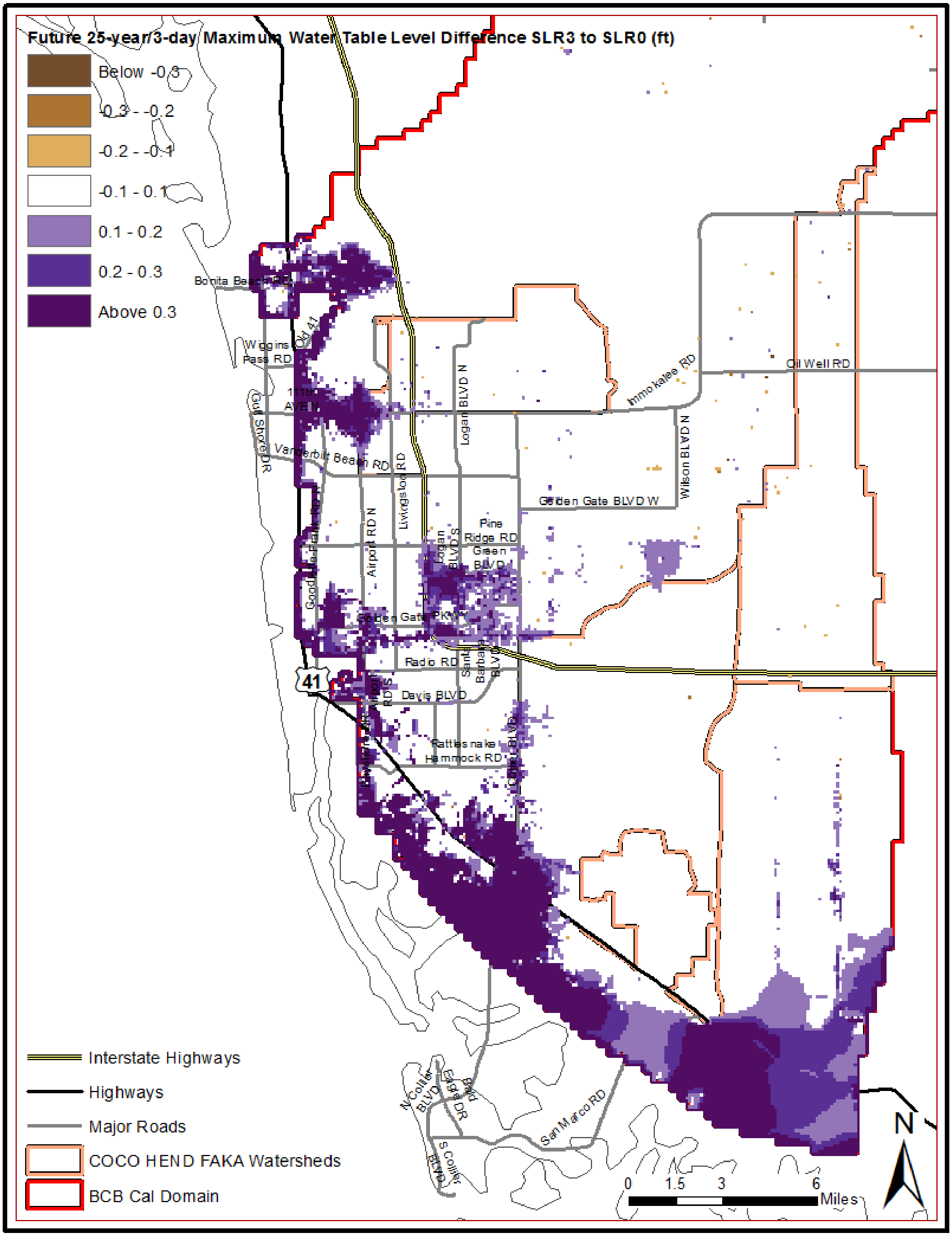

Figure 7.4.5 – Difference Map of Maximum 25-Year Water Table Elevation, SLR3 vs. SLR0

7.5 PM #6 – Duration of Flooding, Future Sea Level

PM #6 quantifies the duration of flooding at specific locations of interest within a watershed. For this Study, the length of time the flood elevation is projected to be above a threshold depth was mapped over the entire study area using the gridded model output files for the 2-D overland flow component.

Using a threshold depth of 0.25 feet, the length of time the overland flood depth was predicted to exceed the threshold within each MIKE SHE grid cell was calculated using the statistics tool in MIKE ZERO. The results of the flood duration calculations for all grid cells within all four study watersheds (including the Golden Gate Watershed) are shown on **Figures 7.5.1** through **7.5.3** for the 25-year SLR1, SLR2, and SLR3 scenarios, respectively.

From the figures, it is evident there is little or no difference in the duration of flooding predicted amongst the three scenarios. And in comparing these figures with the comparable figure depicting flood duration for the future conditions SLR0 scenario (**Figure 6.4.3**), no appreciable differences are visible. The explanation for this is that under SLR0 conditions, the areas most susceptible to sea level rise (e.g., east of the Henderson Creek Canal upstream of HC1) are already predicted in SLR0 to flood for durations in excess of 3 days, which is within the maximum duration interval selected for mapping.

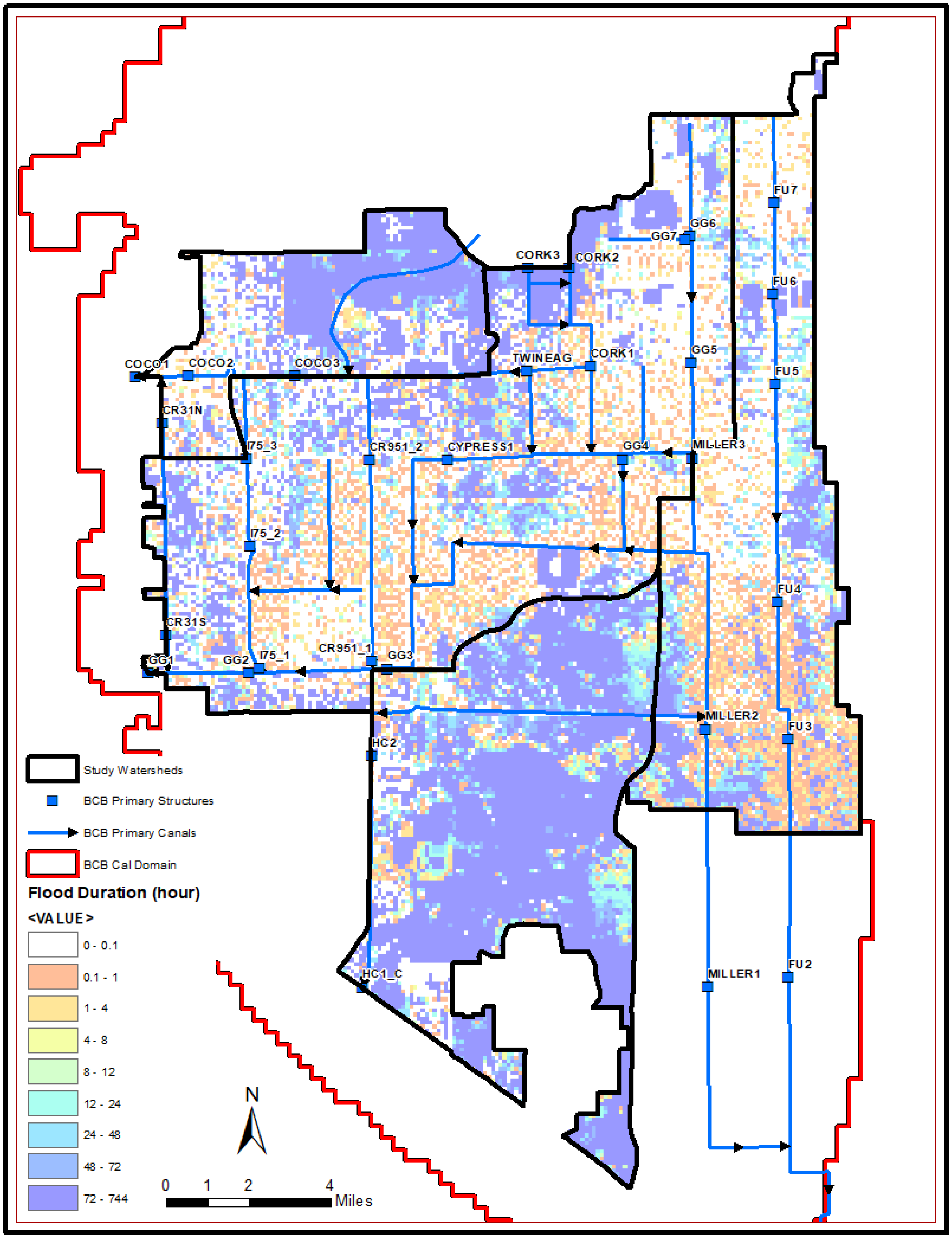

Figure 7.5.1 – Flood Duration Map for 25-year Design Storm Event, Future Condition SLR1

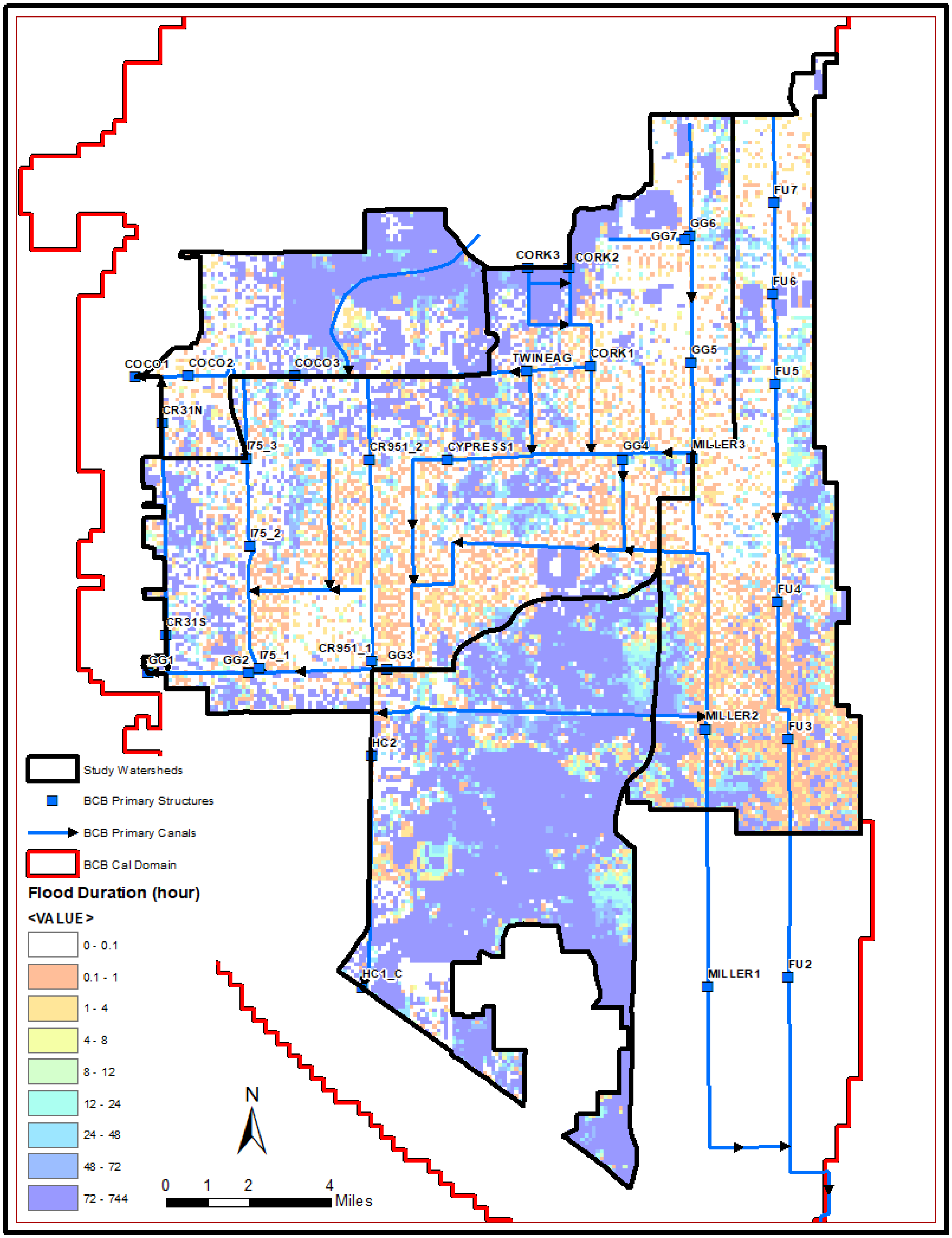

Figure 7.5.2 – Flood Duration Map for 25-year Design Storm Event, Future Condition SLR2

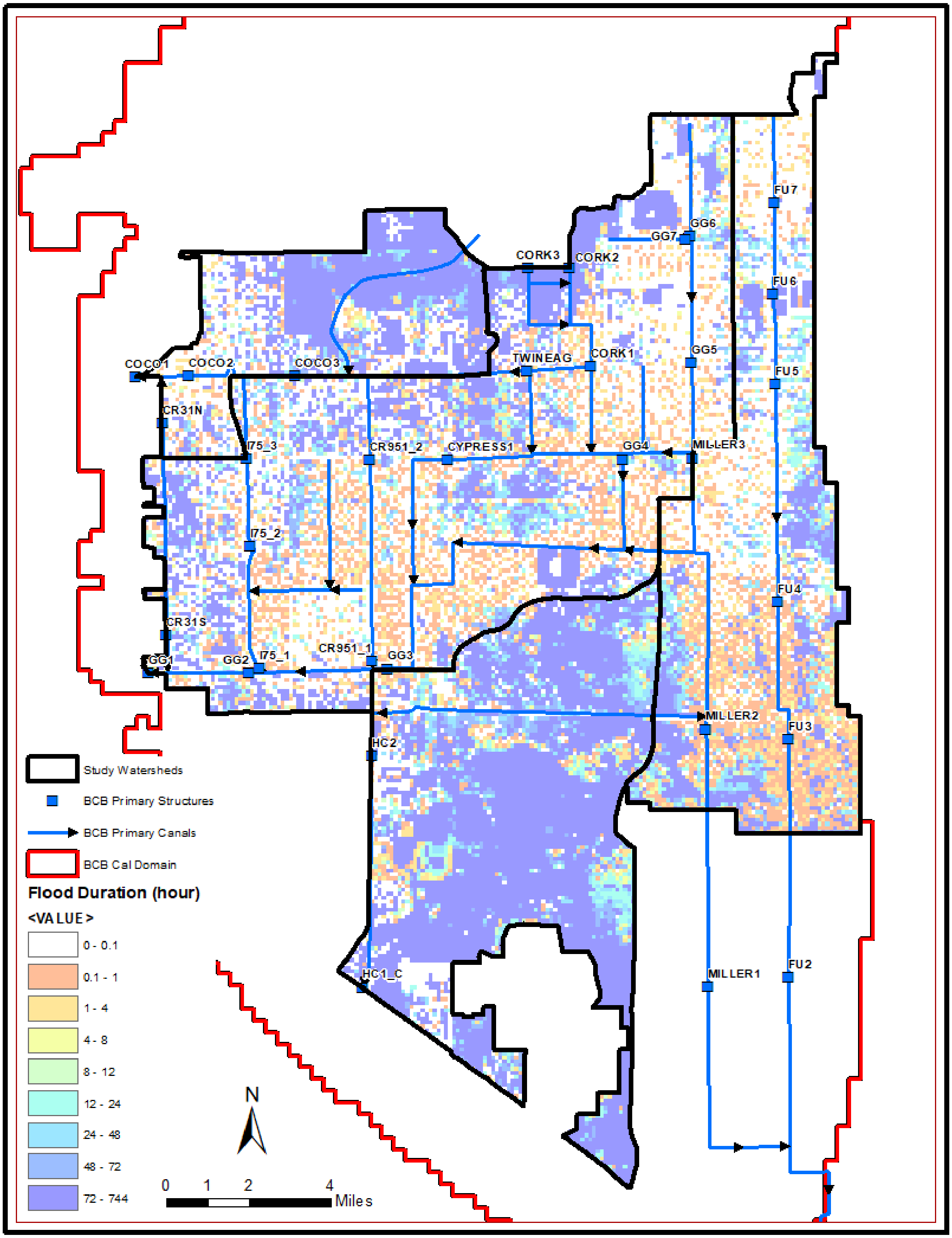

Figure 7.5.3– Flood Duration Map for 25-year Design Storm Event, Future Condition SLR3

8.0 Conclusions

The current and future conditions design storm simulation results were evaluated with respect to four performance measures, which together provide insight into the level of flood protection provided by the current Cocohatchee, Henderson/Belle Meade and Faka Union watershed primary canal network and associated control structures under current stressors (rainfall, land use, and sea level). Potential FPLOS deficiencies were identified as those areas that failed multiple performance measures, for example bank exceedances that corresponded to overland inundation (PM #5 and/or PM #6). In some cases, PM #1 bank exceedances did not manifest as overland inundation, and were thus considered insignificant localized FPLOS deficiencies with inundation extents too small to simulate and map within the MIKE SHE 500' topographic/computational grid.

The future conditions FPLOS was compared with the current conditions FPLOS to identify where significant degradations in FPLOS would likely occur due to potential changes in land use and sea level rise. It is important to understand that the three sea level scenarios evaluated in this modeling effort are only projections. Future changes in sea level are difficult to predict, and actual changes by year 2065 could be outside the range of the three scenarios chosen for this study.

It should also be noted that the model results are subject to certain limitations associated with the scale of the 2-dimensional model grid. Because the grid cells are 500 feet x 500 feet, the model results are suitable for the subregional-scale flood protection LOS evaluation presented herein; however, they should not be extended to local-scale evaluations or regulatory determinations of flooding extents, where considerable variations in topography can occur within the area of each grid cell.

8.1 Current Conditions

8.1.1 Cocohatchee

Based on the results presented herein, it appears that the Cocohatchee primary canal network generally provides a 10-year level of service, with some areas receiving a 25-year level of service or better. The lone exception to the 10-Year FPLOS is near structure COCO3, upstream of Logan Blvd, where the PM #1 profile shows a north bank exceedance for the 10-year event. This bank exceedance corresponds to a significant area of flood inundation on the PM #5 map for the 10-year event.

For the 25-yr and 100-year events, the model results suggest that much of the Cocohatchee Canal would be overwhelmed as most of the watershed would be inundated to some degree during peak flood conditions. Within the developed areas, the inundation appears to be mostly confined to the numerous golf courses, although some roads would flood as well. This is evidenced by the PM #6 results, which show local road flooding north of the intersection of Immokalee Road and Collier Blvd for both the 25-year and 100-year events.[1]

[1] District staff has indicated that the capacity of the tidal conveyance system of the Cocohatchee may be a constraint under high flow conditions.

8.1.2 Henderson/Belle Meade

The Henderson Creek/Belle Meade Watershed is somewhat unique in that much of the watershed is not served by the primary Henderson Creek Canal and its outfall structure HC-1. As discussed in Section 4.2, most of the watershed drains to other structures and culverts along US 41 to the east of HC-1. In addition, the majority of the Henderson Creek/Belle Mead Watershed is undeveloped and contains large wetlands that are seasonally inundated. Therefore, the flood inundation maps (PM #5) show most of the watershed to be inundated for all design storm events.

When the focus is narrowed to the area directly served by the Henderson Creek Canal (Collier Boulevard and the few scattered developments to the east of Collier Boulevard), the primary canal network generally provides a 25-year level of service. One exception to this is a short segment upstream of HC1, where the area west of the canal would be inundated by the combined effects of storm surge and high canal flows for the 5-year event and above. The area of inundation is undeveloped land surrounding two ponds used as water supply for Marco Island Utilities. Although no streets or buildings would be inundated, salty or brackish water flowing into the two ponds may affect their suitability as a drinking water source for some time following a storm surge event. Another area of concern is a channel restriction downstream of HC2. The restriction is a culvert crossing consisting of 4-42” diameter metal pipes located about 5,000 feet downstream of HC2, which should be replaced with an appropriately sized bridge.

8.1.3 Faka Union

The Faka Union Canal Network, including the Miller Canal between Structures MILLER2 and MILLER3, generally provides a 10-year level of service within the study area. Although the PM #1 results show several bank exceedances for the 5-year and 10-year events, these are localized; widespread inundation from canal bank overtopping only occurs for the 25-year event and above. It is also important to note that although PM #5 and PM #6 show developed areas of the Golden Gate Estates to be inundated for the 5-year and 10-year events, these areas appear to flood because of insufficient local drainage (roadside swales and ditches), and do not indicate an FPLOS deficiency in the primary canal network. For both the 5-year and 10-year events, the model results indicate that flood depths and durations are effectively zero in the vicinity of the canals and increase to significant positive values with distance from the canals.

During review of the model results, it was noted that the model representation of impervious runoff in these low density urban lands (via the paved area runoff coefficient) may be causing unrealistically high peak runoff rates. Future model updates using the new “ponded area drainage” routine available in the MIKE SHE 2017 release should provide opportunities to overcome this model limitation.

8.2 Future Land Use

Future land use was simulated based on several assumptions. Among these are implementation of topographic changes based on FEMA requirements, drainage improvements based on modern design standards, and compliance with current regulatory requirements for control of peak discharge rates. Overall, the model results indicate that there would be no widespread degradation of flood protection level of service resulting from the projected land use changes in the Cocohatchee, Henderson/Belle Meade, and Faka Union Watersheds. In many areas the flood depths are projected to decrease under future land use conditions. This results from a combination of higher topography and the resulting displacement of floodwaters within the footprint of the potential developments and simulated drainage improvements associated with the future land use conversions. However, there are some areas, outside the footprints of the potential developed areas, where flood levels are simulated to increase due to future development where future development may result in increased flood levels. The simulated increases can be mitigated in the design process. These increases are manifested both as higher peak flows and stages in the canals, and higher maximum water levels on the land surface as summarized in the following subsections.

8.2.1 Cocohatchee Watershed

Maximum stage increases (PM#1) are projected to occur within the Cocohatchee Canal because of the projected land use changes. The increases are most pronounced upstream of COCO3. At the eastern terminus of the canal, the increases are a result of the land use changes within the Golden Gate Watershed; specifically, within the Corkscrew Canal. A portion of the increase is associated with higher runoff volumes and displacement of floodwaters associated with the Esplanade development and other future development in the area. Increases in maximum overland water depths (PM#5, increases on the order of 0.1 to 0.3 feet) are projected in areas north of the Cocohatchee canal as a result of the PM#1 bank exceedances.

8.2.2 Henderson/Belle Meade Watershed

Maximum stages for the 5-year and 10-year events are projected to increase in the Henderson Creek Canal segment downstream of HC2 due to the potential future development along the canal. The same canal segment is projected to experience decreases in maximum stages for the 25-year and 100-year events, due to the additional storage provided by the connection of the existing mine pits east of the Henderson Canal. Immediately upstream of HC1, higher maximum stages for all design storm events are predicted to occur because of elevated topography, separated overland flow areas, and truncated canal cross-sections associated with the potential future development in the area.

With respect to maximum overland water depths (PM#5), the most significant increases are projected to occur north of the Naples Reserve development. Maximum flood depths are projected to increase by up to 0.3 feet for the 10-year and 25-year events, and to a lesser depth/extent for the 100-year event. Further north in the Henderson Creek / Belle Meade Watershed, areas adjacent to a potential low-density residential development east of the Collier Blvd./Hammock Road intersection are projected to experience slight increases in maximum water level.

8.2.3 Faka Union Watershed

The Faka Union and Miller Canals are both projected to experience increases in maximum stages for the 5-year and 10-year design storm events at the downstream ends of the evaluated segments. The downstream ends of these profiles coincide with the locations of the Faka Union and Miller Pump stations currently under construction as part of the PSRP. The maximum stage increases for the 5-year and 10-year storms, however, are projected to be generally contained within the canal banks. In the Faka Union Canal, peak stages are predicted to be reduced from existing conditions for the 25-year and 100-year events due to pump capacities that exceed the simulated flow rates. Caution should be used when interpreting these results, however, as the model domain does not include the entire contributing area of the Faka Union Canal and the model likely under-predicts flows in that canal as a result.

In the upper reaches of the Miller Canal, increased peak flow from the Golden Gate Canal through the Miller3 structure is projected to cause higher maximum stages throughout the length of the Miller Canal. This is a result of land use changes within the Golden Gate watershed. This increased interbasin flow, combined with projected future build-out of previously platted lots within the northern Golden Gate Estates (and the associated increase in runoff) is projected to strain the capacity of the Miller Pump station for short durations. It is important to note, however, that other performance measures evaluated (depth and duration of flooding) suggest the impacts of this would be localized to areas immediately adjacent to the Miller Canal and would likely not affect critical infrastructure such as I-75. Nevertheless, further study is recommended to determine evaluate the need for potential mitigative measures.

In the upper reaches of the Faka Union Canal, the conversion of native lands to low-density urban is projected to result in higher maximum stages. Again, this increase would result from the projected future build-out of previously platted lots within the northern Golden Gate Estates (and the associated increase in runoff). However, the model representation of impervious runoff in these low density urban lands (via the paved area runoff coefficient) may be causing unrealistically high peak runoff rates. Future model updates will provide opportunities to overcome this model limitation.

In the Faka Union Watershed, maximum overland water depths (PM#5) along the Faka Union Canal upstream of FU4 are projected to increase along a narrow corridor adjacent to the canal. This is consistent with the PM#1 results which show higher flood profiles within the canal, downstream of areas in the northern Faka Union Watershed which are projected to convert to low-density urban.

8.3 Future Sea Level Rise

Changes in flood protection LOS were evaluated in response to three hypothetical sea level rise scenarios. These changes are summarized separately for each of the three watersheds in the following subsections.

8.3.1 Cocohatchee Watershed

As expected, the largest increases in maximum canal stages would occur under the most severe of the three sea level rise (SLR3) conditions. The location most affected would be the downstream end of the Cocohatchee Canal, with a maximum rise of about 2 feet. The magnitude of the increase diminishes with distance upstream. Just west of COCO2 the projected rise is about 0.5 feet, and just east of COCO2, the projected rise under SLR3 conditions is less than 0.2 feet. Under SLR2, the maximum increase is projected to be approximately 1.0 foot upstream of COCO1; this increase rapidly diminishes to approximately 0.25 feet at the COCO2 structure. Stage increases associated with the SLR1 scenario are projected to be minimal within the Cocohatchee Watershed.

Discharge capacity at the tidally affected COCO1 structure would decrease due to sea level rise, most significantly under the SLR3 scenario. Under the 25-year design storm conditions, the higher tailwater elevations associated with SLR1 and SLR2 would result in negligible changes in discharge capacity, while SLR3 would result in a reduction in discharge capacity of approximately 26 cfs, about a 2.3% reduction.

8.3.2 Henderson/Belle Meade Watershed

Due to the low topography and relatively flat water surface profile of the Henderson Creek Canal, the tidally affected HC1 structure is particularly susceptible to backwater effects associated with storm surge and sea level rise. The characteristics of the fixed crest weir at HC1 may also impact performance. In the Henderson Creek Canal, significant stage increases are associated with all sea level rise scenarios over essentially the entire 8-mile length of the canal. Under SLR3, the maximum increase is projected to be approximately 1.7 feet upstream of HC1. Upstream of HC2, significant increases (approximately 0.5 feet) are projected for the 25-year event under SLR1, SLR2, and SLR3 conditions. The FPLOS of the upper reaches of the Henderson Creek Canal could be improved by removing the channel restriction downstream of HC2 (noted in **Section 8.1.2**). Further analysis is required to determine whether downstream improvements would be required to accommodate the increased peak flows likely to result from removing the restriction.

Within the Henderson Creek / Belle Meade Watershed, some significant increases in the 25-year overland inundation depths would occur, particularly under the SLR3 scenario. The largest increases (more than 1.5 feet) within the Henderson Creek / Belle Meade Watershed would occur adjacent to the downstream portion of the Henderson Creek Canal, between HC1 and HC2. Under the SLR2 scenario, the differences in inundation depths for the 25-year event would still be significant (up to 0.8 feet) but would be confined to a smaller region immediately upstream of HC1. Under the SLR1 scenario, the differences in inundation depths for the 25-year event would be negligible.

Although not a formal performance metric, maximum water table stages were also evaluated under the future sea level rise scenarios. Higher water table stages can affect infrastructure such as septic tanks, older sanitary sewers (through higher infiltration), and roadway base material. Pronounced differences in maximum water table stages are evident in coastal areas west and south of the Cocohatchee and Henderson/Belle Meade Watersheds. Within the Henderson/Belle Meade Watershed, the largest and most widespread increases in maximum water table stage would occur under the SLR3 scenario, in an area north of U.S. 41.

When the effects of future land use changes, future sea level rise, and an existing undersized culvert crossing are considered together, the Henderson Creek Canal is an area of concern for all three reasons.

8.3.3 Faka Union Watershed

Within the portion of the Faka Union Watershed evaluated under this task, no increases were projected under the SLR1, SLR2, and SLR3 scenarios. This is because the profiles only include the Faka Union and Miller Canal segments to remain under future conditions, which are the segments upstream of the Miller and Faka Union pump stations now under construction as part of the PSRP.

Annex A

Performance Measure #3 Evaluations for COCO1, GG1, and HC1

Prepared by Lichun Zhang, SFWMD

- THE EFFECTS OF SEA LEVEL RISE ON COCO1 PERFORMANCE
- THE EFFECTS OF SEA LEVEL RISE ON GG1 PERFORMANCE
- THE EFFECTS OF SEA LEVEL RISE ON HC1 PERFORMANCE

THE EFFECTS OF SEA LEVEL RISE ON COCO1 PERFORMANCE

DRAFT, Lichun Zhang, June 22, 2017

Background

The Coco1 structure is the tidal structure for the Cocohatchee Canal in Collier County. It controls water levels in a 1.3 mile reach of the canal using a two-bay gated spillway and a fixed crest weir (SFWMD, 2017). The design capacity of the structure is 1380 cfs.

This memo describes an assessment on the impact of sea level rise on the capacity of this tidal structure. The assessment is developed from simulations of structure flow where the upstream stage is fixed at the design headwater stage while the downstream stage oscillates with the tide. In each simulation, a storm surge of known intensity (as indicated by its return period) raises the tail-water stage, potentially suppressing flow[1]. A 12-hour average of flows were used to filter the effect of tidal oscillations. A range of tailwater levels and sea levels were examined. For this analysis, four sea-level scenarios were modeled and, for each scenario, a range of six design tailwater levels were considered, resulting a total of 24 model simulations.. The minimum flow values from these 24 model runs were then compared on a single plot.

Hydraulic Modeling of the COCO1 Structure

A simple hydraulic model was used to carry out the simulations. It is comprised of the structure, a short length of canal upstream of the structure and a short length of canal downstream of the structure. The upstream boundary condition is a fixed head equal to the design head water of Coco1, while the downstream boundary condition is based on a suite of time-varying tidal boundary conditions. The model also simulates all structure operations established for the operations protocol (SFWMD, 2017). This assessment is independent of rainfall and basin runoff.

Since the Coco1 structure is not part of the C&SF system, there are no design report available for the structure. Design headwater stage and flow are previously in another study using an integrated groundwater/surface water model conducted for the watershed in the area (SFWMD, 2017). The estimated discharge for a 25-year storm is about 1380 cfs. The design headwater stage is estimated to be 7.15 ft NGVD.

The tidal boundary conditions are depicted in both **Figure 1** and **Figure 2.** **Figure 1** shows an example of the suite of tidal boundaries (2-yr to 100-yr return period) associated with the tailwater stages for current (i.e. 2015) sea level conditions. **Figure 2** shows the tidal boundary water levels for six different sea level rise scenarios, all reflecting a 5-yr tailwater. These sea level rise scenarios represent one estimates of current (2015) sea level conditions: IPCCAR5-MEDIAN (2015); and three estimates of future sea level conditions: 2065 LOW (USACE-INTERMEDIATE 2065), 2065 INTM (IPCCAR5-MEDIAN 2065), and 2065 HIGH (USACE-HIGH 2065).

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[1] There are ongoing discussions about the cause of the observed high tailwater conditions and the correlation between high tailwater and high flow. For this paper, we assume that tailwater is directly correlated to flow and that sea level rise translates to a matching rise in tailwater.

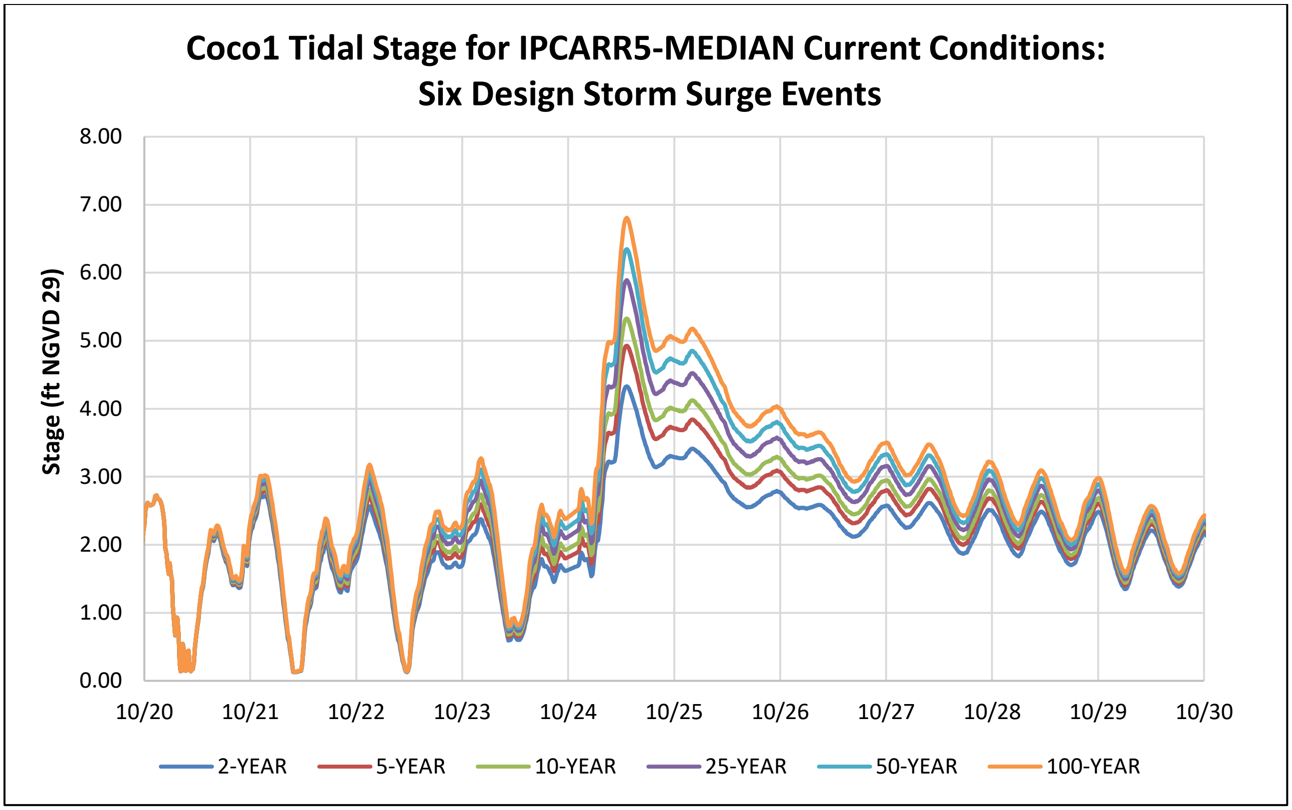

Figure 1. COCO1 tidal stage for existing (2015) conditions: six design tailwater events

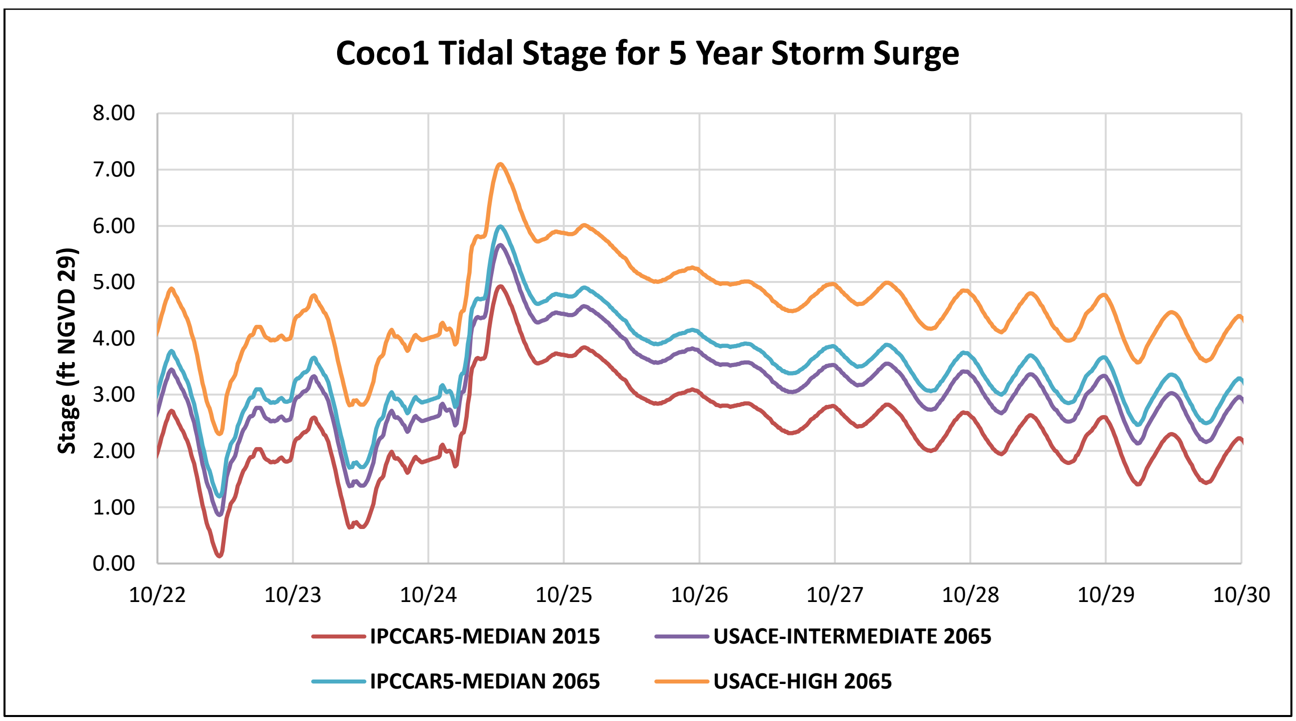

Figure 2. COCO1 tidal stage for six sea level rise scenarios (5-year tailwater for all scenarios)

As shown in **Figure 2**, the tidal stage for current sea level conditions is IPCCAR5-MEDIAN 2015 (CSL).

For the three future sea level rise conditions:

SLR1 = YEAR2065 USACE-INTERMEDIATE = CSL + 0.7313 feet = CSL + **0.73** feet
SLR2 = YEAR2065 IPCCAR4-MEDIAN = CSL + 1.0609 feet = CSL + **1.06** feet
SLR3 = YEAR2065 USACE-HIGH = CSL + 2.1687 feet = CSL + **2.17** feet

Example stage and flow values simulated during a tailwater event are shown in **Figure 3** as instantaneous values and in **Figure 4** as 12-hour moving average values.

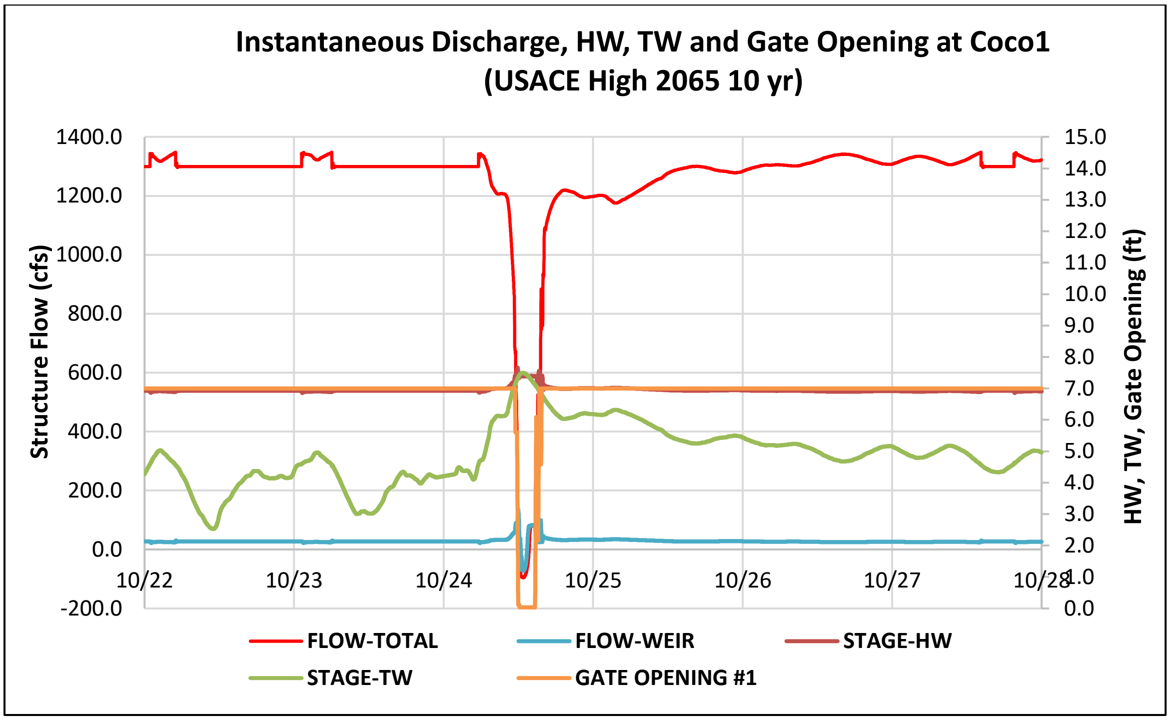

Figure 3. Instantaneous Flow, Stages and gate opening at COCO1: Design Headwater with USACE High 2065 10-year tailwater level

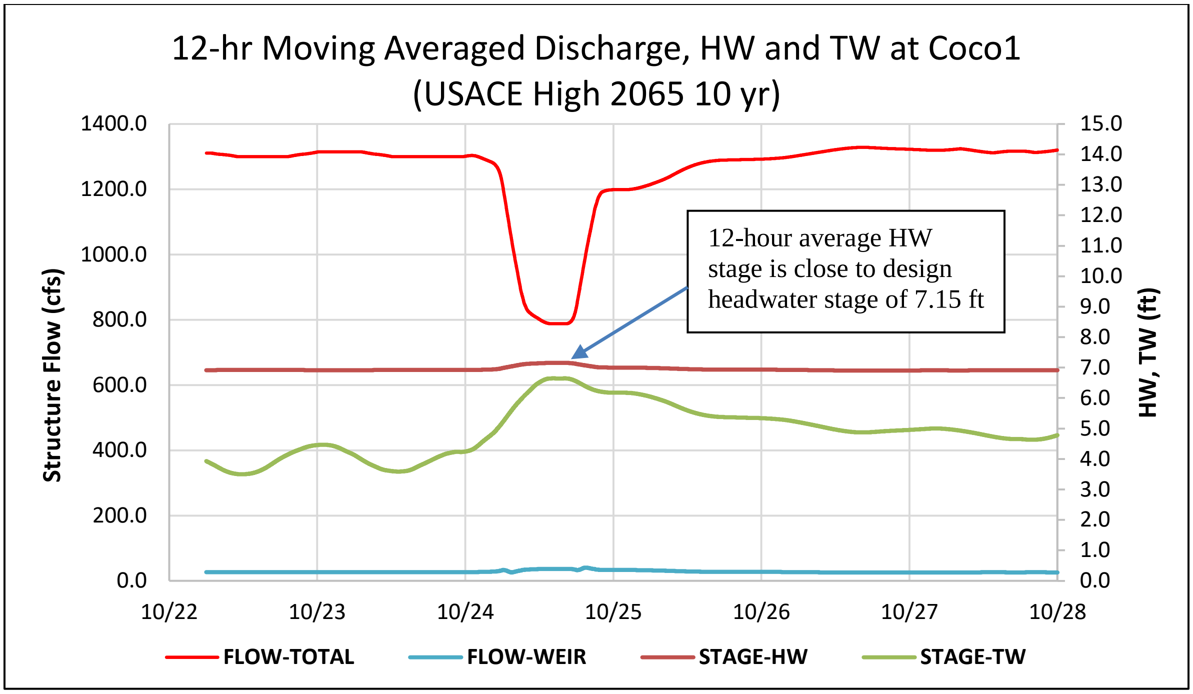

Figure 4. 12-hour moving average Flow and Stages at Coco1: Design Headwater with USACE High 2065 10-year tailwater level

Figure 3 and **Figure 4** depicts head water stage, tail water stage and flow through Coco1 for USACE High 2065 10-year tailwater level. The 12-hour average minimum flow is 788 cfs, which means that the structure lost some of its capacity (the flow at 25-yr storm is approximately 1380 cfs) at this tailwater condition. This minimum flow is plotted on **Figure 5** below, along with all other simulation results.

Model Simulation Results

Figure 5 shows the performance of Coco1 as affected by sea level rise and tailwater. Displayed are the minimum flow values (derived in the manner discussed in the previous section) for each of the twenty-four simulations (four sea level scenarios each with six design tailwater events). The structure flows associated with IPCCAR5-MEDIAN 2015 conditions are shown as a purple x's; 2015 Current (IPCCAR5-MEDIAN 2015), 2065 Low (USACE-INTERMEDIATE 2065), 2065 INTM (IPCCAR5-MEDIAN 2065), and 2065 HIGH (USACE-HIGH 2065) conditions are shown using the red markers indicated. Furthermore, each line shows the short-term suppression of flows caused by tailwater along with long-term flow suppression caused by seal level rise alone.

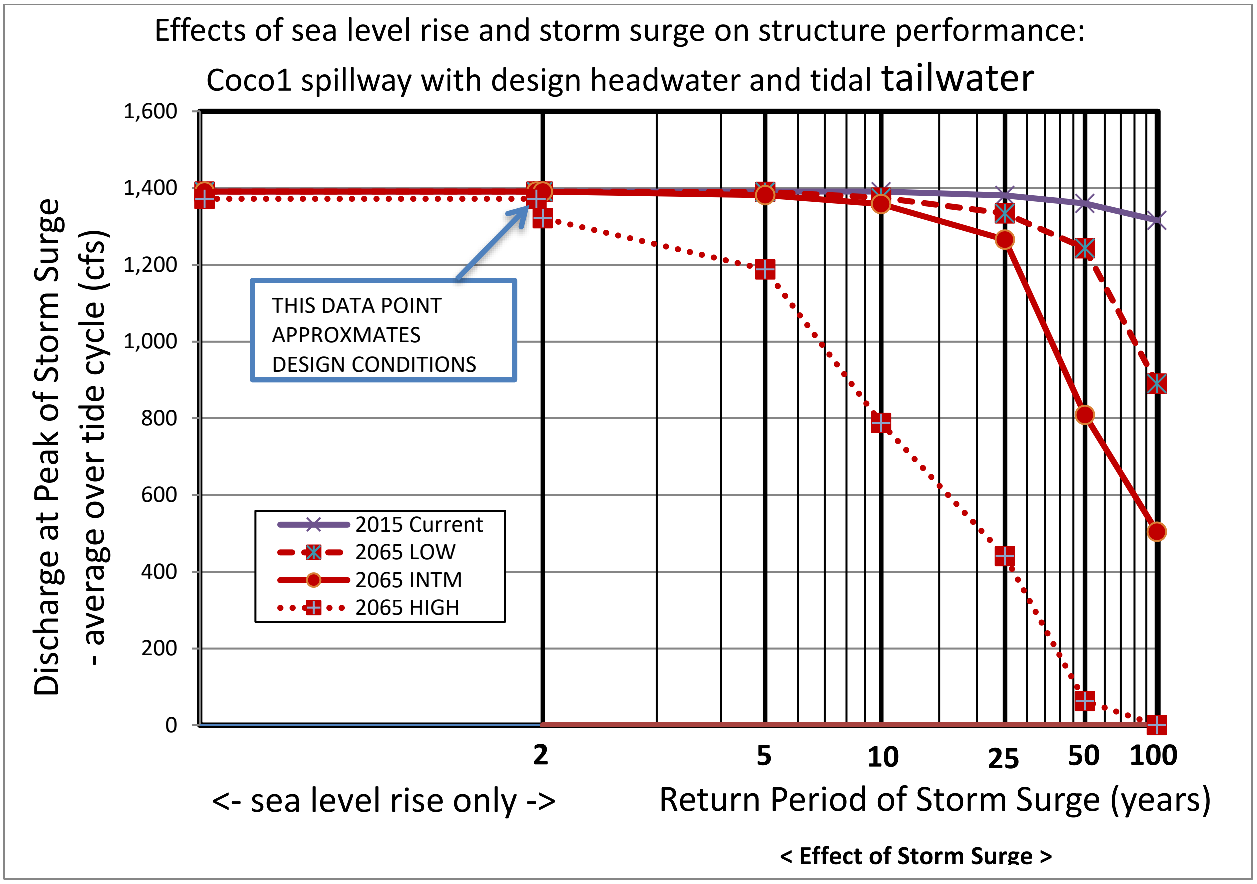

Figure 5. Effects of sea level rise and tailwater levels on the capacity of COCO1

Figure 5 reveals that Coco1, under current (2015) sea levels, can pass its design flow under all but the largest (50-year and 100-year) tailwater conditions. For a 0.73 foot sea level rise, the structure can pass its design flow up to a 10 year return period surge. For a 1.06 foot seal level rise, the structure can pass its design flow up to 10 year return period surge events. For a sea level rise of 2.17 feet, tailwater above 2 year return period will suppress flow at the structure. However, even with a 2.17 foot rise, the structure can carry its design flow as long as tailwater is minimal.

Conclusions

Based on the assumptions of this study, the COCO1 structure will not be significantly impacted by low (0.73 foot) and moderate (1.06 foot) sea level rise. High (2.17 foot) sea level rise will suppress flows under moderate tailwater conditions.

Reference

South Florida Water Management District, 2017. Water Control Operations Atlas: big Cypress Basin System. H&H Bureau and BCB Service Center, SFWMD, West Palm Beach, FL. 180 pp. April 20, 2017.

THE EFFECTS OF SEA LEVEL RISE ON GG1 PERFORMANCE

DRAFT, Lichun Zhang, June 22, 2017

Background

The GG1 structure is the tidal structure for the Golden Gate Main Canal in Collier County. It controls water levels in a 2 ½ mile reach of the canal using a three-bay bottom-hinged variable weir (SFWMD, 2017). The GG1 canal is the largest managed watershed in the Big Cypress Basin, drainage an area of 71,000 acres. The design capacity of the structure is 4,625 cfs[1].

This memo describes an assessment on the impact of sea level rise on the capacity of this tidal structure. The assessment is developed from simulations of structure flow where the upstream stage is fixed at the design headwater stage while the downstream stage oscillates with the tide. In each simulation, a storm surge of known intensity (as indicated by its return period) raises the tail-water stage, potentially suppressing flow[2]. A 12-hour average of flows were used to filter the effect of tidal oscillations. A range of tailwater levels and sea levels were examined. For this analysis, four sea-level scenarios were modeled and, for each scenario, a range of six design tailwater levels were considered, resulting a total of 24 model simulations. The minimum flow values from these 24 model runs were then compared on a single plot.

Hydraulic Modeling of the GG1 Structure

A simple hydraulic model was used to carry out the simulations. It is comprised of the structure, a short length of canal upstream of the structure and a short length of canal downstream of the structure. The upstream boundary condition is a fixed head equal to the design head water of GG1, while the downstream boundary condition is based on a suite of time-varying tidal boundary conditions. The model also simulates all structure operations established for the operations protocol (SFWMD, 2017). This assessment is independent of rainfall and basin runoff.

Since the GG1 structure is not part of the C&SF system, there are no design report available for the structure. Design headwater stage and flow were estimated previously in another study using an integrated groundwater/surface water model conducted for the watershed in the area (SFWMD, 2017). The estimated discharge for a 25-yr storm is about 3600 cfs (shown as the bottom line of the gray box in **Figure 5**). The design headwater stage is estimated to be 7.0 ft NGVD.

The tidal boundary conditions are depicted in both **Figure 1** and **Figure 2. Figure 1** shows an example of the suite of tidal boundaries (2-yr to 100-yr return period) associated with the tailwater stages for current (i.e. 2015) sea level conditions. **Figure 2** shows the tidal boundary water levels for six different sea level rise scenarios, all reflecting tailwater stages with a 5-year return period. These sea level rise scenarios represent one estimates of current (2015) sea level conditions: IPCCAR5-MEDIAN (2015); and three

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[1] *This report is based on the District's current flow rating equation for GG1. Due to concerns about the current rating for flow under high tailwater conditions, a Computational Fluid Dynamic (CFD) model is being developed to provide a more accurate estimation of flow at GG1.*

[2] *There are ongoing discussions about the cause of the observed high tailwater conditions and the correlation between high tailwater and high flow. For this paper, we assume that tailwater is directly correlated to flow and that sea level rise translates to a matching rise in tailwater.*

estimates of future sea level conditions: 2065 LOW (USACE-INTERMEDIATE 2065), 2065 INTM (IPCCAR5-MEDIAN 2065), and 2065 HIGH (USACE-HIGH 2065).

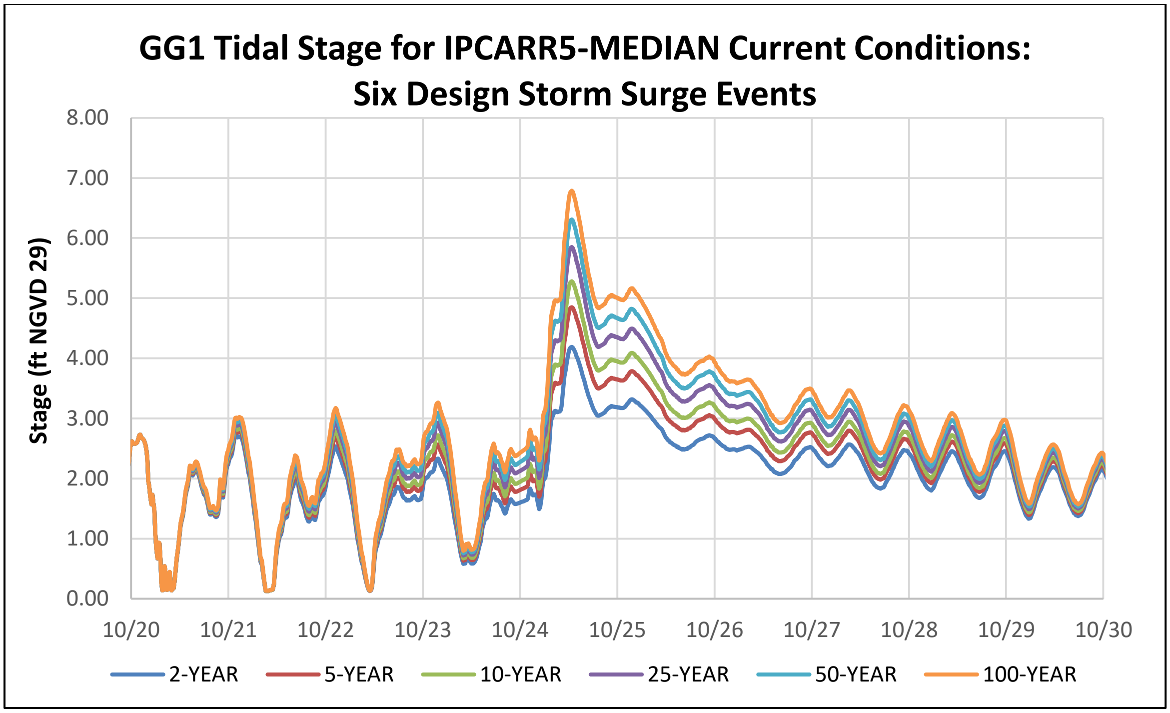

Figure 1. GG1 tidal stage for existing (2015) conditions: six design tailwater events

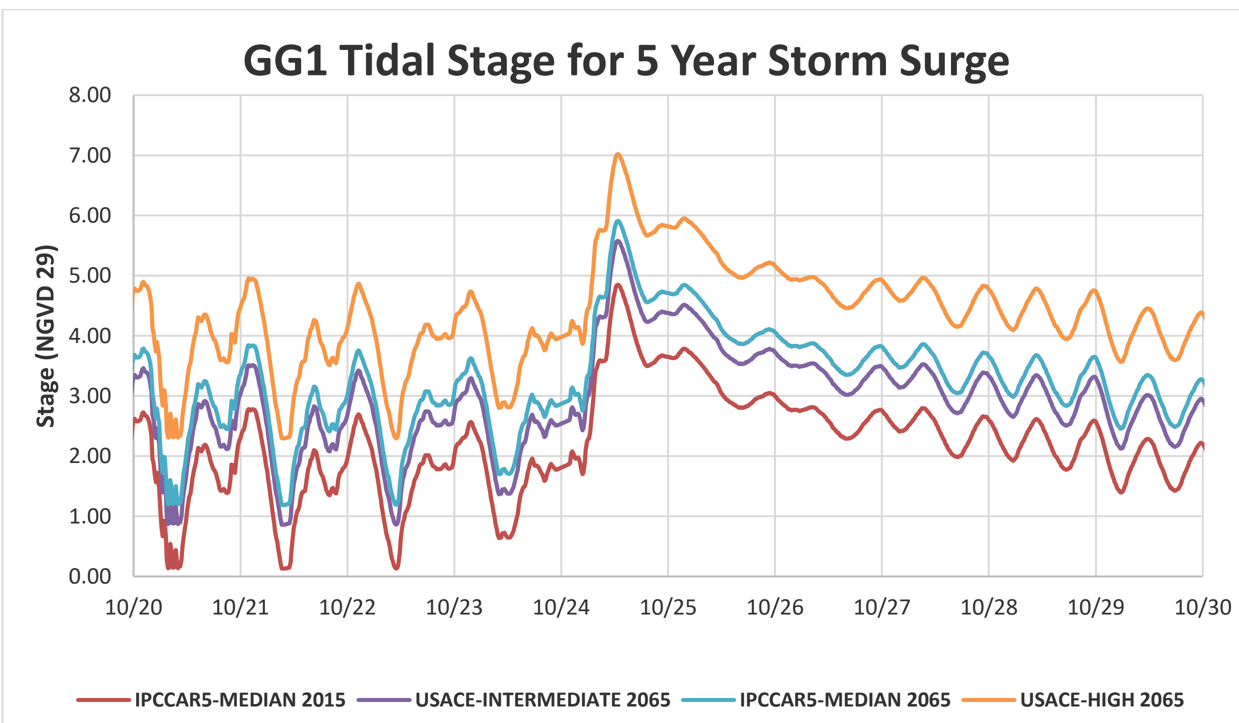

Figure 2. GG1 tidal stage for six sea level rise scenarios (5-year tailwater for all scenarios)

As shown in **Figure 2**, the tidal stage for current sea level conditions is IPCCAR5-MEDIAN 2015 (CSL).

For the three future sea level rise conditions, it should be noted that:

SLR1 = YEAR2065 USACE-INTERMEDIATE = CSL + 0.7313 feet = CSL + **0.73** feet
SLR2 = YEAR2065 IPCCAR4-MEDIAN = CSL + 1.0609 feet = CSL + **1.06** feet
SLR3 = YEAR2065 USACE-HIGH = CSL + 2.1687 feet = CSL + **2.17** feet

Example stage and flow values are shown in **Figure 3** as instantaneous values and in **Figure 4** as 12-hour moving average values.

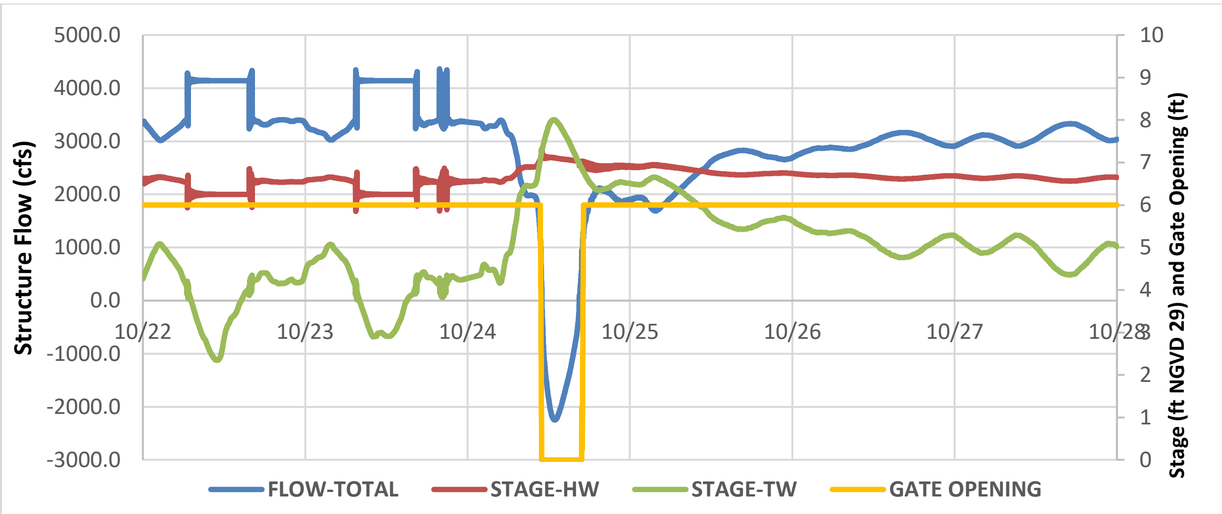

Figure 3. Instantaneous flow, stages and gate opening at GG1 simulated using the design headwater stage in conjunction with the USACE High 2065 25-year tailwater level

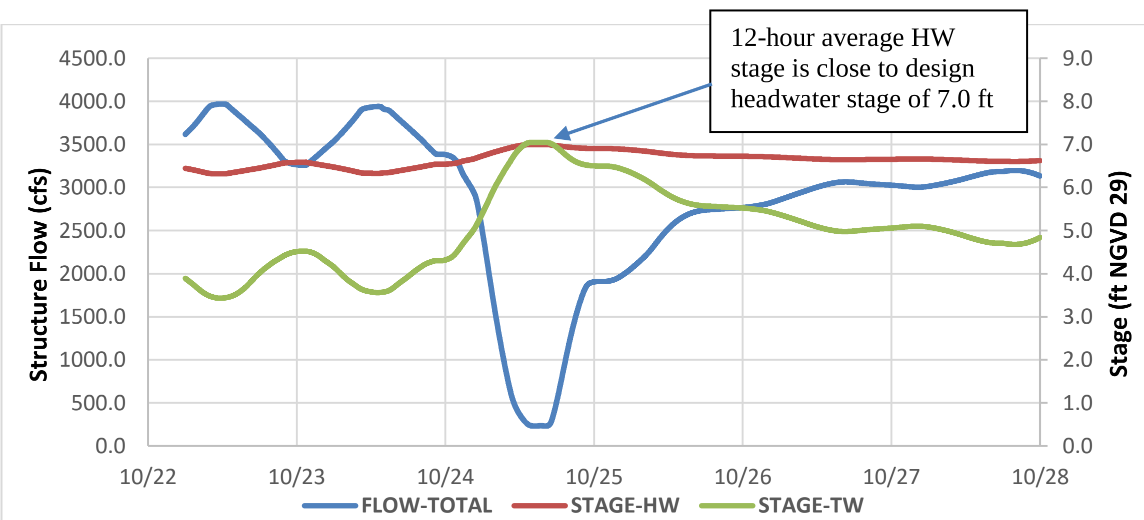

Figure 4. 12-hour moving average flow and stages at GG1 simulated using the design headwater along with USACE High 2065 25-year tailwater level

Figure 3 and **Figure 4** depicts head water stage, tail water stage and flow through GG1 for USACE High 2065 25-year tailwater level. The variable weir gates of GG1 close when the tailwater is higher than headwater. This suppresses flow. As **Figure 4** shows, the 12-hour average minimum flow is 231 cfs, which means that the structure lost most of its capacity (the flow at 25-yr storm is approximately 3600 cfs) at this tailwater condition. This minimum flow is plotted on **Figure 5** below, along with all other simulation results.

Model Simulation Results

Figure 5 shows the performance of GG1 as affected by sea level rise and tailwater. Displayed are the minimum flow values (derived in the manner discussed in the previous section) for each of the twenty-four simulations (four sea level scenarios each with six design tailwater events). The structure flows

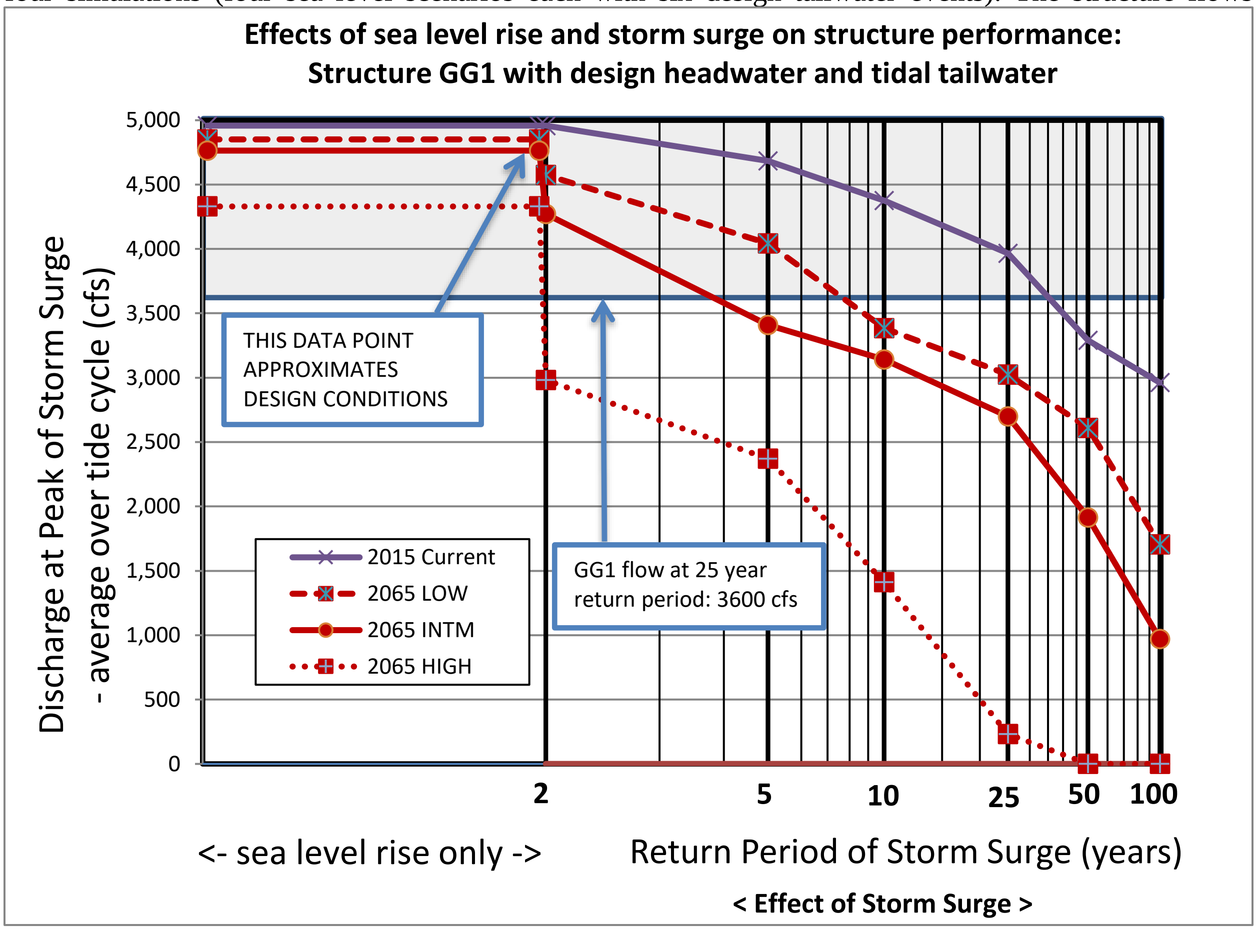

Figure 5. Effects of sea level rise and tailwater levels on the capacity of GG1

associated with IPCCAR5-MEDIAN 2015 conditions are shown as a purple x's; 2015 Current (IPCCAR5-MEDIAN 2015), 2065 Low (USACE-INTERMEDIATE 2065), 2065 INTM (IPCCAR5-MEDIAN 2065), and 2065 HIGH (USACE-HIGH 2065) conditions are shown using the red markers indicated. Furthermore, each line shows the short-term suppression of flows caused by tailwater along with long-term flow suppression caused by seal level rise alone.

Figure 5 reveals that GG1, under current (2015) sea levels, can pass its design flow under all but the largest (50-year and 100-year) tailwater conditions. For a 0.73 foot sea level rise, the structure can pass its design flow up to a 5 year return period surge. For a 1.06 foot seal level rise, the structure can pass its design flow for 2 year return period surge events only. For a sea level rise of 2.17 feet, any tailwater will suppress flow at the structure. However, even with a 2.17 foot rise, the structure can carry its design flow as long as tailwater is minimal.

Conclusions

Based on the assumptions of this study, the GG1 structure will not be significantly impacted by low (0.73 foot) and moderate (1.06 foot) sea level rise. High (2.17 foot) sea level rise will suppress flows under moderate tailwater conditions.

Reference

South Florida Water Management District, 2017. Water Control Operations Atlas: big Cypress Basin System. H&H Bureau and BCB Service Center, SFWMD, West Palm Beach, FL. 180 pp. April 20, 2017.

THE EFFECTS OF SEA LEVEL RISE ON HC1 PERFORMANCE

DRAFT, Lichun Zhang, June 29, 2017

Background

The HC1 structure is the tidal structure for the Henderson Creek Canal in Collier County. It controls water levels in a 5.66 mile reach of the canal using a two-bay gated variable crest weir, a fixed crest overflow weir, and an emergency box culvert with an upstream sluice gate and a downstream flap gate (SFWMD, 2017). The design capacity of the structure is 780 cfs (SFWMD, 2017).

This memo describes an assessment on the impact of sea level rise on the capacity of this tidal structure. The assessment is developed from simulations of structure flow where the upstream stage is fixed at the design headwater stage while the downstream stage oscillates with the tide. In each simulation, a storm surge of known intensity (as indicated by its return period) raises the tail-water stage, potentially suppressing flow[1]. A 12-hour average of flows were used to filter the effect of tidal oscillations. A range of tailwater levels and sea levels were considered, resulting a total of 24 model simulations. For this analysis, four sea-level scenarios were modeled and, for each scenario, a range of six design tailwater levels were examined. The minimum flow values from these 24 model runs were then compared on a single plot.

Hydraulic Modeling of the HC1 Structure

A simple hydraulic model was used to carry out the simulations. It is comprised of the structure, a short length of canal upstream of the structure and a short length of canal downstream of the structure. The upstream boundary condition is a fixed head equal to the design headwater of HC2, while the downstream boundary condition is based on a suite of time-varying tidal boundary conditions. The model also simulates all structure operations established for the operations protocol (SFWMD, 2017). This assessment is independent of rainfall and basin runoff.

The HC1 structure is not part of the C&SF system and there are no design report available for the structure. Design headwater stage and flow are estimated previously in another study using an integrated groundwater/surface water model developed for the watershed in the area (SFWMD, 2017). The estimated discharge for a 25-year storm is about 780 cfs. The design headwater stage is estimated to be 6.63 ft NGVD.

The tidal boundary conditions are depicted in both **Figure 1** and **Figure 2. Figure 1** shows an example of the suite of tidal boundaries (2-yr to 100-yr return period) associated with the tailwaters for current (i.e. 2015) sea level conditions. **Figure 2** shows the tidal boundary water levels for six different sea level rise scenarios, all reflecting a 5-yr tailwater. These sea level rise scenarios represent one estimates of current (2015) sea level conditions: IPCCAR5-MEDIAN (2015); and three estimates of future sea level conditions: 2065 LOW (USACE-INTERMEDIATE 2065), 2065 INTM (IPCCAR5-MEDIAN 2065), and 2065 HIGH (USACE-HIGH 2065).

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[1] There are ongoing discussions about the cause of the observed high tailwater conditions and the correlation between high tailwater and high flow. For this paper, we assume that tailwater is directly correlated to flow and that sea level rise translates to a matching rise in tailwater.

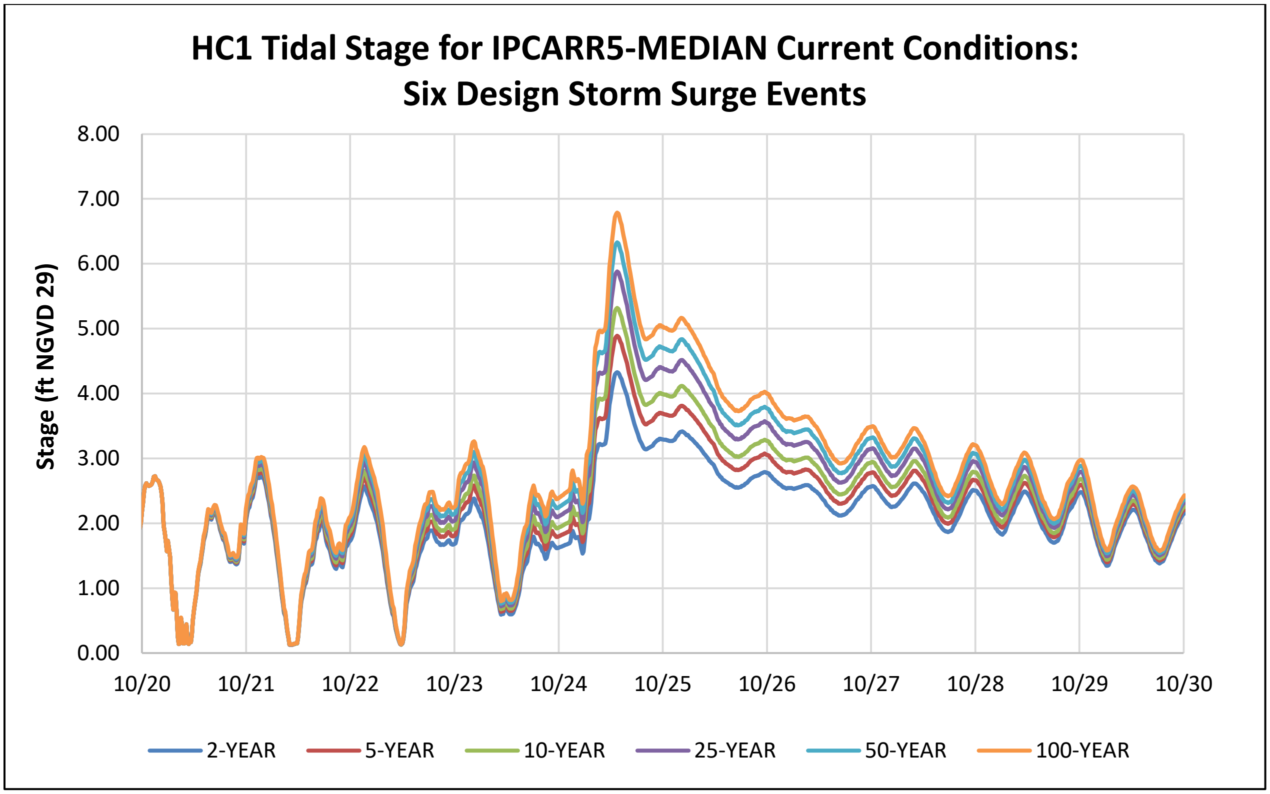

Figure 1. HC1 tidal stage for existing (2015) conditions: six design tailwater events

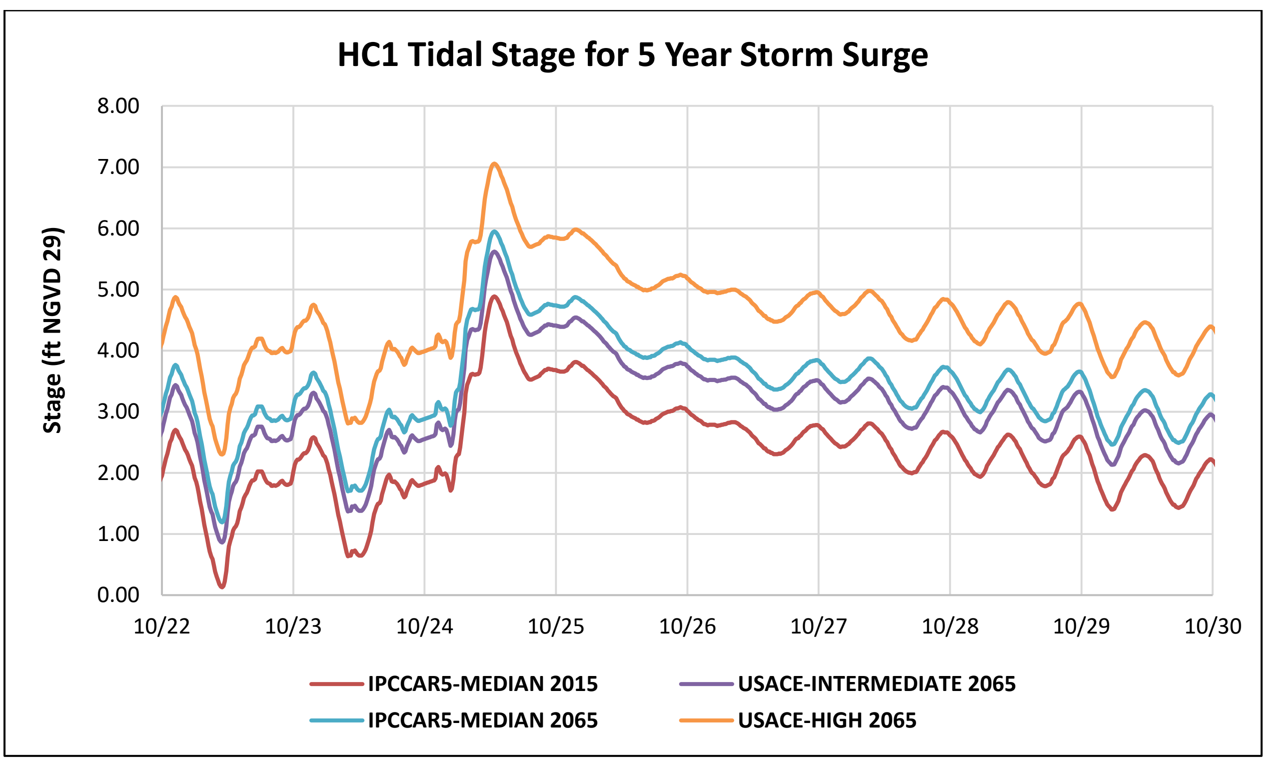

Figure 2. HC1 tidal stage for six sea level rise scenarios (5-year tailwater for all scenarios)

As shown in **Figure 2**, the tidal stage for current sea level conditions is IPCCAR5-MEDIAN 2015 (CSL).

For the three future sea level rise conditions:

SLR1 = YEAR2065 USACE-INTERMEDIATE = CSL + 0.7313 feet = CSL + **0.73** feet
SLR2 = YEAR2065 IPCCAR4-MEDIAN = CSL + 1.0609 feet = CSL + **1.06** feet
SLR3 = YEAR2065 USACE-HIGH = CSL + 2.1687 feet = CSL + **2.17** feet

Example stage and flow values simulated during a tailwater event are shown in **Figure 3** as instantaneous values and in **Figure 4** as 12-hour moving average values.

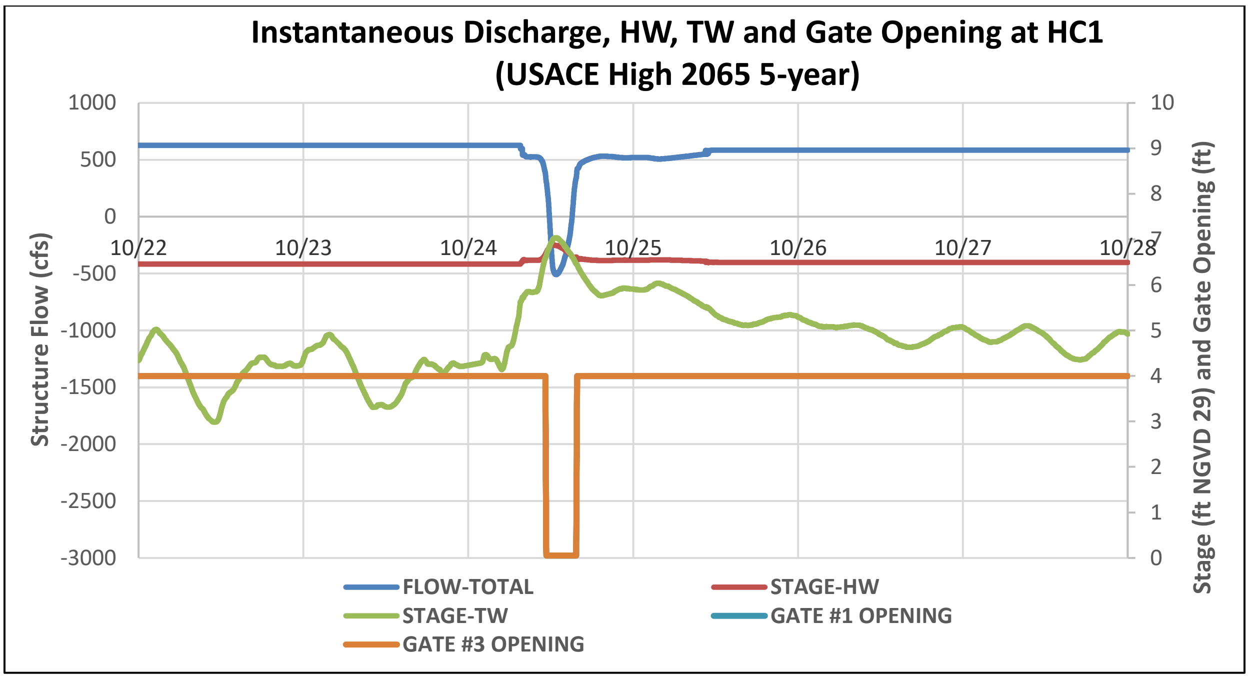

Figure 3. Instantaneous Flow, Stages and gate opening at HC1: Design Headwater with USACE High 2065 5-year tailwater level

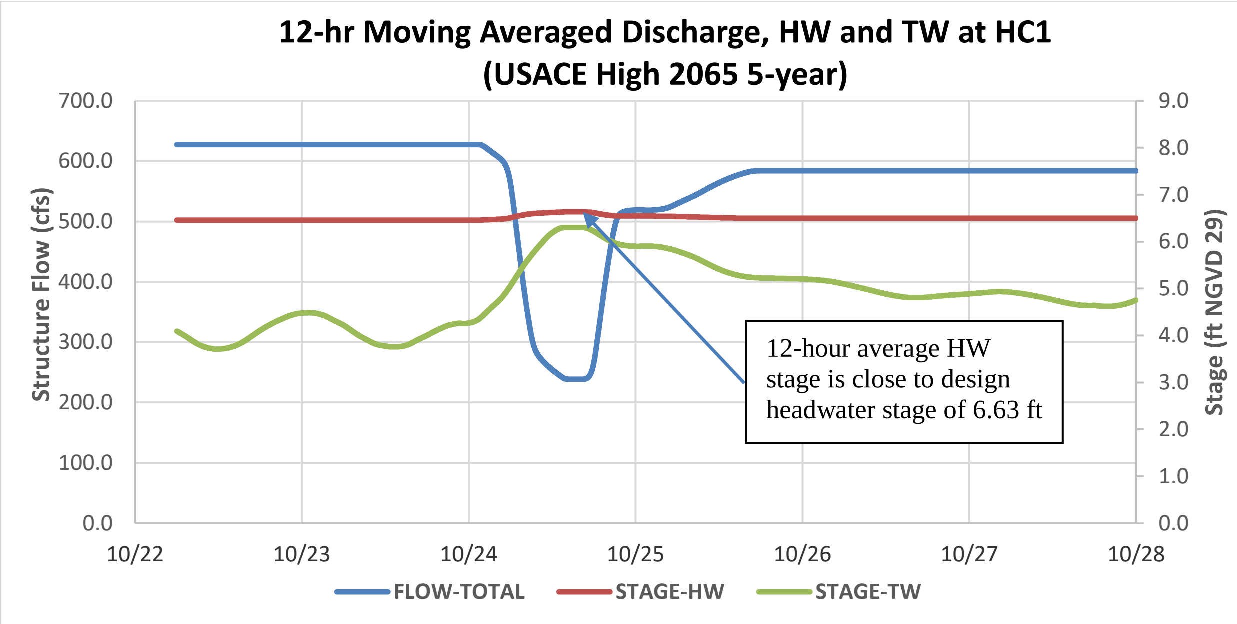

Figure 4. 12-hour moving average Flow and Stages at HC1: Design Headwater with USACE High 2065 5-year tailwater level

Figure 3 and **Figure 4** depicts head water stage, tail water stage and flow through HC1 for USACE High 2065 5-year tailwater level. The 12-hour average minimum flow is 239 cfs, which means that the structure lost a large part of its capacity (the flow at 25-yr storm is approximately 780 cfs) at this tailwater condition. This minimum flow is plotted on **Figure 5** below, along with all other simulation results.

Model Simulation Results

Figure 5 shows the performance of HC1 as affected by sea level rise and tailwater. Displayed are the minimum flow values (derived in the manner discussed in the previous section) for each of the twenty-four simulations (four sea level scenarios each with six design tailwater events). The structure flows associated with IPCCAR5-MEDIAN 2015 conditions are shown as a purple x's; 2015 Current (IPCCAR5-MEDIAN 2015), 2065 Low (USACE-INTERMEDIATE 2065), 2065 INTM (IPCCAR5-MEDIAN 2065), and 2065 HIGH (USACE-HIGH 2065) conditions are shown using the red markers indicated. Furthermore, each line shows the short-term suppression of flows caused by tailwater along with long-term flow suppression caused by seal level rise alone.

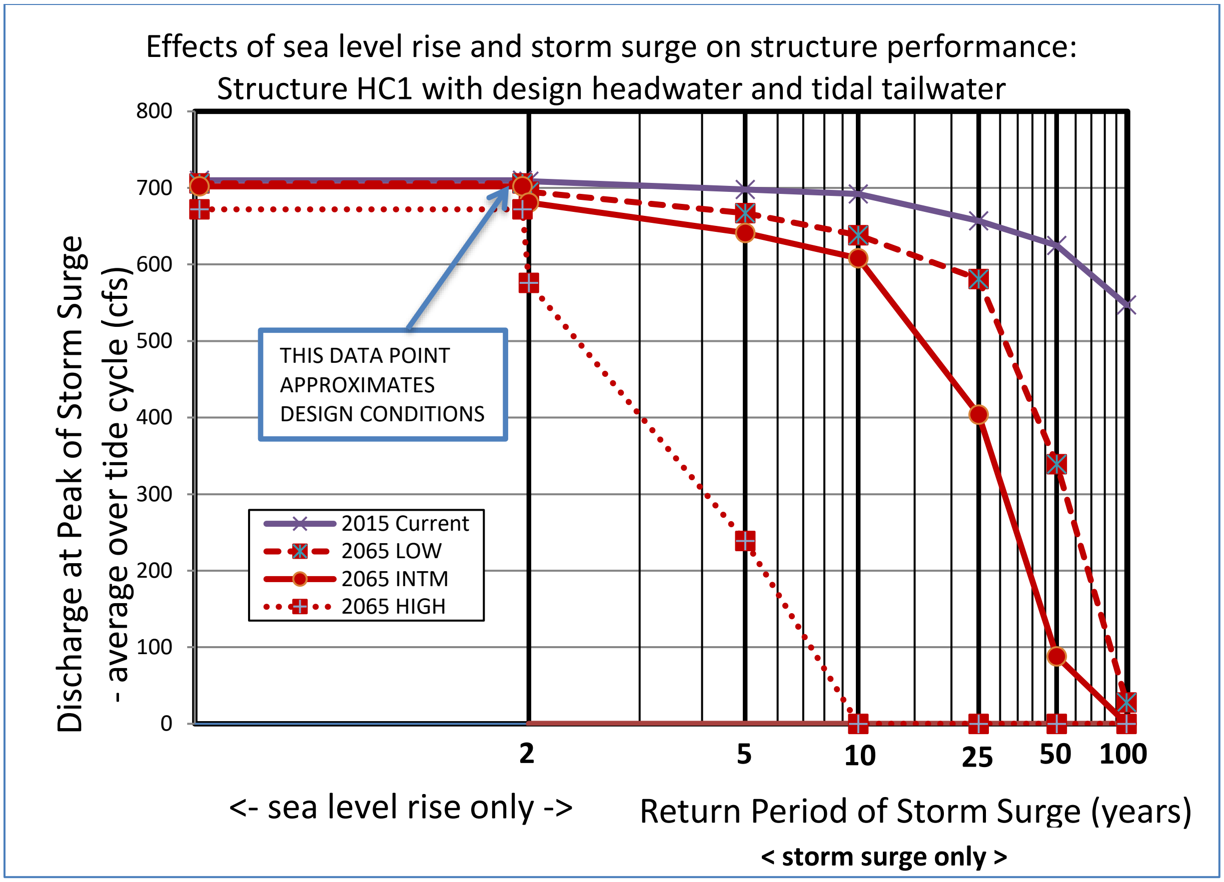

Figure 5. Effects of sea level rise and tailwater levels on the capacity of HC1

Figure 5 reveals that HC1, under current (2015) sea levels, can pass its design flow under 25 year tailwater conditions. For a 0.73 foot sea level rise, the structure can pass its design flow up to a 10 year return period surge. For a 1.06 foot seal level rise, the structure can pass its design flow up to 10 year return period surge events. For a sea level rise of 2.17 feet, tailwater above 2 year return period will suppress flow at the structure. However, even with a 2.17 foot rise, the structure can carry its design flow as long as tailwater is minimal.

Conclusions

Based on the assumptions of this study, the HC1 structure will not be significantly impacted by low (0.73 foot) and moderate (1.06 foot) sea level rise. High (2.17 foot) sea level rise will suppress flows under moderate tailwater conditions.

Reference

South Florida Water Management District, 2017. Water Control Operations Atlas: big Cypress Basin System. H&H Bureau and BCB Service Center, SFWMD, West Palm Beach, FL. 180 pp. April 20, 2017.

< Effect of Storm Surge >

A – Peak Stage and Discharge Rate Summary Tables

(Current and Future Conditions)

Table 1 Peak Stage Summary

Structure	Mike 11 Chainage	Peak Stage (ft, NAVD)										
		5yr		10yr		25yr					100yr	
		Current	Future	Current	Future	Current	Future-SLR0	Future-SLR1	Future-SLR2	Future-SLR3	Current	Future
COCO1	50433.48	6.63	6.71	6.96	7.01	7.45	7.49	7.80	8.00	8.60	8.04	8.06
COCO2	43635.17	10.10	10.26	10.69	10.82	11.12	11.23	11.28	11.31	11.40	11.44	11.50
COCO3	27022.08	11.54	11.90	12.05	12.34	12.52	12.81	12.81	12.81	12.82	12.99	13.15
HC1	44904.86	4.71	4.76	5.10	5.16	5.50	5.65	6.16	6.39	7.33	6.15	6.40
HC2	15370.73	8.69	8.68	8.69	8.68	9.20	8.75	9.20	9.21	9.26	9.83	9.24
Miller2	36417.32	6.93		7.98		9.30					10.23	
Miller3	0.00	11.01	11.73	12.15	12.44	12.56	12.76	12.76	12.76	12.76	12.91	13.03
Miller Pump	47147.7		7.56		7.80		8.42	8.38	8.39	8.40		9.34
FU Pump	92296.9		6.76		7.15		7.11	7.10	7.10	7.11		7.60
FU3	80111.55	7.53		8.17		8.82					10.12	
FU4	61043.31	9.62	10.45	10.91	11.20	11.85	11.52	11.52	11.54	11.51	11.85	11.88
FU5	33628.61	13.46	14.44	14.52	14.91	14.79	15.05	15.05	15.06	15.05	14.95	15.19
FU6	21744.92	15.55	17.37	16.90	18.11	17.43	18.55	18.57	18.61	18.59	17.89	18.92
FU7	9994.77	17.47	19.38	18.45	20.90	19.42	21.72	21.73	21.69	21.73	20.04	21.96

	N/A for Current conditions
	N/A for Future conditions

Table 2 Peak Discharge Rate Summary

Structure	Mike11 Chainage	Peak Discharge (cfs)										
		5yr		10yr		25yr					100yr	
		Current	Future	Current	Future	Current	Future-SLR0	Future-SLR1	Future-SLR2	Future-SLR3	Current	Future
COCO1	50524.93	1536	1557	1665	1691	1778	1799	1769	1768	1688	1858	1872
COCO2	43779.53	918	953	1028	1059	1101	1131	1117	1116	1090	1145	1165
COCO3	27329.40	214	436	272	338	343	368	368	369	385	423	445
HC1	45311.68	232	234	397	375	604	568	578	585	698	801	763
HC2	15403.54	86	112	154	166	219	245	240	244	217	215	288
Miller2	37237.53	471		708		969					1037	
Miller3	173.88	349	517	616	687	672	749	749	749	749	720	768
Miller Pump	47859.1		780		1015		1250	1250	1250	1250		1250
FU Pump	92523.2		1240		1710		1710	1710	1710	1710		2180
FU3	80754.59	1032		1308		1499					1701	
FU4	61712.60	1123	1239	1305	1371	1715	1537	1534	1537	1531	1528	1489
FU5	33841.86	763	1040	964	1171	1031	1191	1190	1188	1186	1079	1228
FU6	22522.97	567	1033	833	1256	1118	1382	1383	1378	1383	1176	1439
FU7	10990.81	270	783	505	1126	789	1260	1263	1244	1262	942	1279

	N/A for Current conditions
	N/A for Future conditions

B – MIKE 11 Detailed Time Series for 5-year Storm, Current Conditions

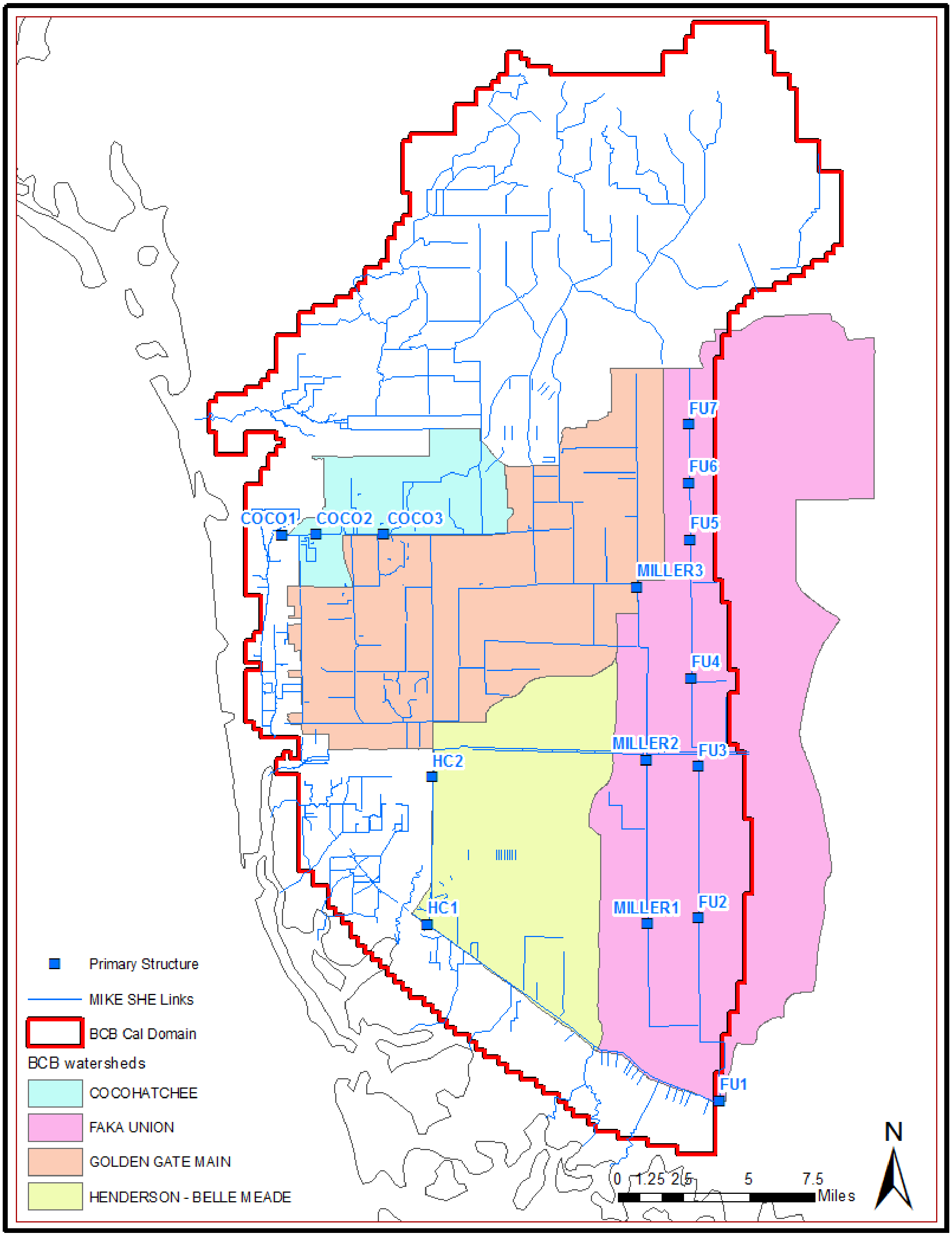

Figure 1 Mike SHE Stations within COCO, FAKA and HEND Watersheds

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Figure 2. COCO1 Discharge (cfs)

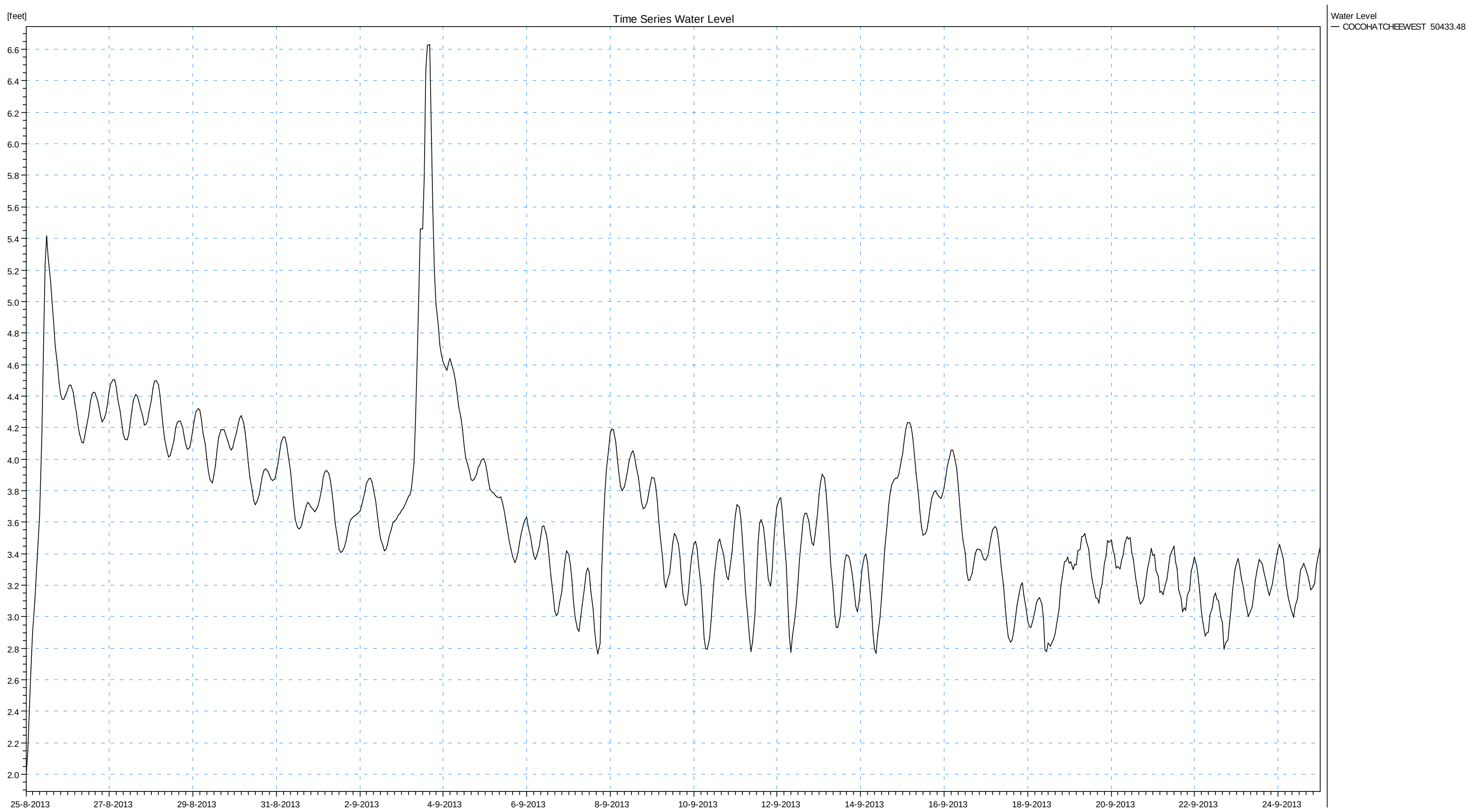

Figure 3. COCO1 Headwater Stage (ft)

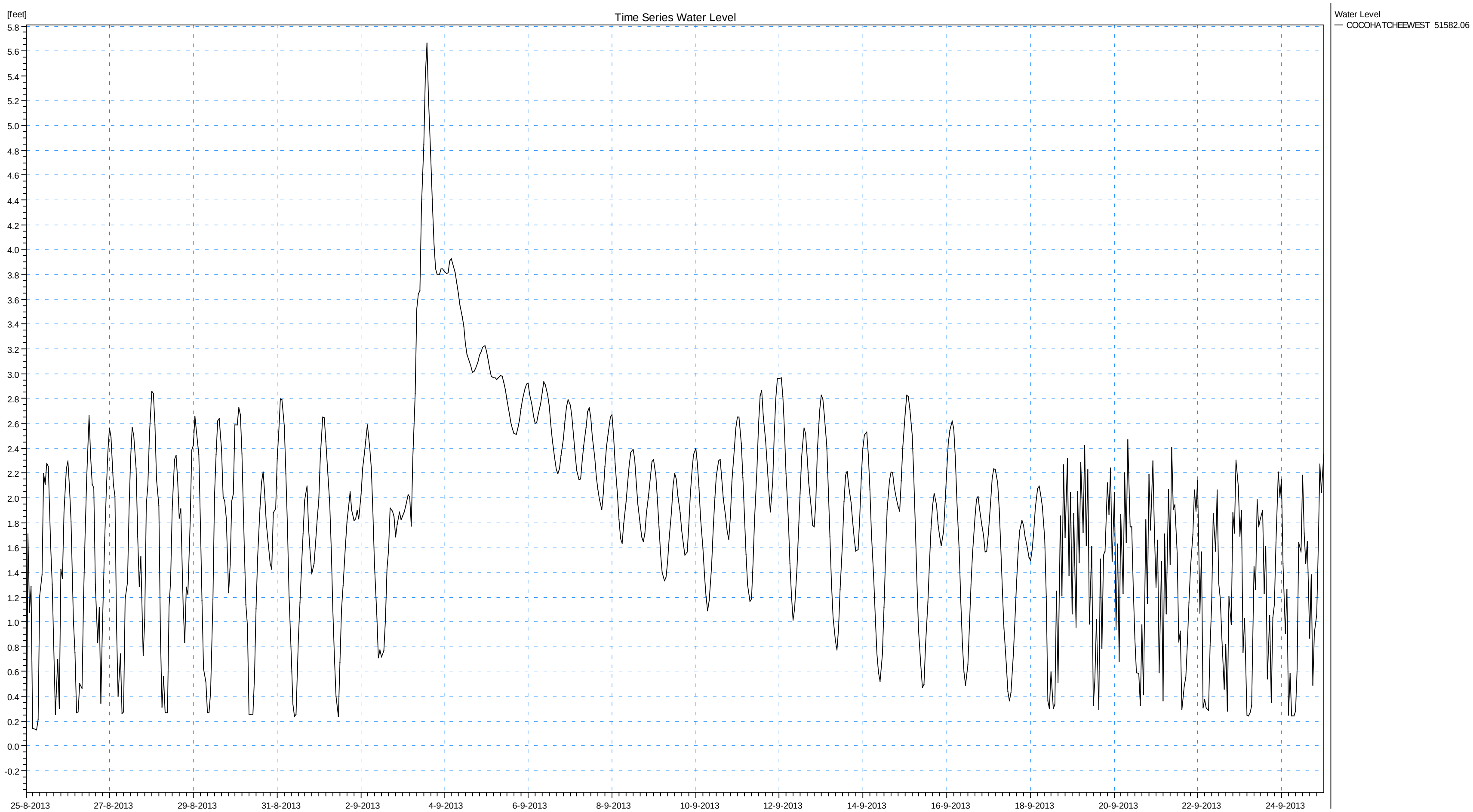

Figure 4. COCO1 Tailwater Stage (ft)

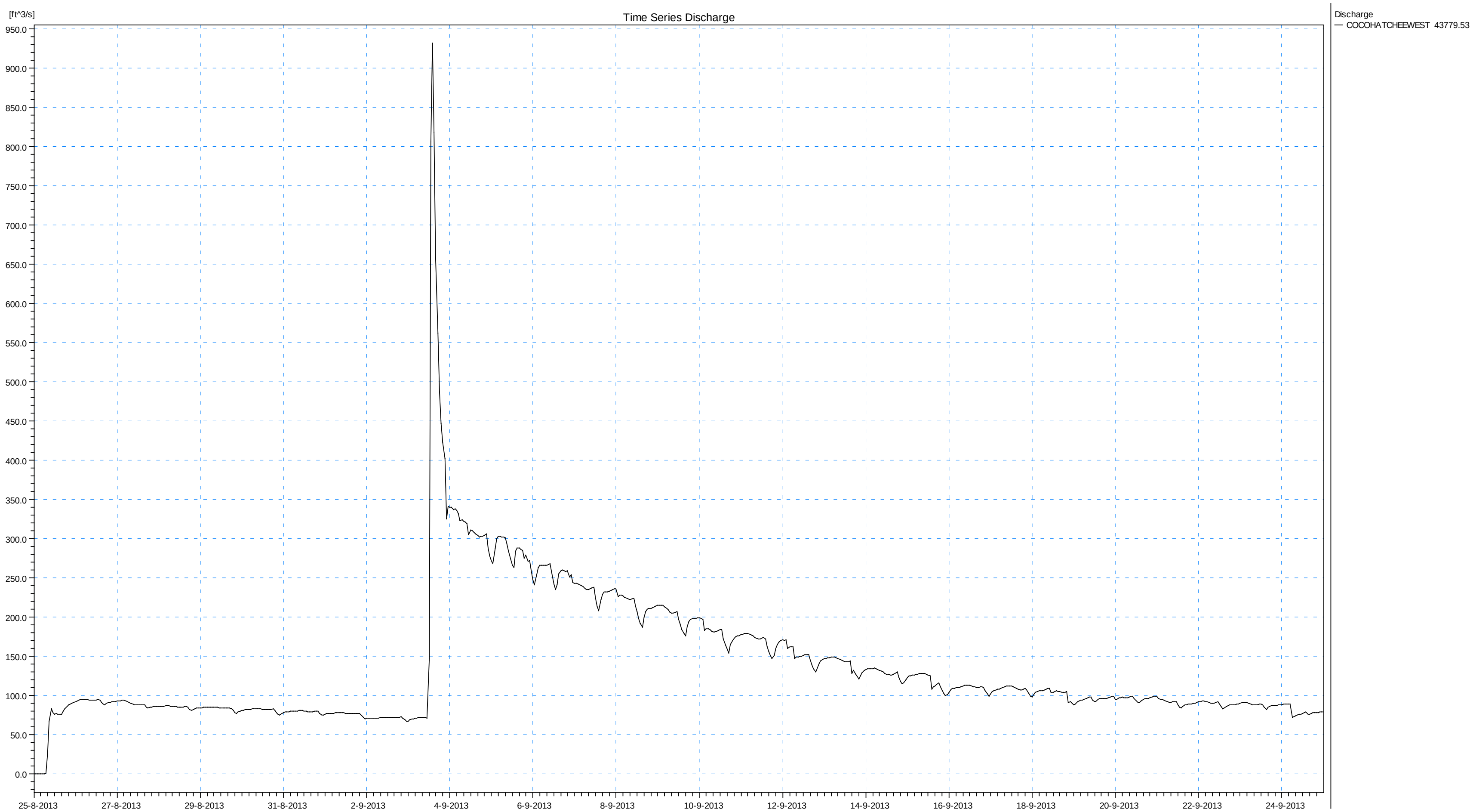

Figure 5. COCO2 Discharge (cfs)

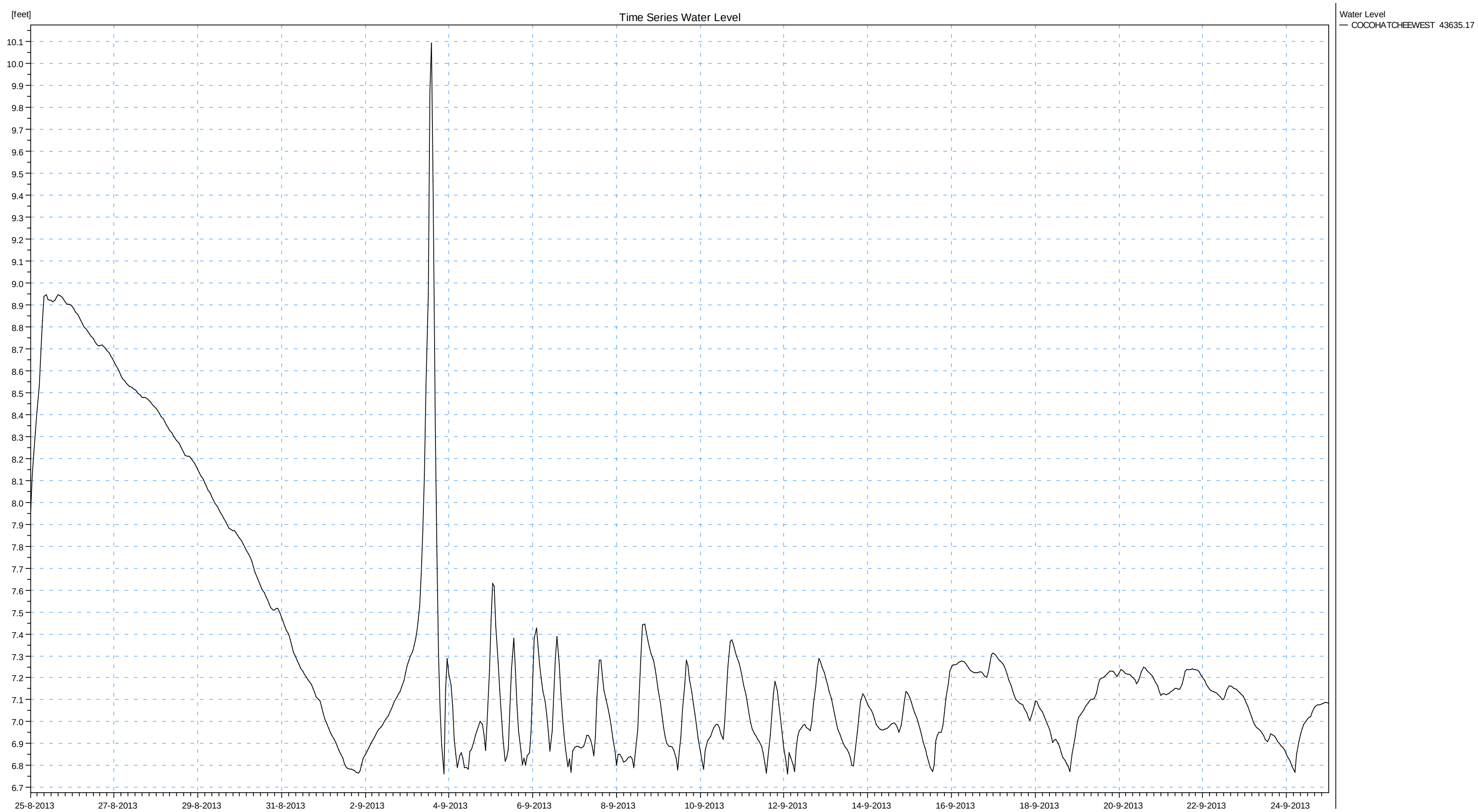

Figure 6. COCO2 Headwater Stage (ft)

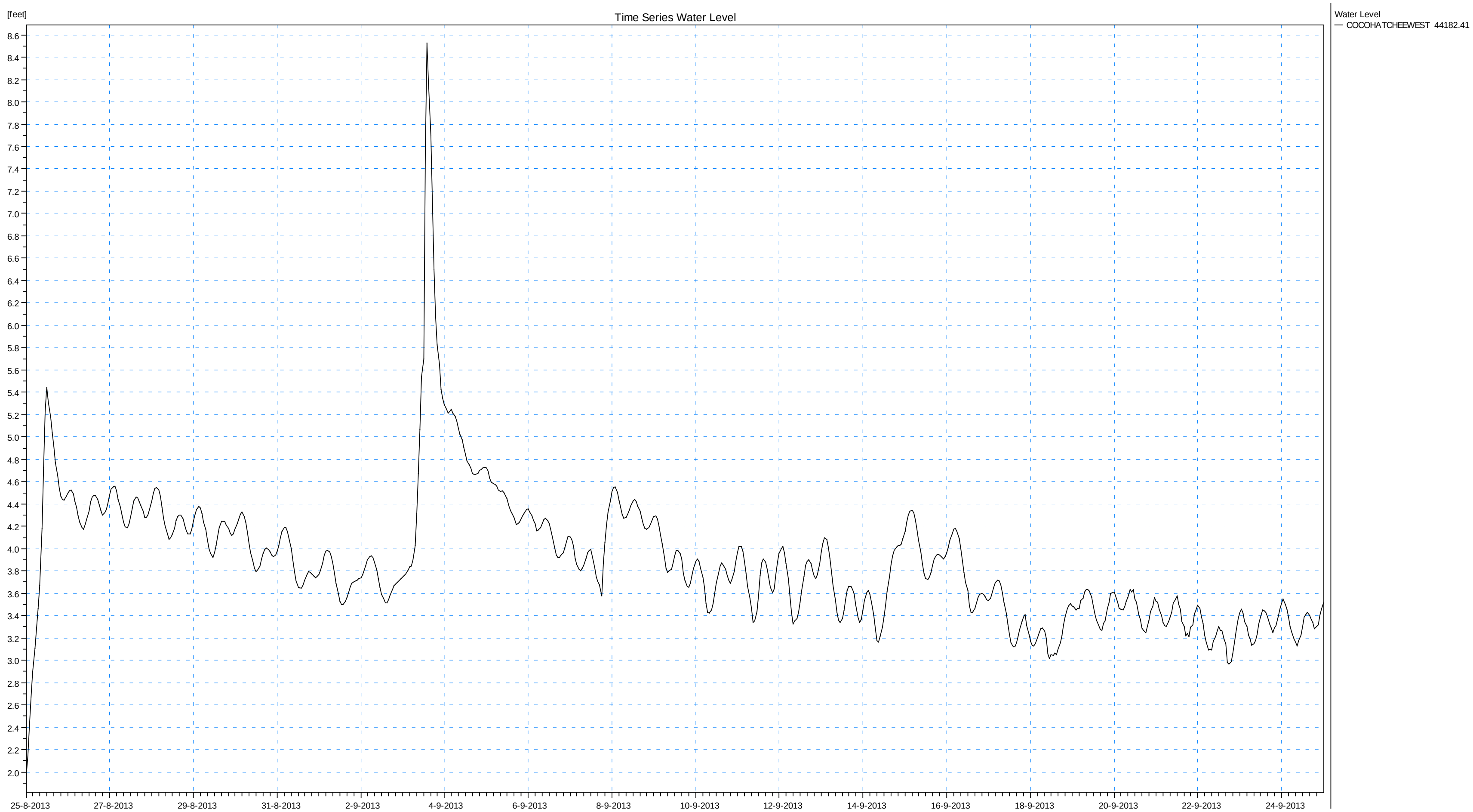

Figure 7. COCO2 Tailwater Stage (ft)

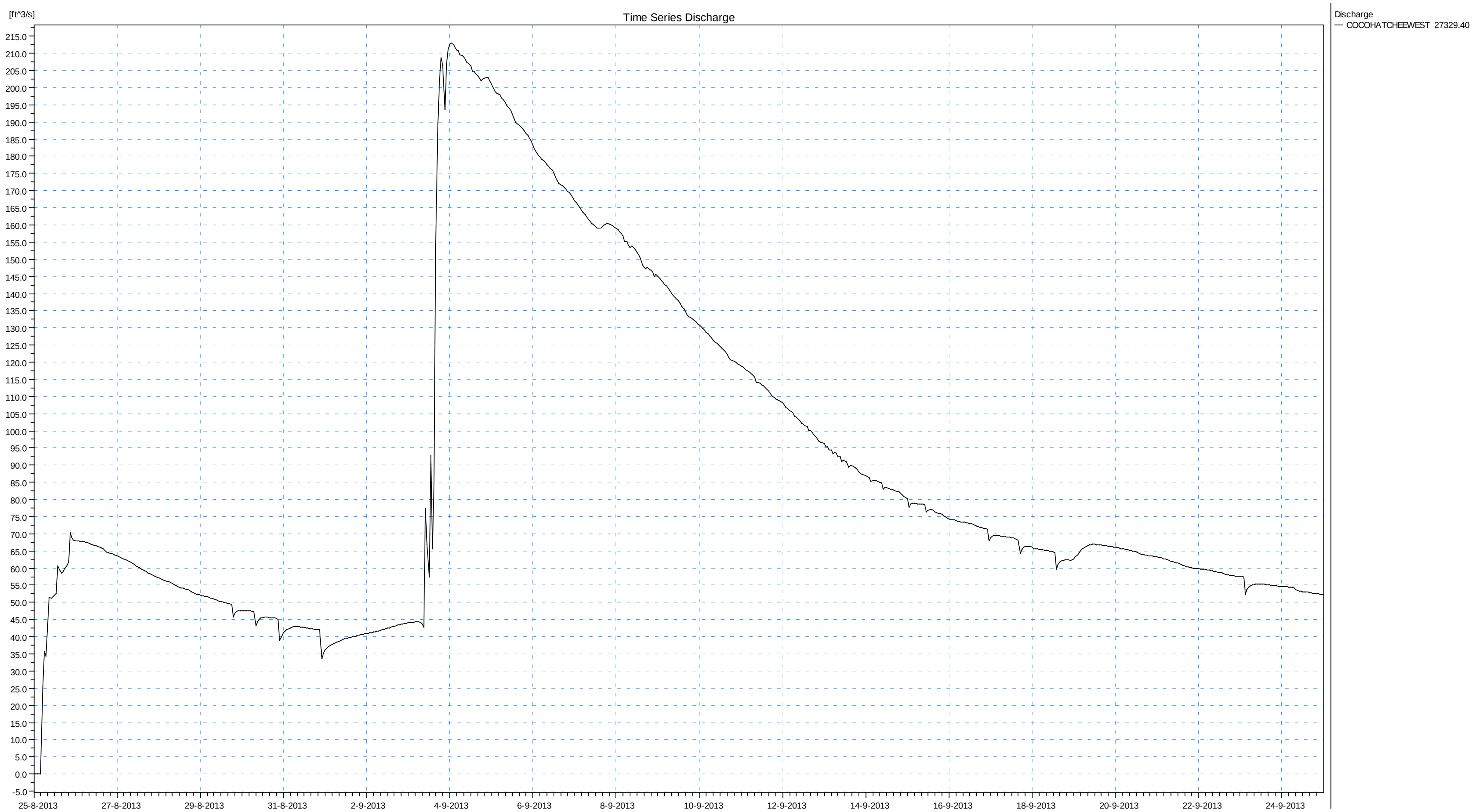

Figure 8. COCO3 Discharge (cfs)

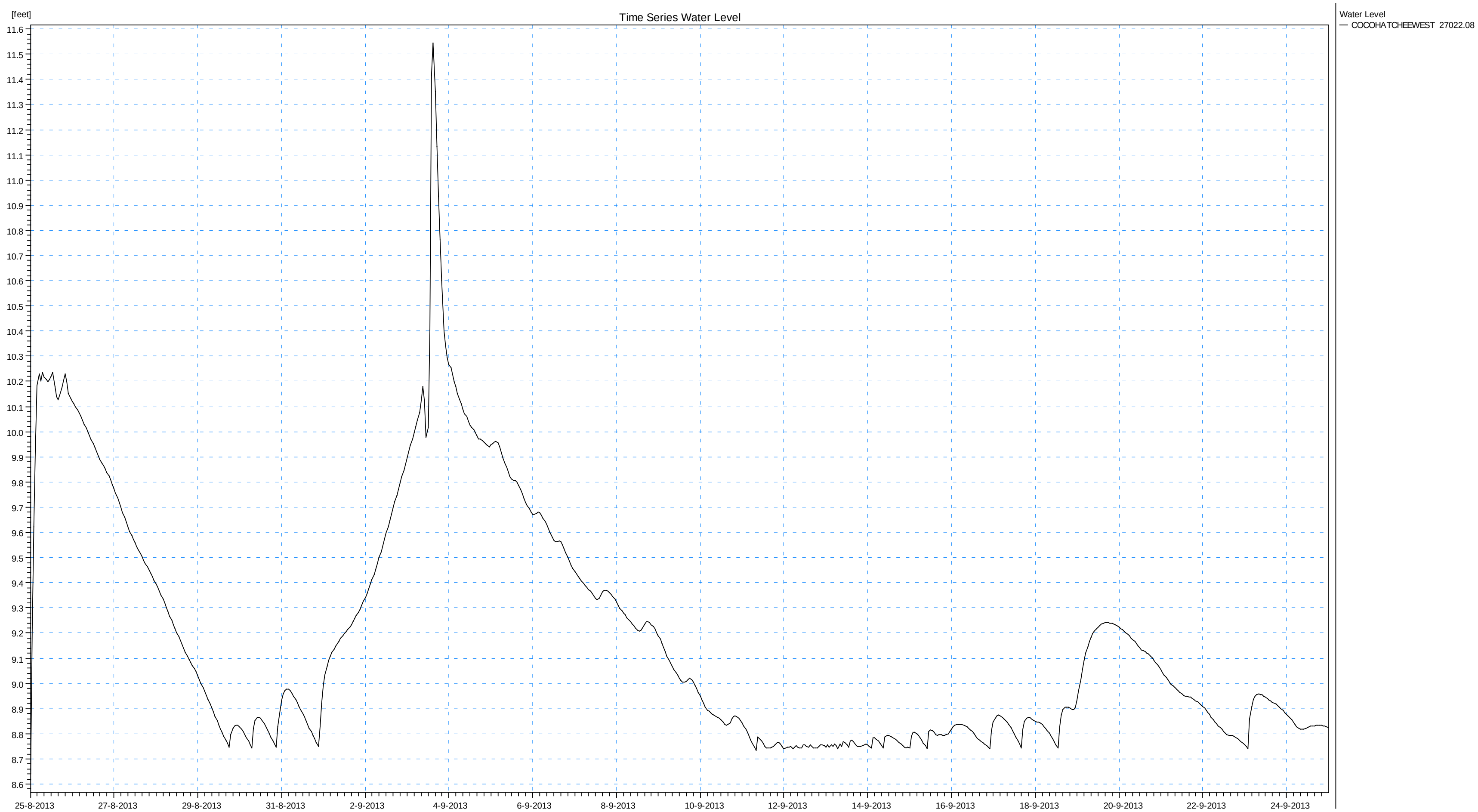

Figure 9. COCO3 Headwater Stage (ft)

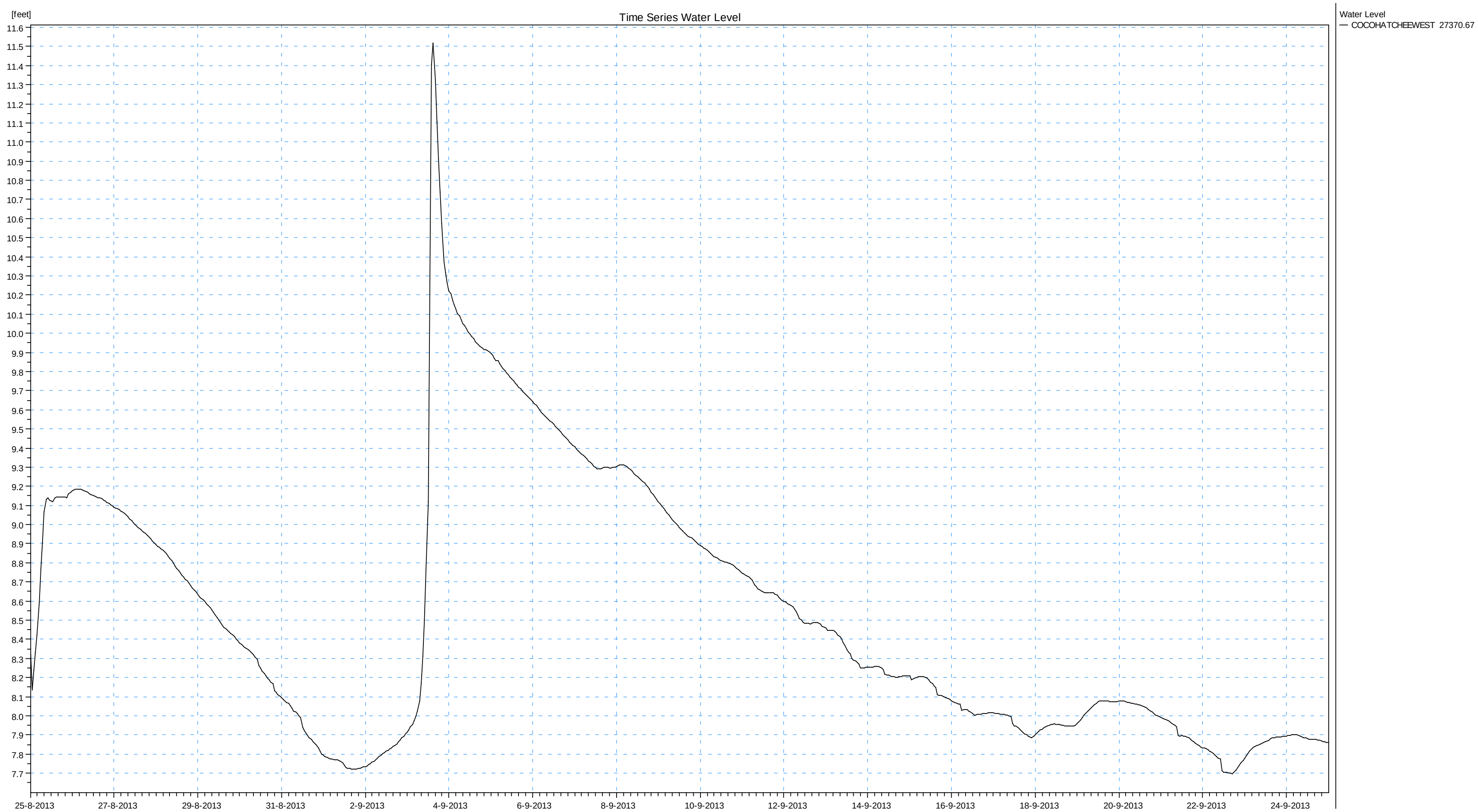

Figure 10. COCO3 Tailwater Stage (ft)

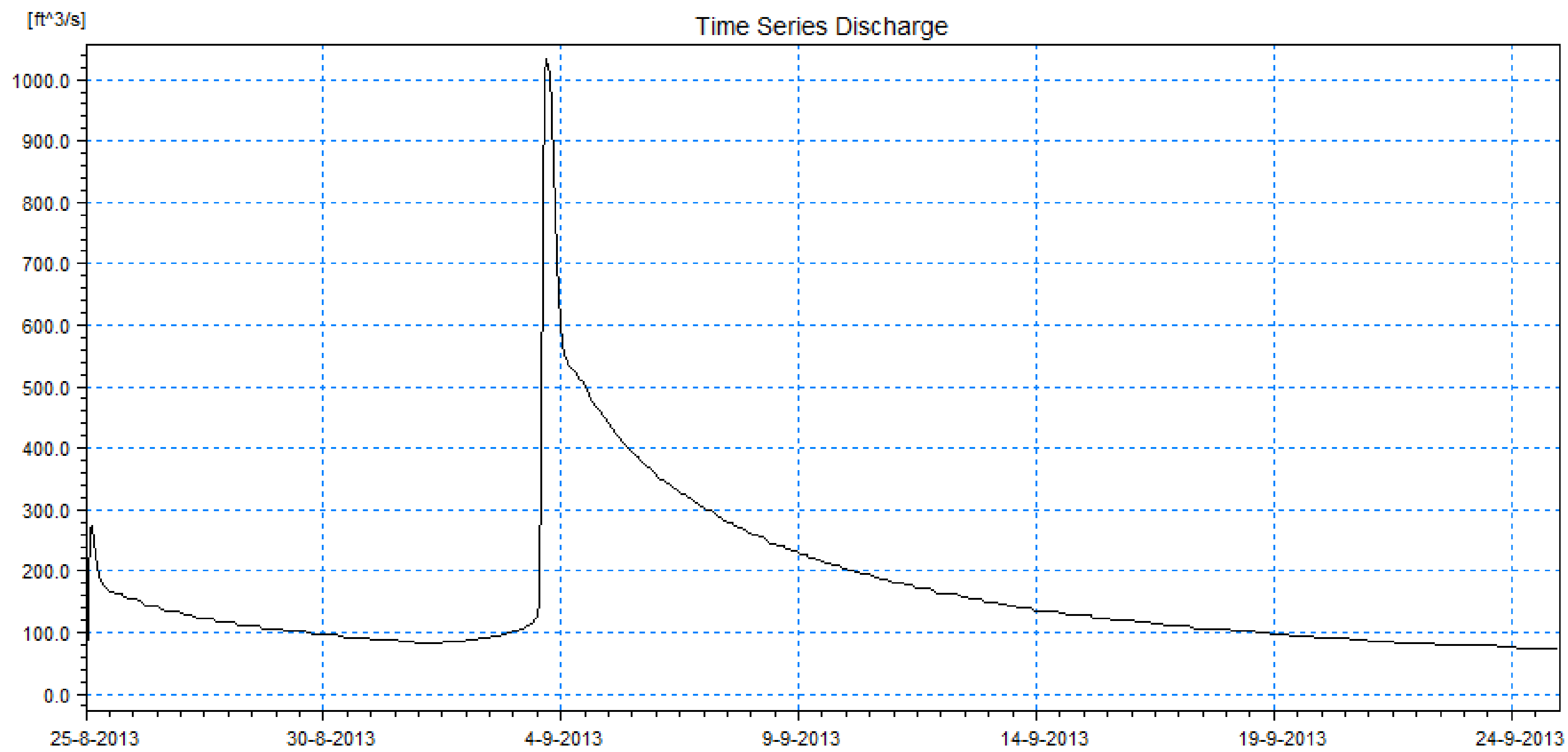

Figure 11. FU3 Discharge (cfs)

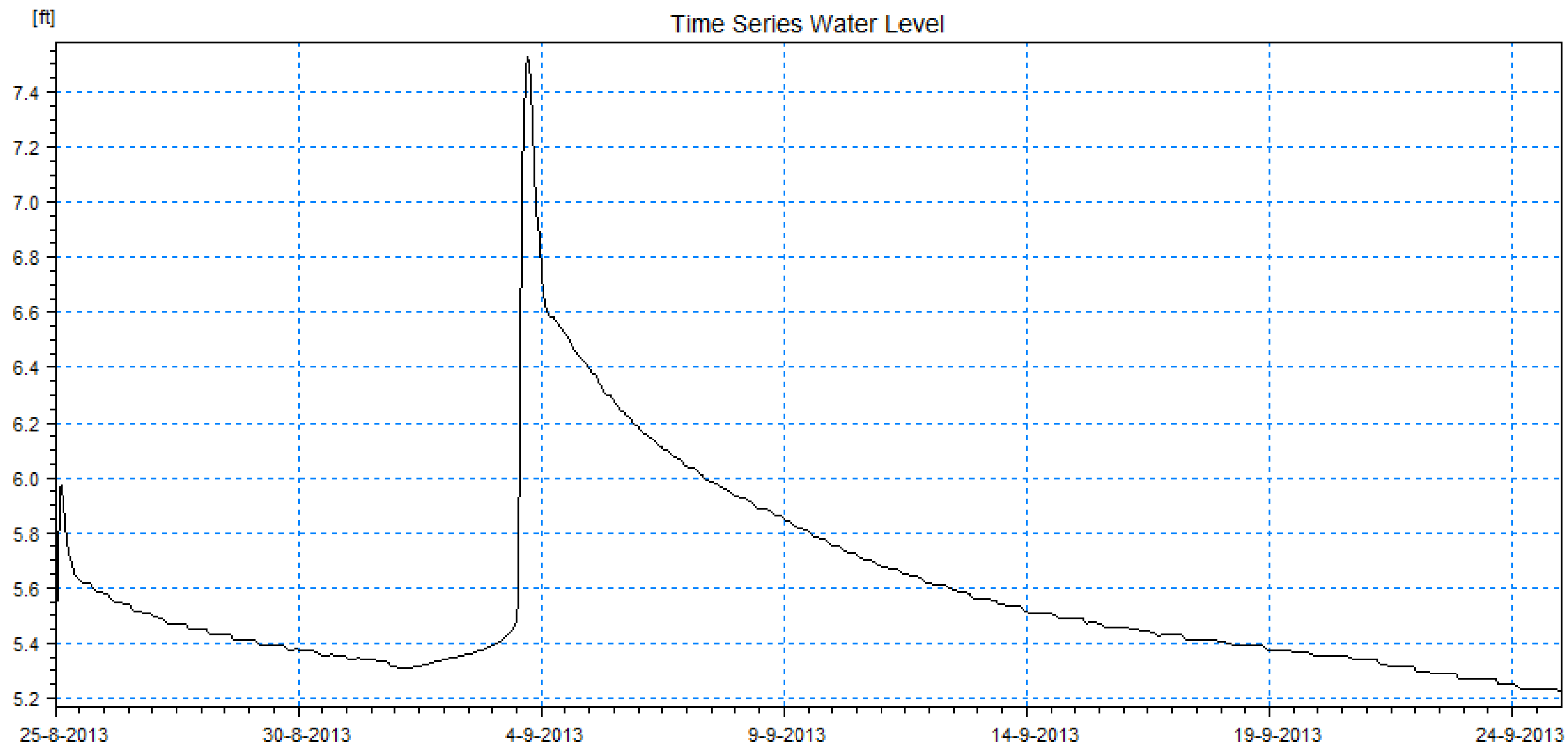

Figure 12. FU3 Headwater Stage (ft)

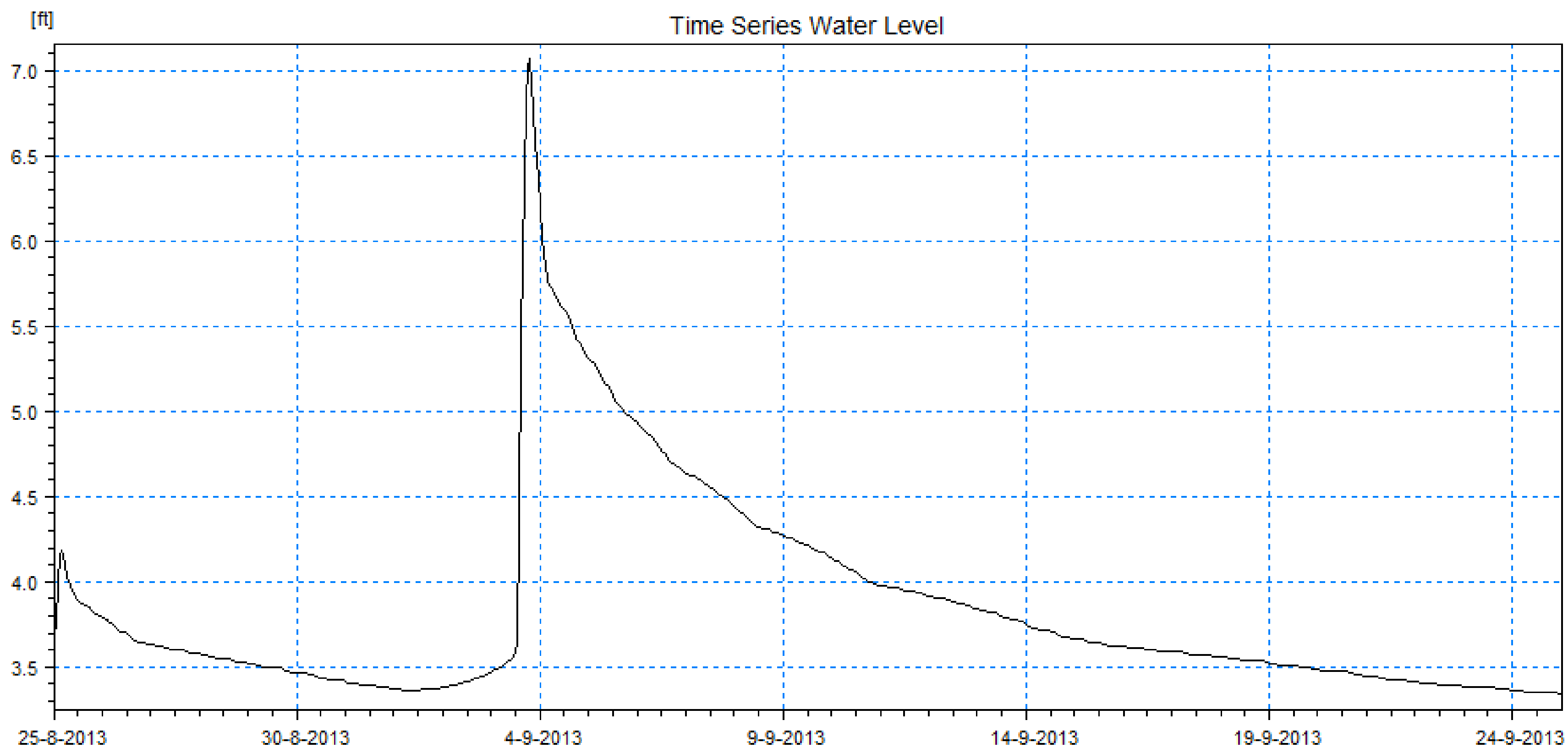

Figure 13. FU3 Tailwater Stage (ft)

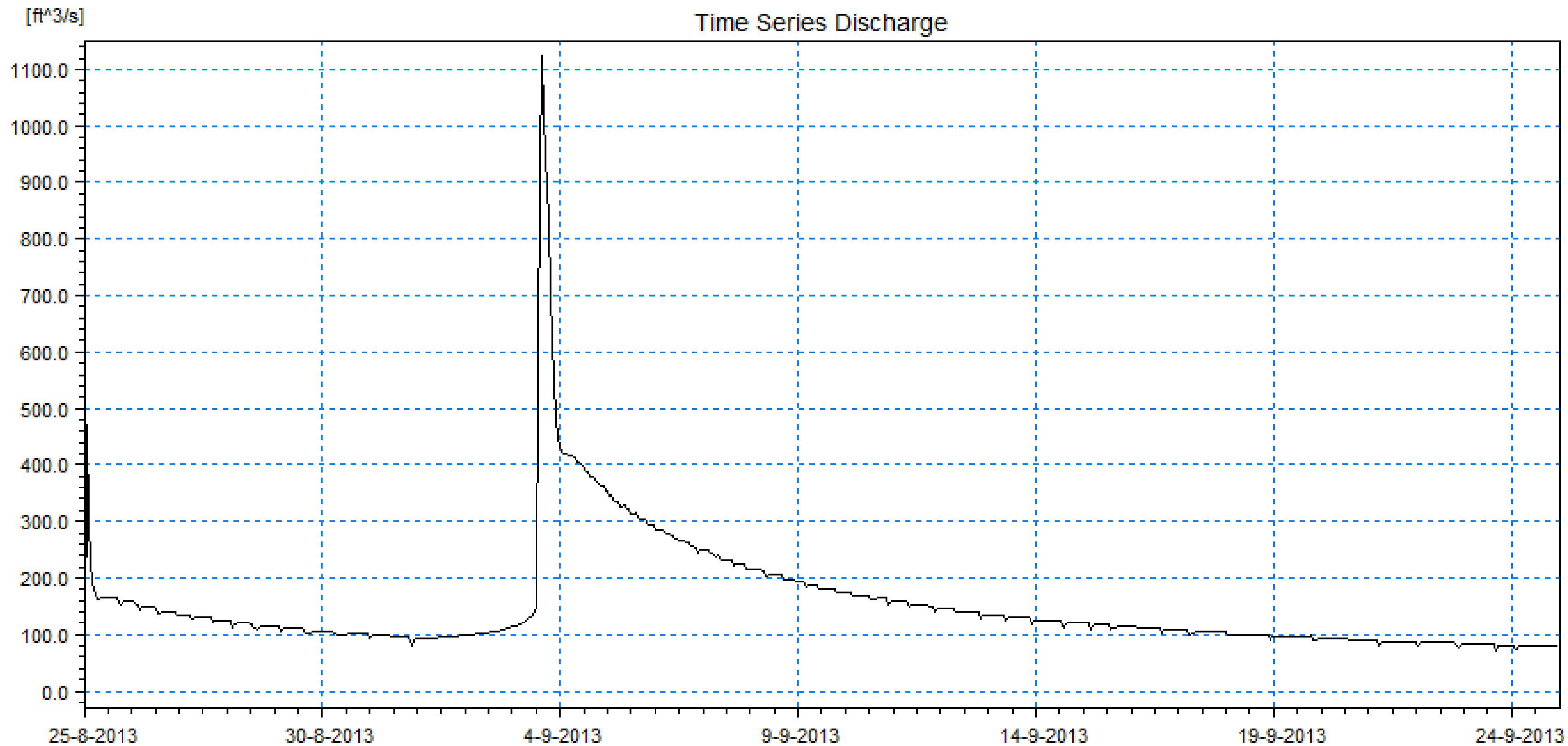

Figure 14. FU4 Discharge (cfs)

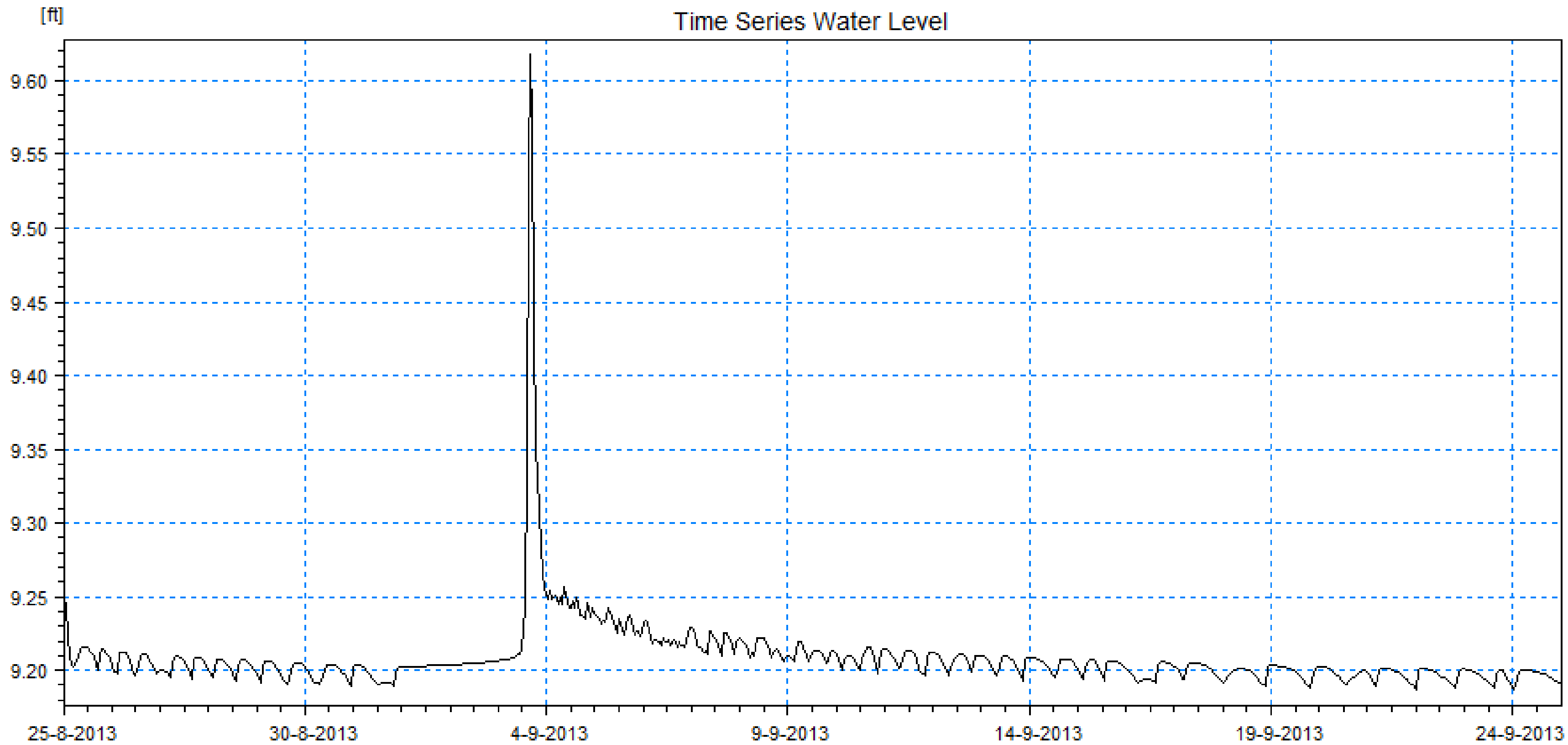

Figure 15. FU4 Headwater Stage (ft)

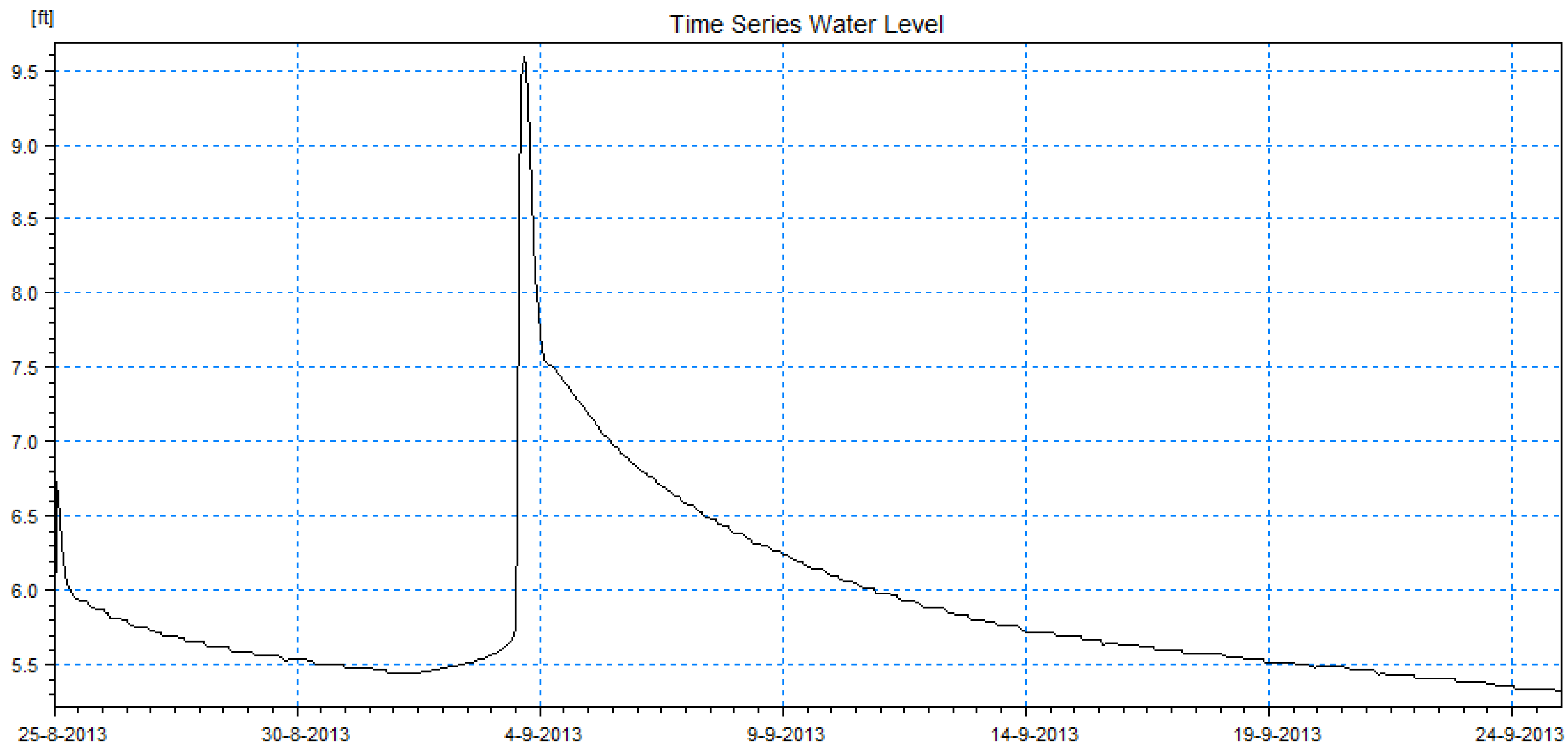

Figure 16. FU4 Tailwater Stage (ft)

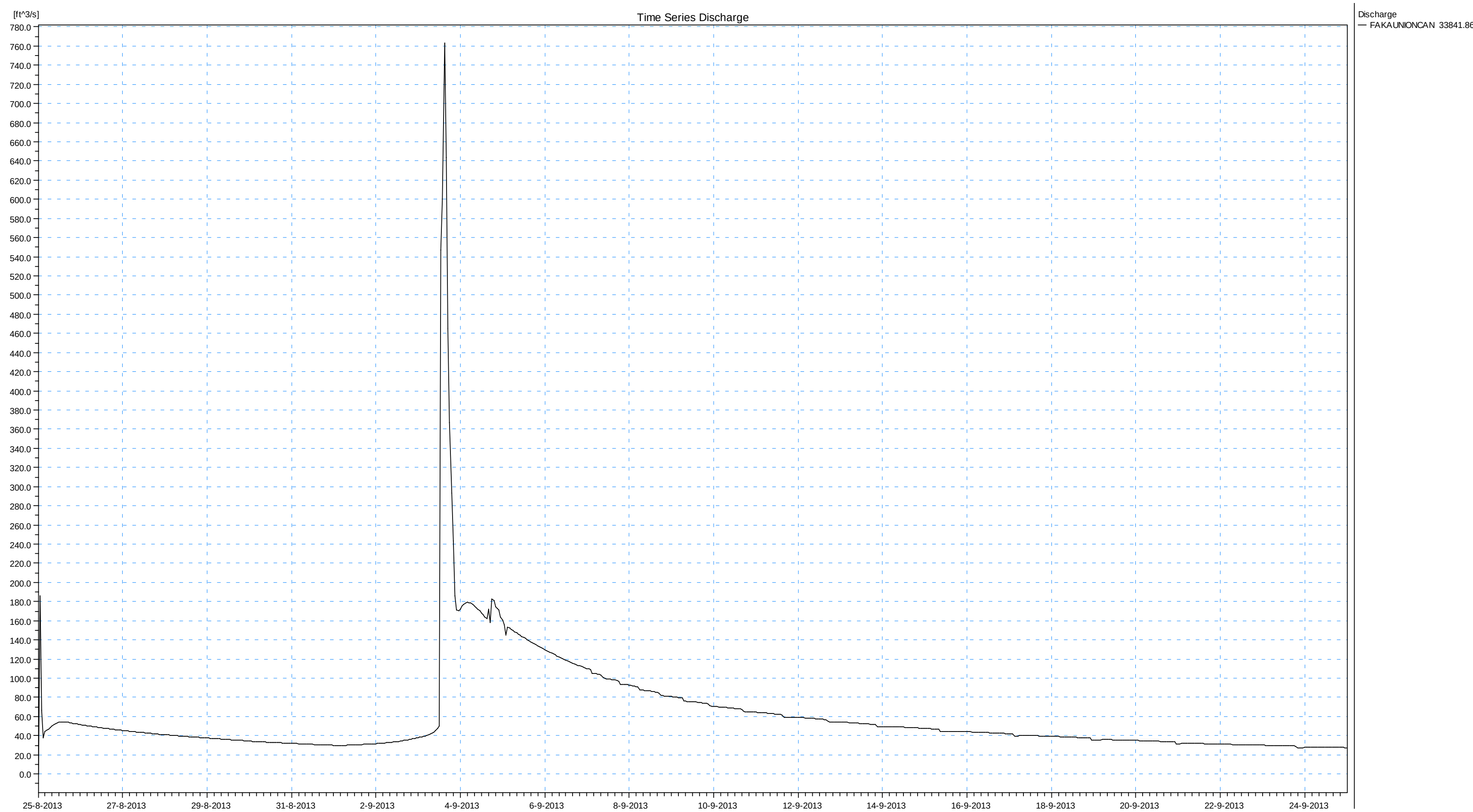

Figure 17. FU5 Discharge (cfs)

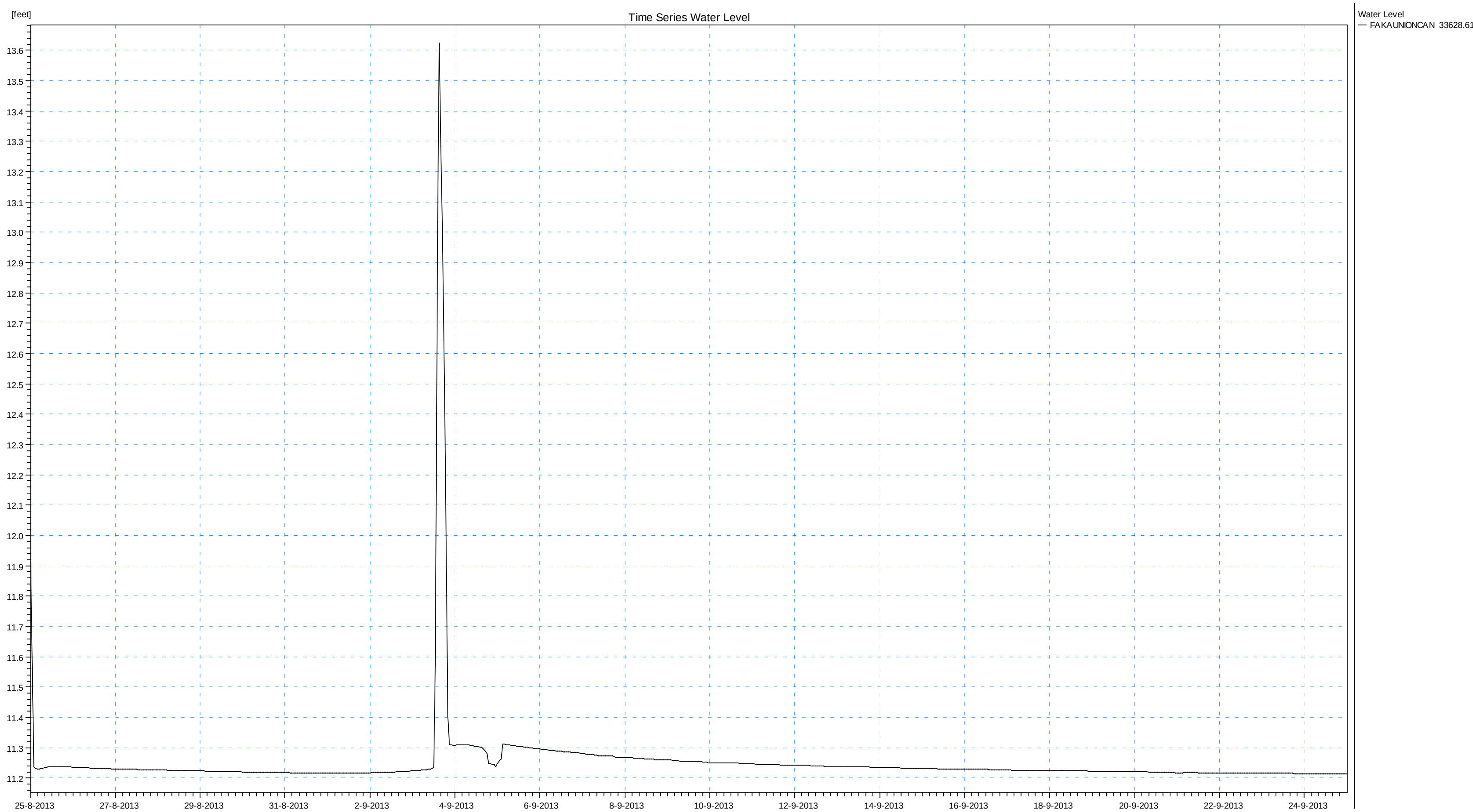

Figure 18. FU5 Headwater Stage (ft)

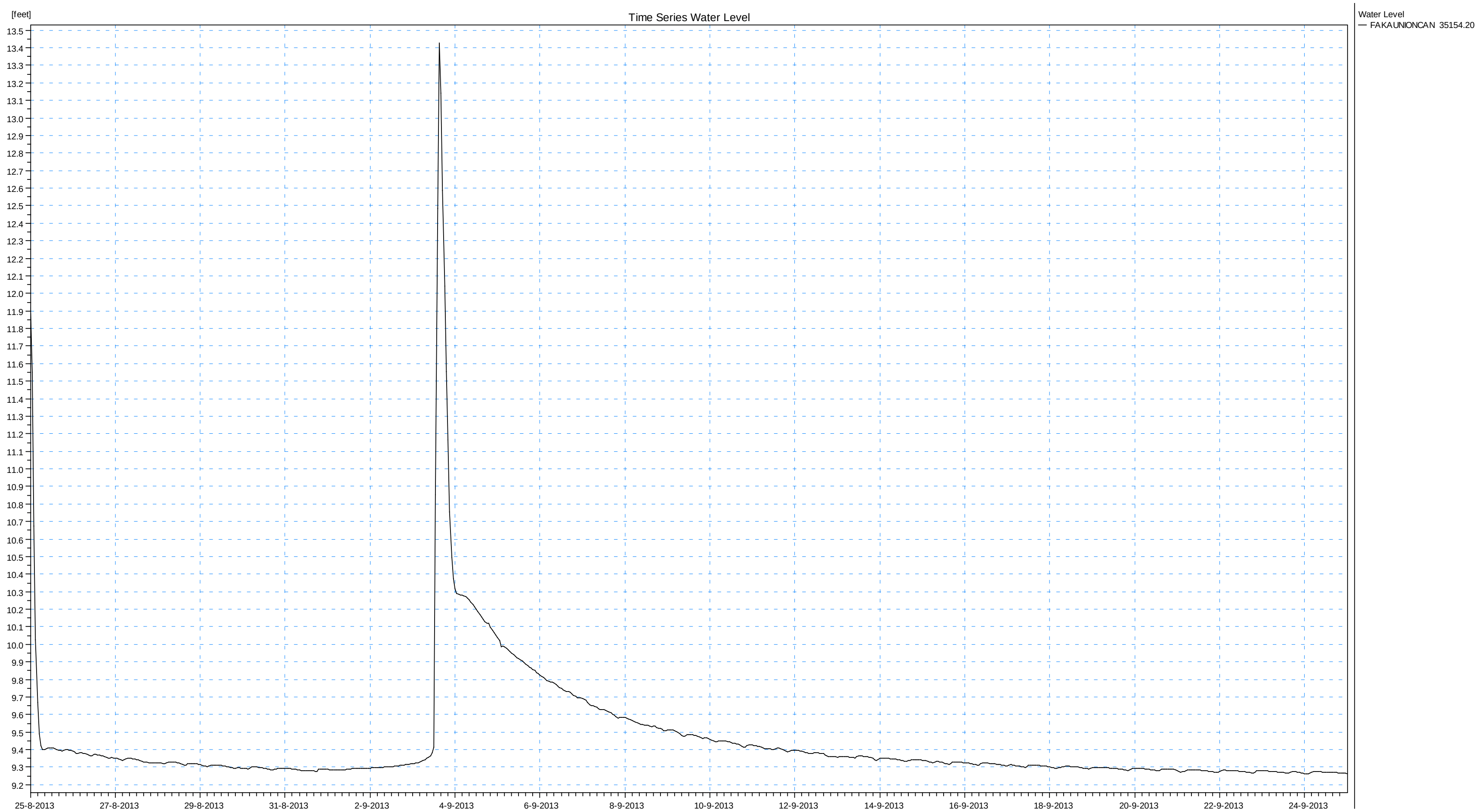

Figure 19. FU5 Tailwater Stage (ft)

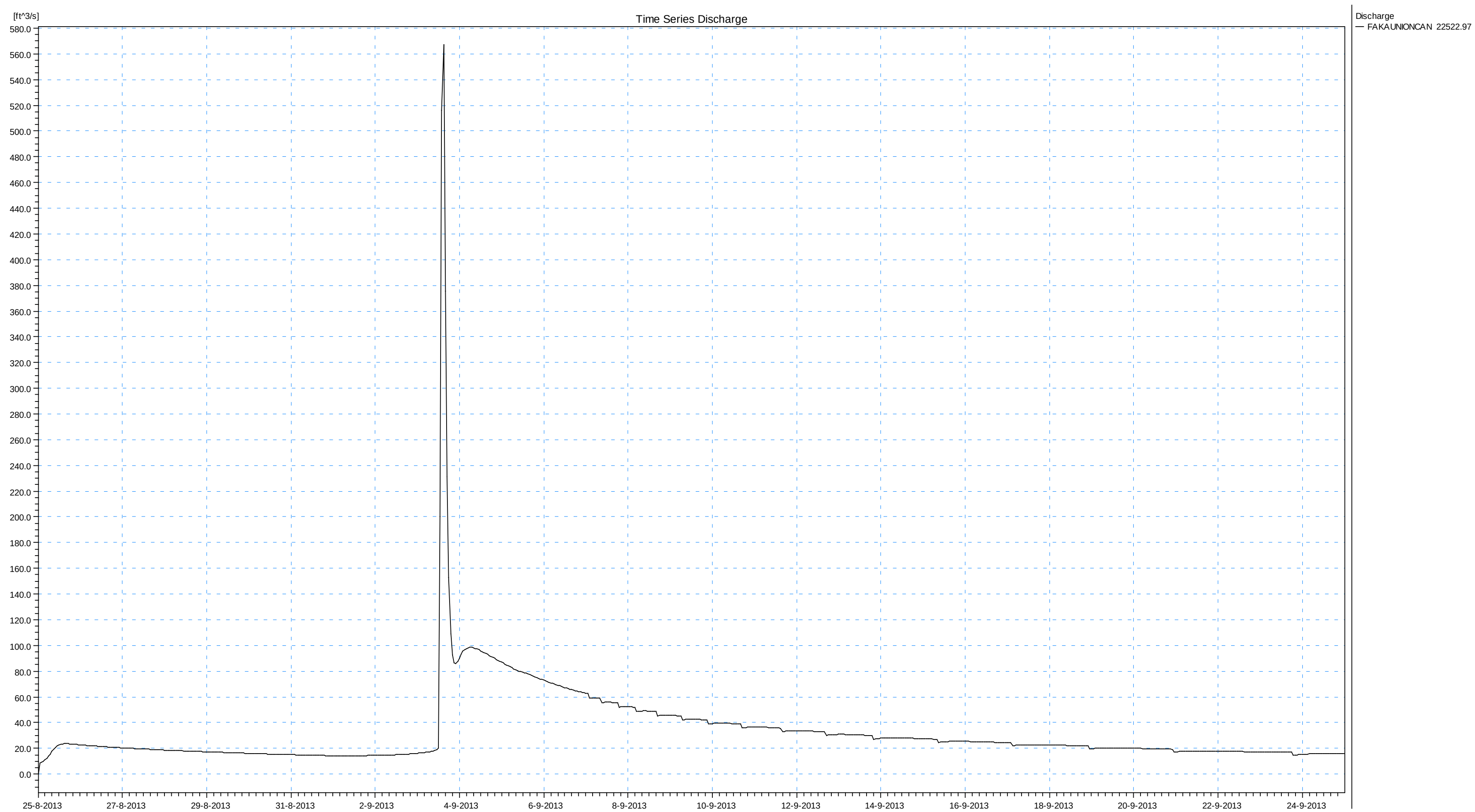

Figure 20. FU6 Discharge (cfs)

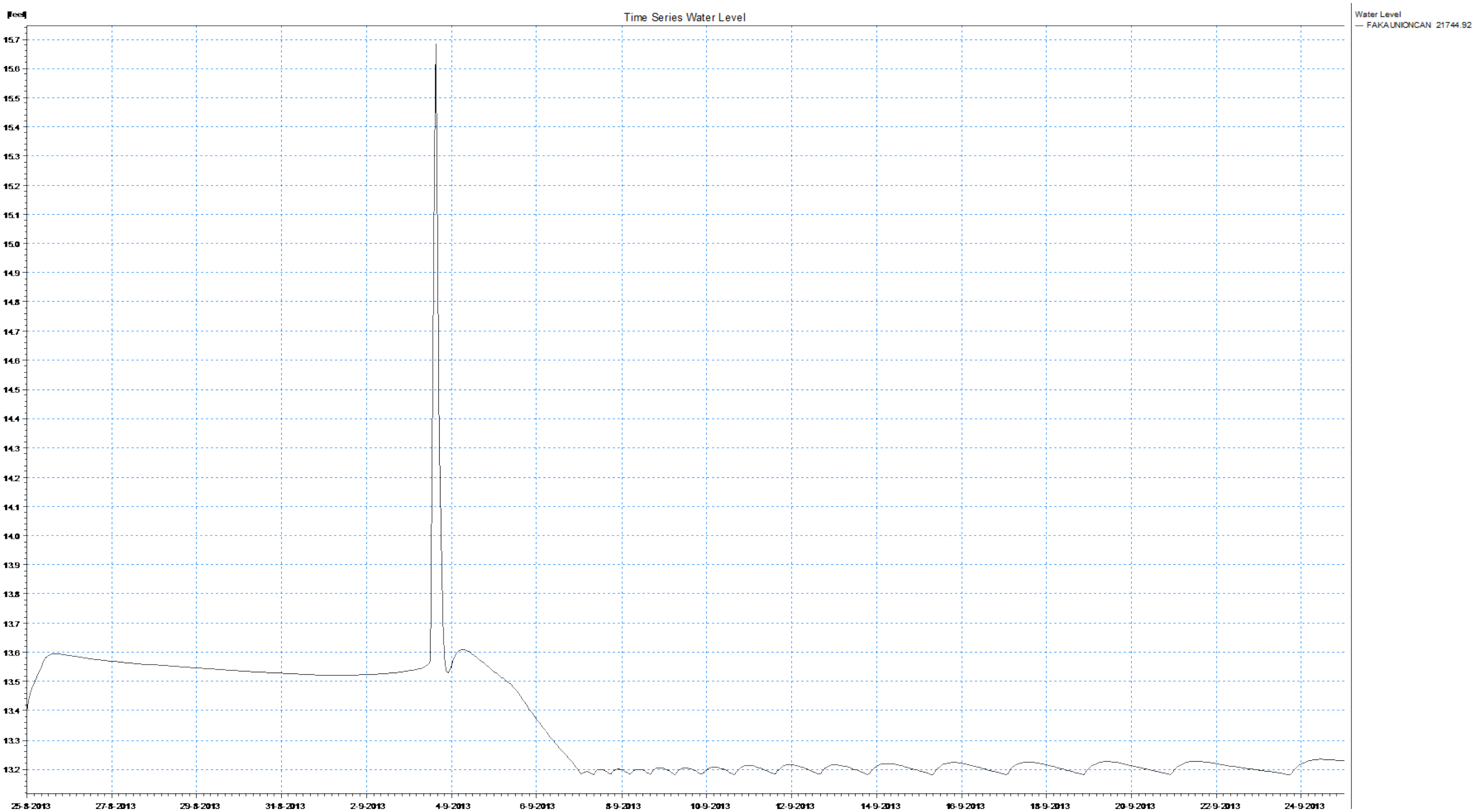

Figure 21. FU6 Headwater Stage (ft)

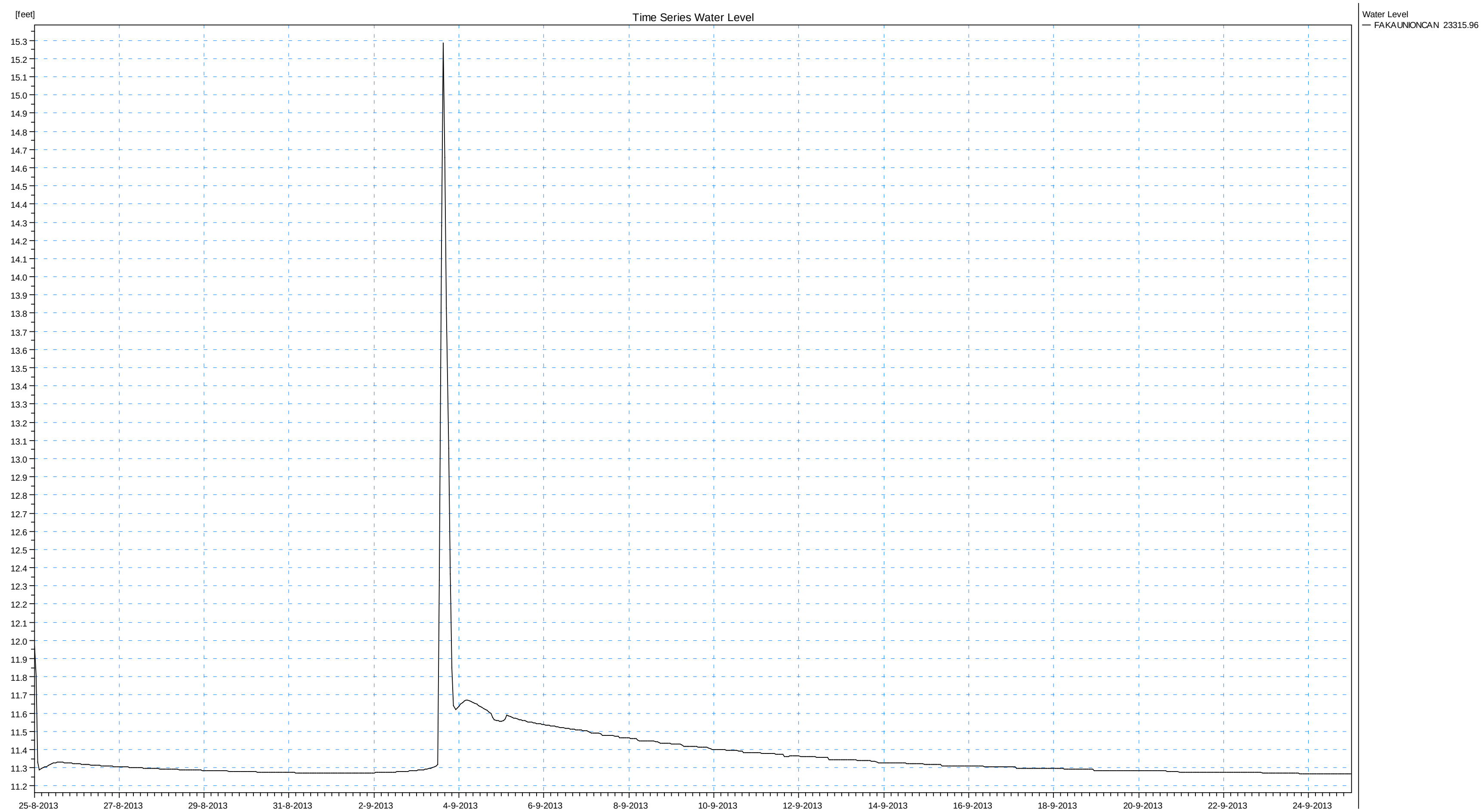

Figure 22. FU6 Tailwater Stage (ft)

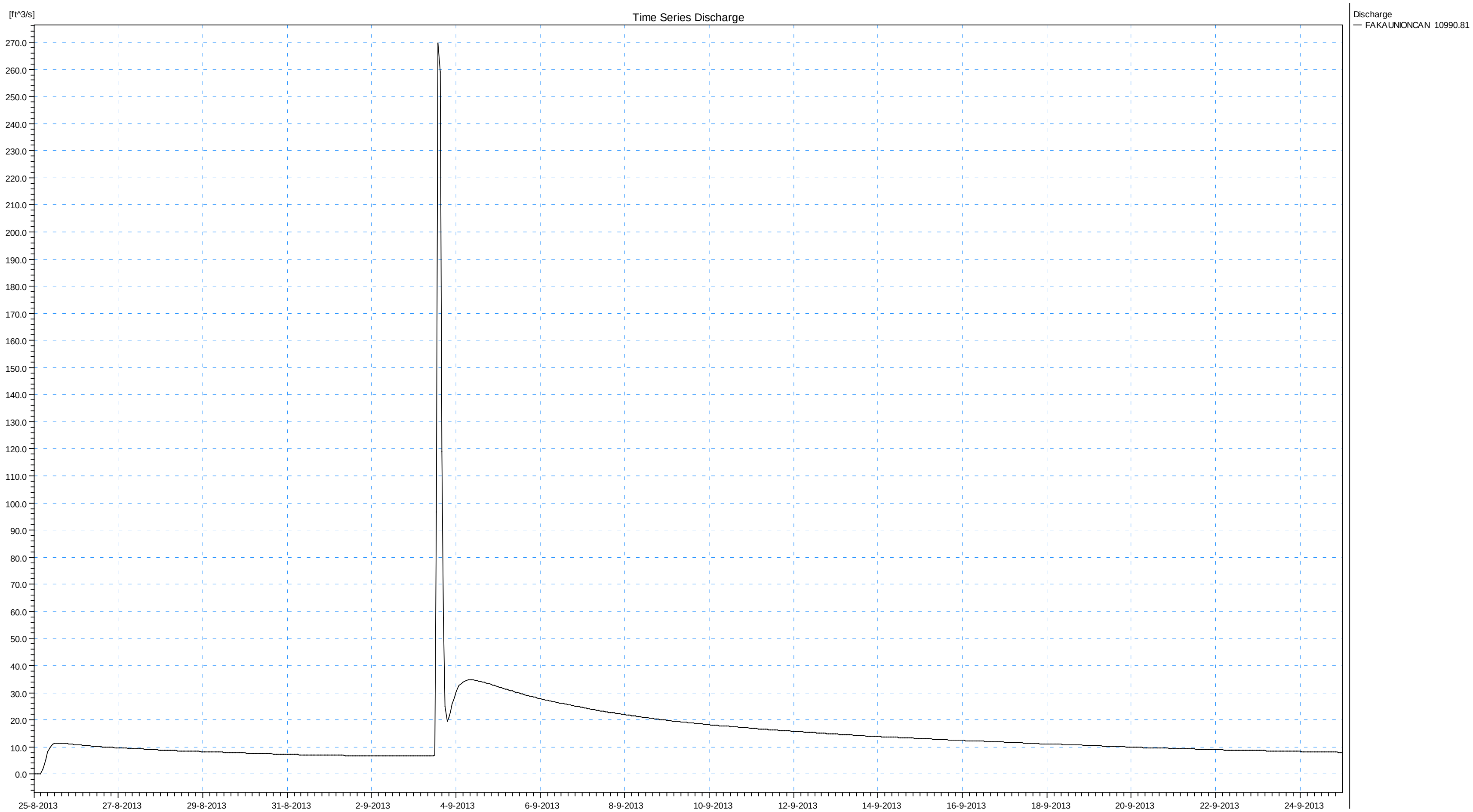

Figure 23. FU7 Discharge (cfs)

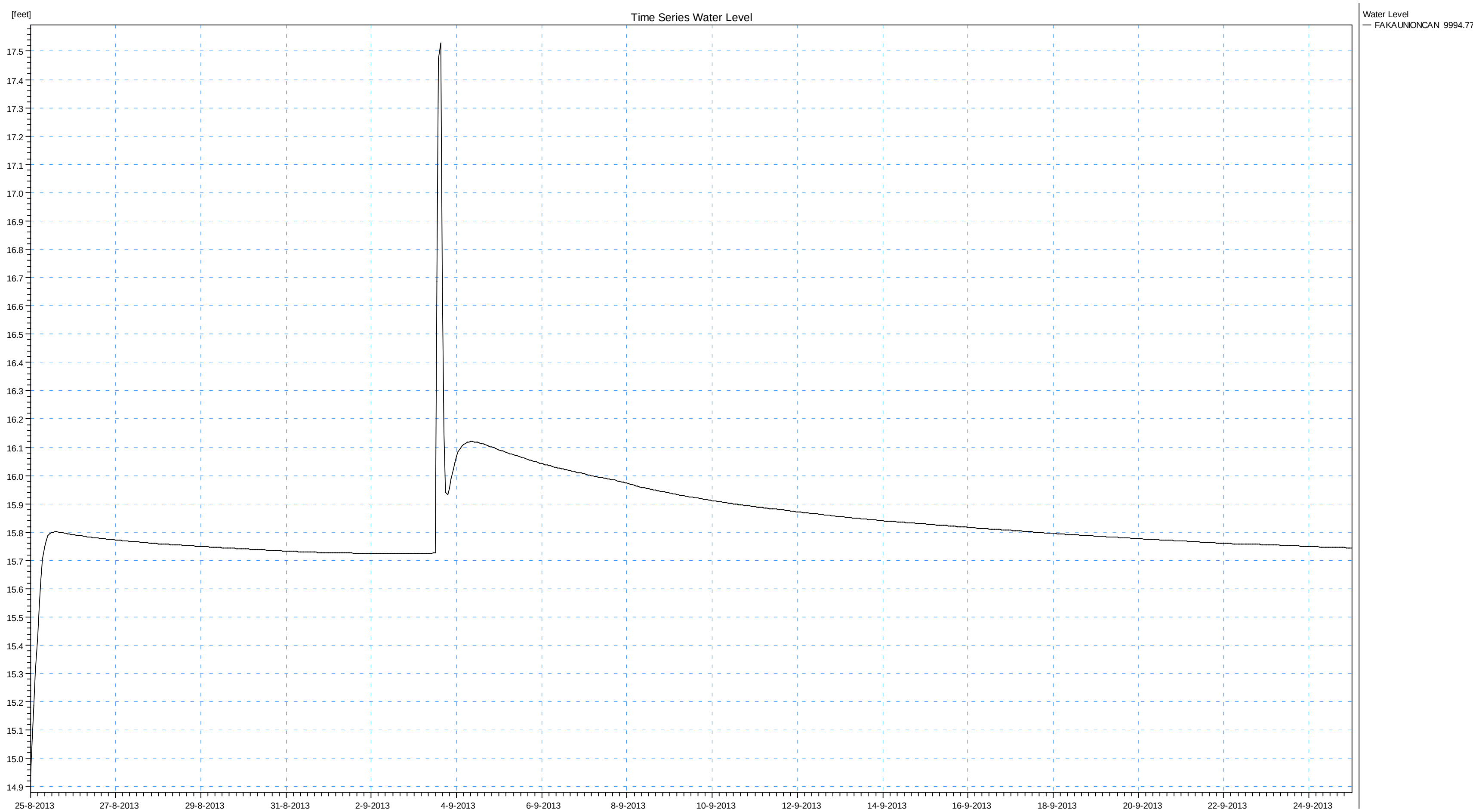

Figure 24. FU7 Headwater Stage (ft)

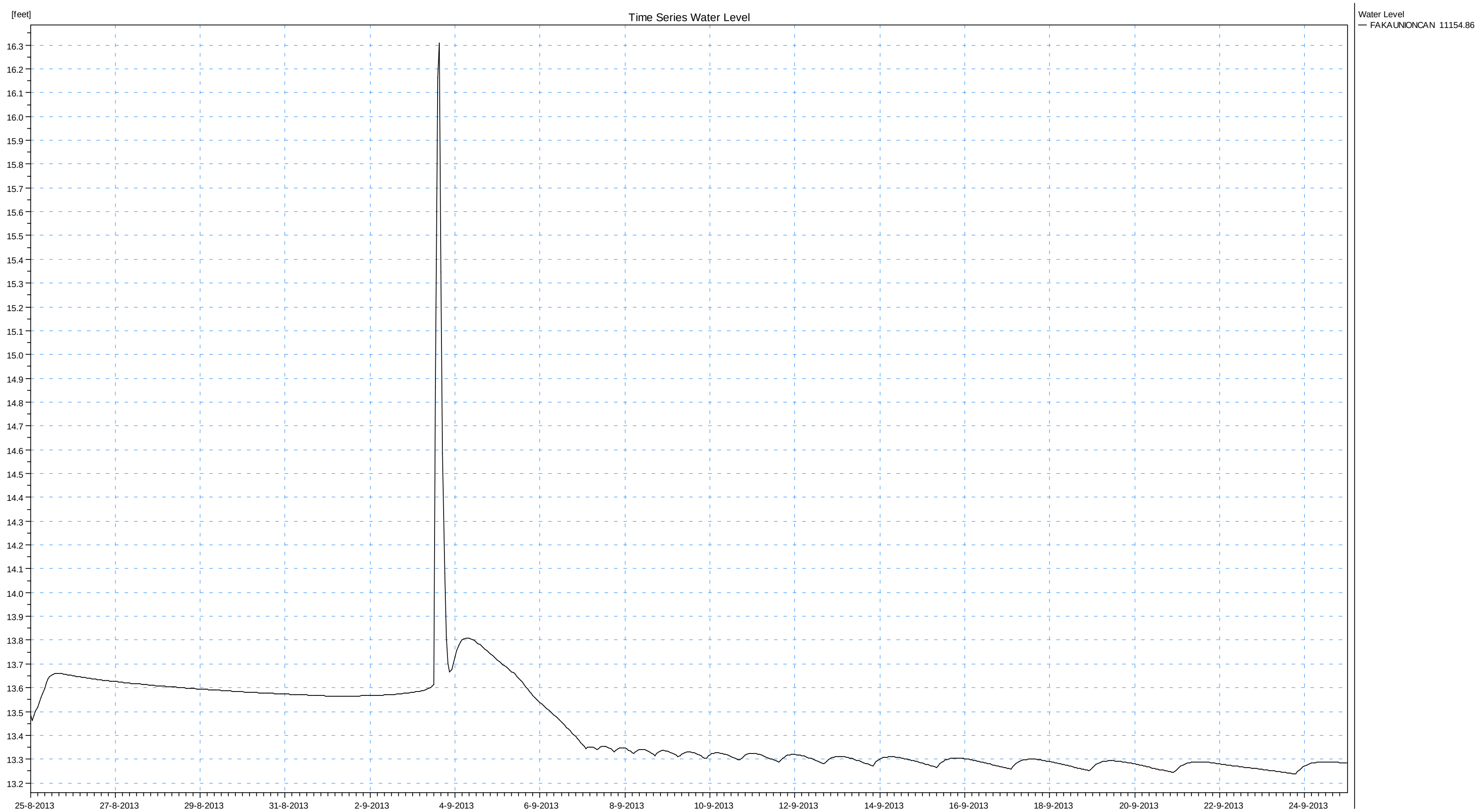

Figure 25. FU7 Tailwater Stage (ft)

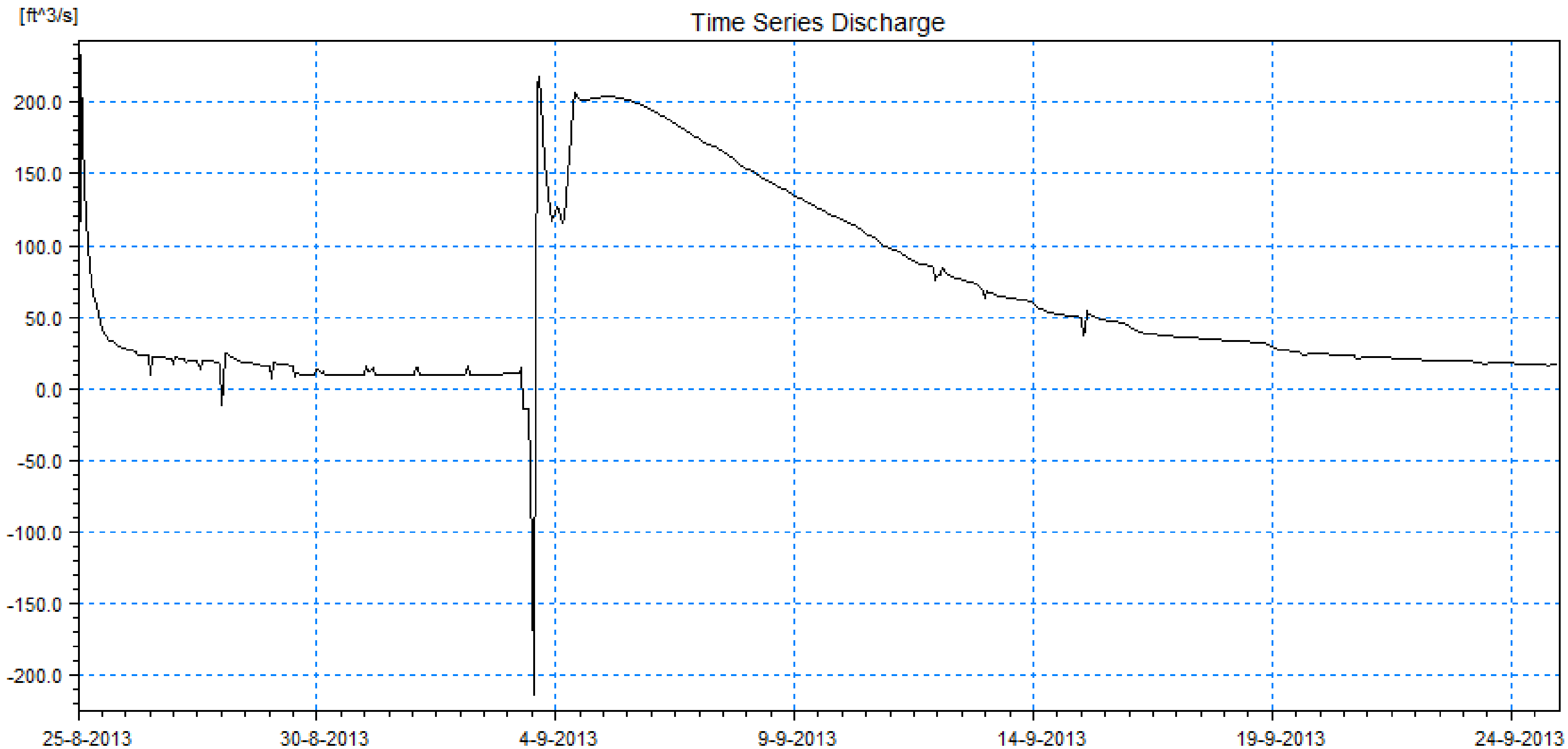

Figure 26. HC1 Discharge (cfs)

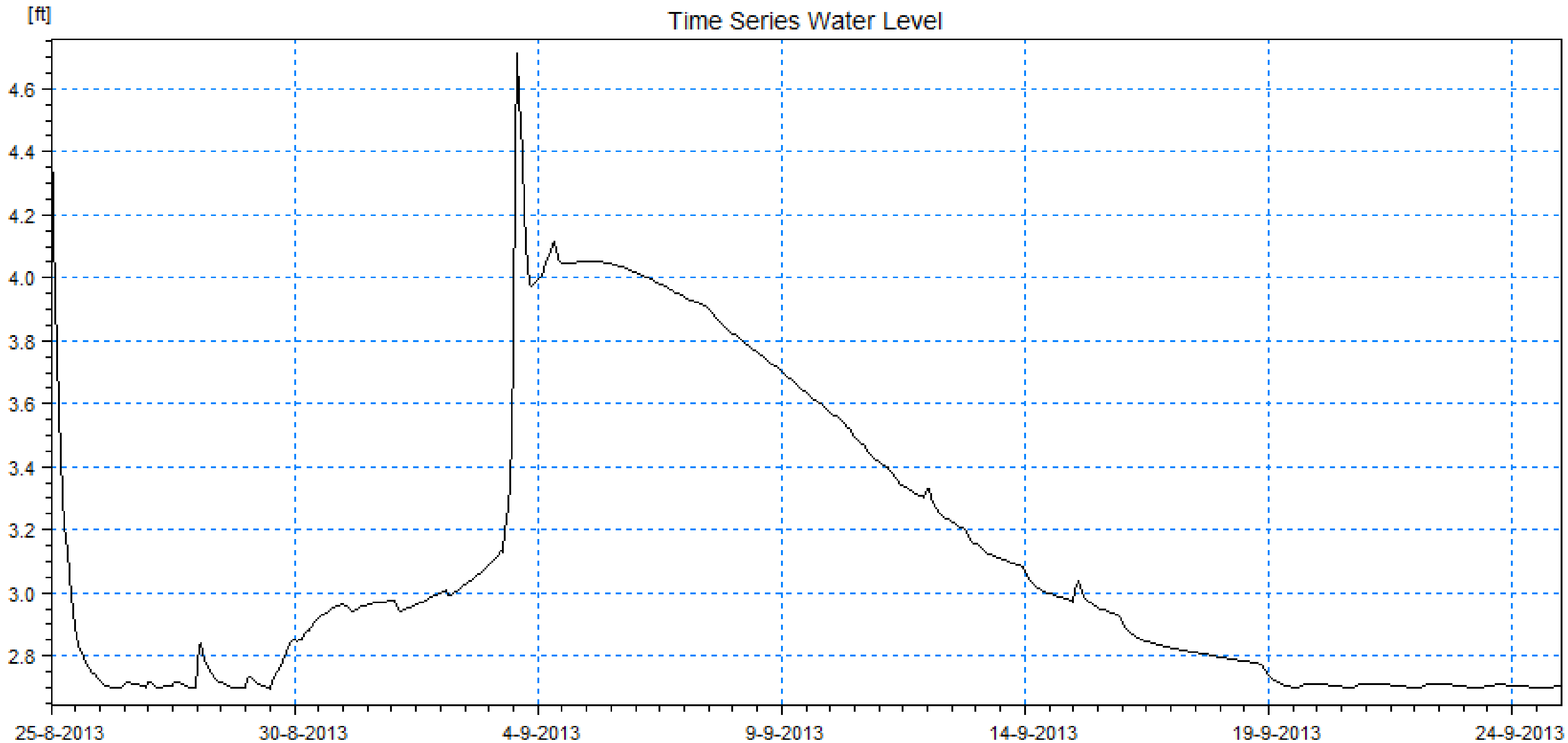

Figure 27. HC1 Headwater Stage (ft)

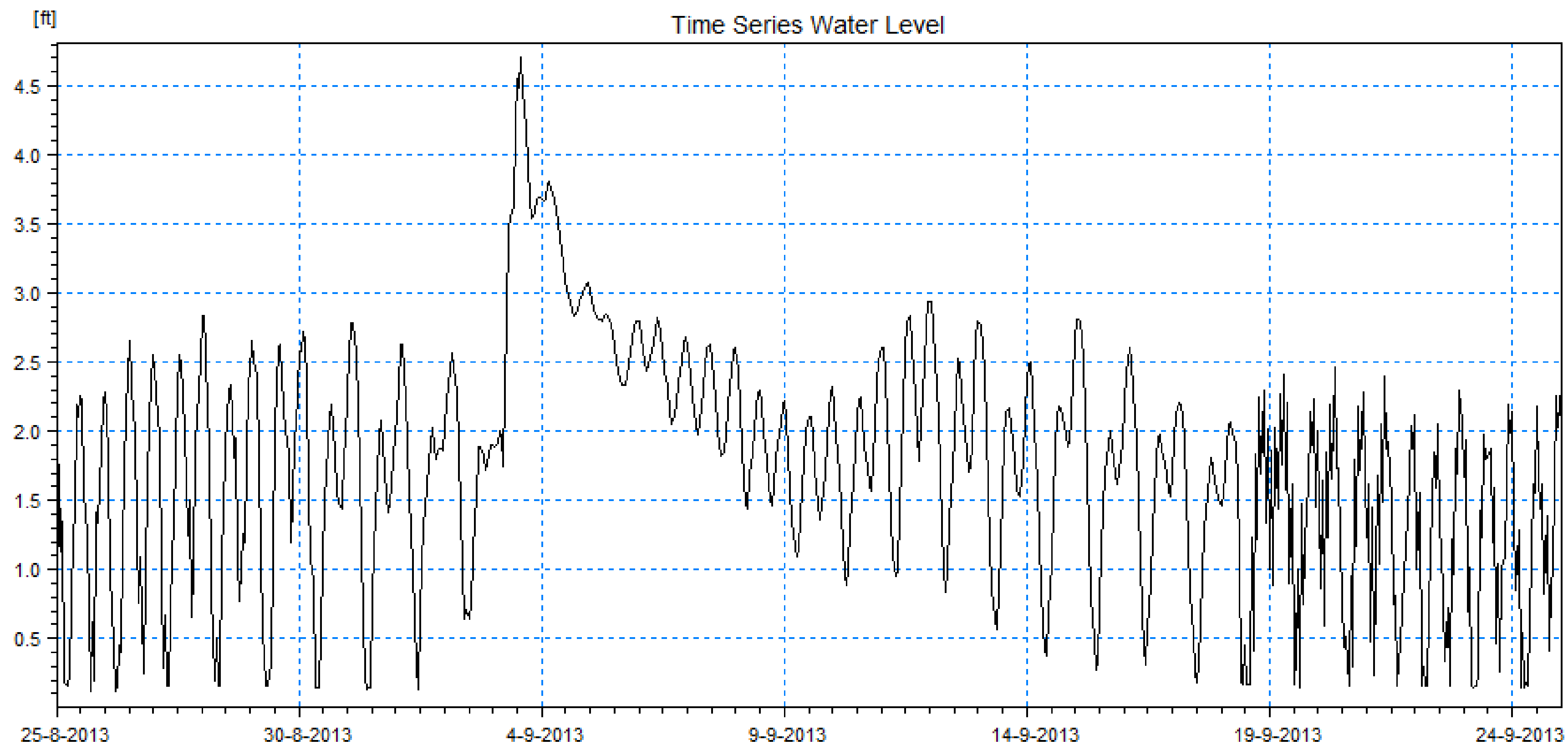

Figure 28. HC1 Tailwater Stage (ft)

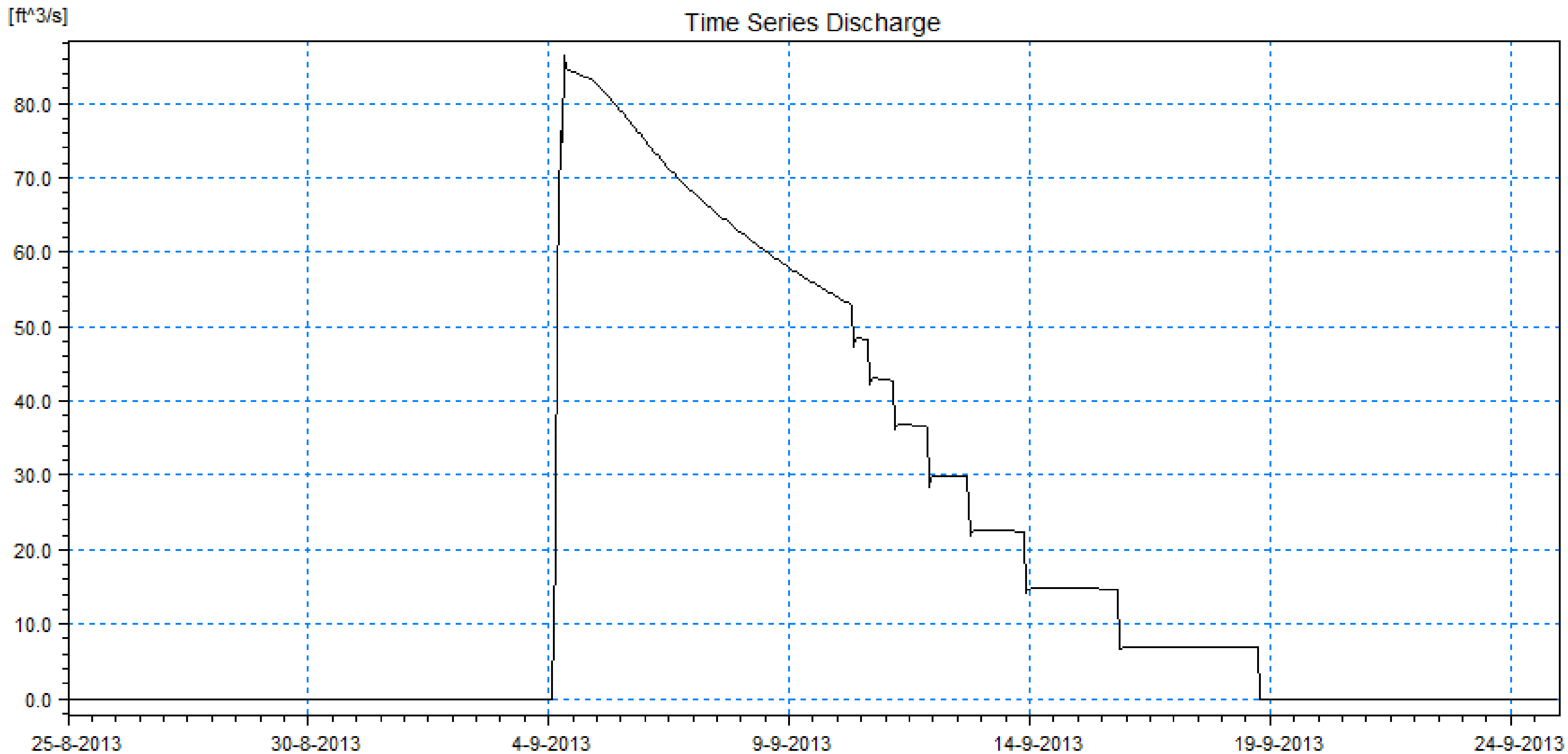

Figure 29. HC2 Discharge (cfs)

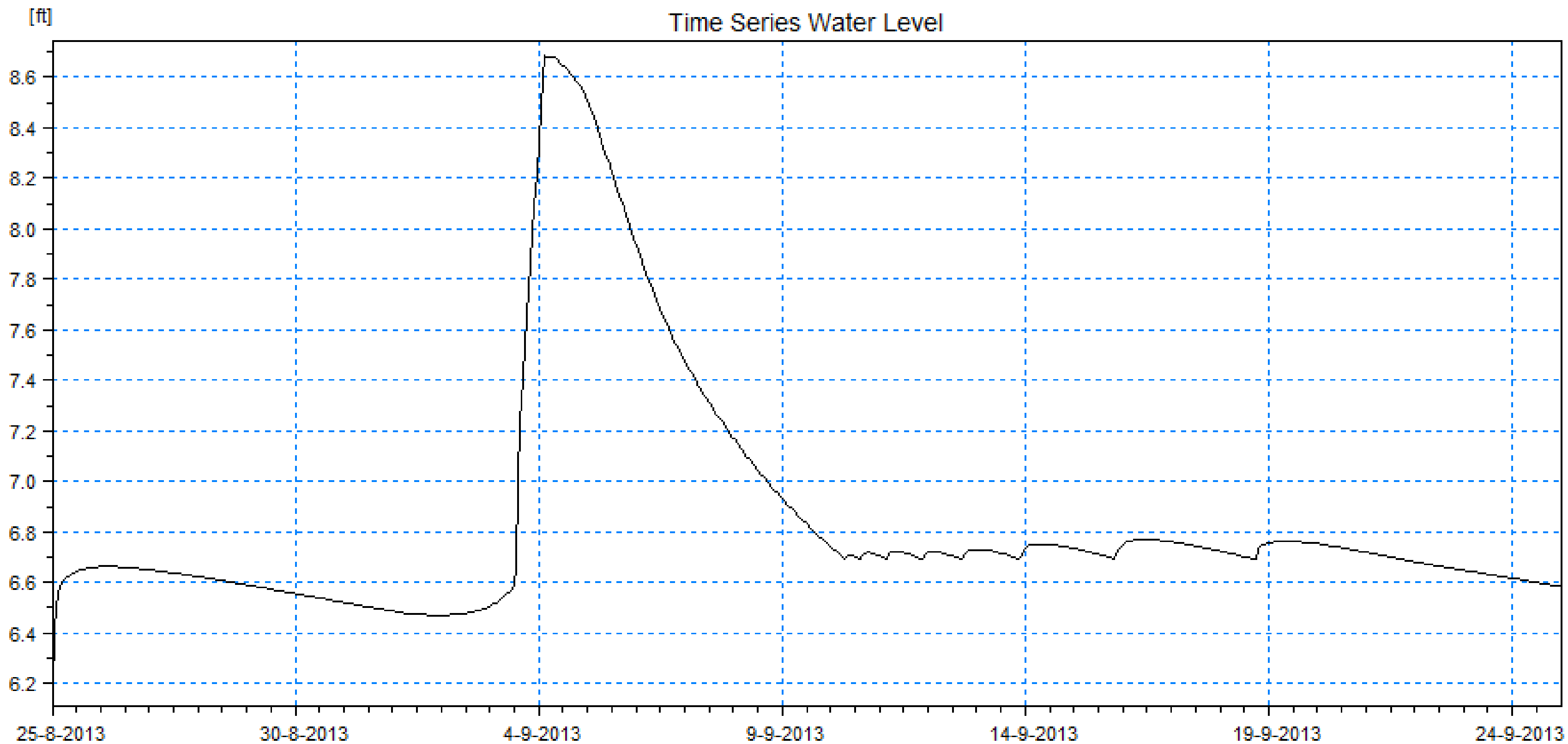

Figure 30. HC2 Headwater Stage (ft)

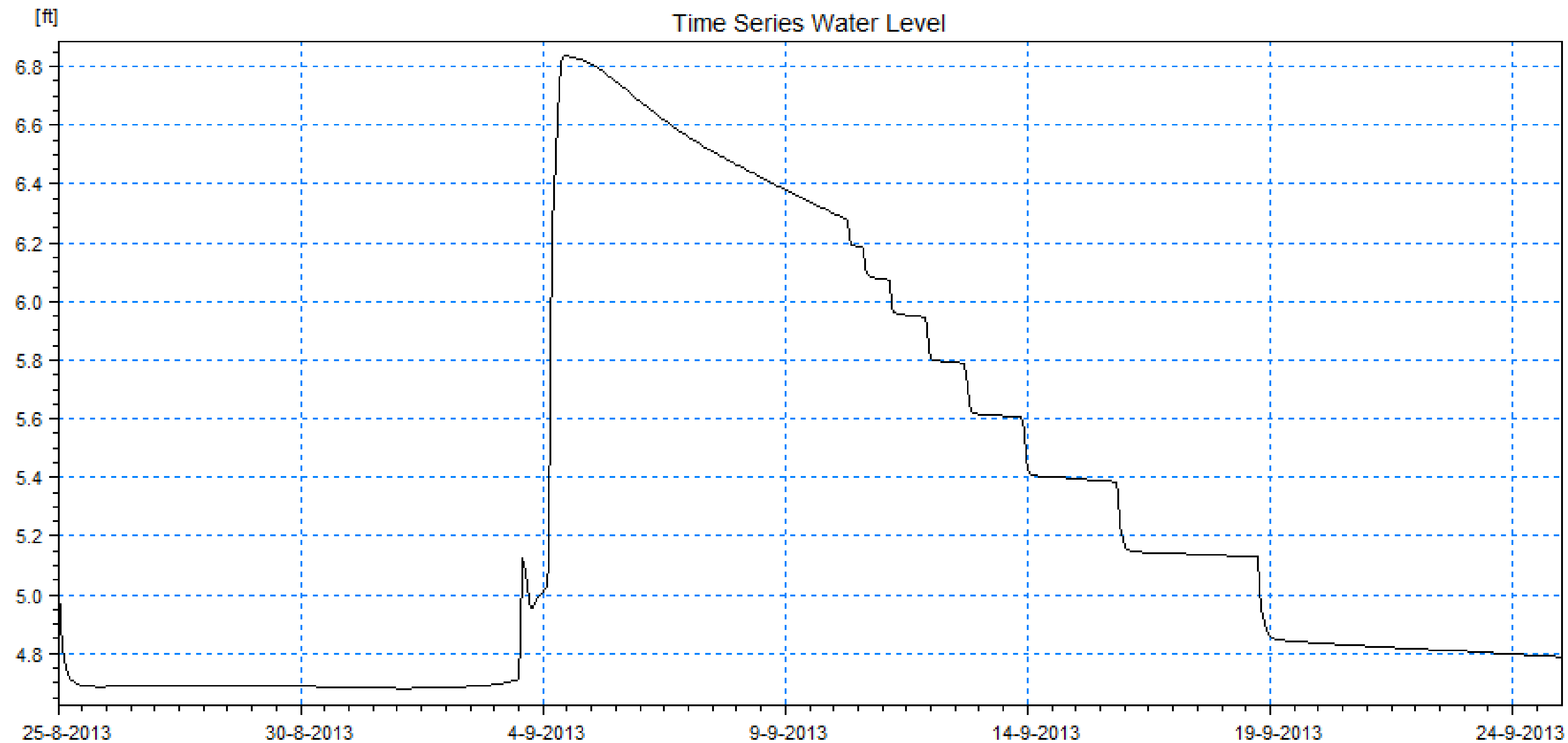

Figure 31. HC2 Tailwater Stage (ft)

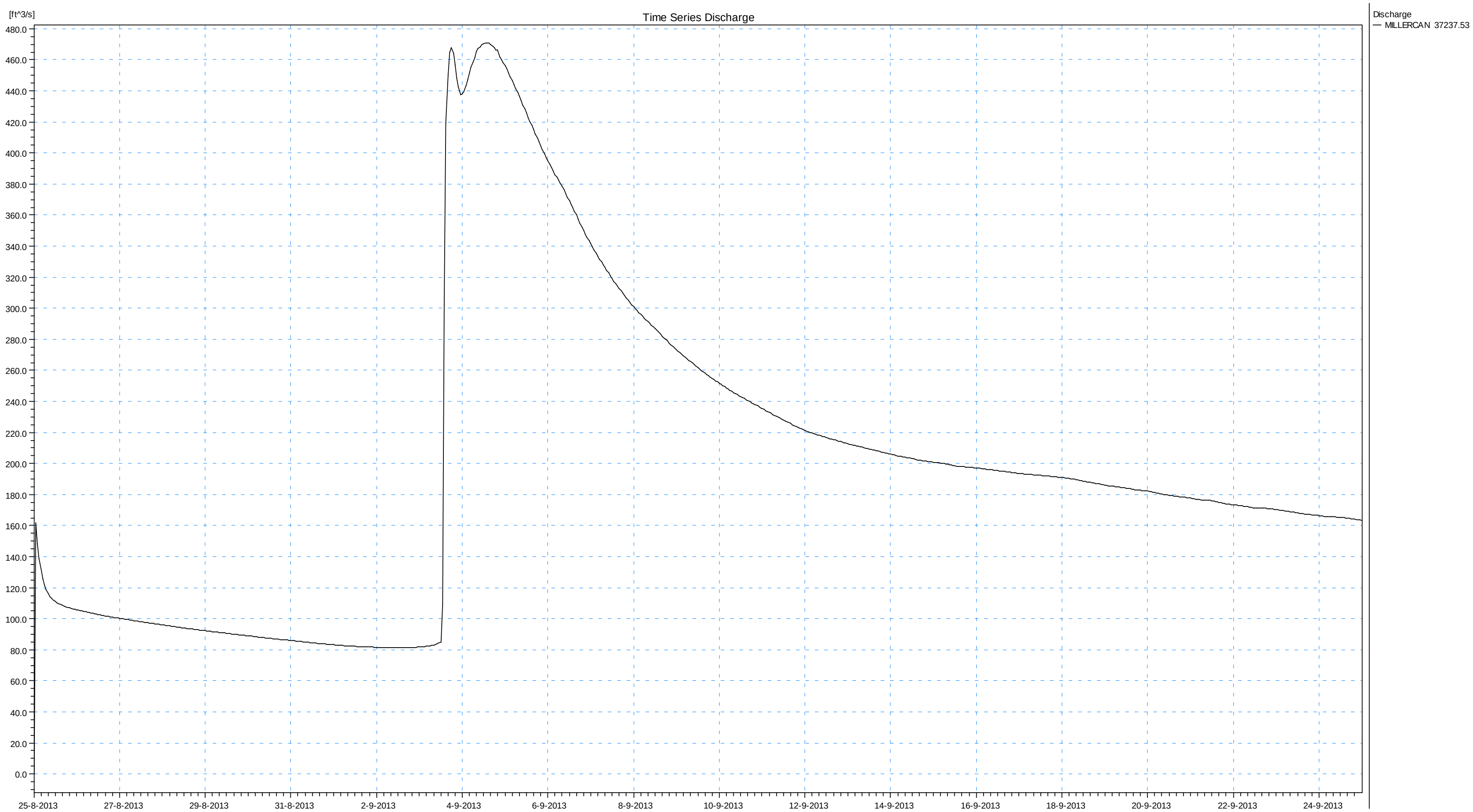

Figure 32. Miller2 Discharge (cfs)

Figure 33. Miller2 Headwater Stage (ft)

Figure 34. Miller2 Tailwater Stage (ft)

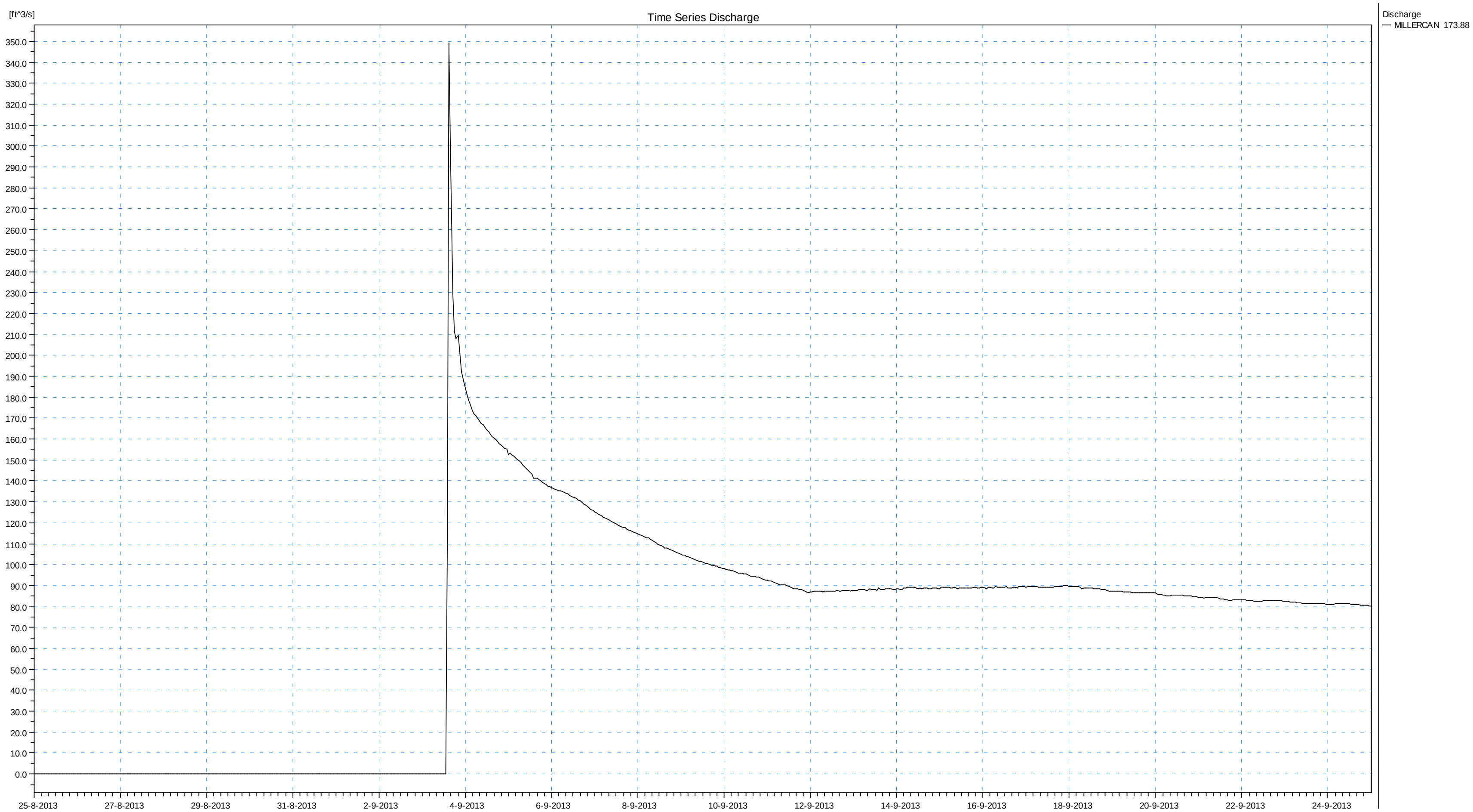

Figure 35. Miller3 Discharge (cfs)

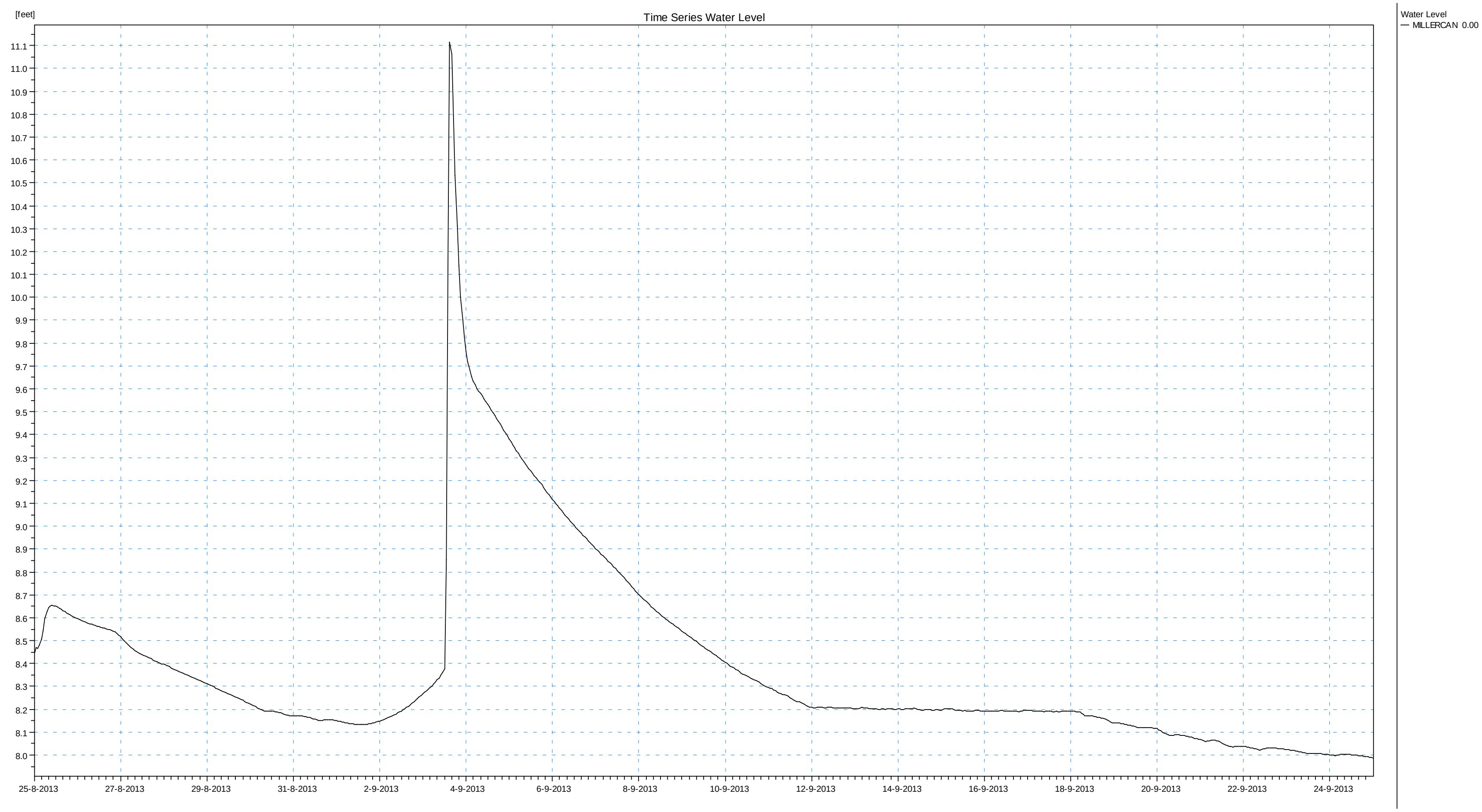

Figure 36. Miller3 Headwater Stage (ft)

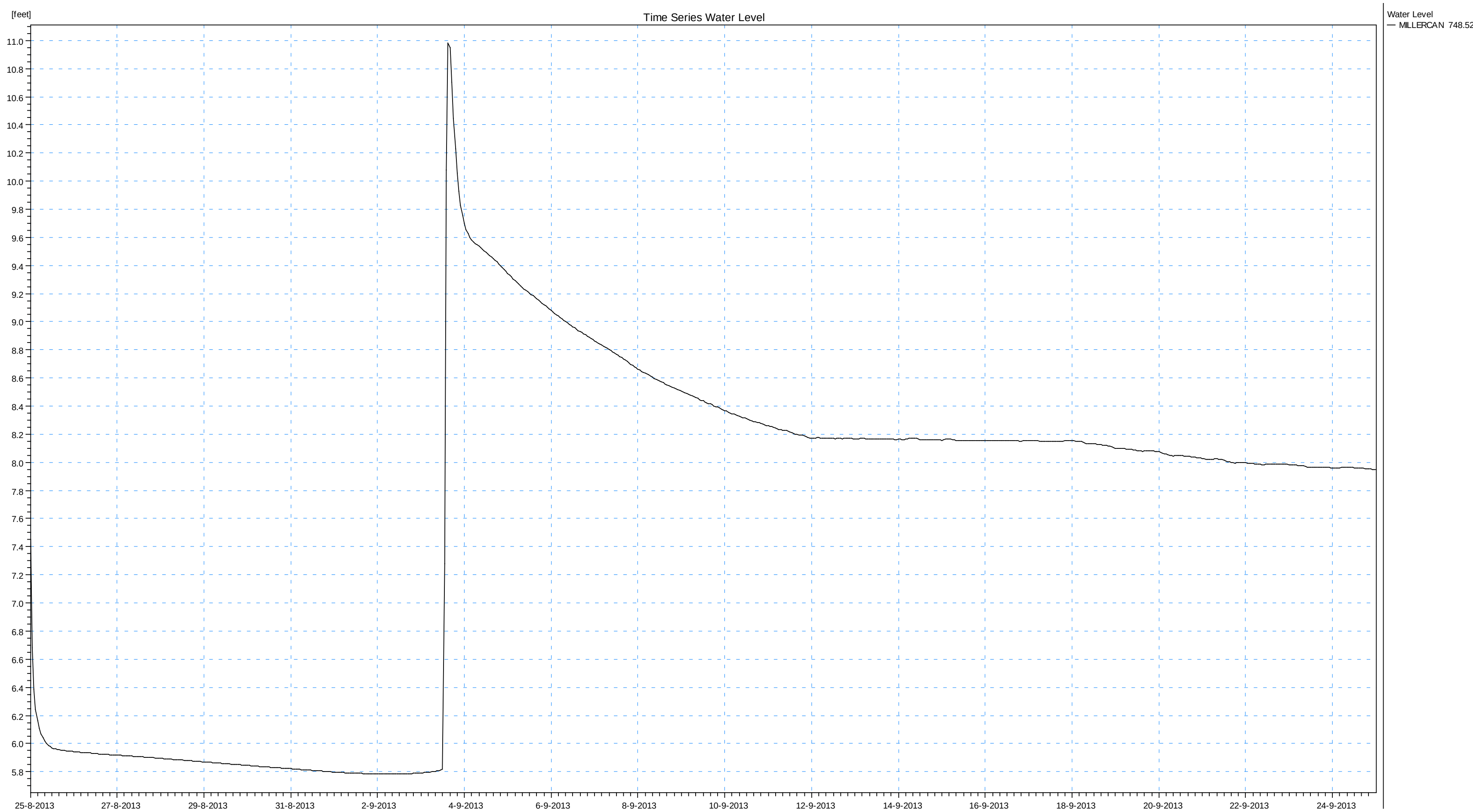

Figure 37. Miller3 Tailwater Stage (ft)

C – MIKE 11 Detailed Time Series for 10-year Storm, Current Conditions

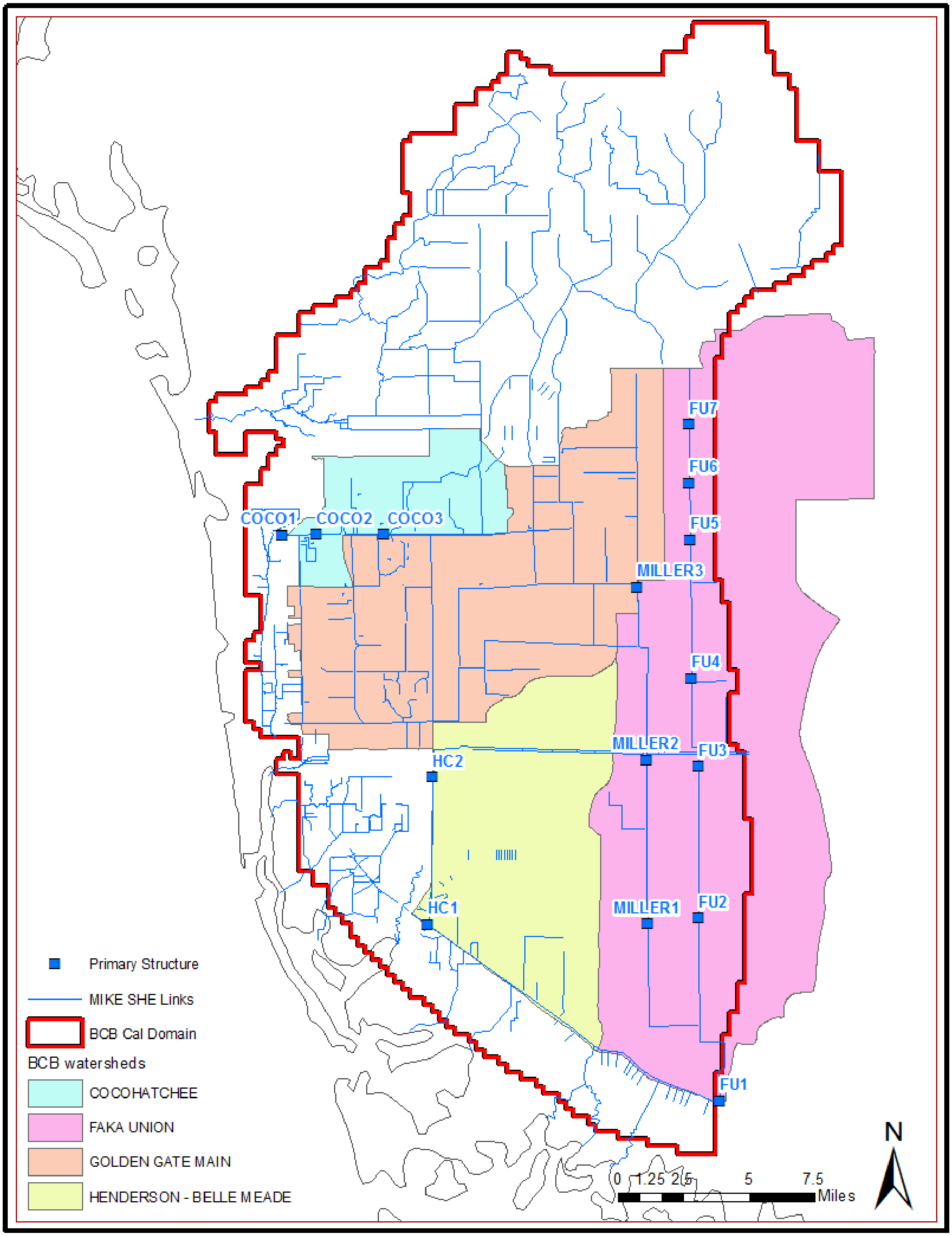

Figure 1 Mike SHE Stations within COCO, FAKA and HEND Watersheds

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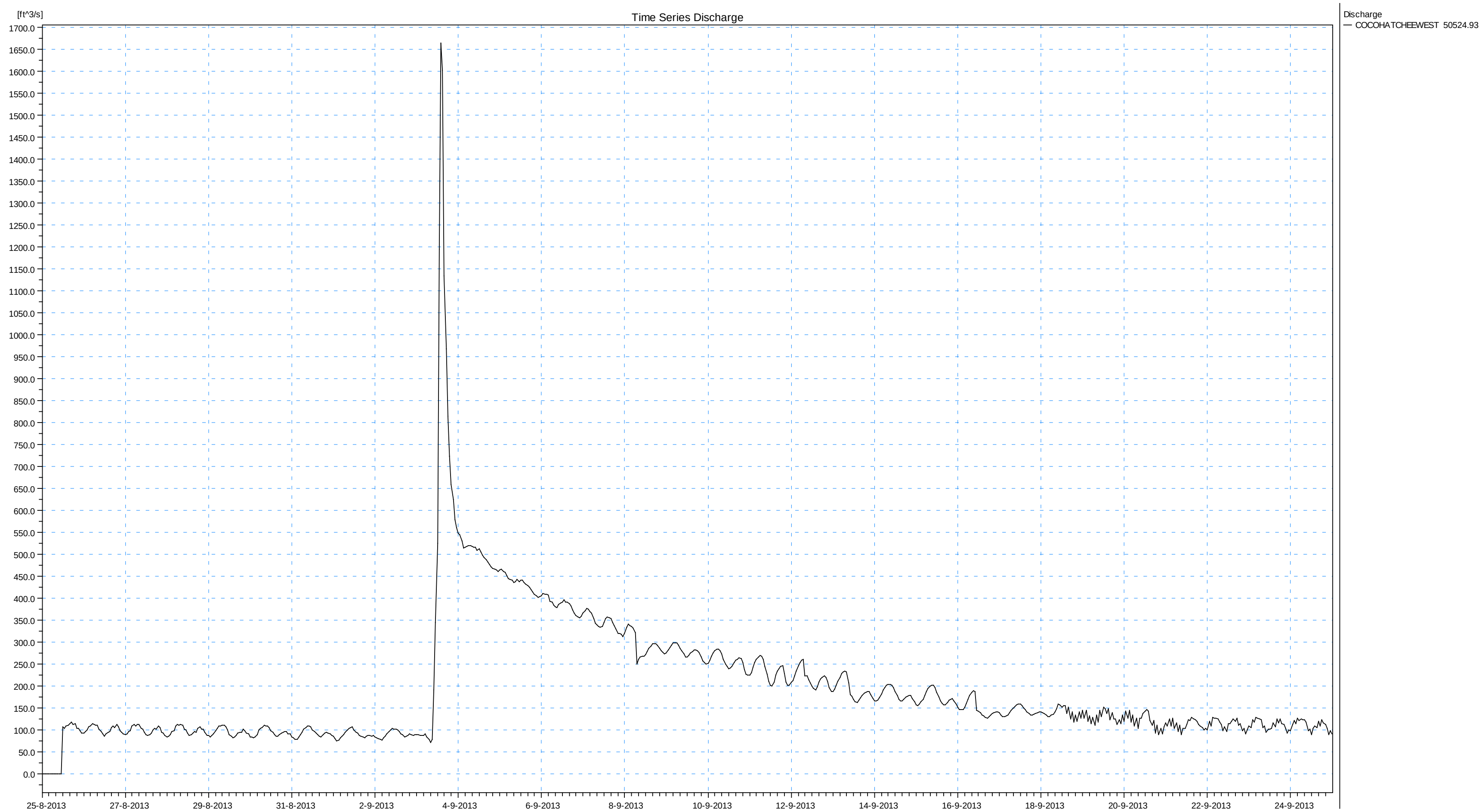


Figure 2. COCO1 Discharge (cfs)

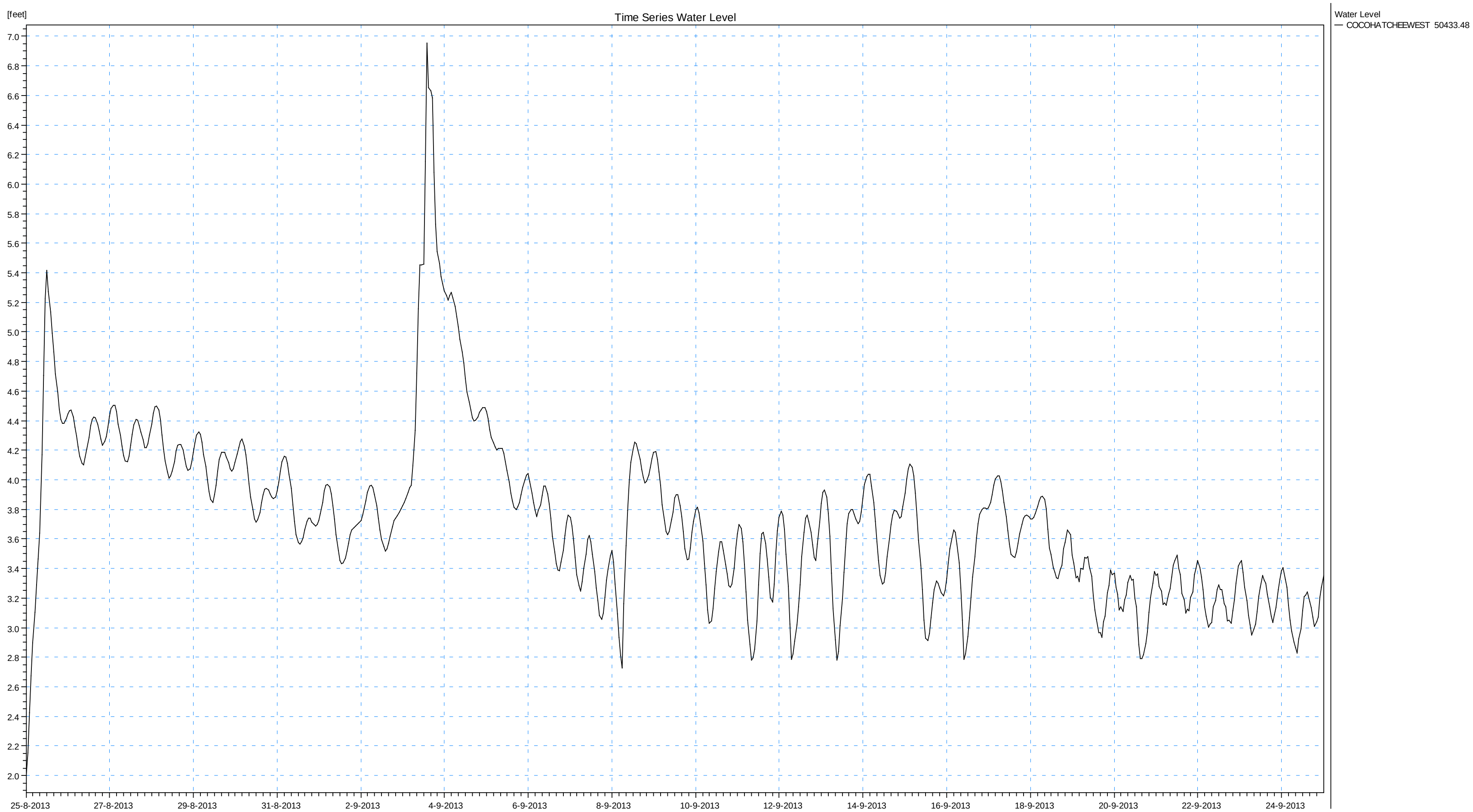


Figure 3. COCO1 Headwater Stage (ft)

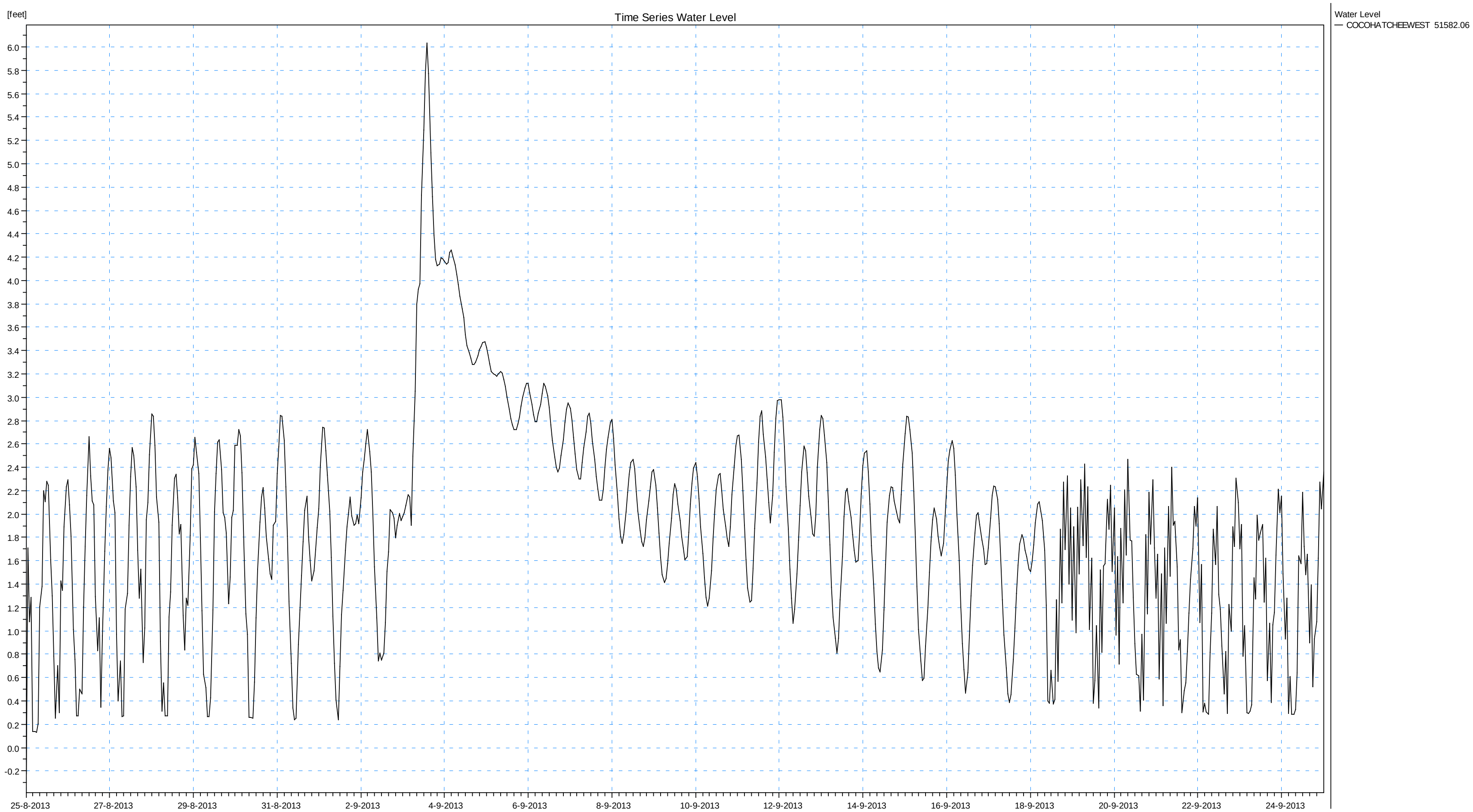


Figure 4. COCO1 Tailwater Stage (ft)

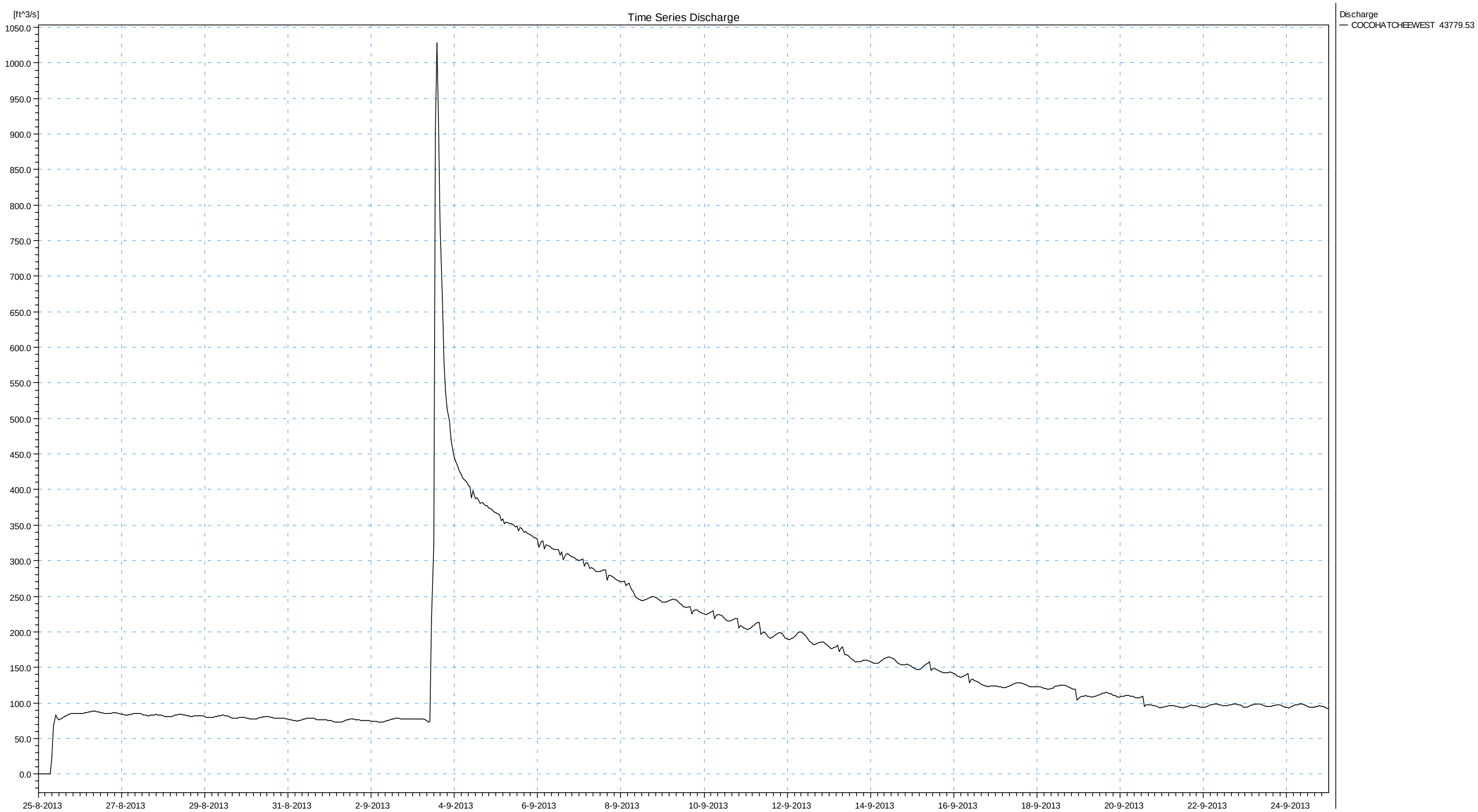


Figure 5. COCO2 Discharge (cfs)

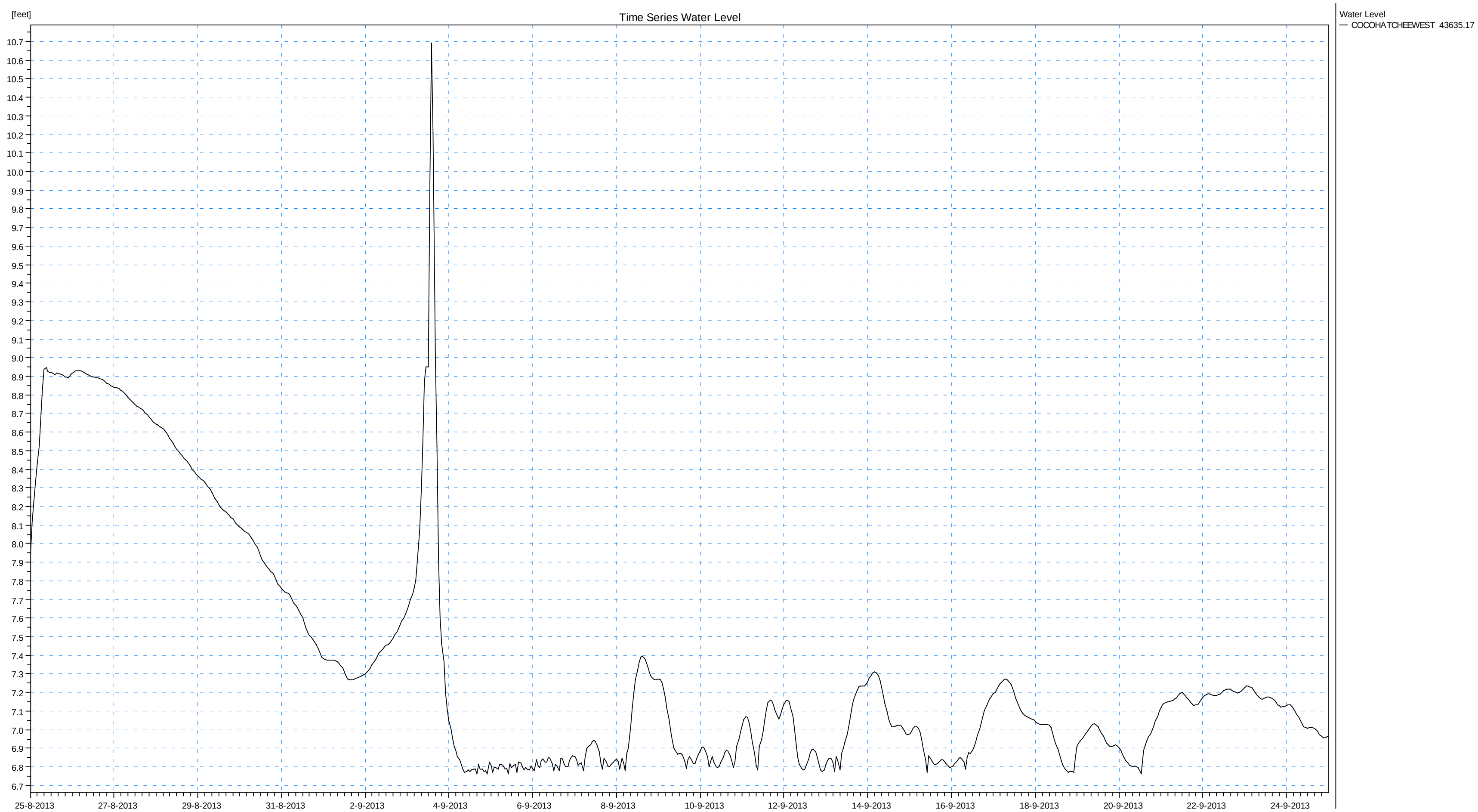
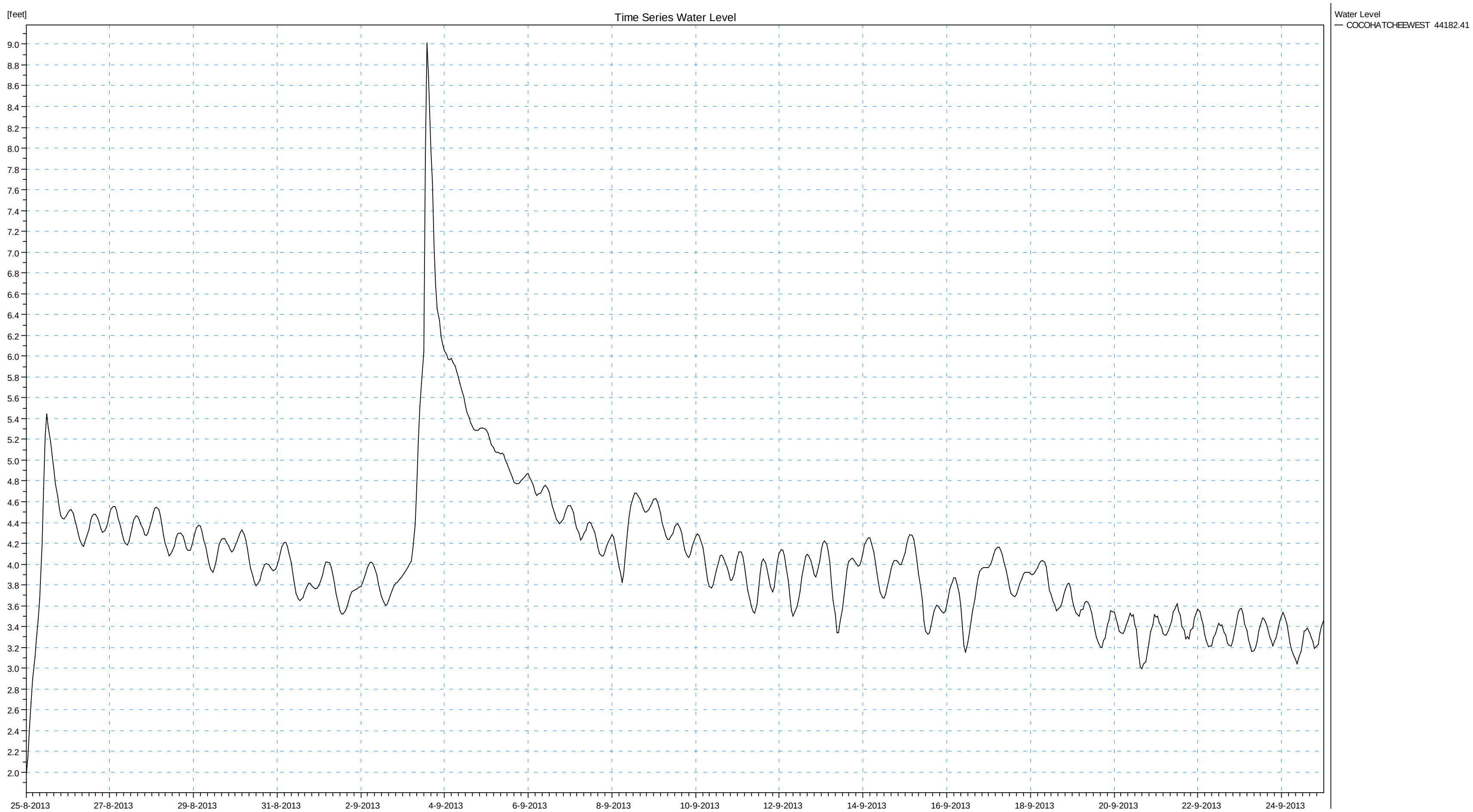


Figure 6. COCO2 Headwater Stage (ft)



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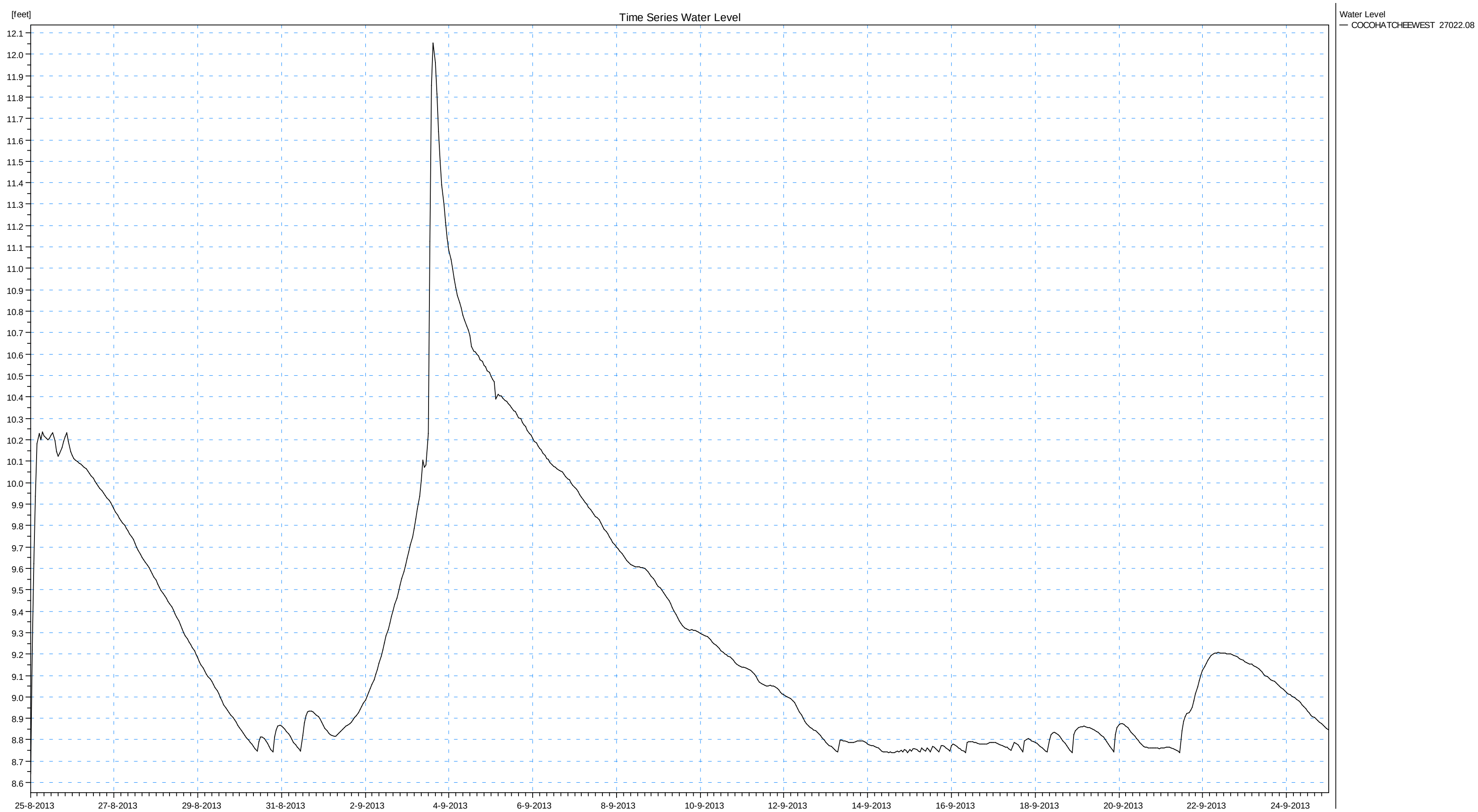

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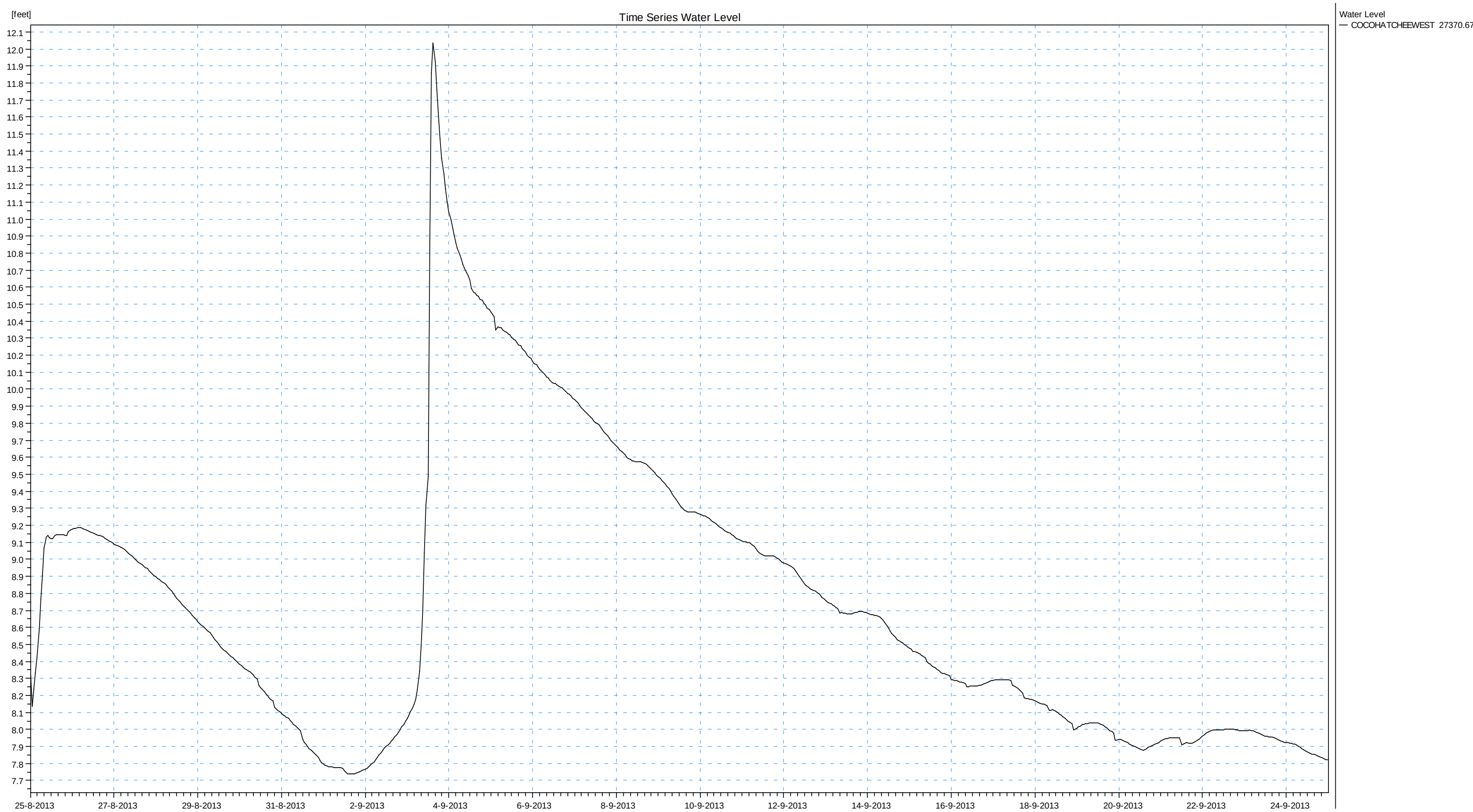

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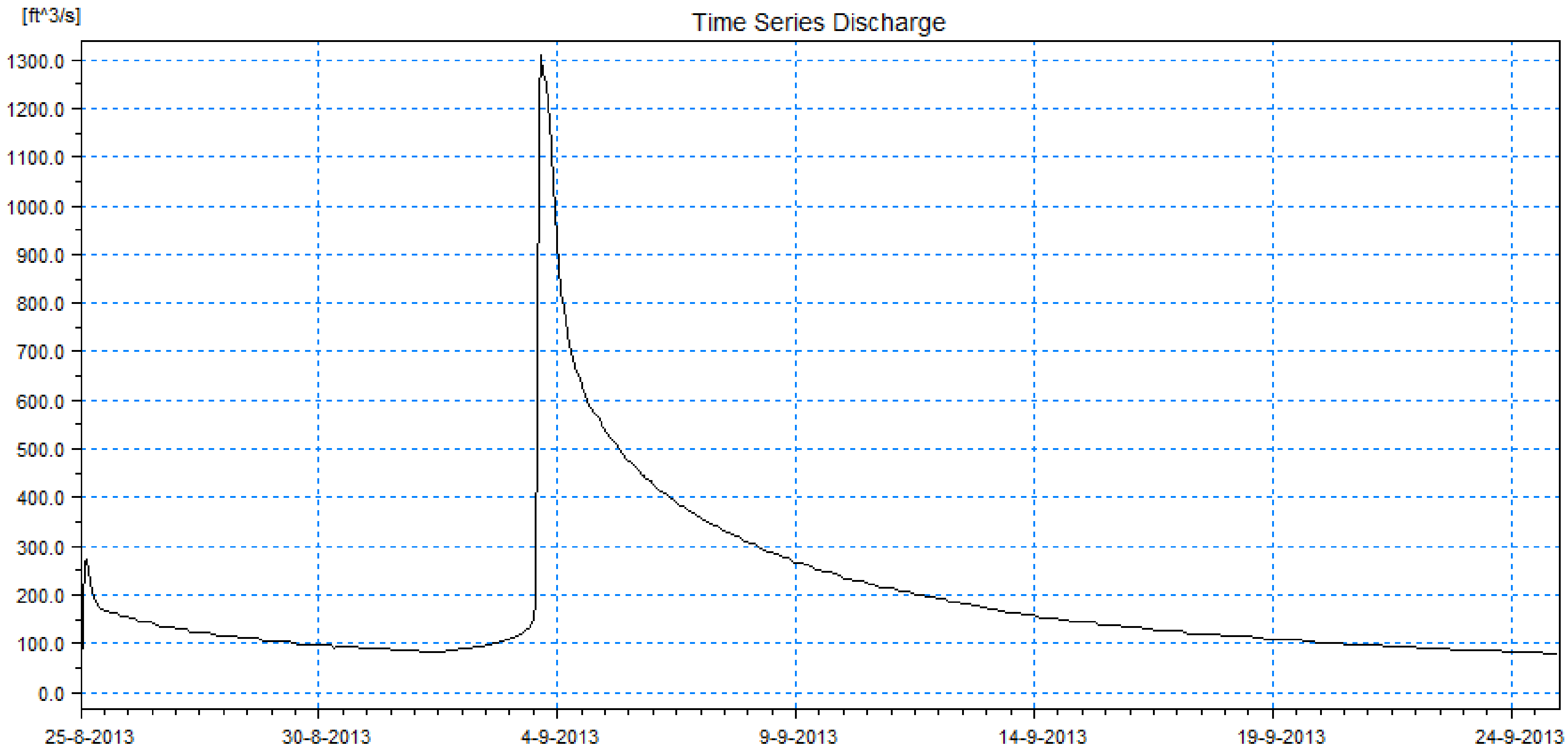

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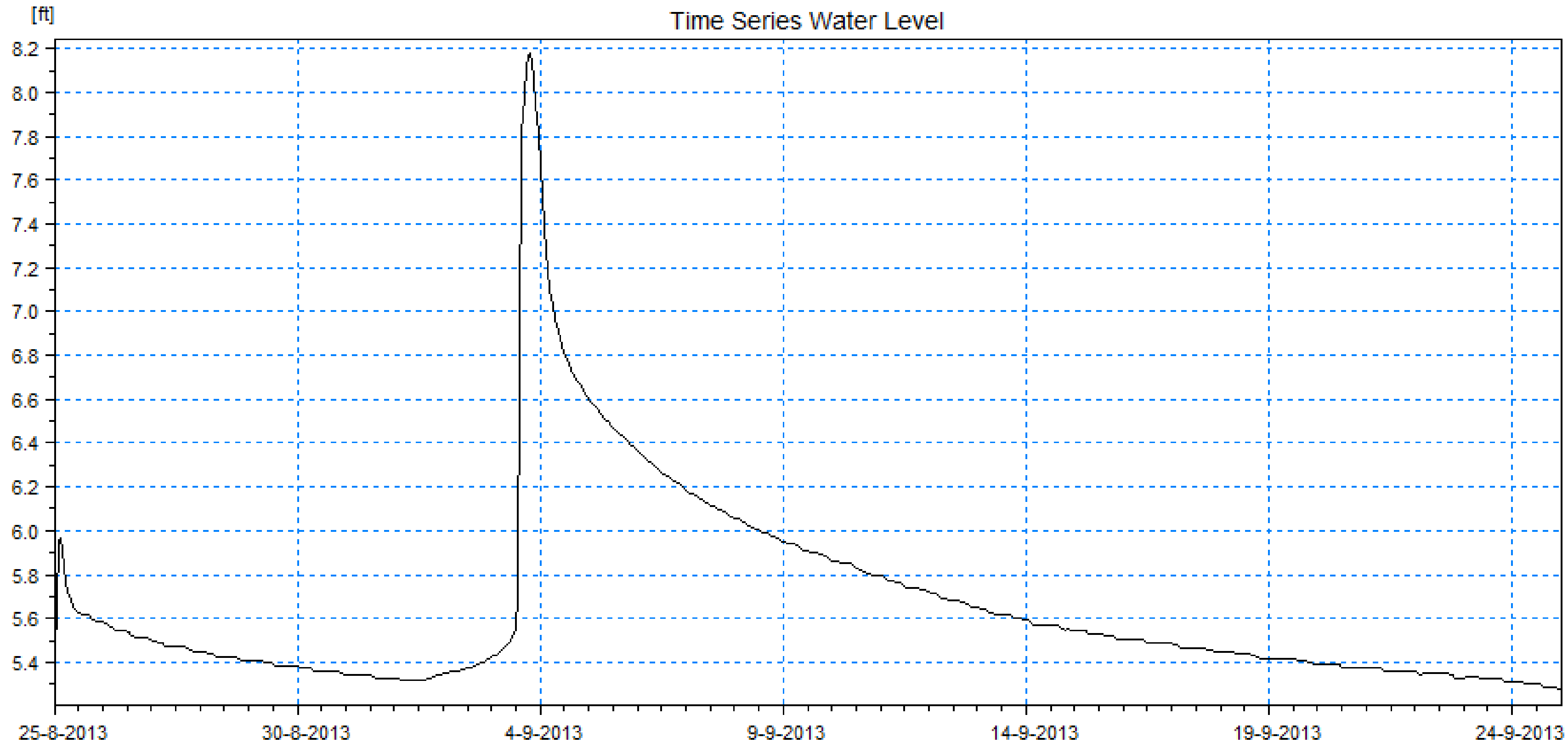

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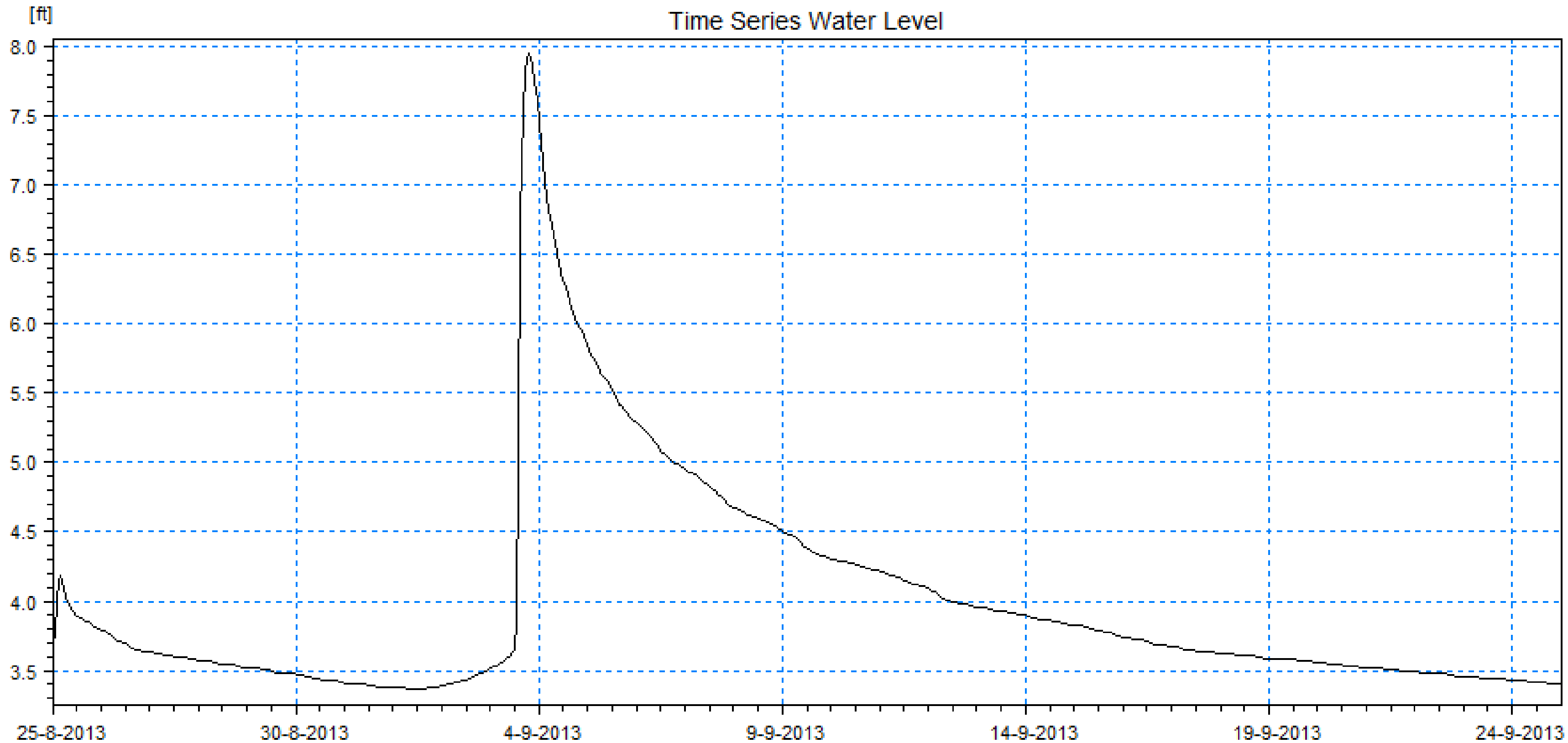

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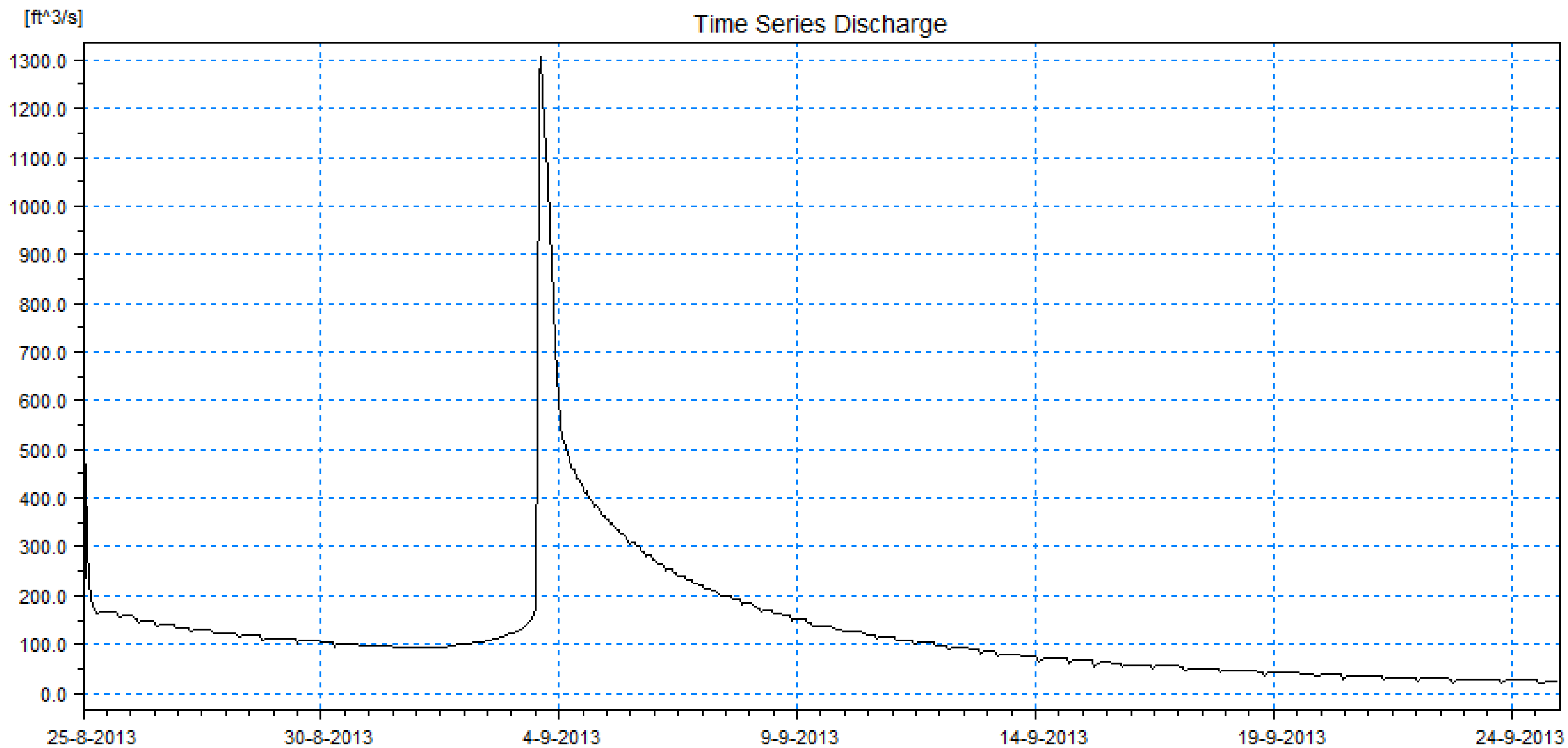

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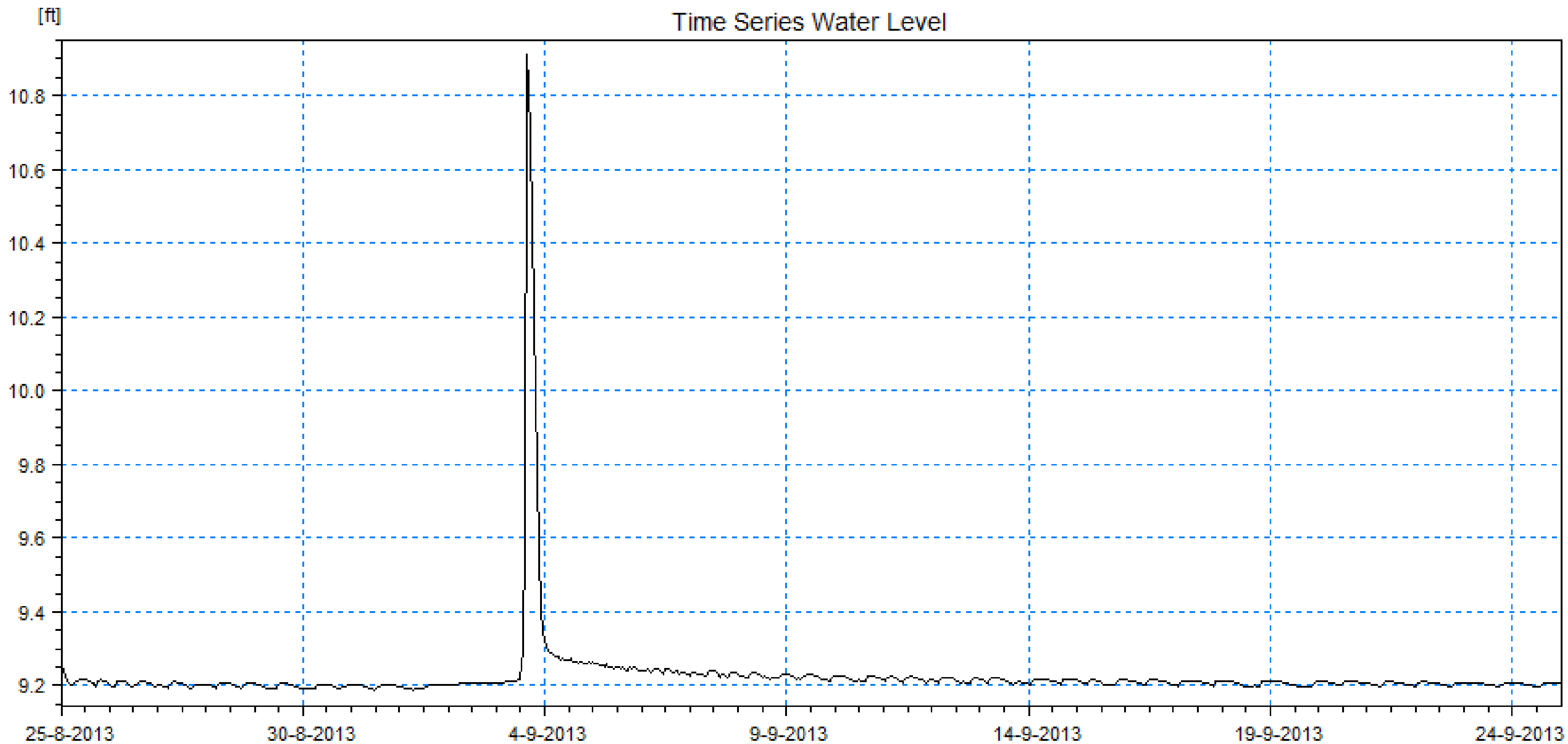

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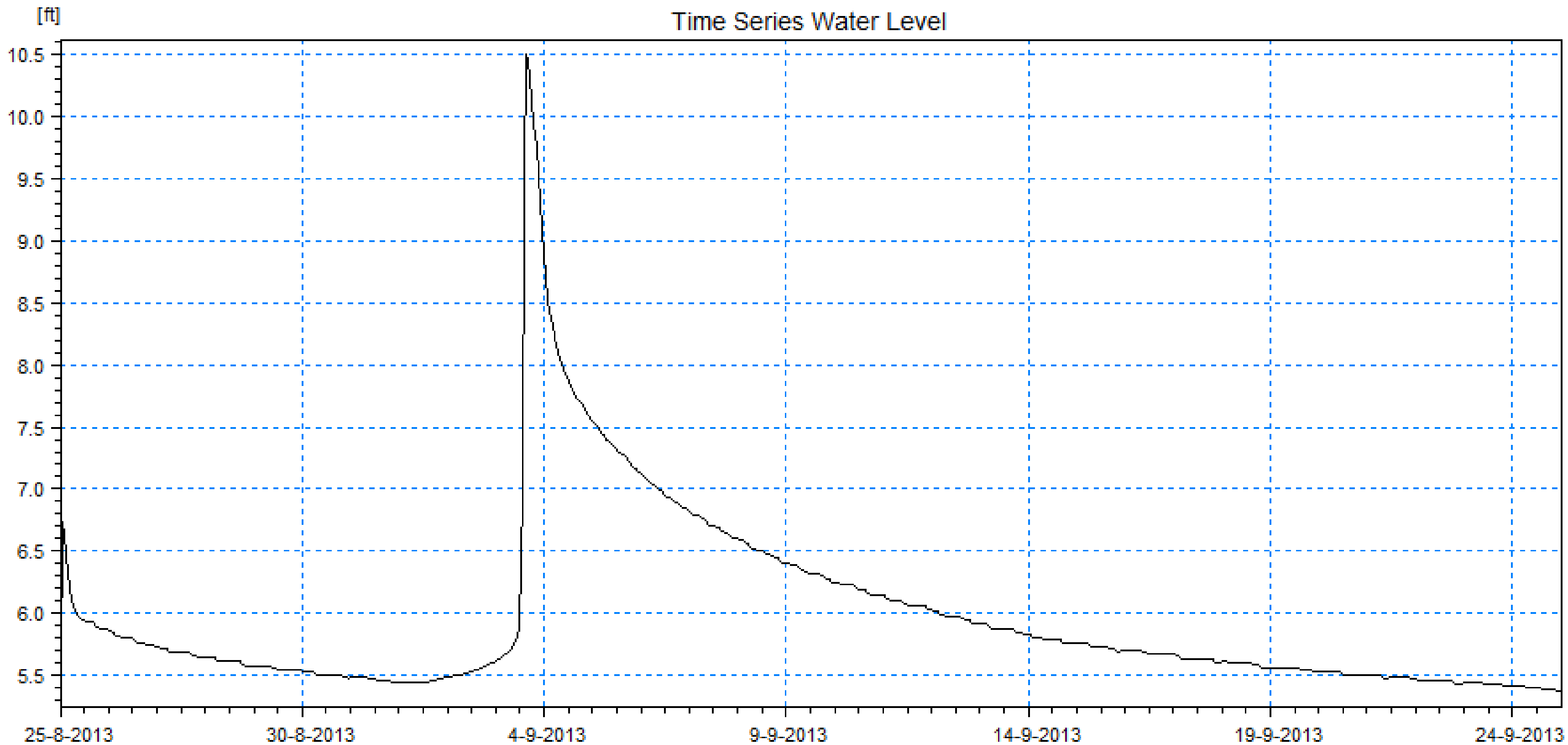

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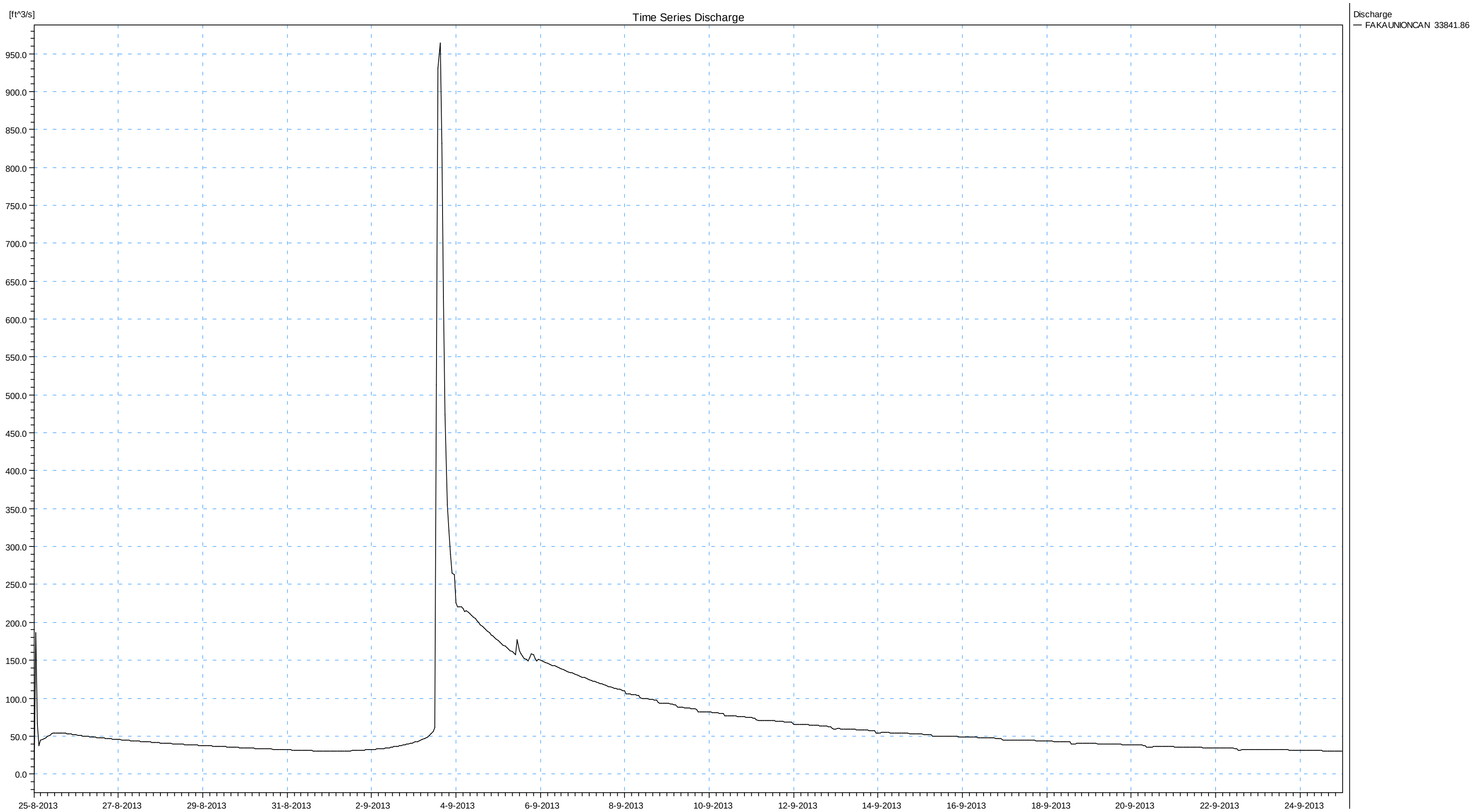

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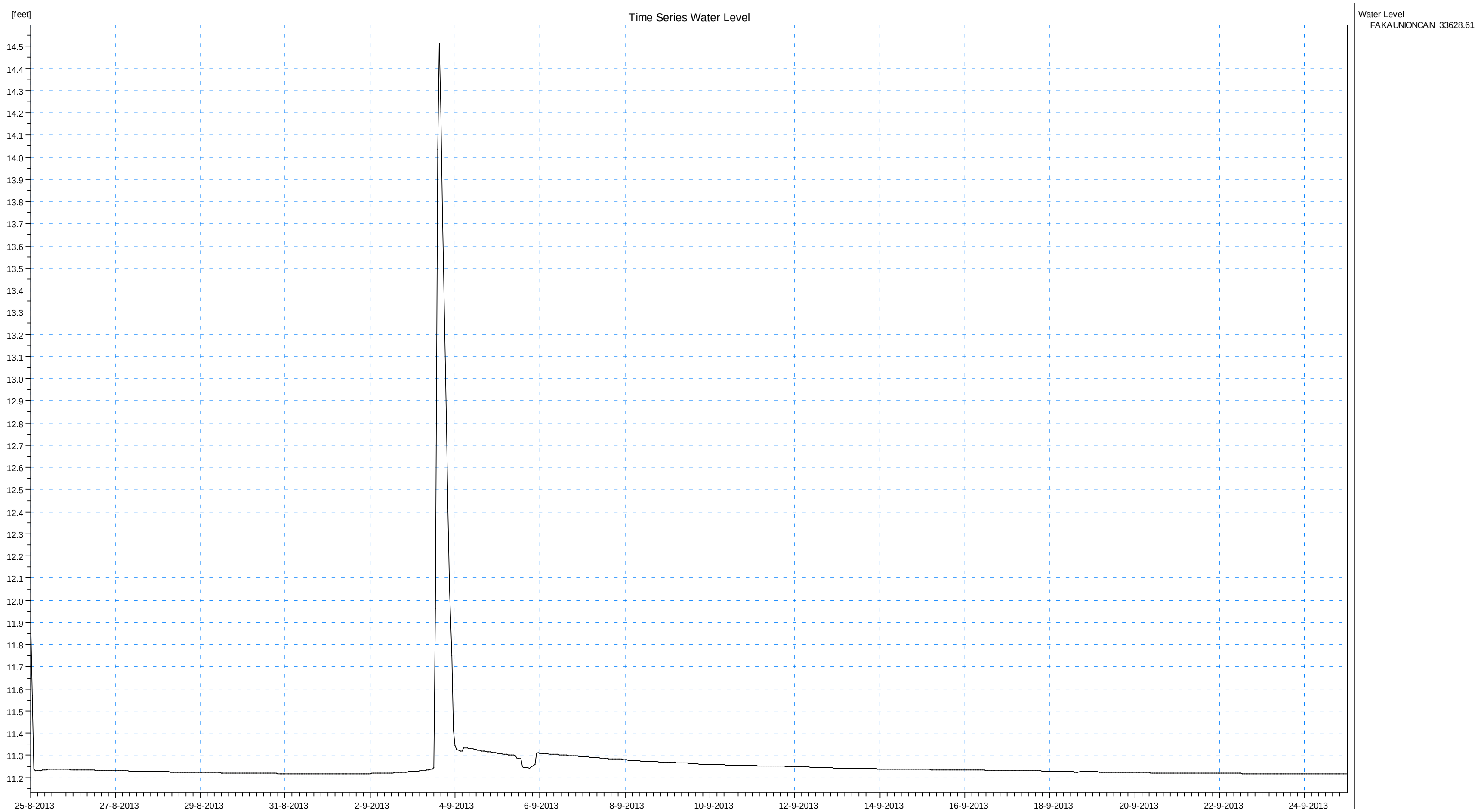

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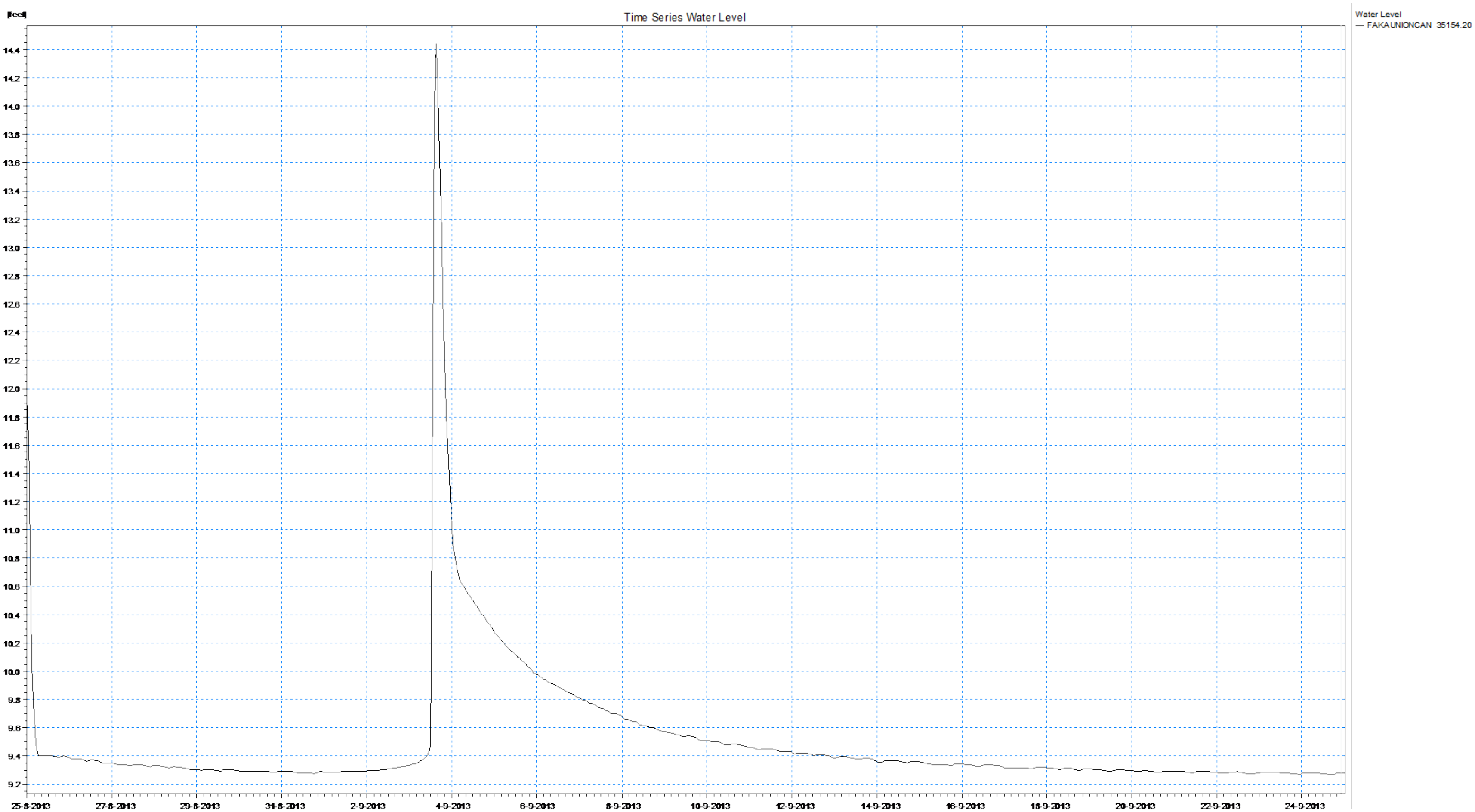

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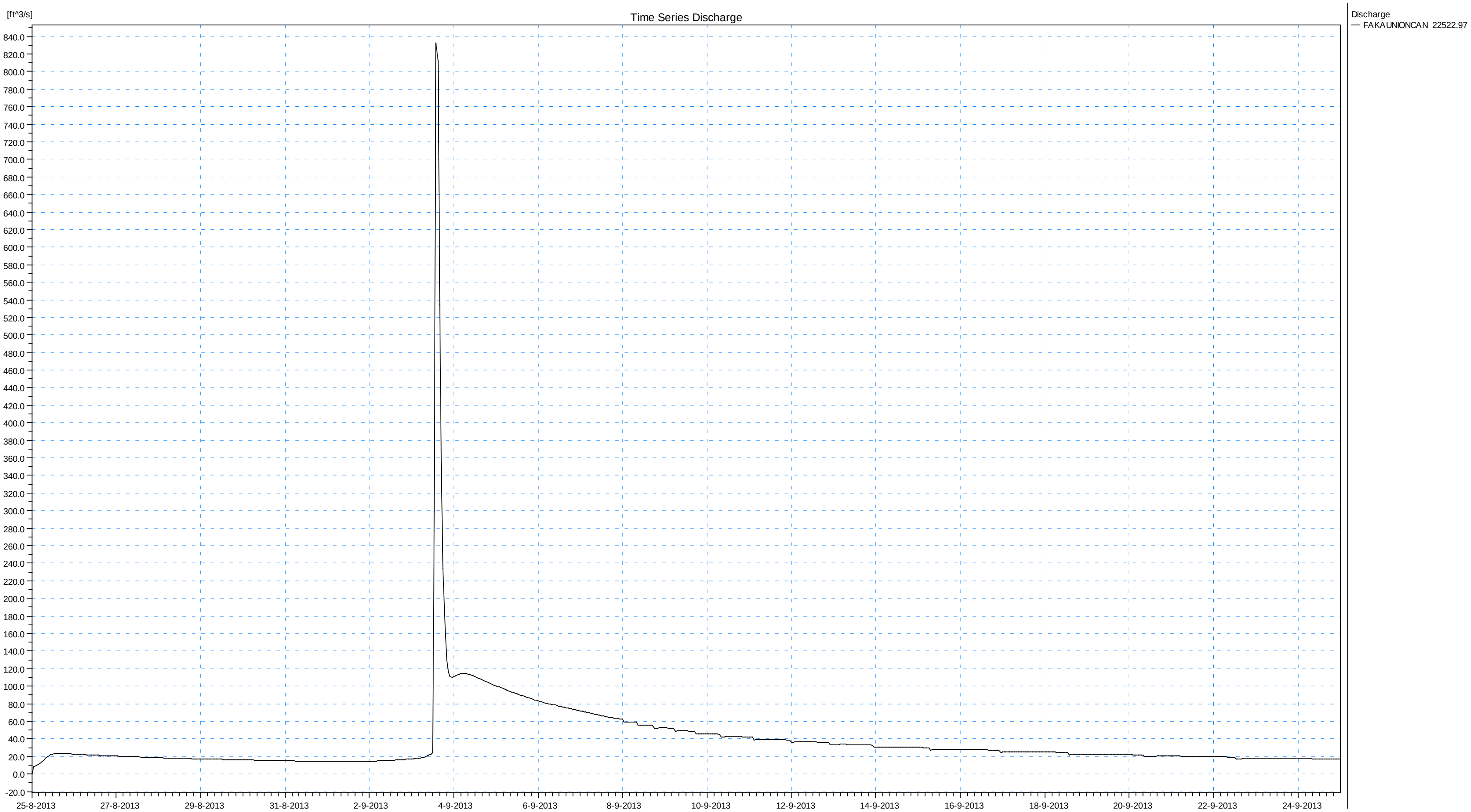

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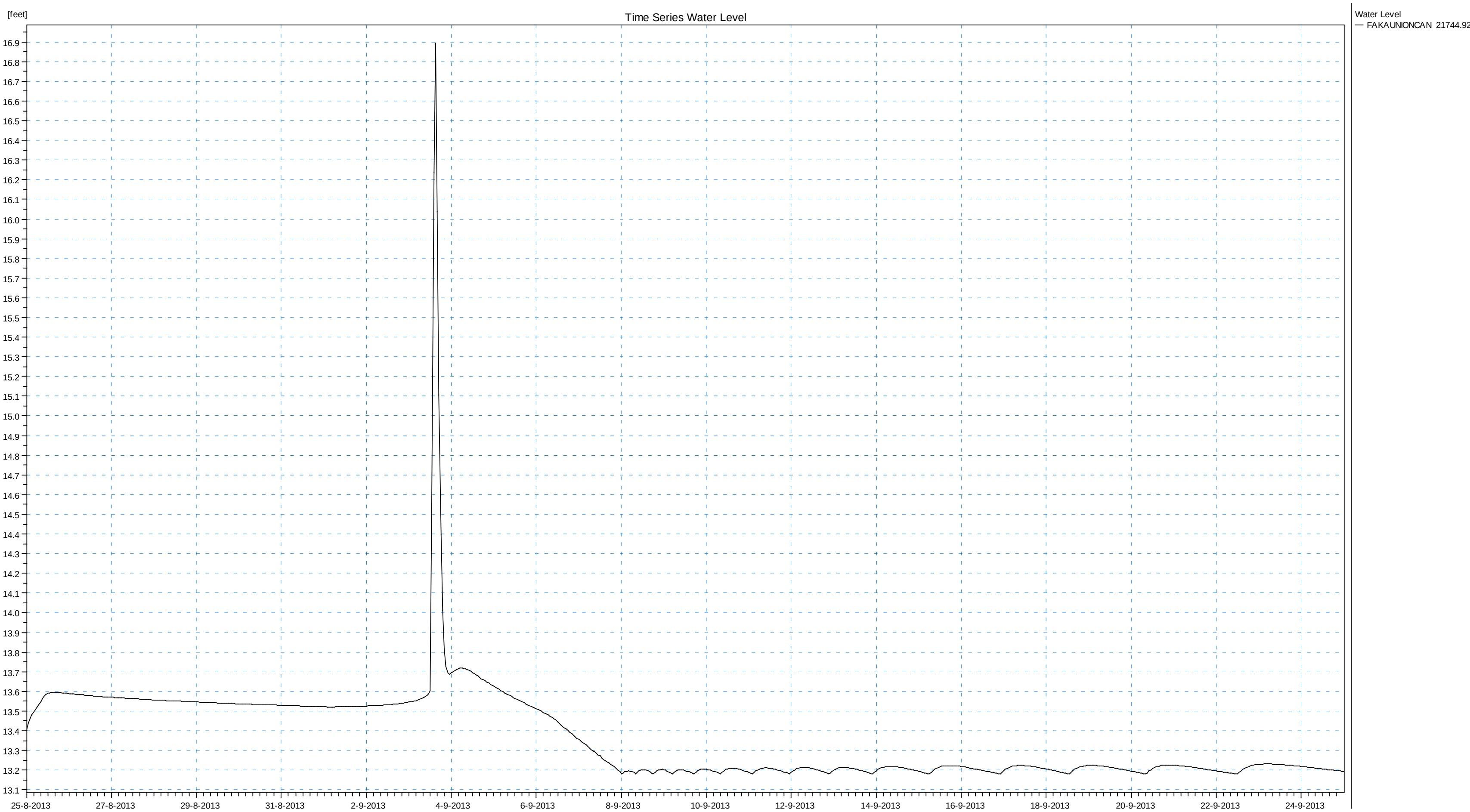

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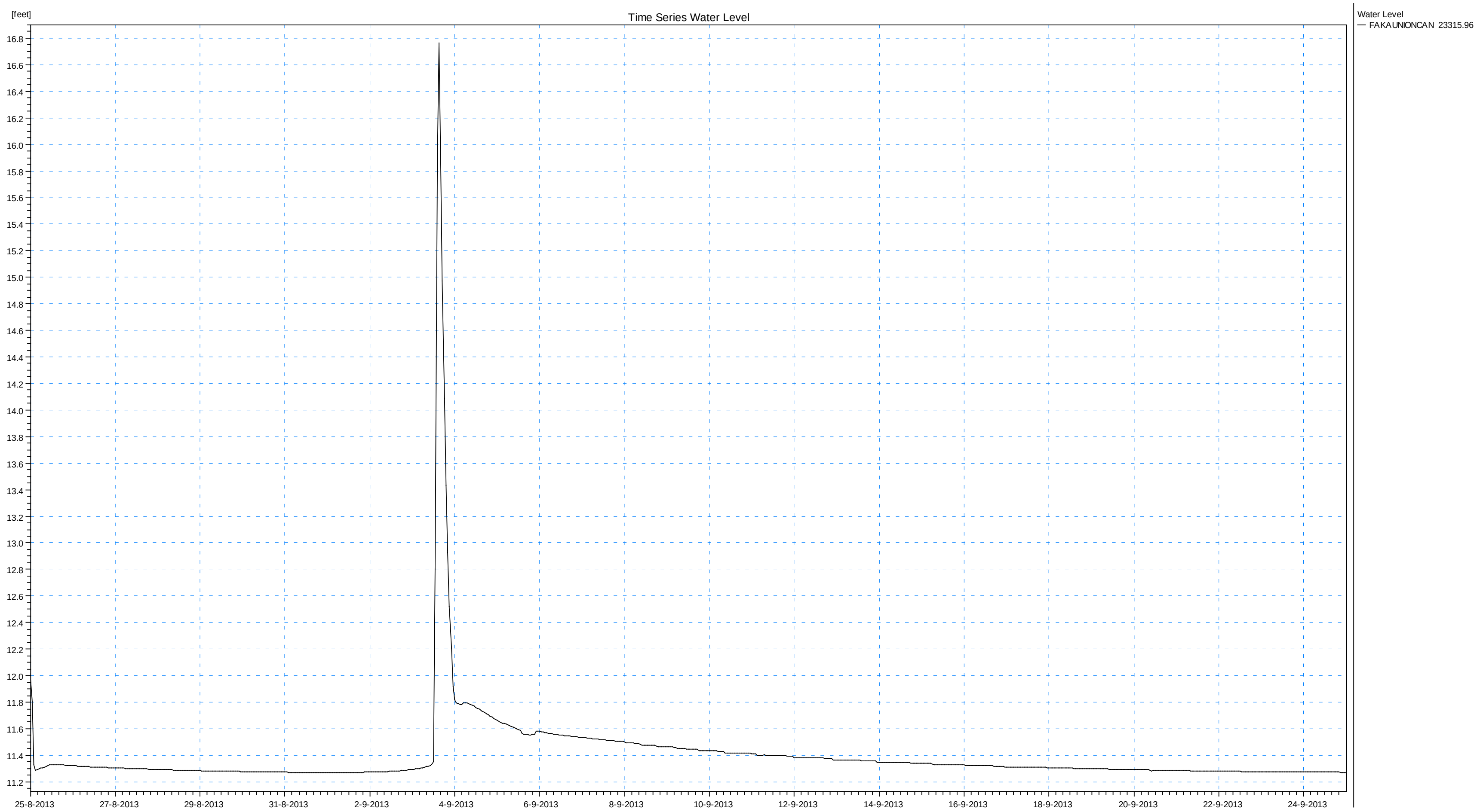

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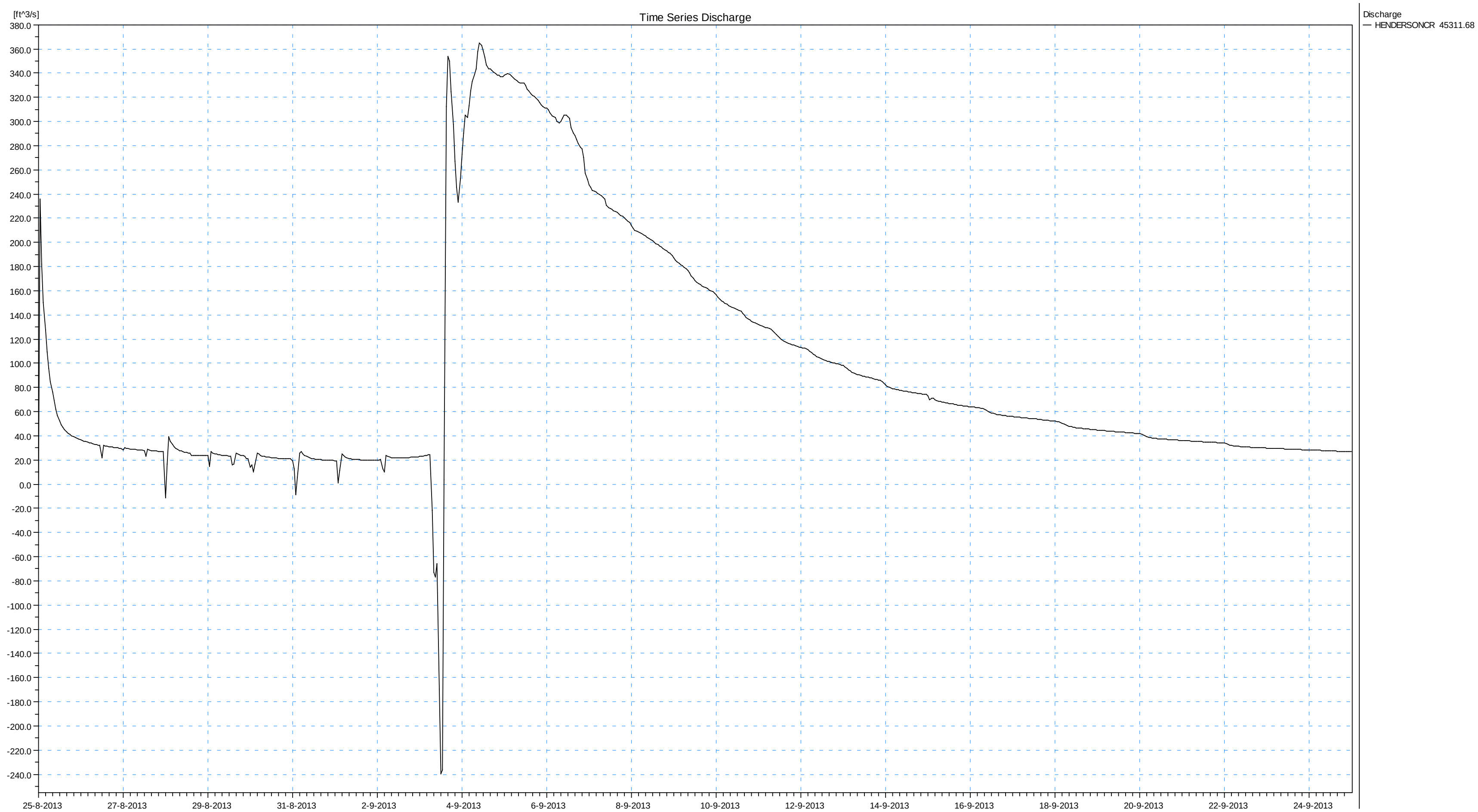

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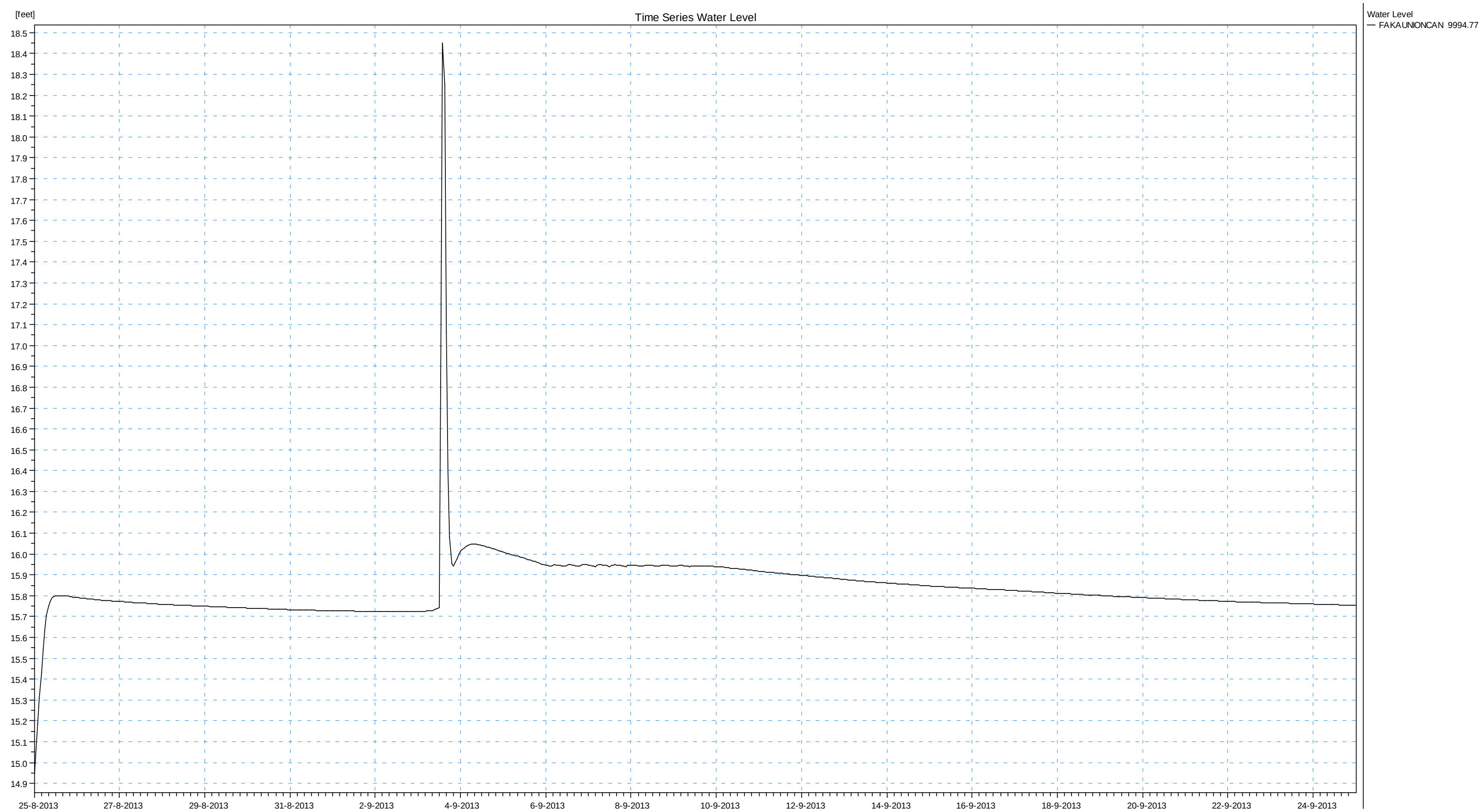

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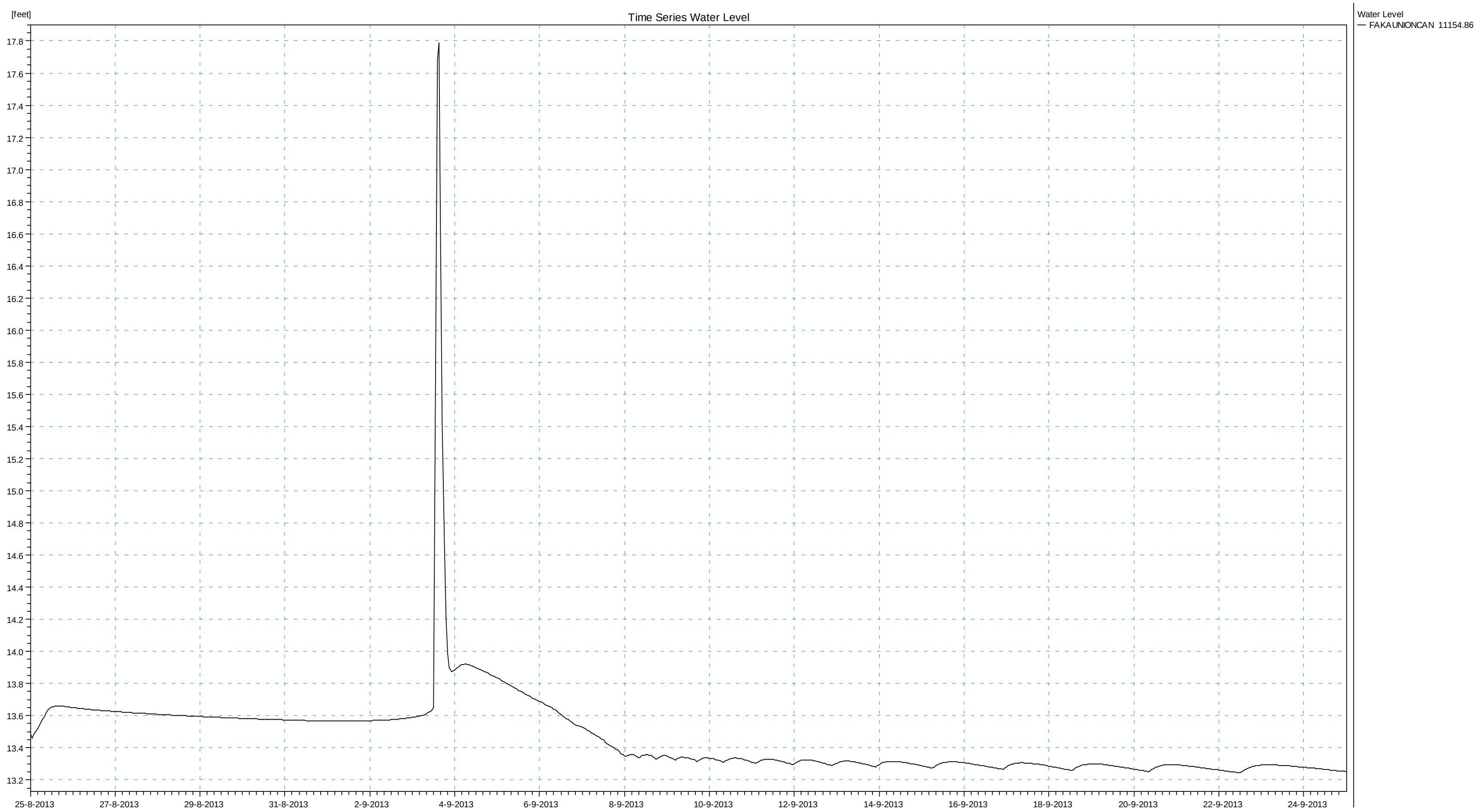

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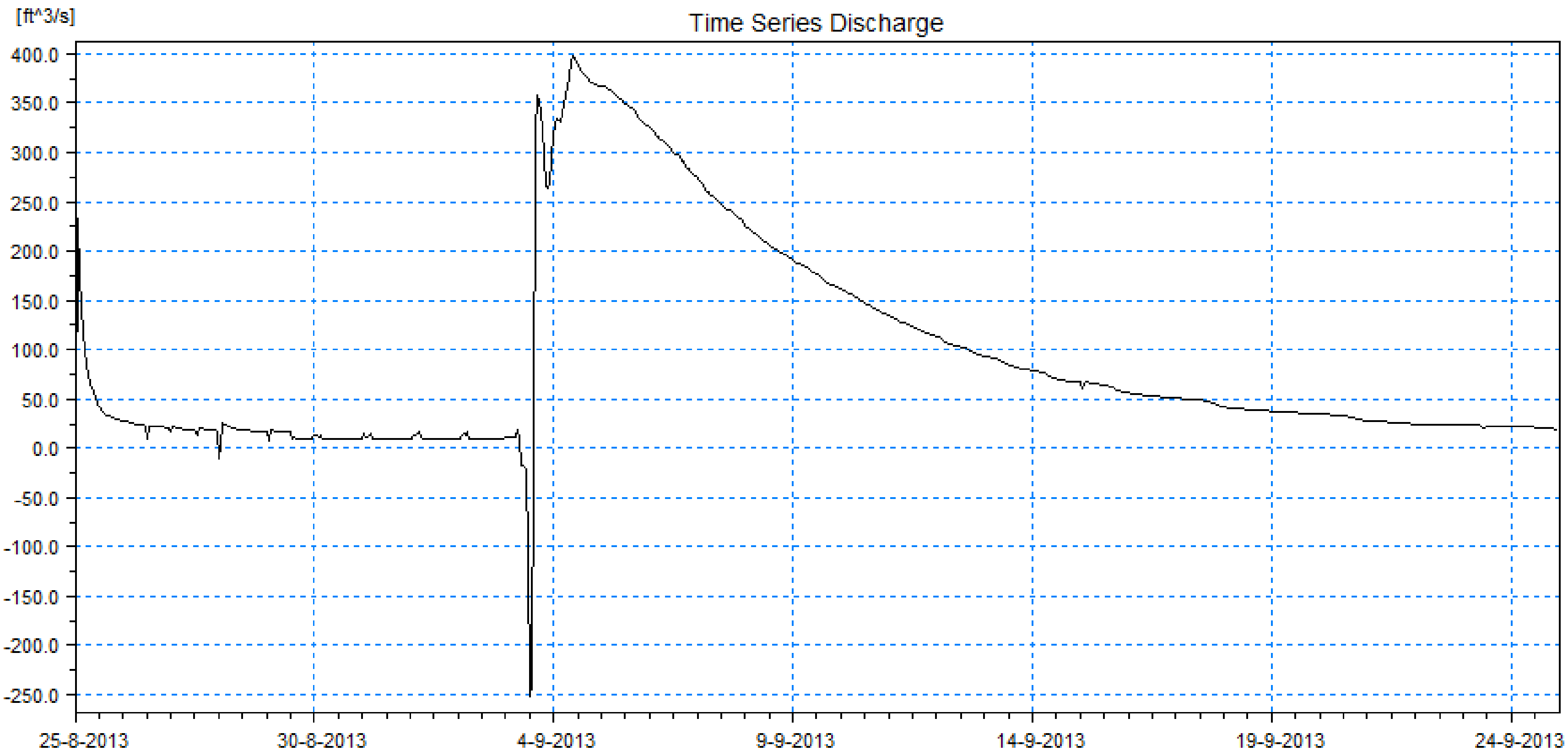

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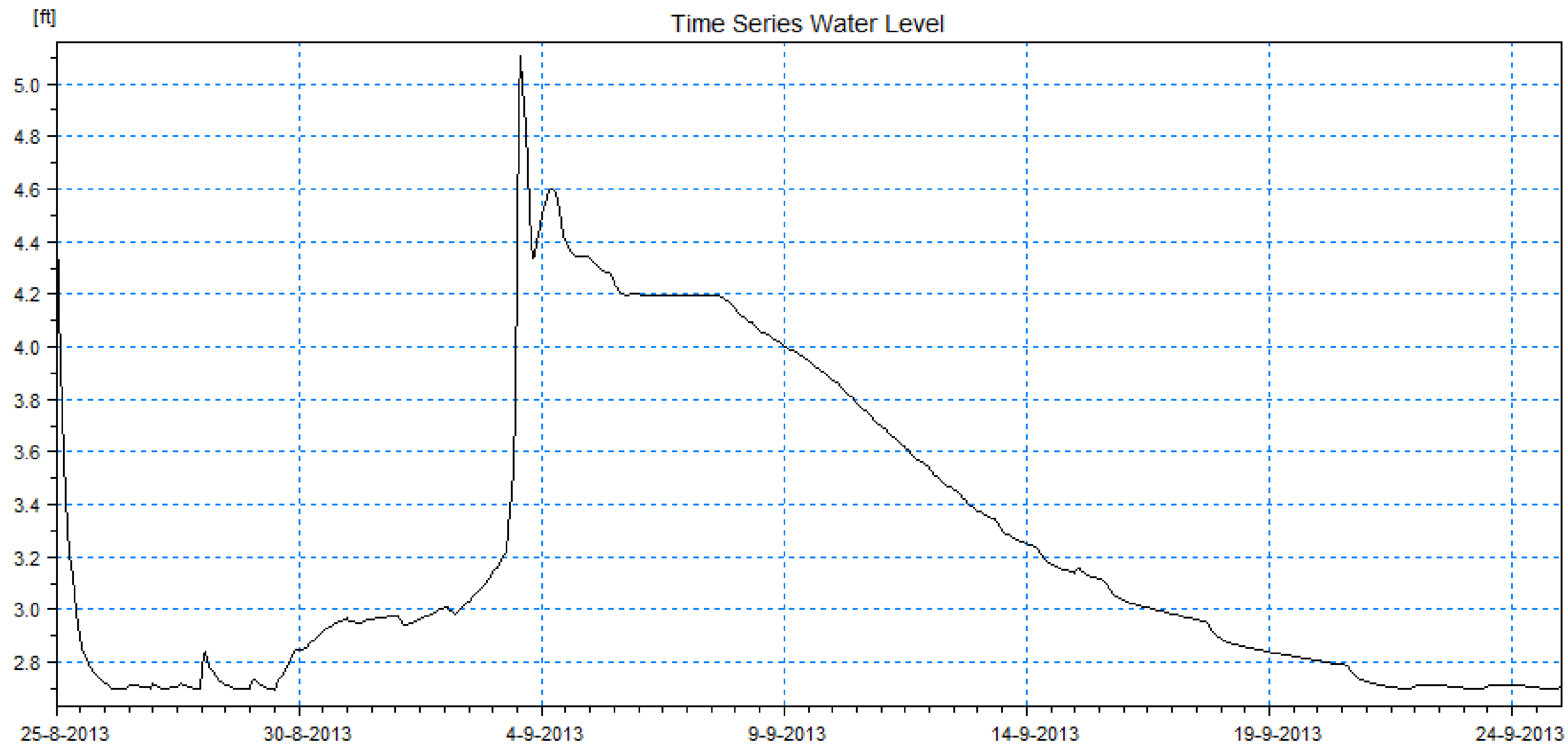

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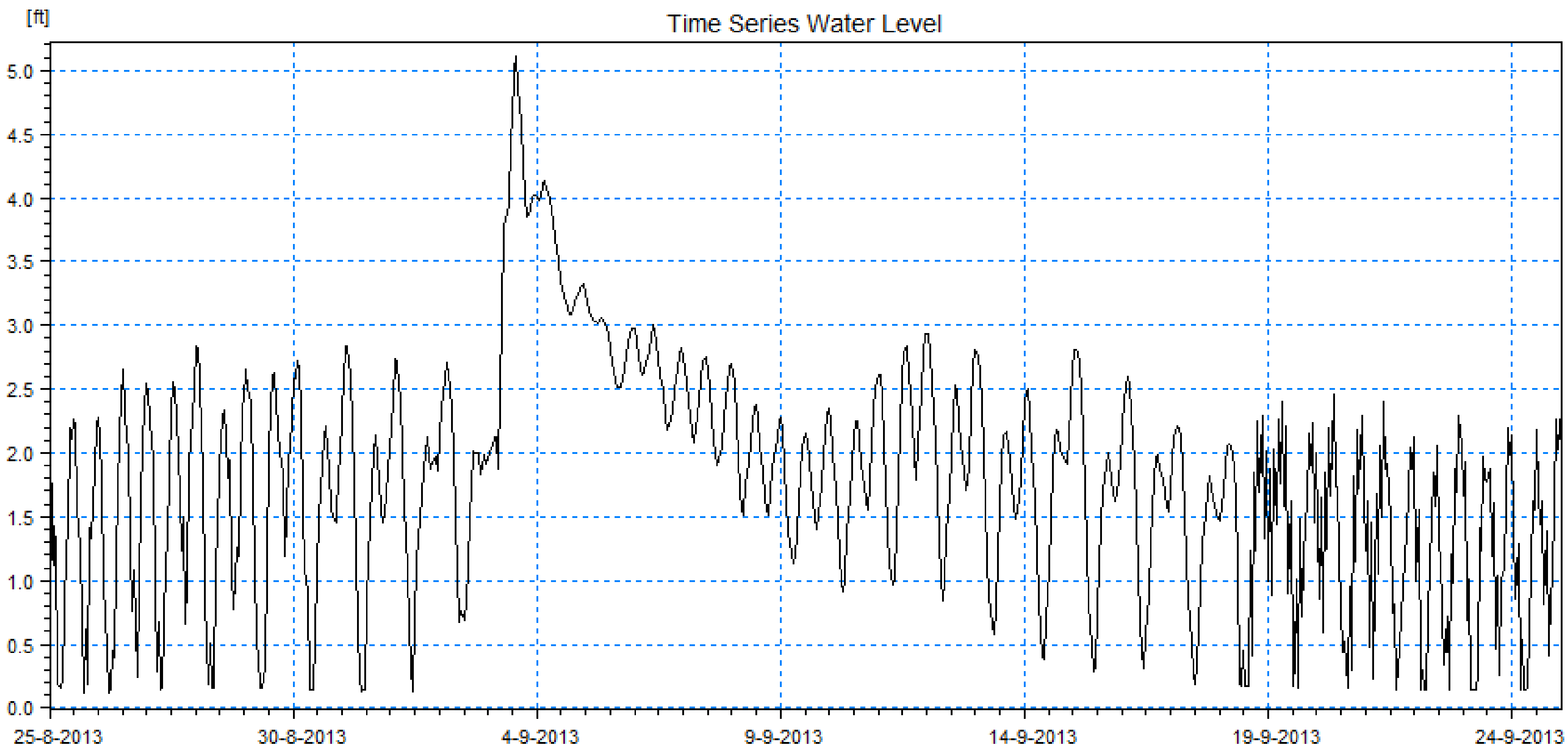

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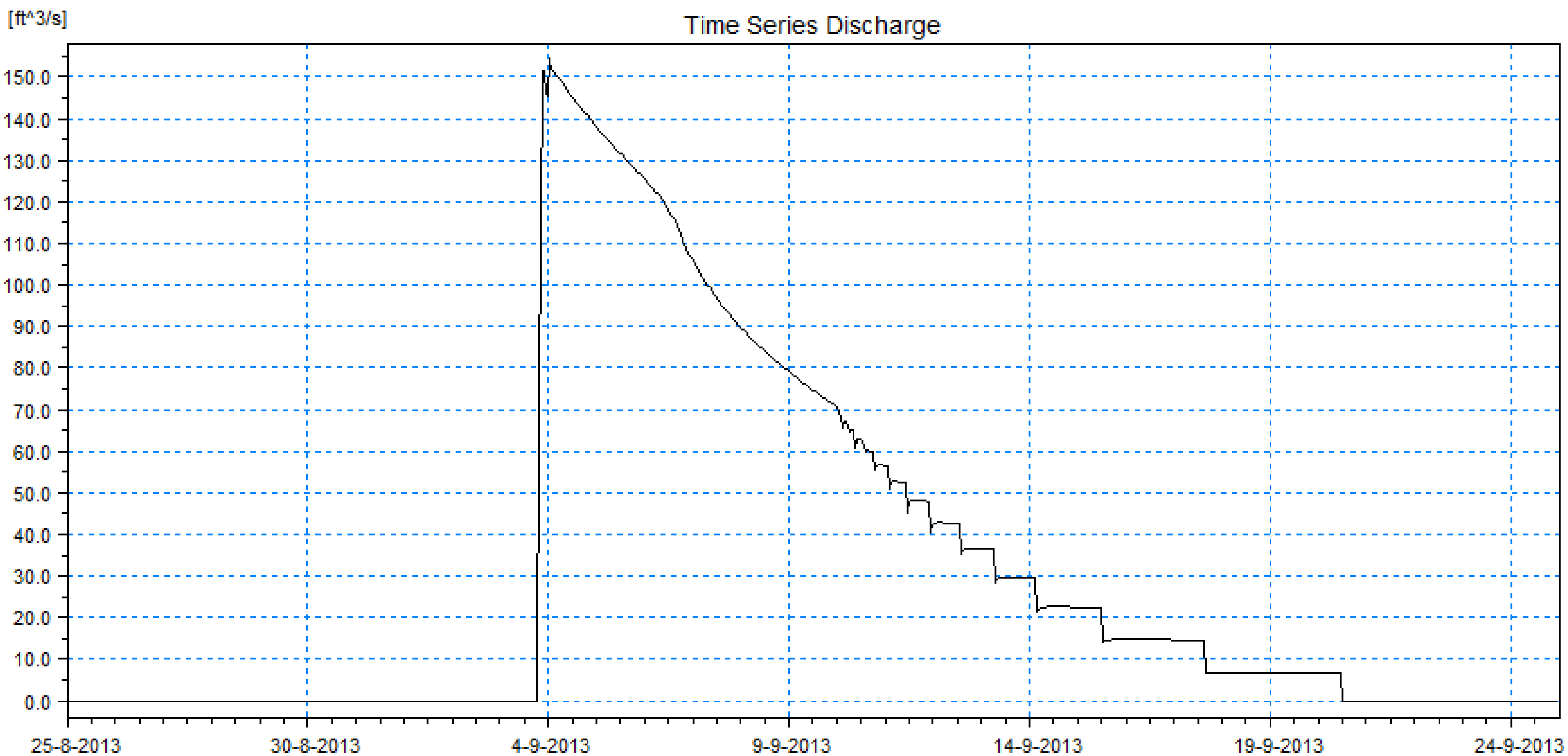

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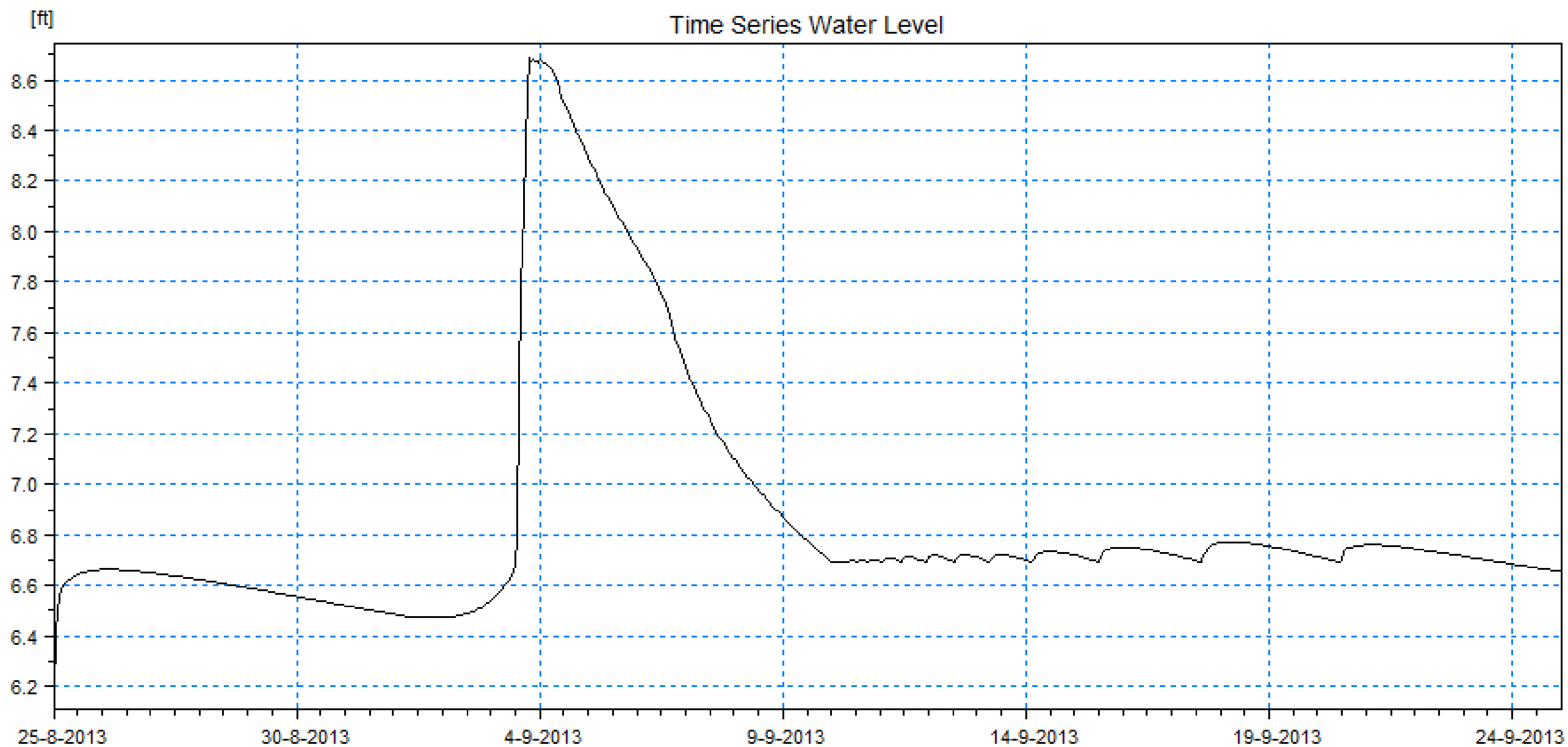

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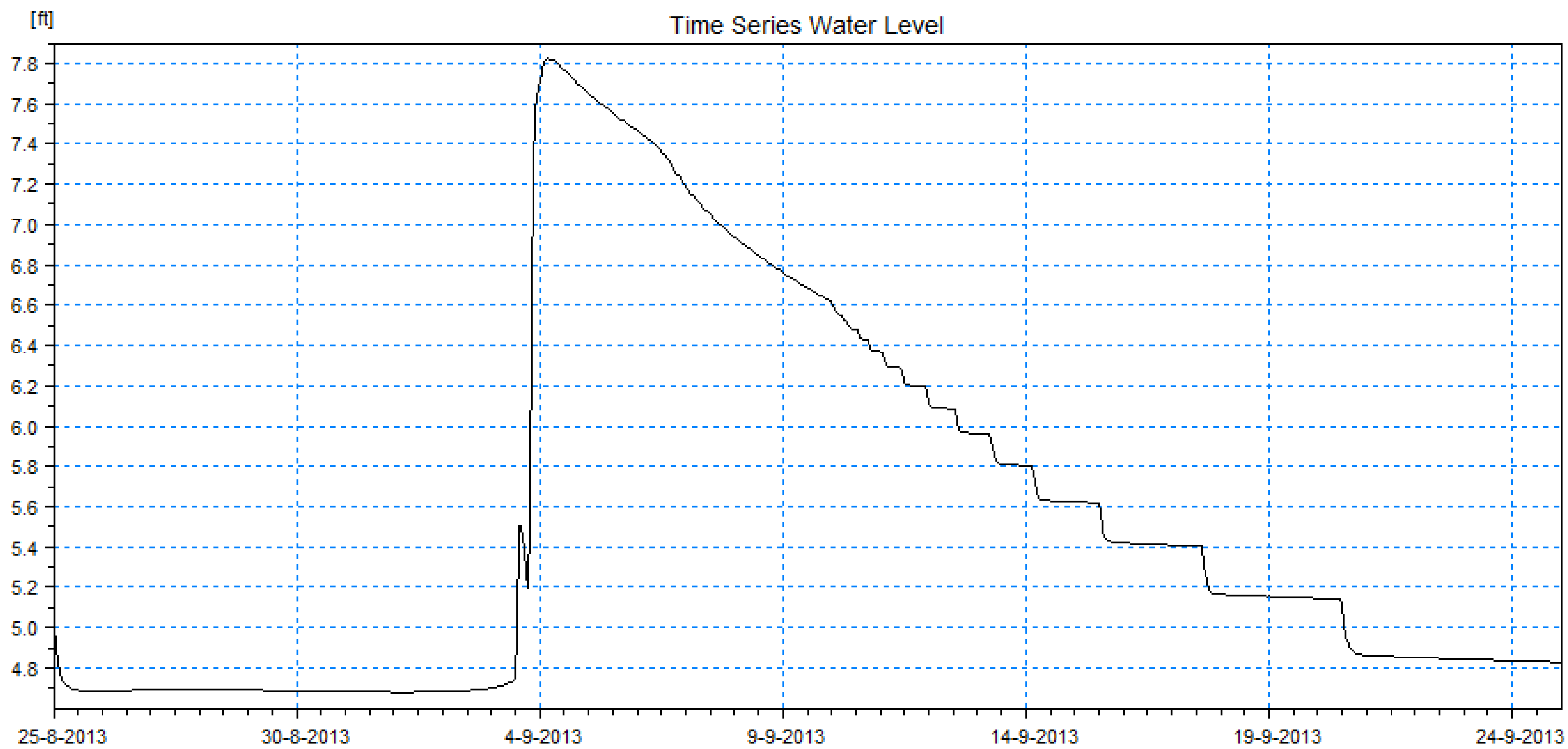

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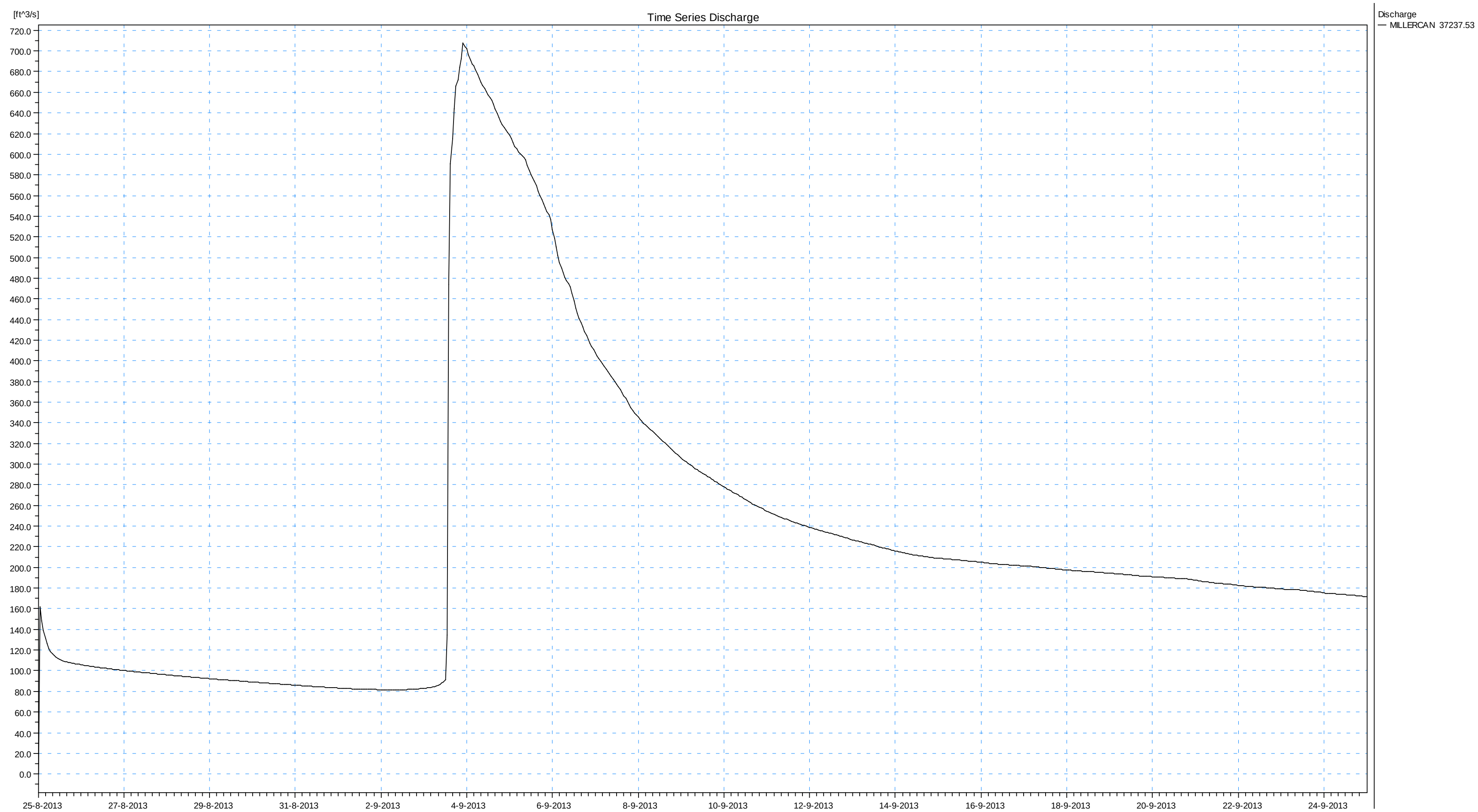

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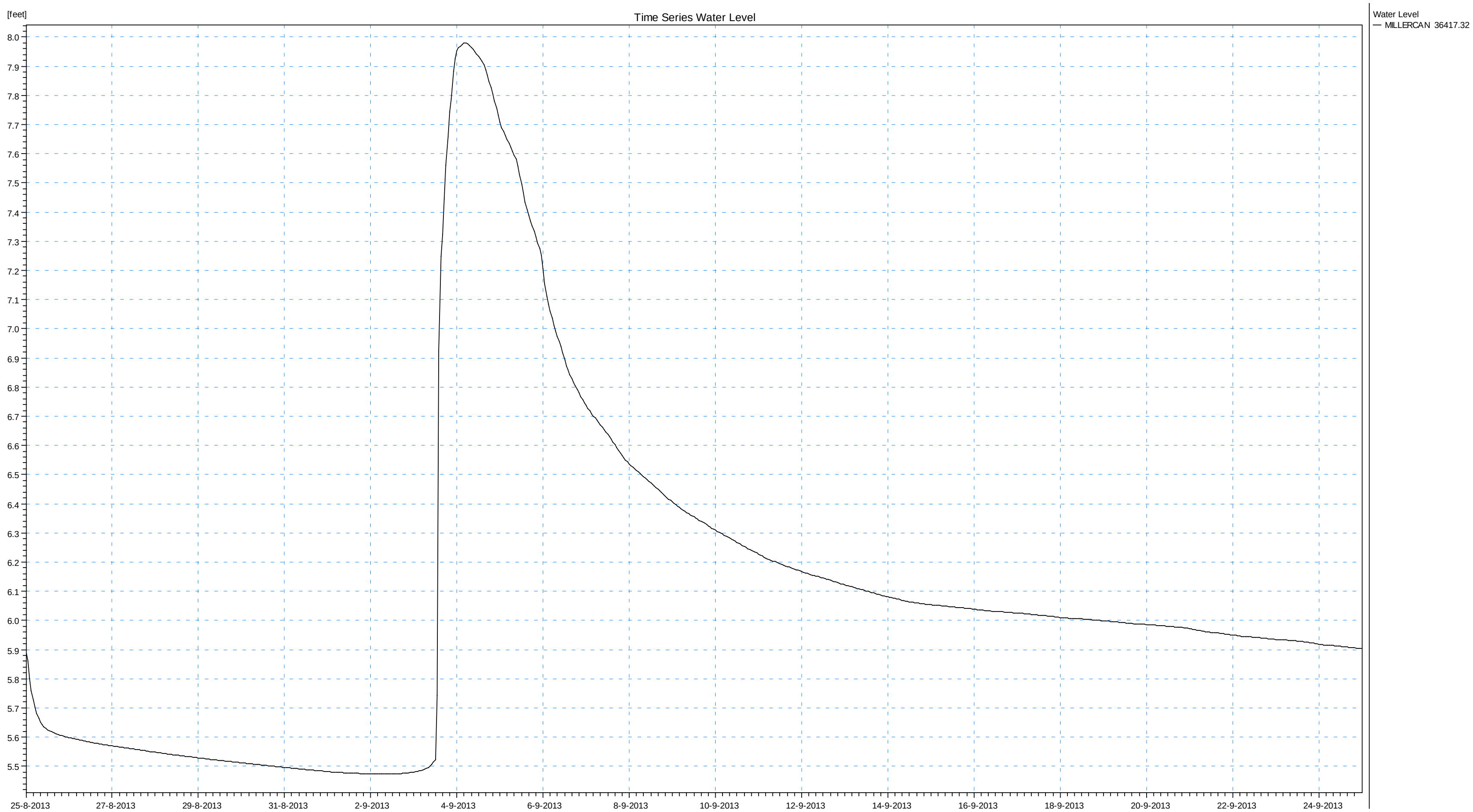

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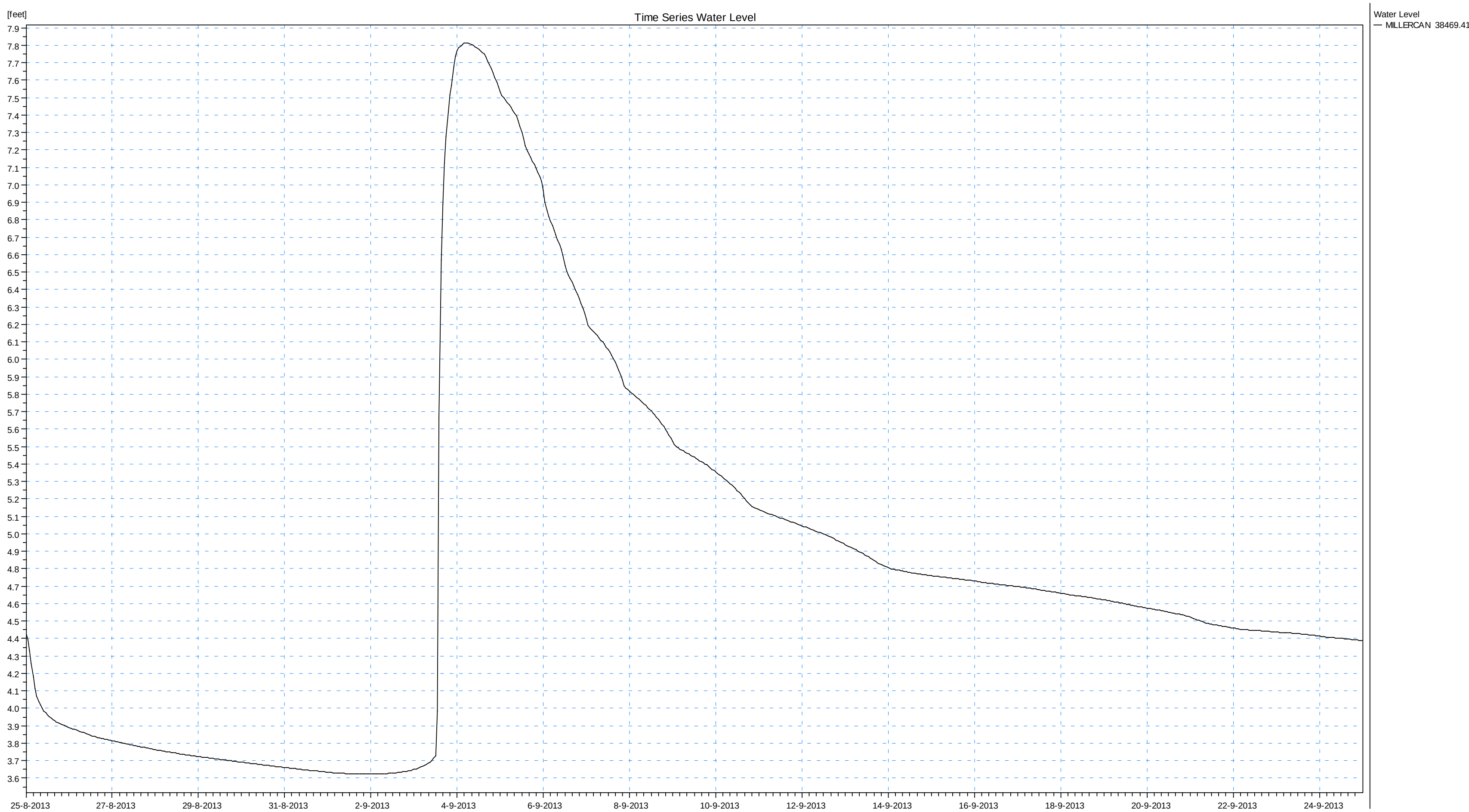

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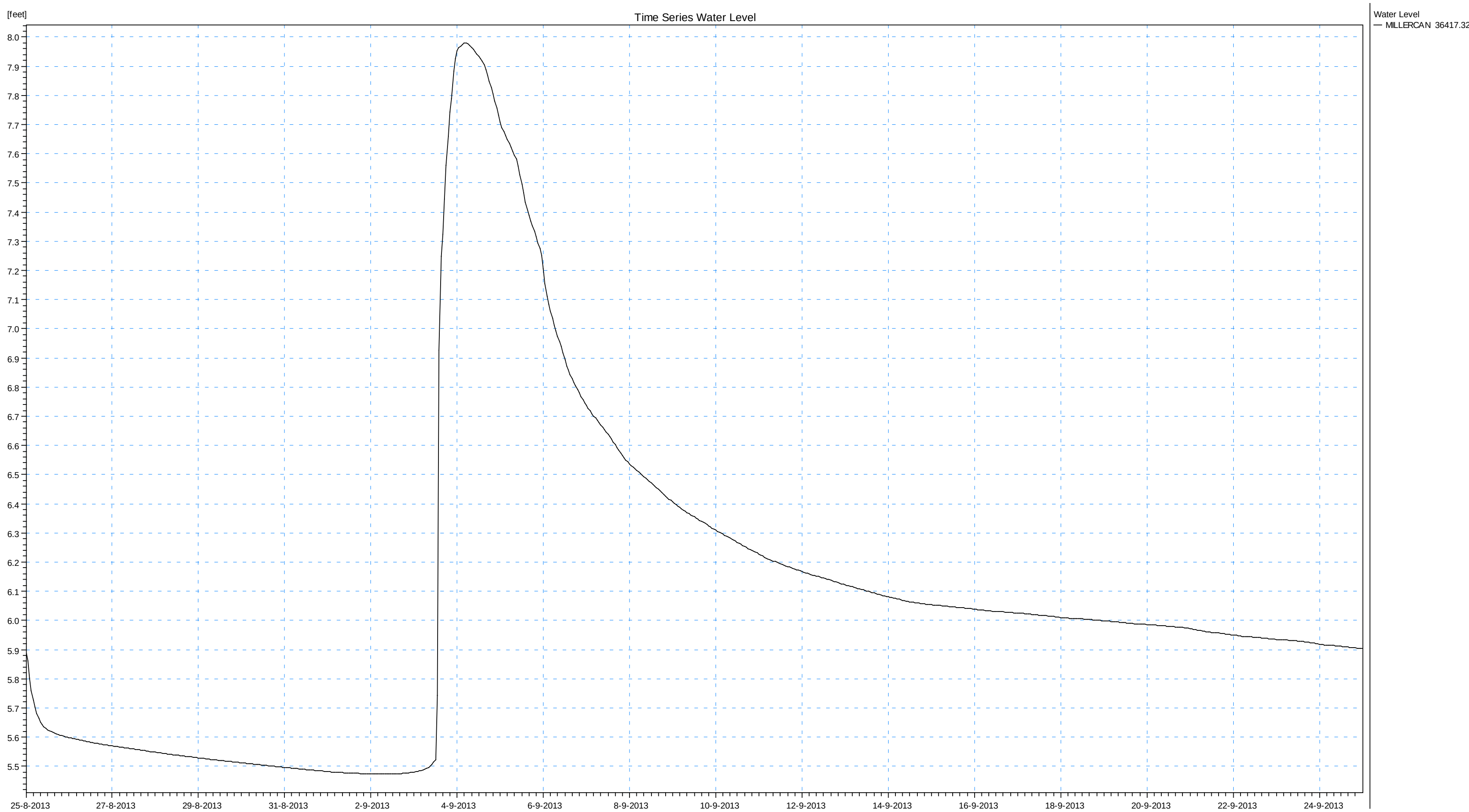

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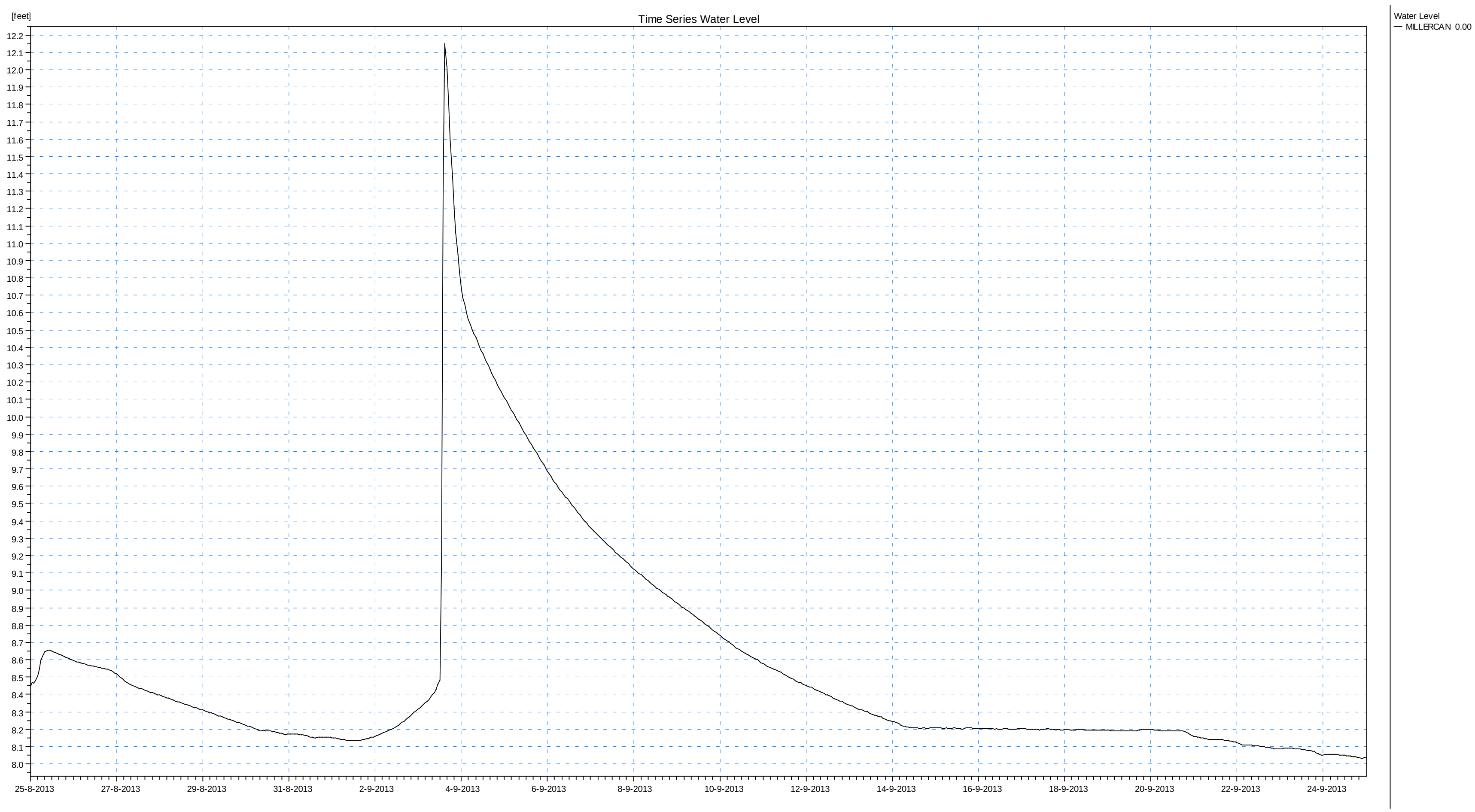

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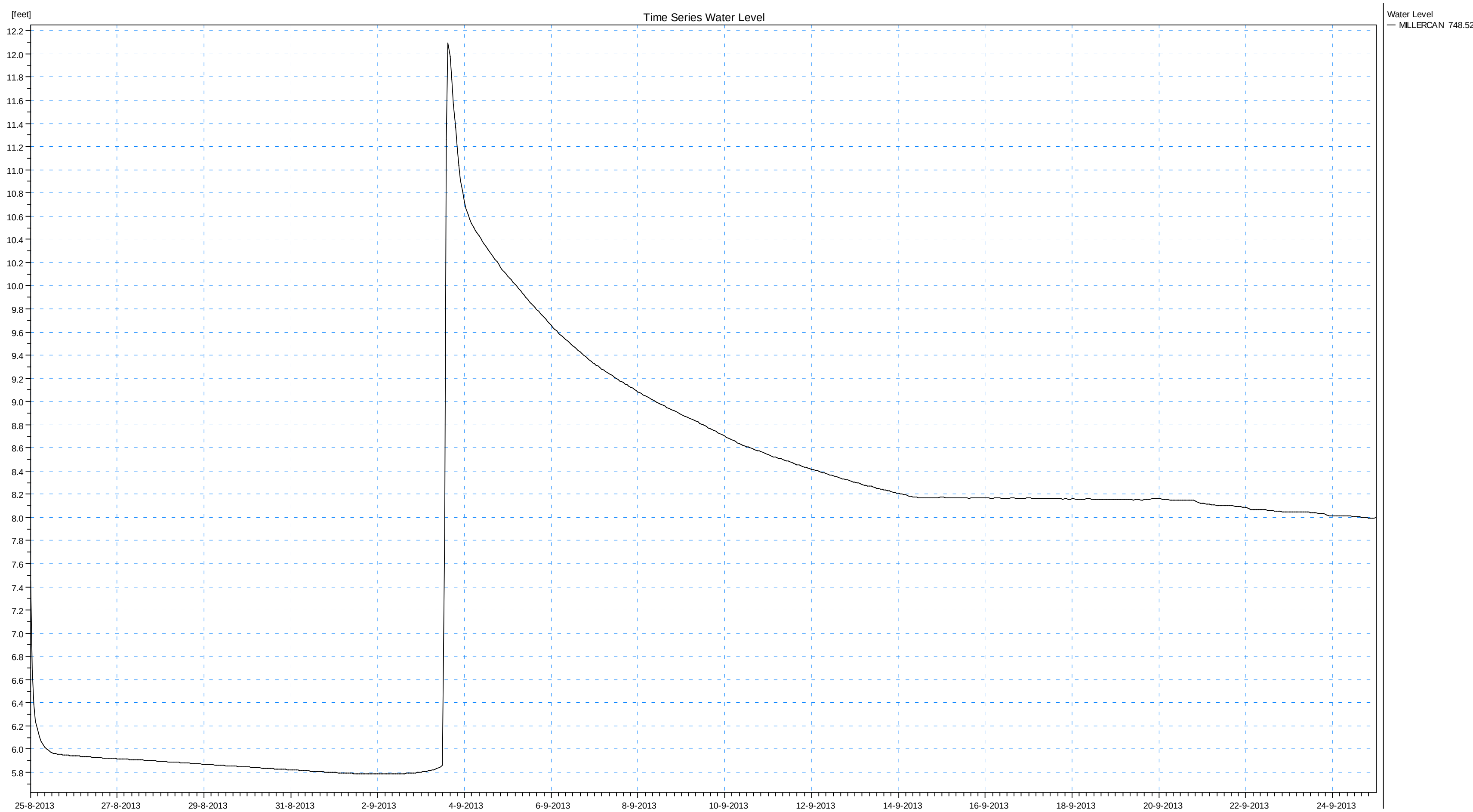

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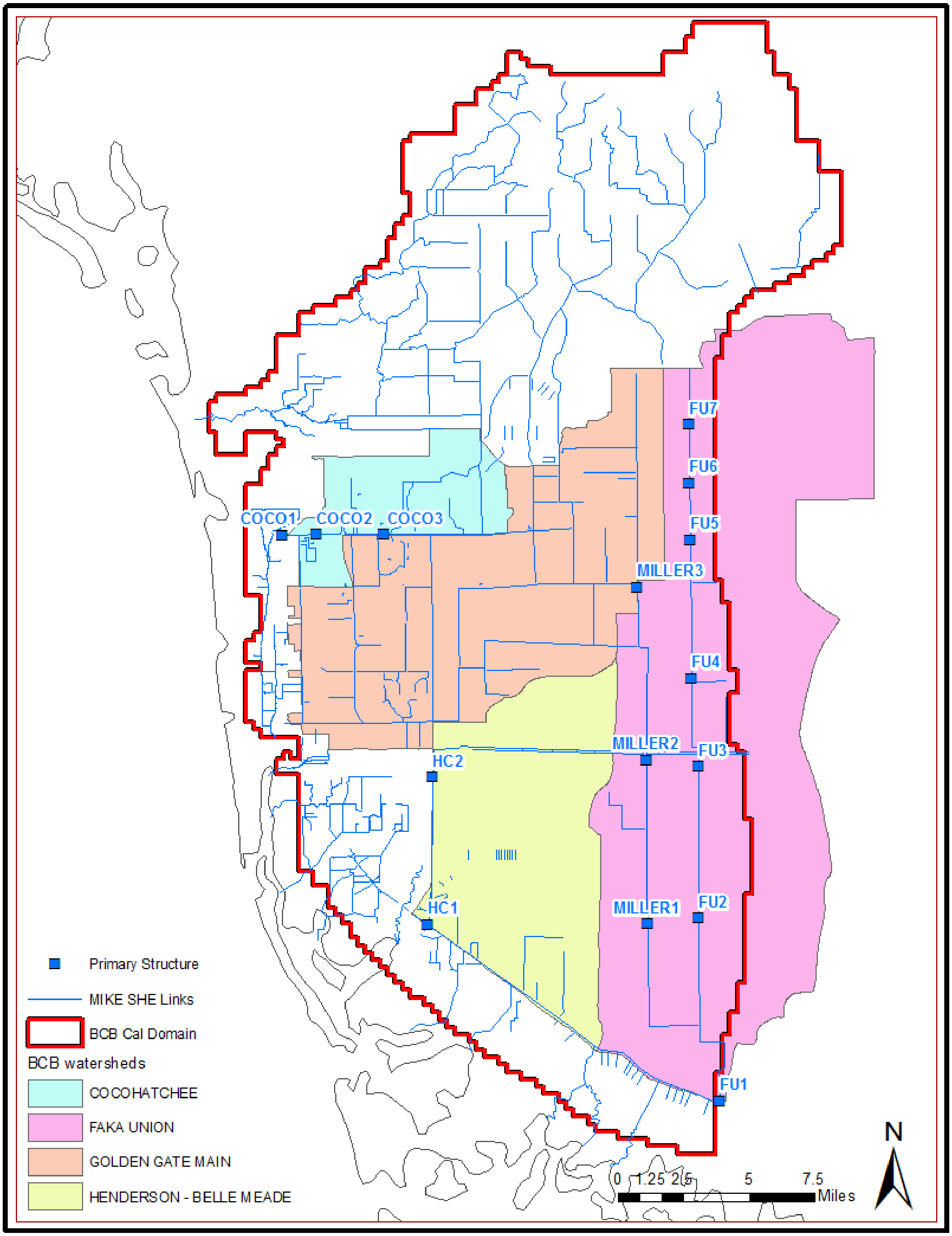

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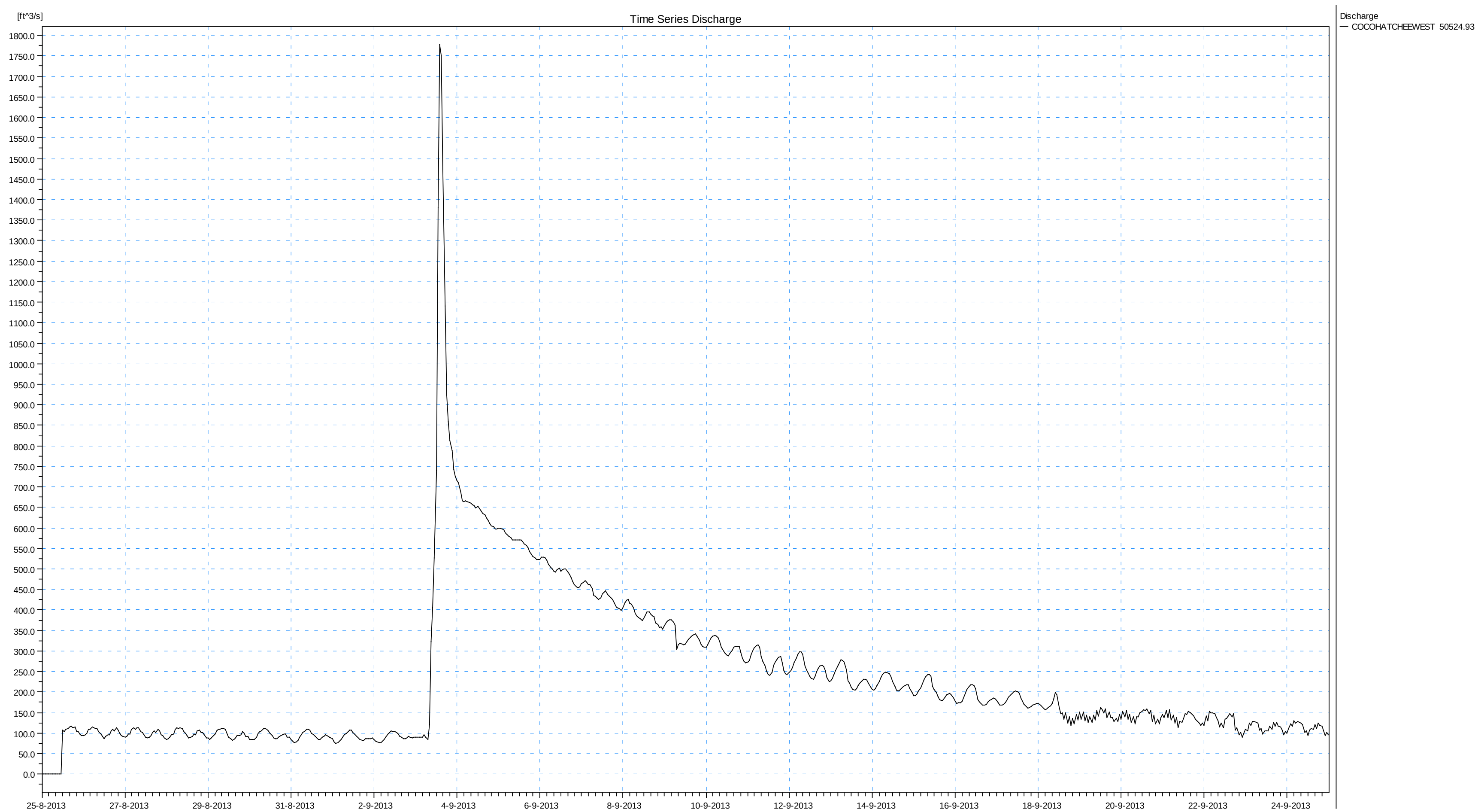

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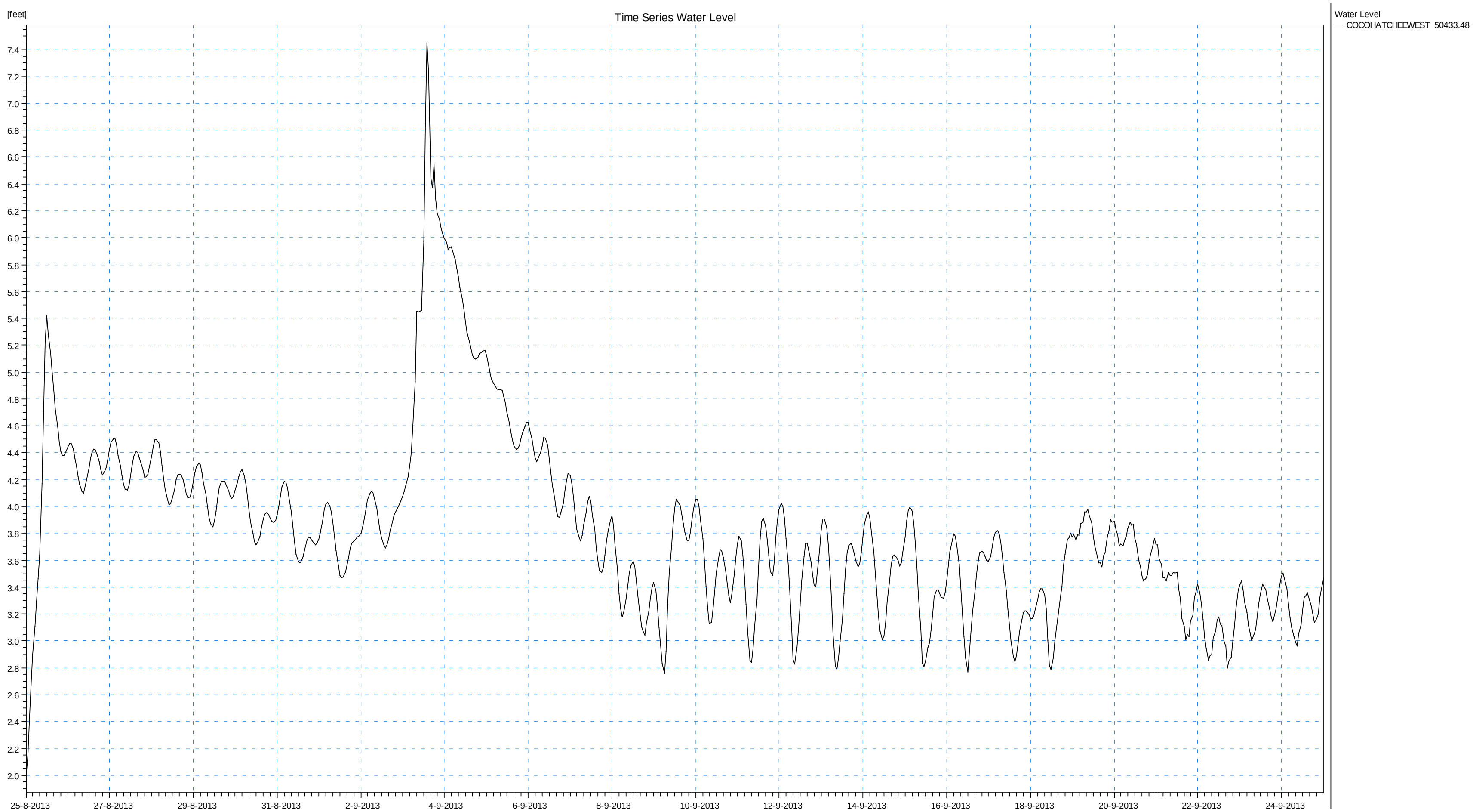

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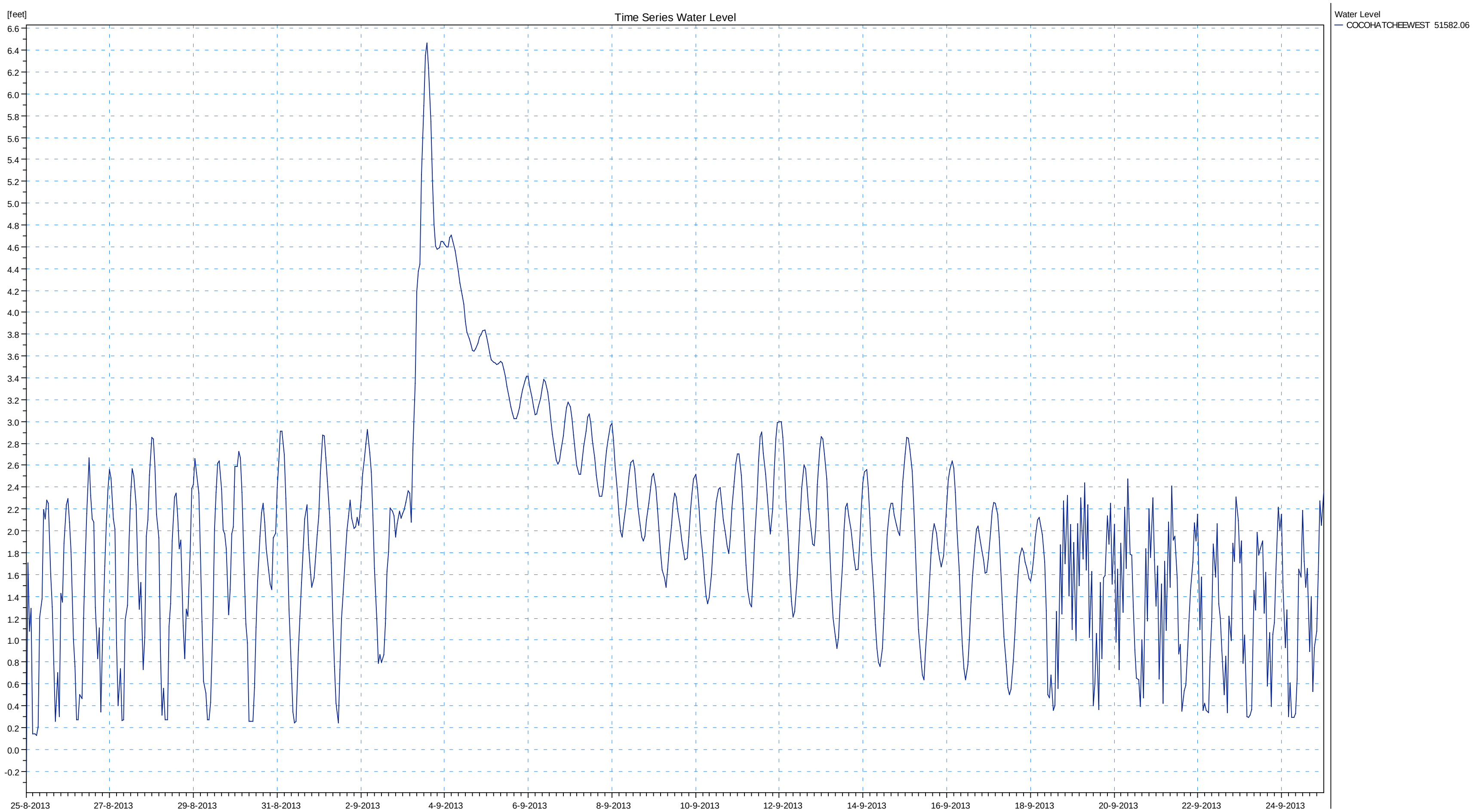

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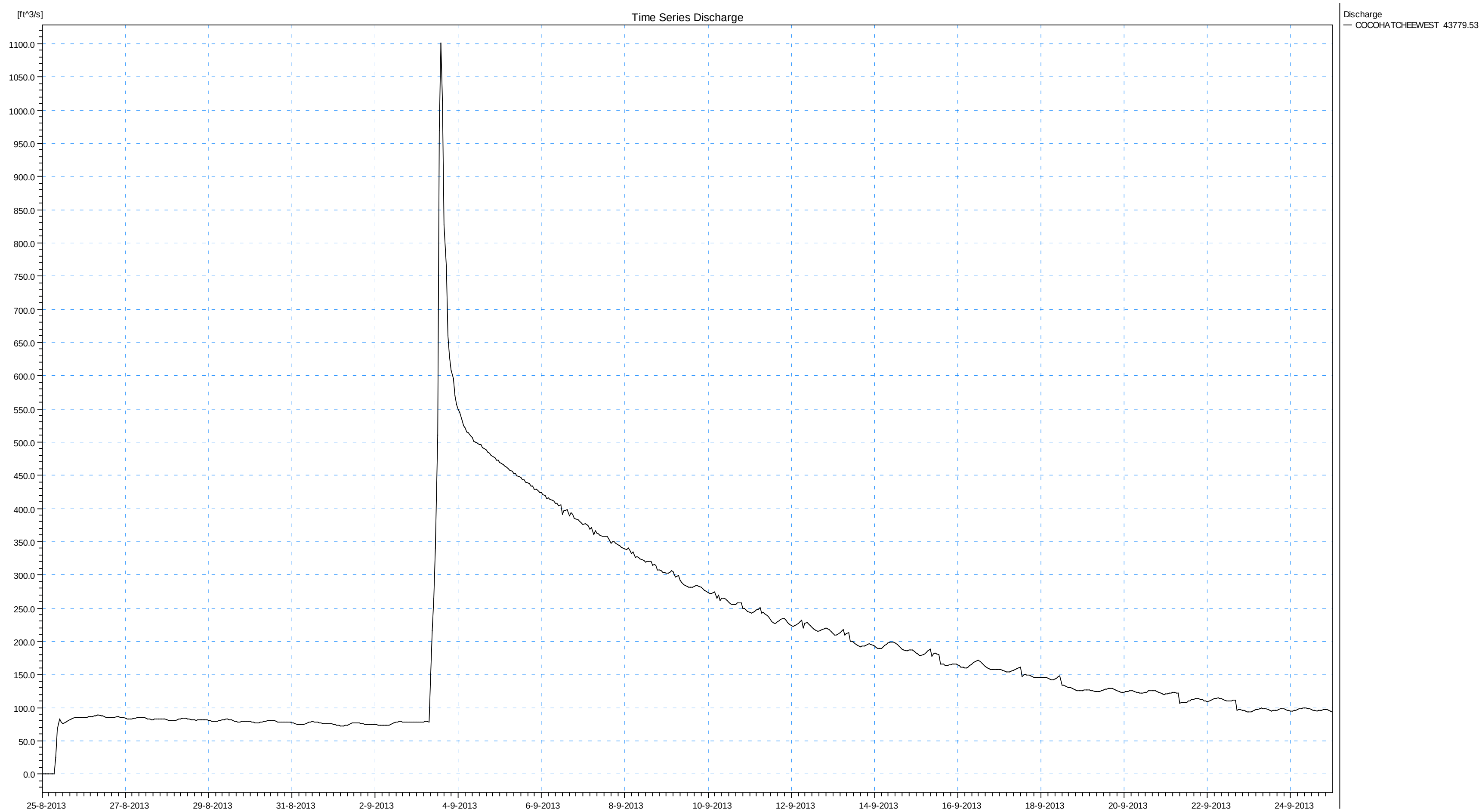

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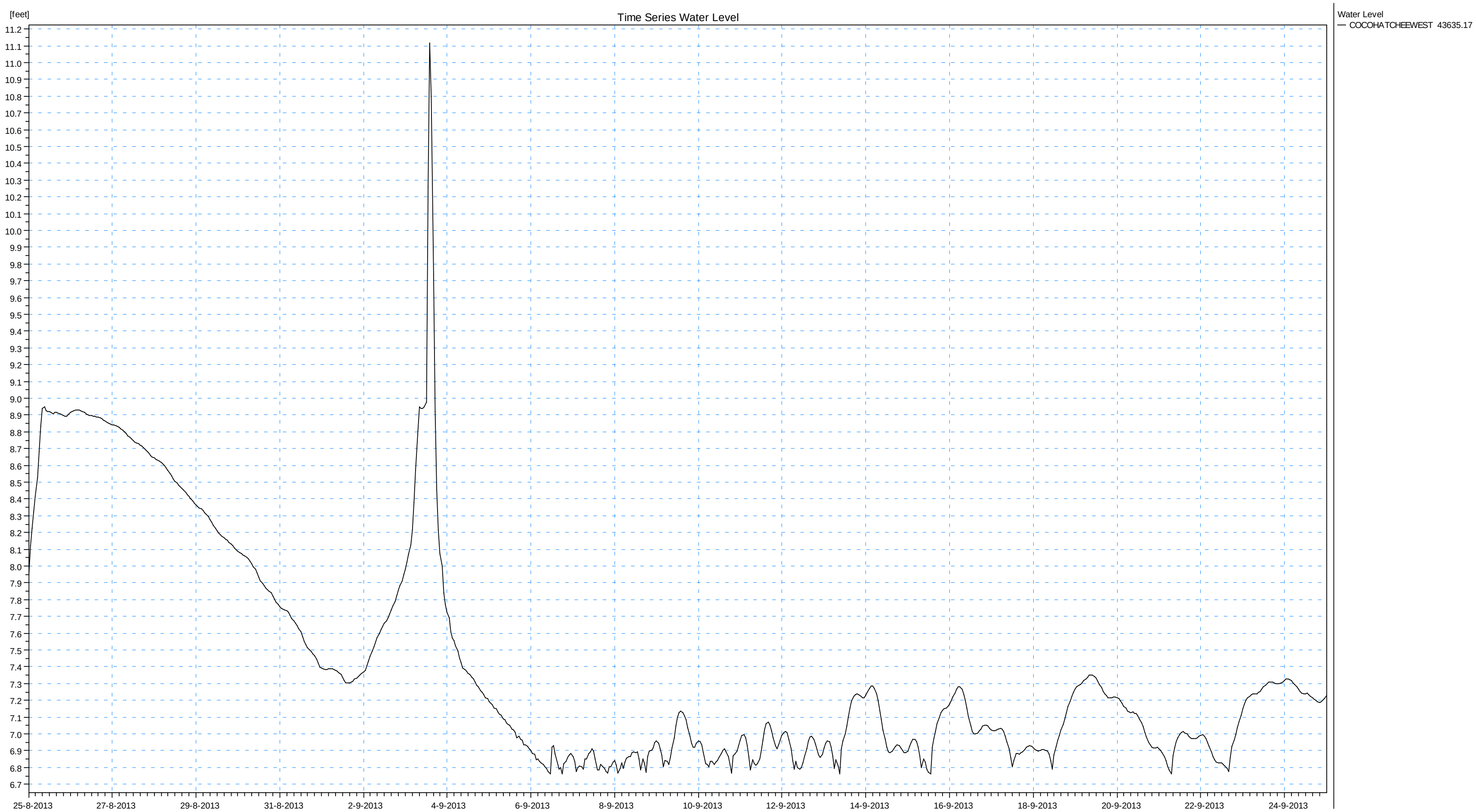

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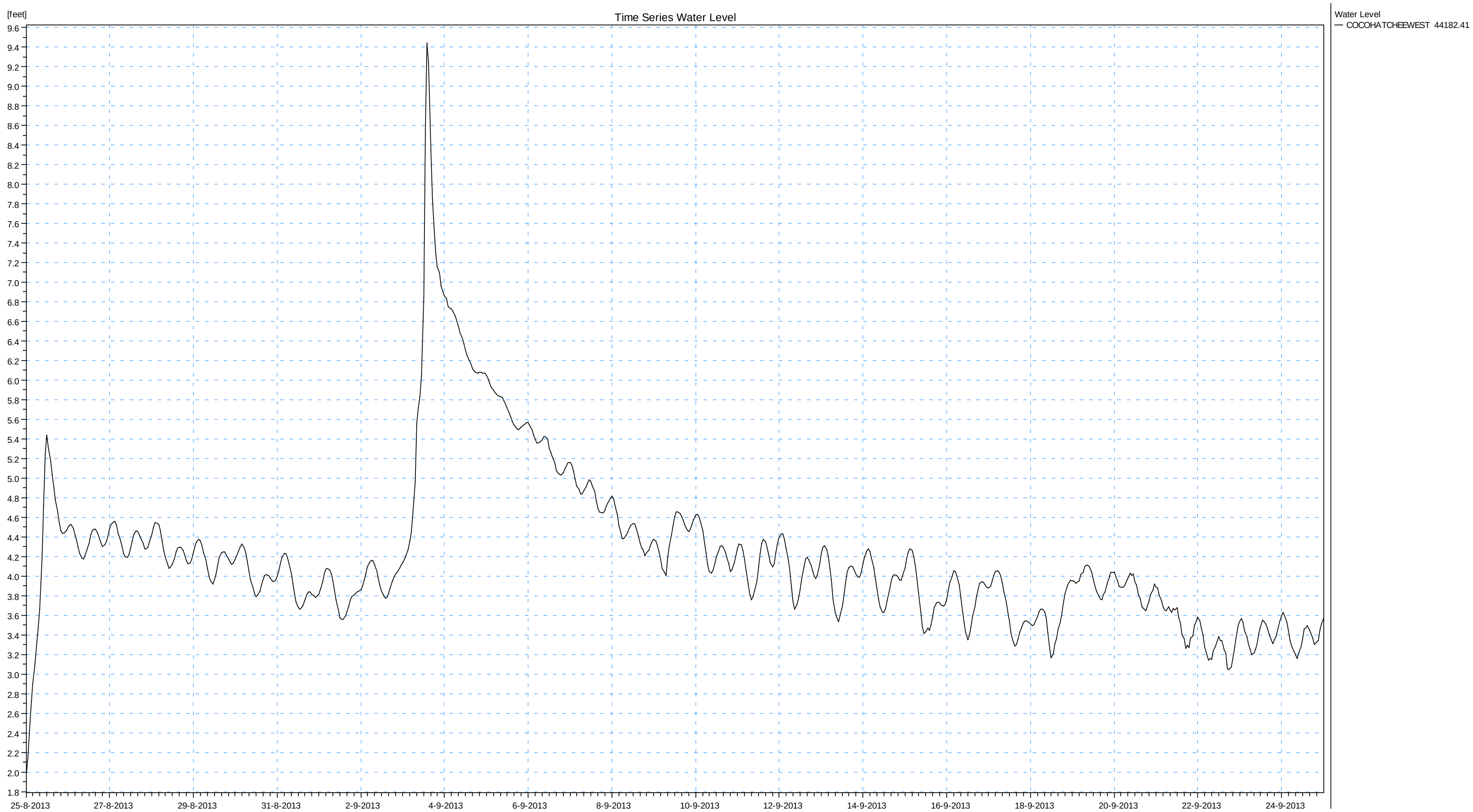

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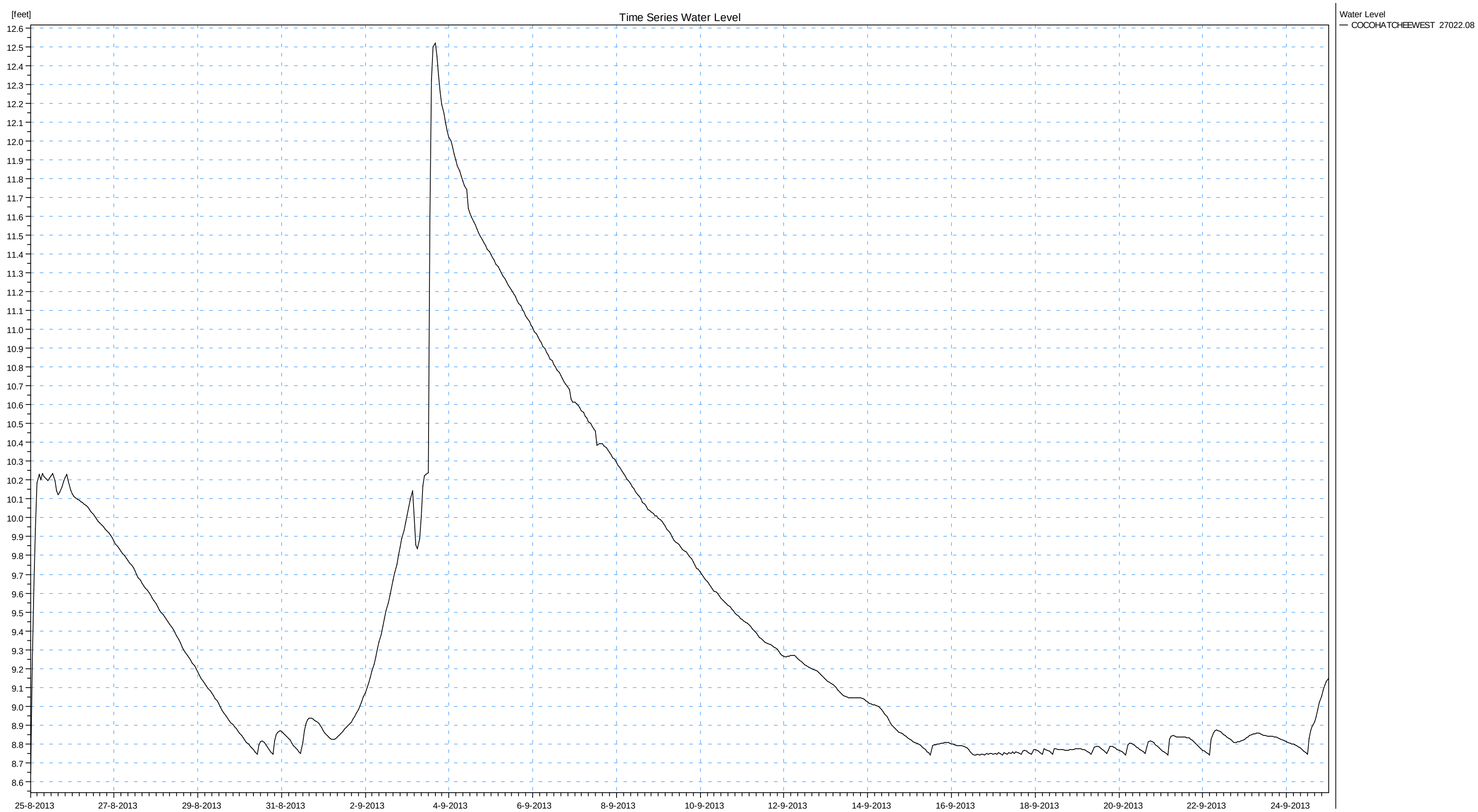

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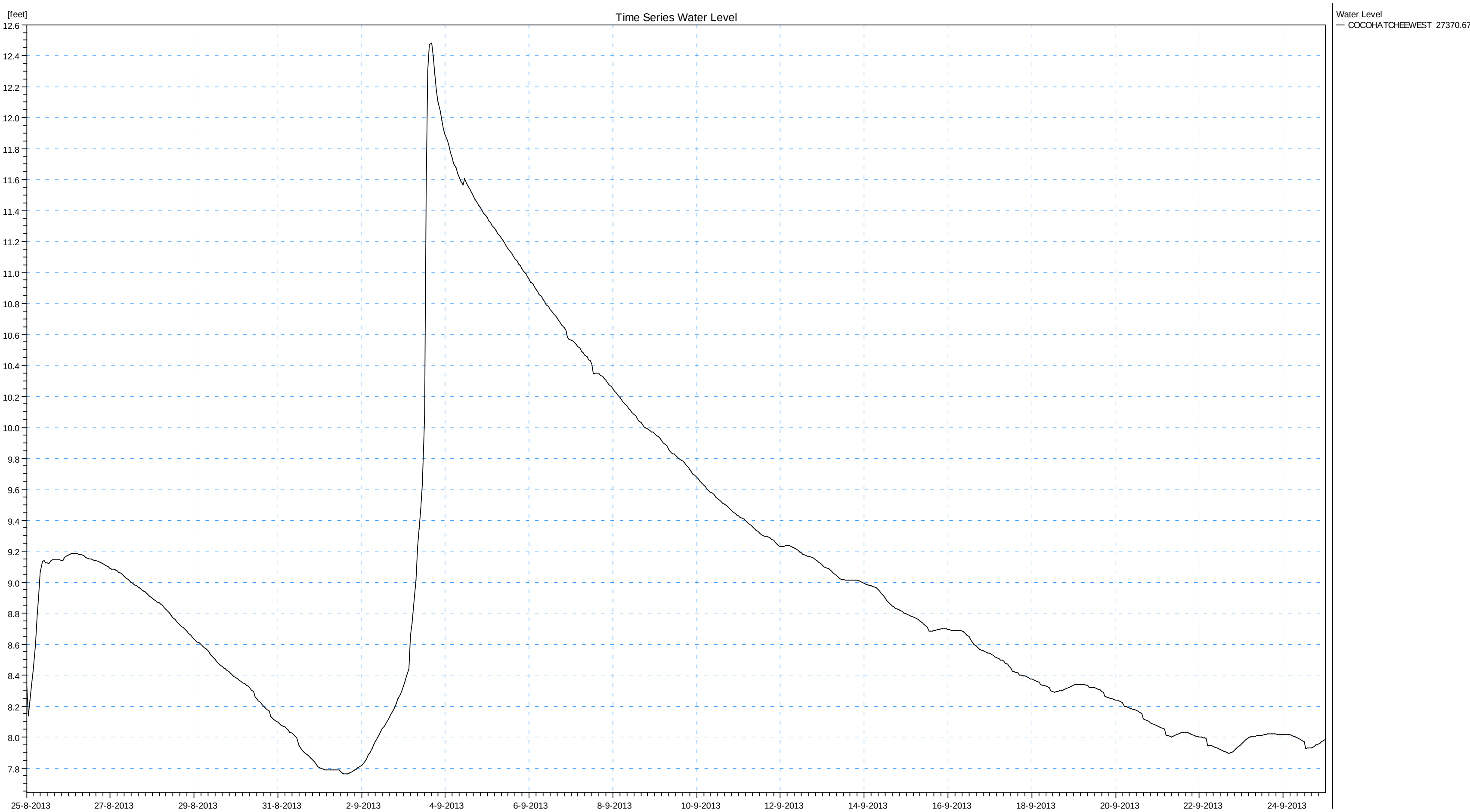

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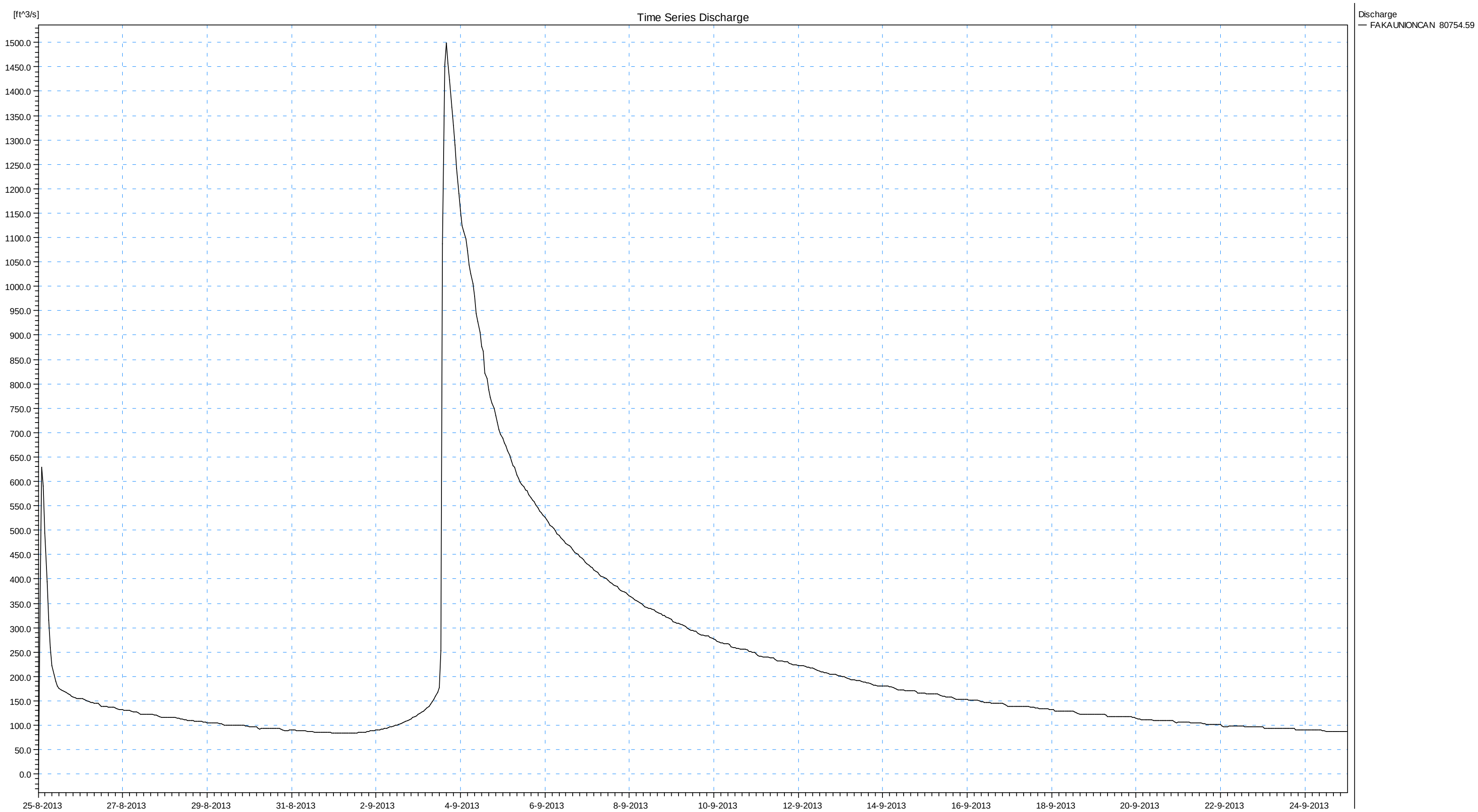

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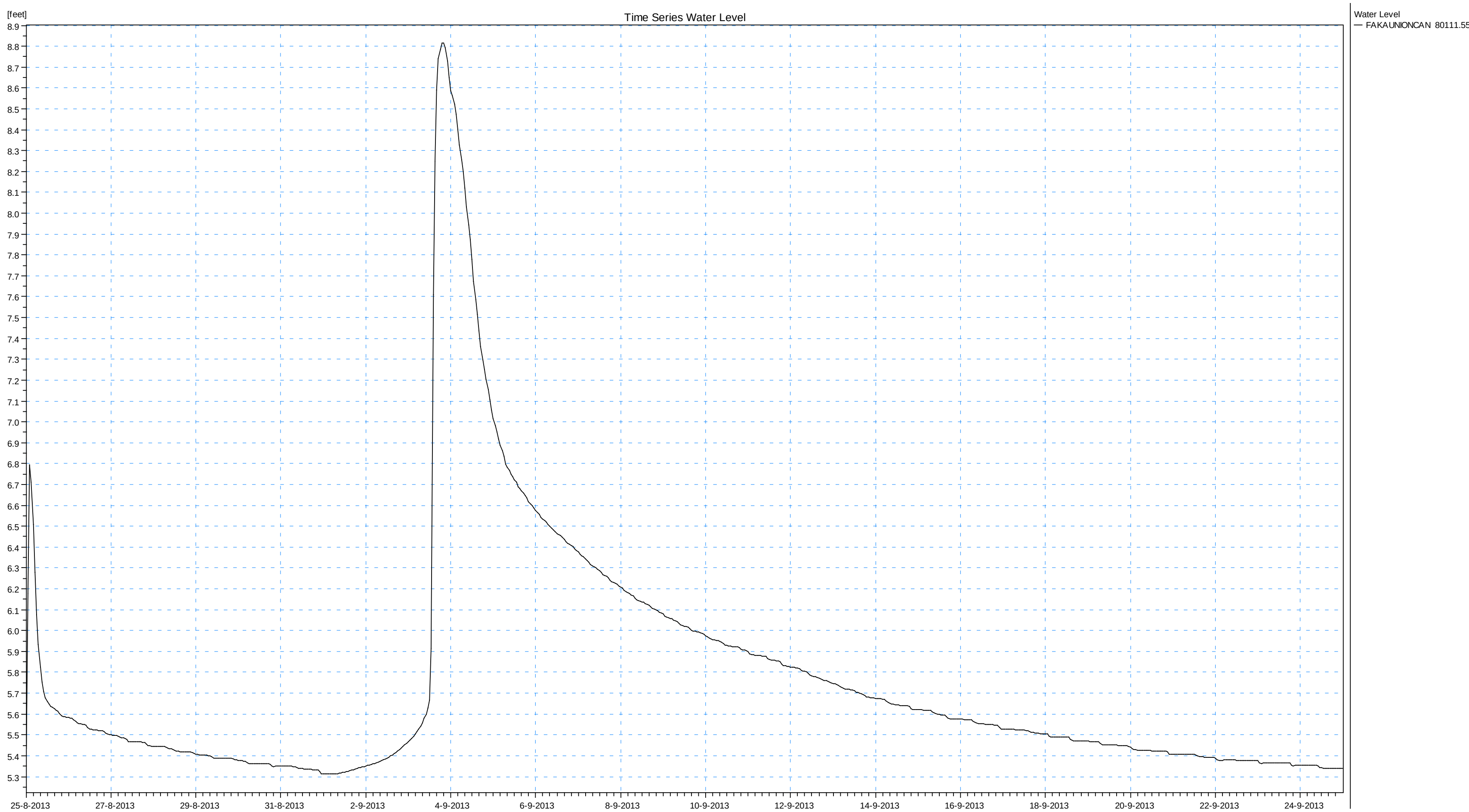

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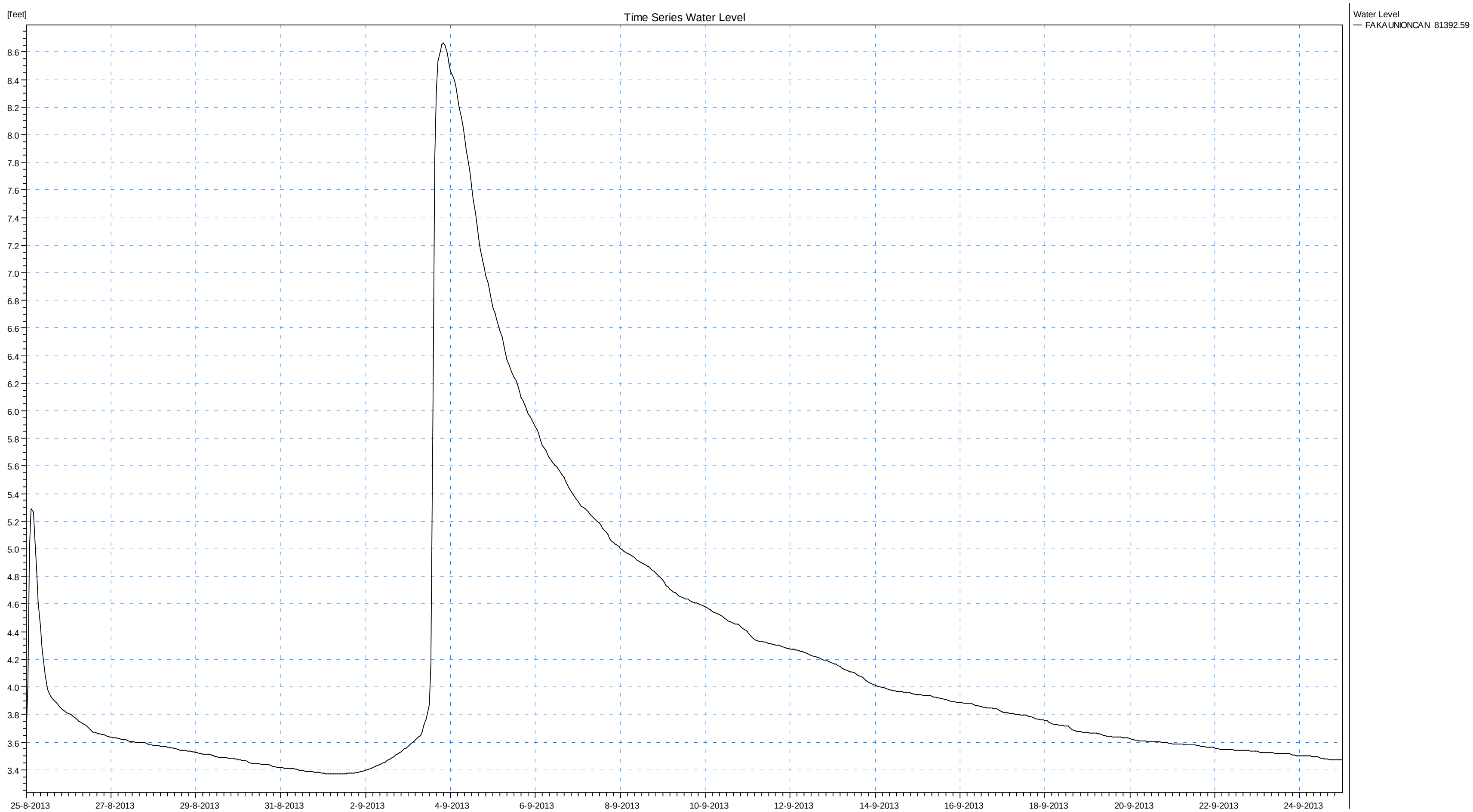

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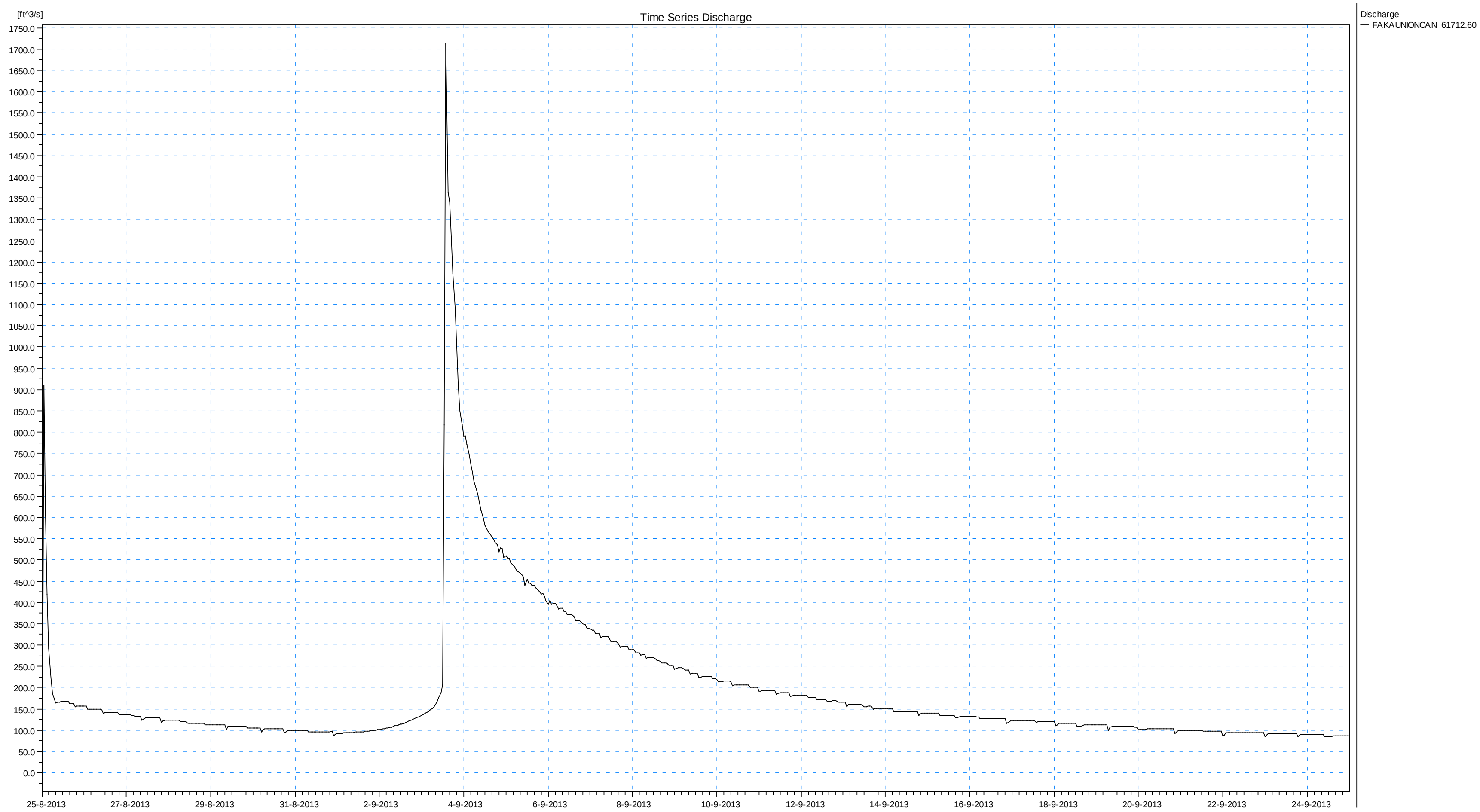

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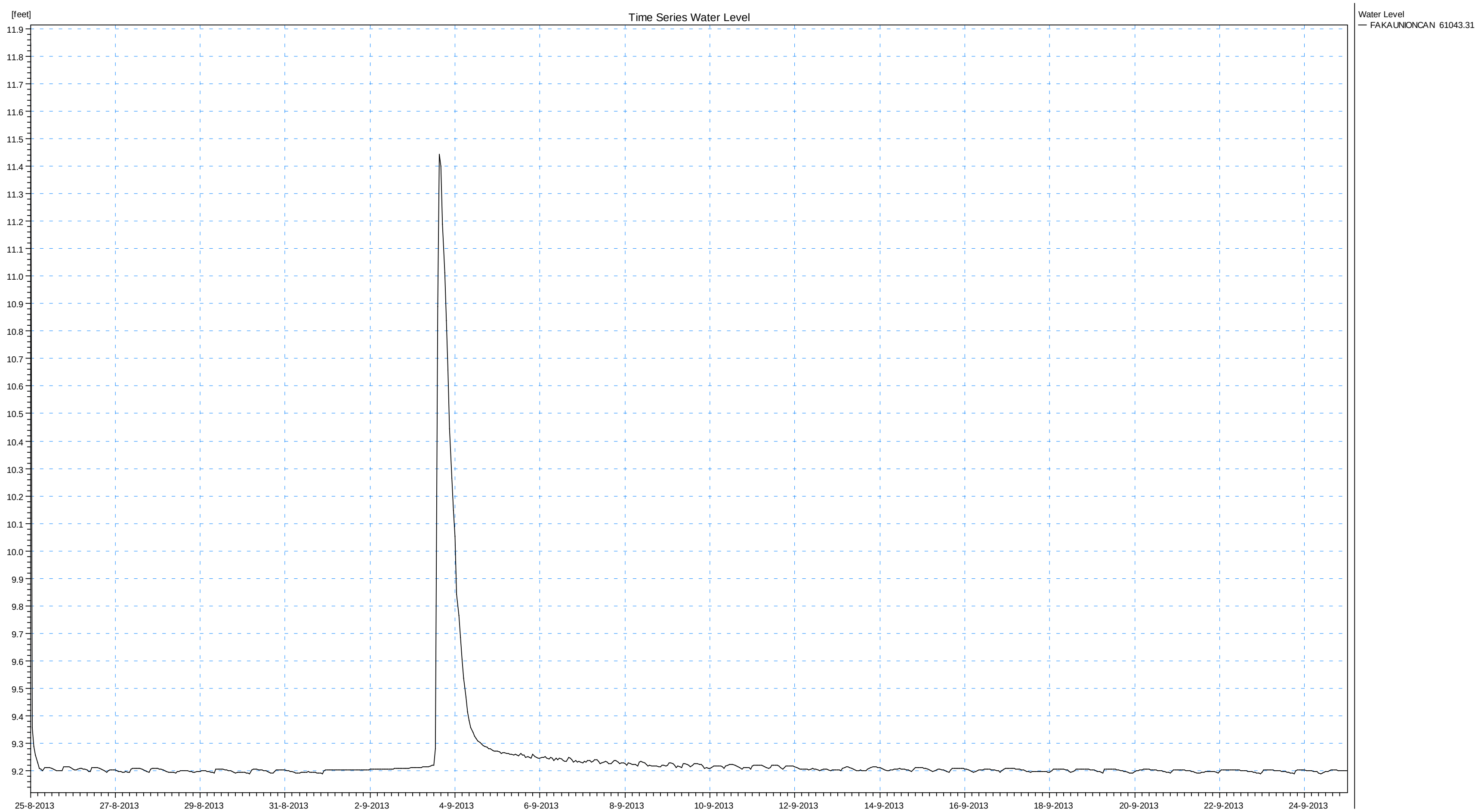

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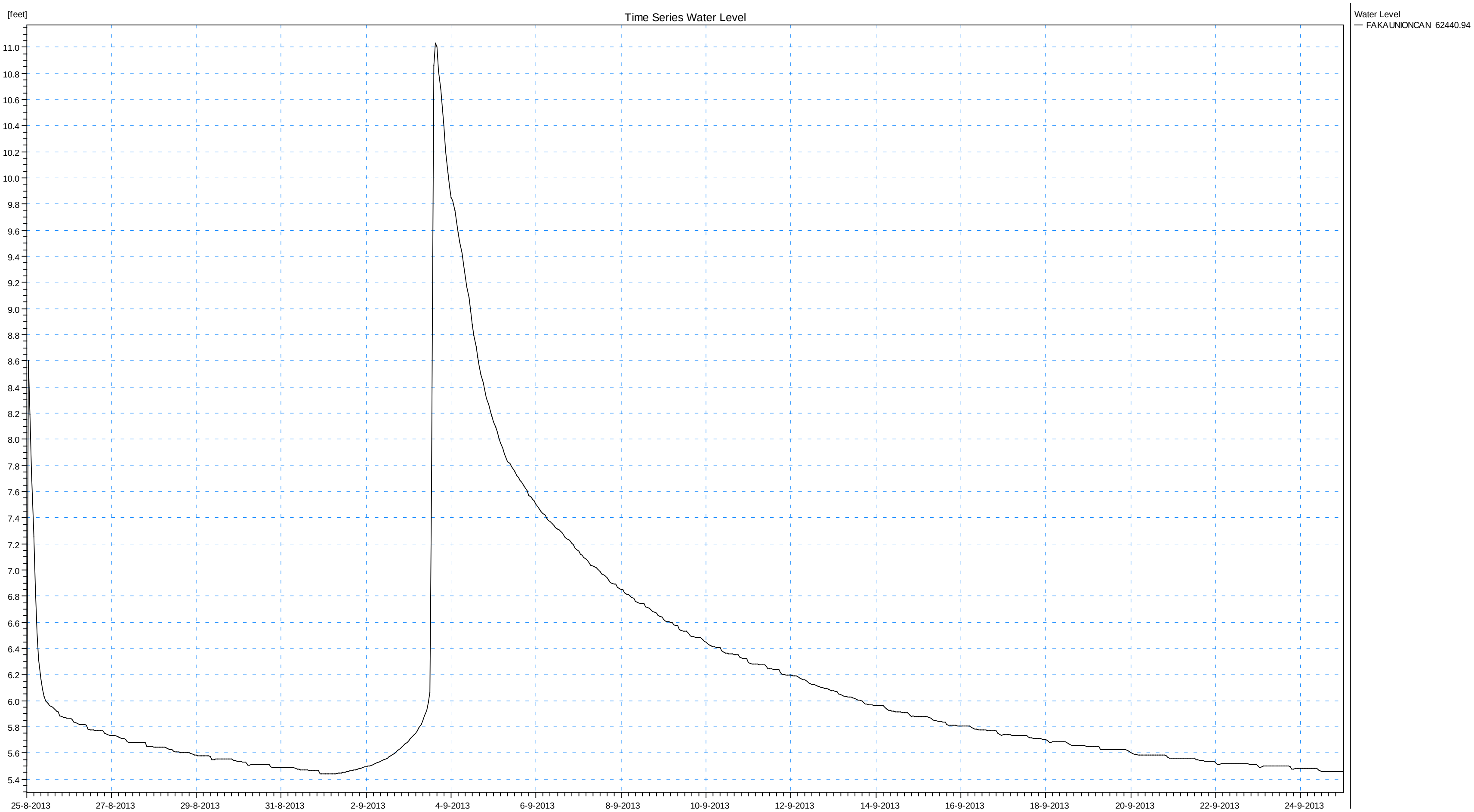

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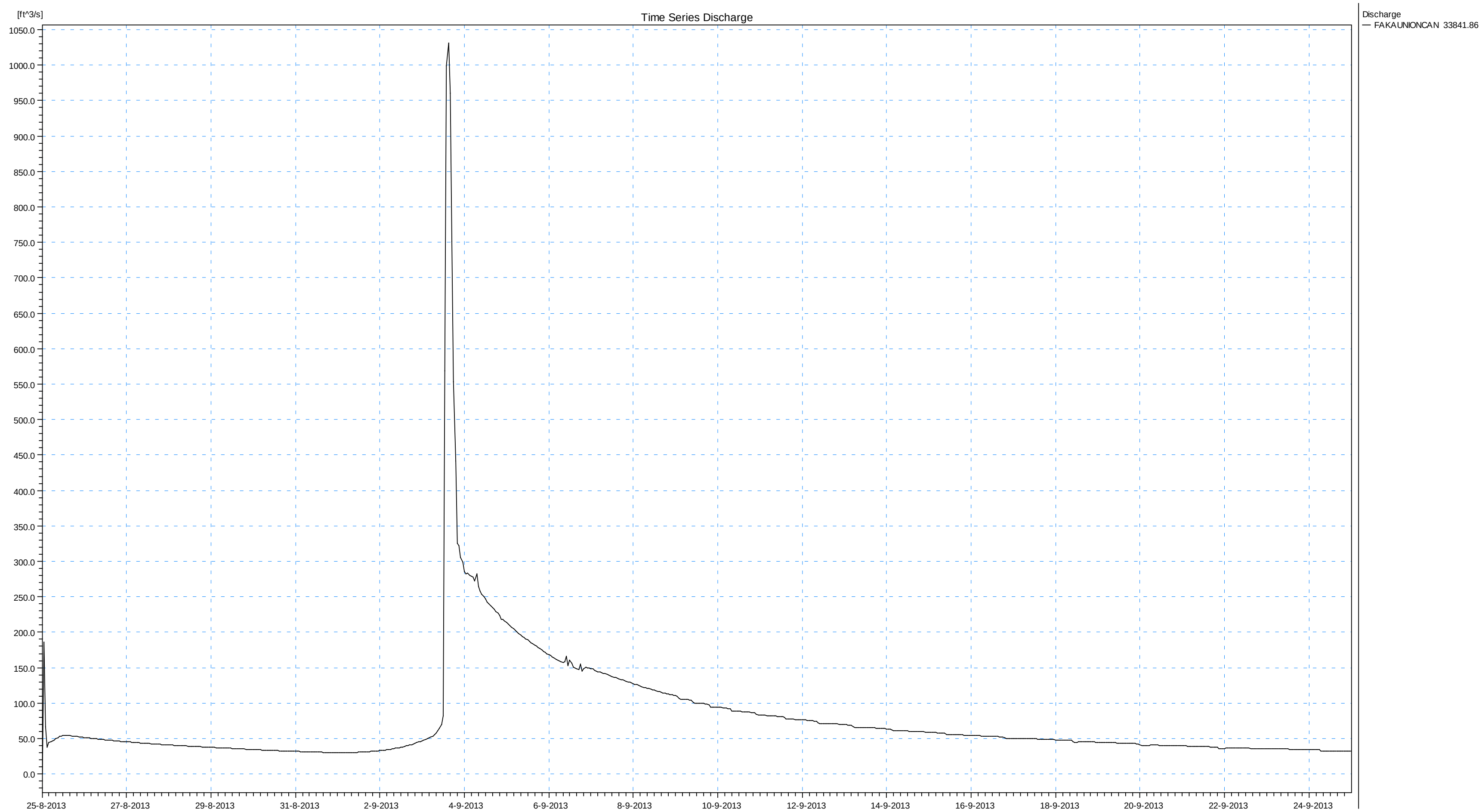

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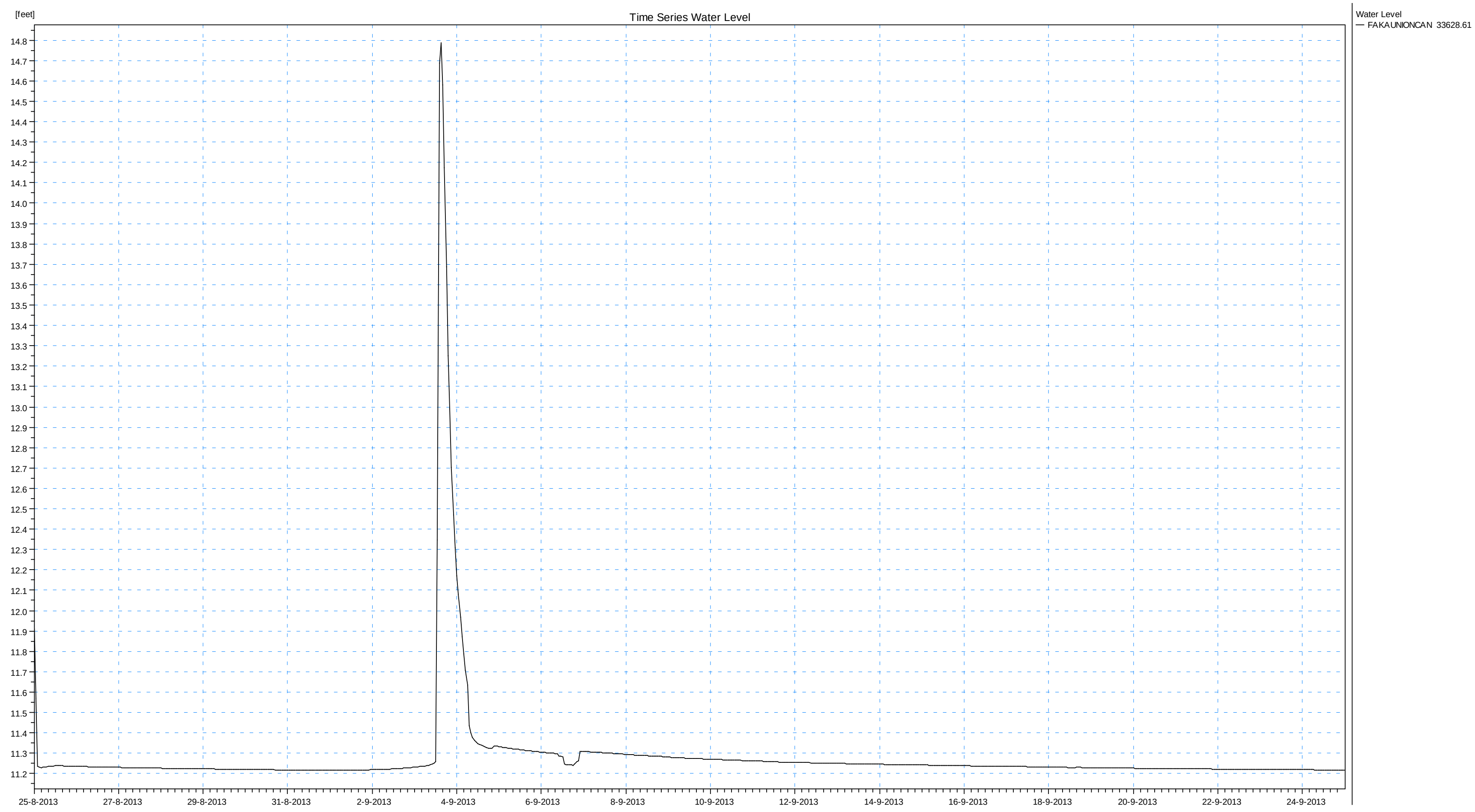

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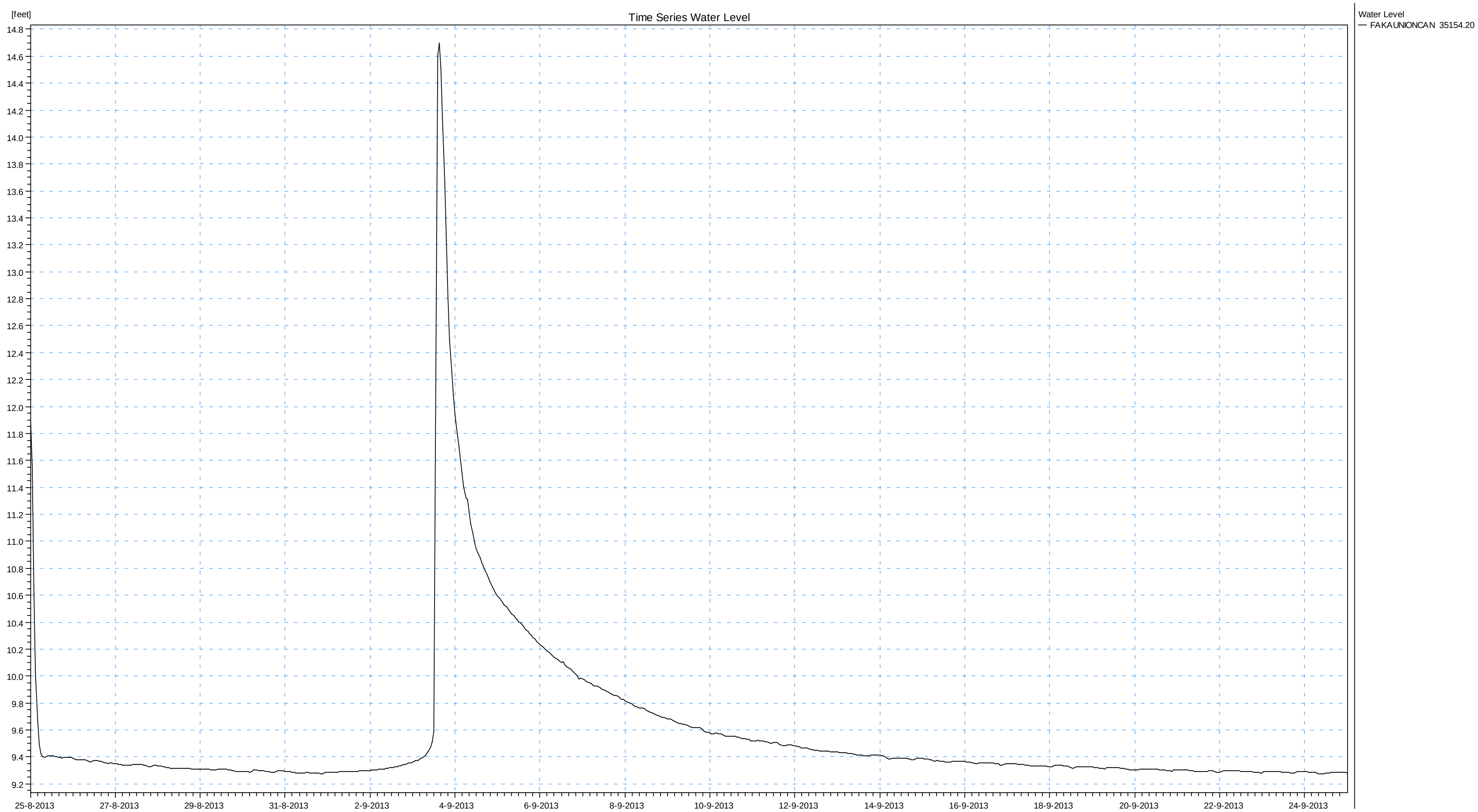

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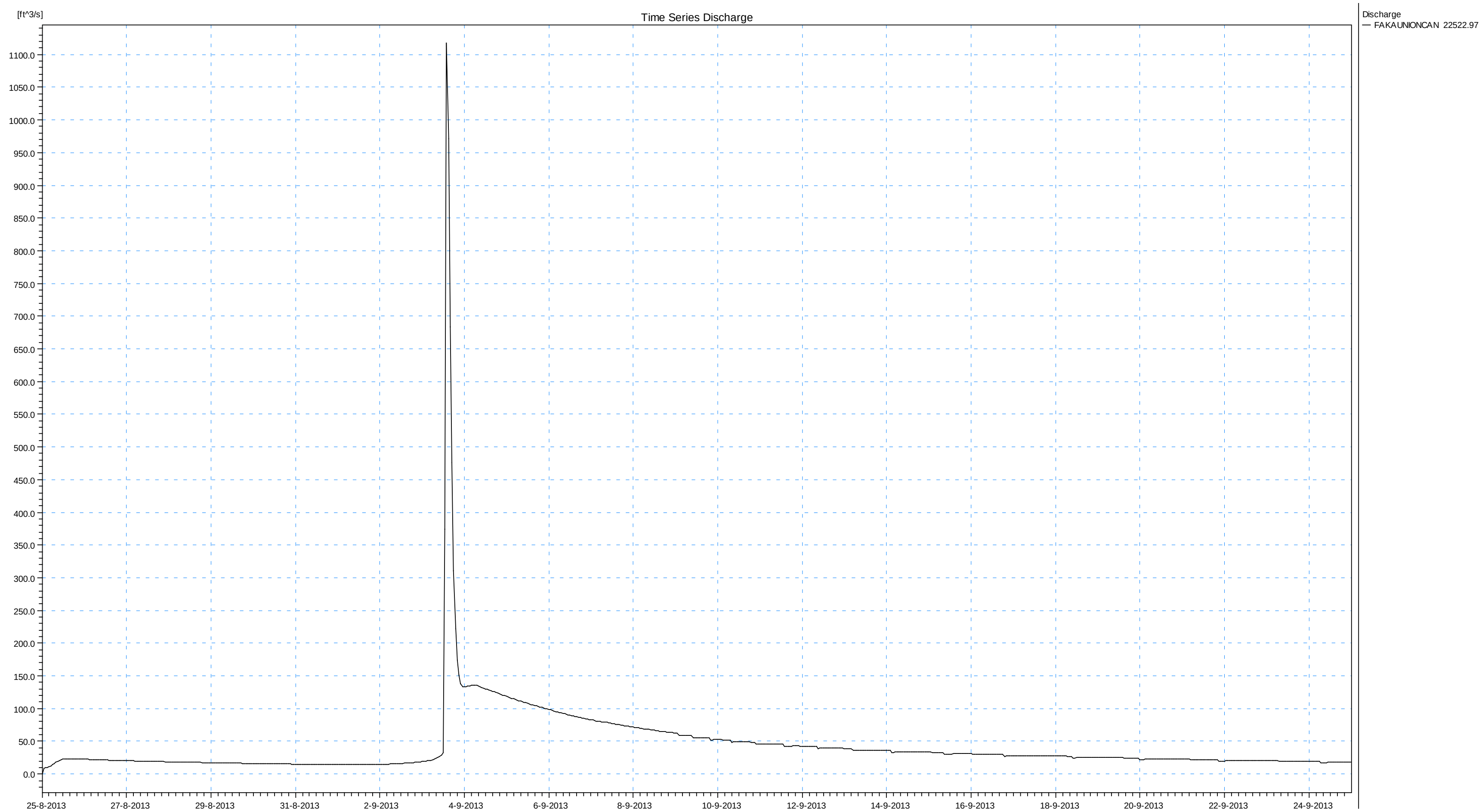

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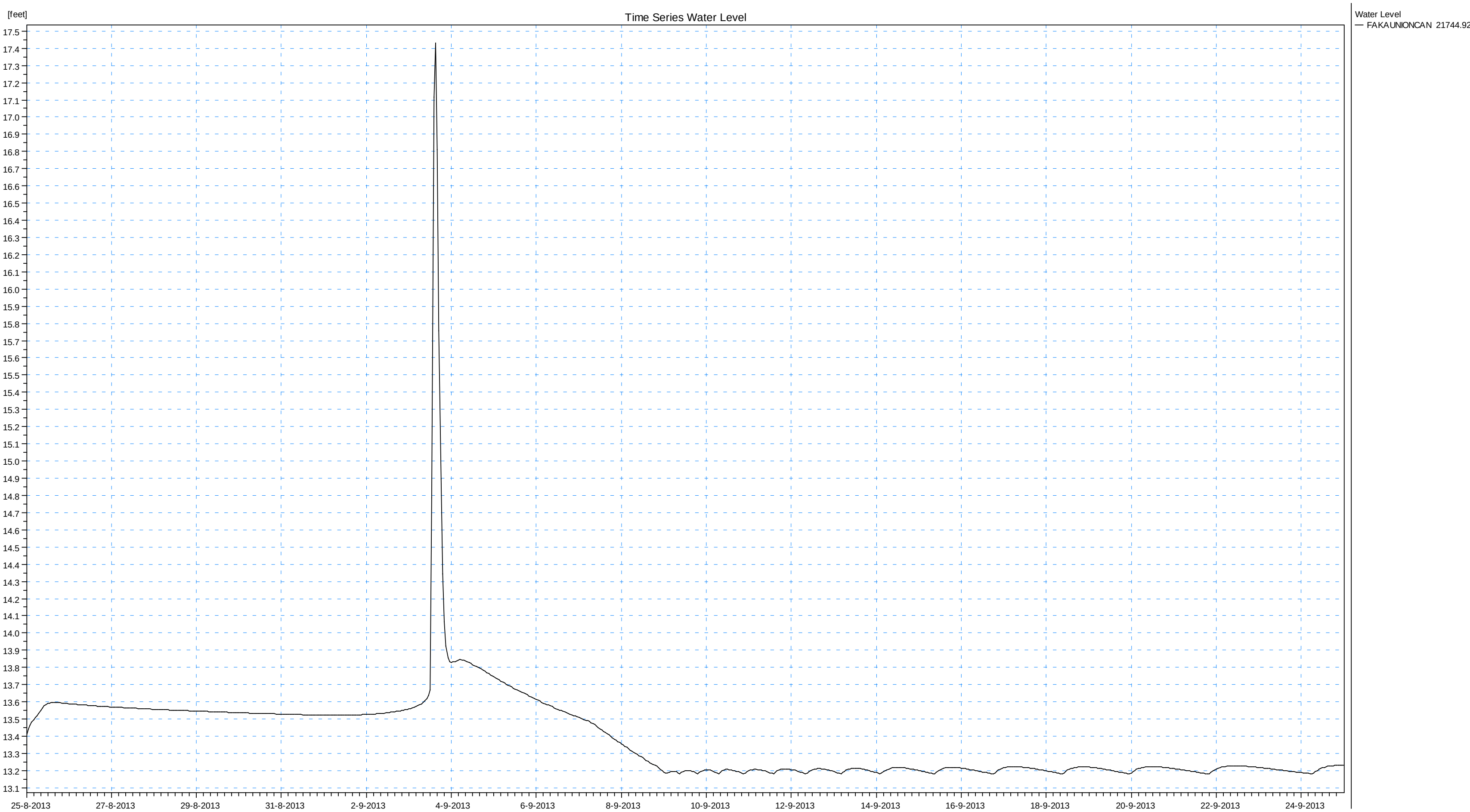

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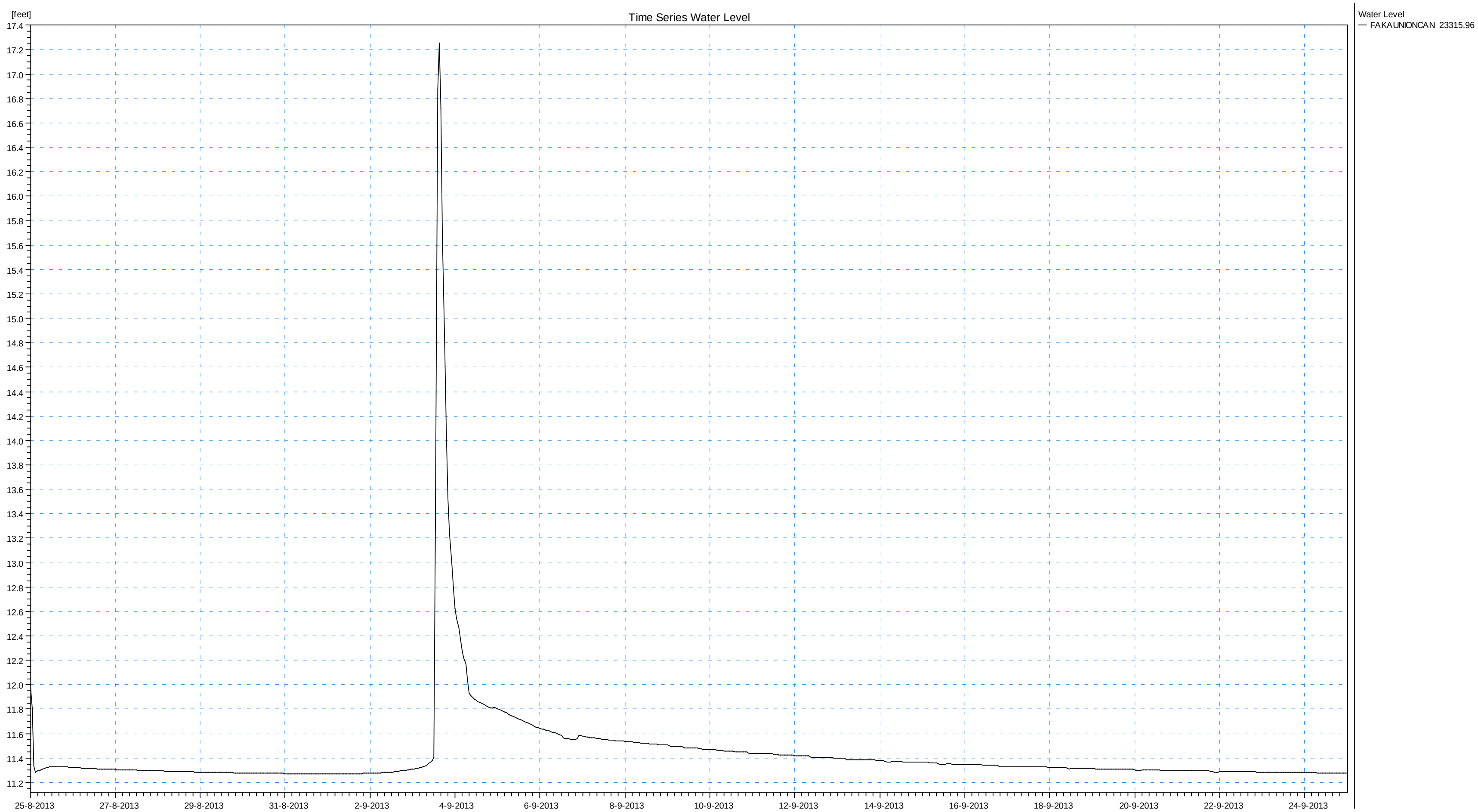

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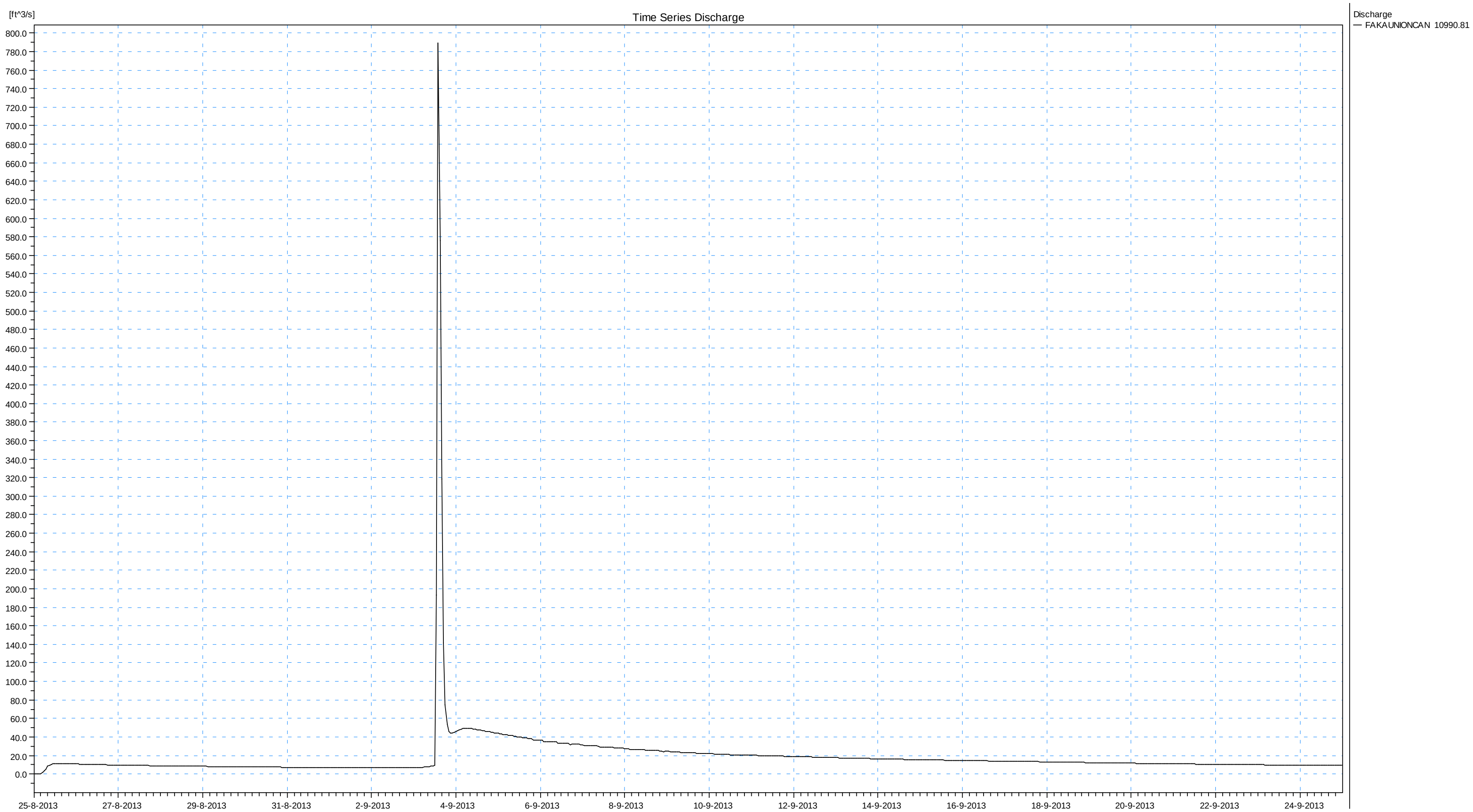

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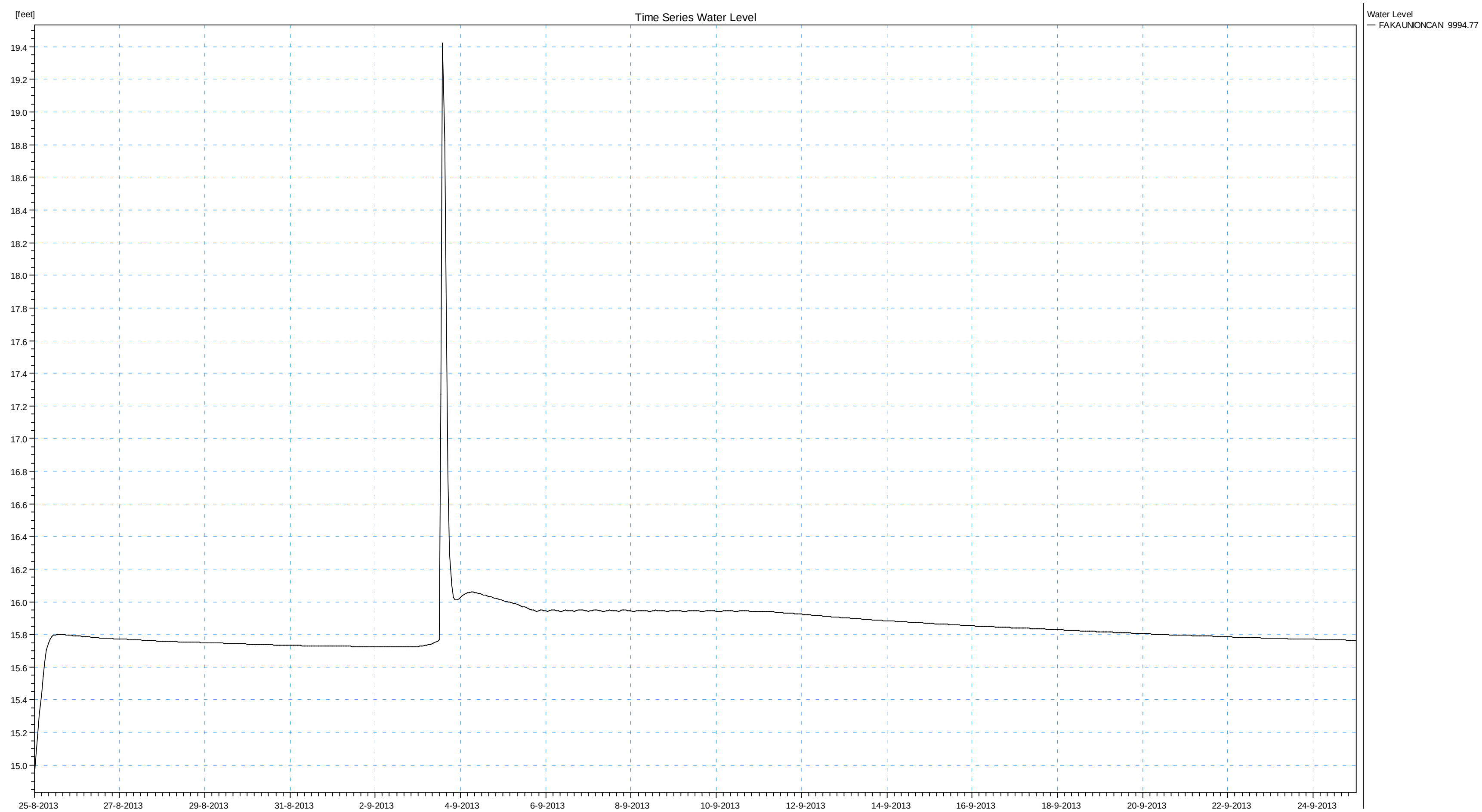

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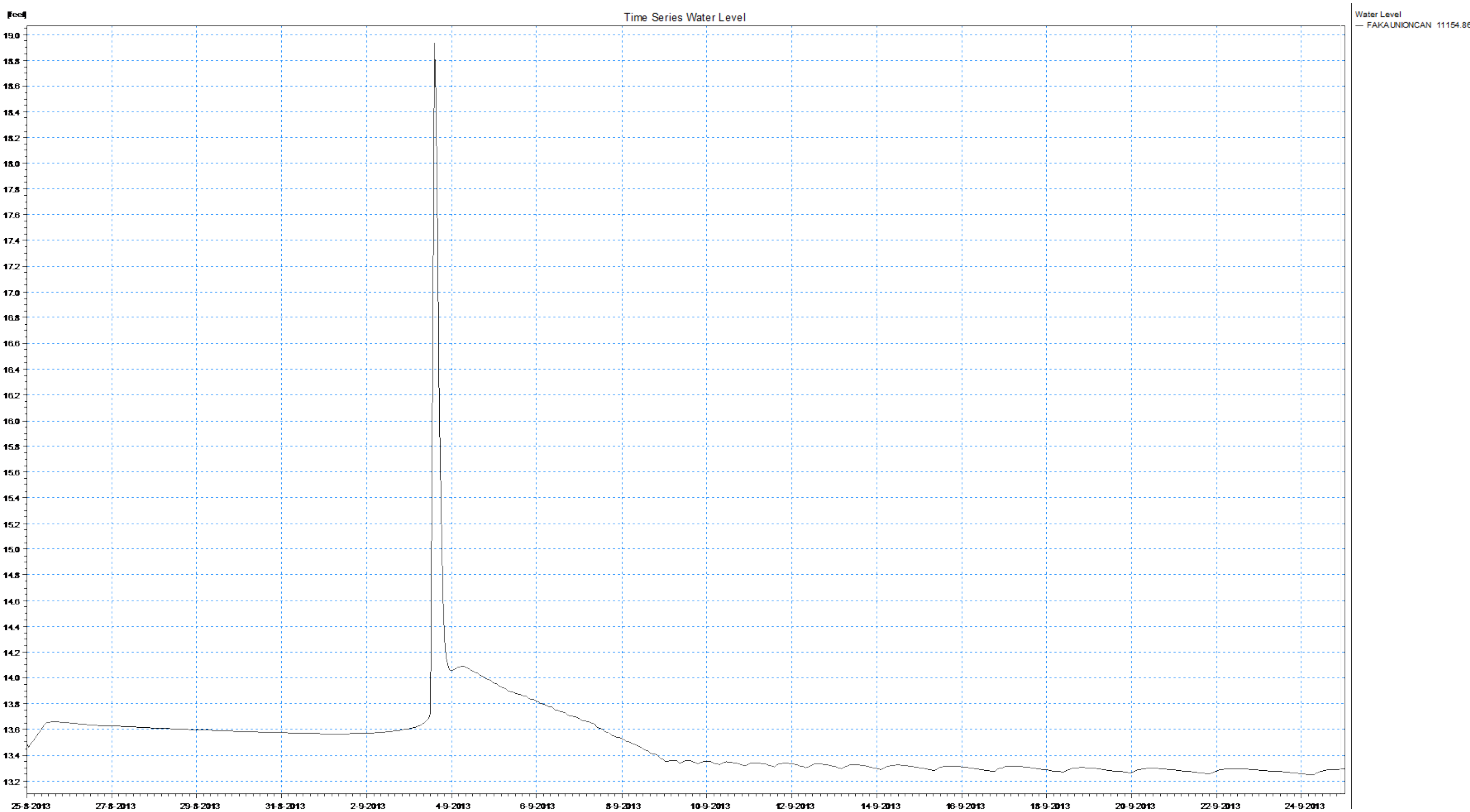

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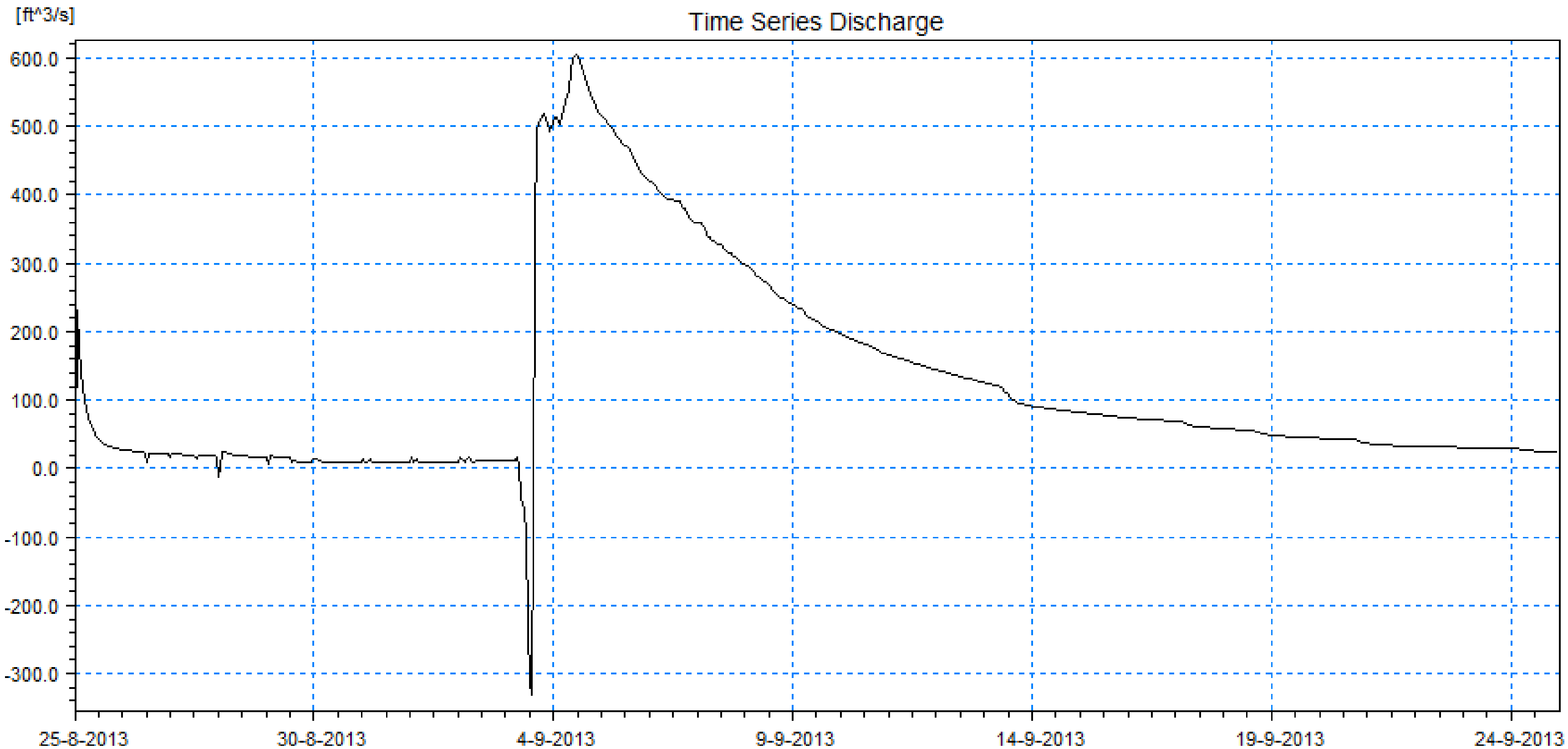

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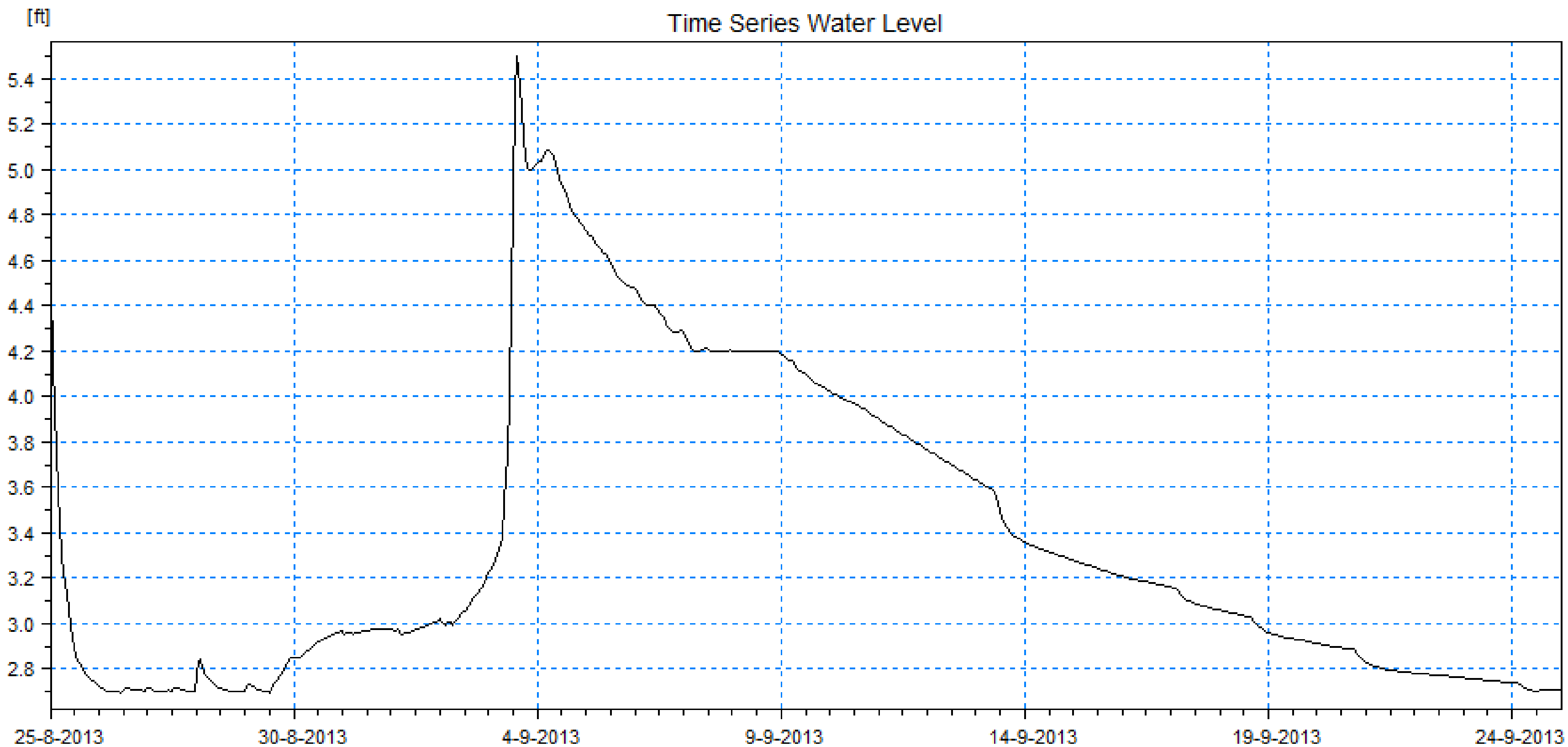

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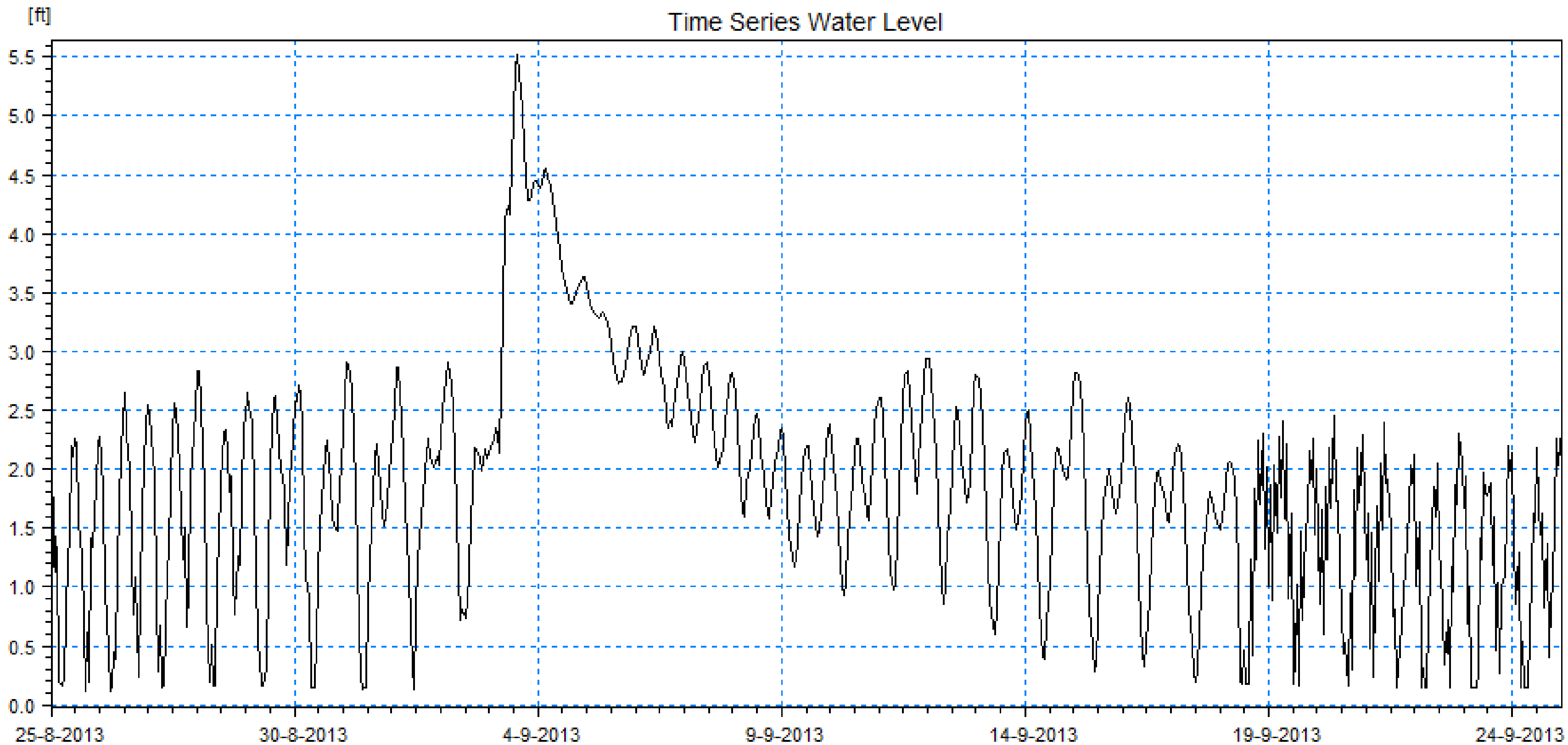

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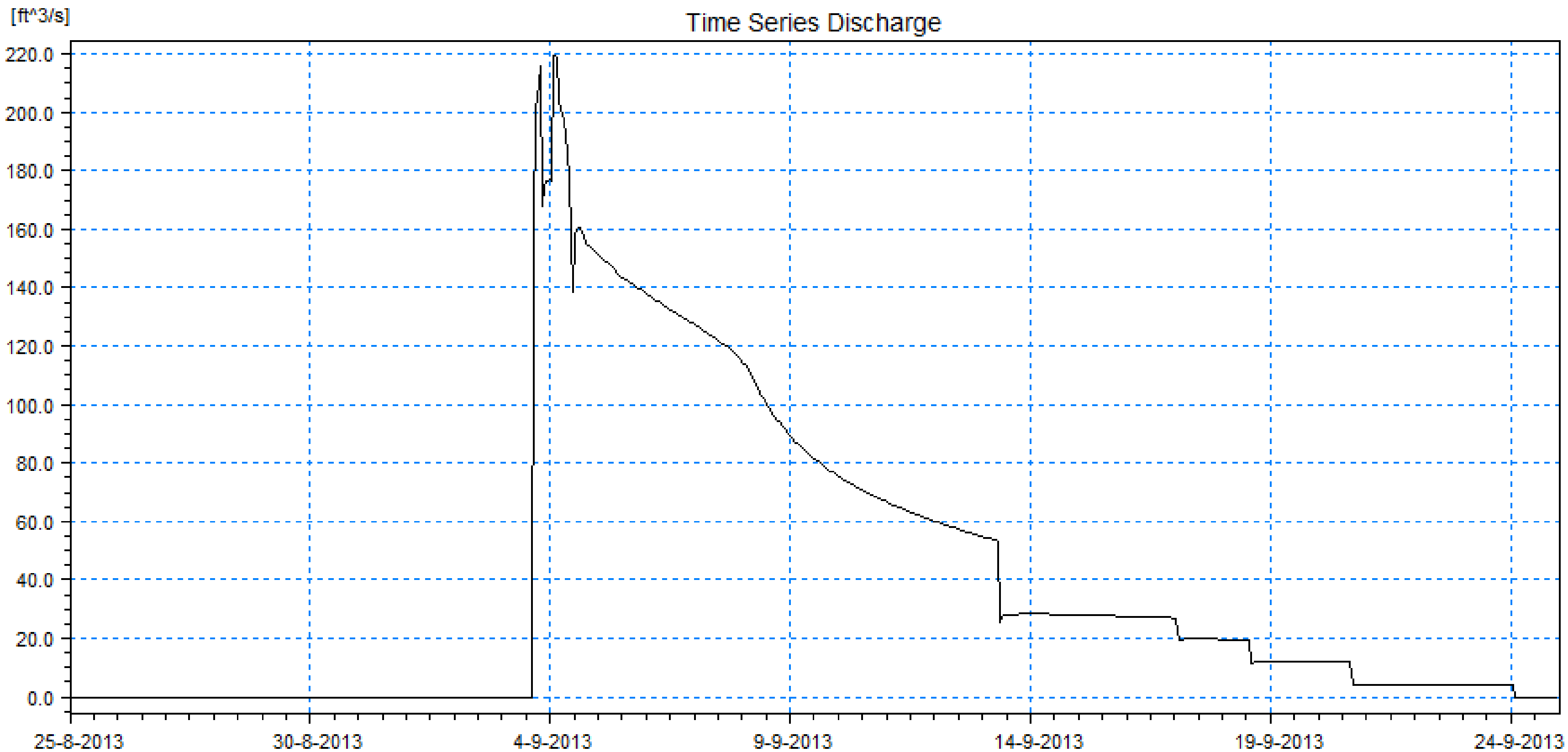

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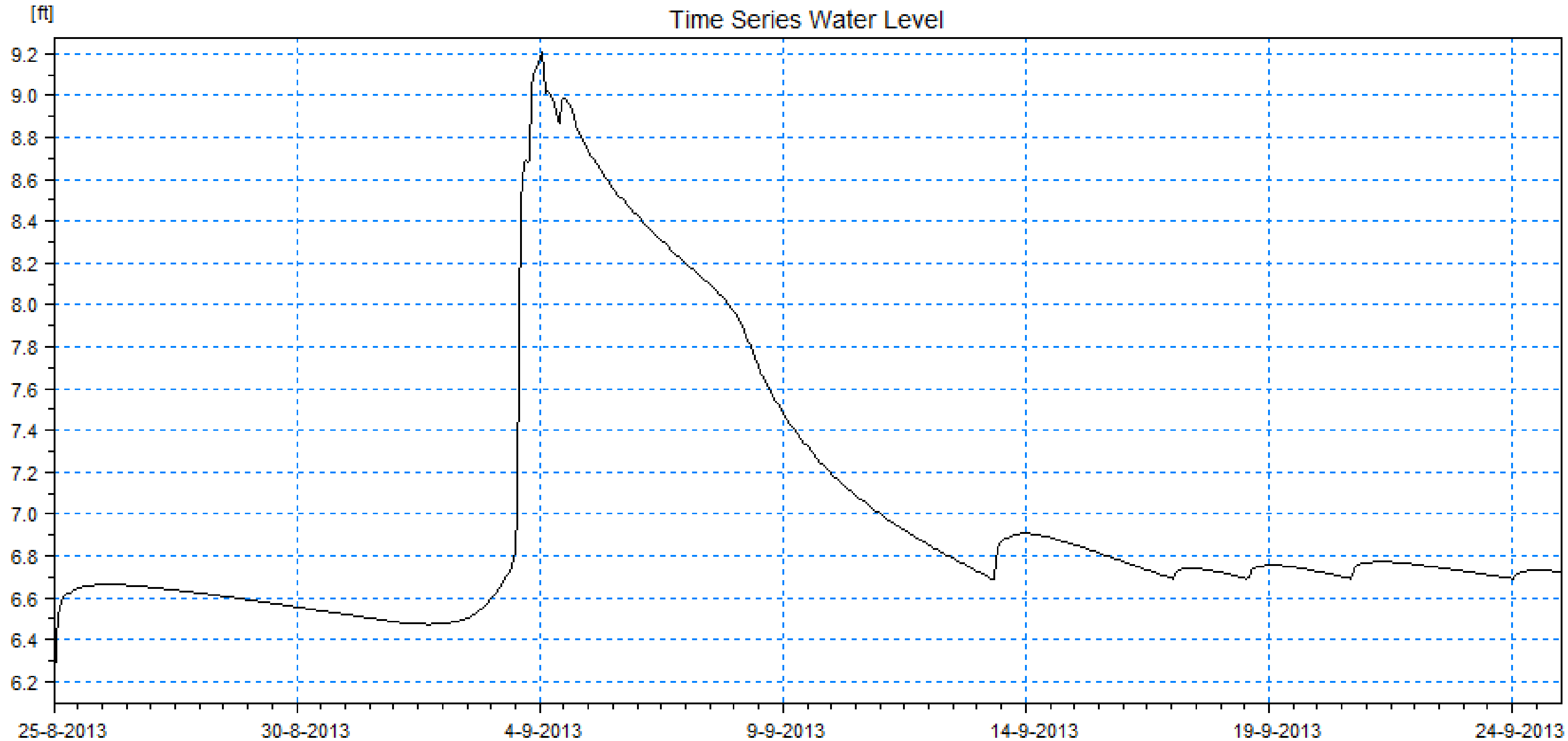

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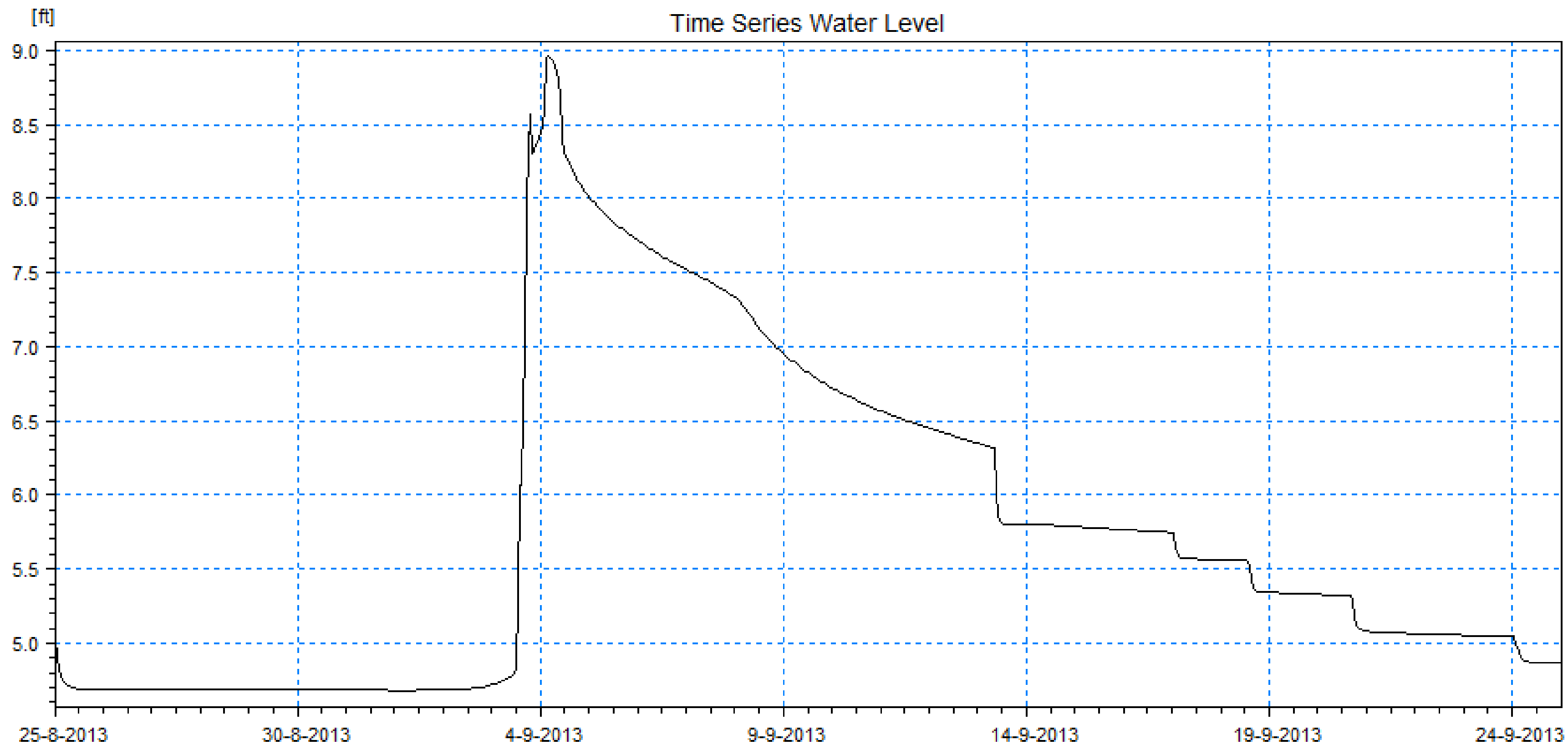

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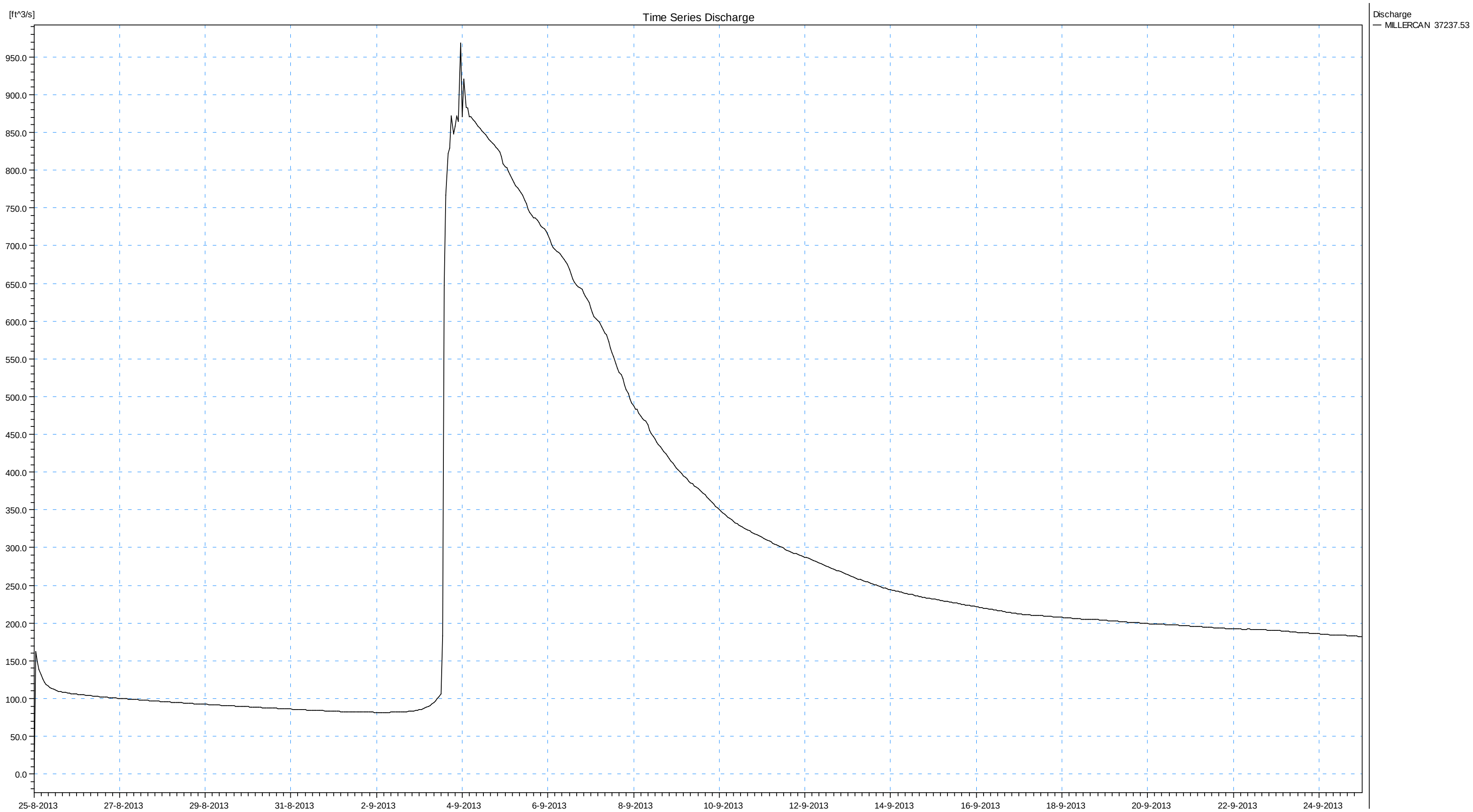

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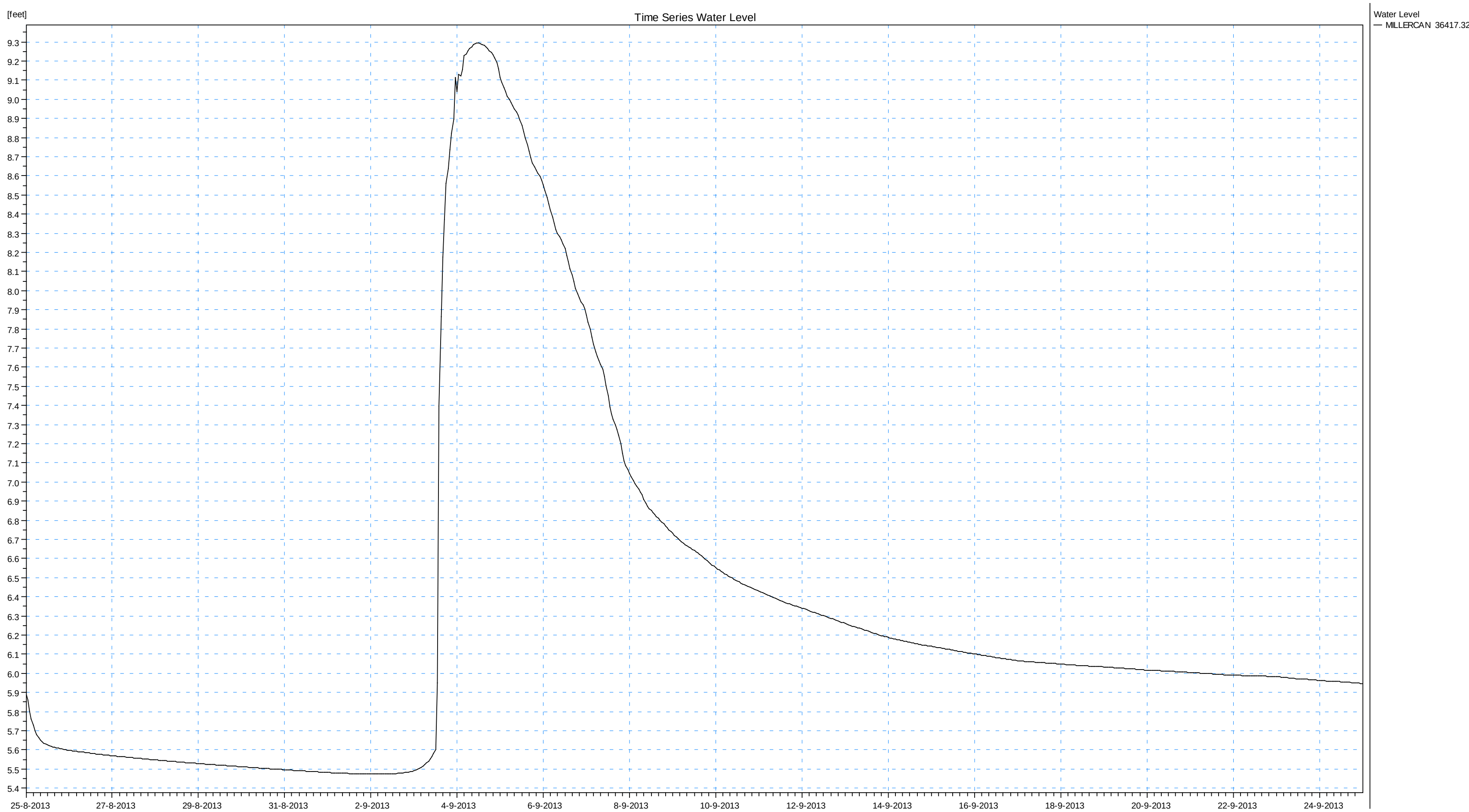

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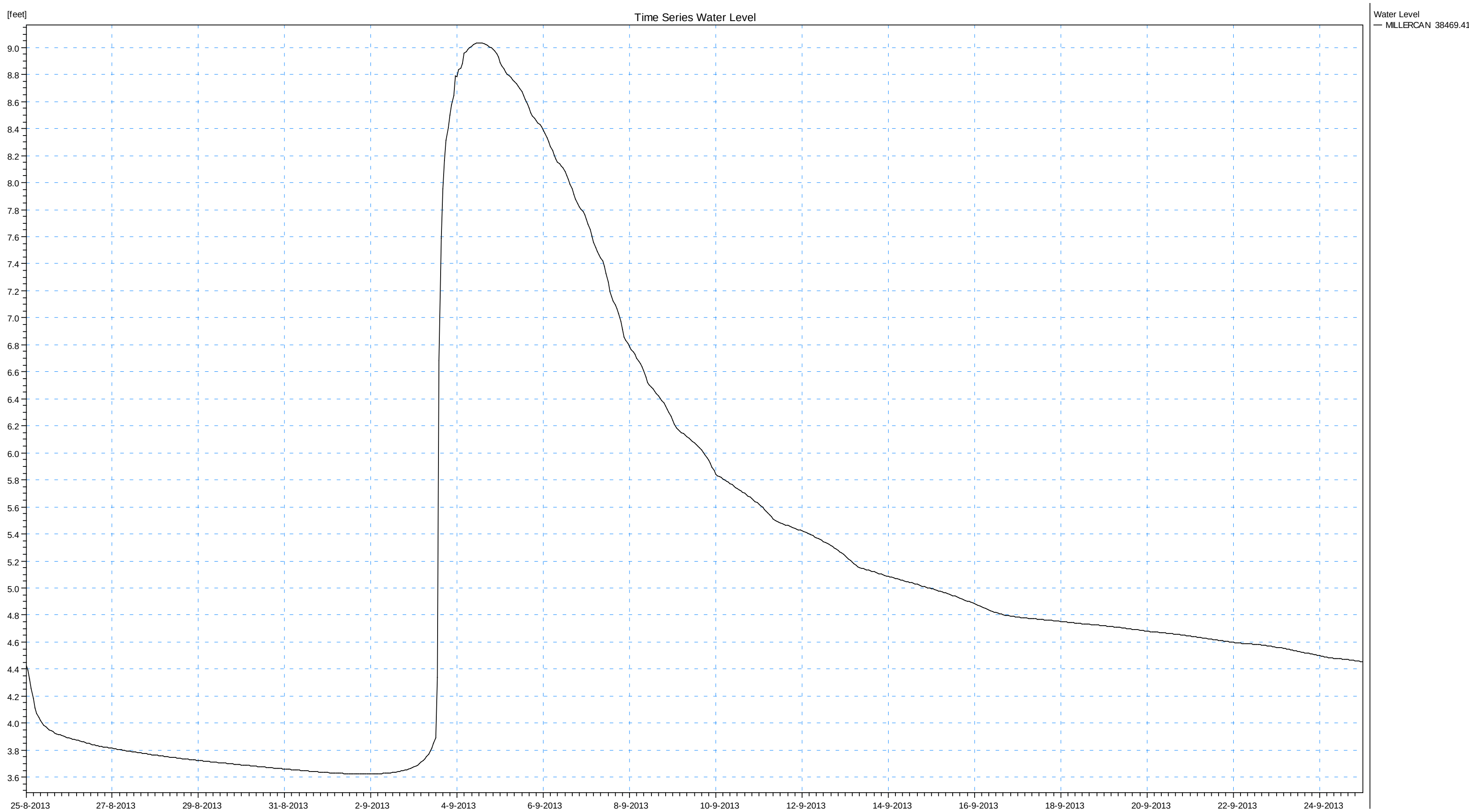

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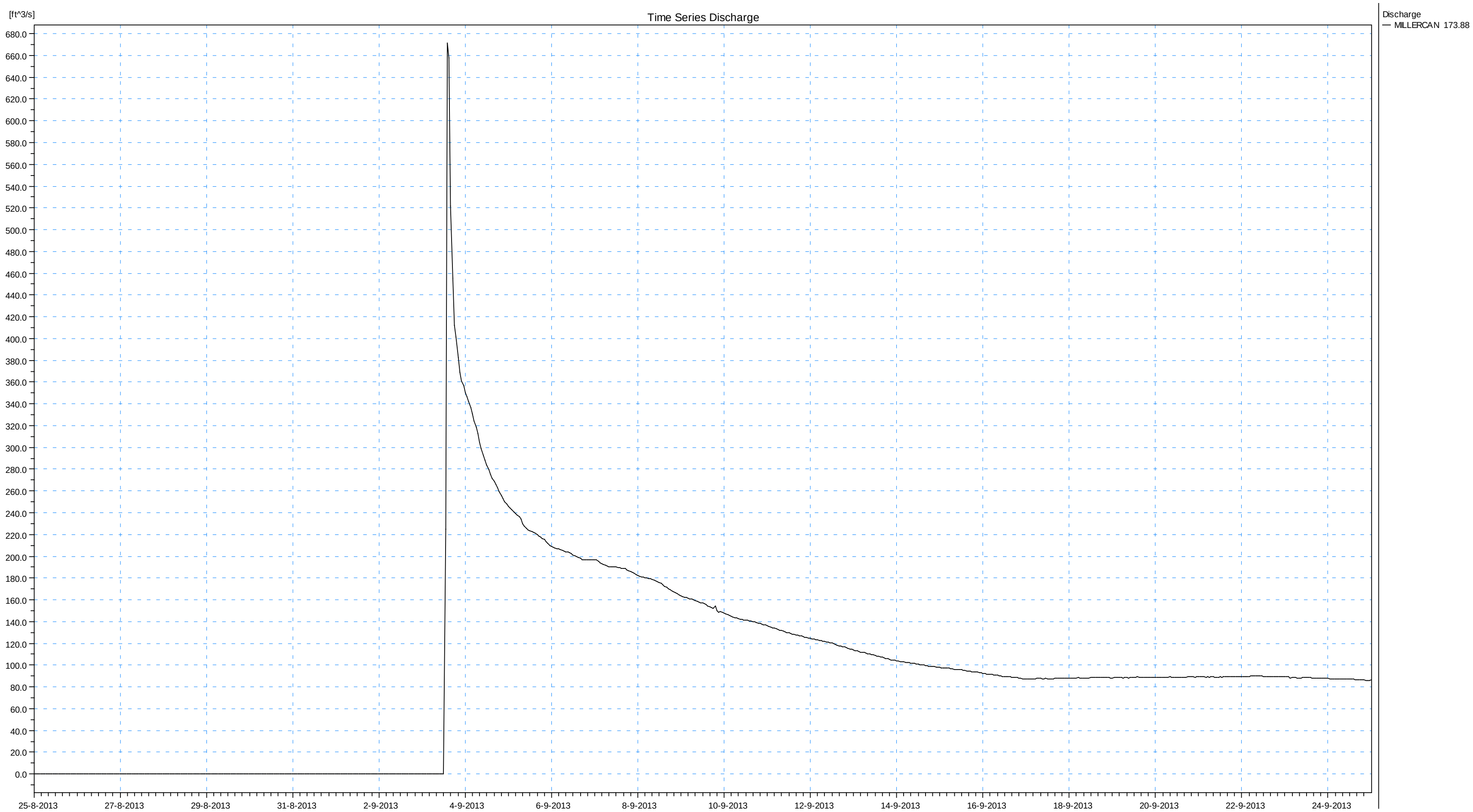

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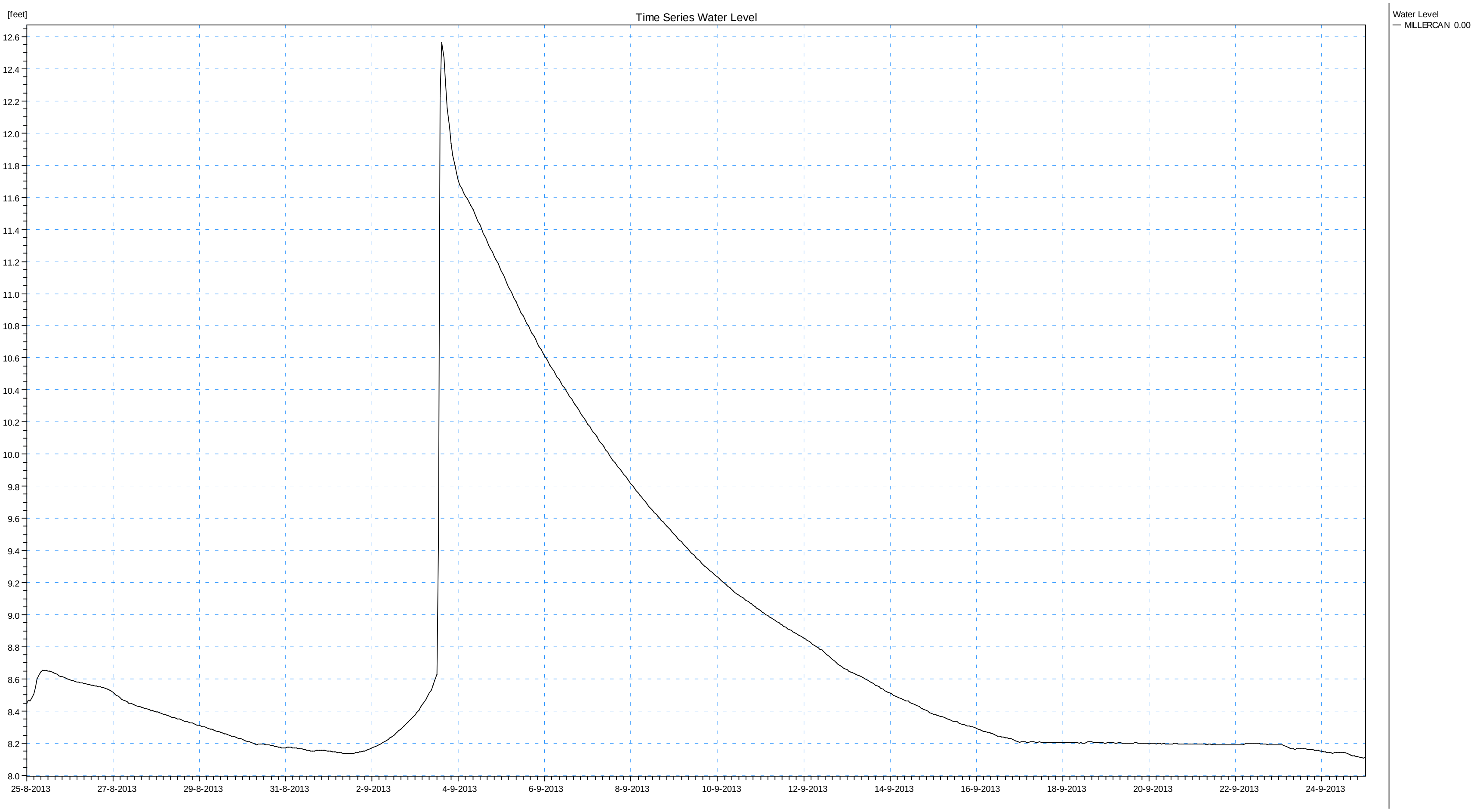

Figure 36. Miller3 Headwater Stage (ft)

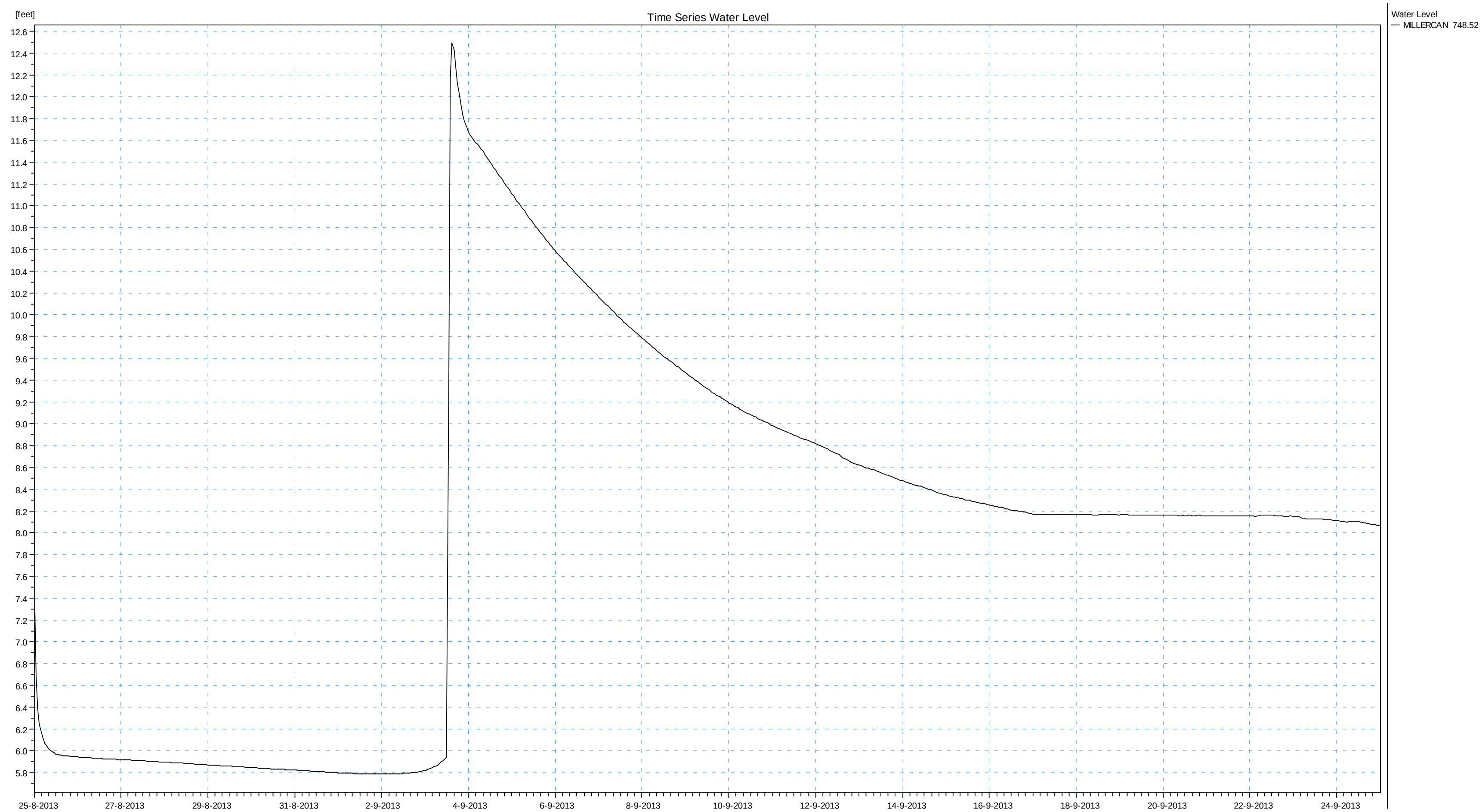

Figure 37. Miller3 Tailwater Stage (ft)

E – MIKE 11 Detailed Time Series for 100-year Storm, Current Conditions

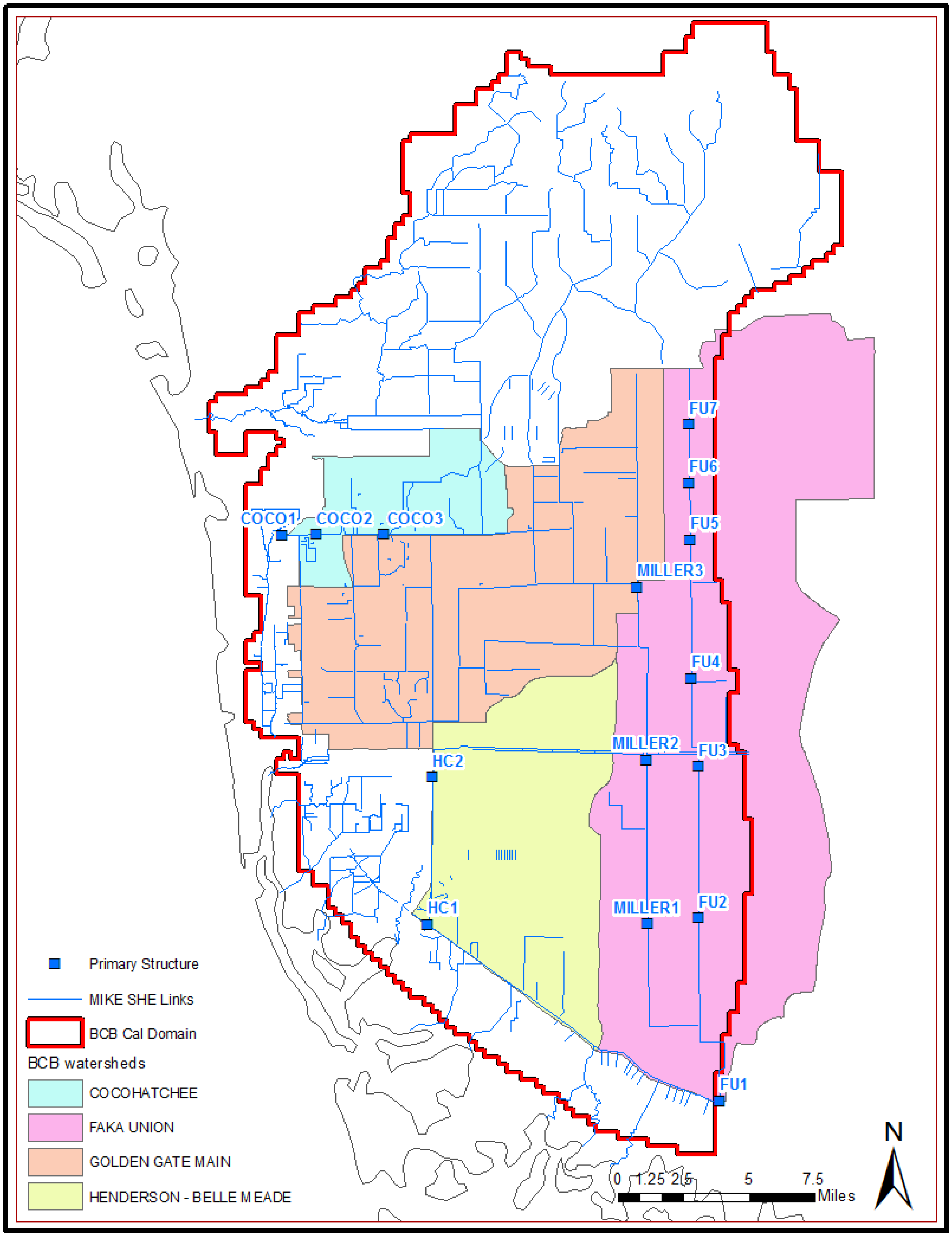

Figure 1 Mike SHE Stations within COCO, FAKA and HEND Watersheds

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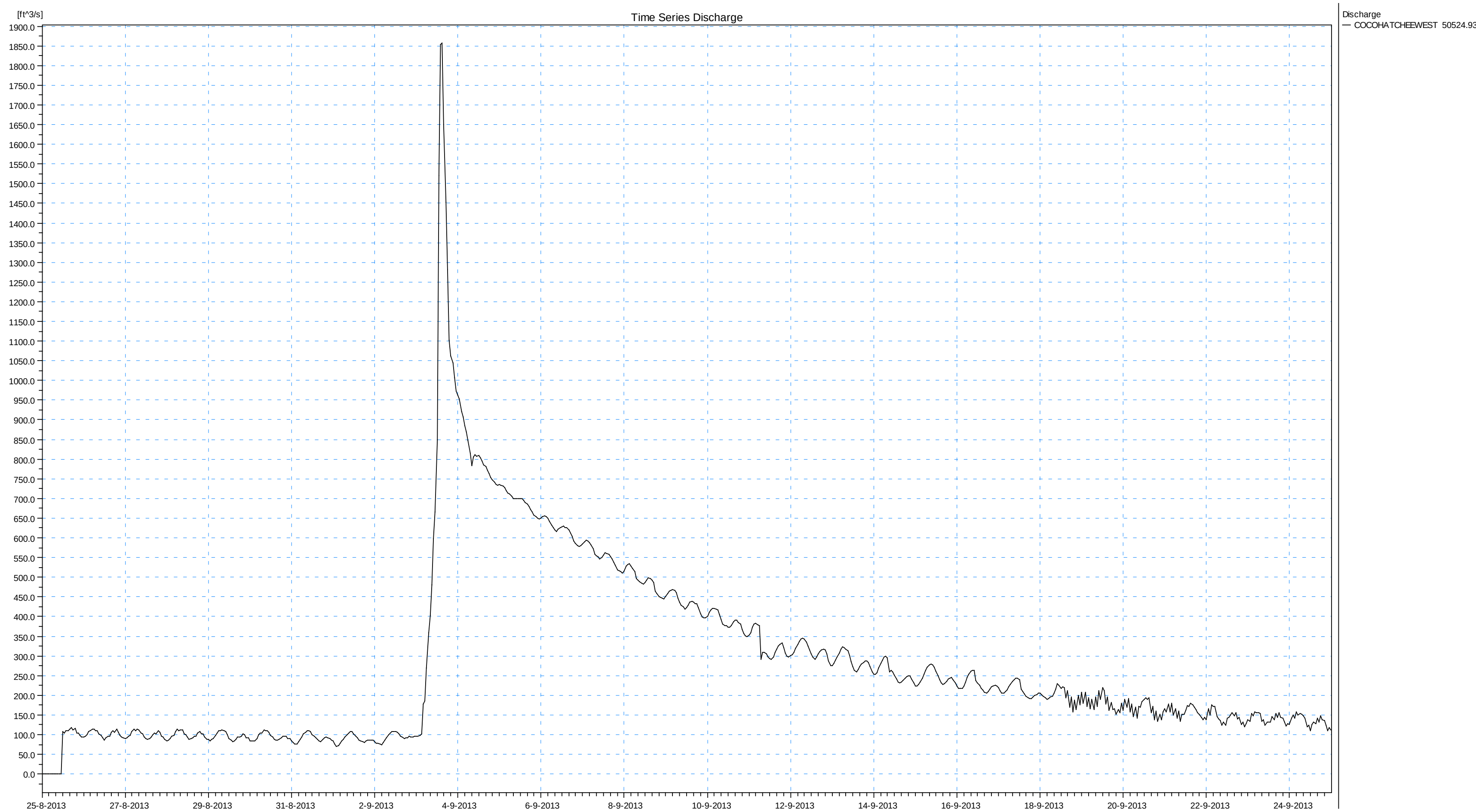


Figure 2. COCO1 Discharge (cfs)

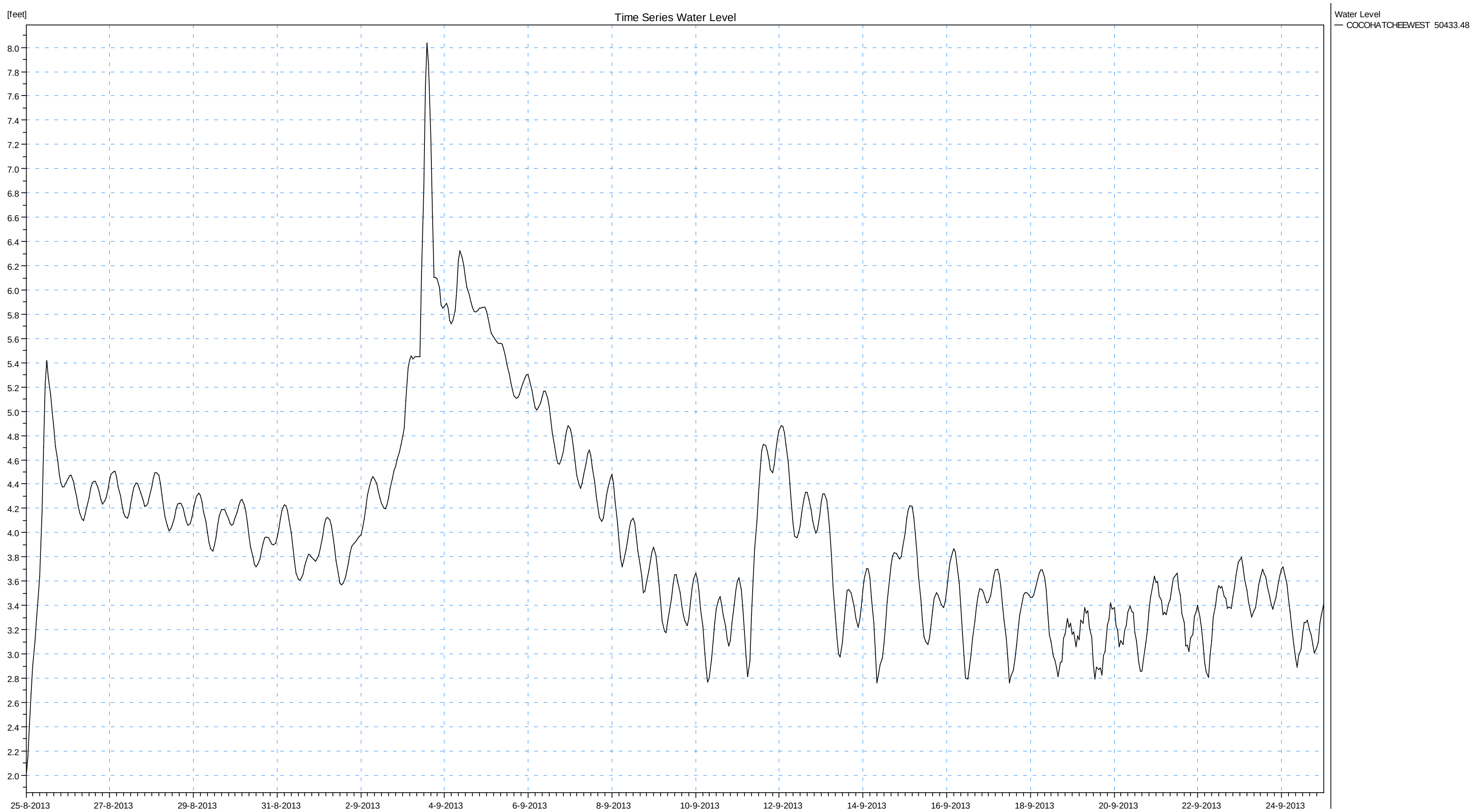


Figure 3. COCO1 Headwater Stage (ft)

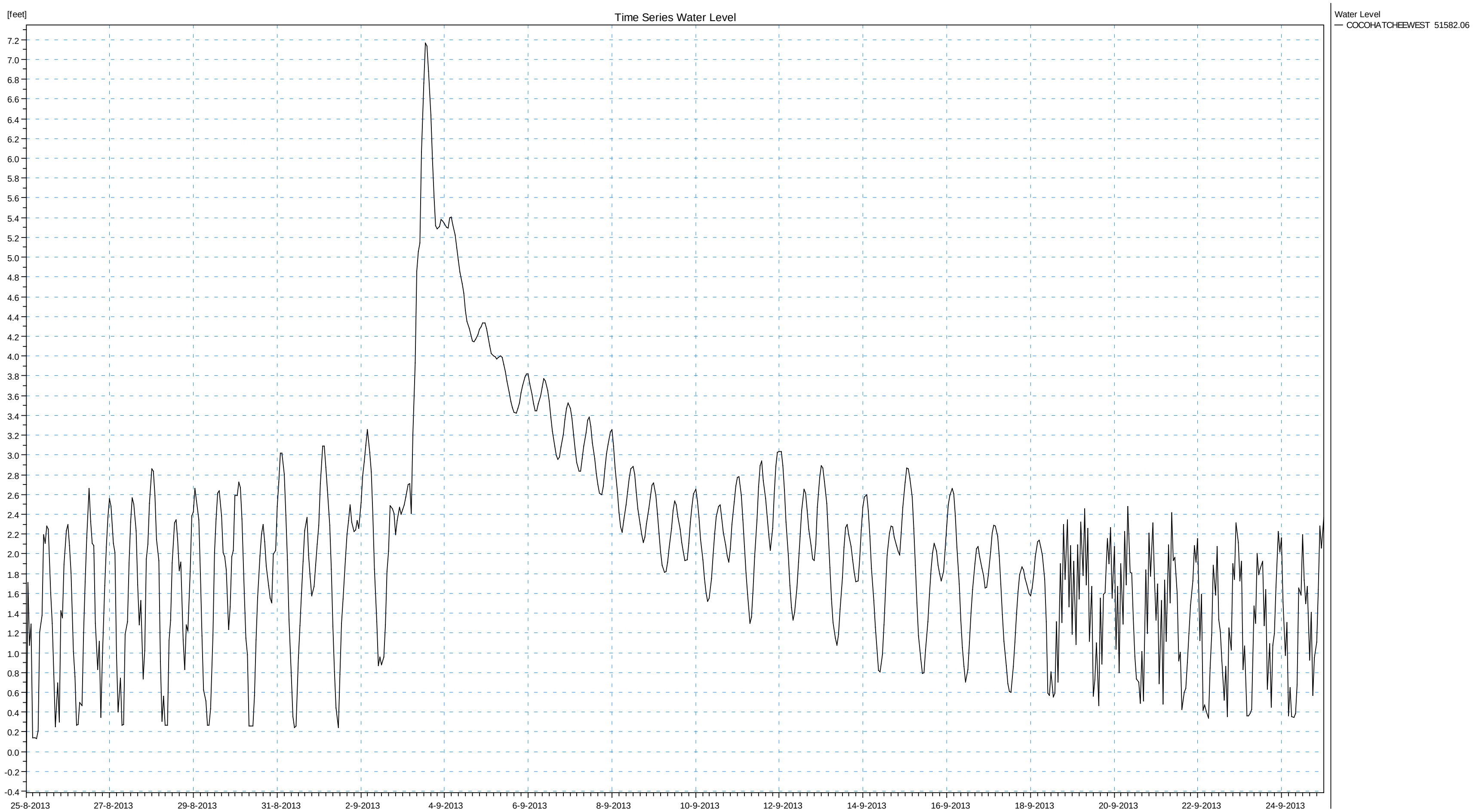


Figure 4. COCO1 Tailwater Stage (ft)

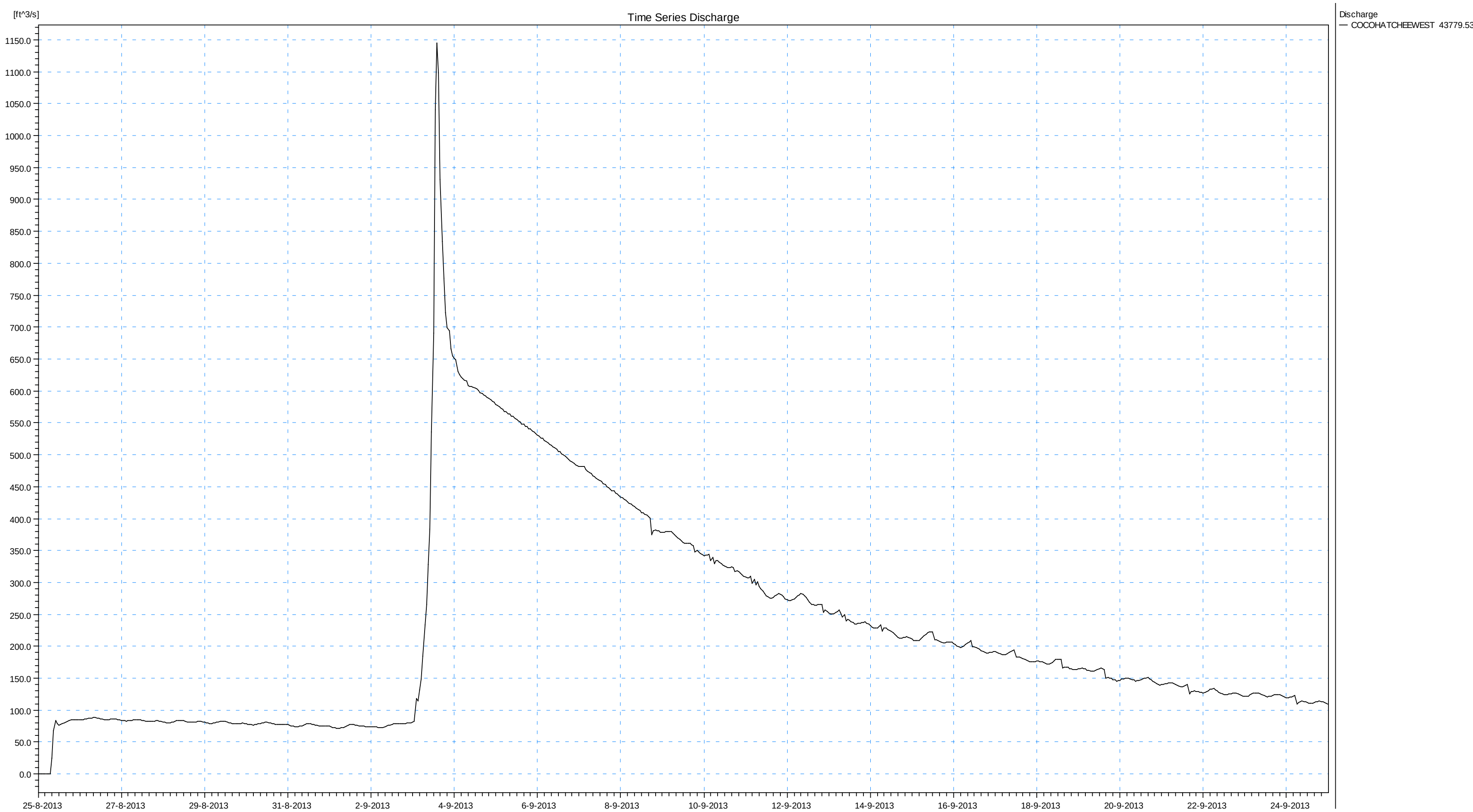


Figure 5. COCO2 Discharge (cfs)

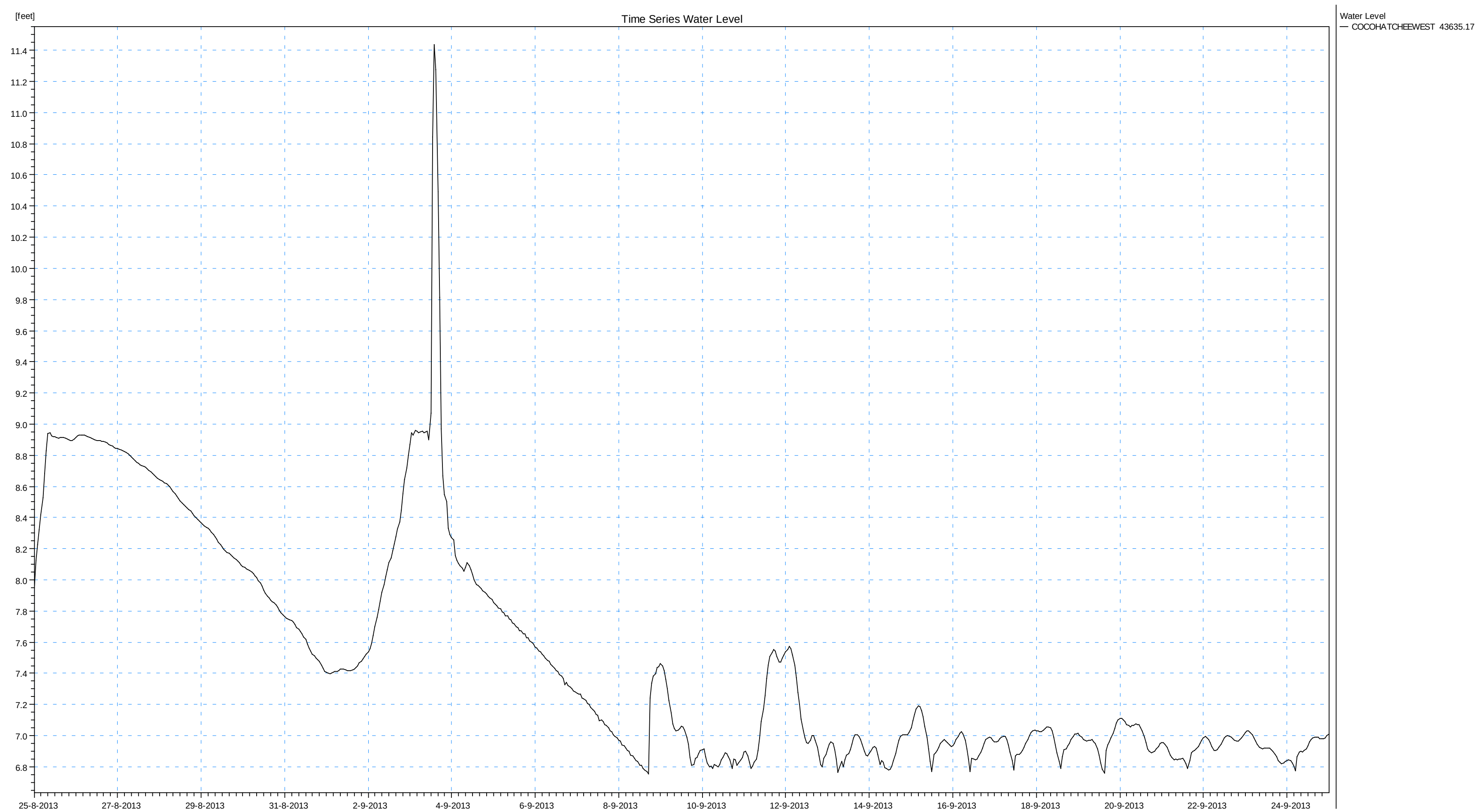


Figure 6. COCO2 Headwater Stage (ft)

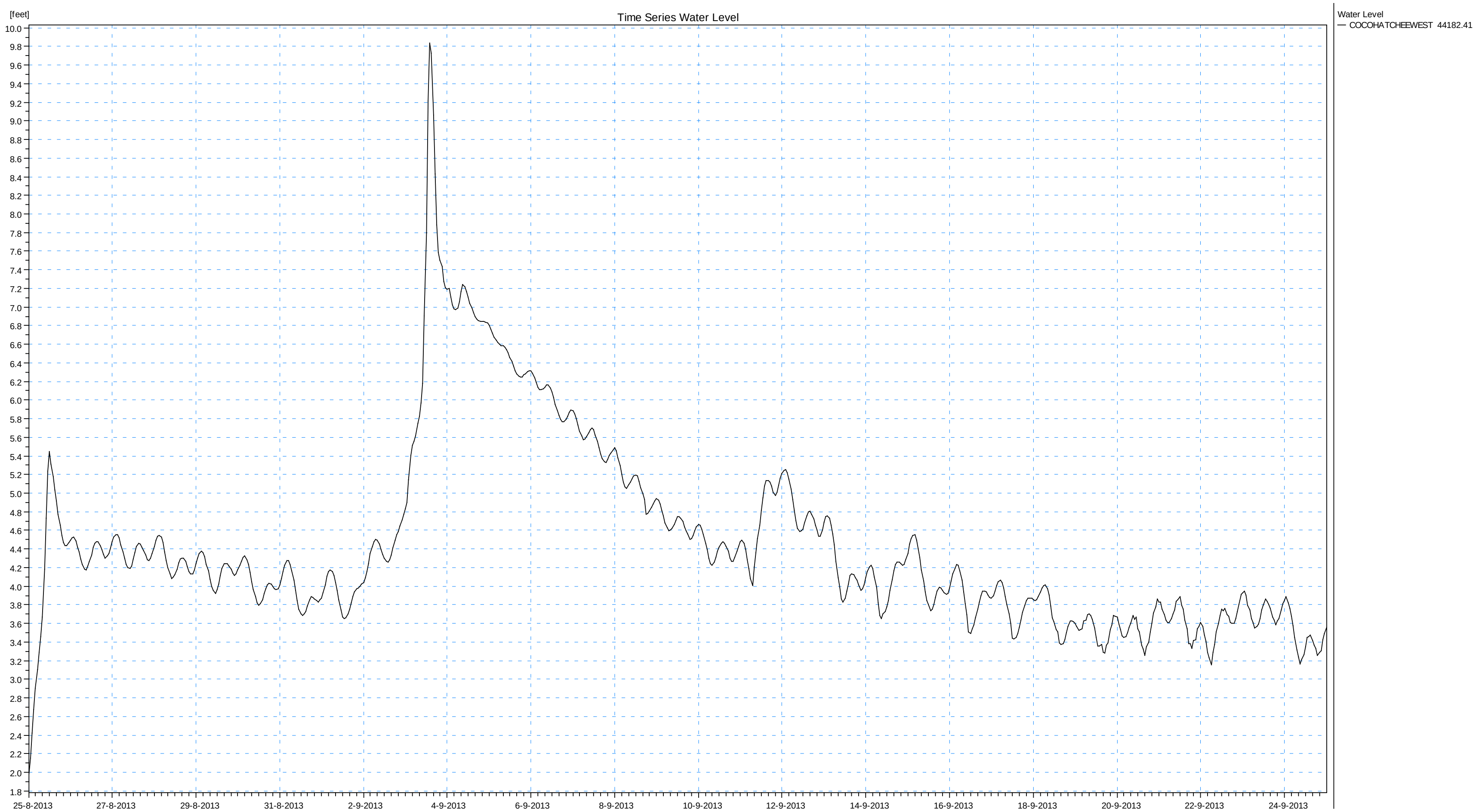


Figure 7. COCO2 Tailwater Stage (ft)

Figure 8. COCO3 Discharge (cfs)

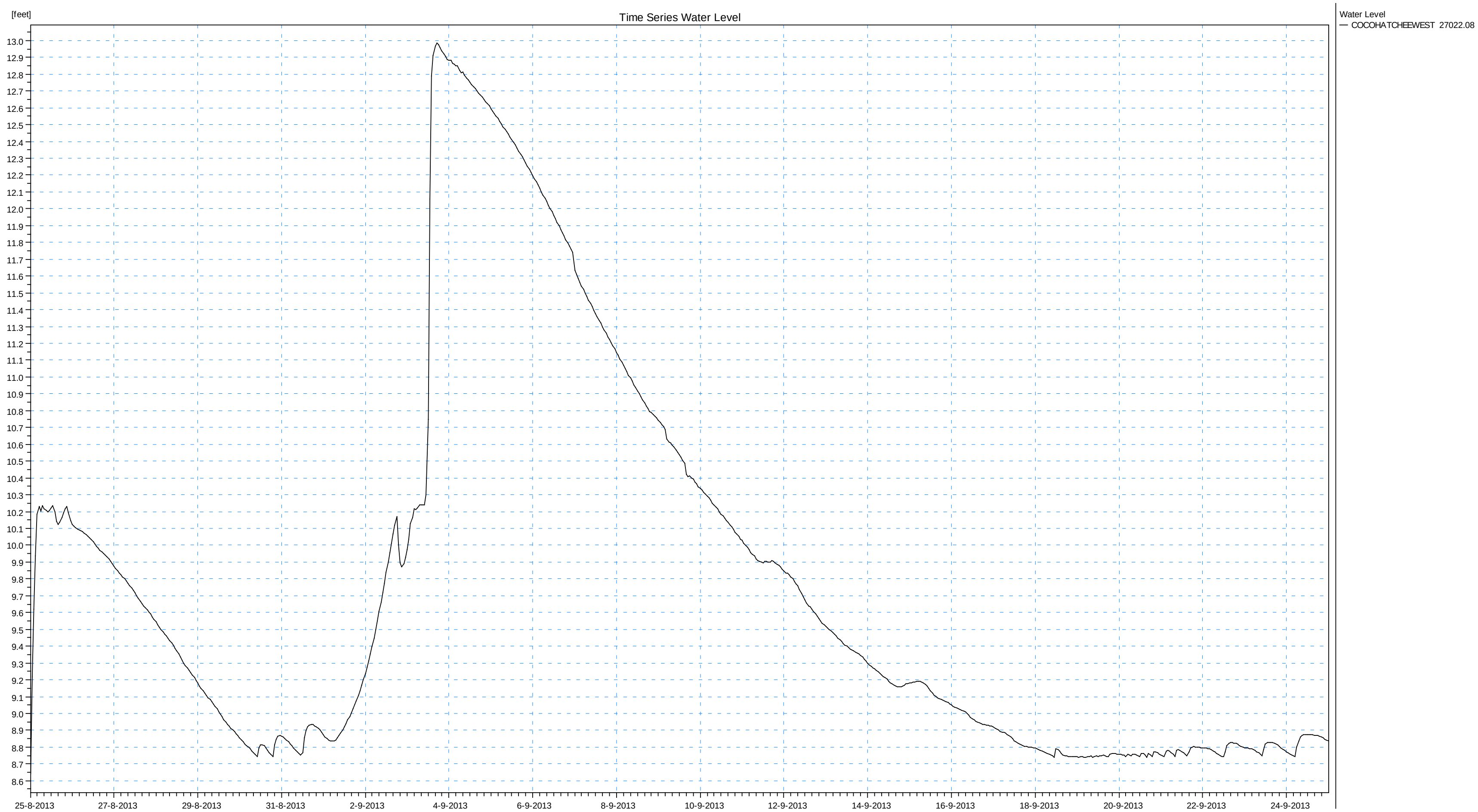


Figure 9. COCO3 Headwater Stage (ft)

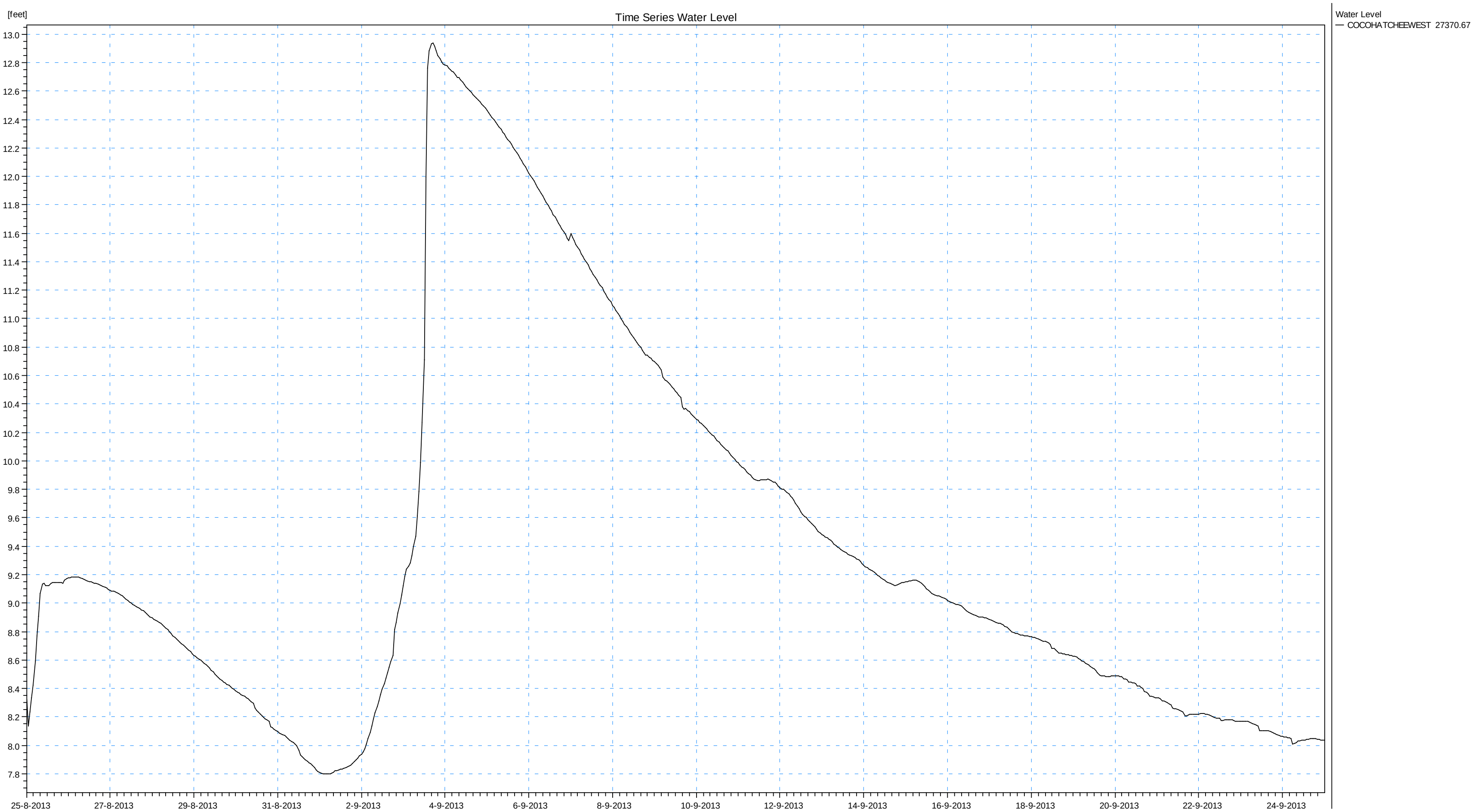


Figure 10. COCO3 Tailwater Stage (ft)

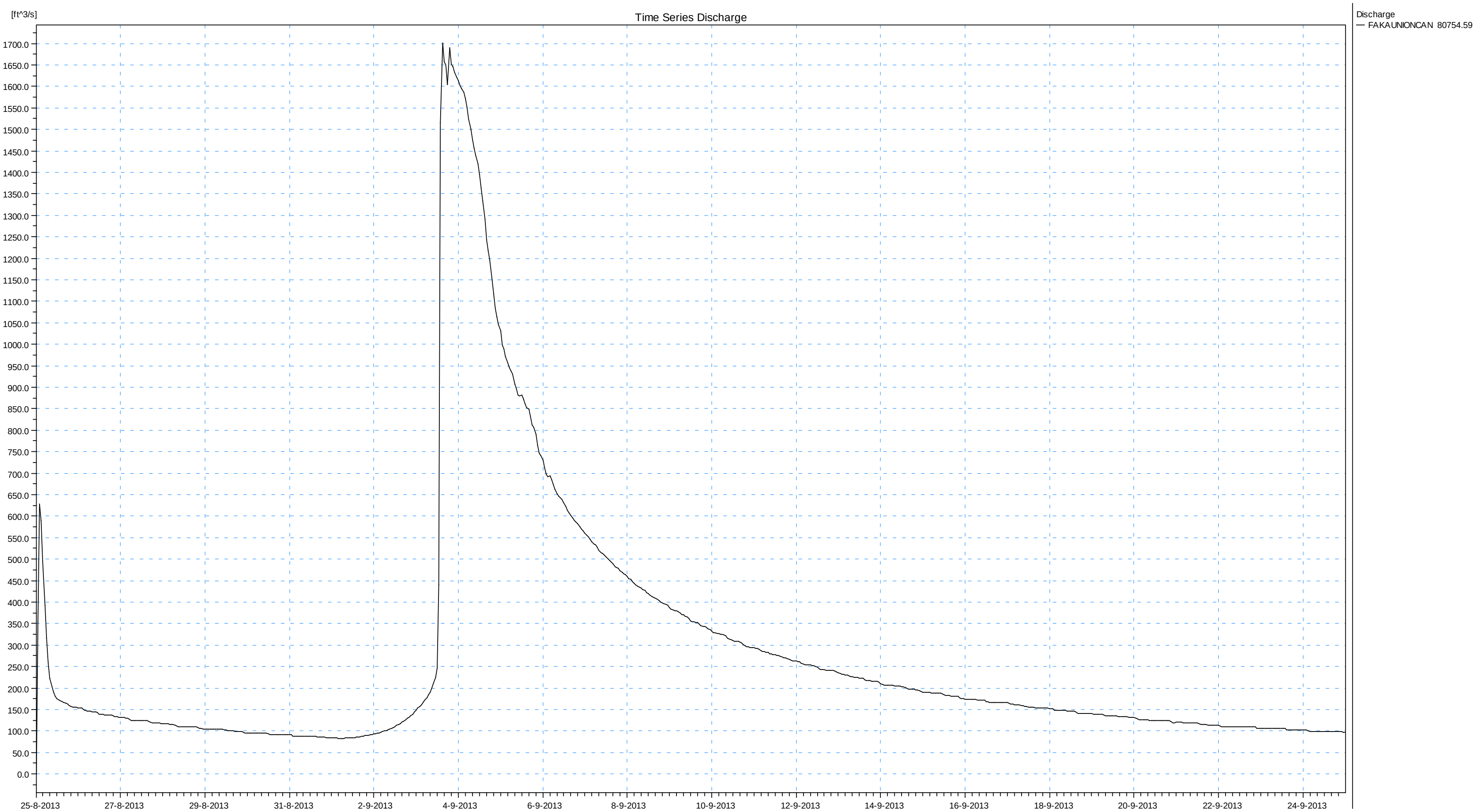
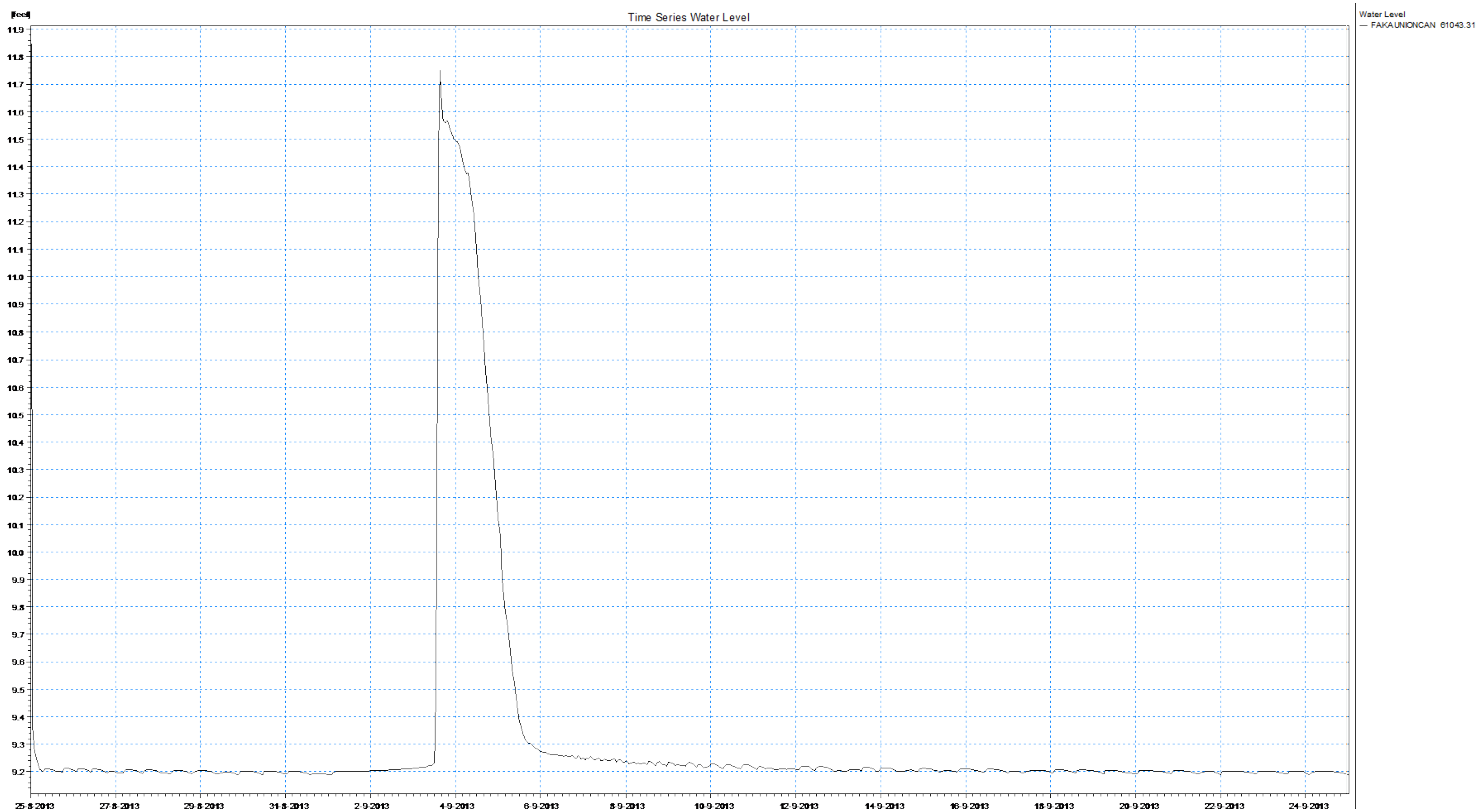


Figure 11. FU3 Discharge (cfs)



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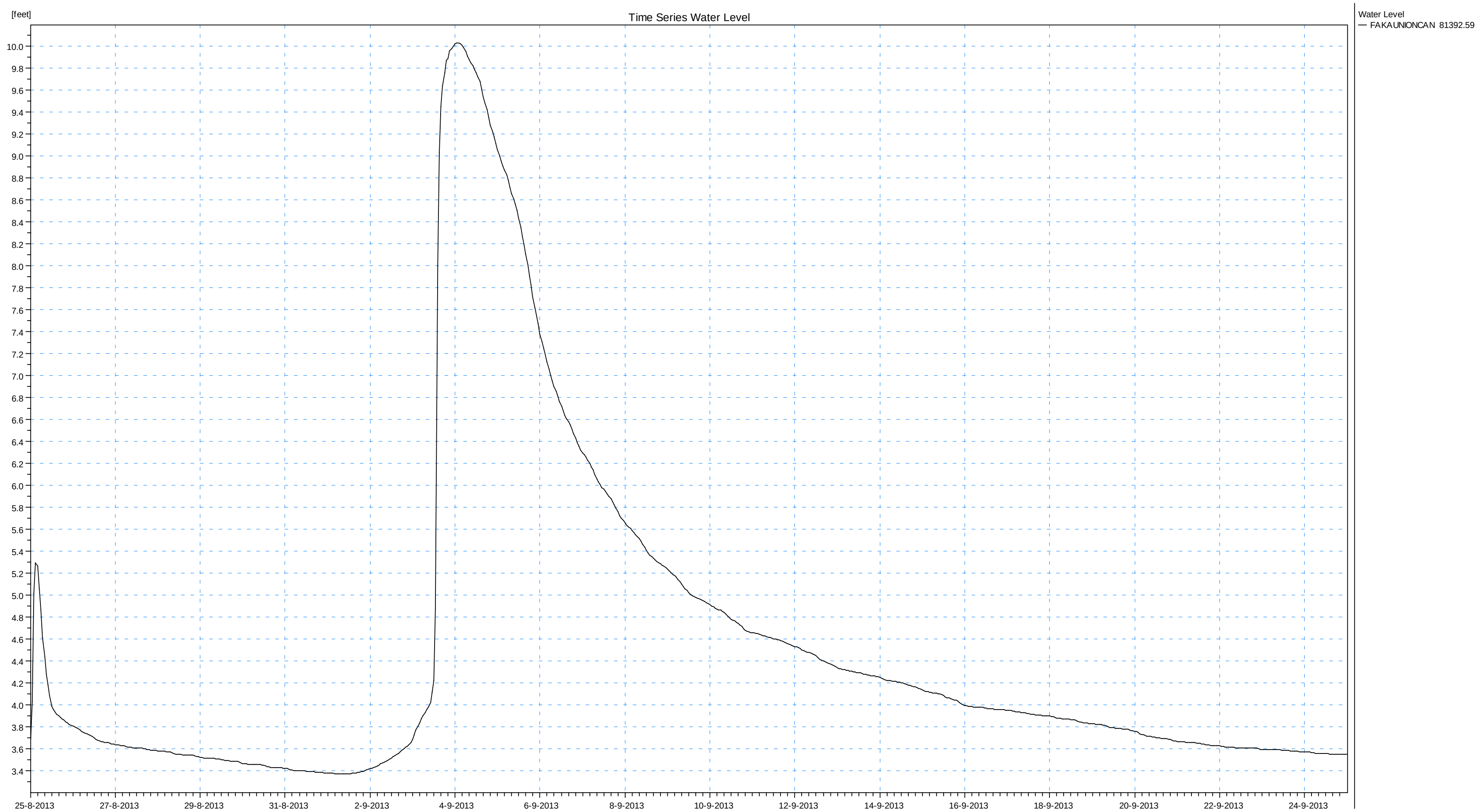
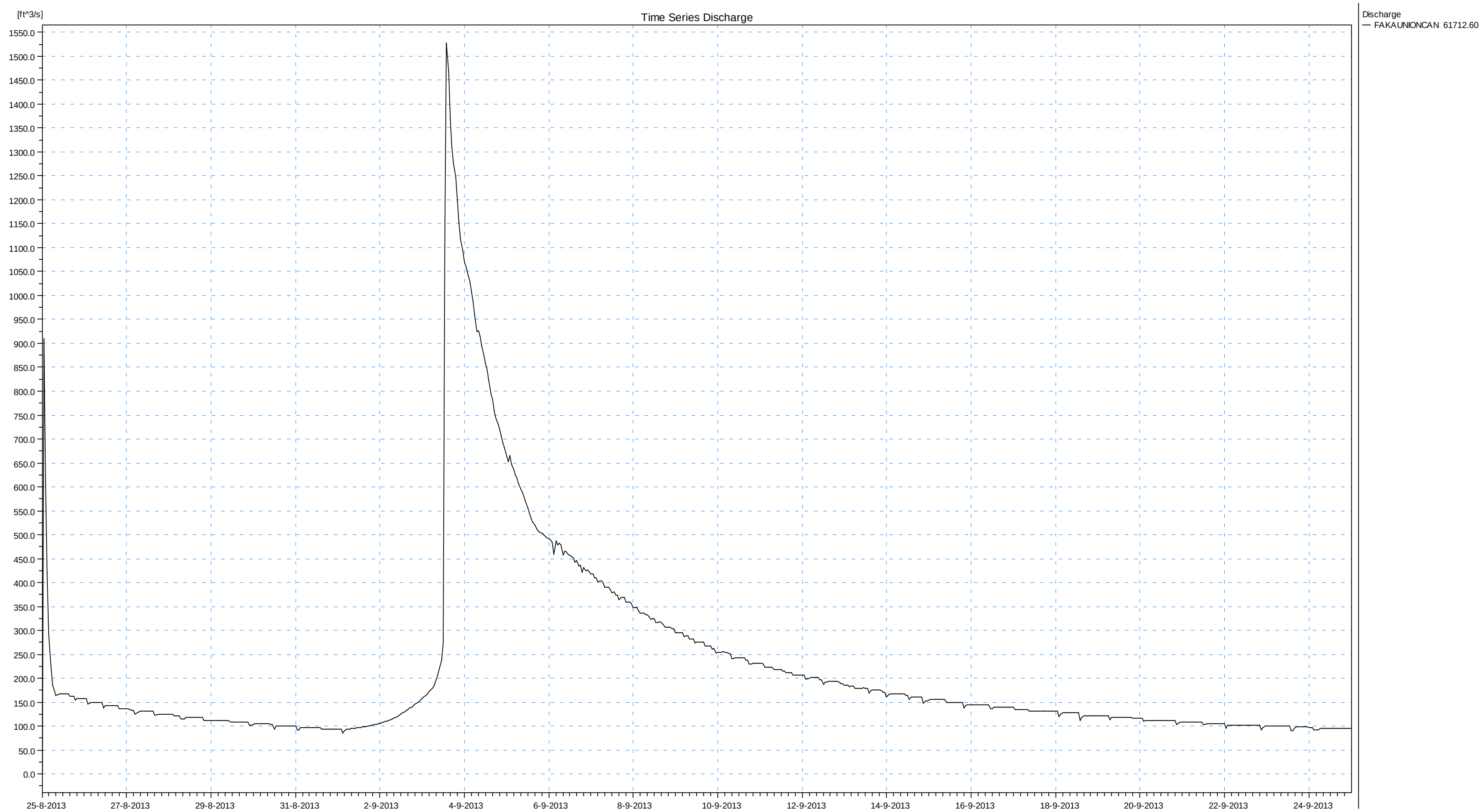


Figure 13. FU3 Tailwater Stage (ft)



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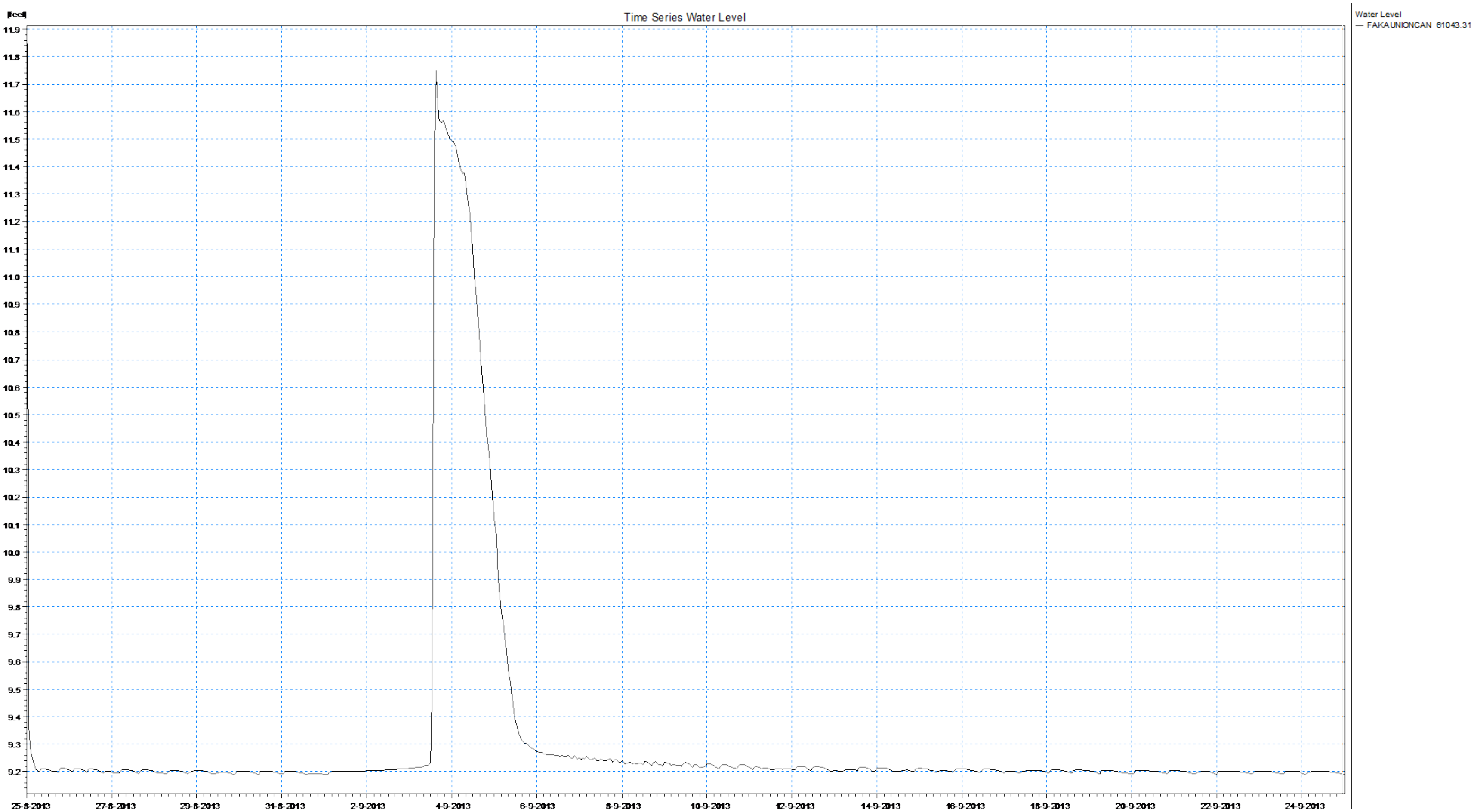


Figure 15. FU4 Headwater Stage (ft)

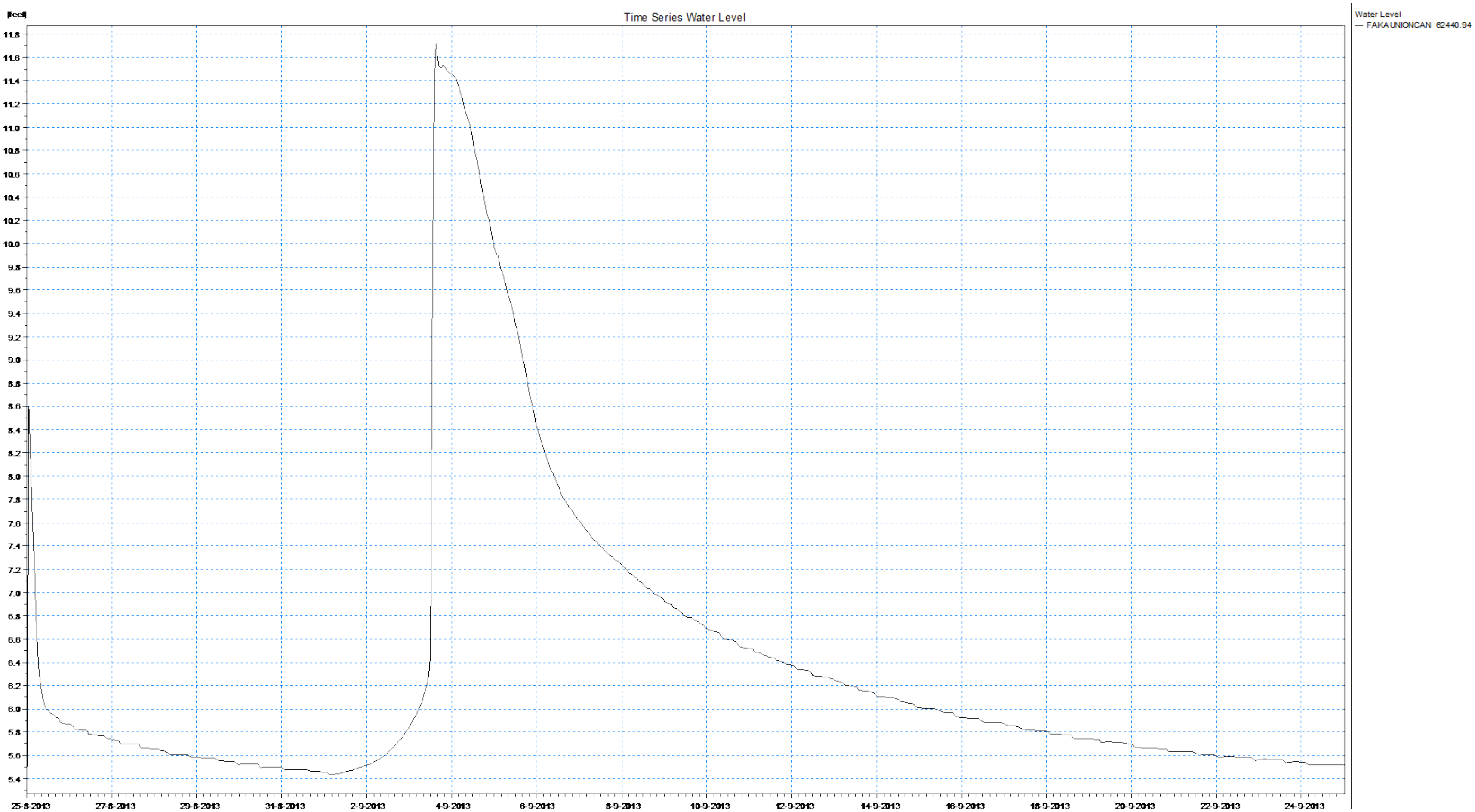


Figure 16. FU4 Tailwater Stage (ft)

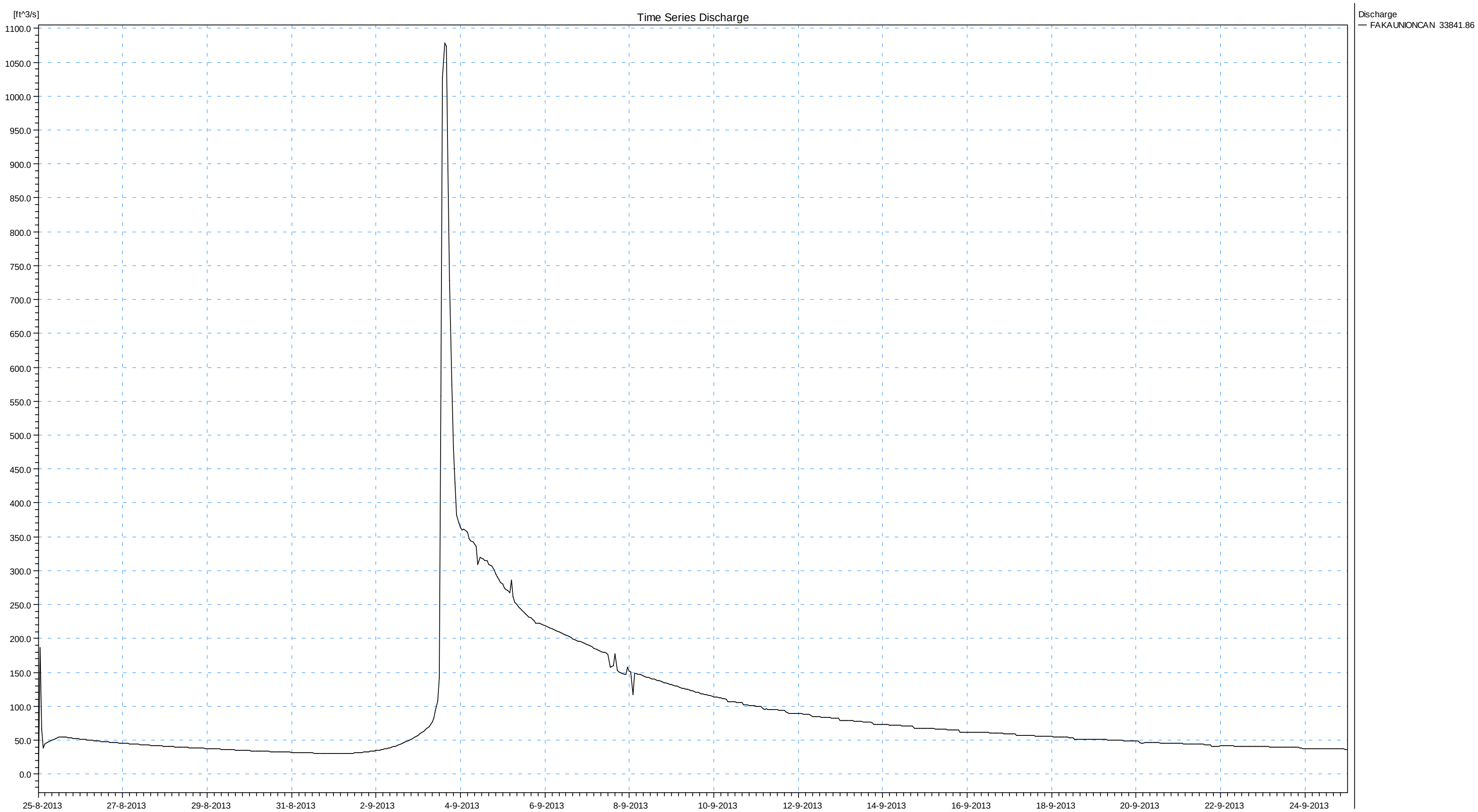
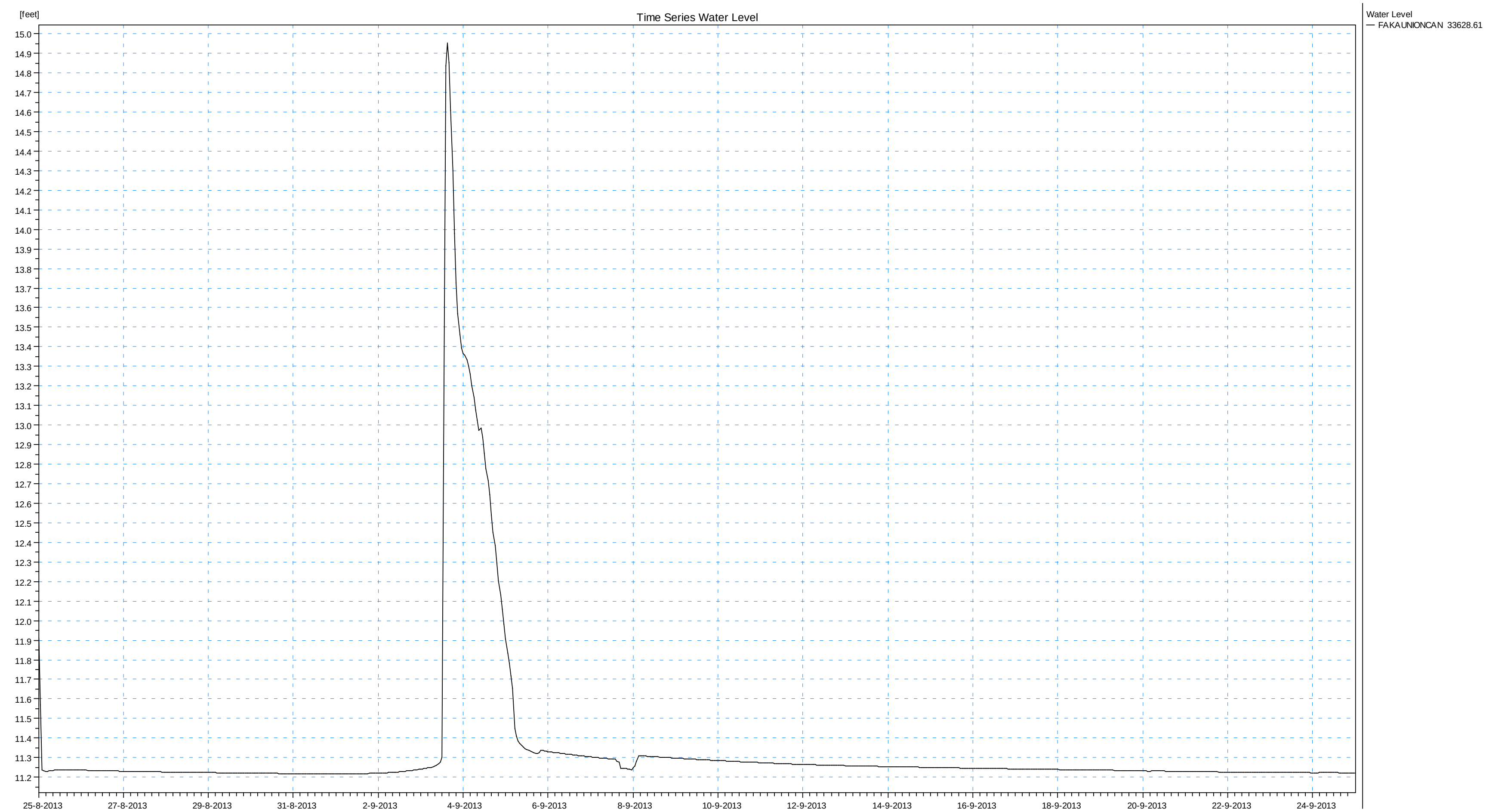


Figure 17. FU5 Discharge (cfs)



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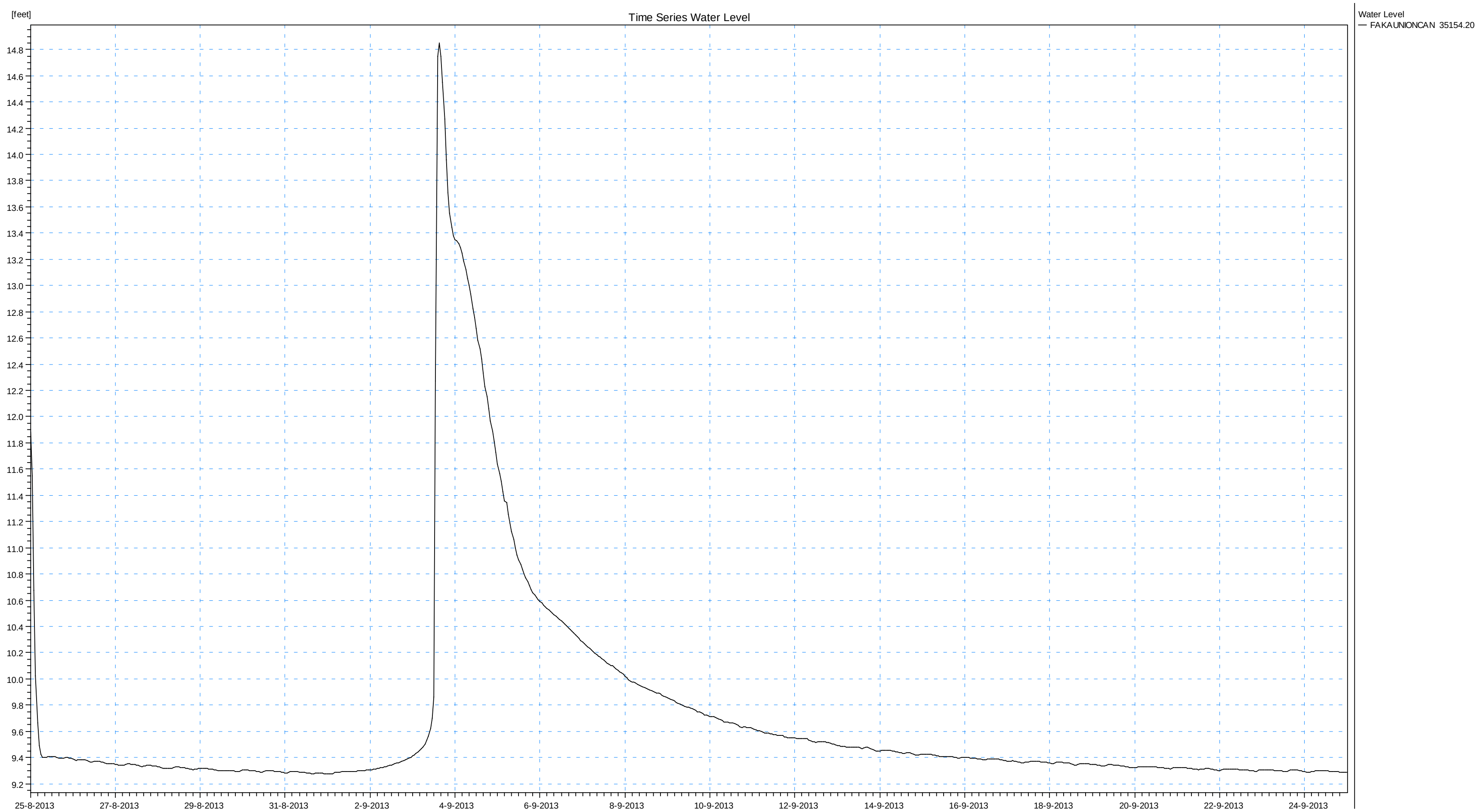


Figure 19. FU5 Tailwater Stage (ft)

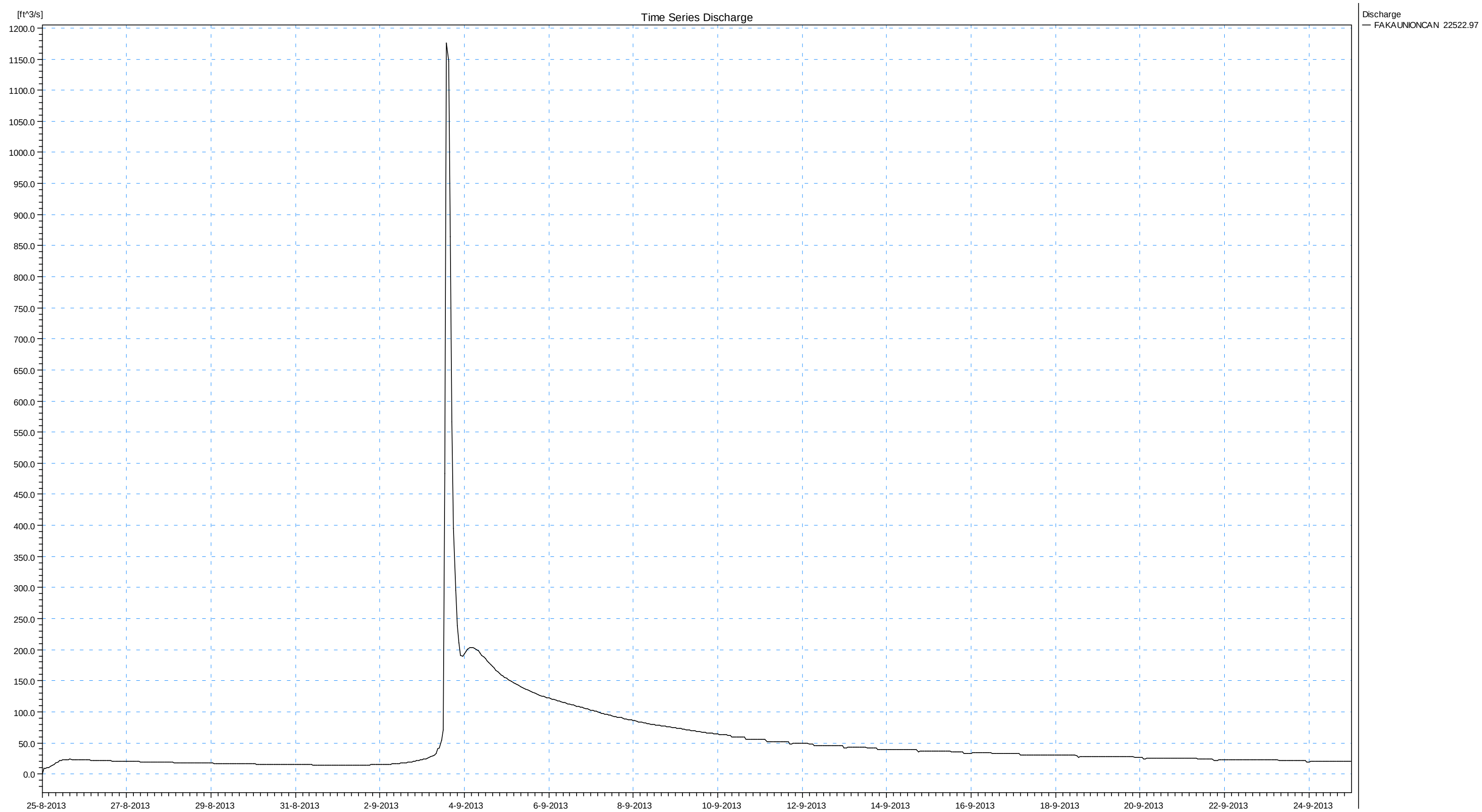


Figure 20. FU6 Discharge (cfs)

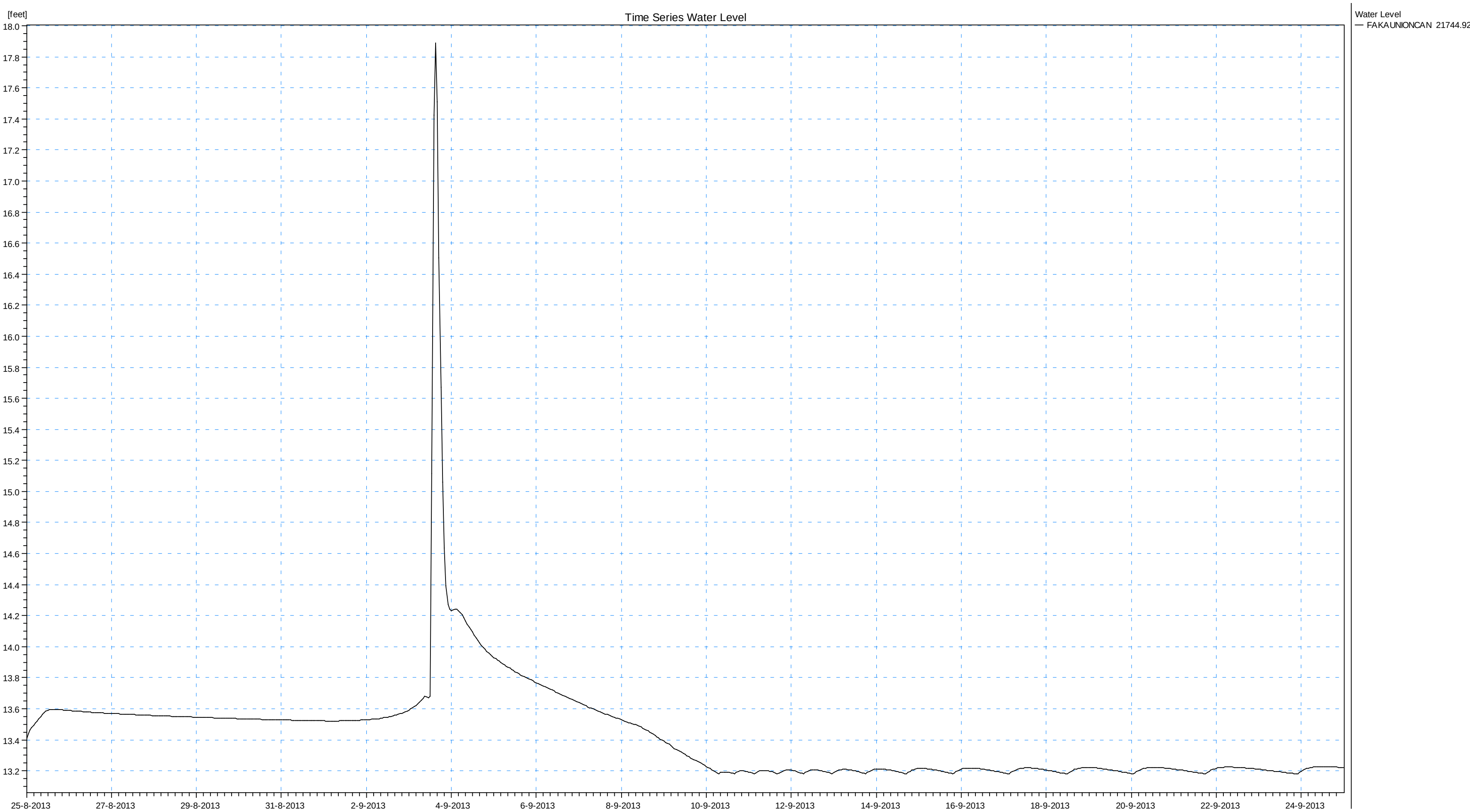


Figure 21. FU6 Headwater Stage (ft)

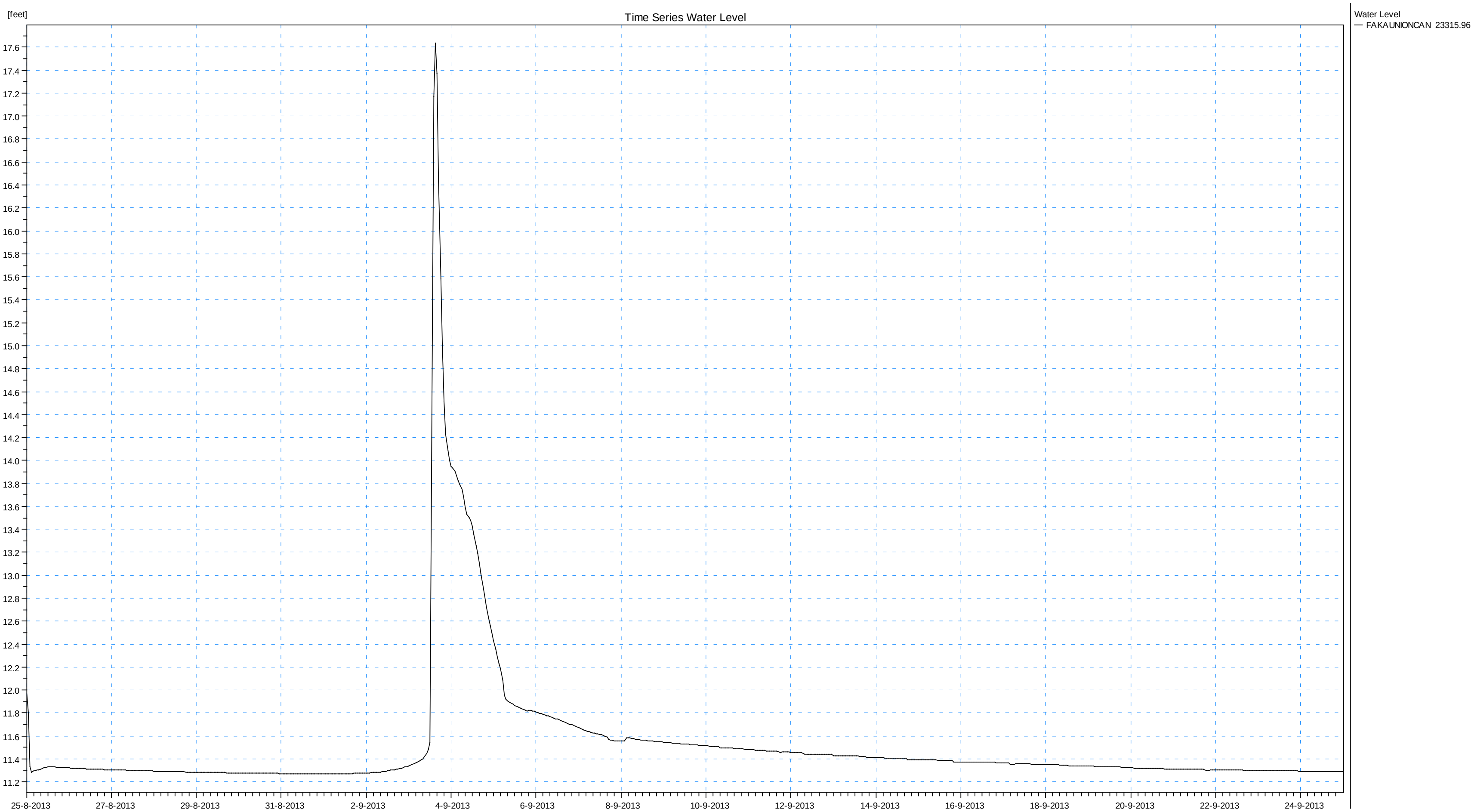


Figure 22. FU6 Tailwater Stage (ft)

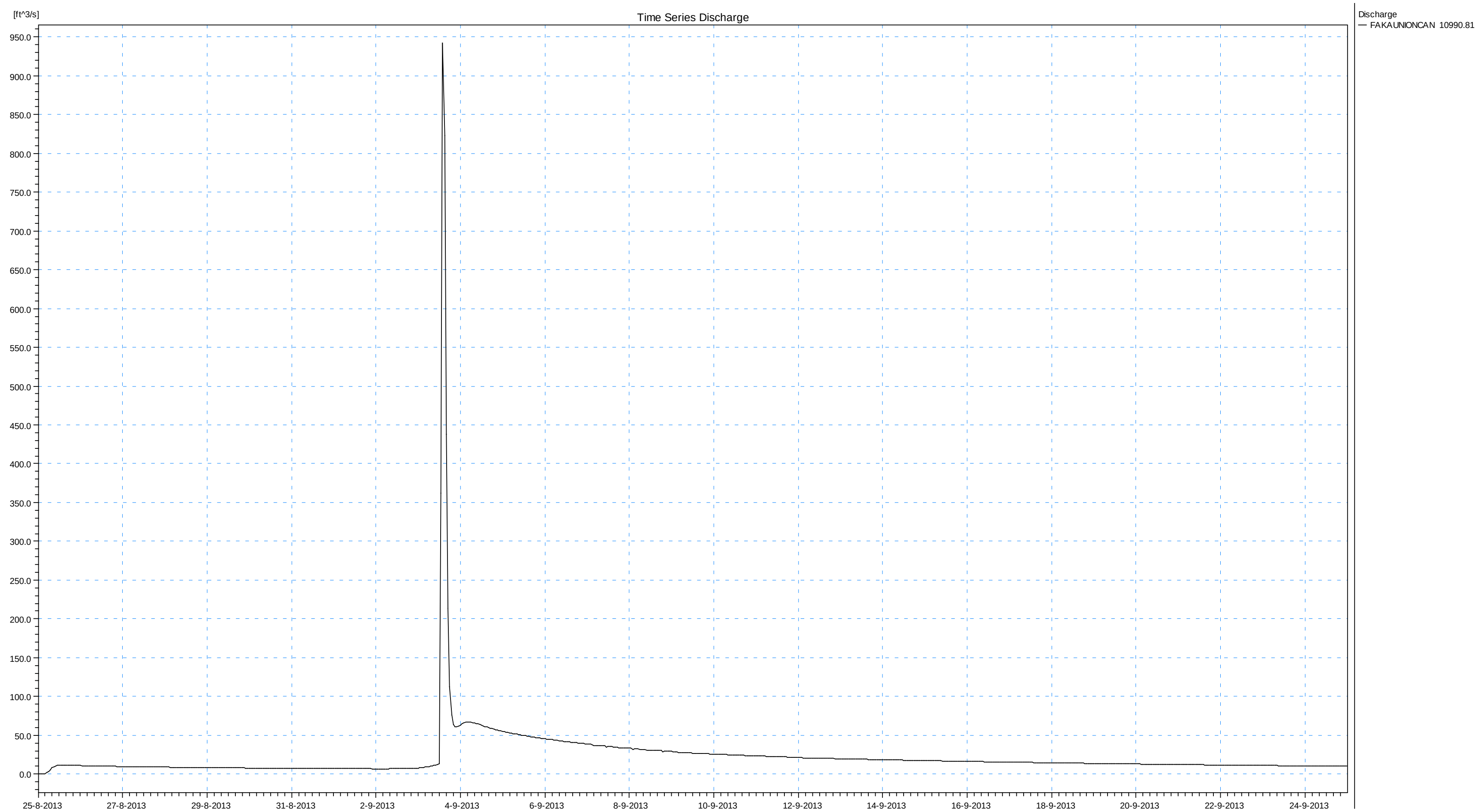


Figure 23. FU7 Discharge (cfs)

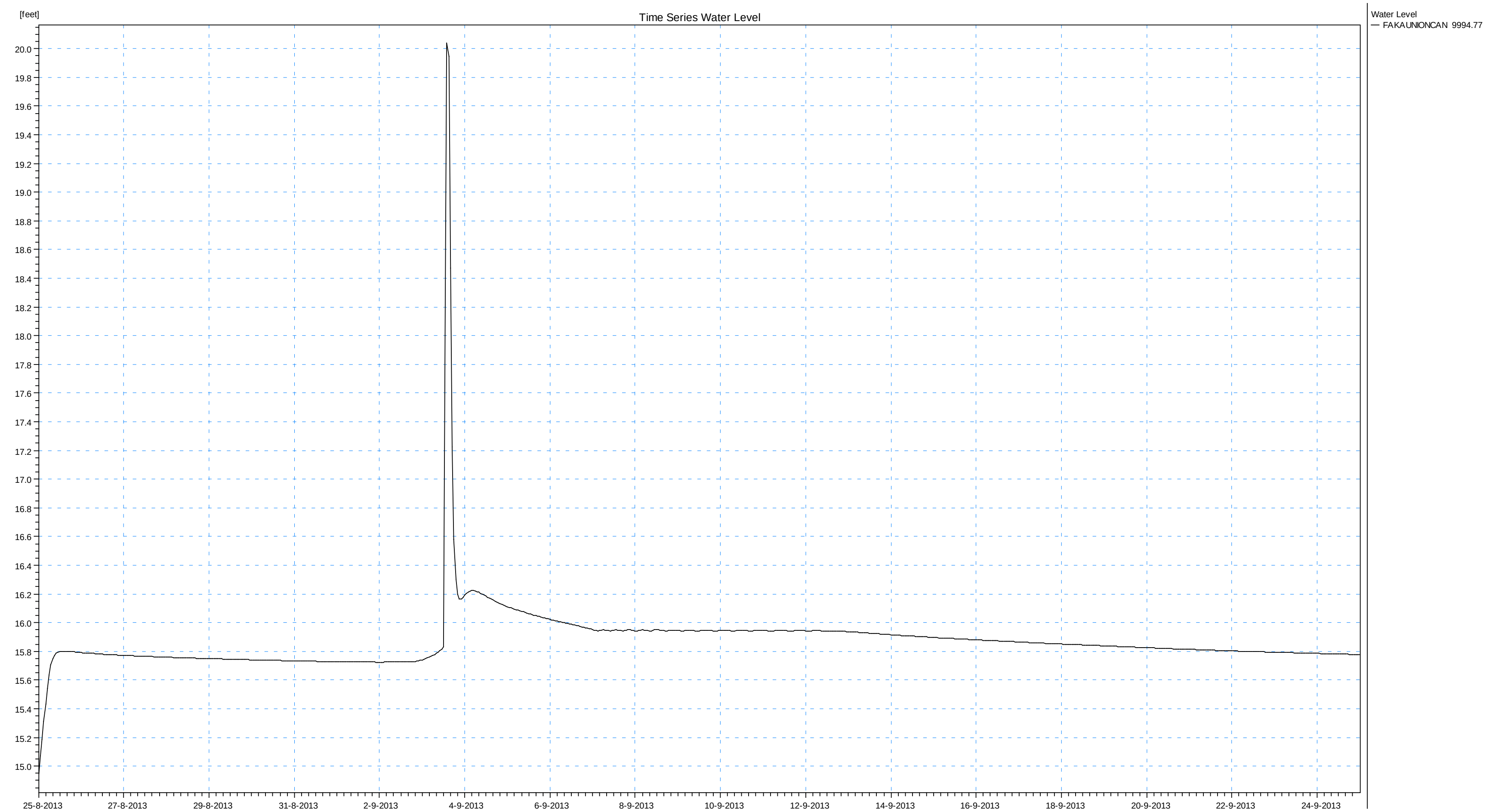


Figure 24. FU7 Headwater Stage (ft)

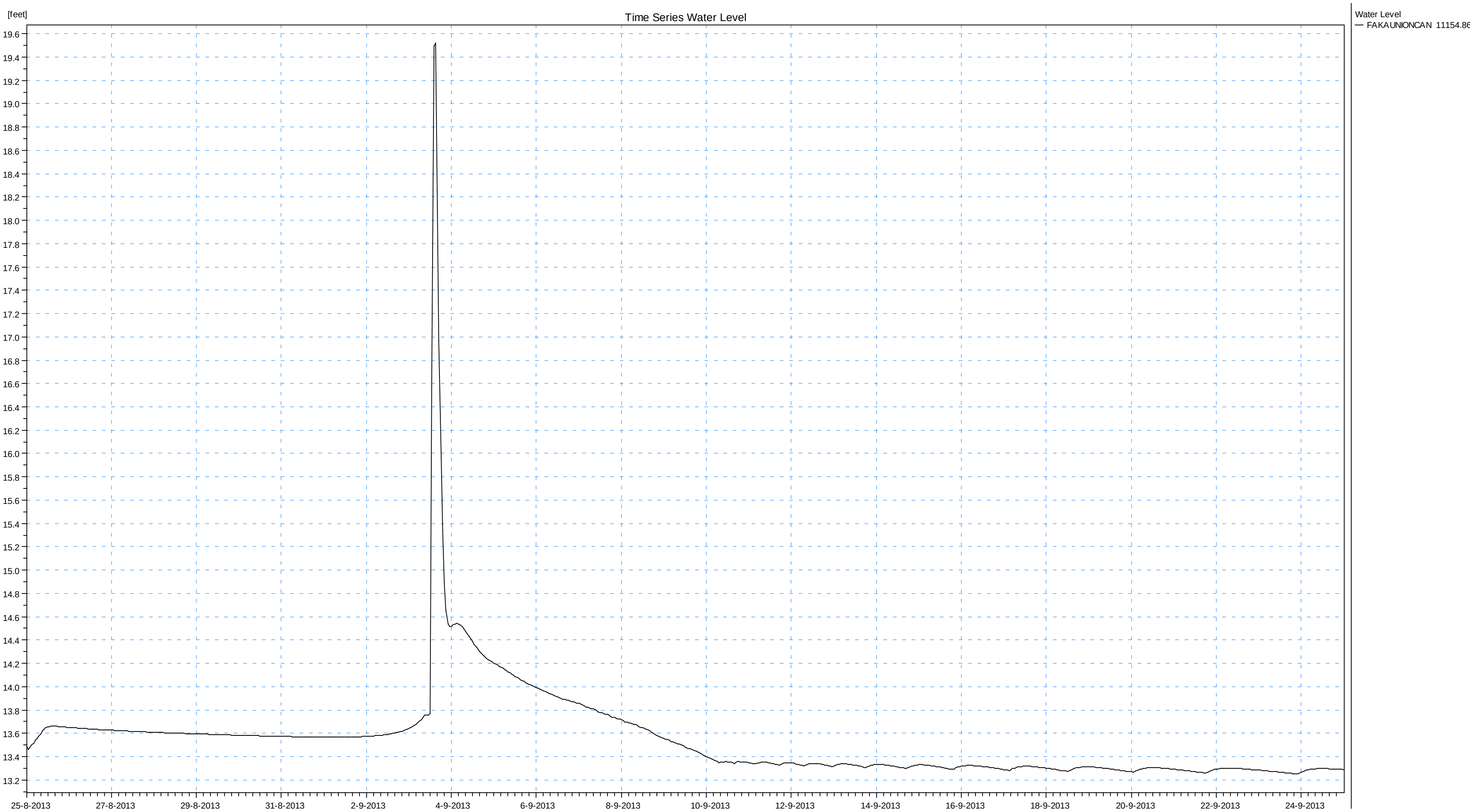


Figure 25. FU7 Tailwater Stage (ft)

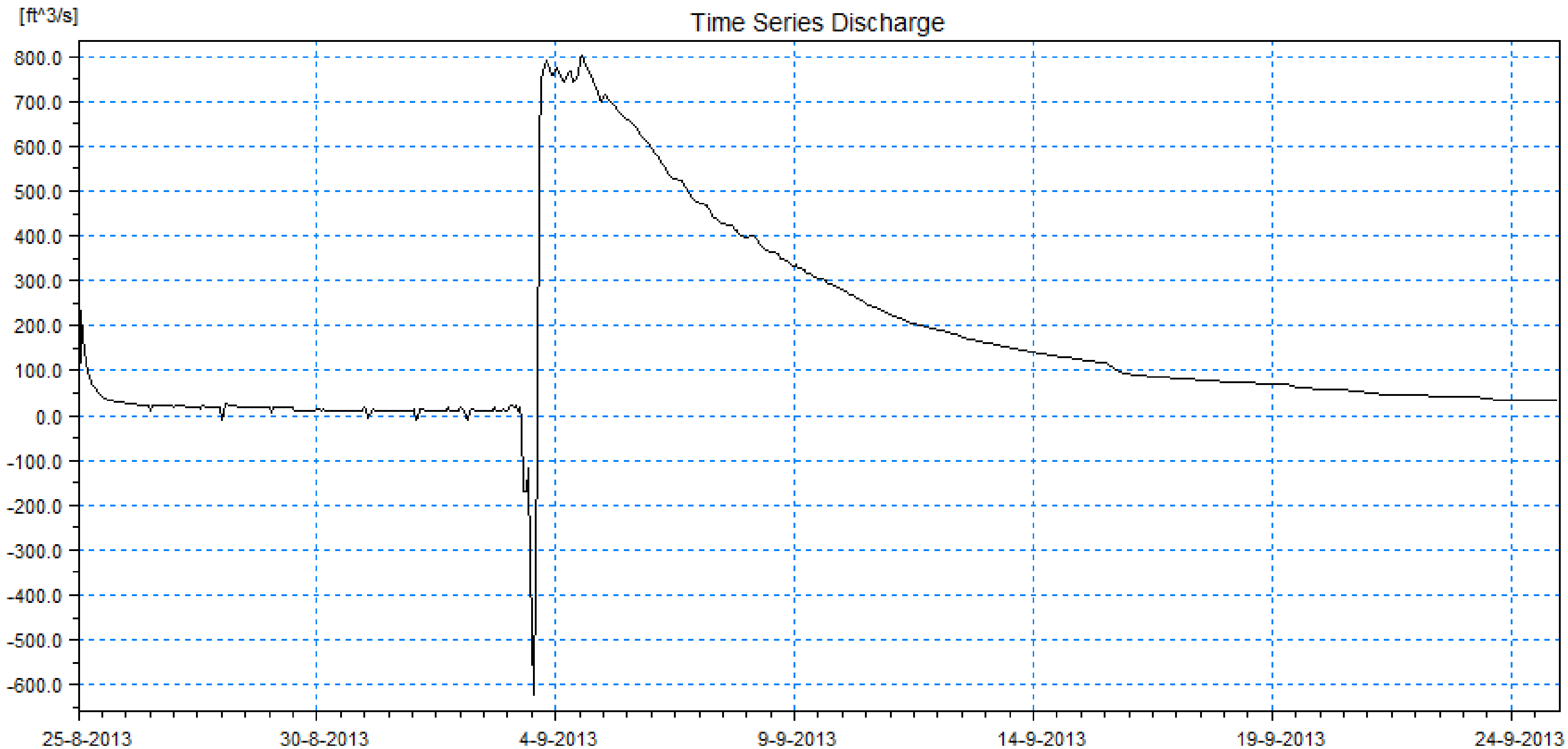


Figure 26. HC1 Discharge (cfs)

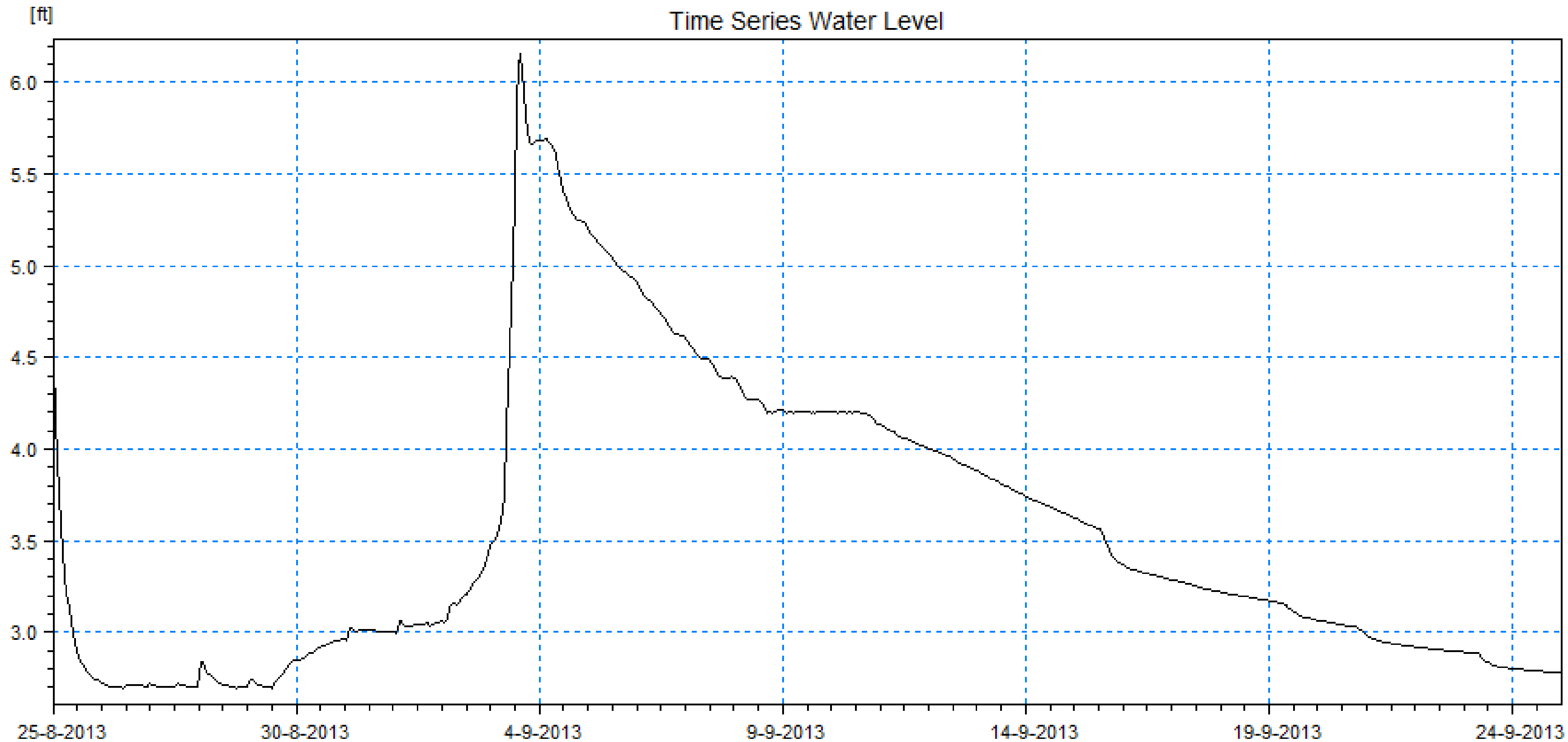


Figure 27. HC1 Headwater Stage (ft)

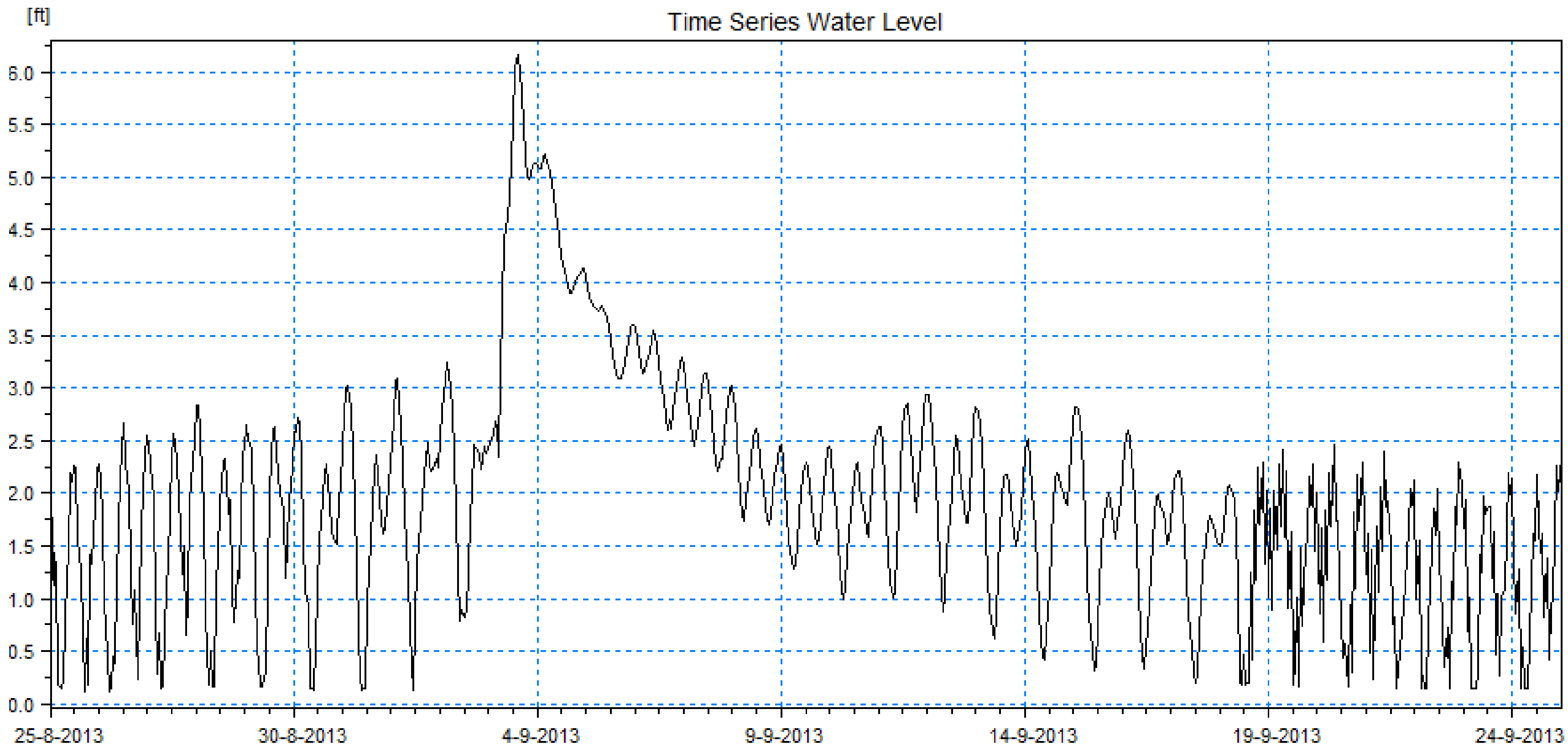


Figure 28. HC1 Tailwater Stage (ft)

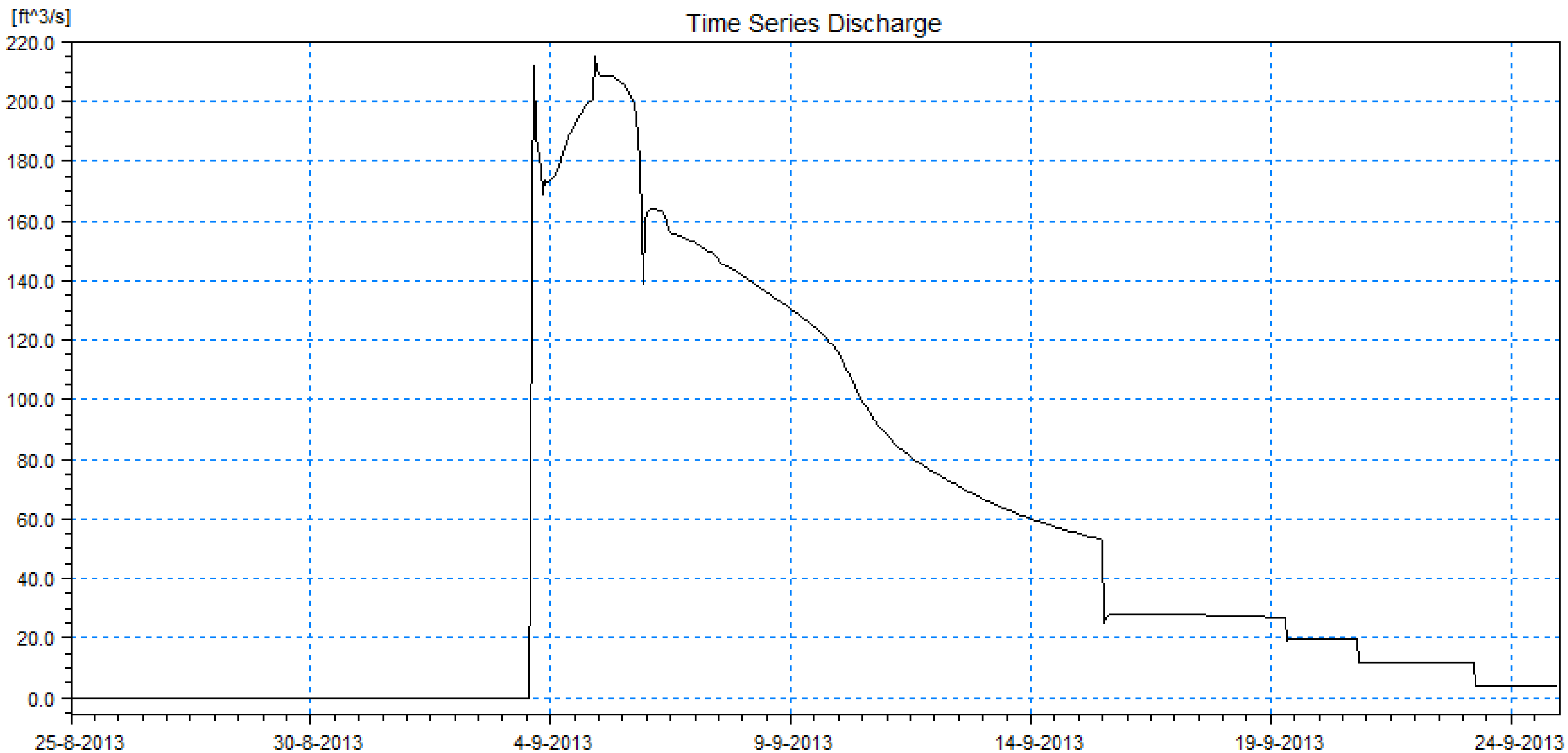


Figure 29. HC2 Discharge (cfs)

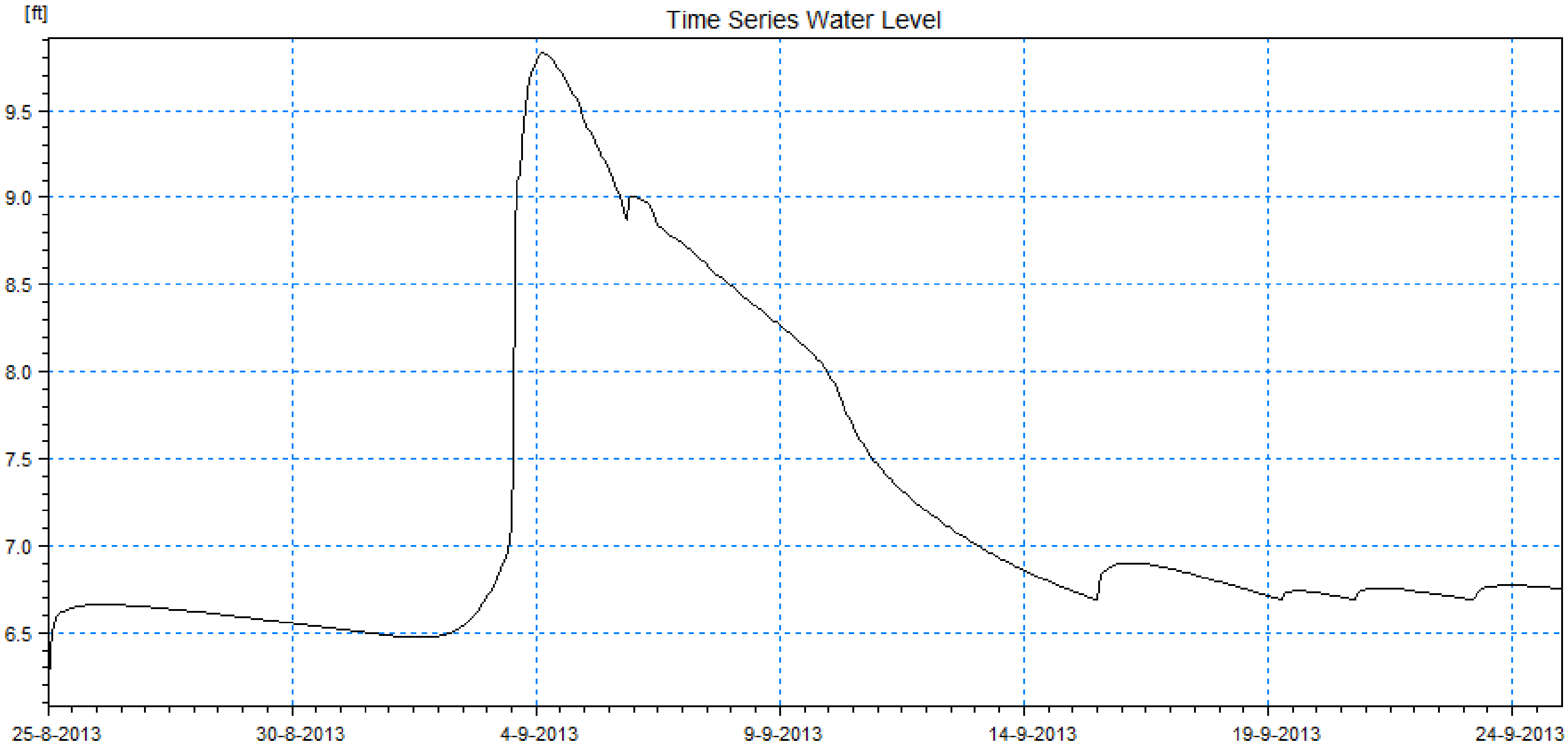


Figure 30. HC2 Headwater Stage (ft)

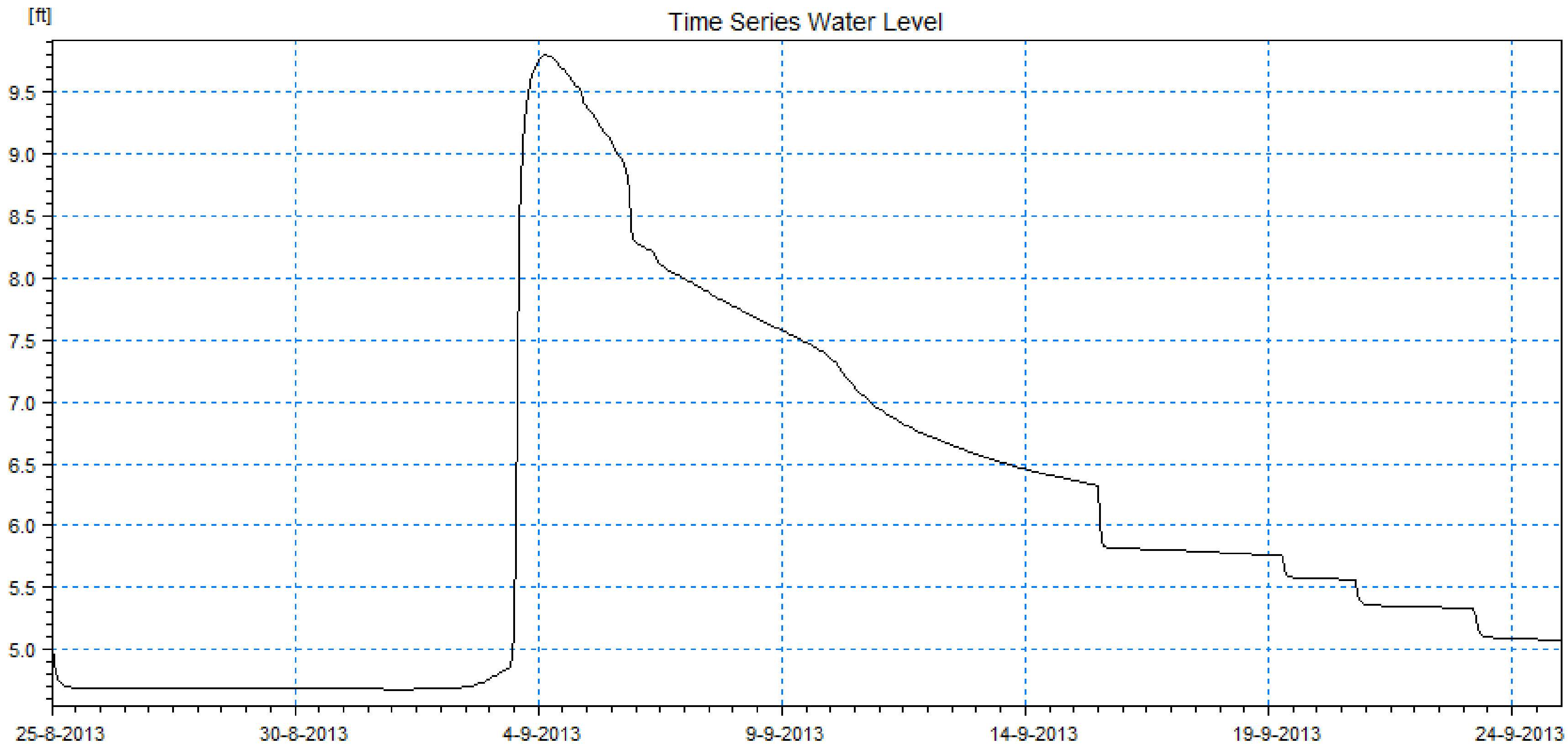


Figure 31. HC2 Tailwater Stage (ft)

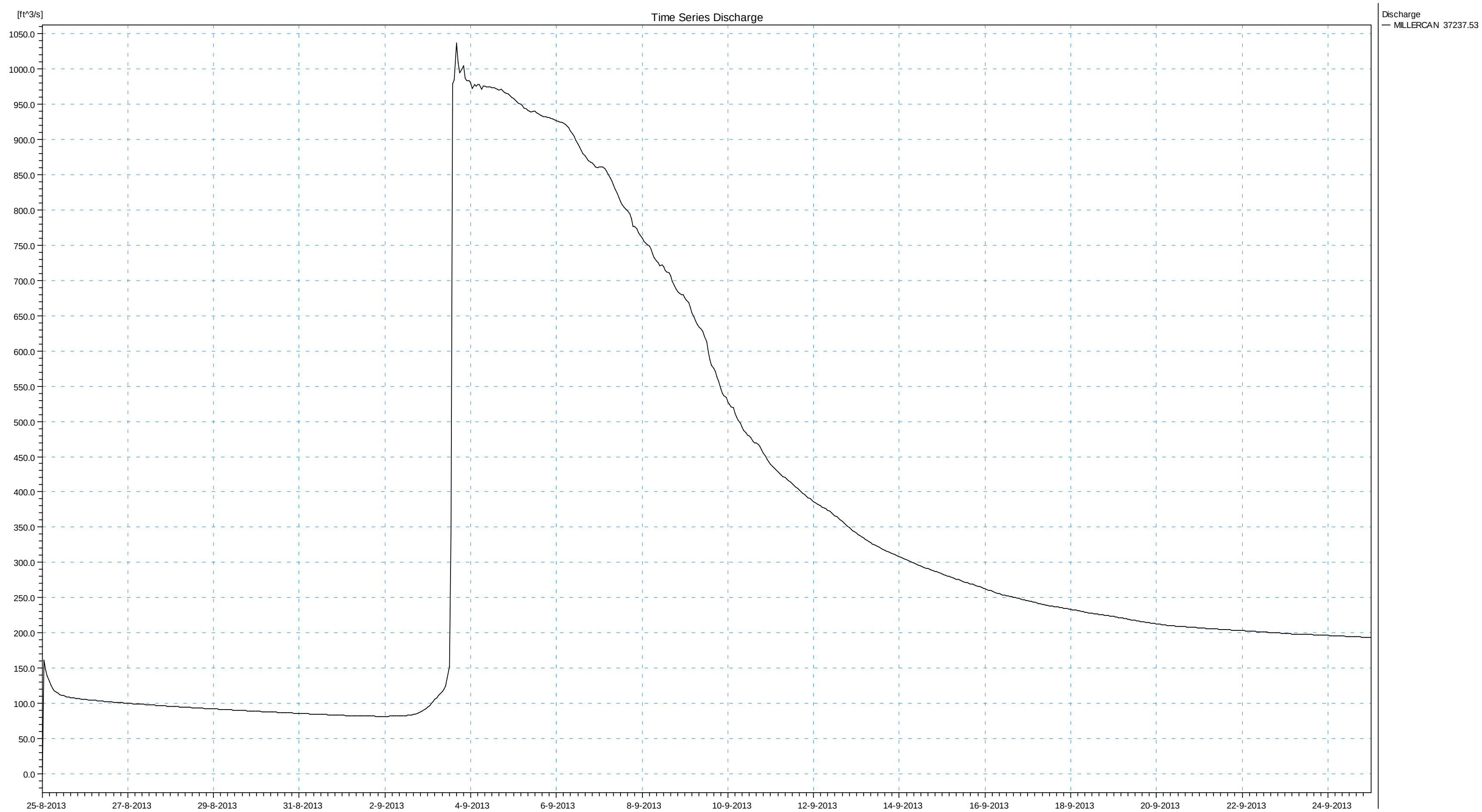


Figure 32. Miller2 Discharge (cfs)

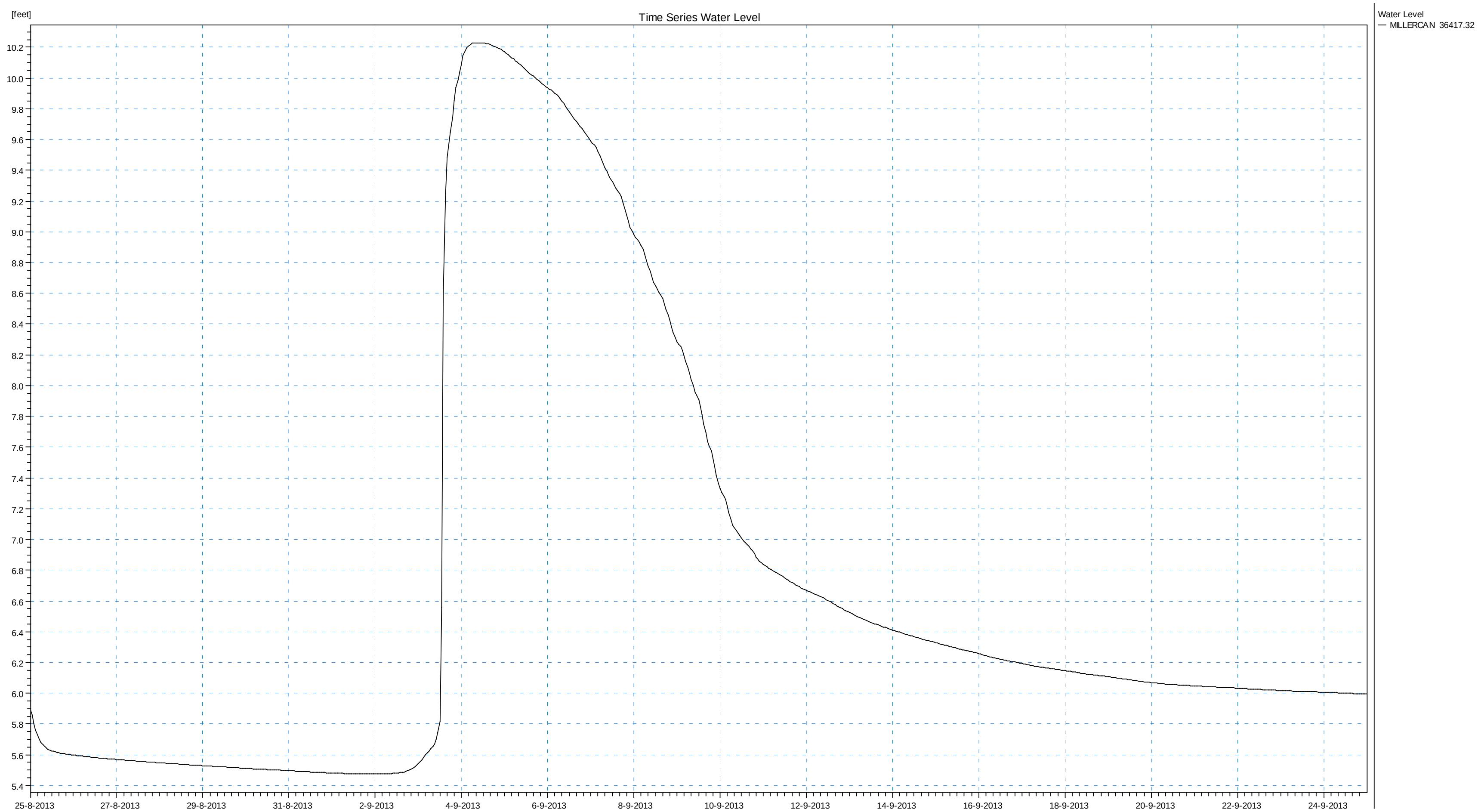


Figure 33. Miller2 Headwater Stage (ft)

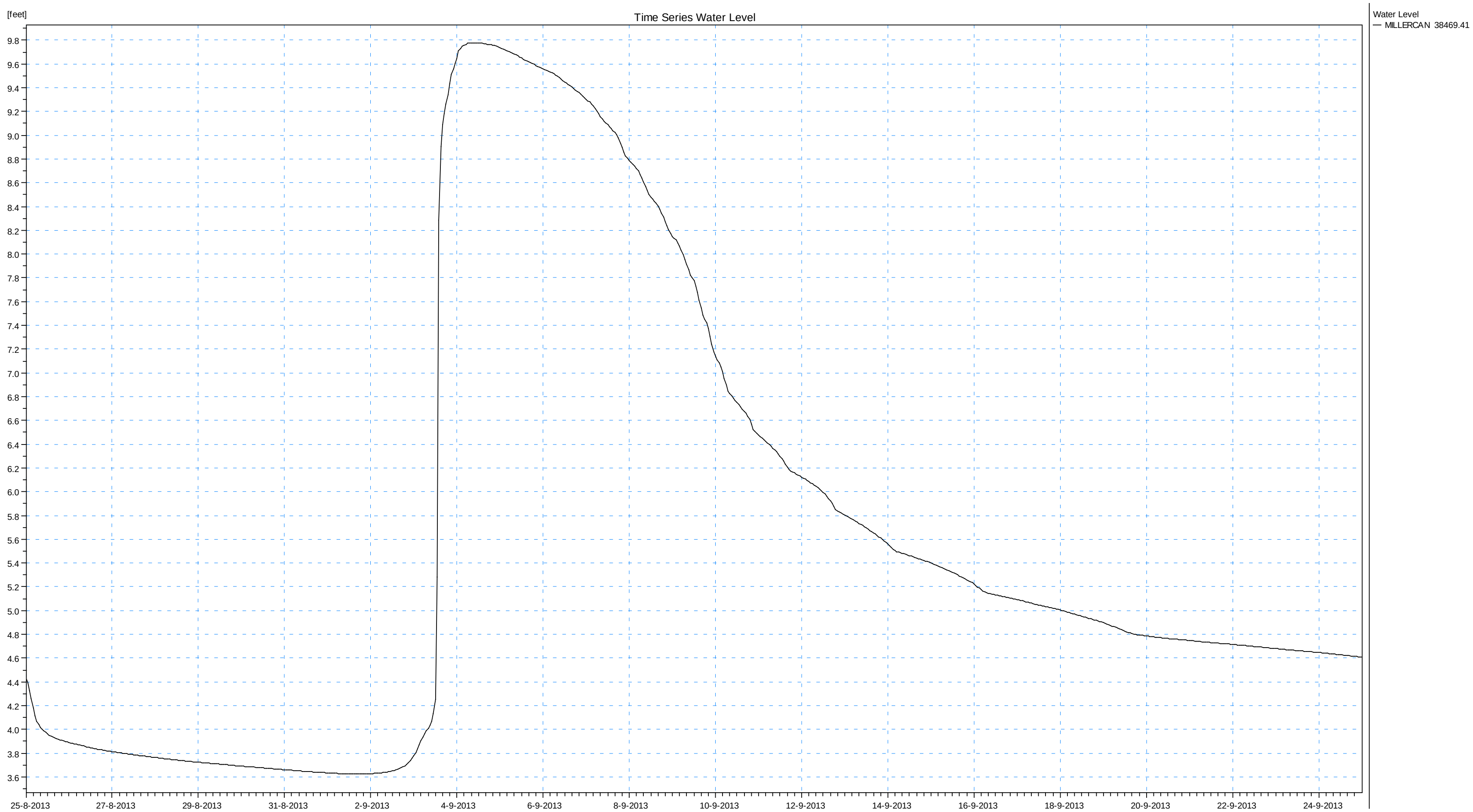


Figure 34. Miller2 Tailwater Stage (ft)

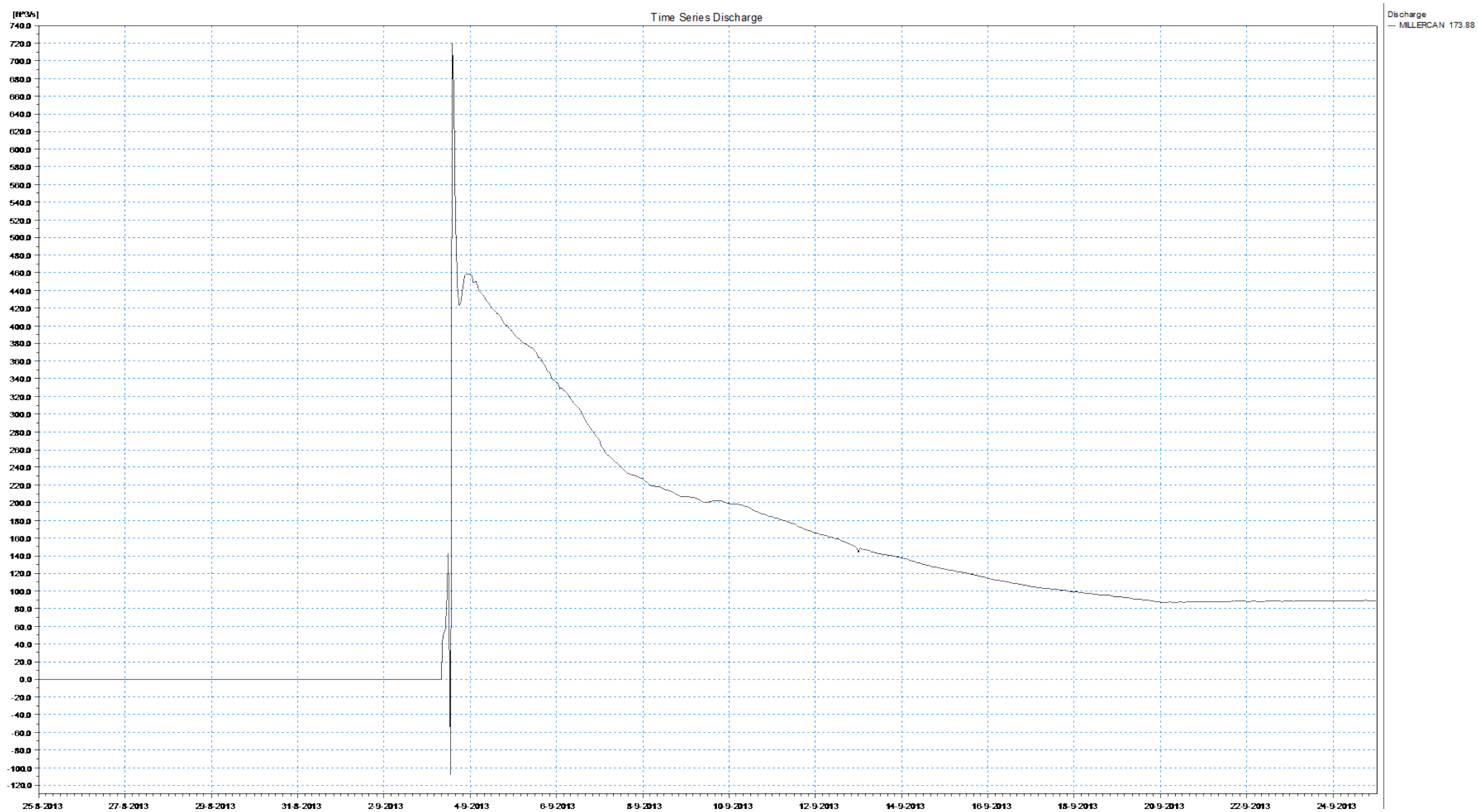


Figure 35. Miller3 Discharge (cfs)

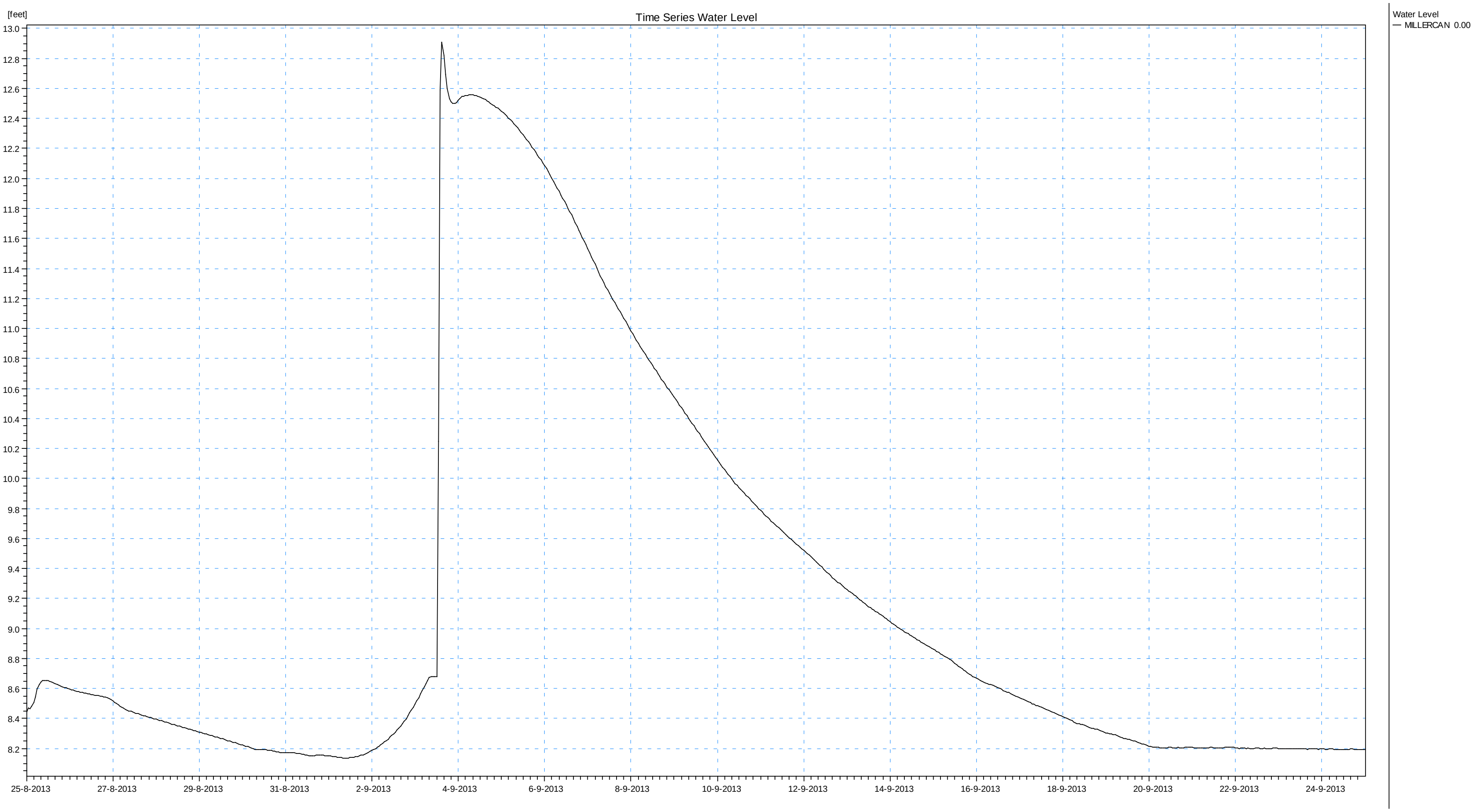


Figure 36. Miller3 Headwater Stage (ft)

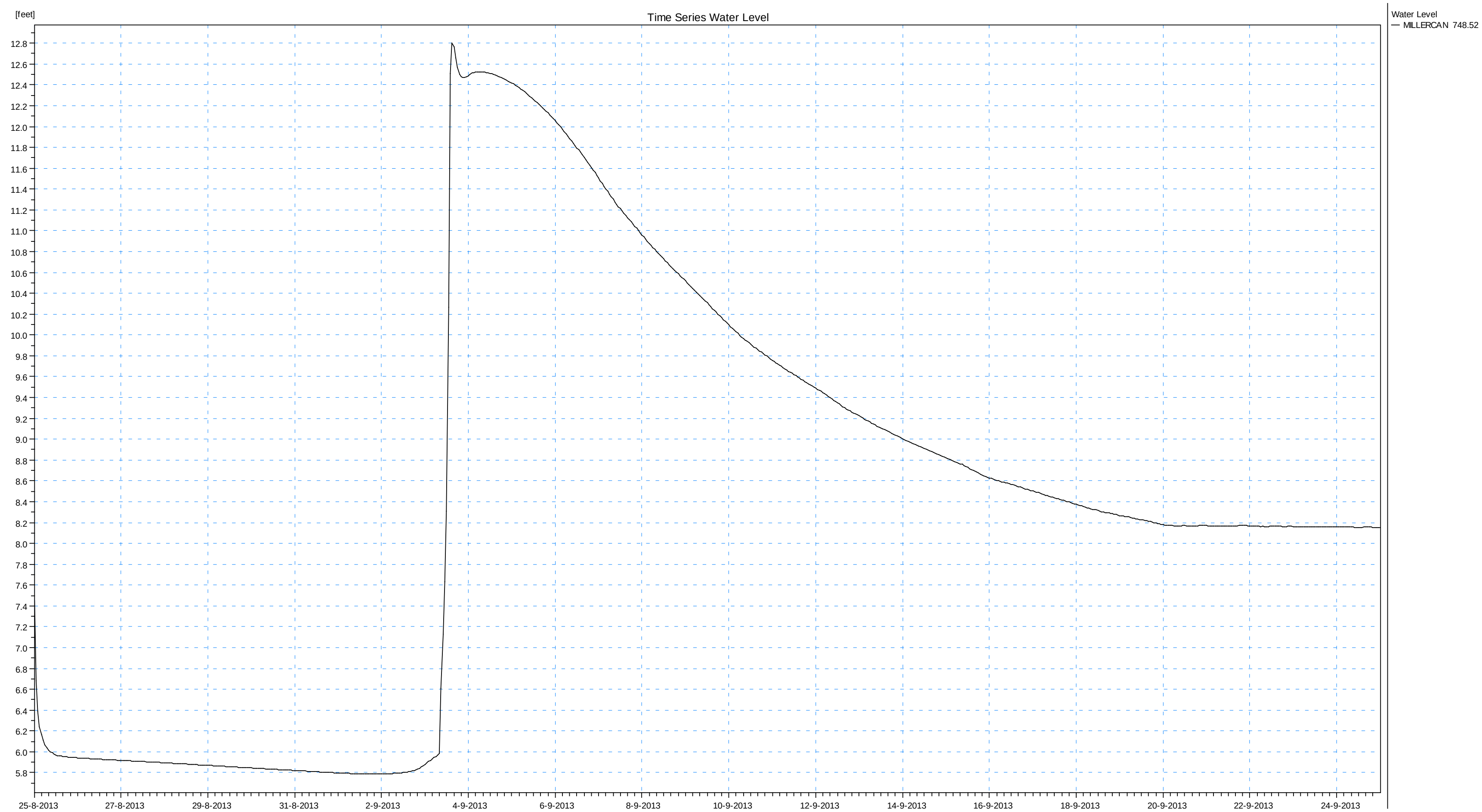
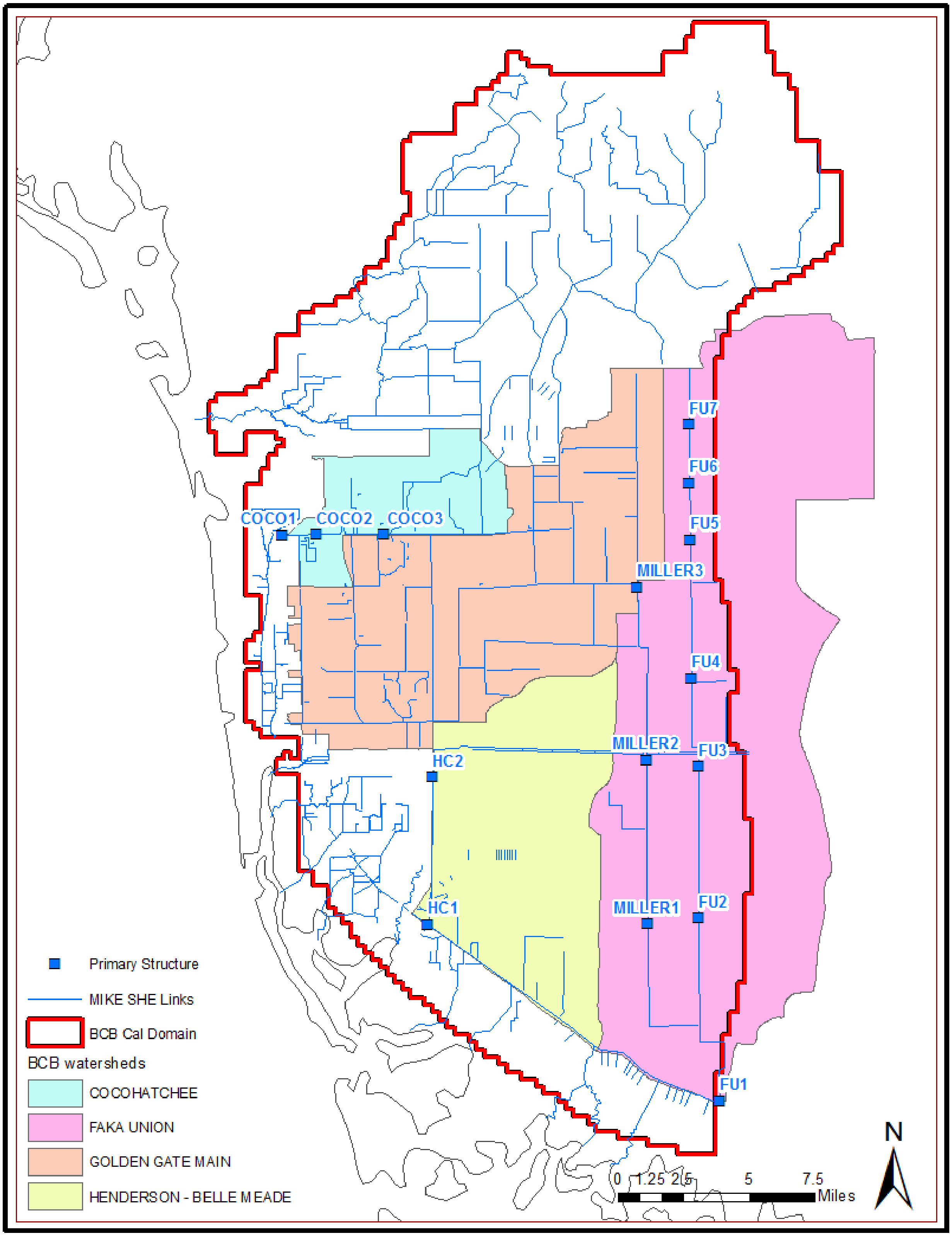


Figure 37. Miller3 Tailwater Stage (ft)

F – MIKE 11 Detailed Time Series for 5-year Storm, Future Conditions



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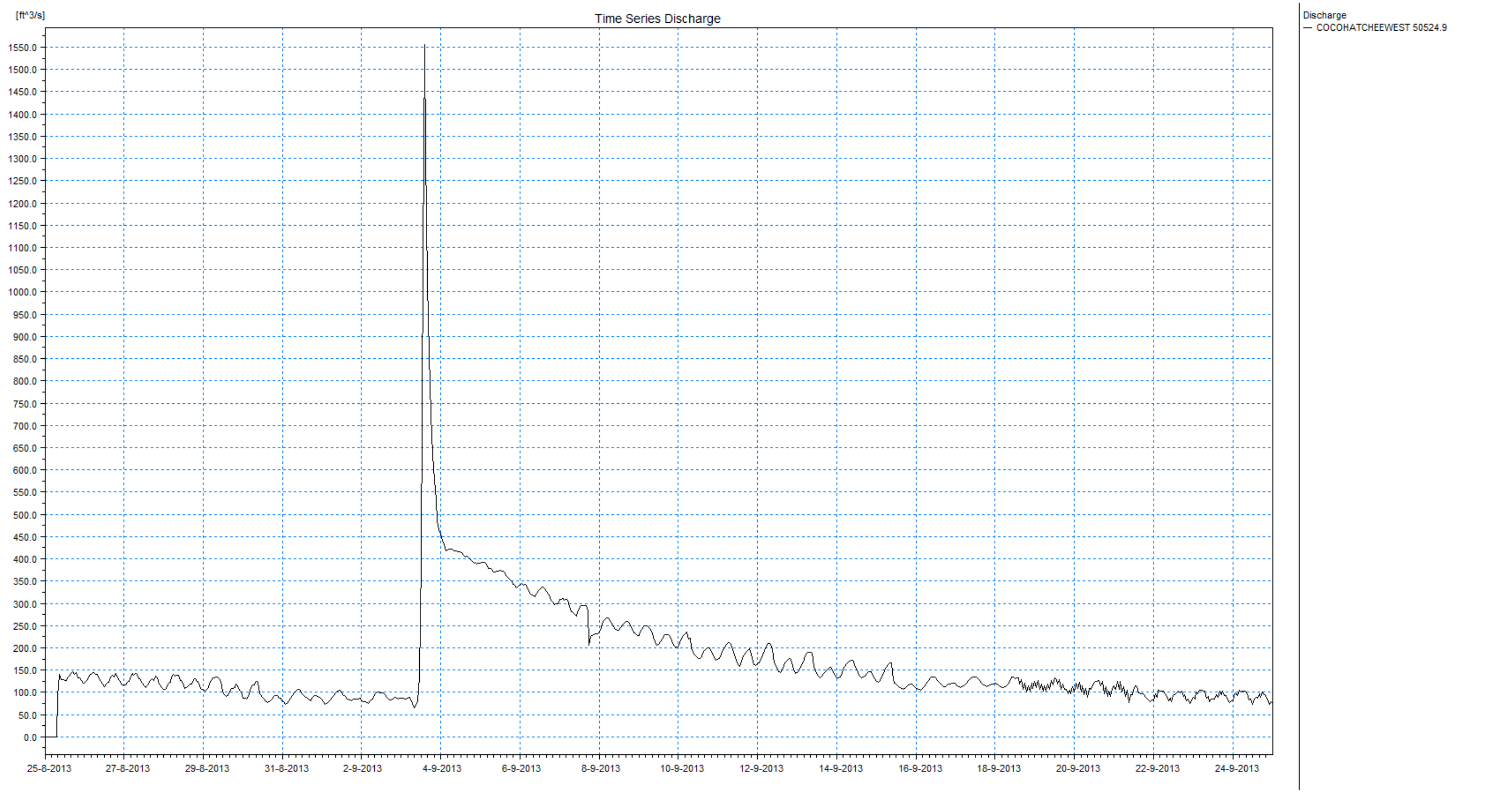

Figure 2. COCO1 Discharge (cfs)

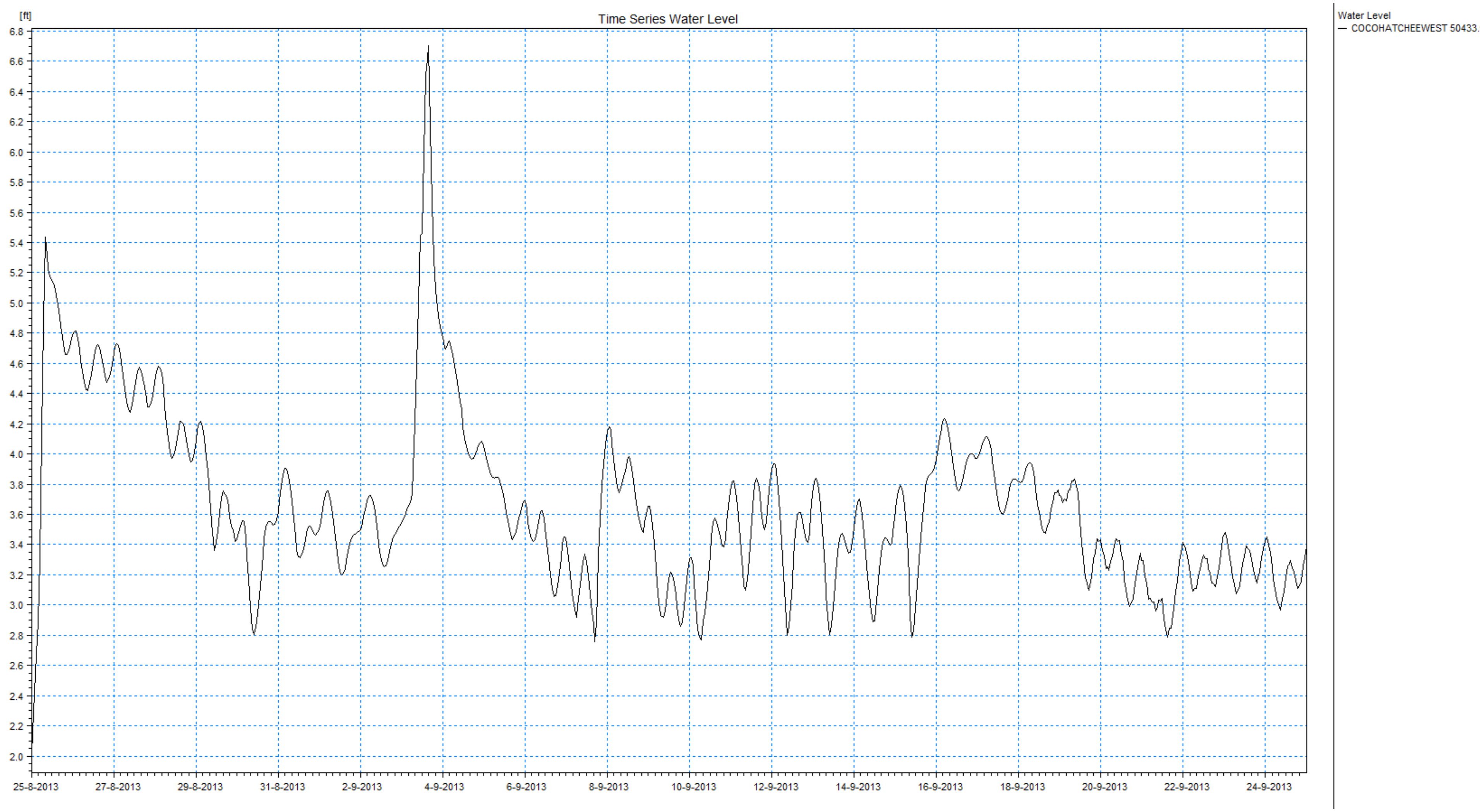

Figure 3. COCO1 Headwater Stage (ft)

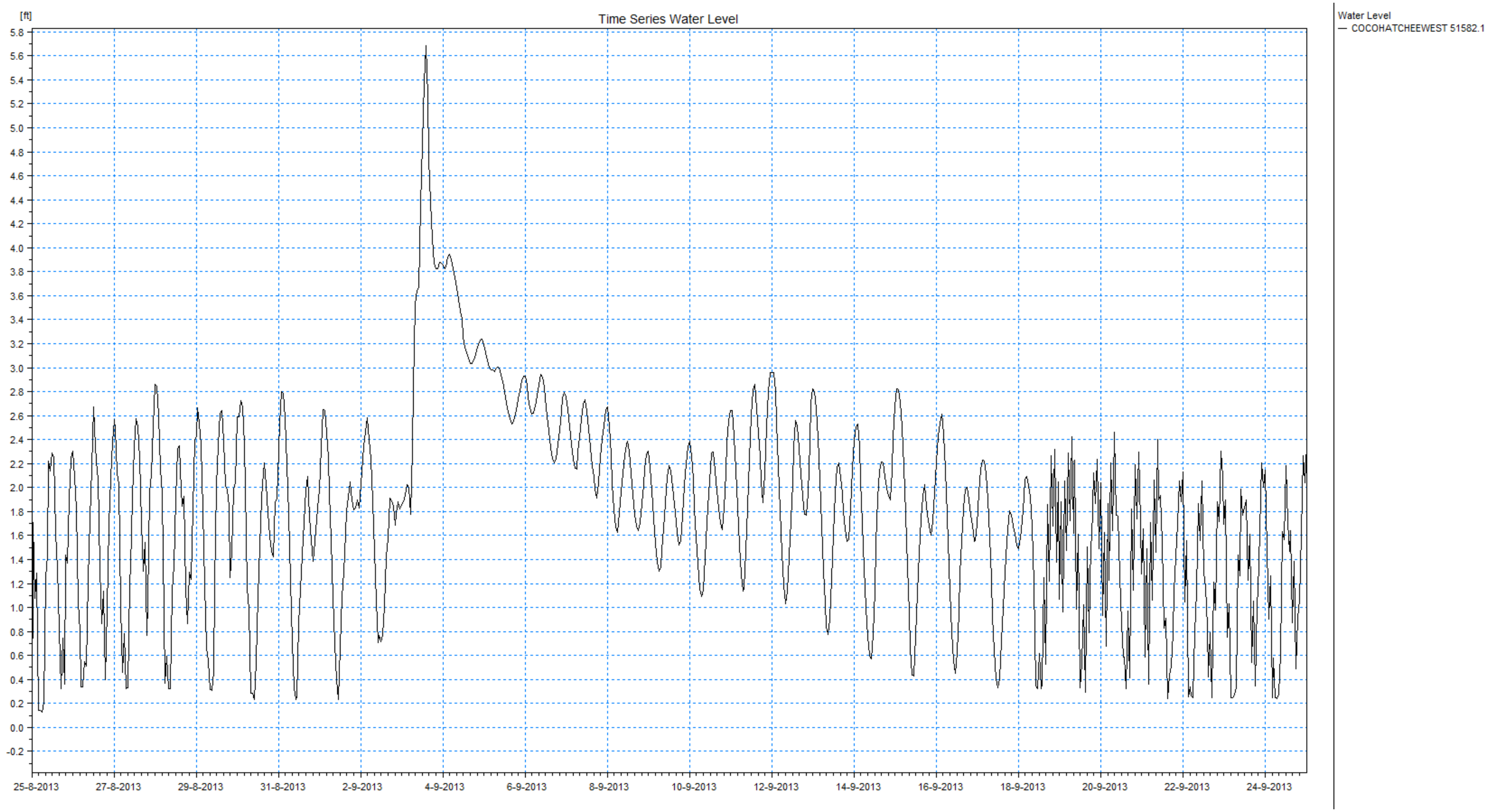

Figure 4. COCO1 Tailwater Stage (ft)

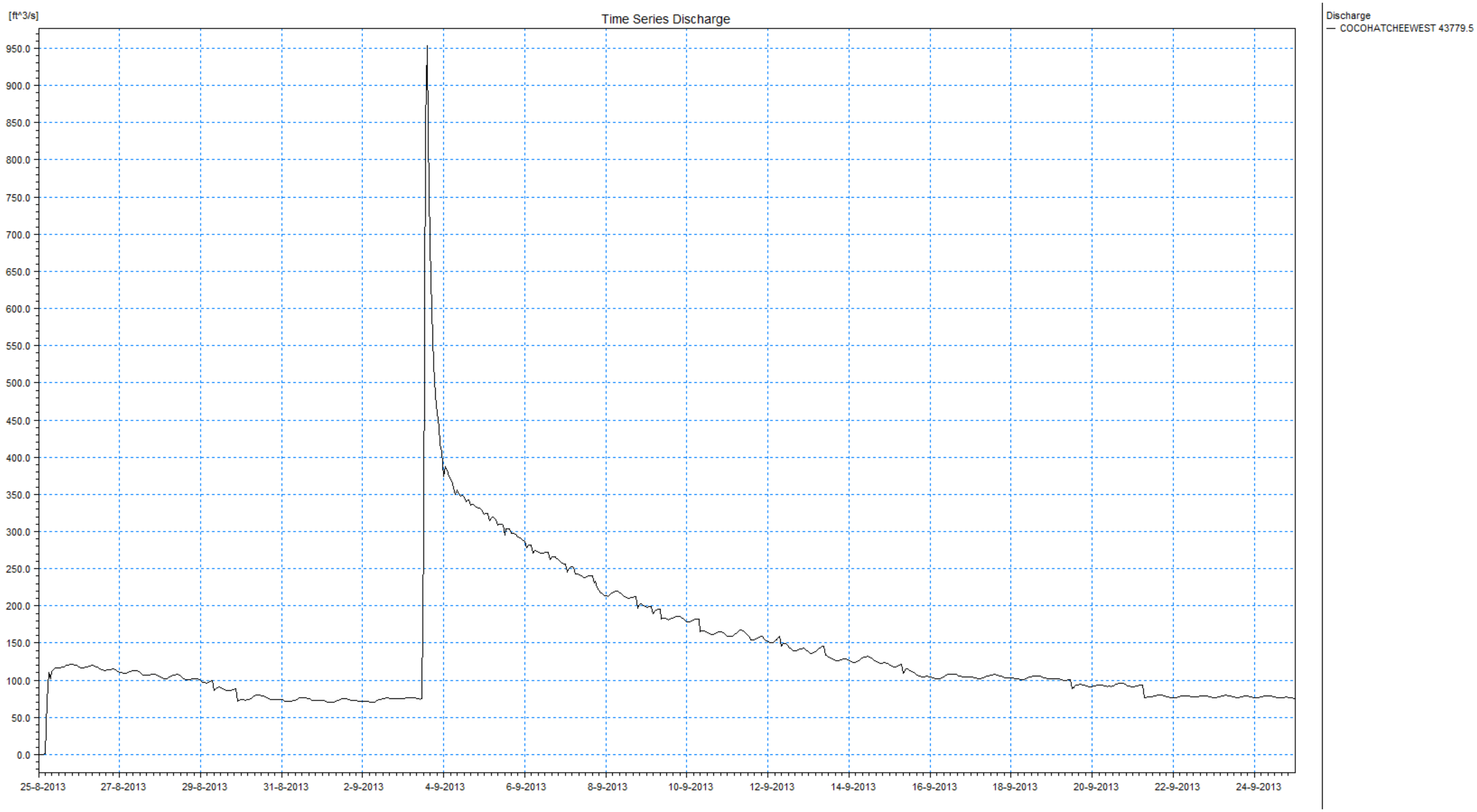

Figure 5. COCO2 Discharge (cfs)

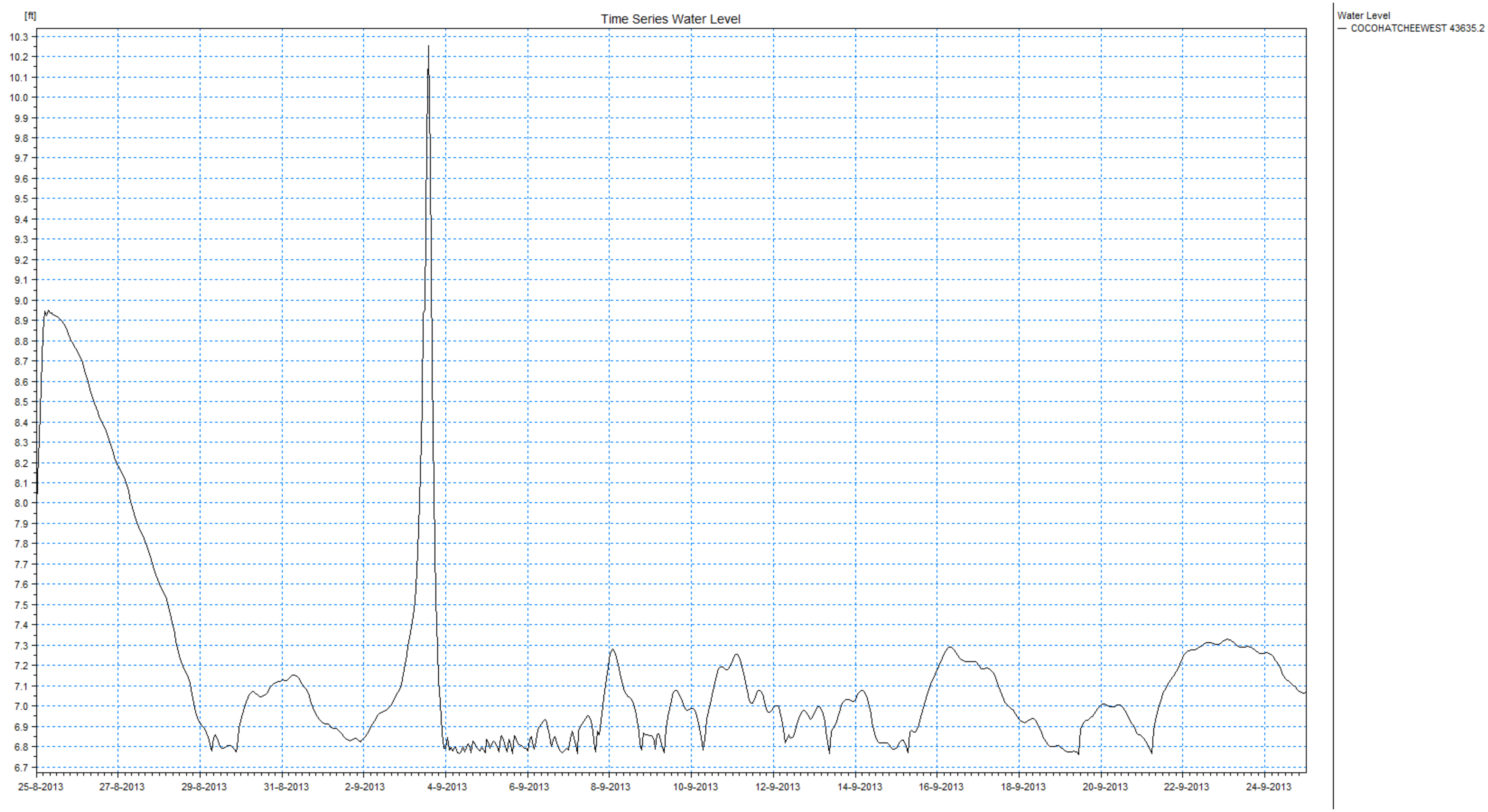

Figure 6. COCO2 Headwater Stage (ft)

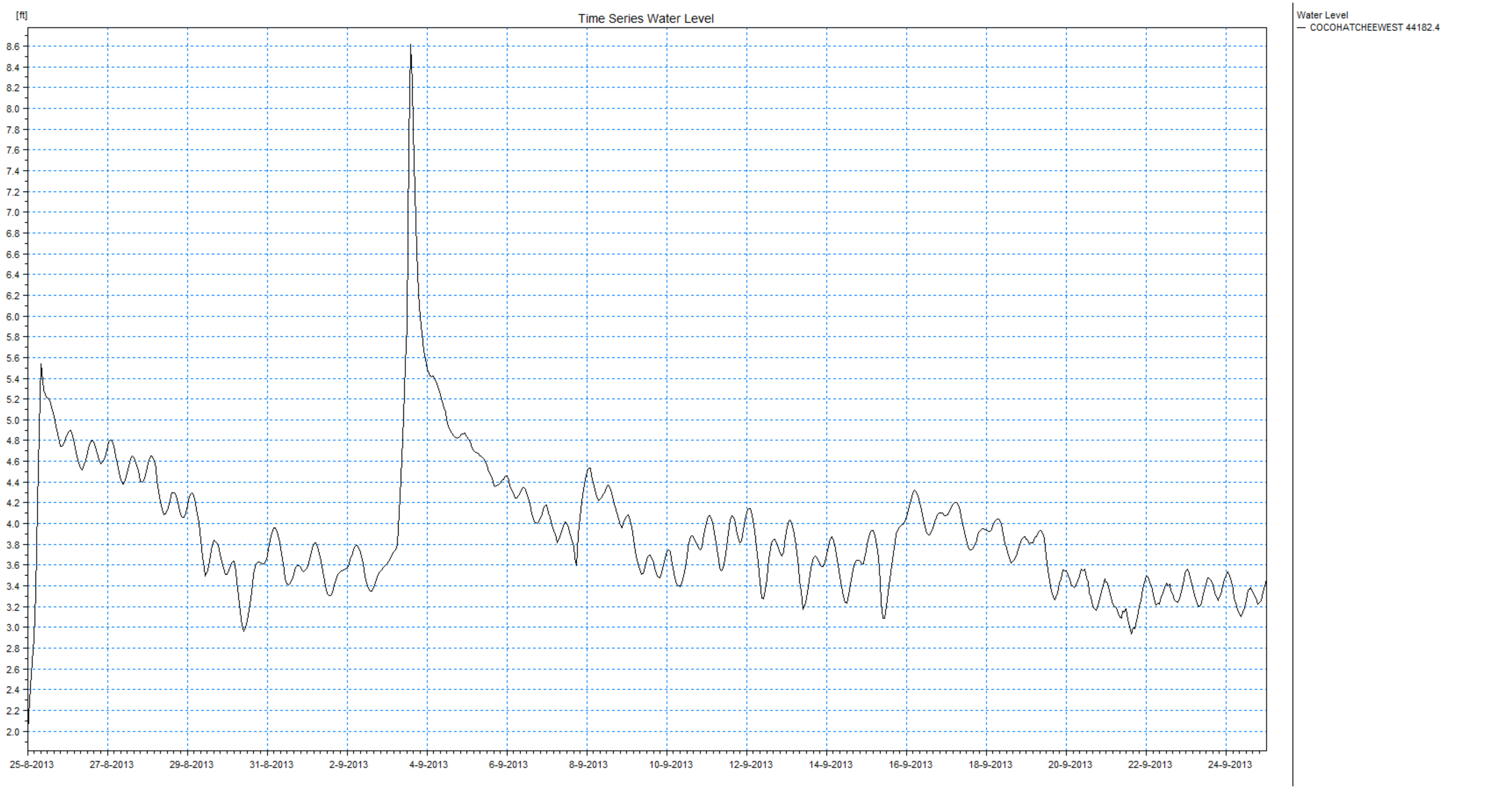

Figure 7. COCO2 Tailwater Stage (ft)

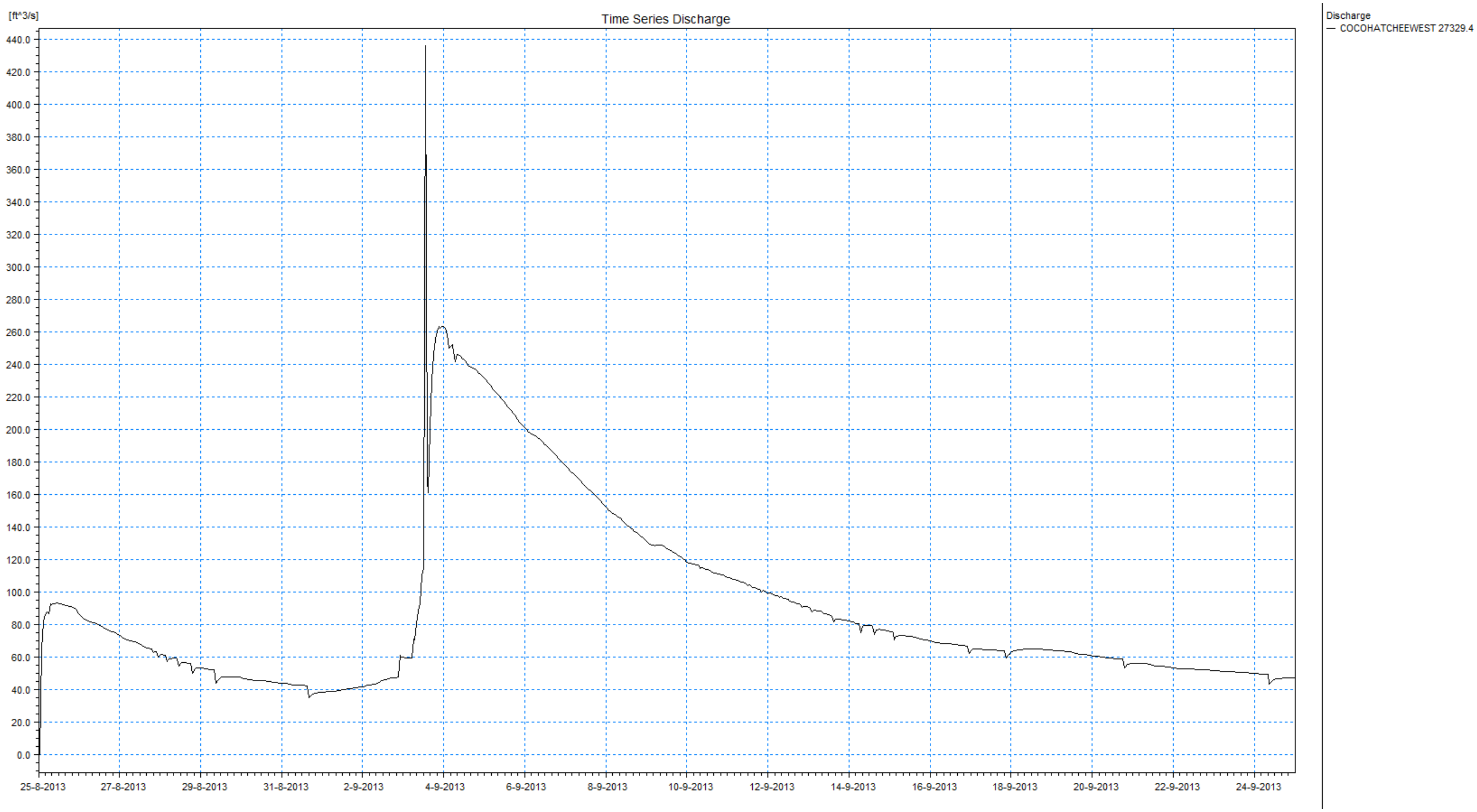

Figure 8. COCO3 Discharge (cfs)

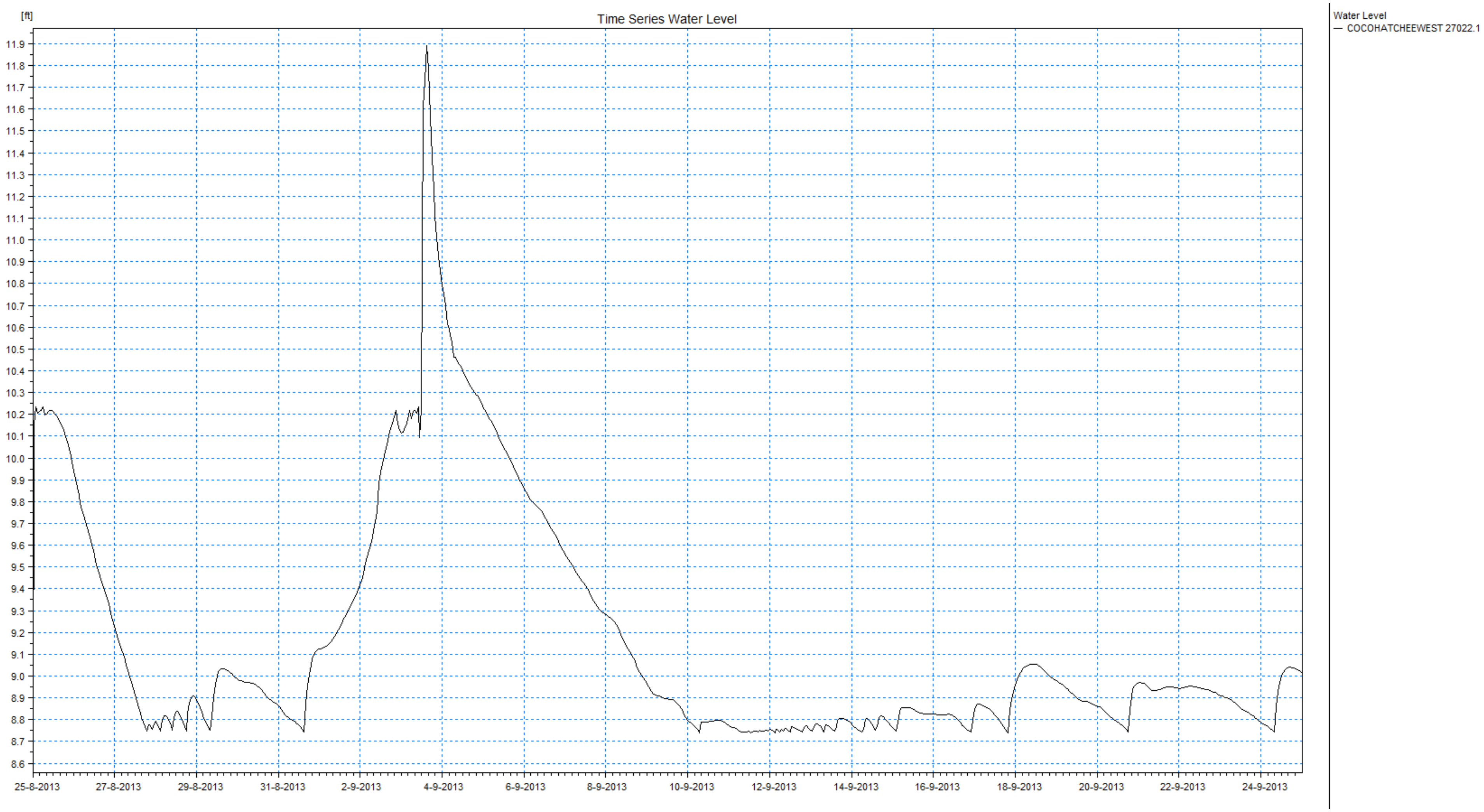

Figure 9. COCO3 Headwater Stage (ft)

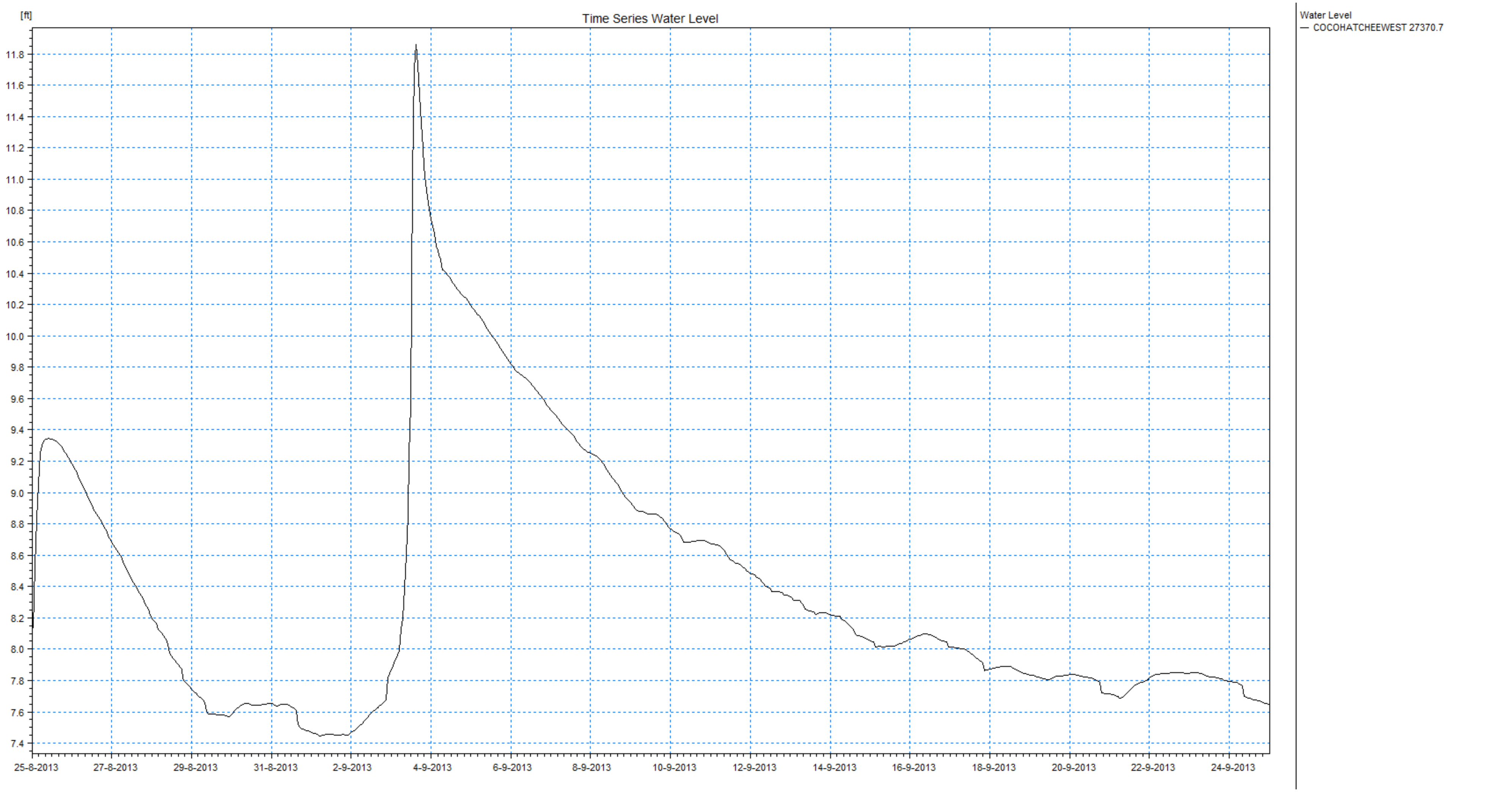

Figure 10. COCO3 Tailwater Stage (ft)

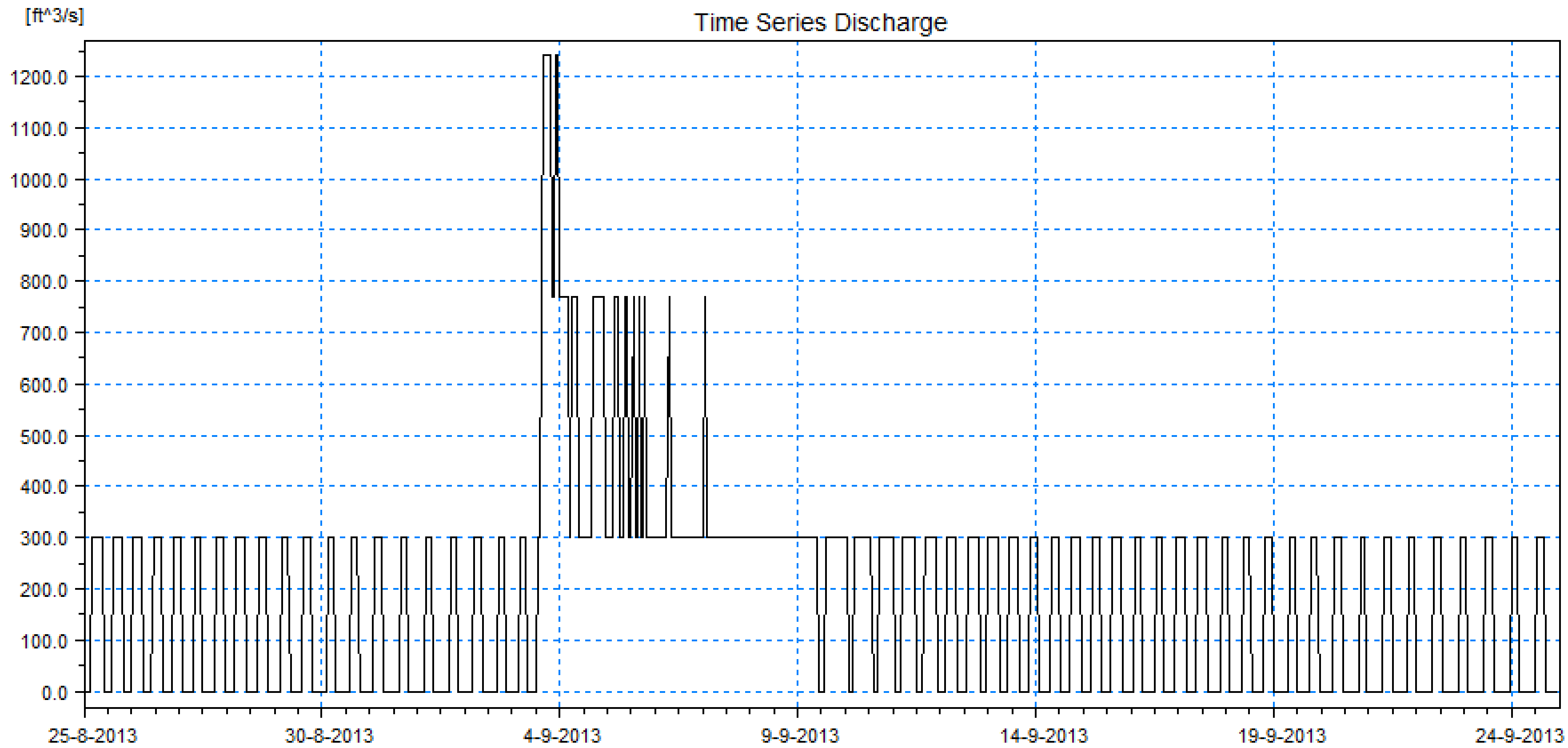

Figure 11: FU Pump Discharge (cfs)

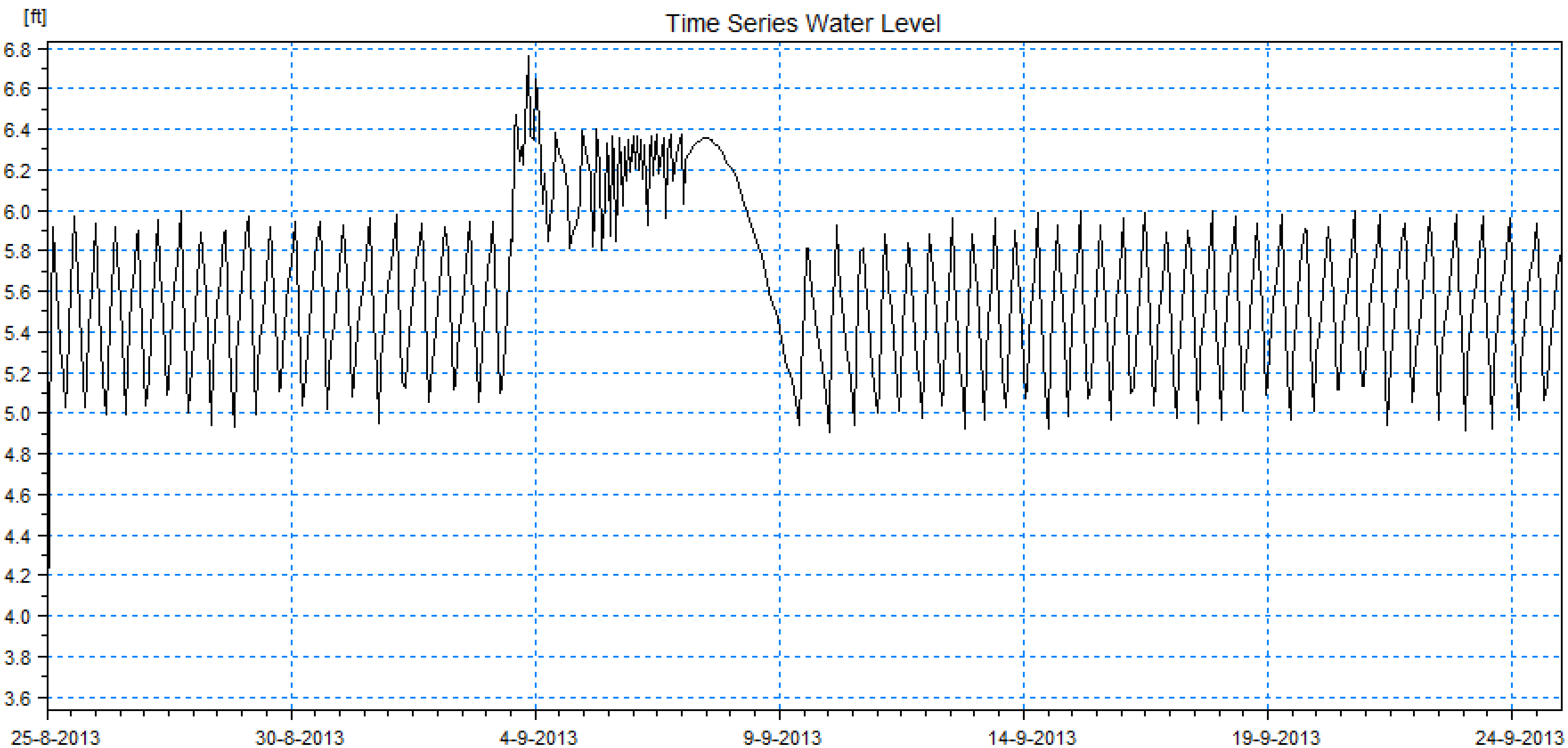

Figure 12- FU Pump Headwater Stage (ft)

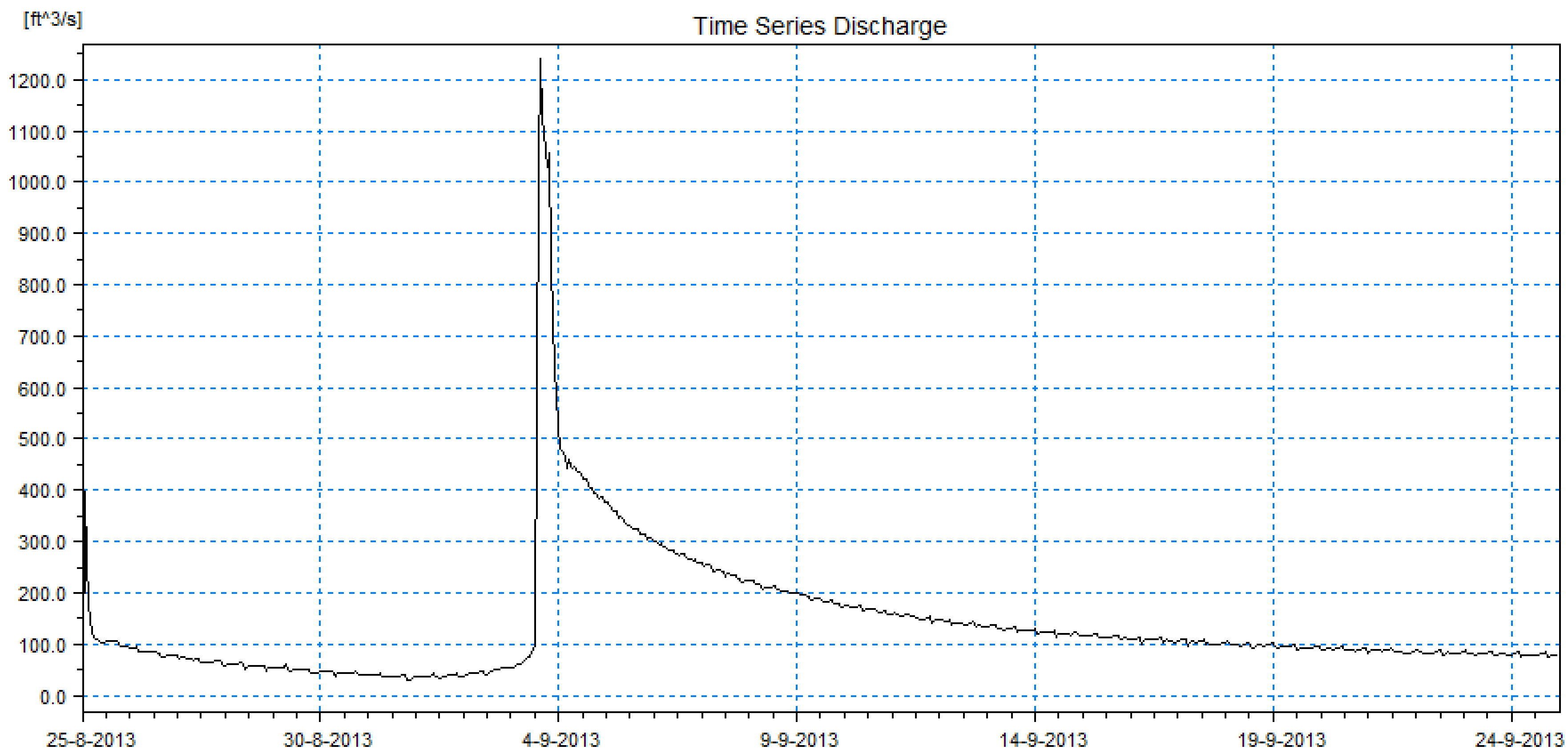

Figure 13. FU4 Discharge (cfs)

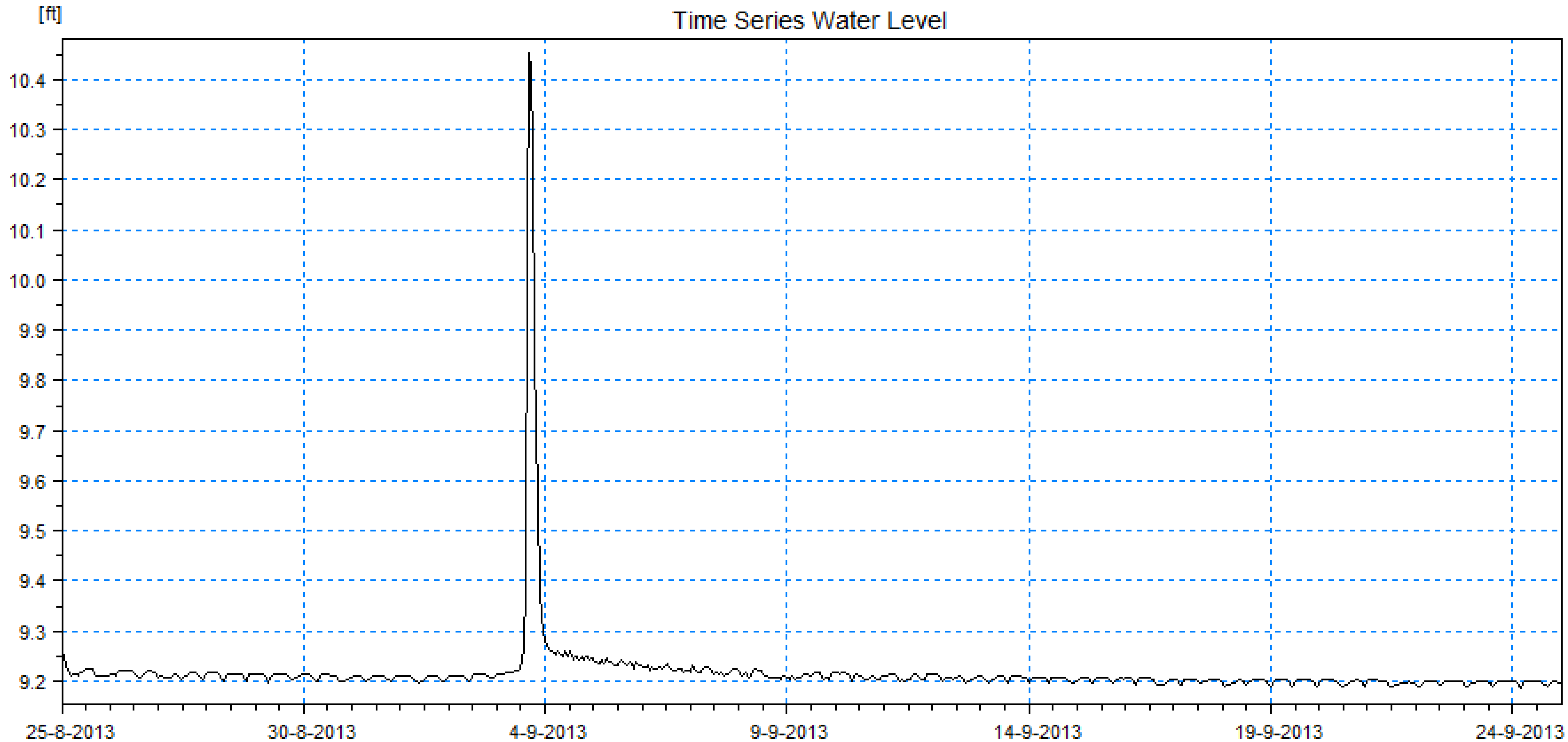

Figure 14. FU4 Headwater Stage (ft)

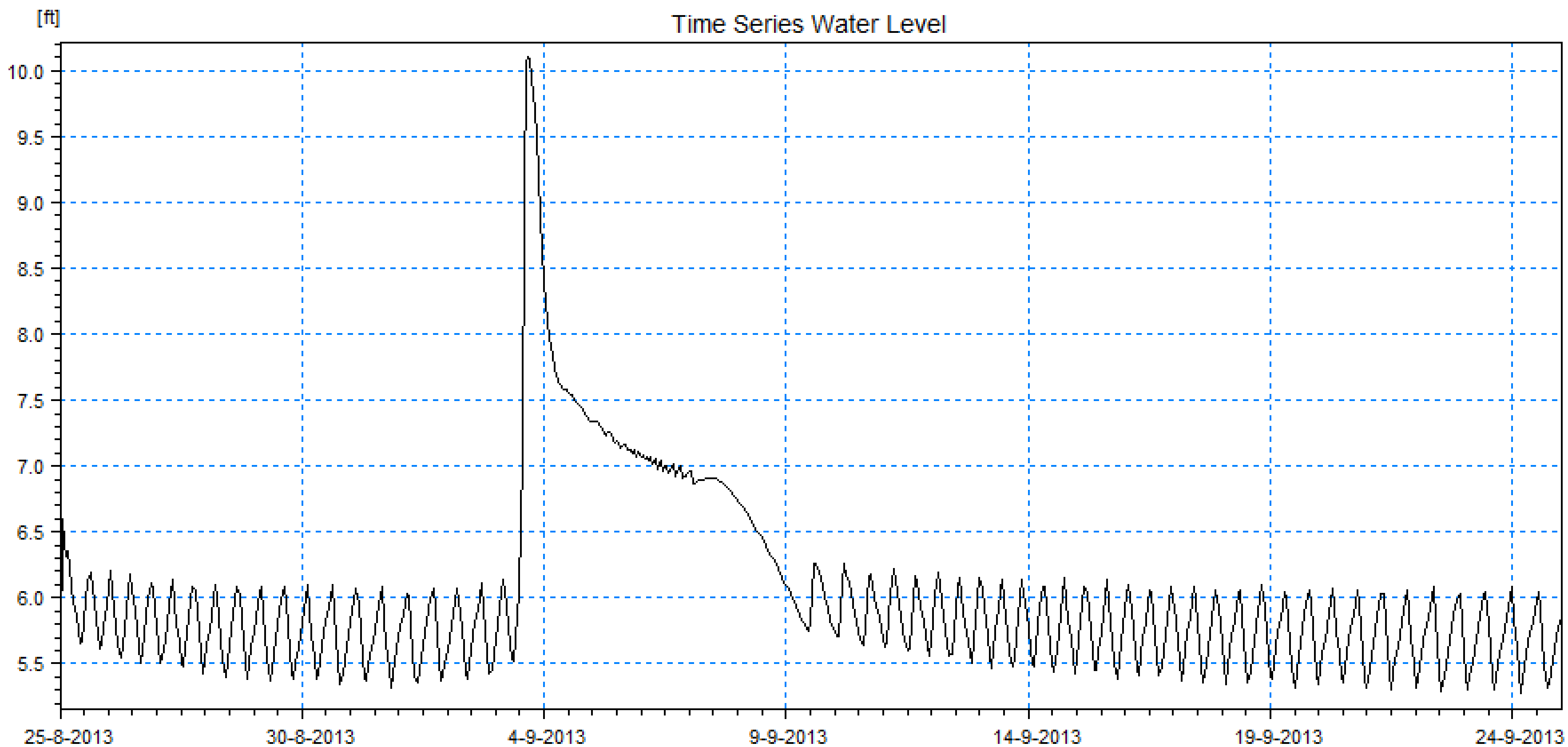

Figure 15. FU4 Tailwater Stage (ft)

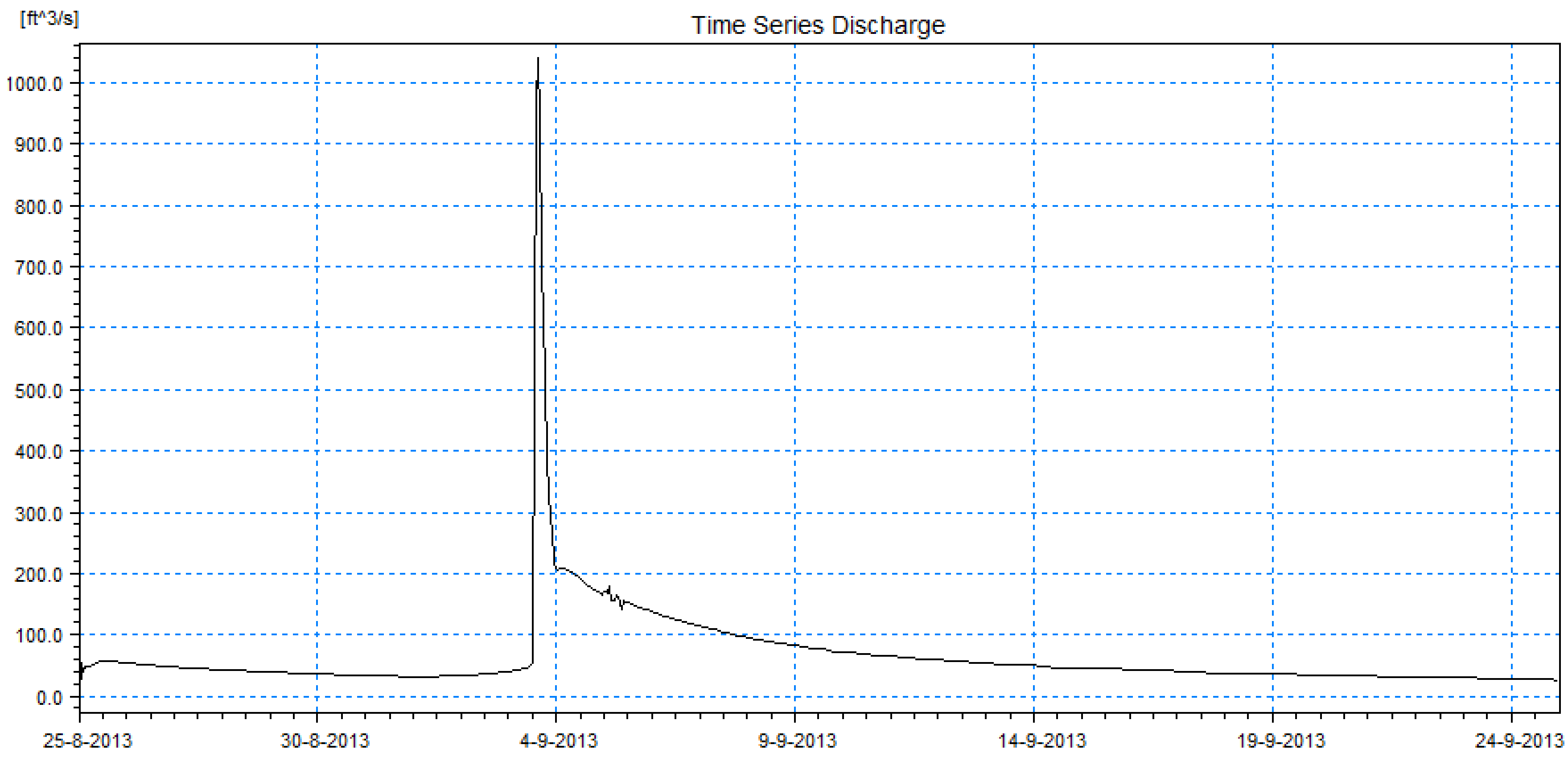

Figure 16. FU5 Discharge (cfs)

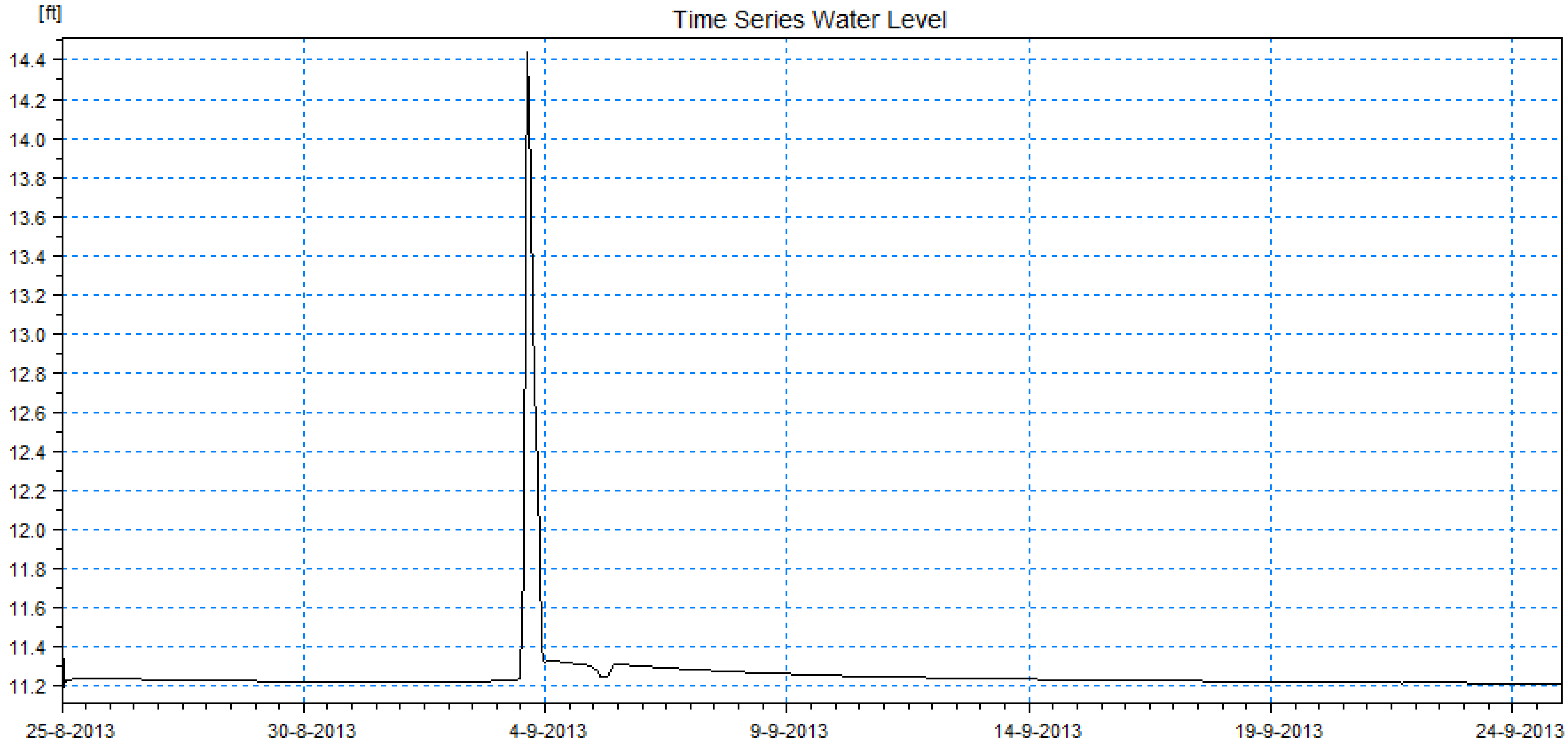

Figure 17. FU5 Headwater Stage (ft)

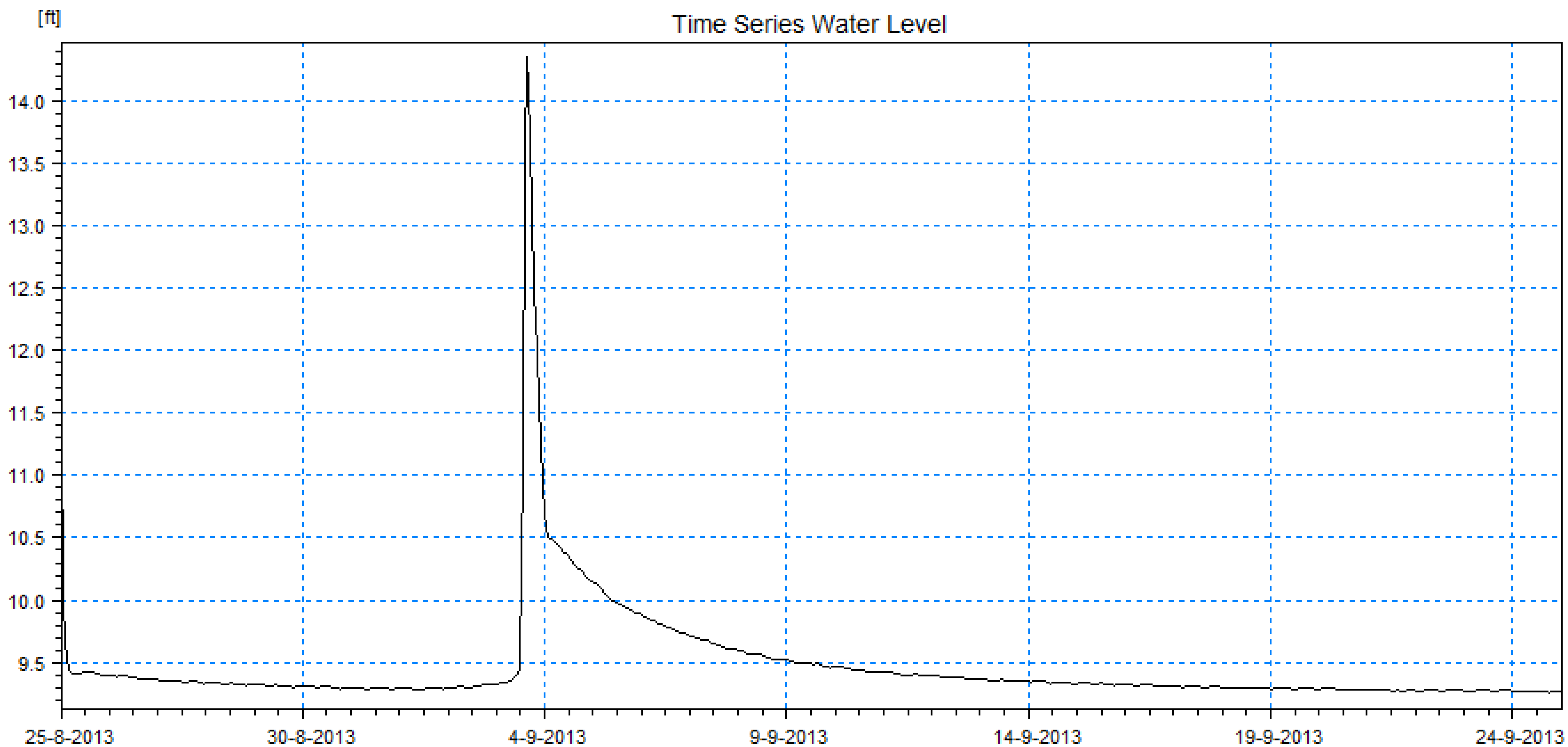

Figure 18. FU5 Tailwater Stage (ft)

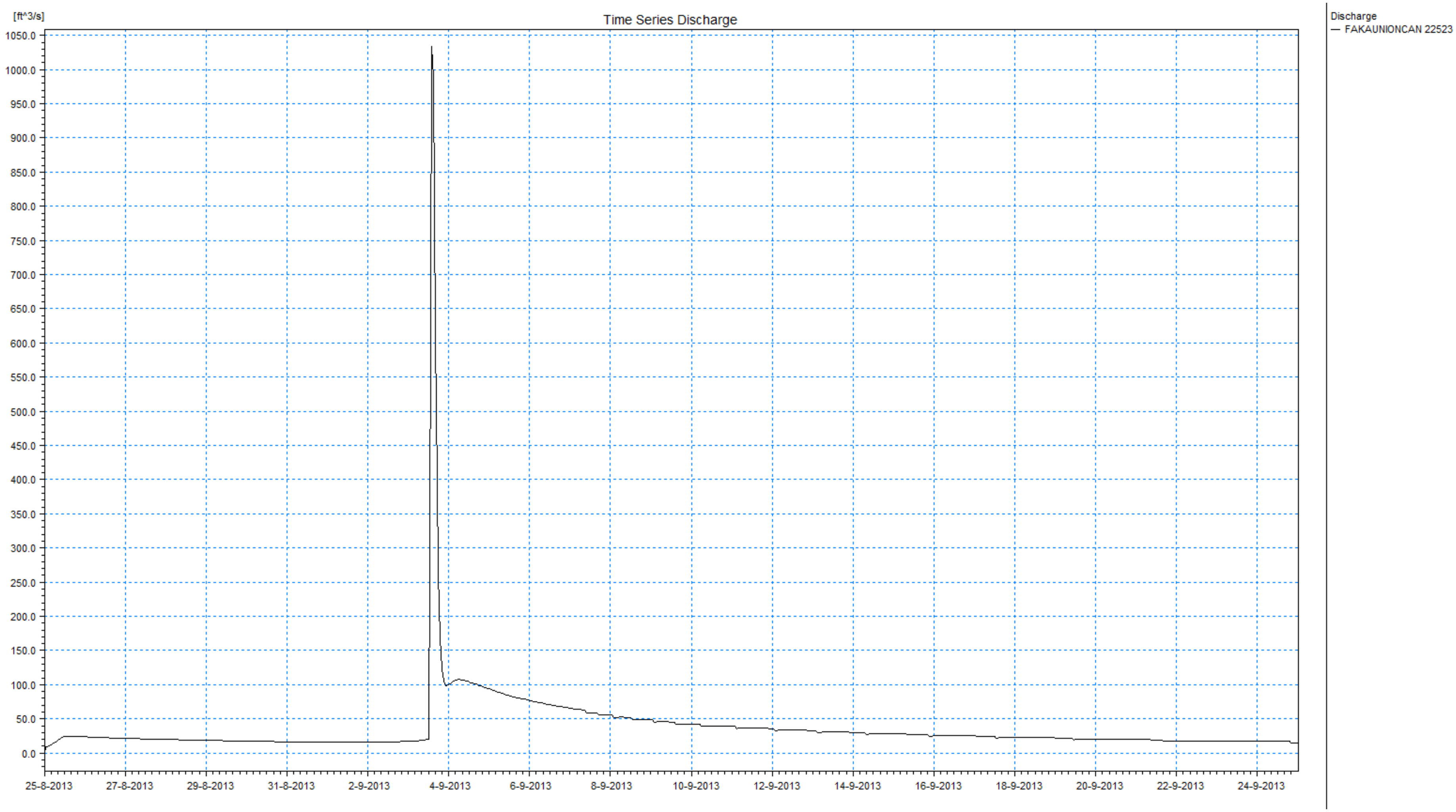

Figure 19. FU6 Discharge (cfs)

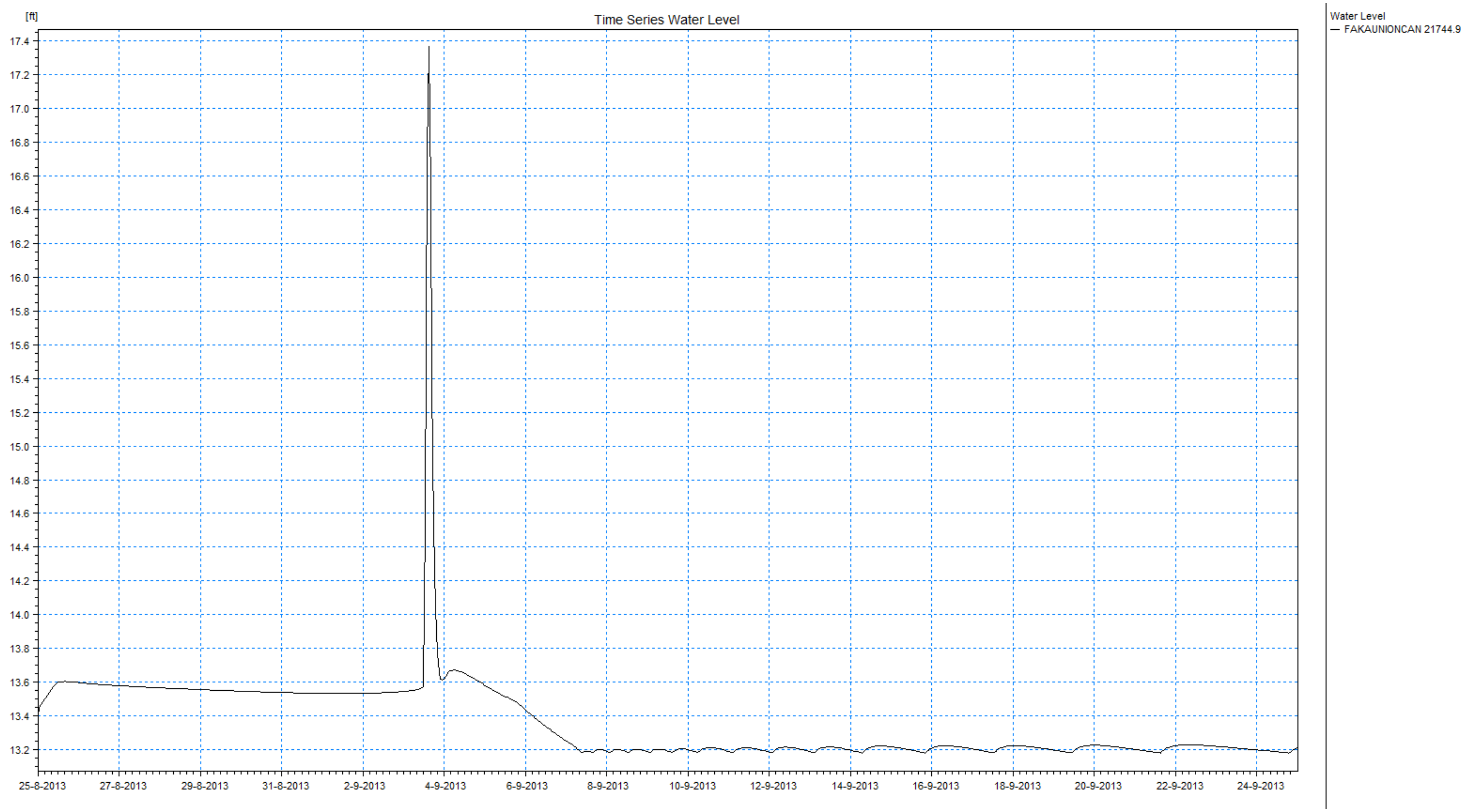

Figure 20. FU6 Headwater Stage (ft)

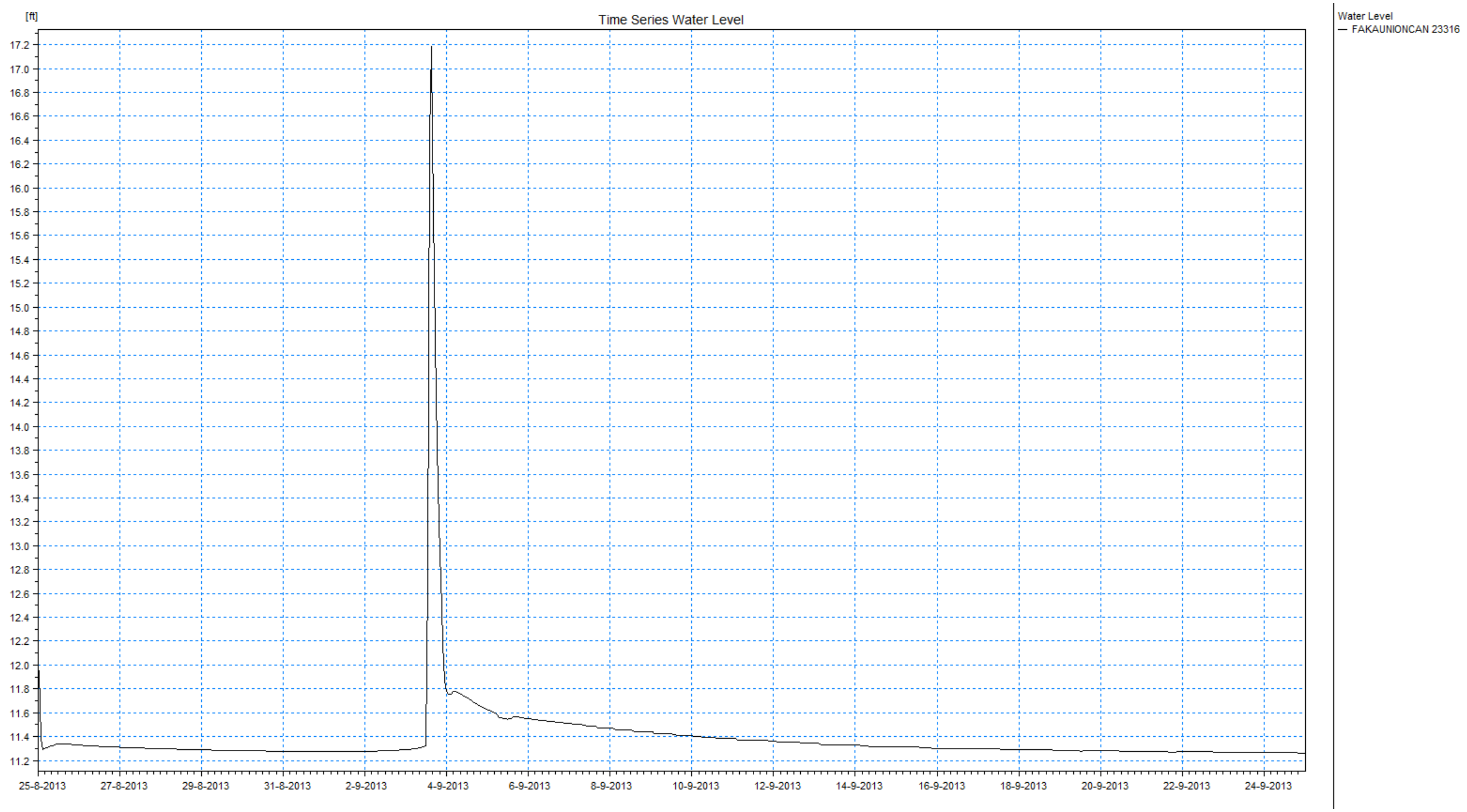

Figure 21. FU6 Tailwater Stage (ft)

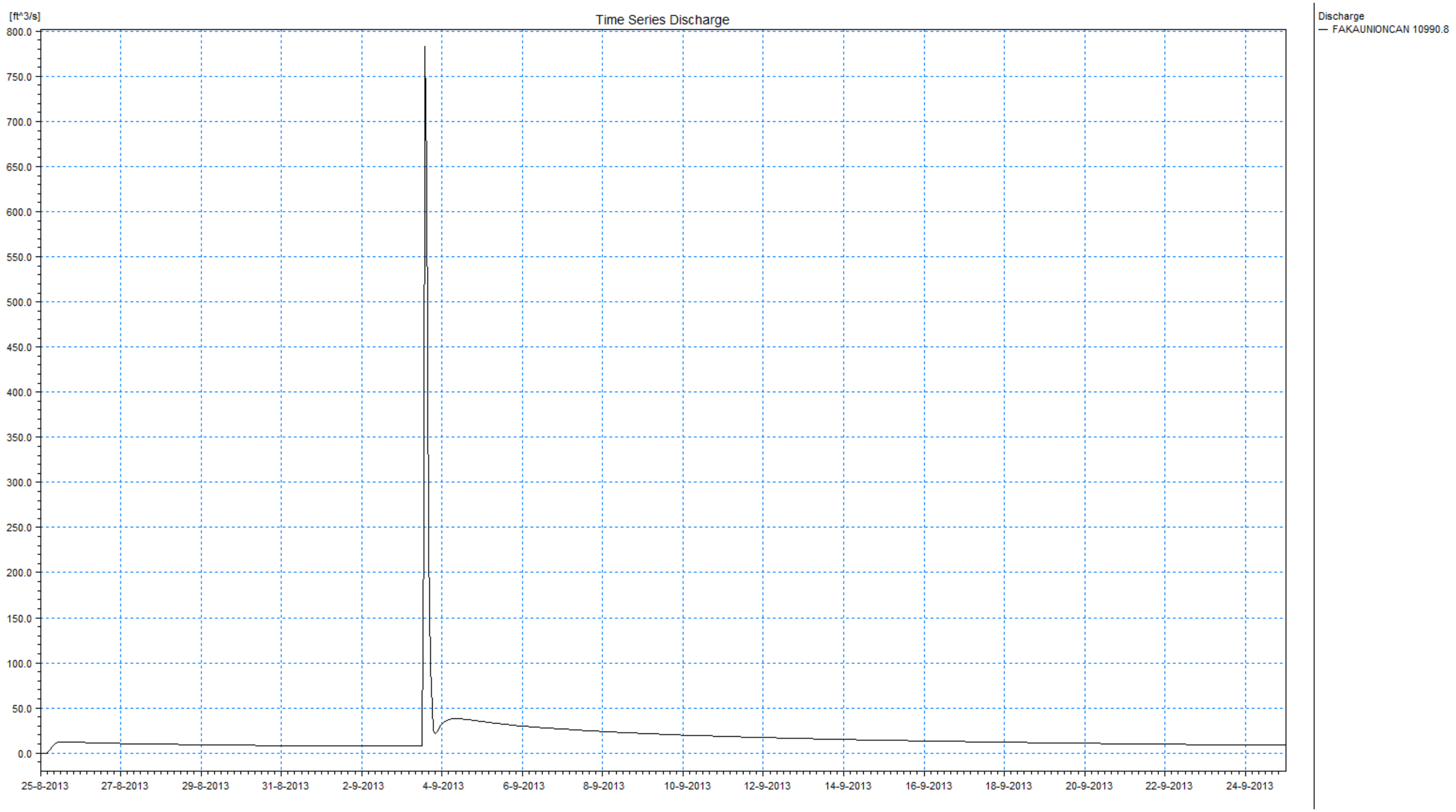

Figure 22. FU7 Discharge (cfs)

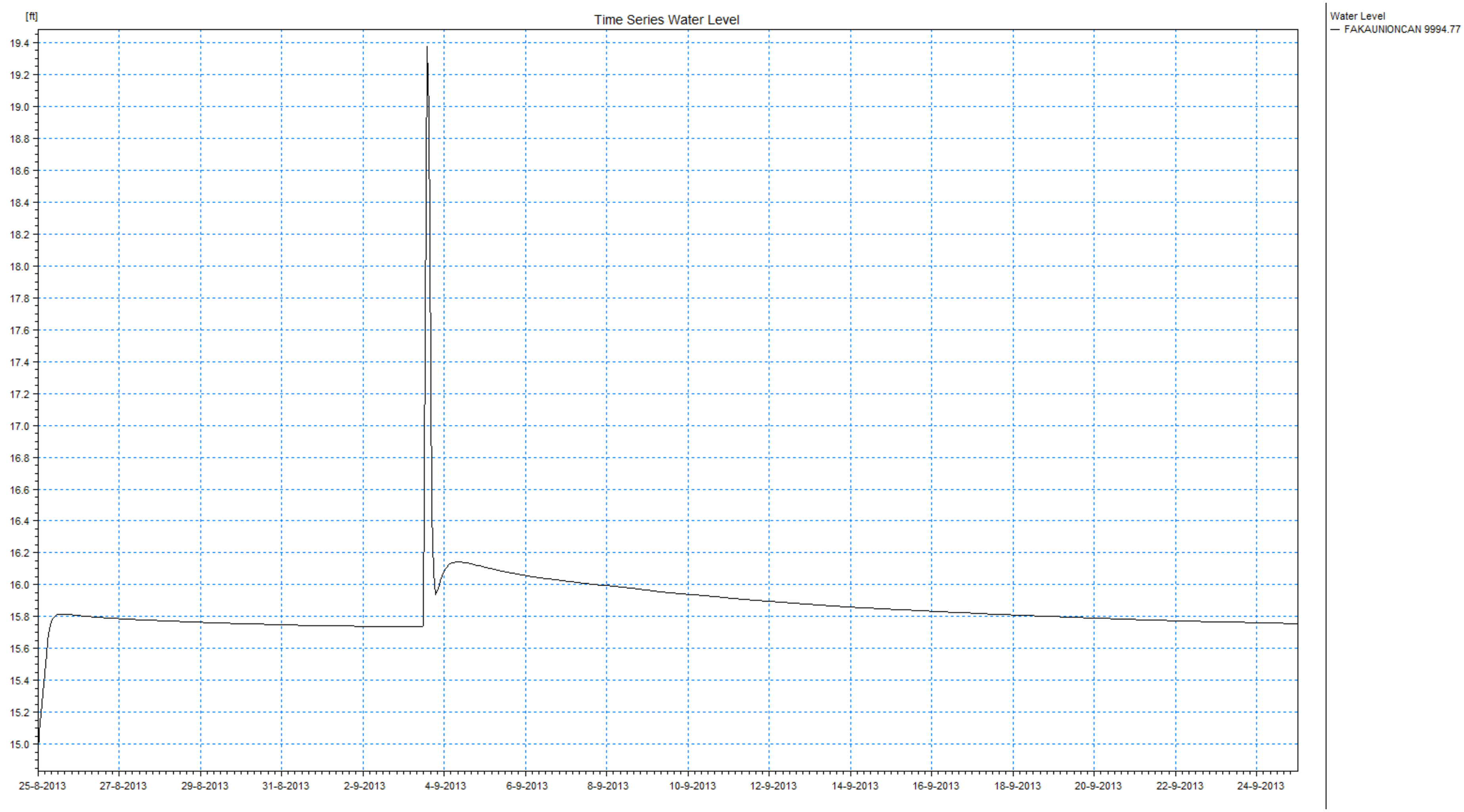

Figure 23. FU7 Headwater Stage (ft)

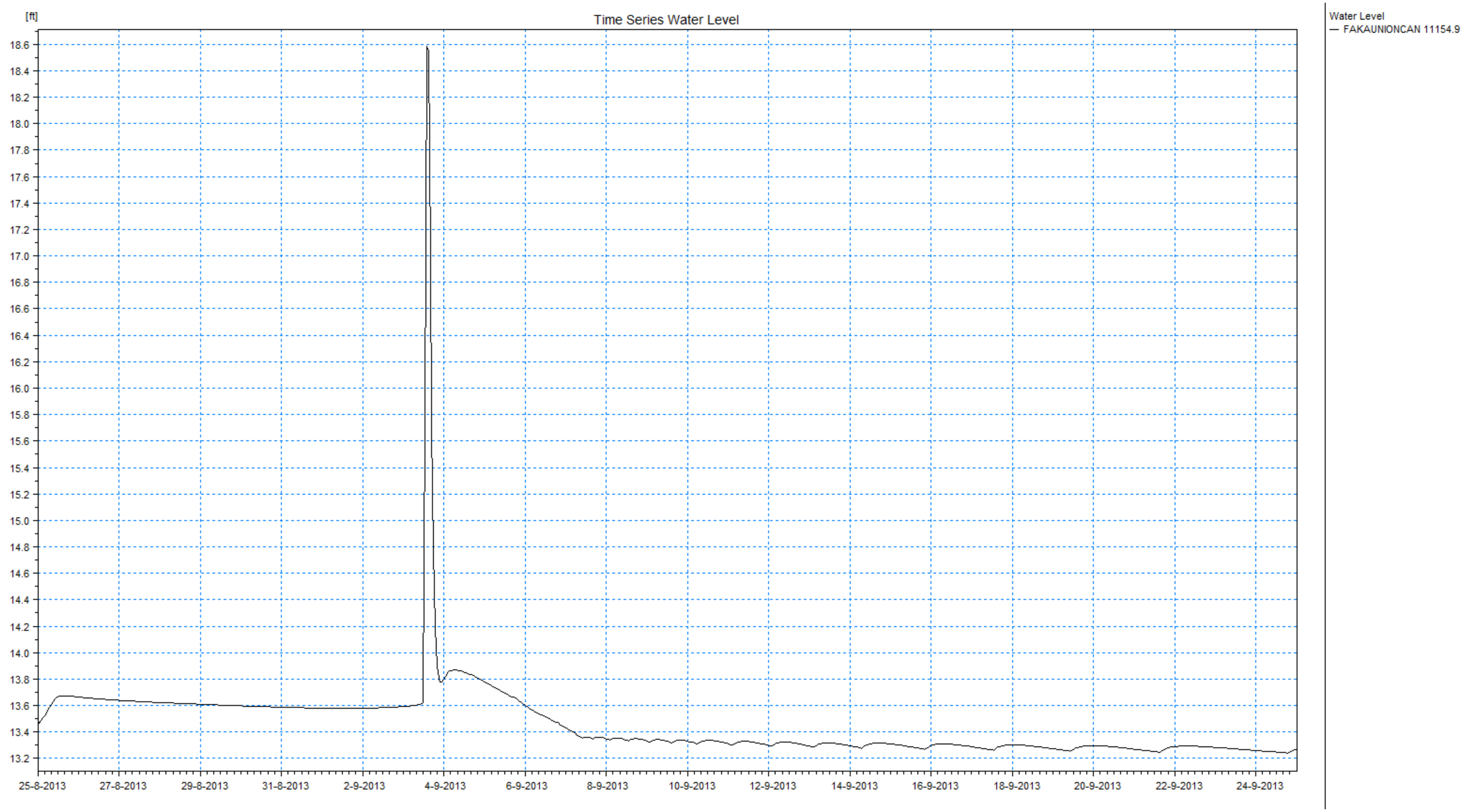

Figure 24. FU7 Tailwater Stage (ft)

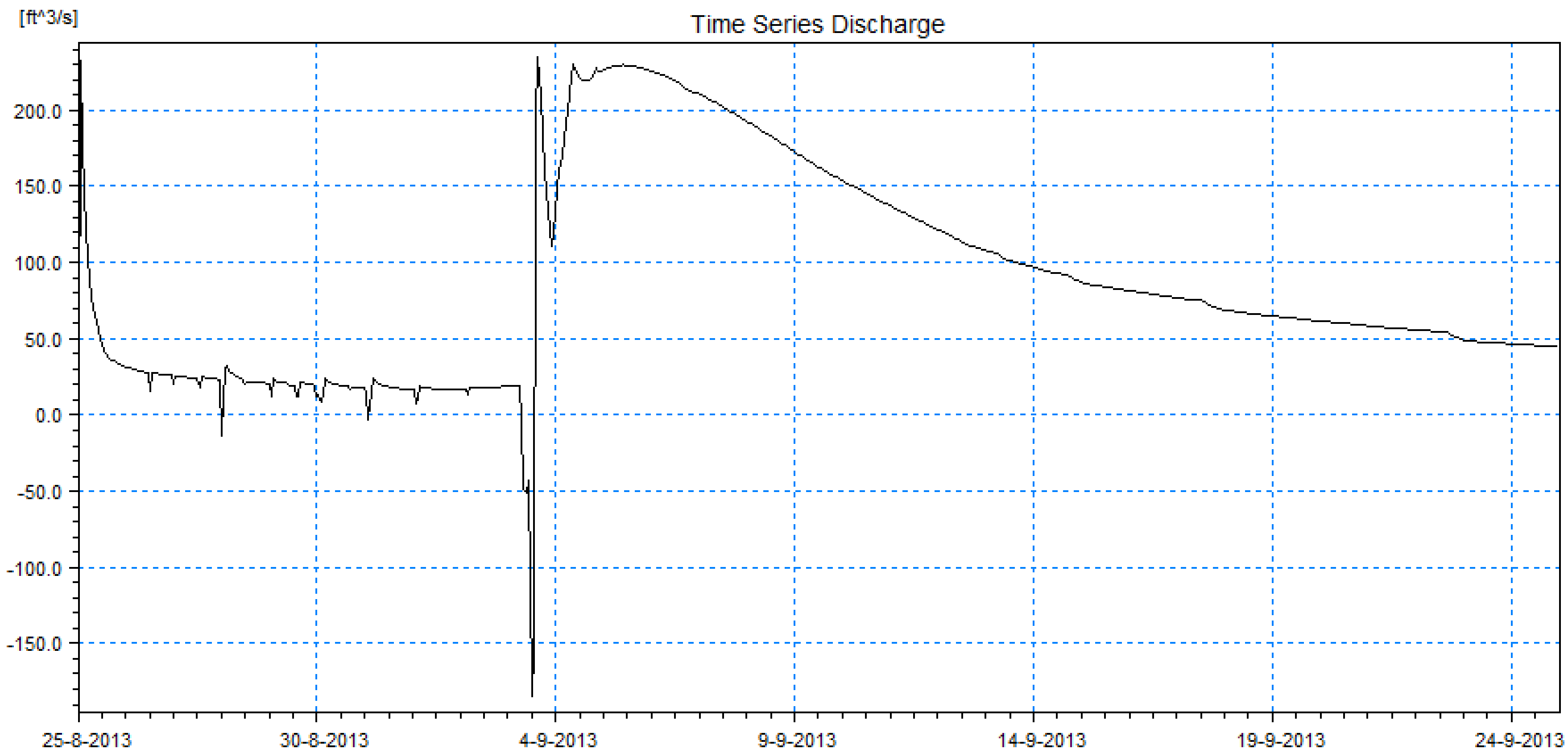

Figure 25. HC1 Discharge (cfs)

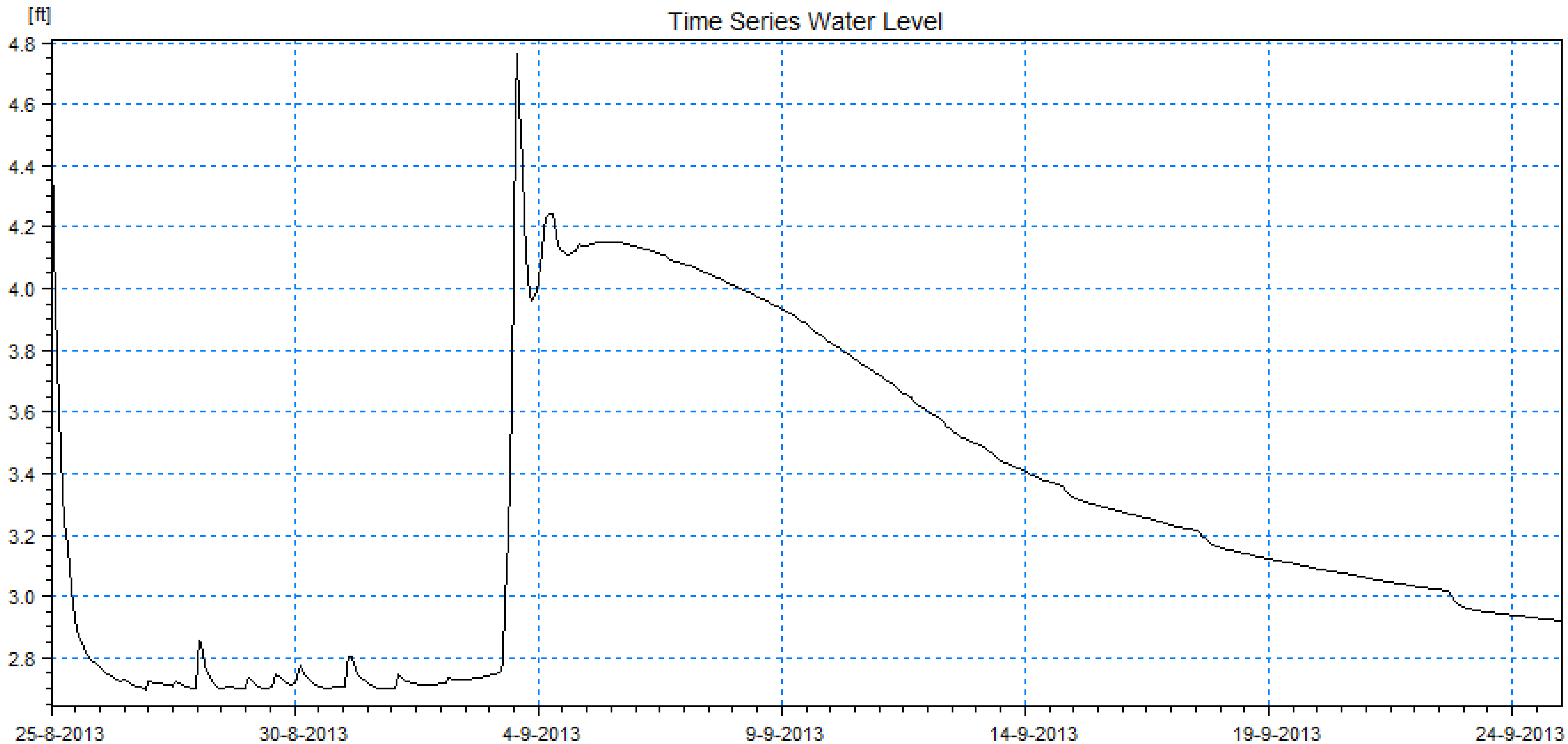

Figure 26. HC1 Headwater Stage (ft)

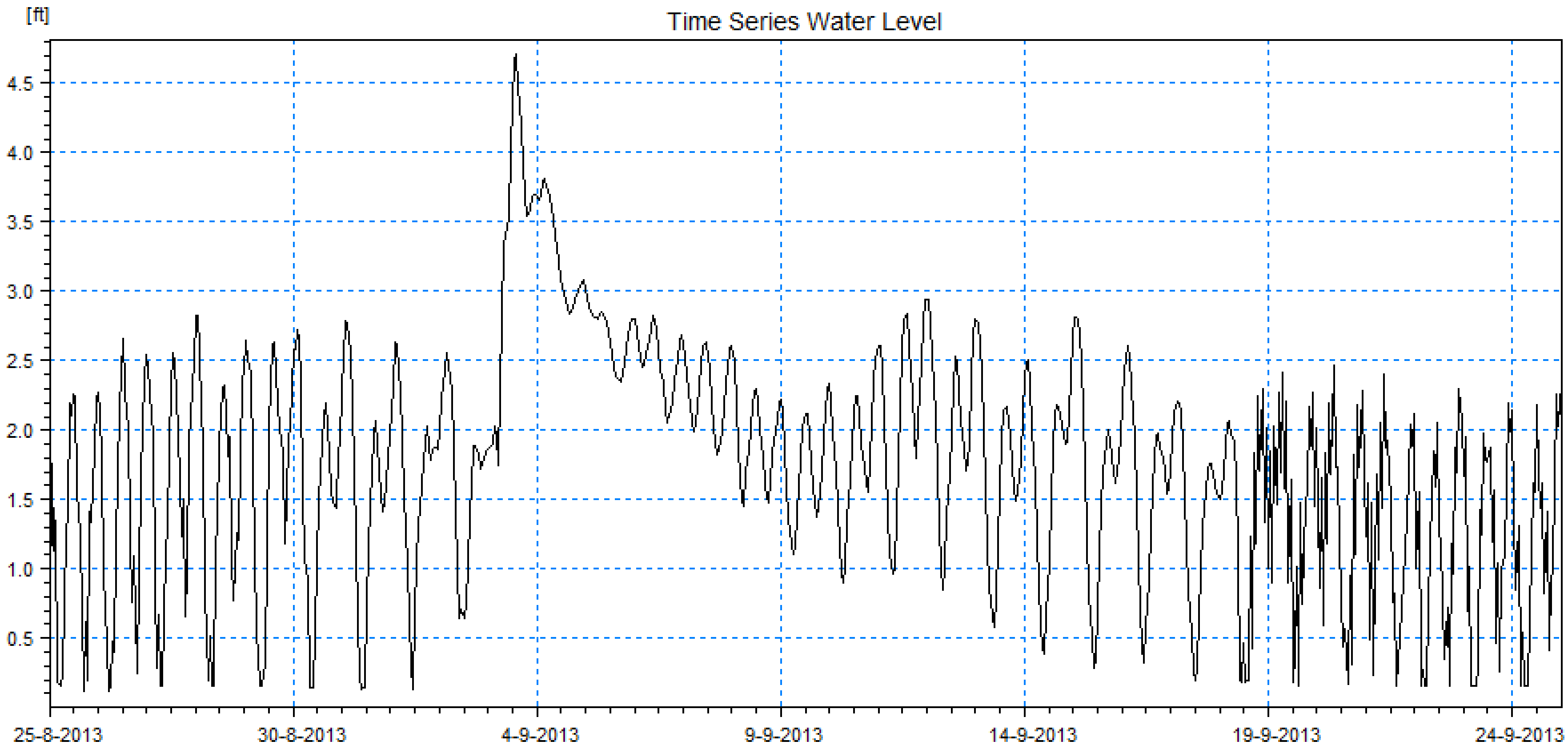

Figure 27. HC1 Tailwater Stage (ft)

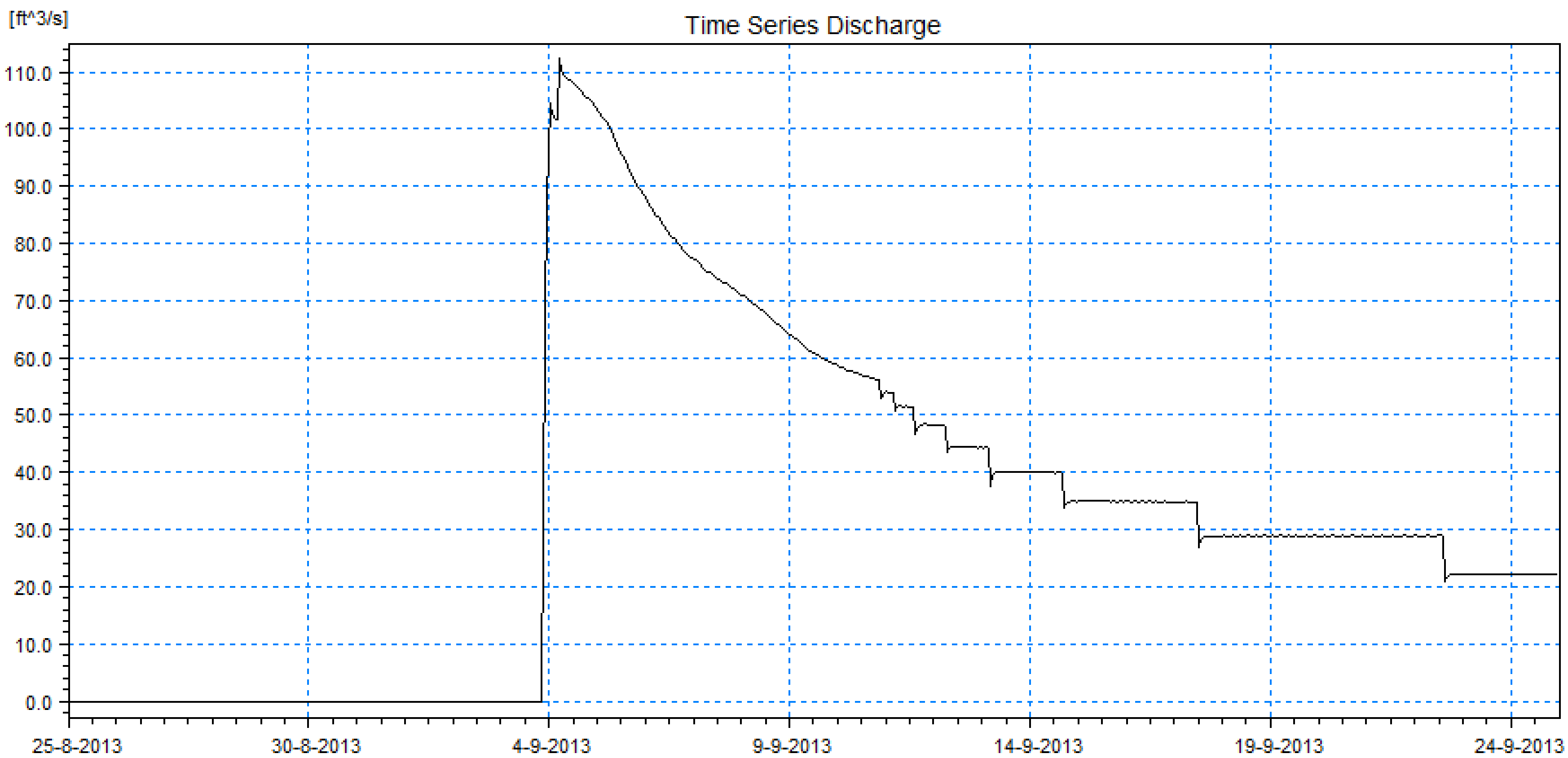

Figure 28. HC2 Discharge (cfs)

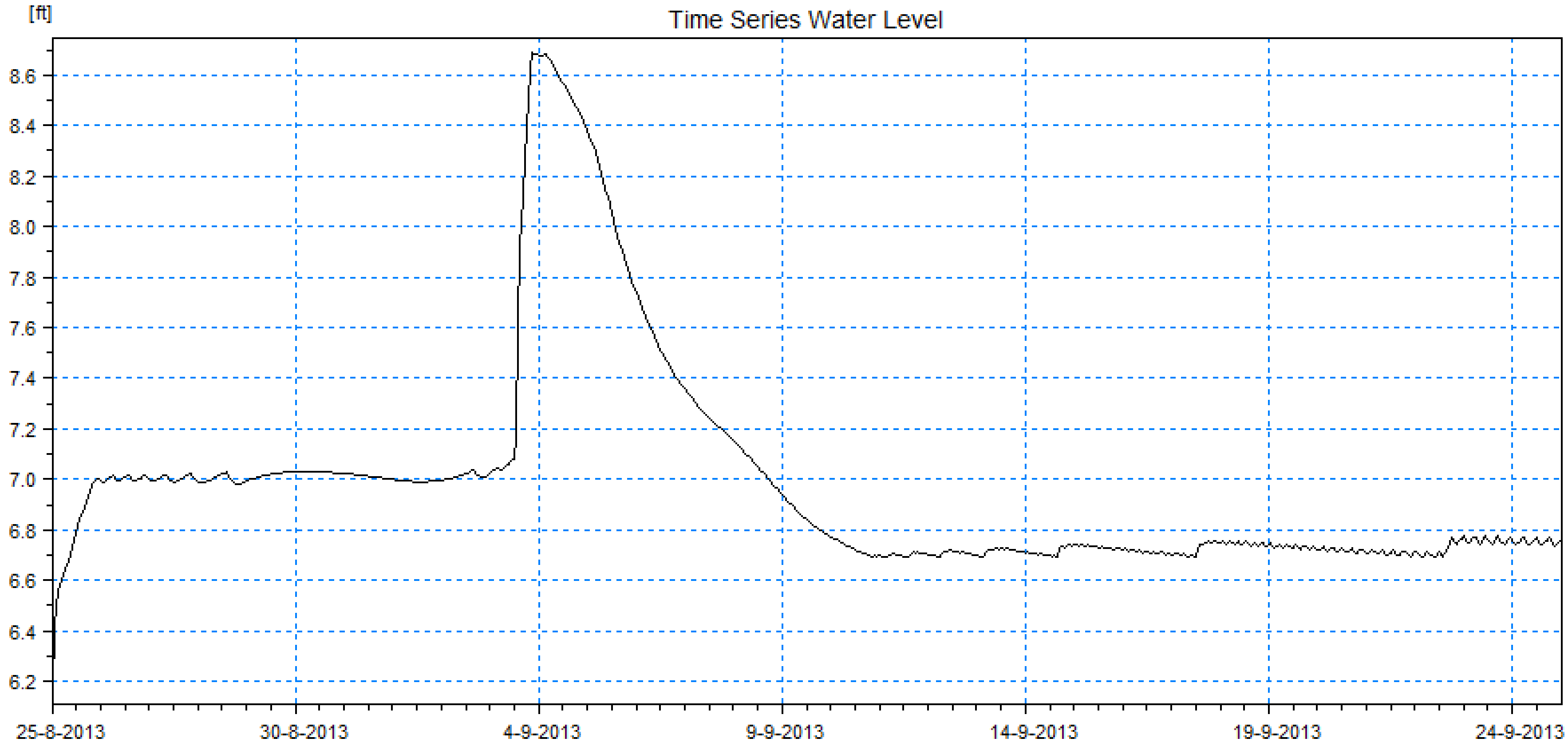

Figure 29. HC2 Headwater Stage (ft)

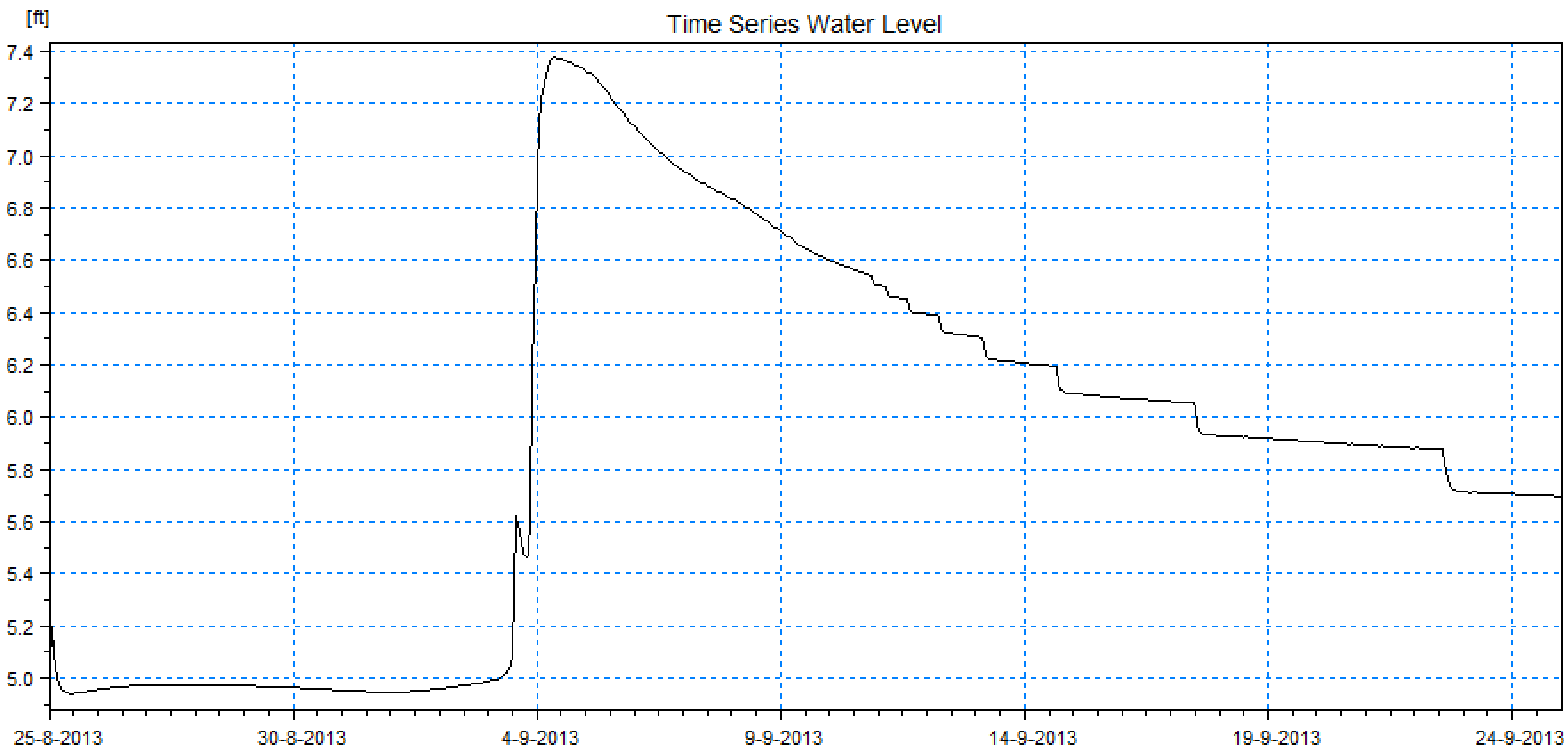

Figure 30. HC2 Tailwater Stage (ft)

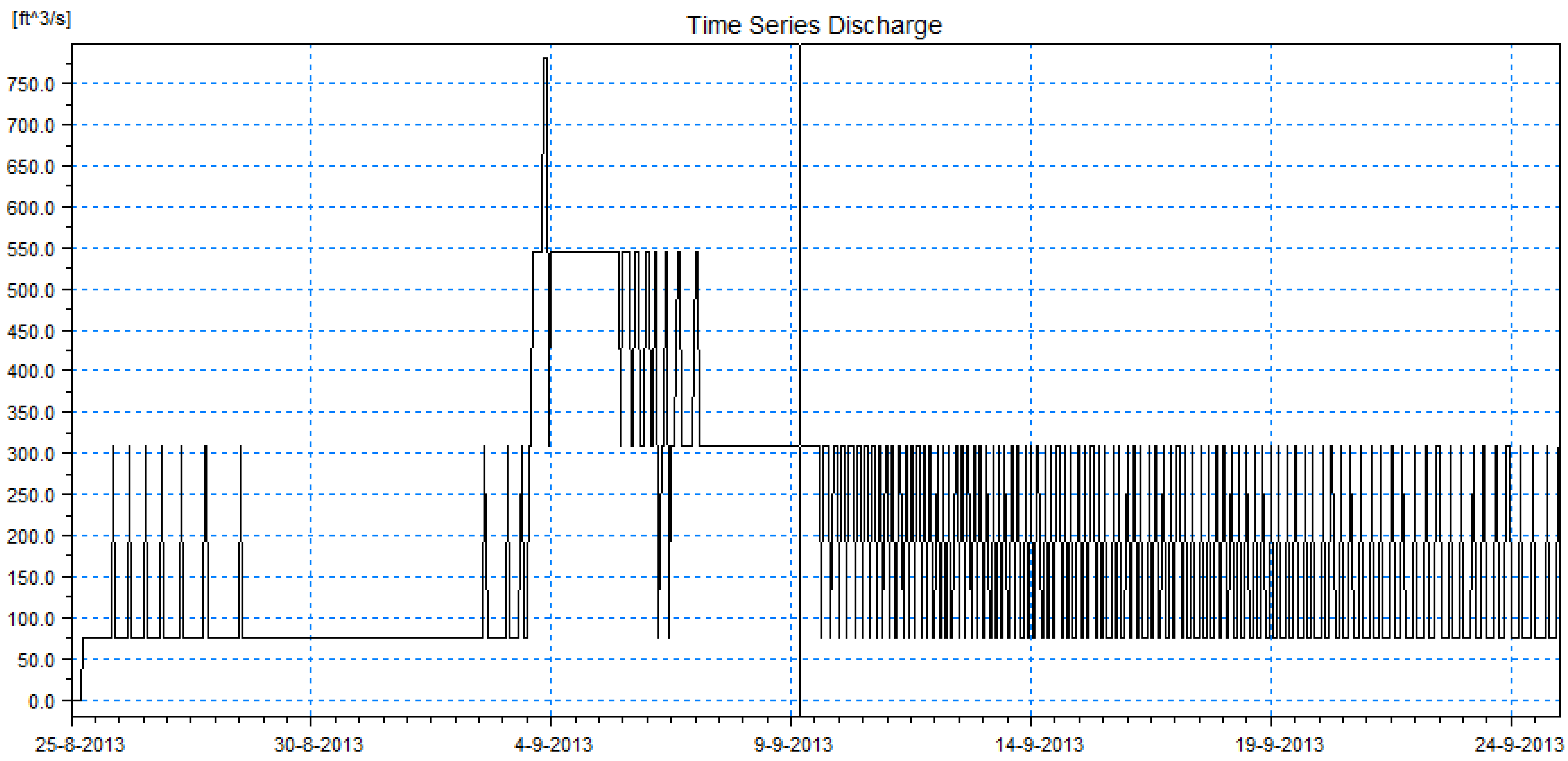

Figure 31: Miller pump Discharge (cfs)

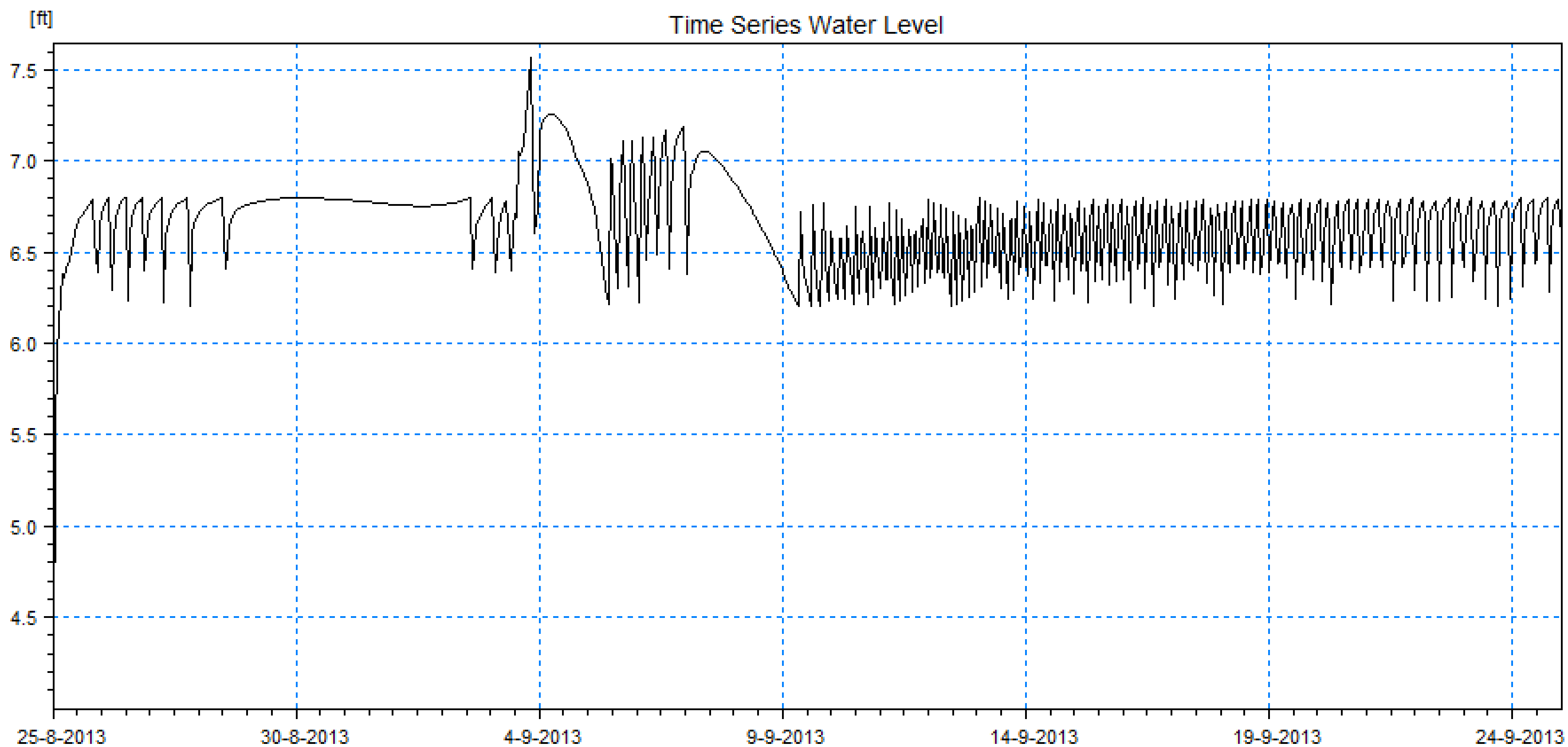

Figure 32- Miller Pump Headwater Stage (ft)

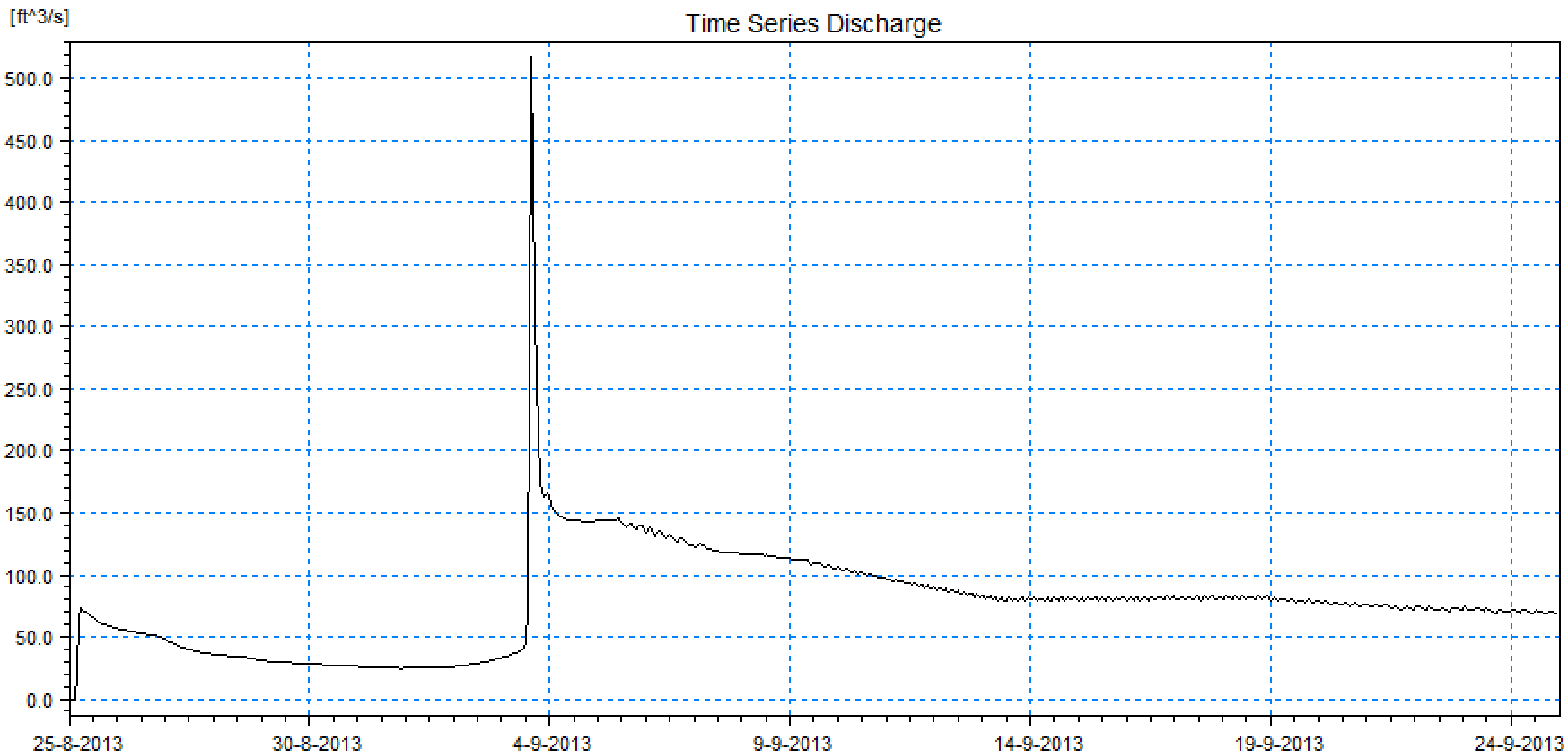

Figure 33. Miller3 Discharge (cfs)

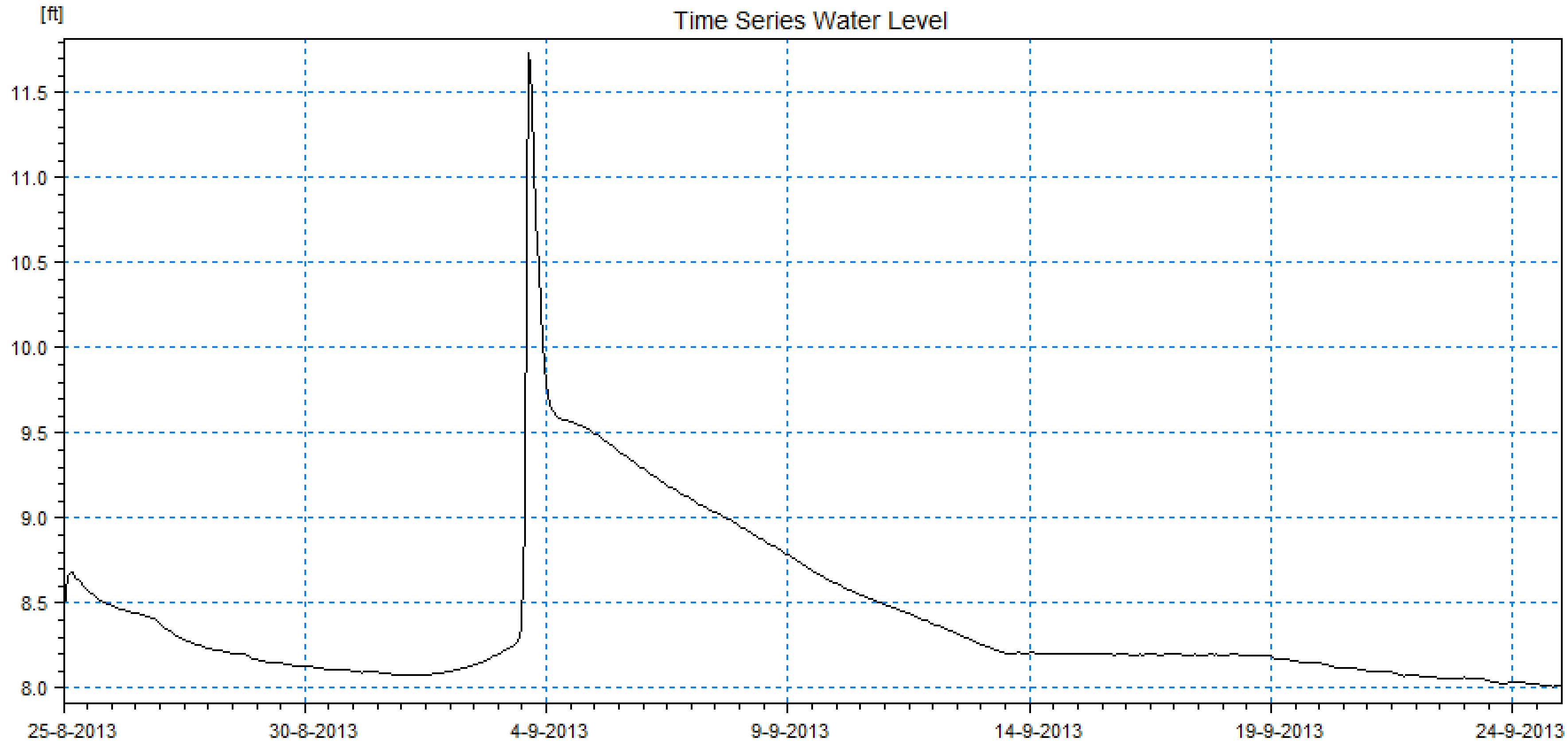

Figure 34. Miller3 Headwater Stage (ft)

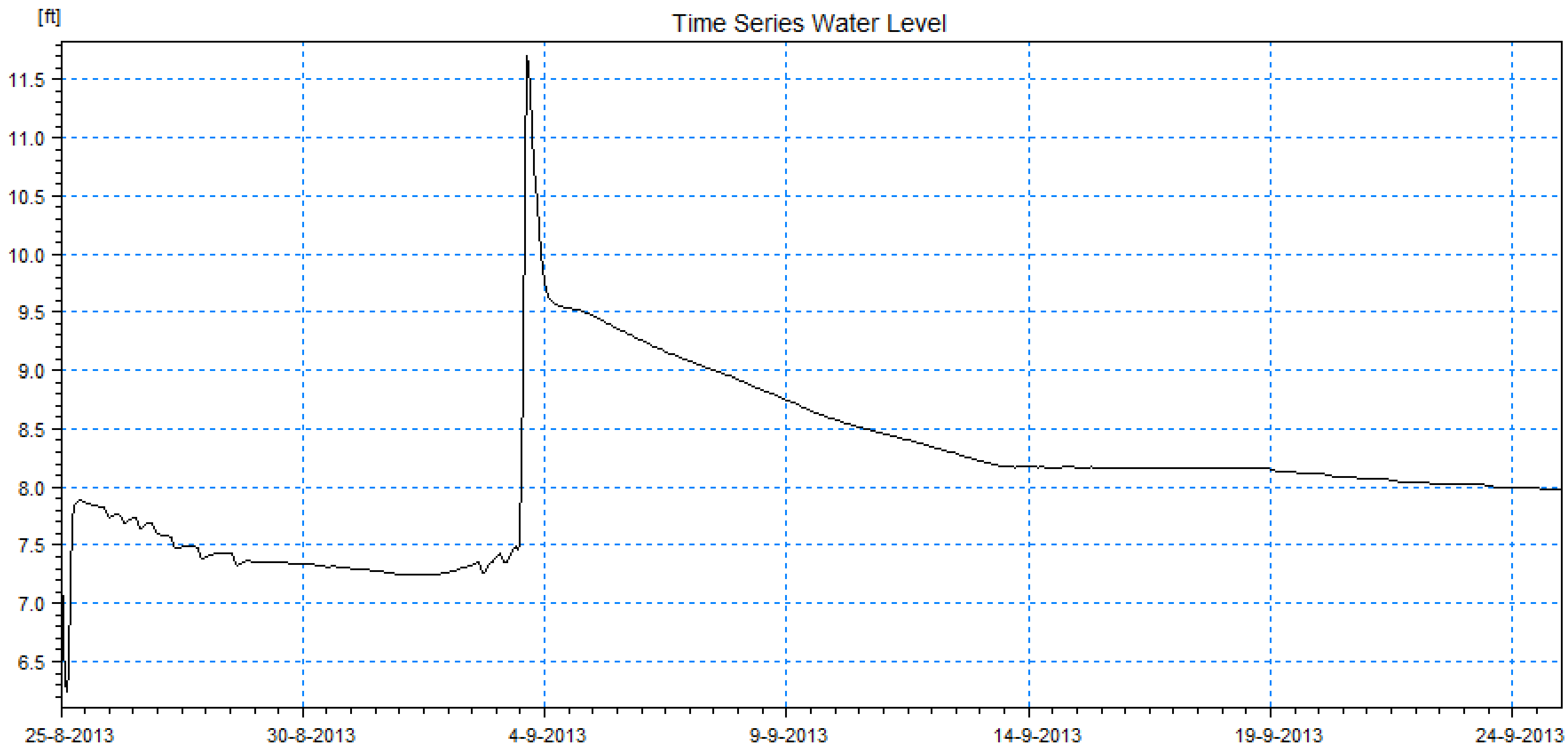

Figure 35. Miller3 Tailwater Stage (ft)

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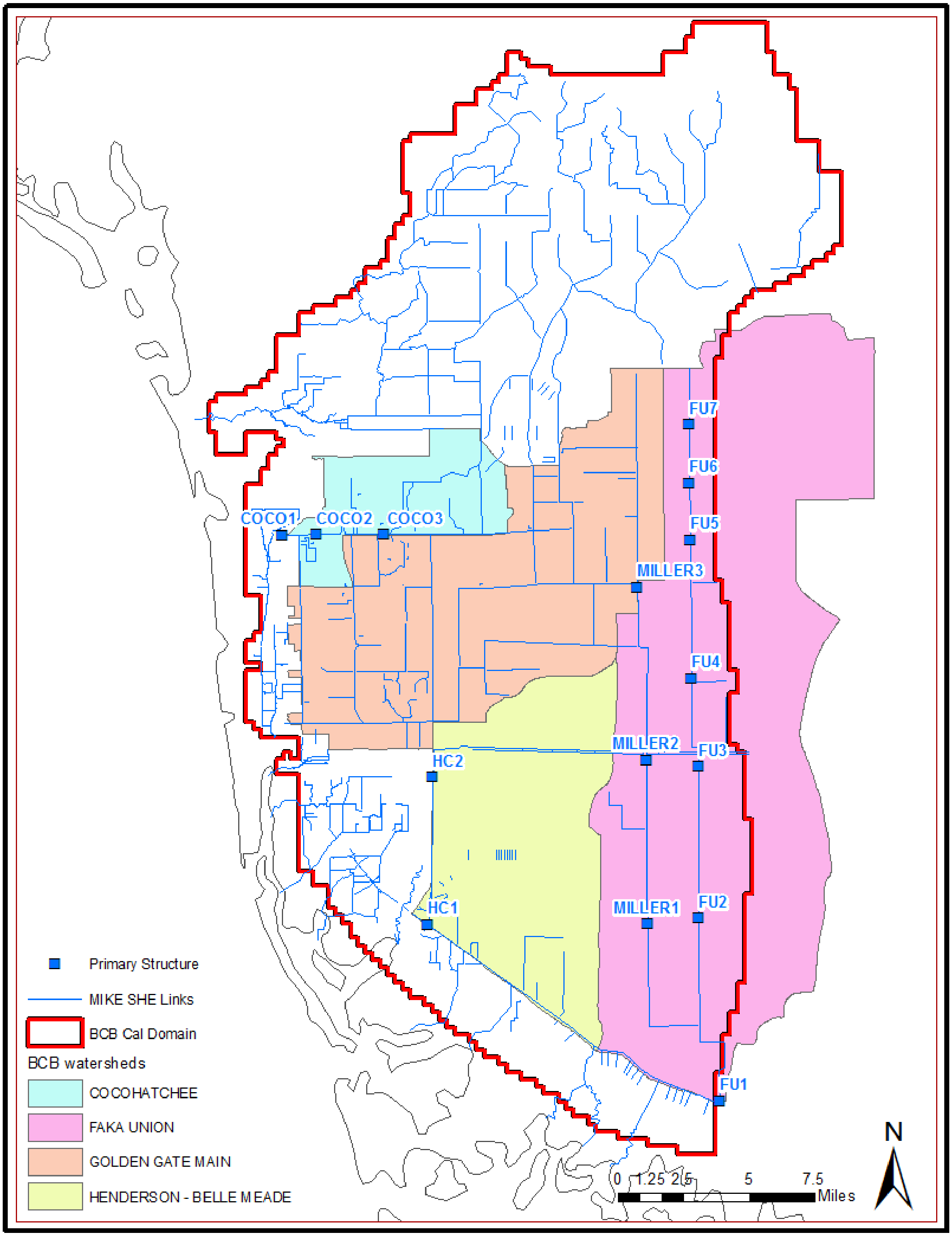

Figure 1 Mike SHE Stations within COCO, FAKA and HEND Watersheds

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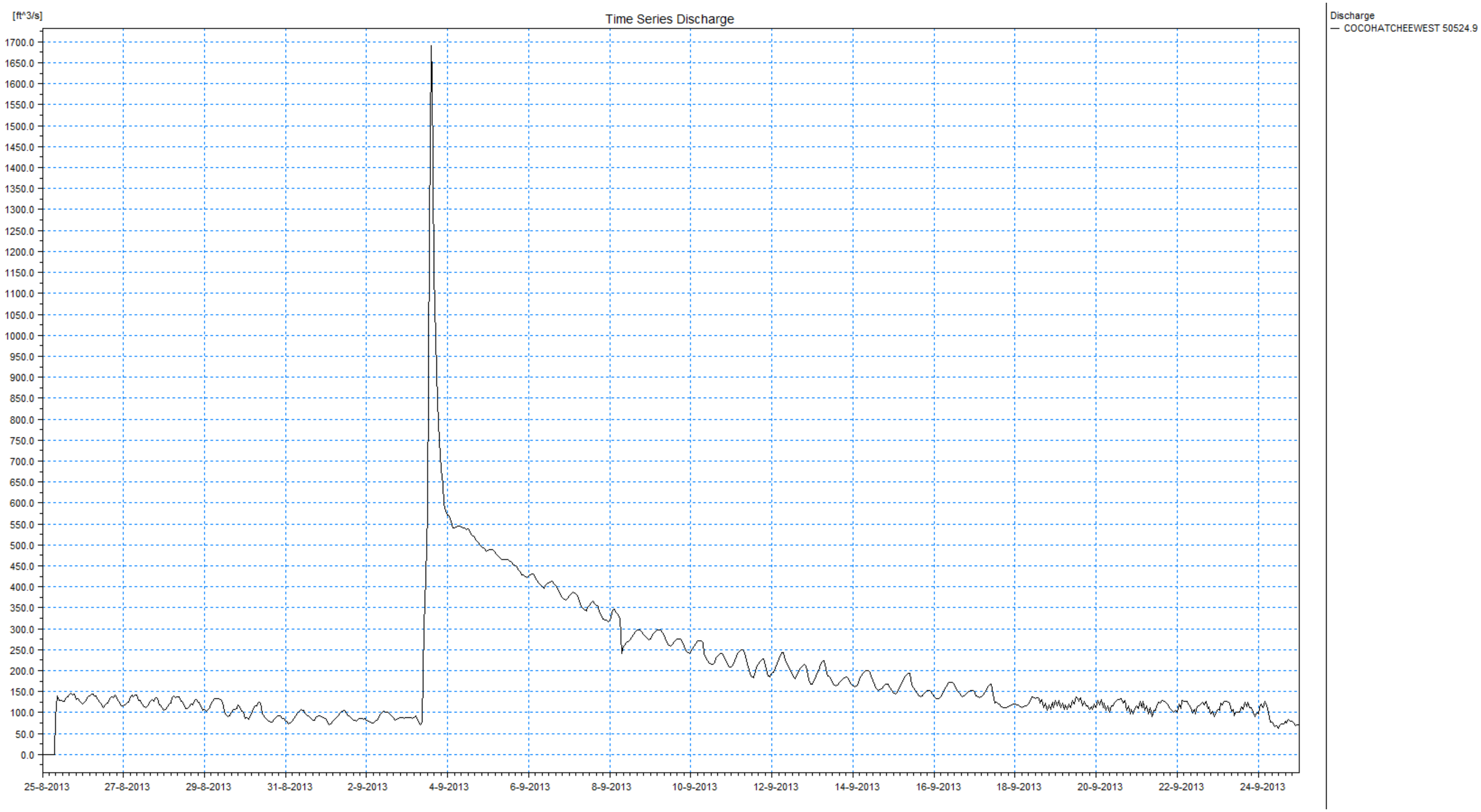

Figure 2. COCO1 Discharge (cfs)

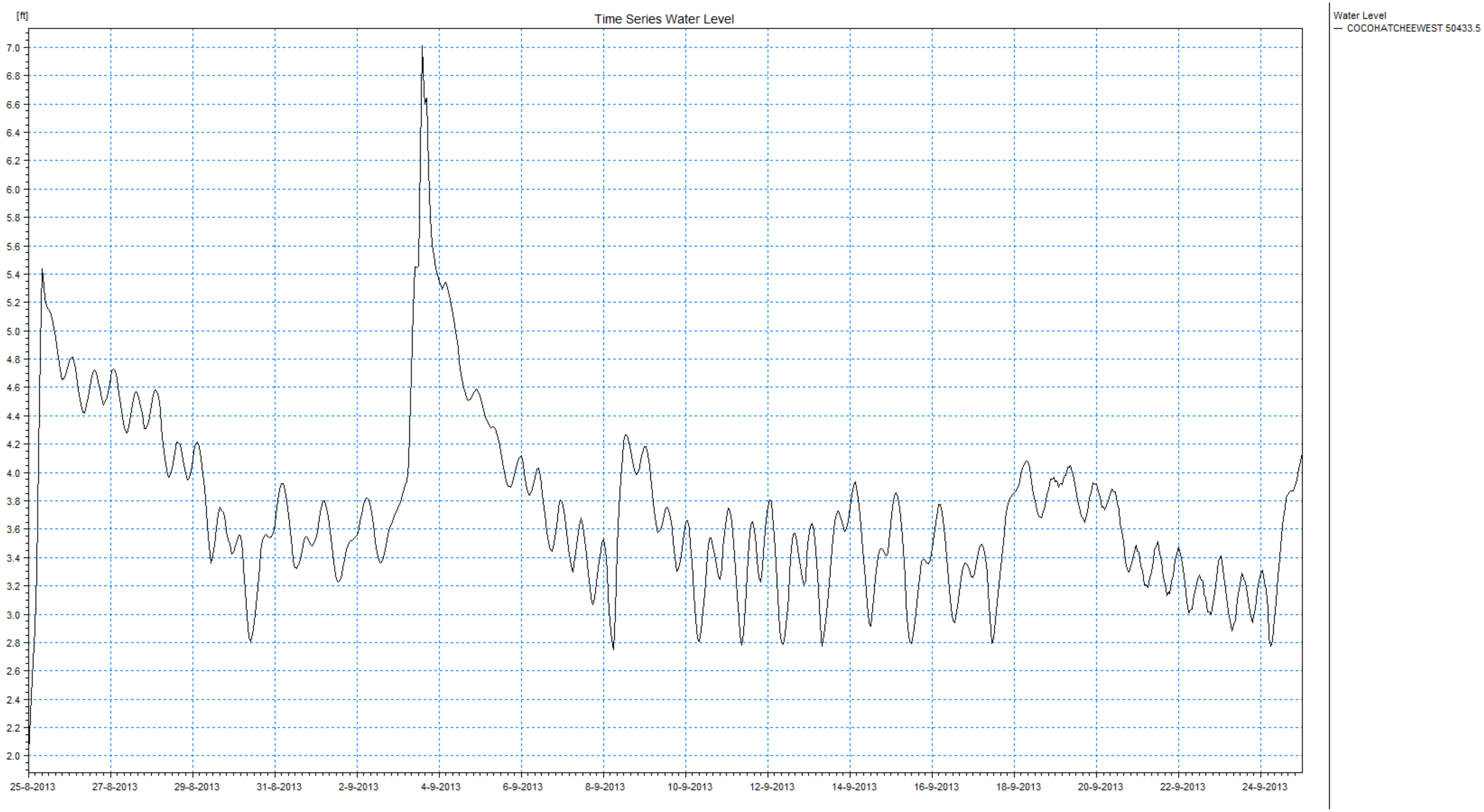

Figure 3. COCO1 Headwater Stage (ft)

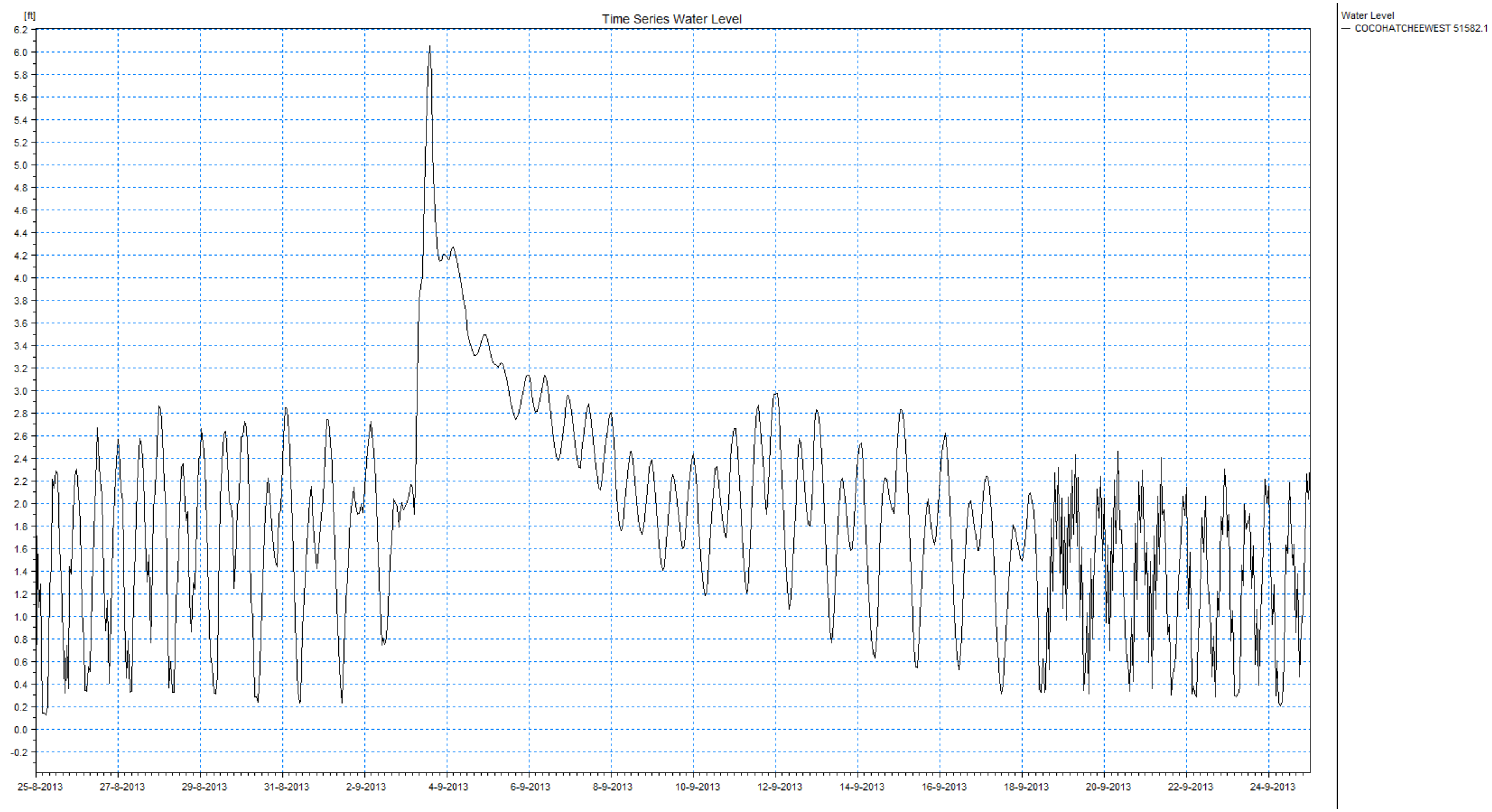

Figure 4. COCO1 Tailwater Stage (ft)

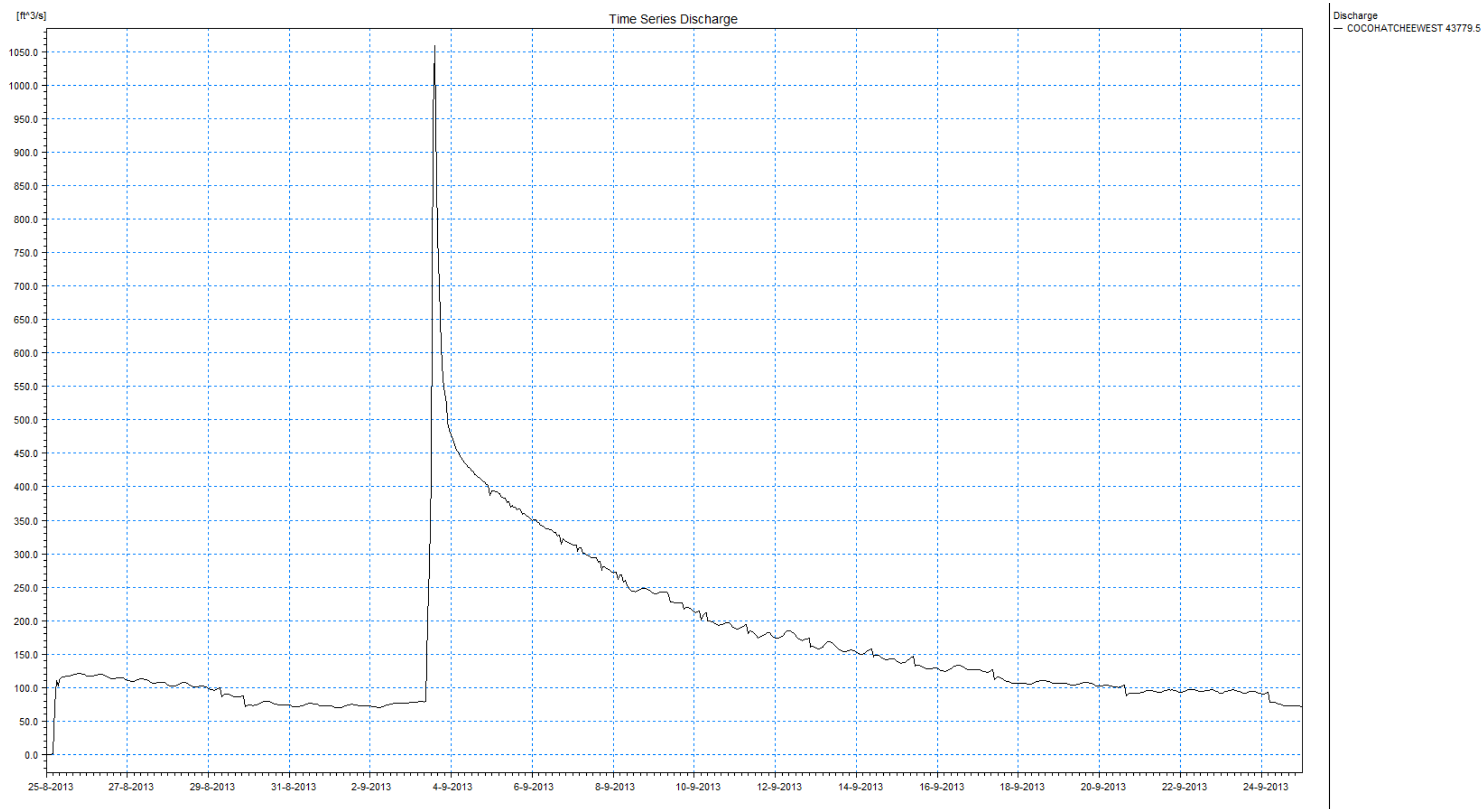

Figure 5. COCO2 Discharge (cfs)

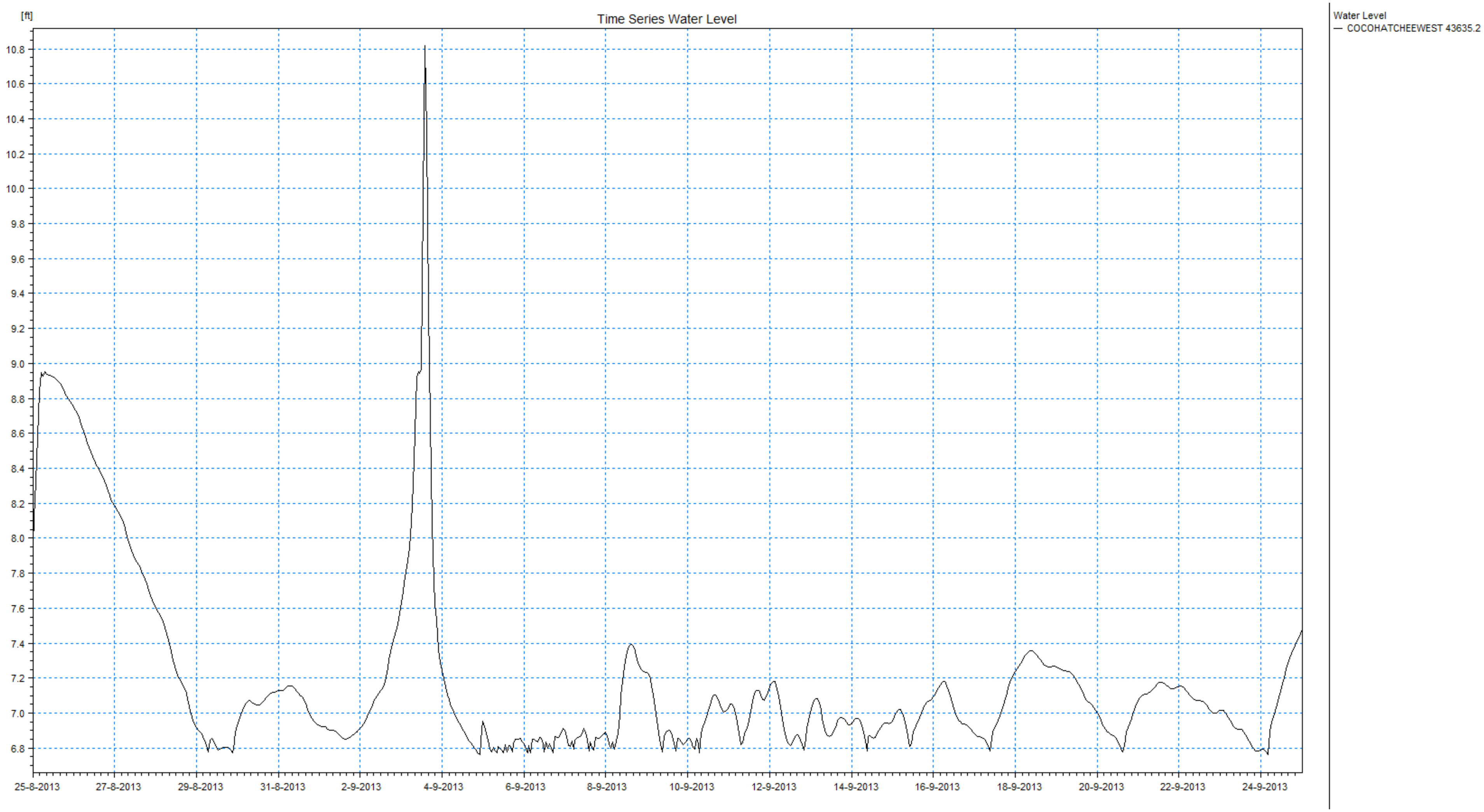

Figure 6. COCO2 Headwater Stage (ft)

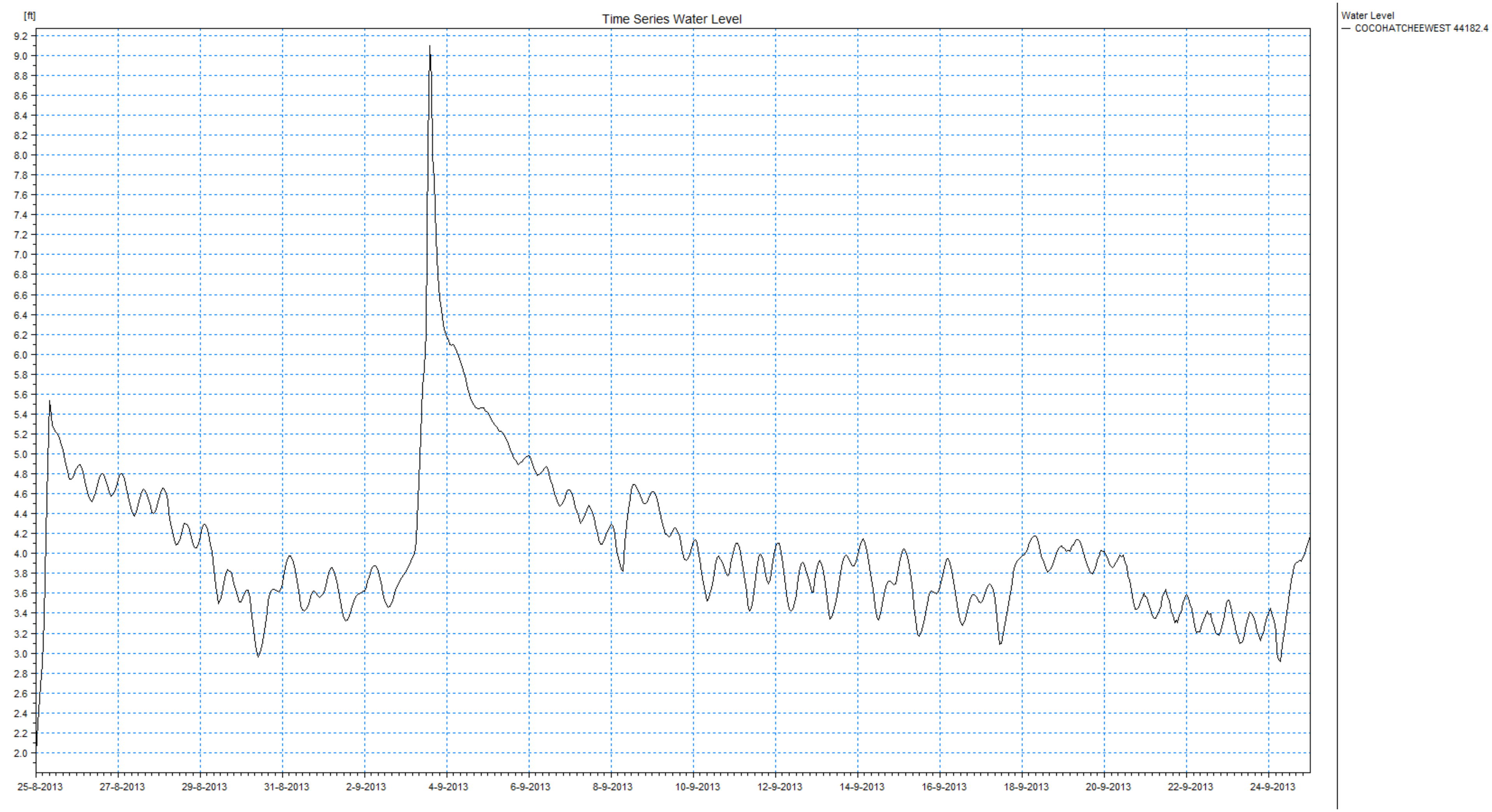

Figure 7. COCO2 Tailwater Stage (ft)

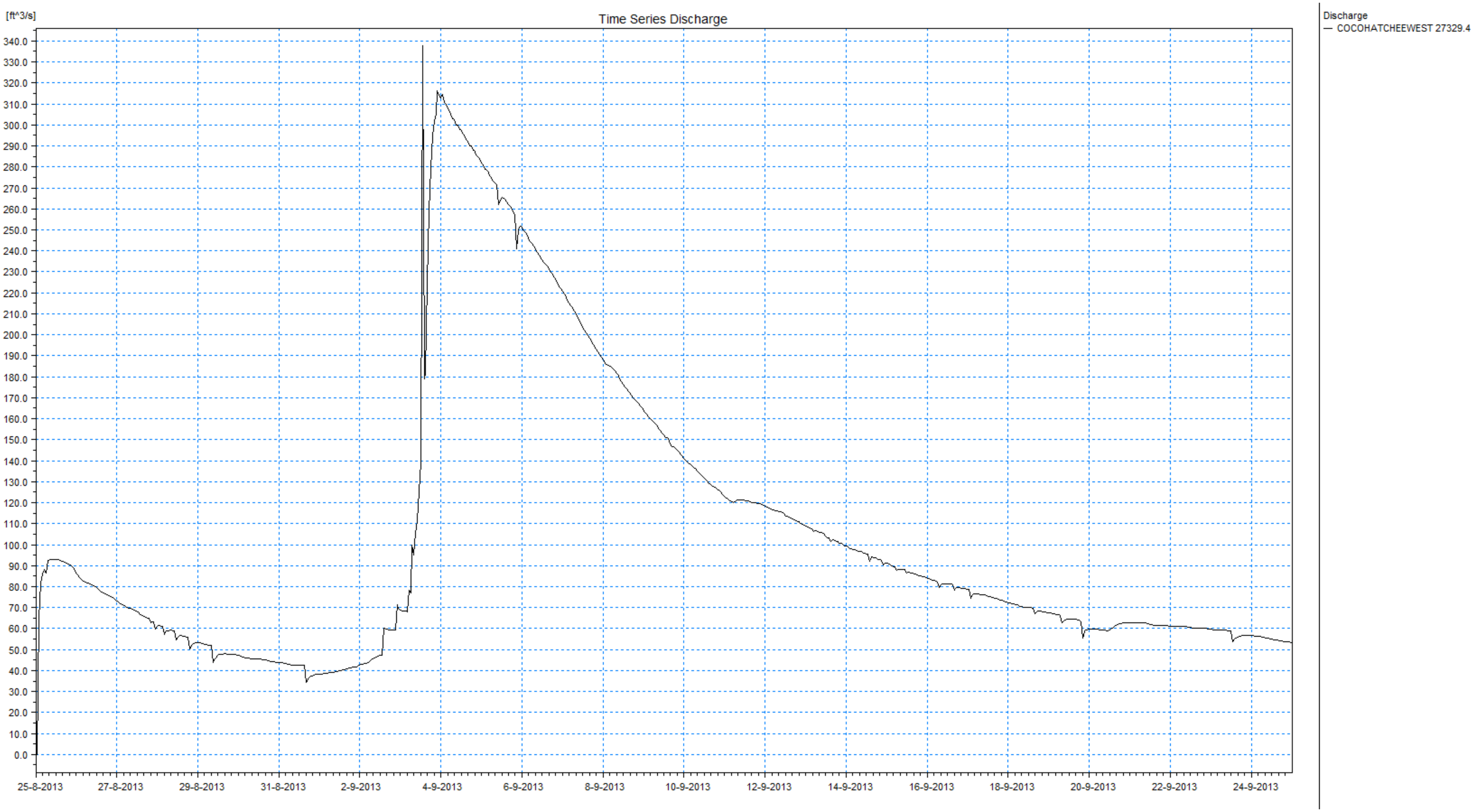

Figure 8. COCO3 Discharge (cfs)

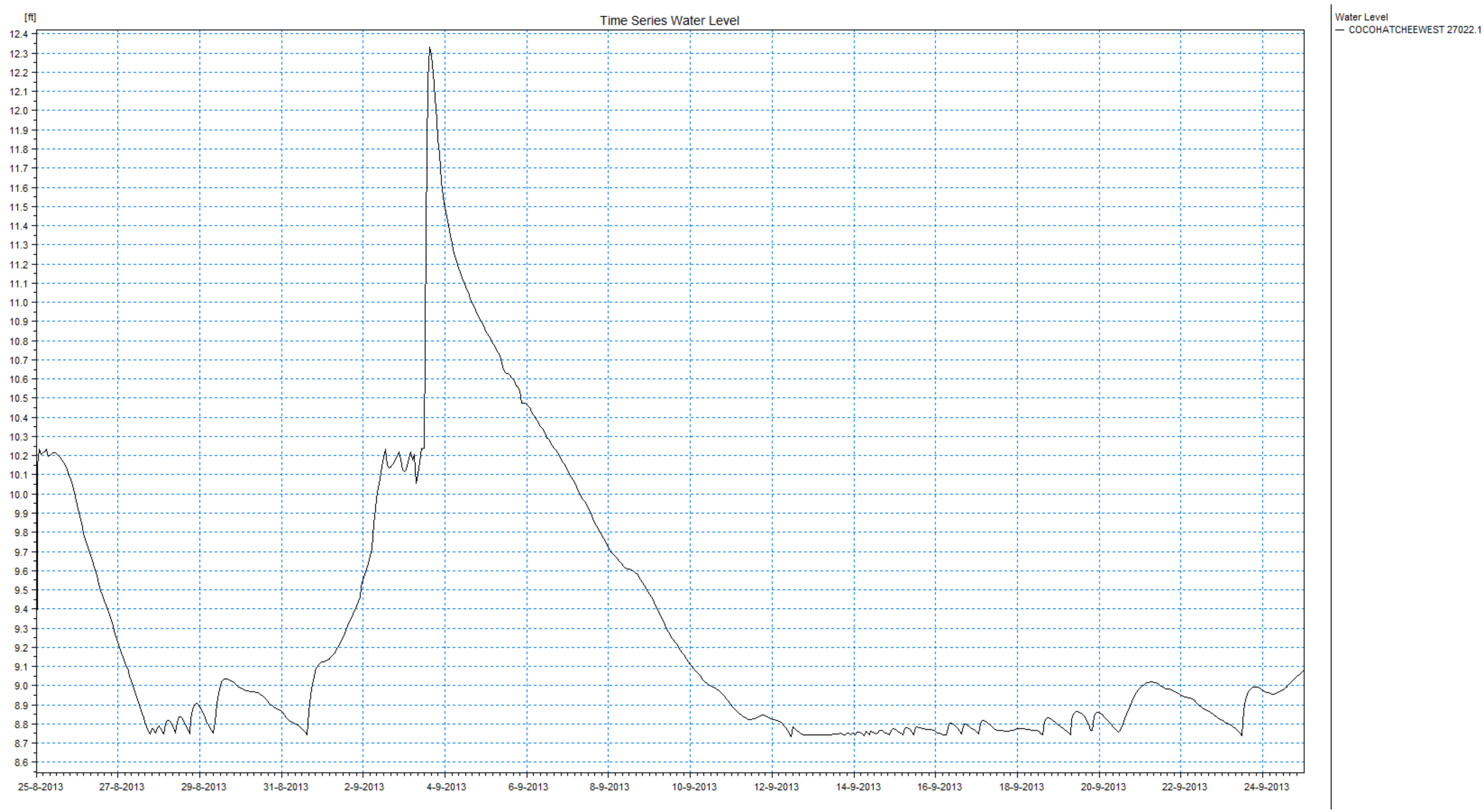

Figure 9. COCO3 Headwater Stage (ft)

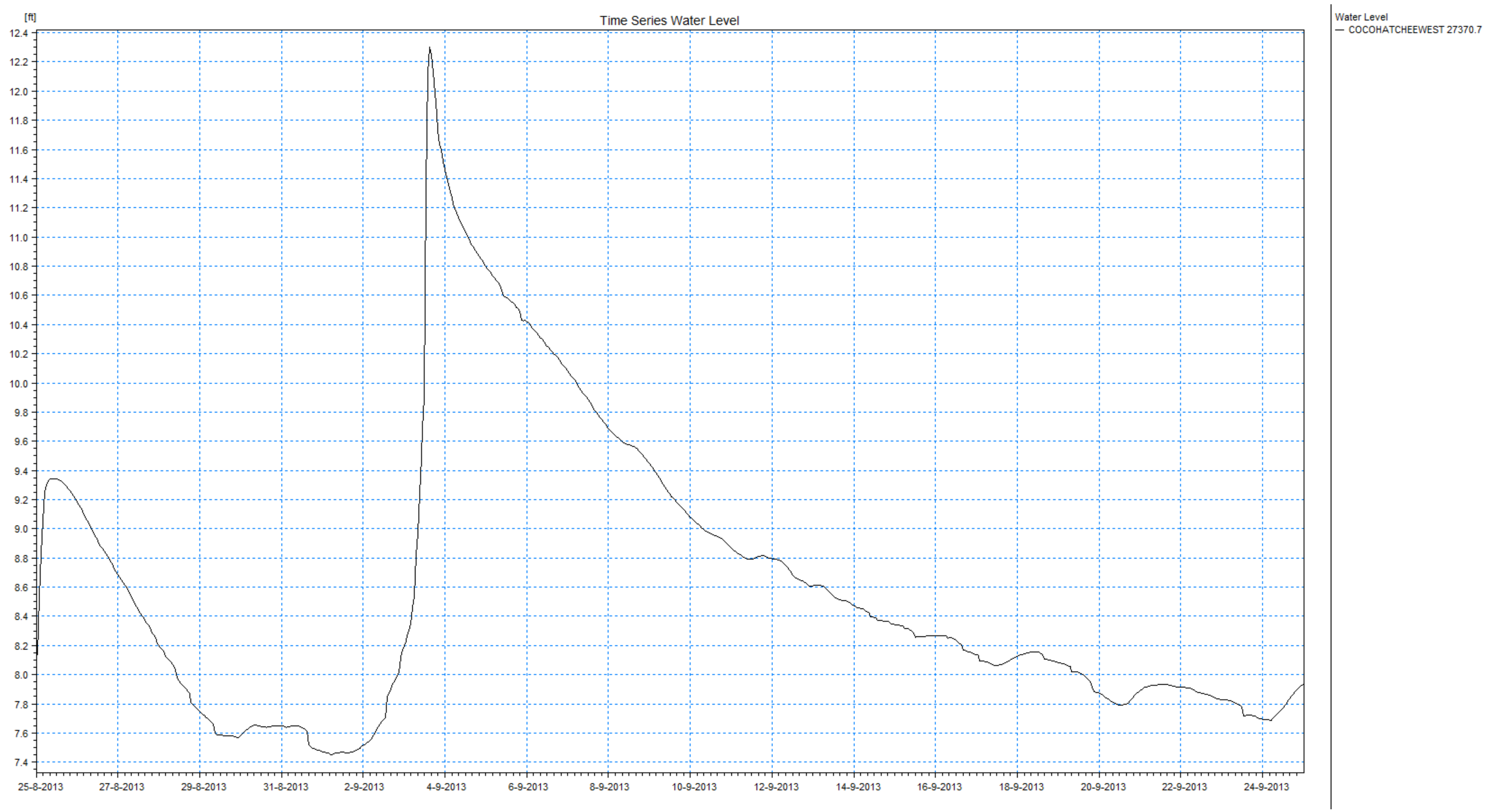

Figure 10. COCO3 Tailwater Stage (ft)

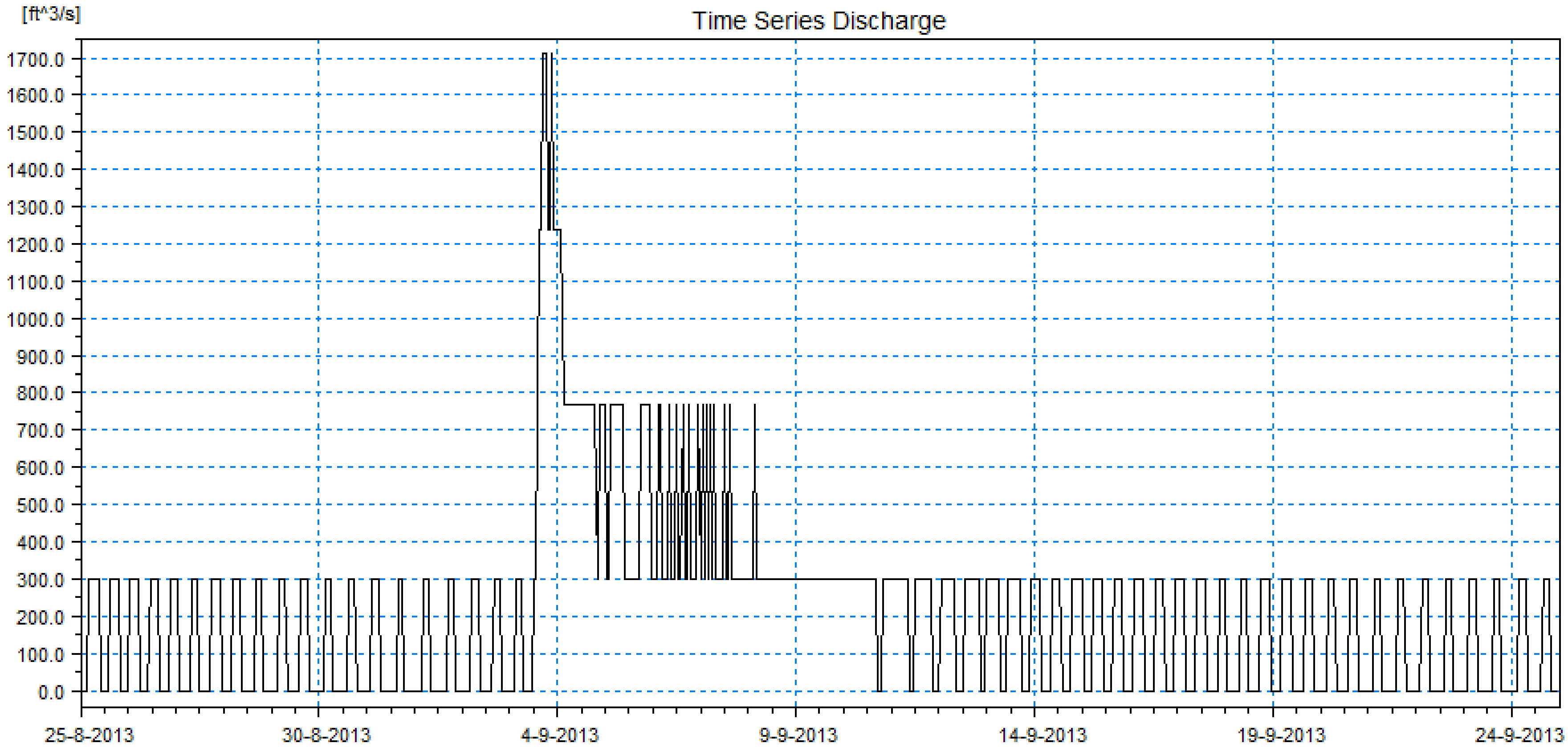

Figure 11. FU Pump Discharge (cfs)

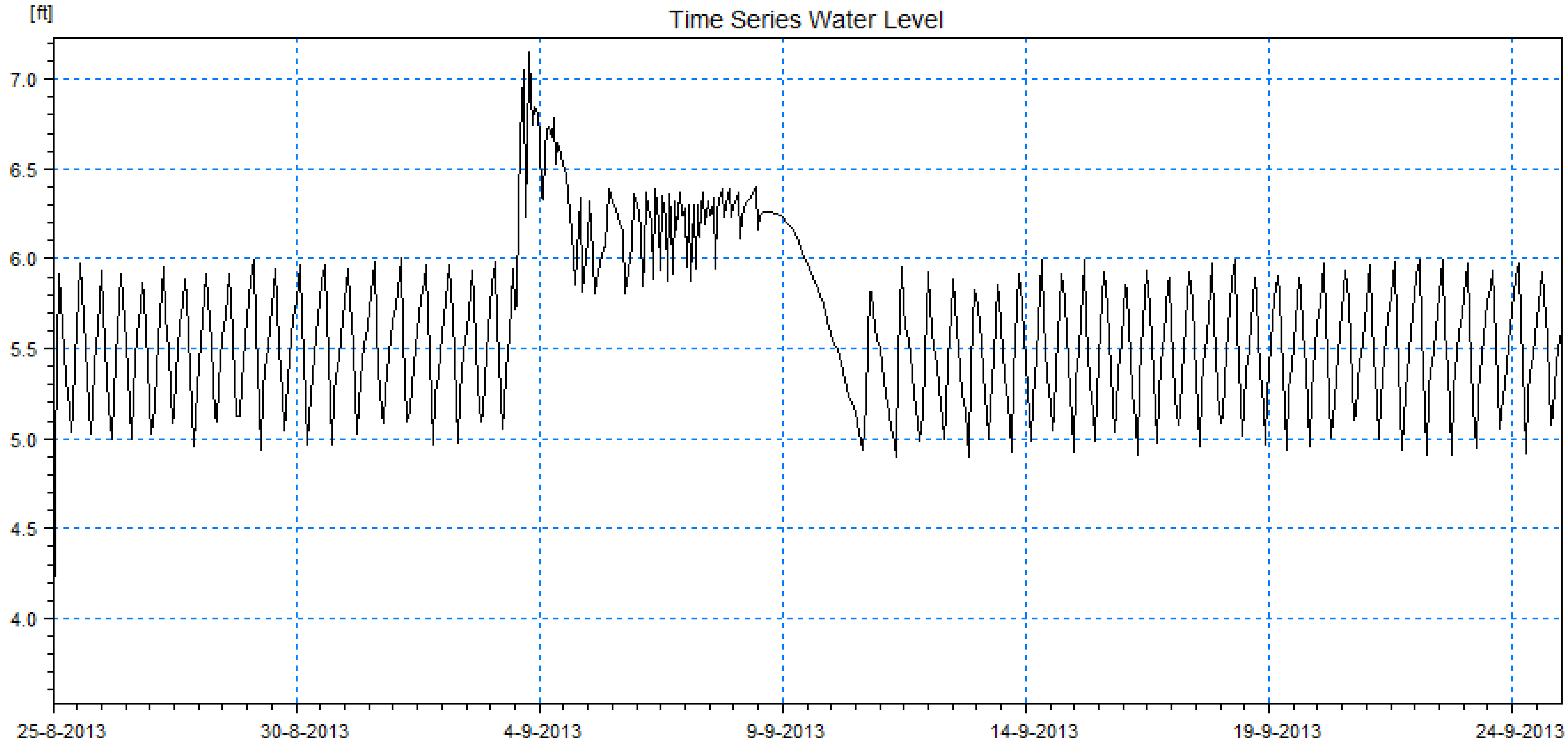

Figure 12. FU Pump Headwater Stage (ft)

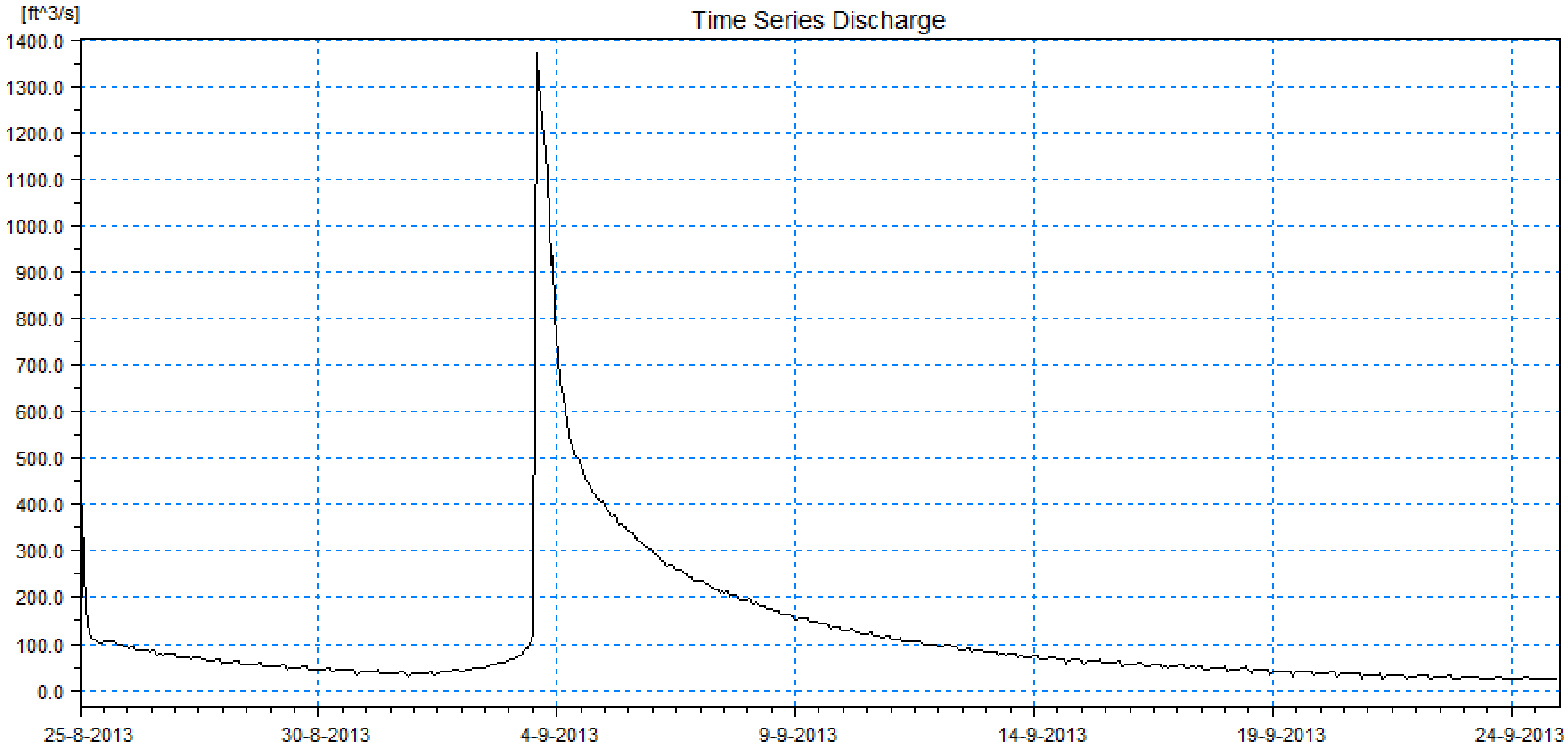

Figure 13. FU4 Discharge (cfs)

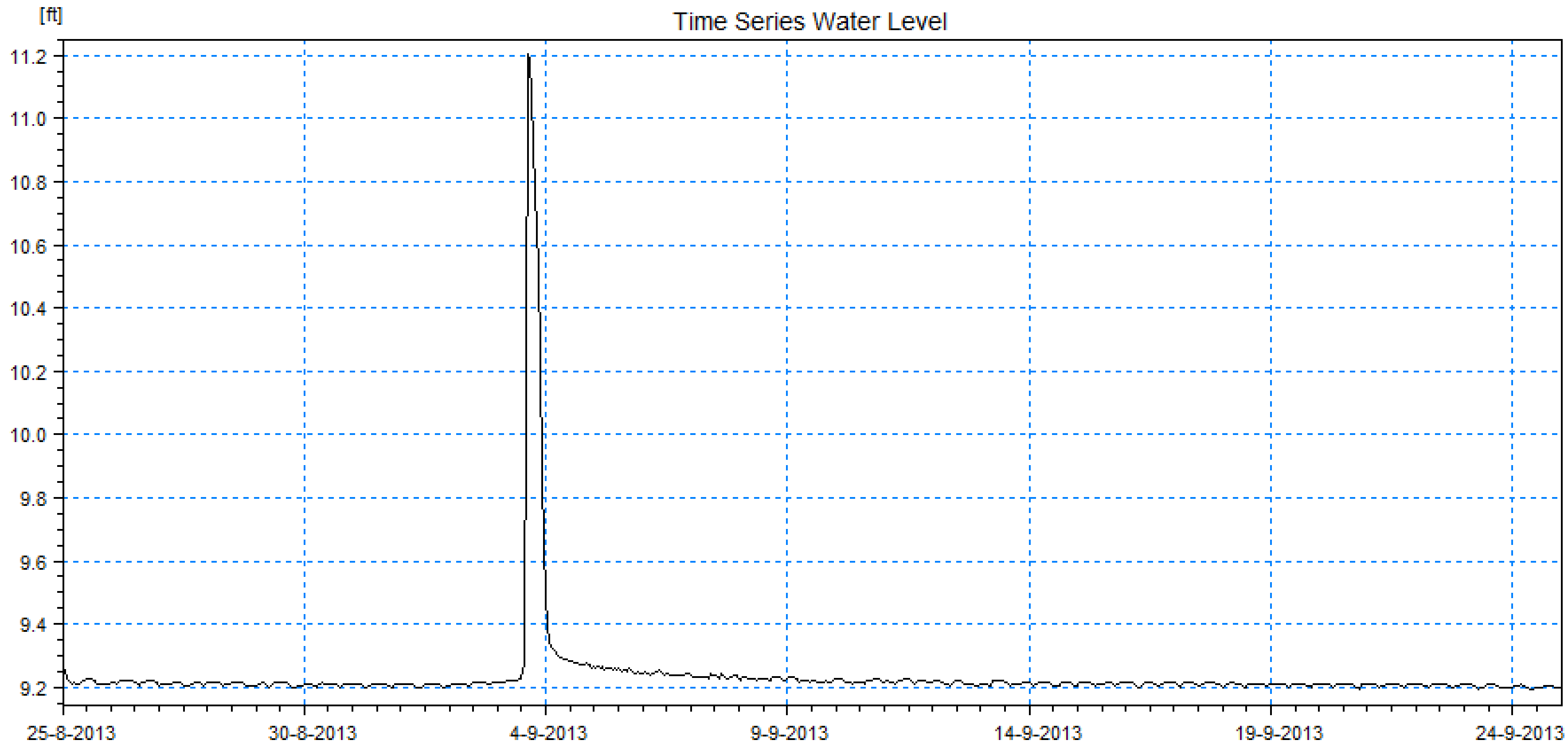

Figure 14. FU4 Headwater Stage (ft)

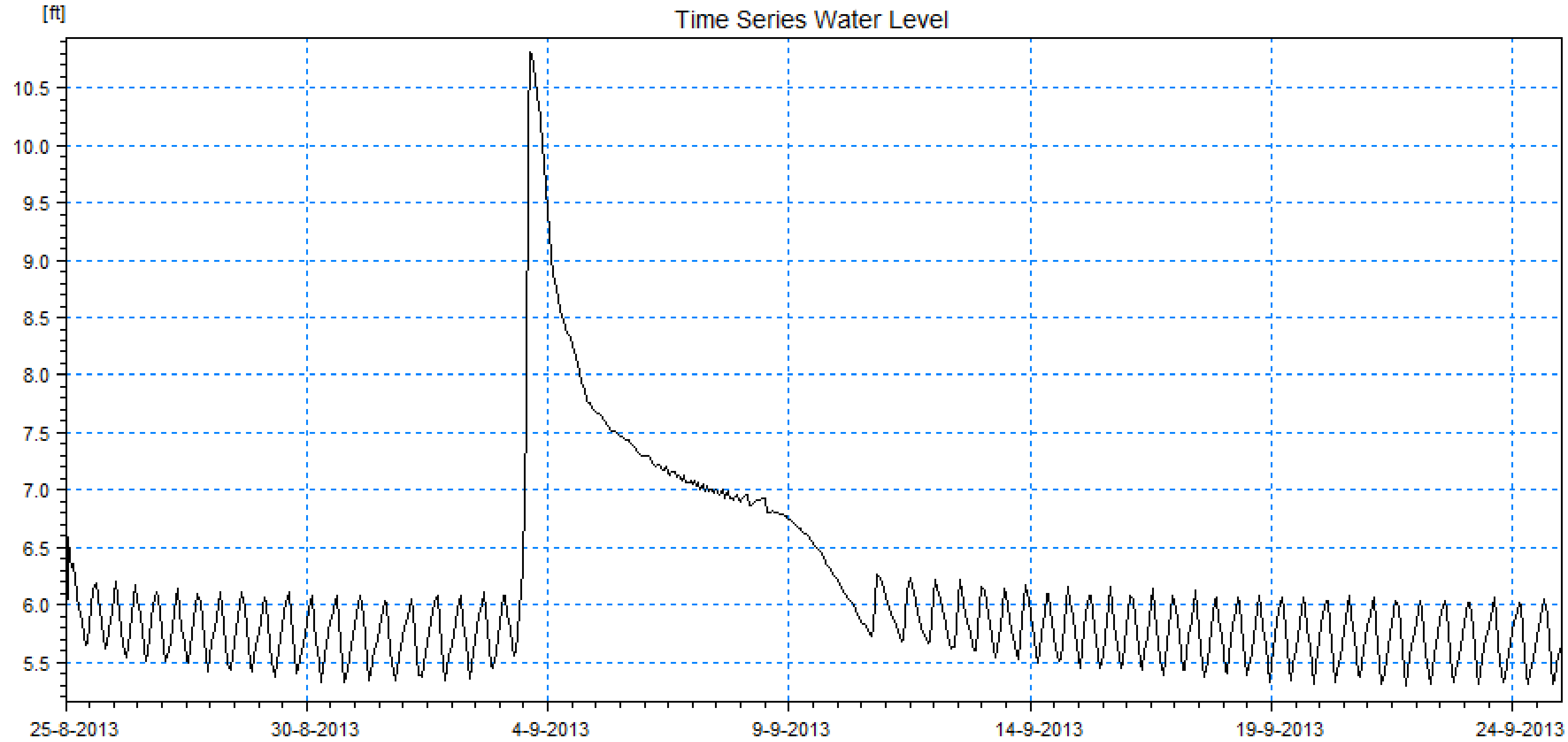

Figure 15. FU4 Tailwater Stage (ft)

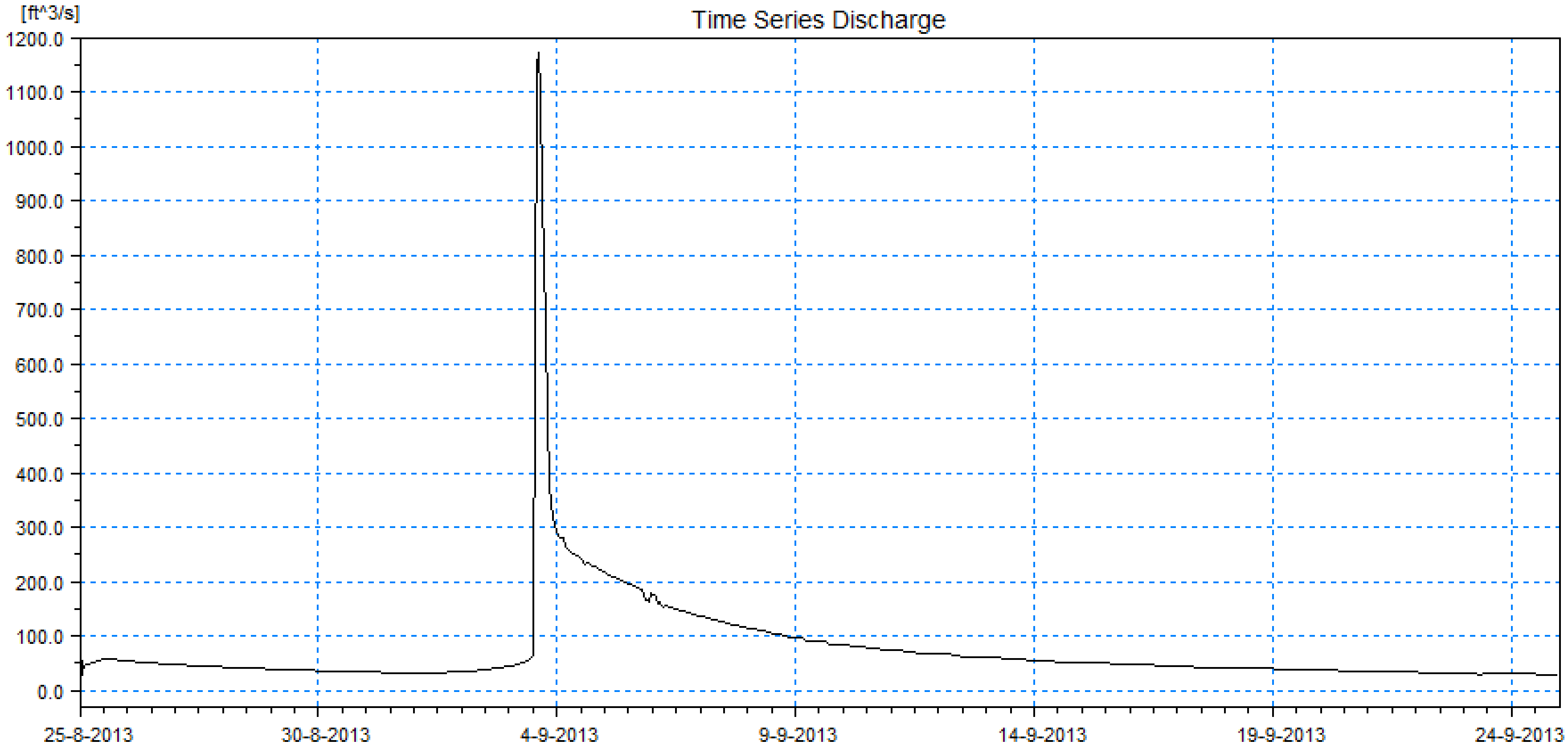

Figure 16. FU5 Discharge (cfs)

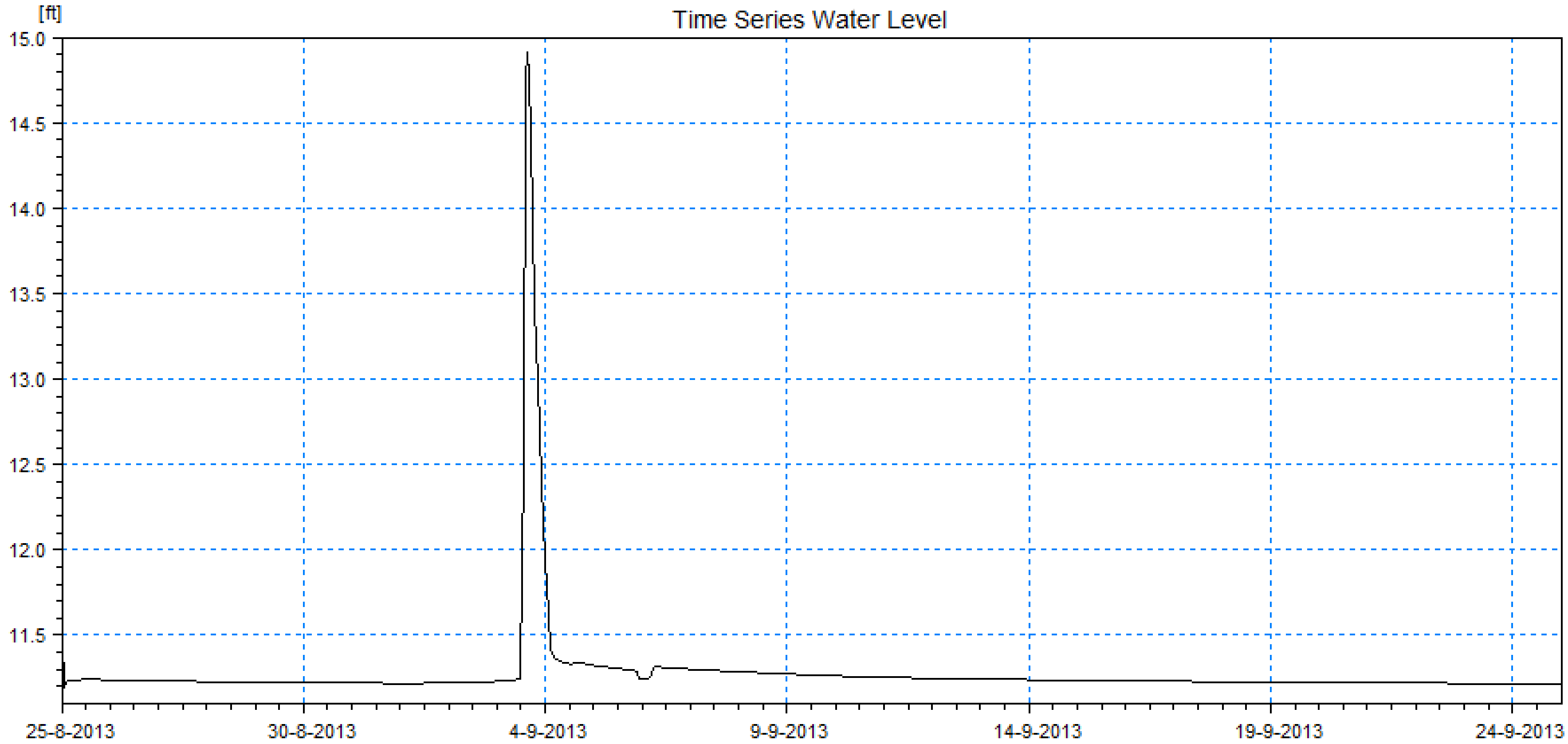

Figure 17. FU5 Headwater Stage (ft)

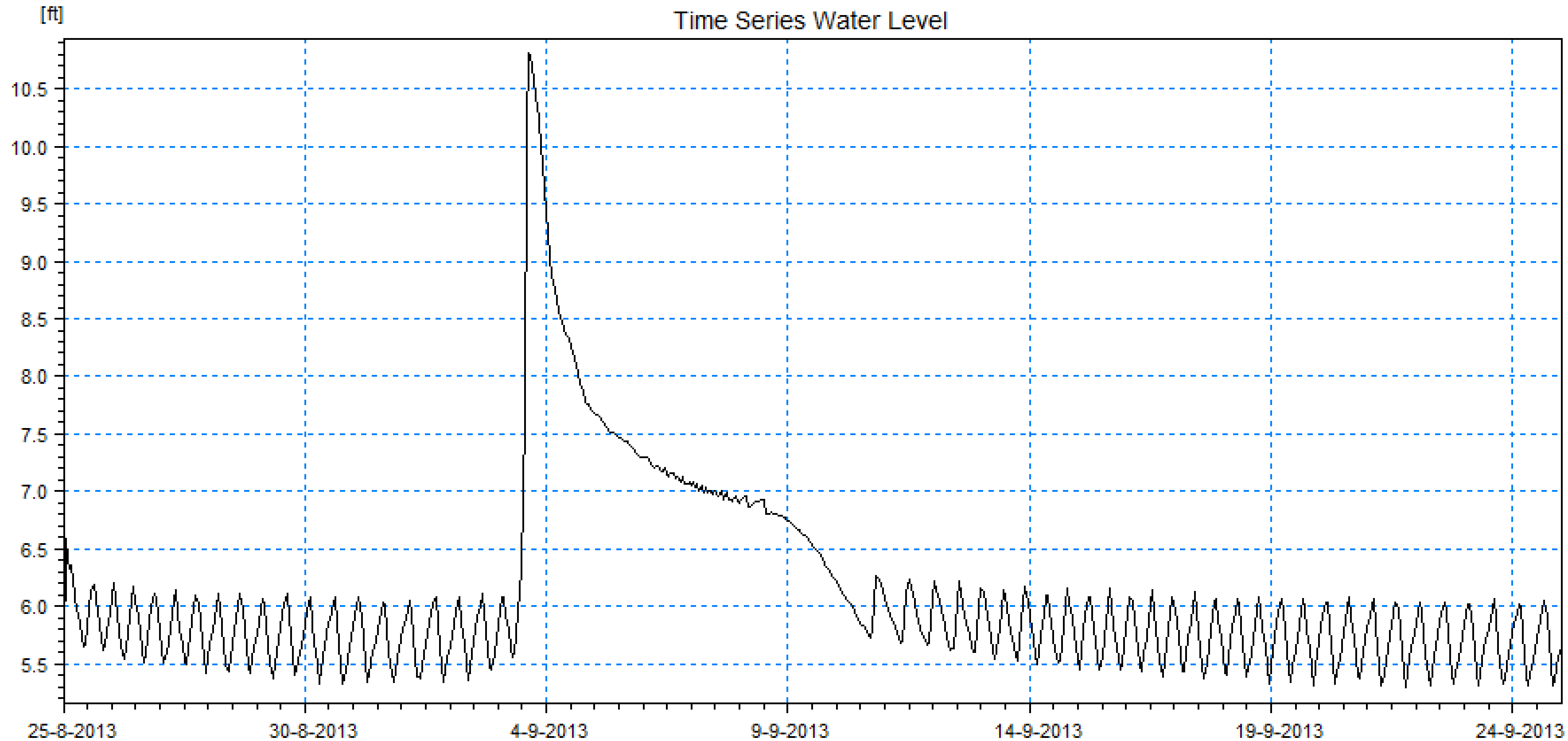

Figure 18. FU5 Tailwater Stage (ft)

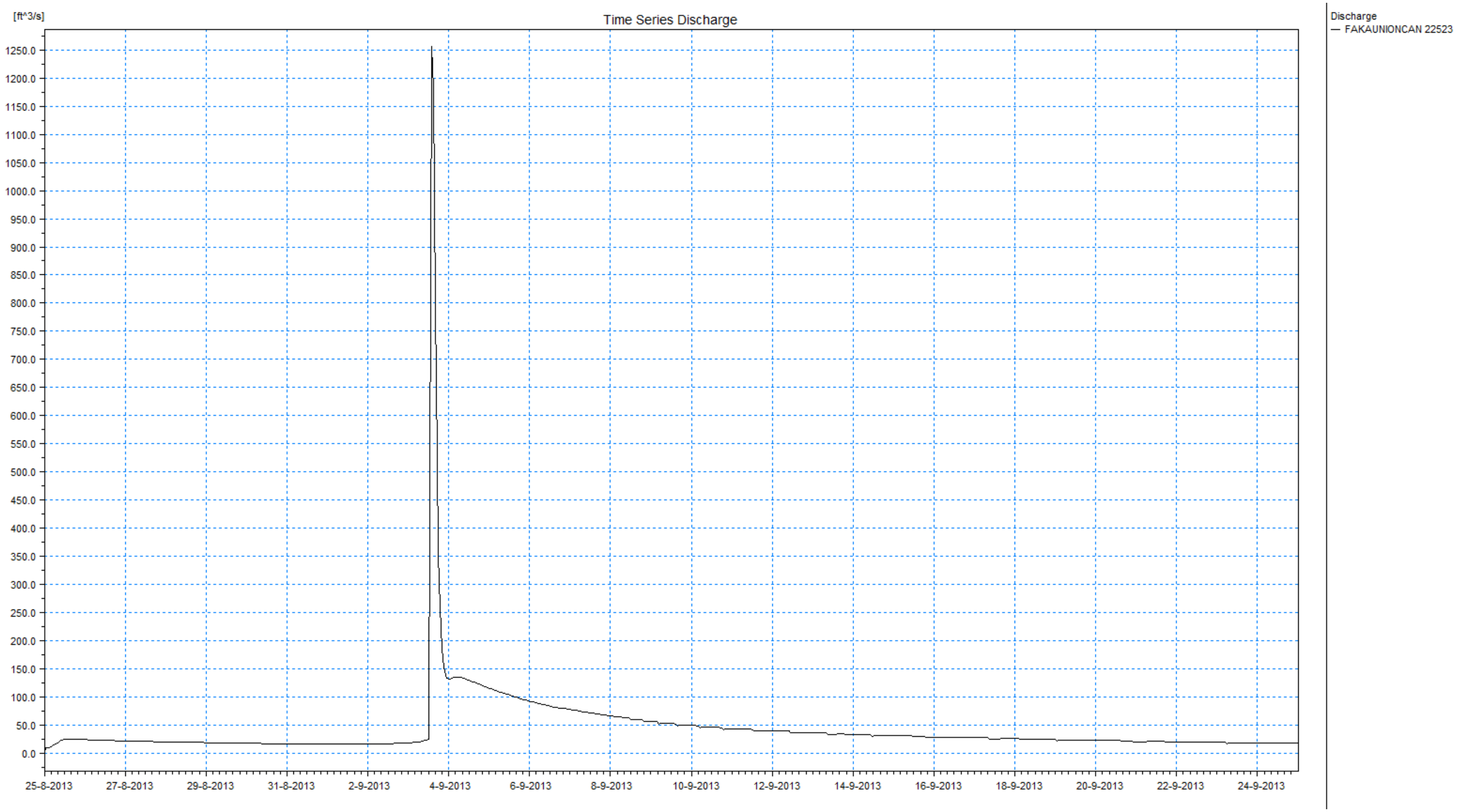

Figure 19. FU6 Discharge (cfs)

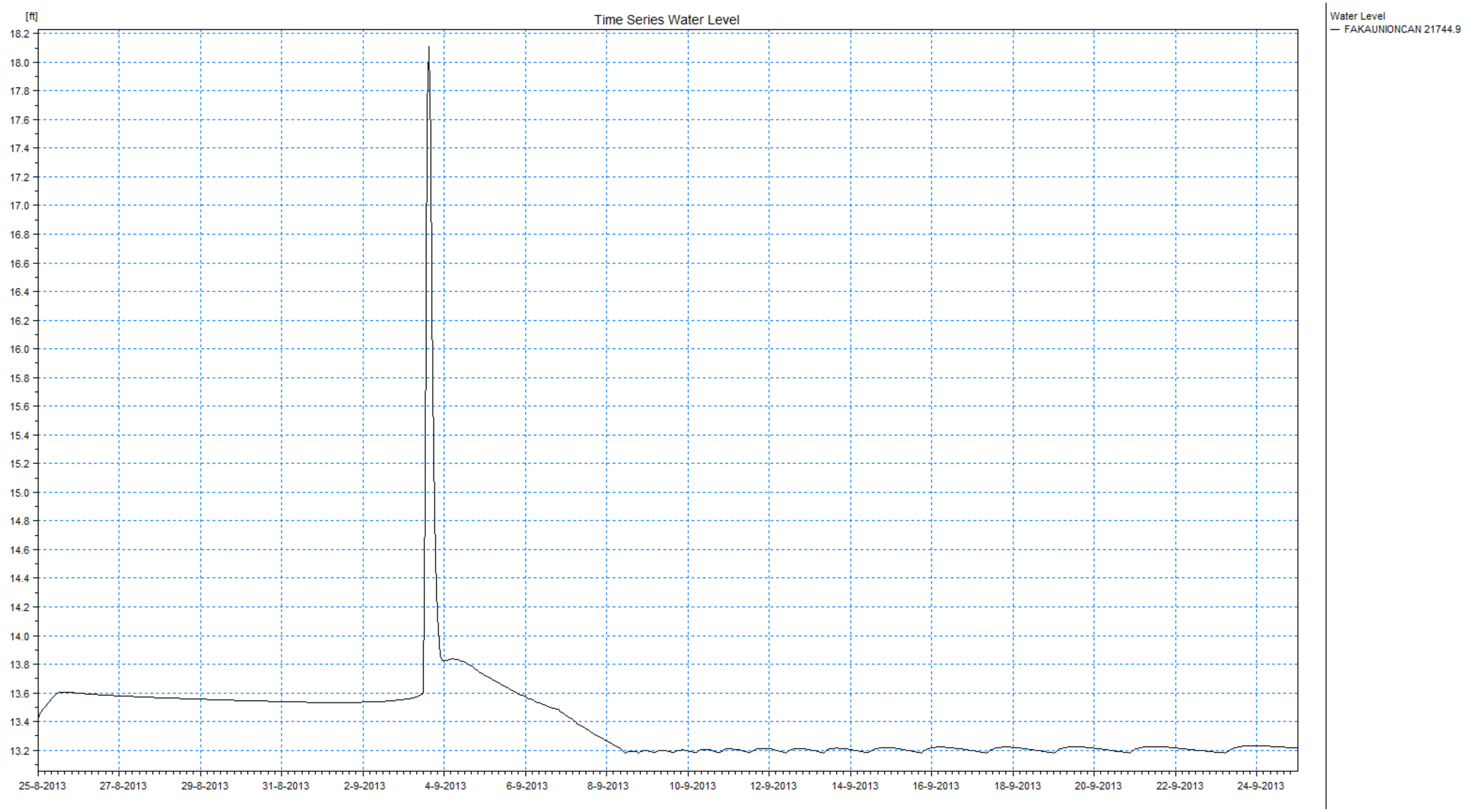

Figure 20. FU6 Headwater Stage (ft)

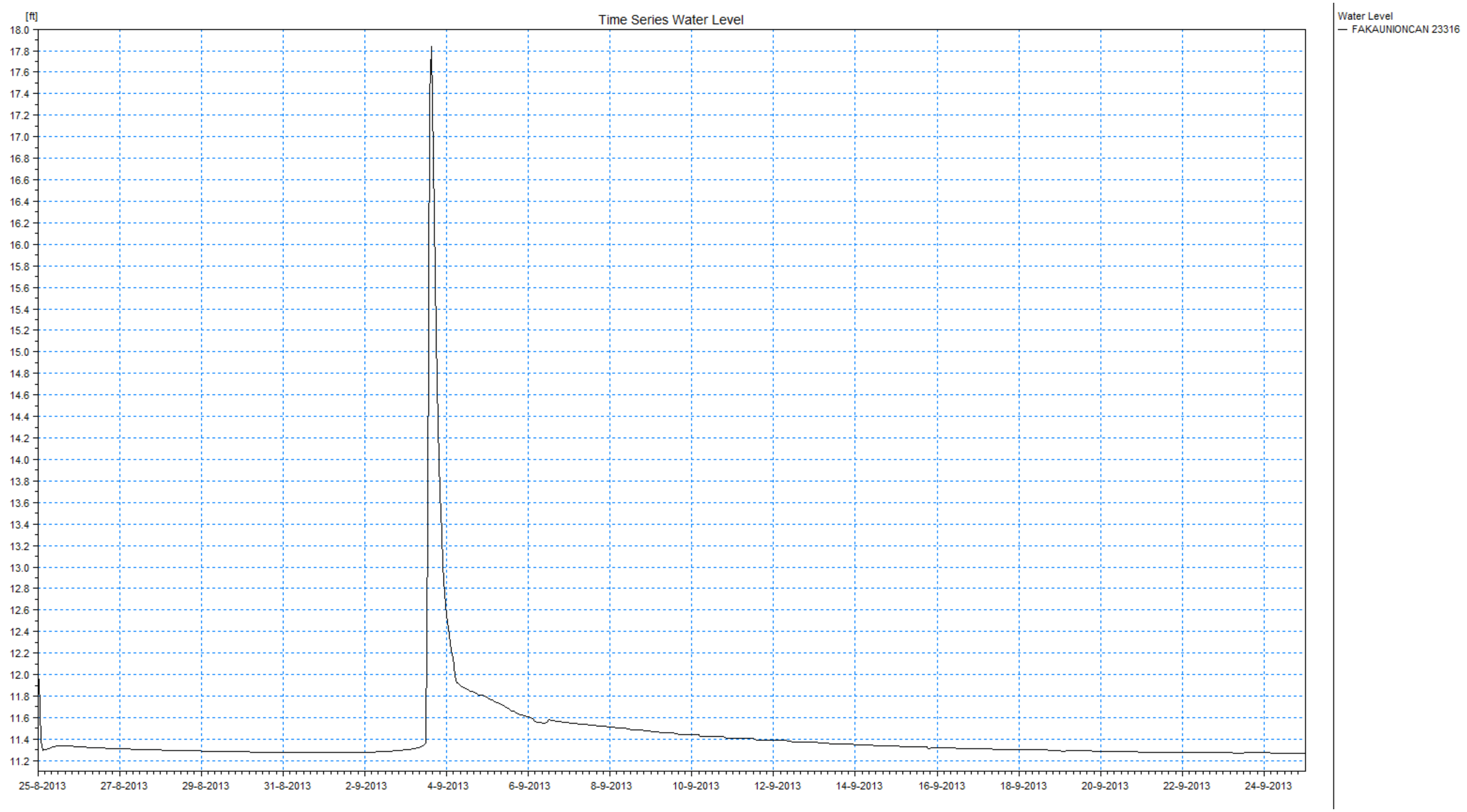

Figure 21. FU6 Tailwater Stage (ft)

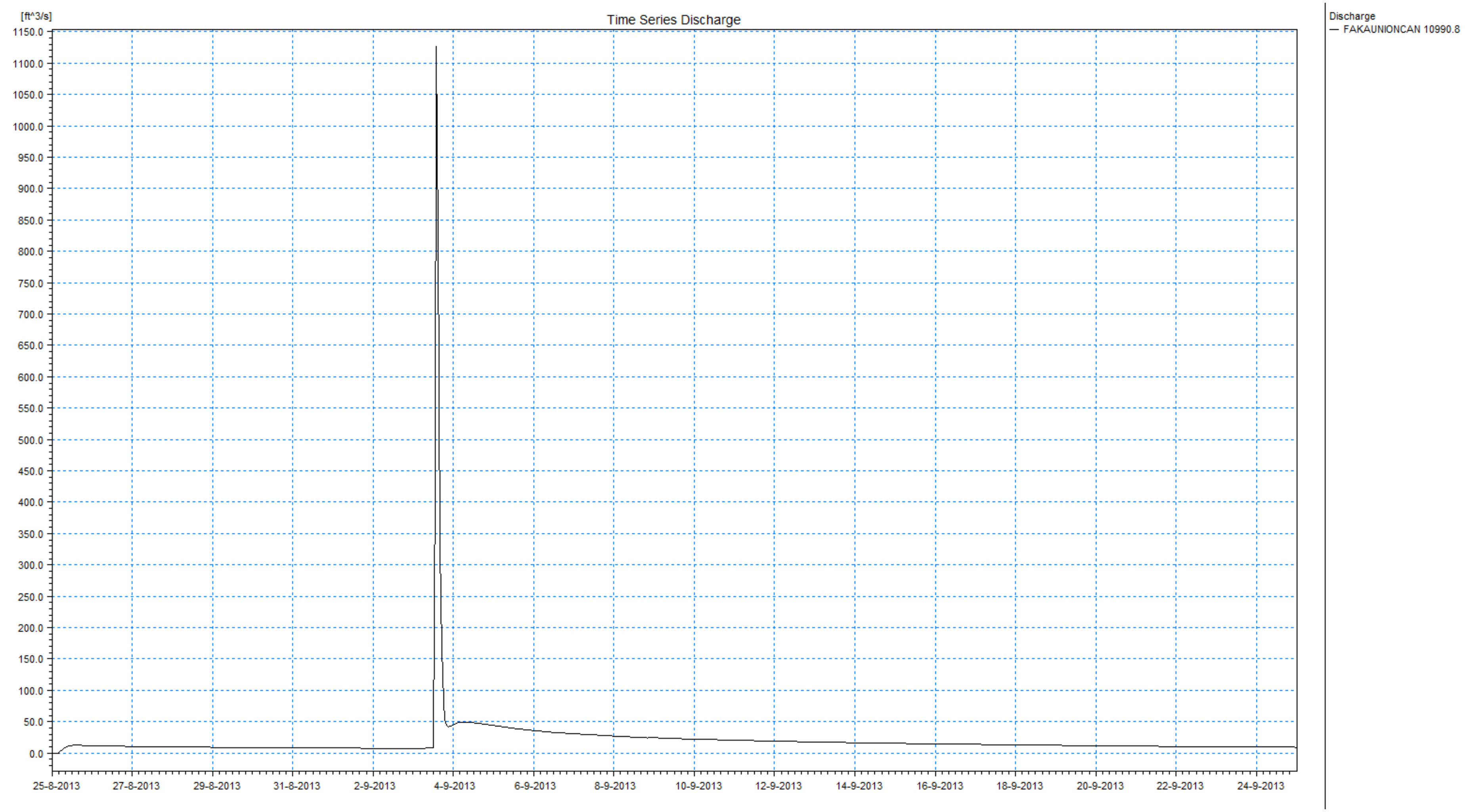

Figure 22. FU7 Discharge (cfs)

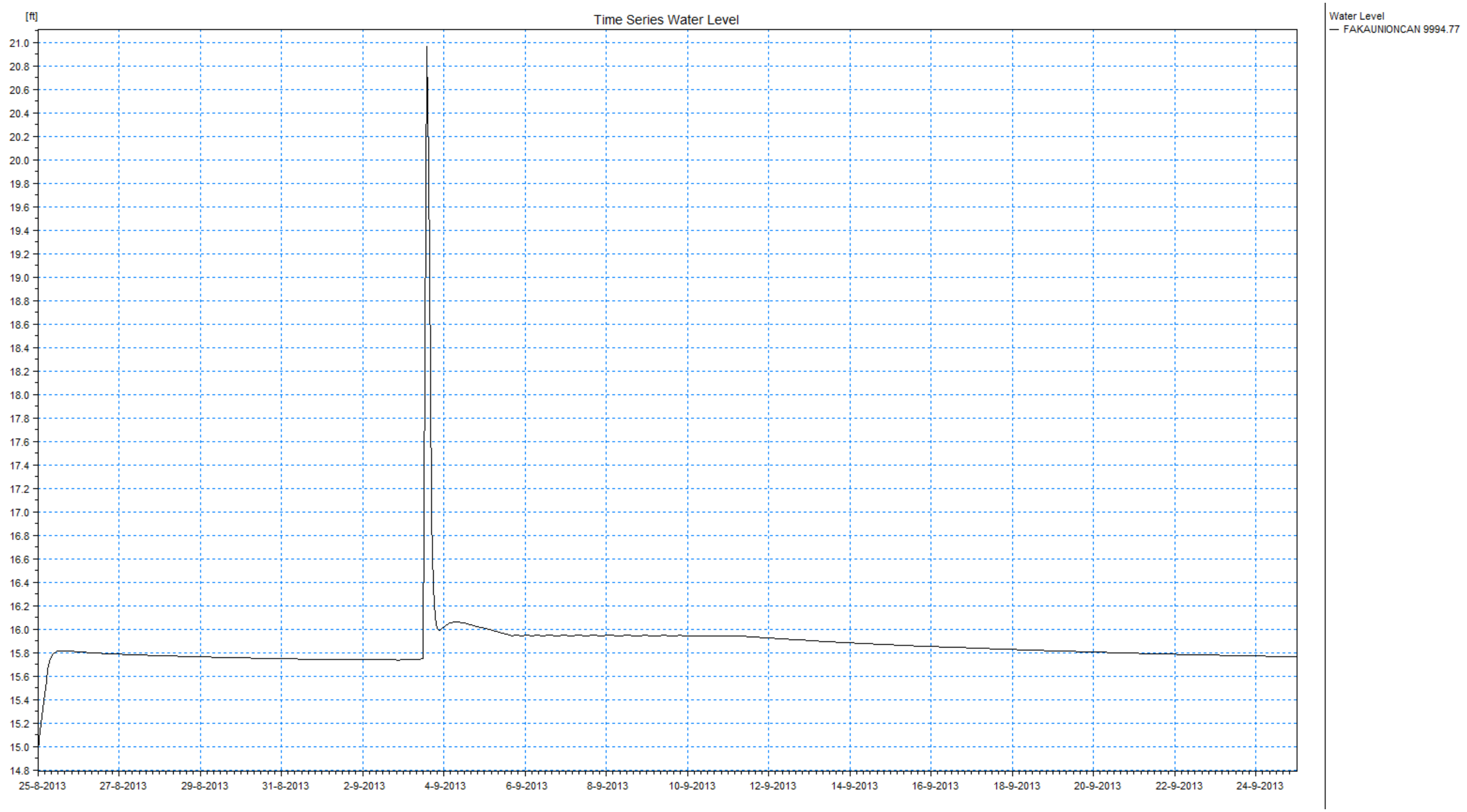

Figure 23. FU7 Headwater Stage (ft)

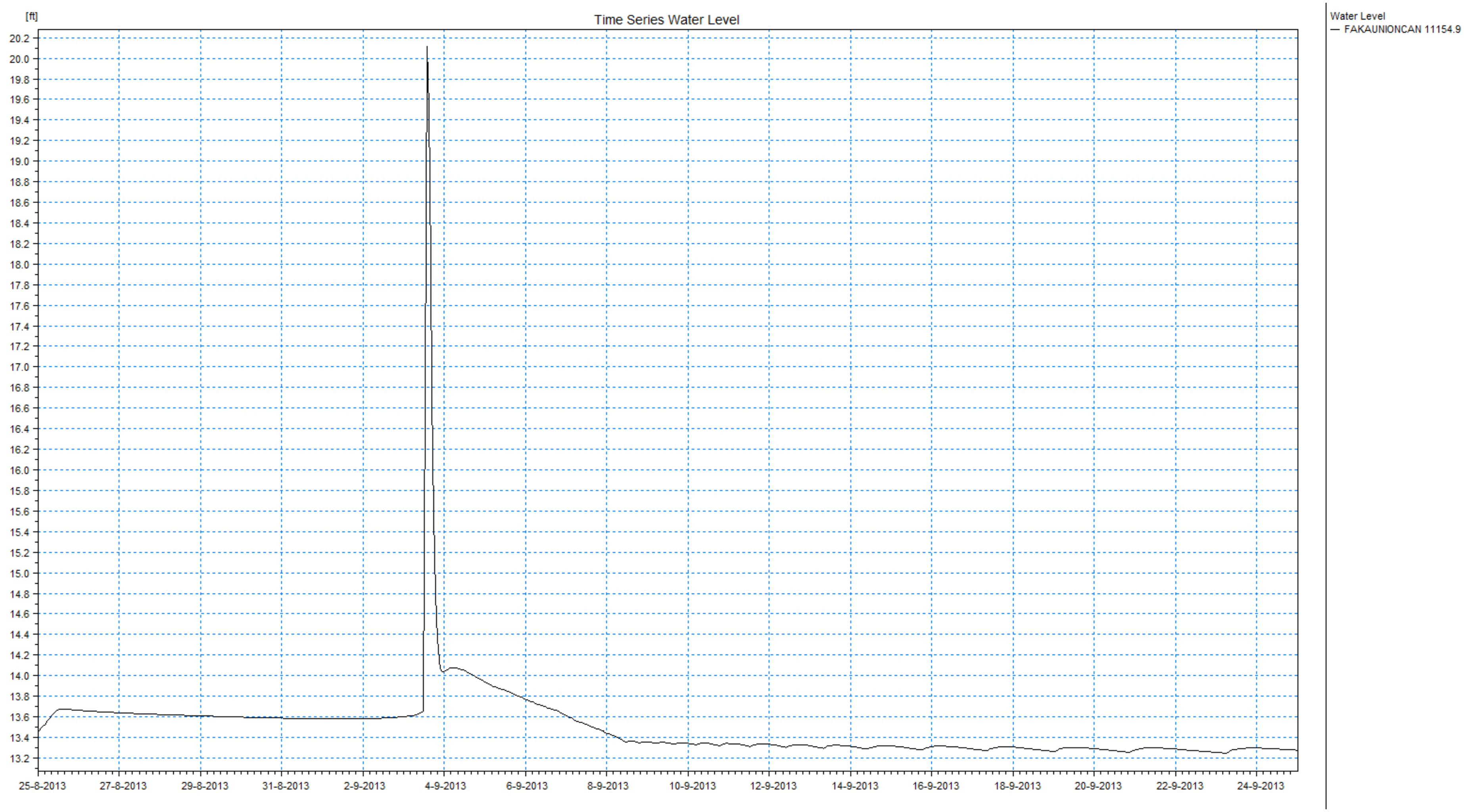

Figure 24. FU7 Tailwater Stage (ft)

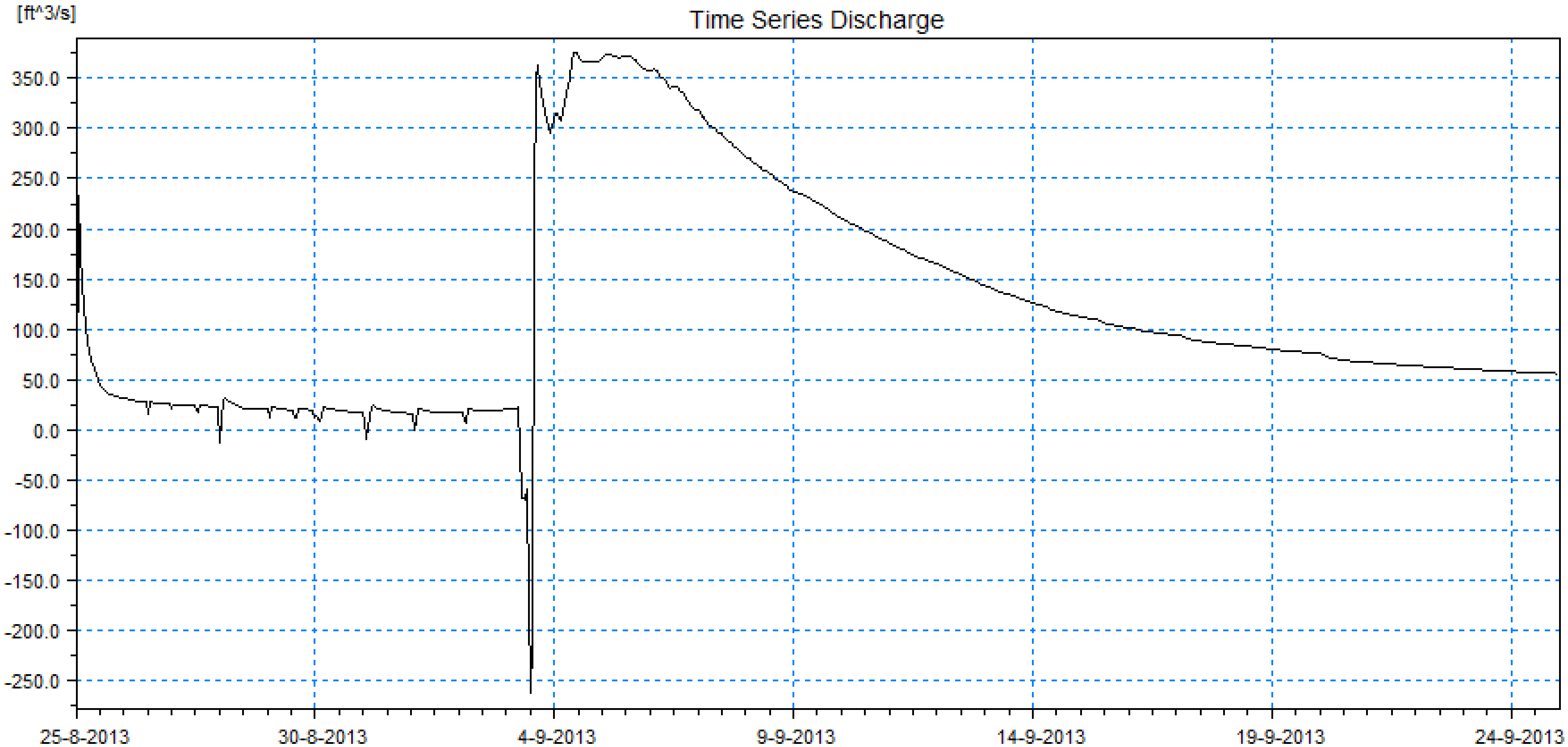

Figure 25. HC1 Discharge (cfs)

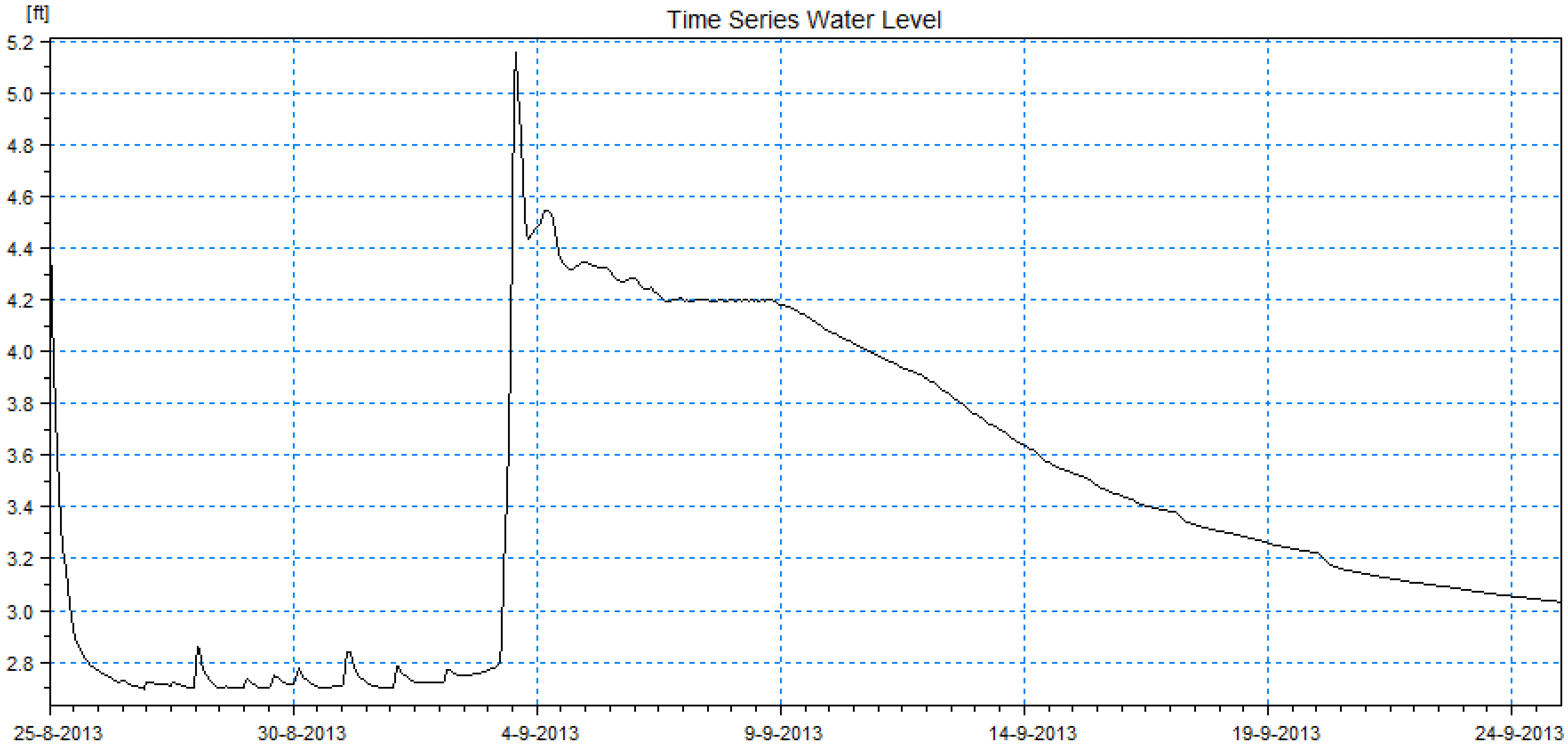

Figure 26. HC1 Headwater Stage (ft)

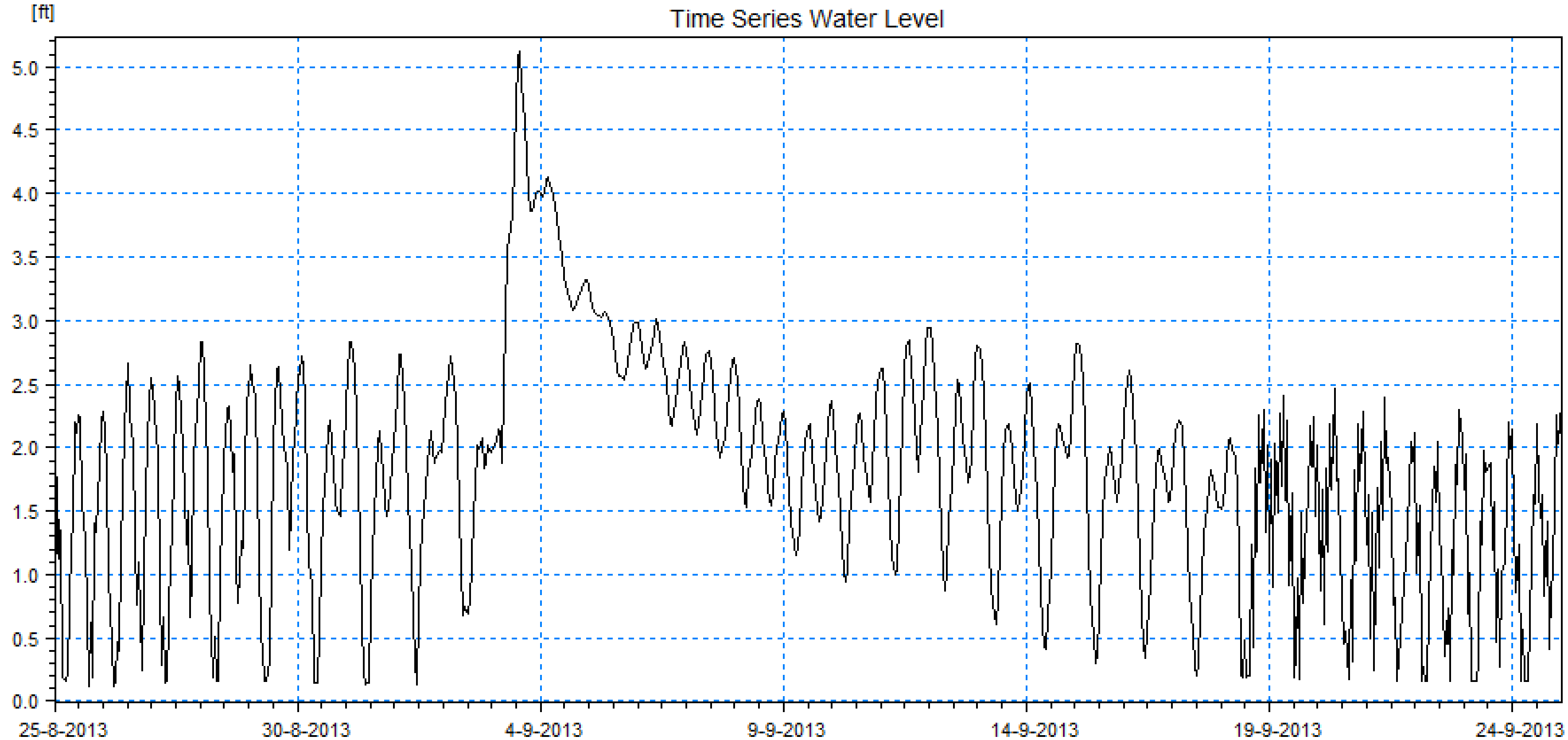

Figure 27. HC1 Tailwater Stage (ft)

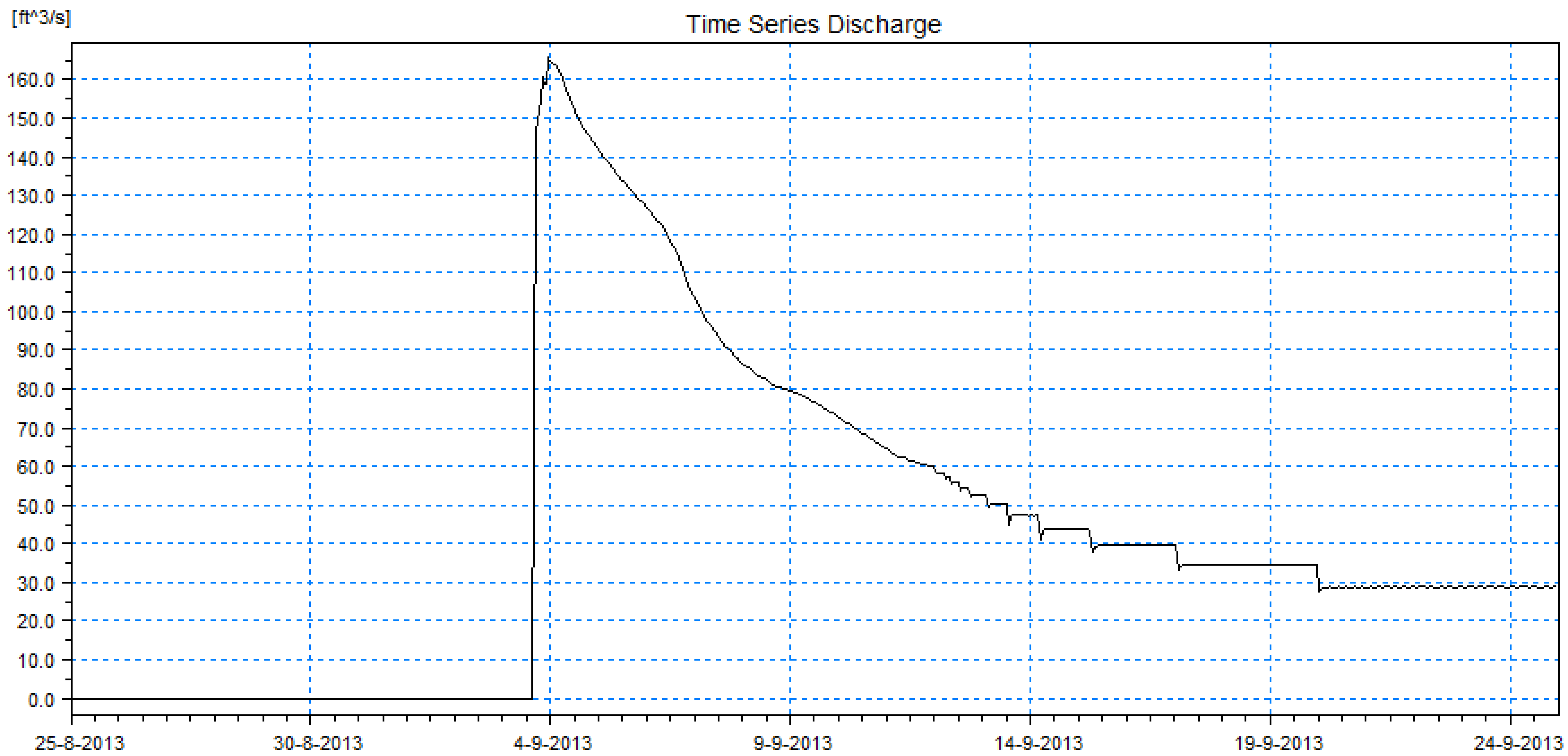

Figure 28. HC2 Discharge (cfs)

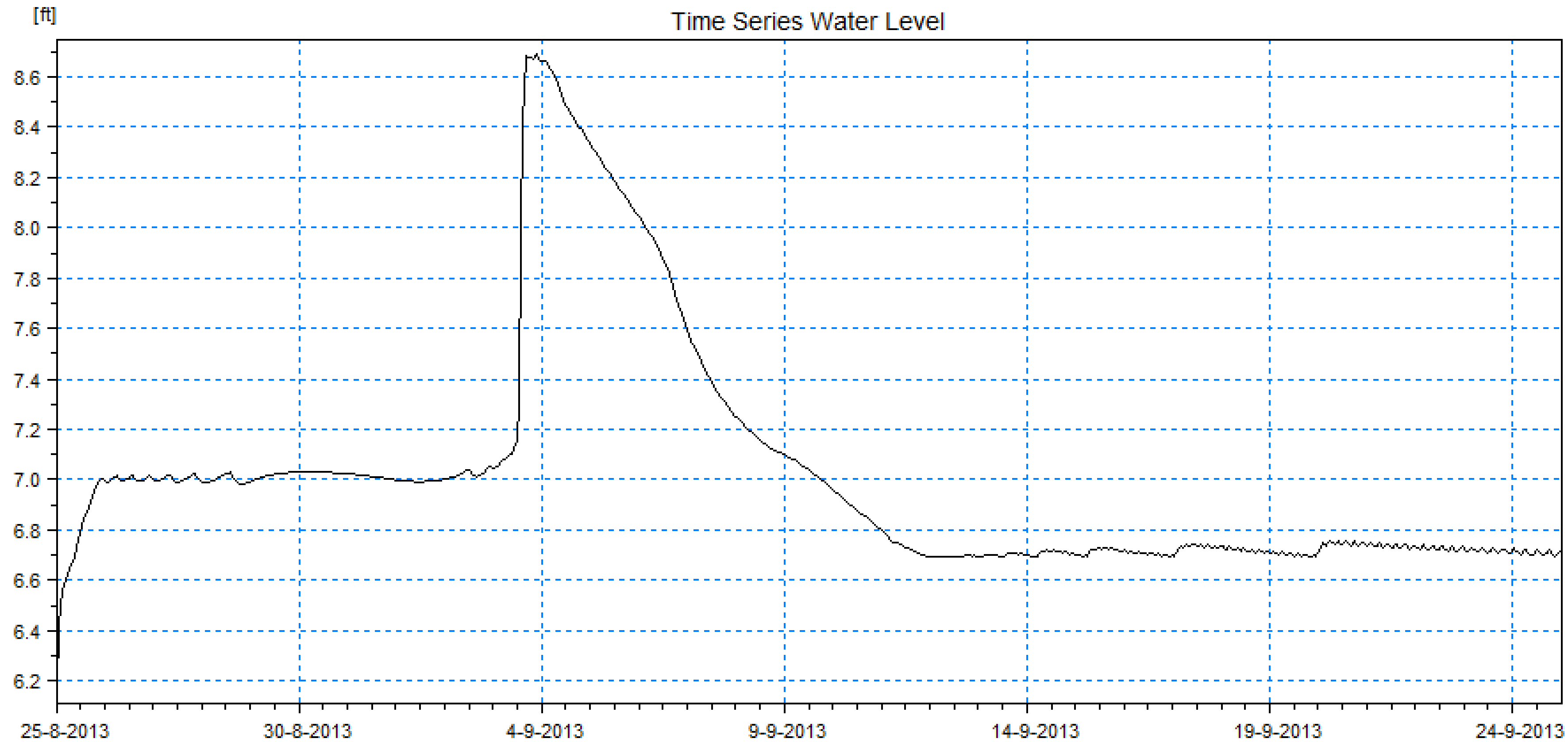

Figure 29. HC2 Headwater Stage (ft)

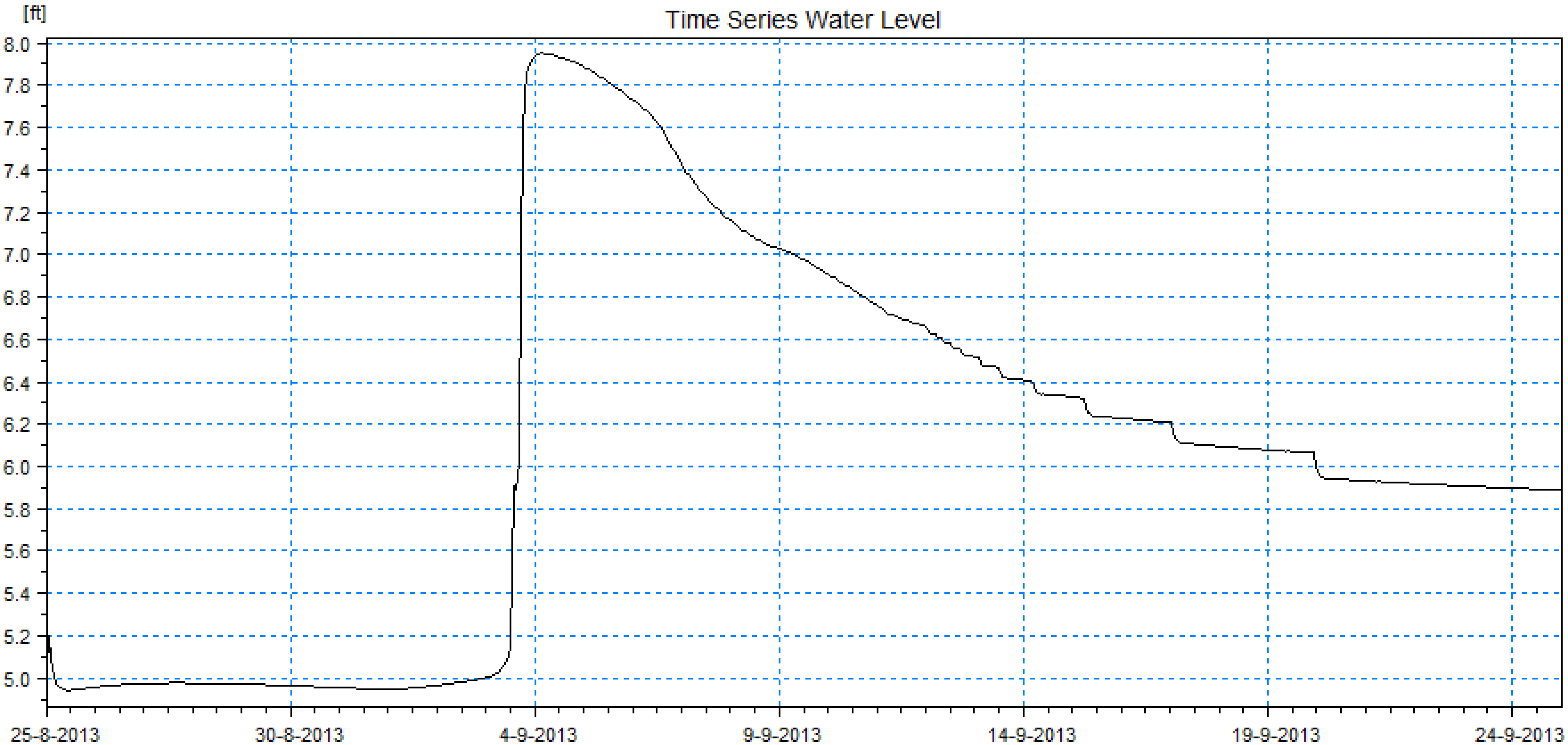

Figure 30. HC2 Tailwater Stage (ft)

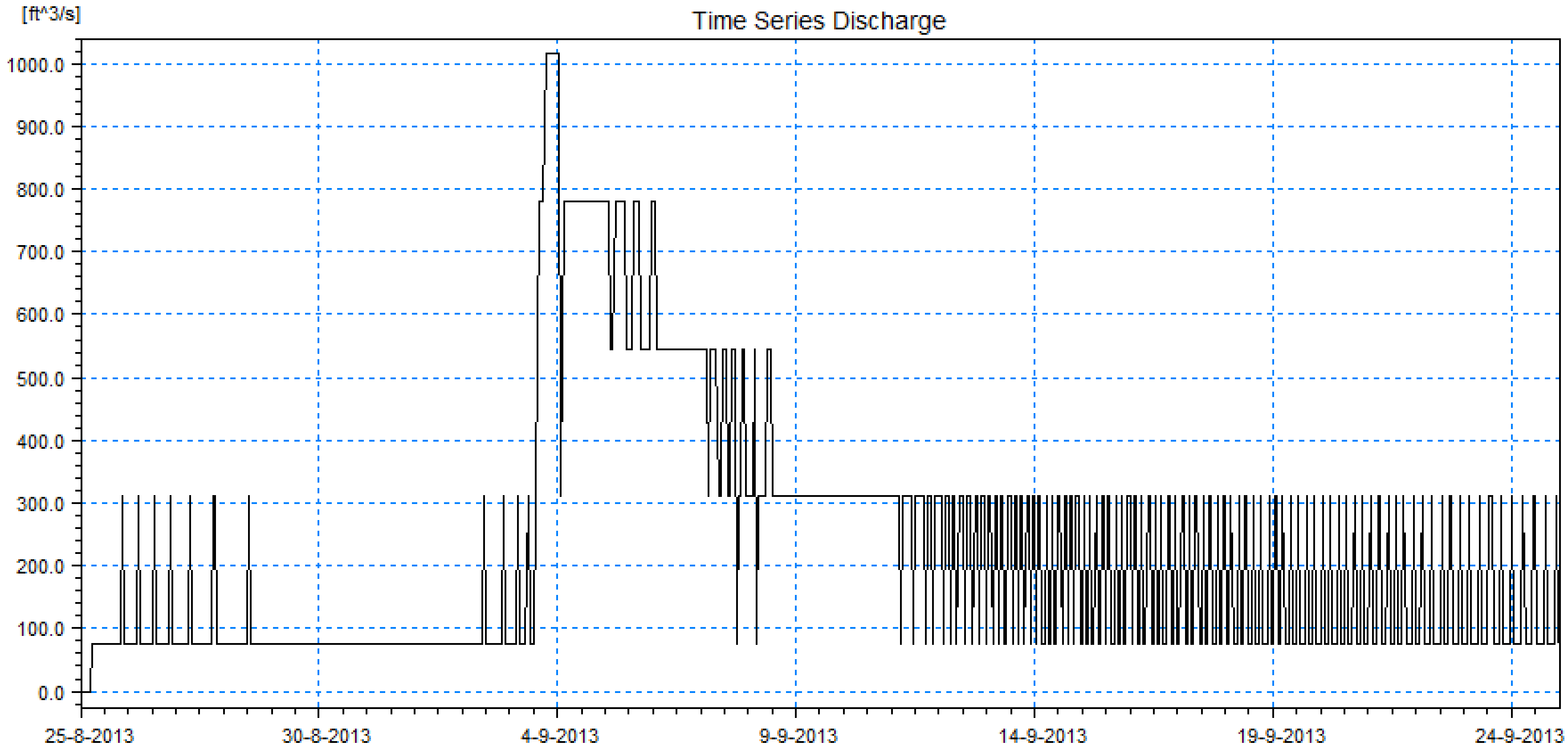

Figure 31- Miller Pump Discharge (cfs)

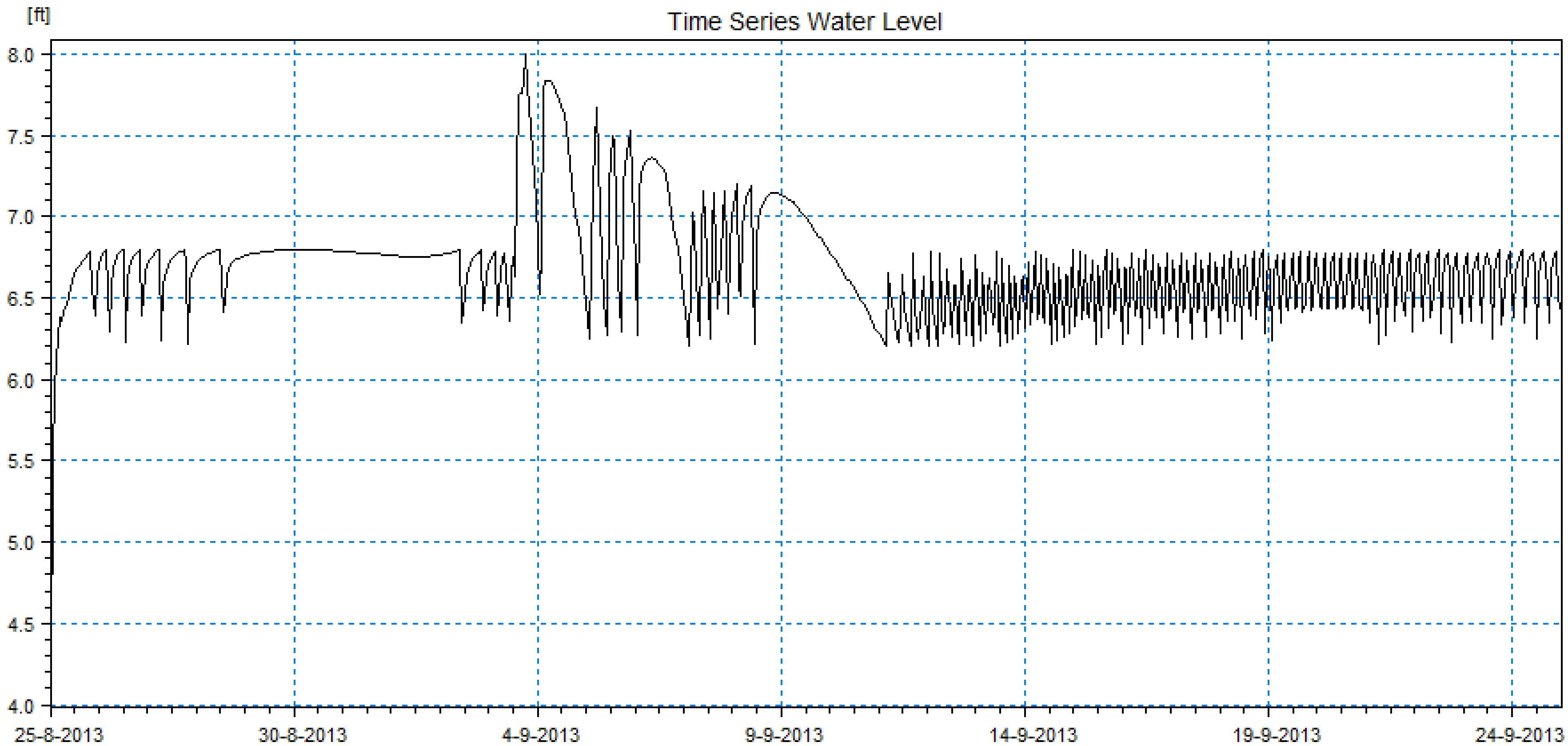

Figure 32- Miller Pump Headwater Stage (ft)

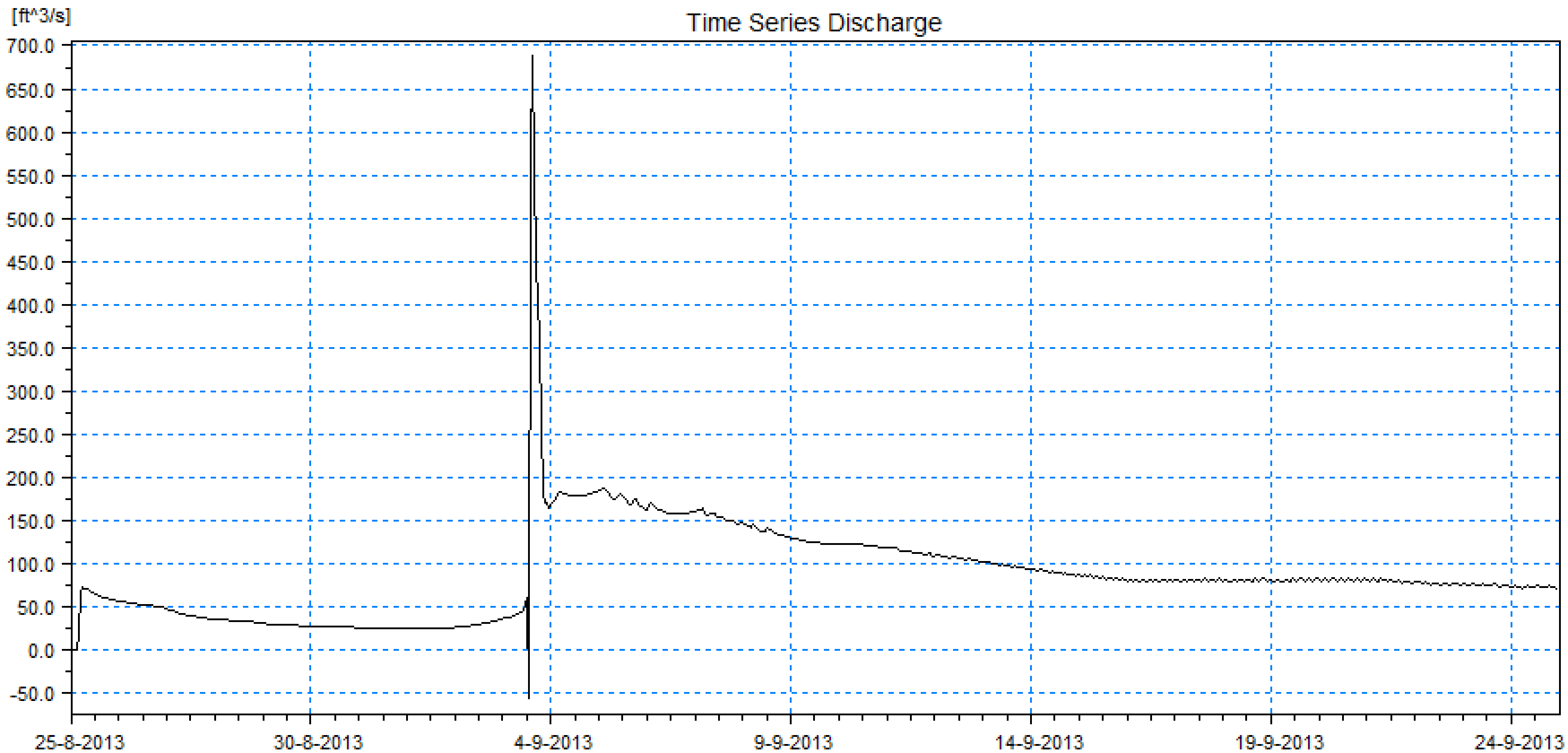

Figure 33. Miller3 Discharge (cfs)

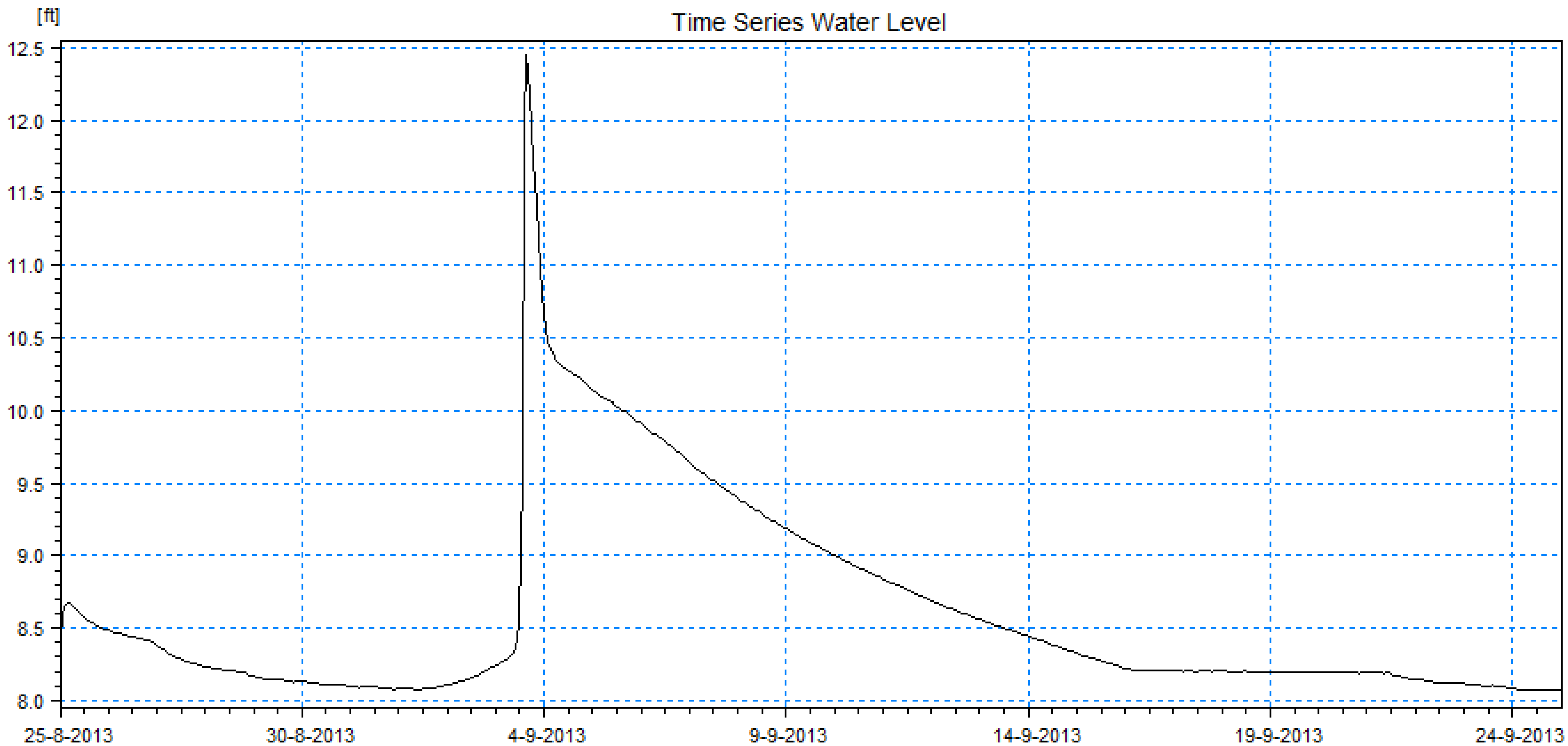

Figure 34. Miller3 Headwater Stage (ft)

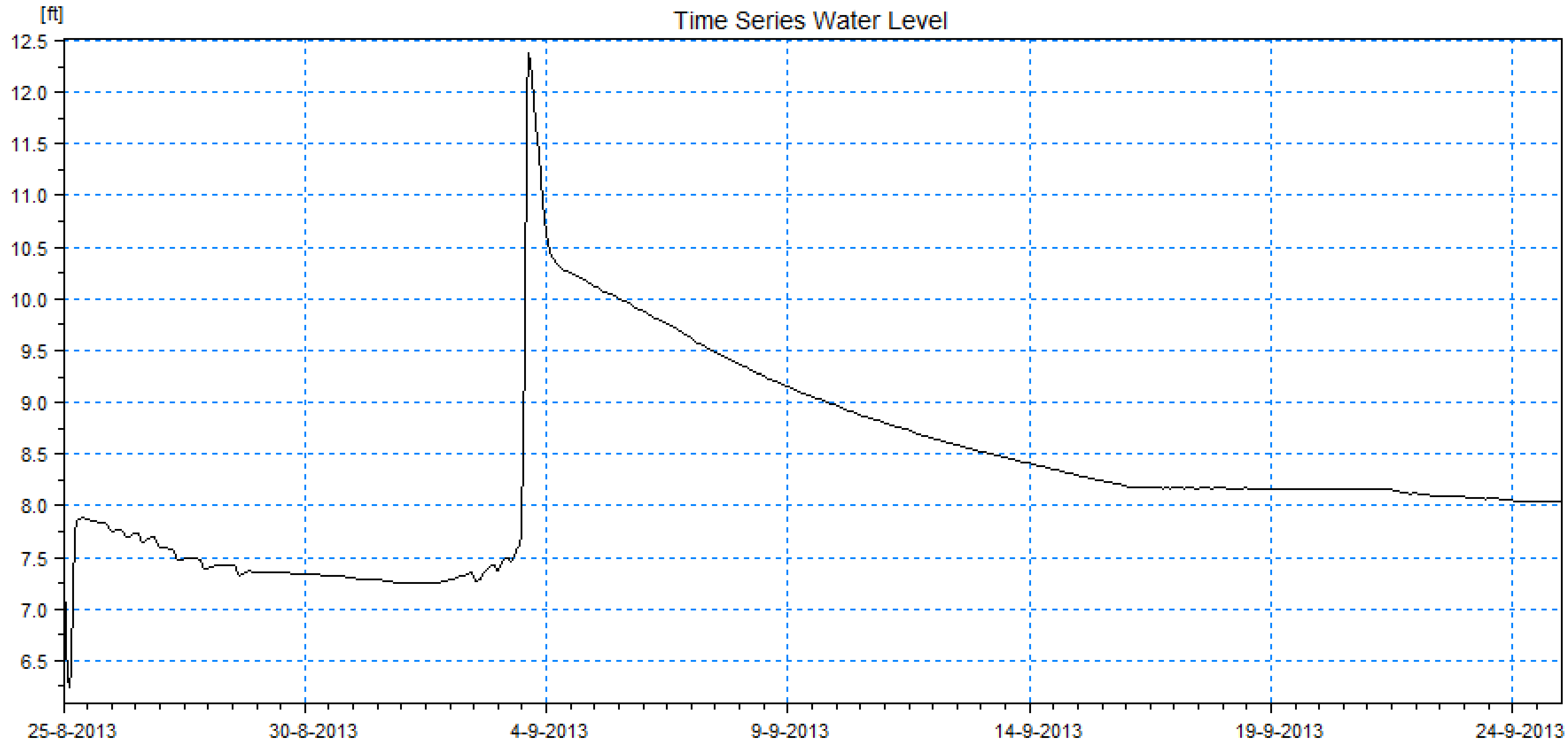

Figure 35. Miller3 Tailwater Stage (ft)

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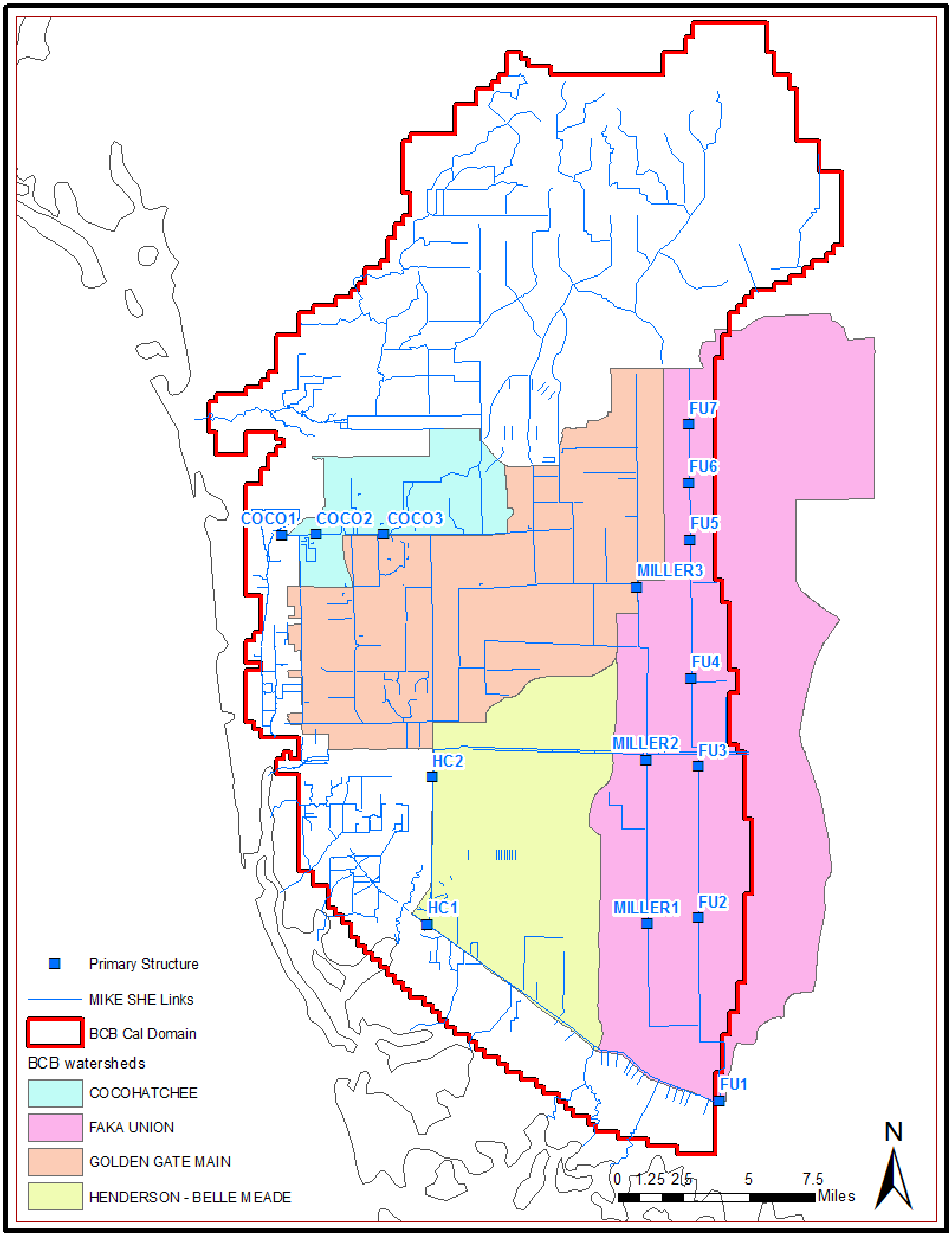

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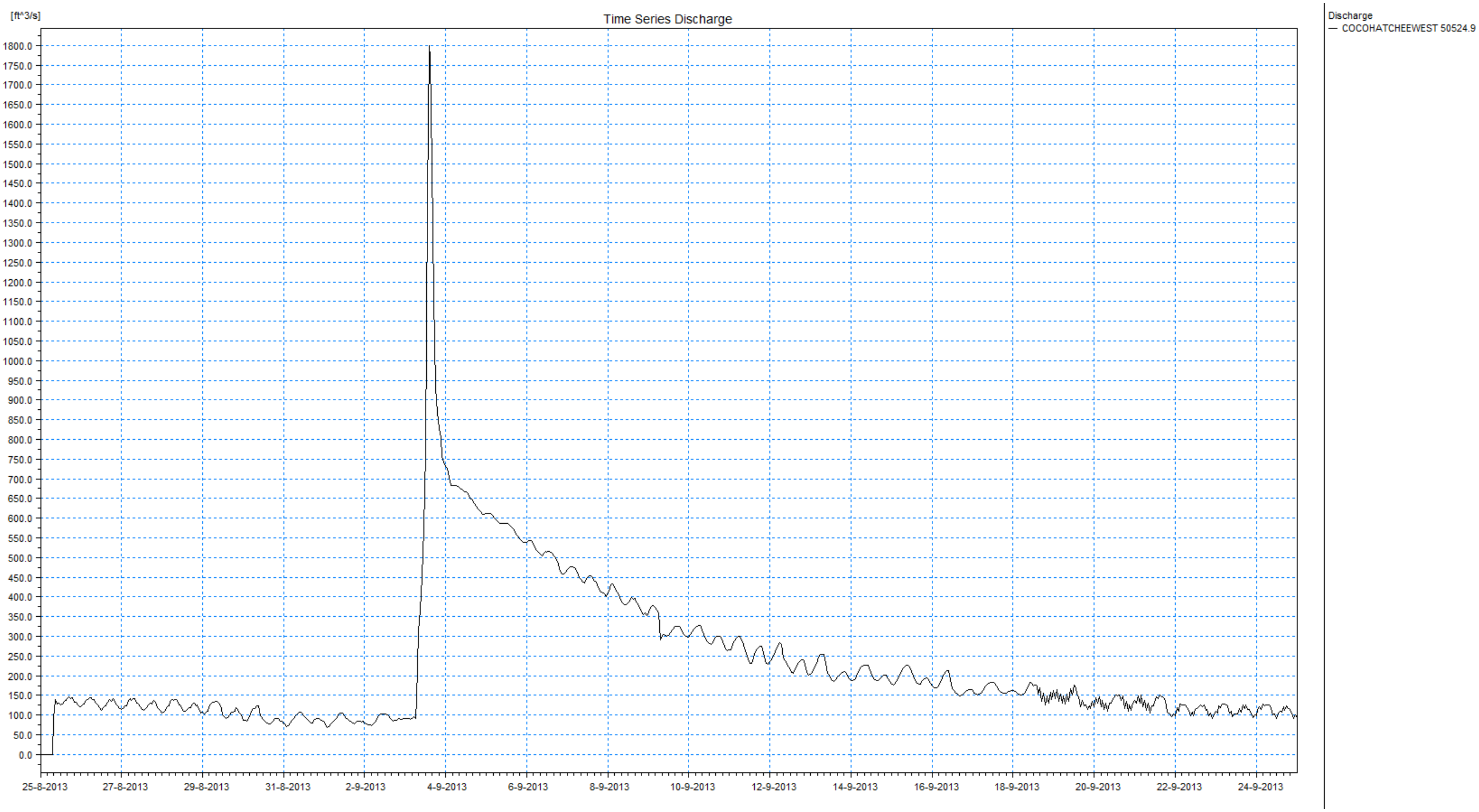

Figure 2. COCO1 Discharge (cfs)

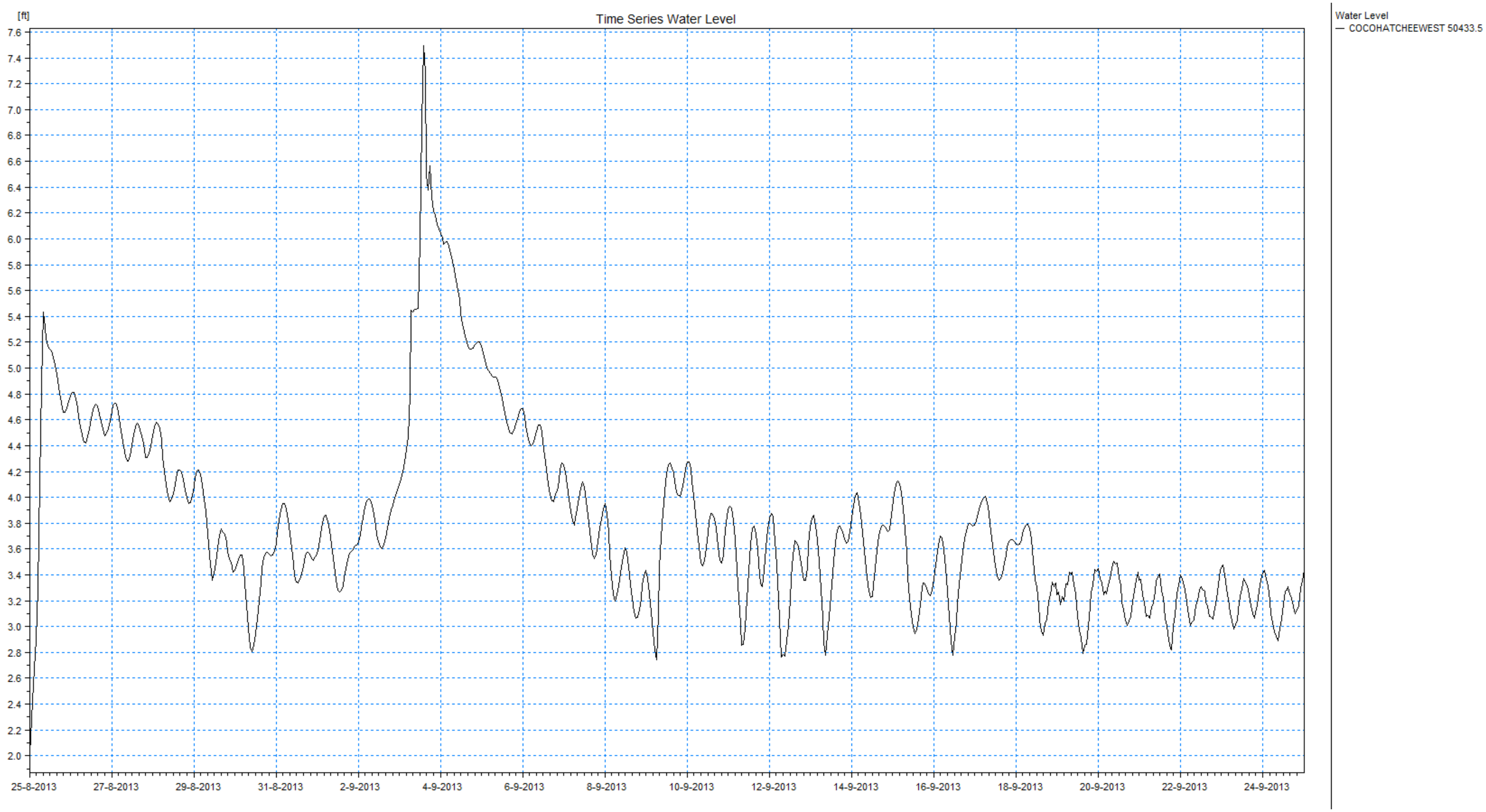

Figure 3. COCO1 Headwater Stage (ft)

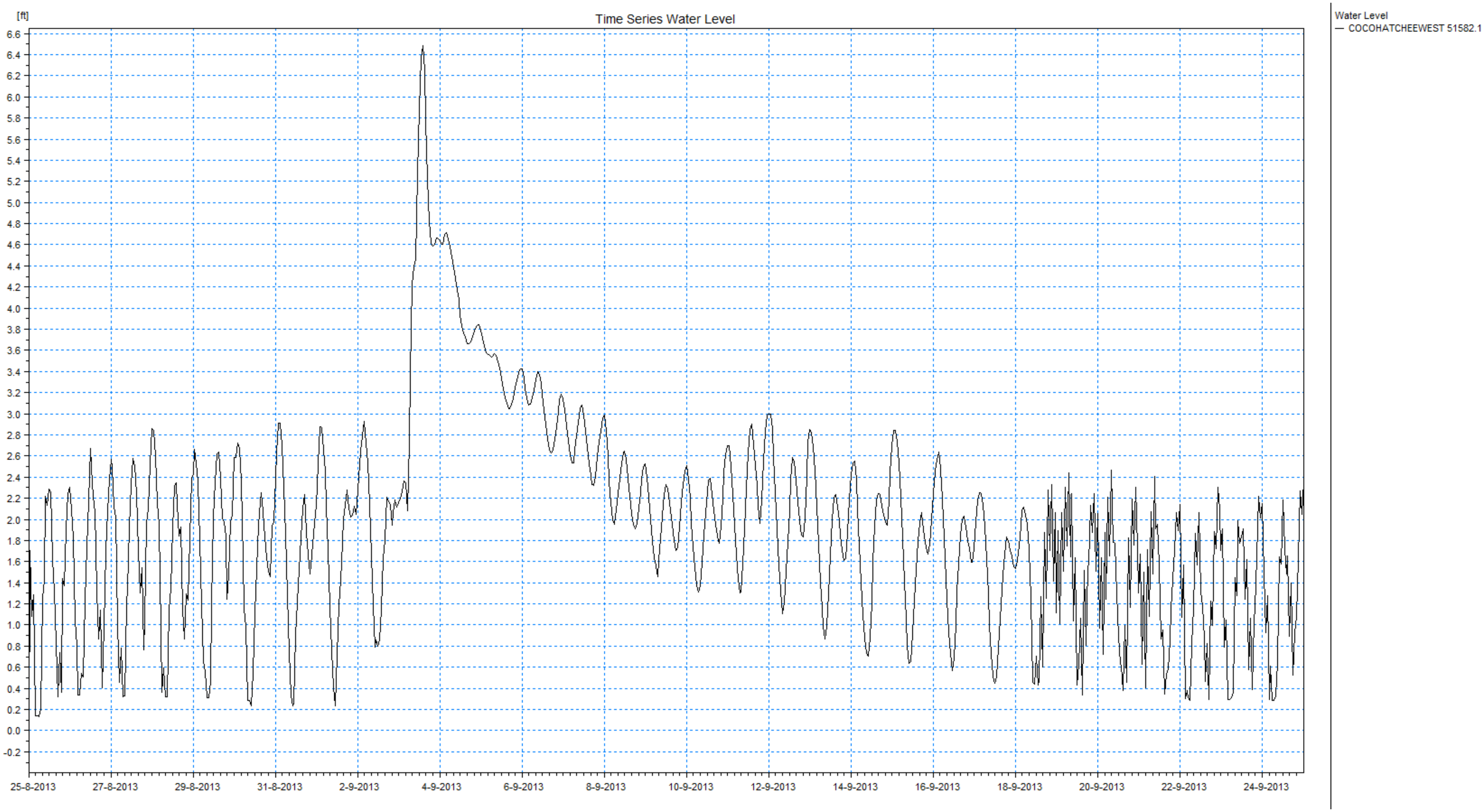

Figure 4. COCO1 Tailwater Stage (ft)

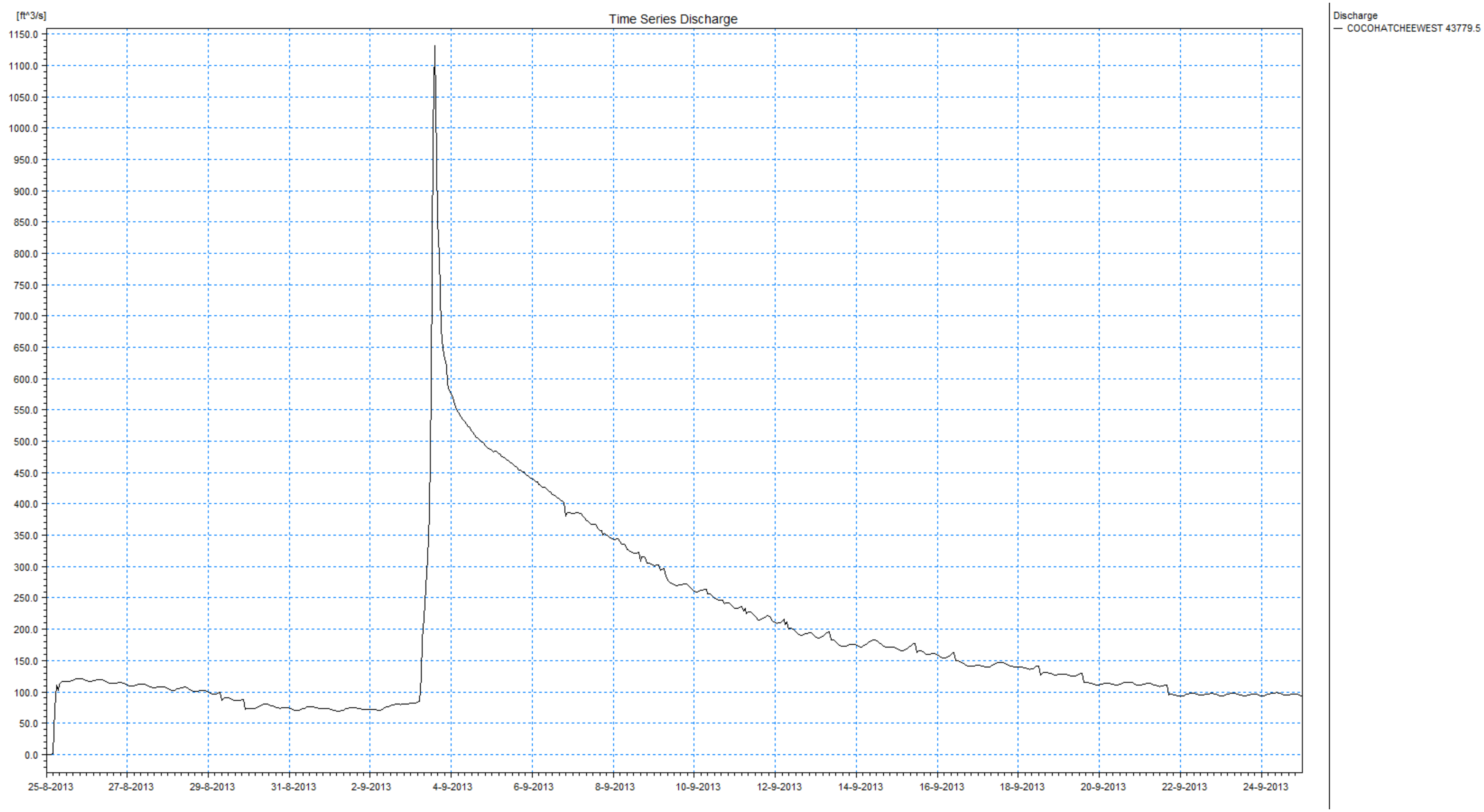

Figure 5. COCO2 Discharge (cfs)

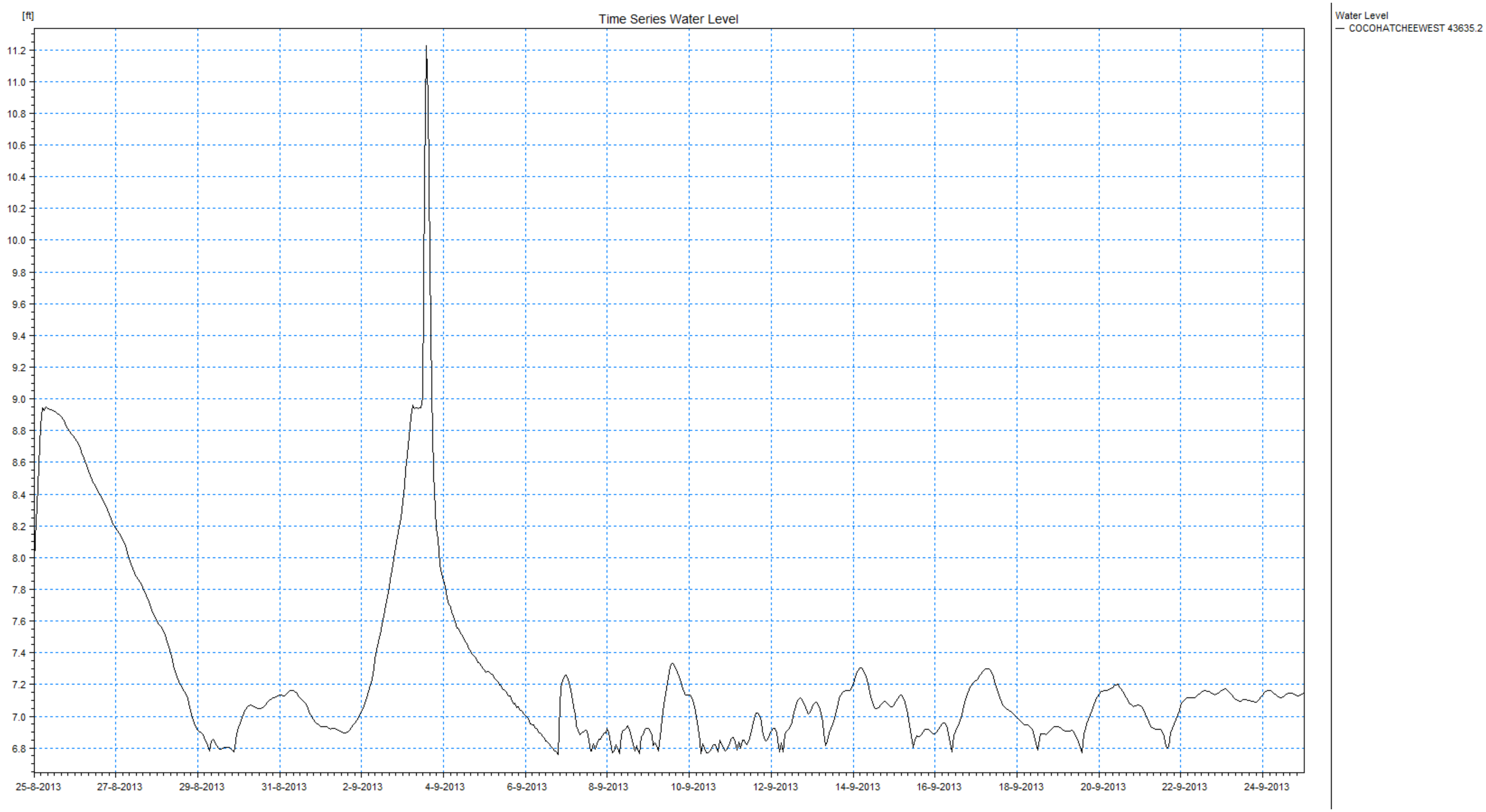

Figure 6. COCO2 Headwater Stage (ft)

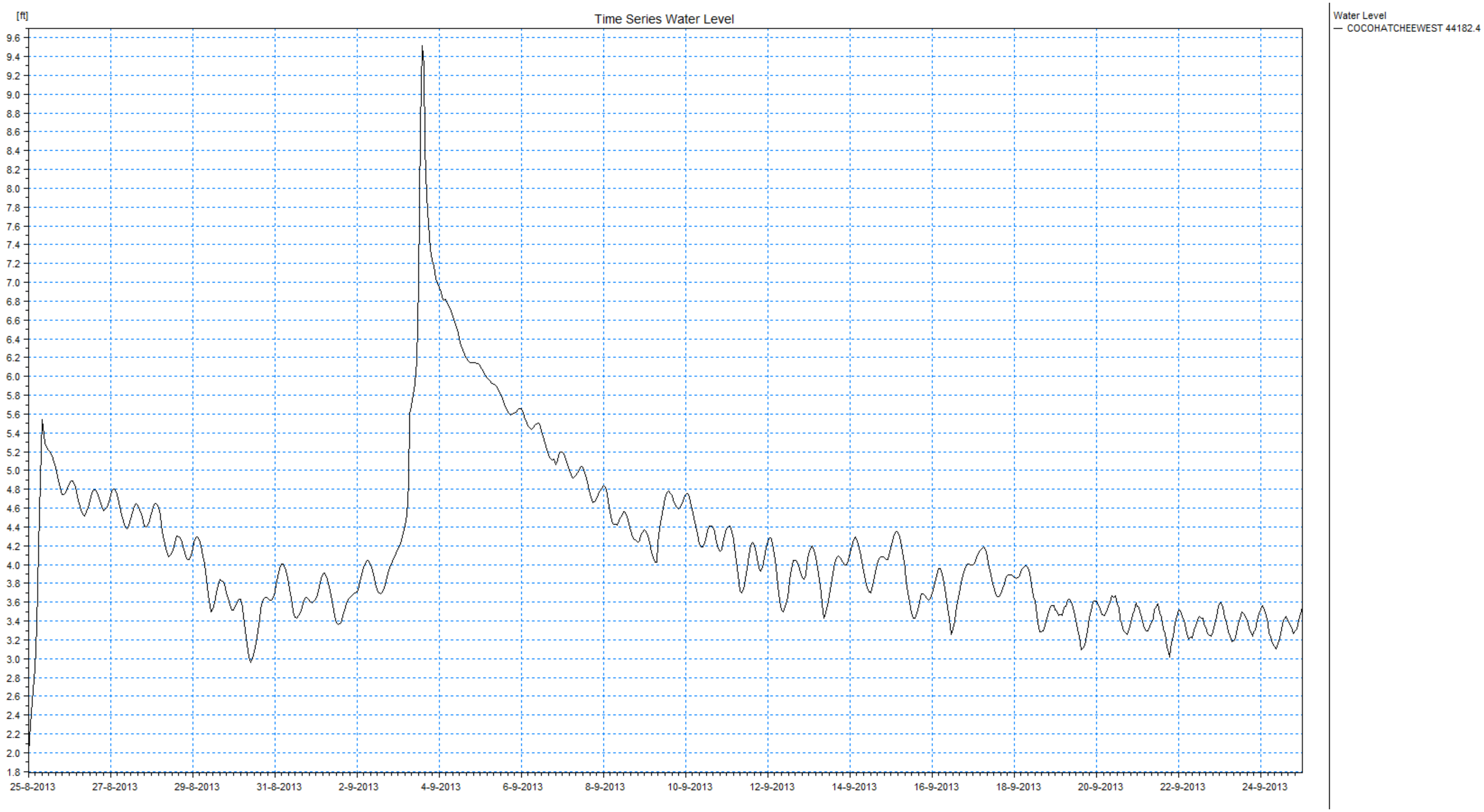

Figure 7. COCO2 Tailwater Stage (ft)

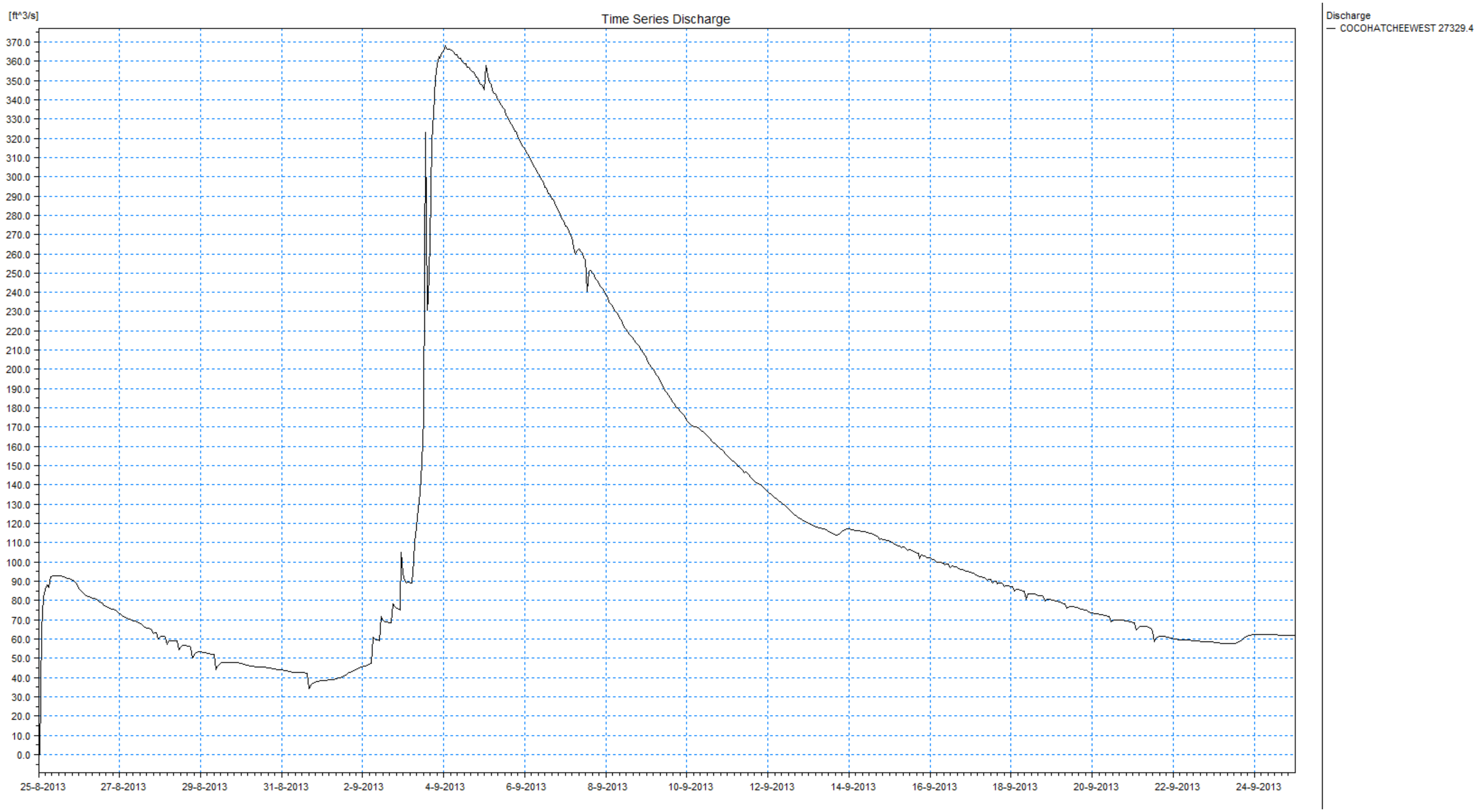

Figure 8. COCO3 Discharge (cfs)

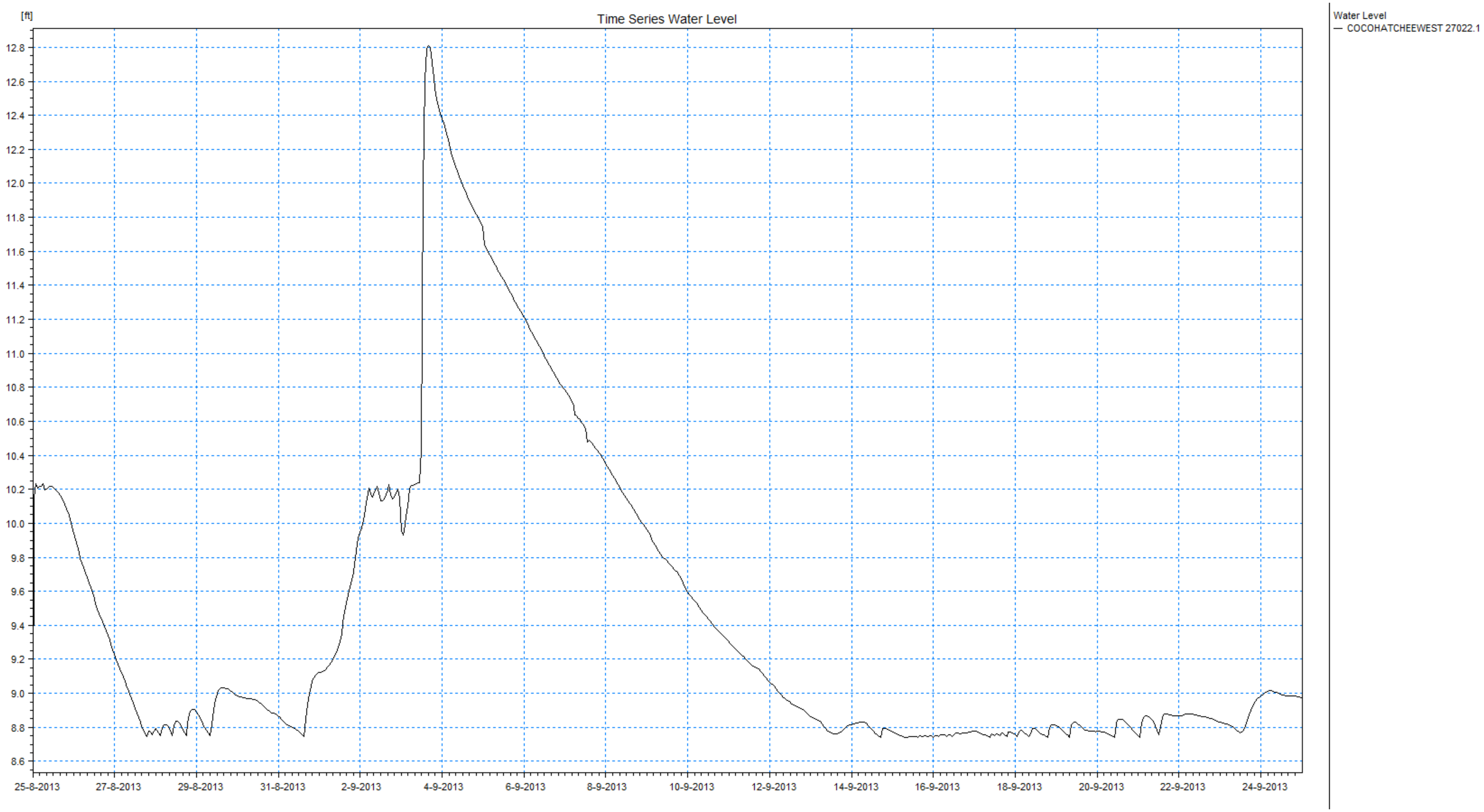

Figure 9. COCO3 Headwater Stage (ft)

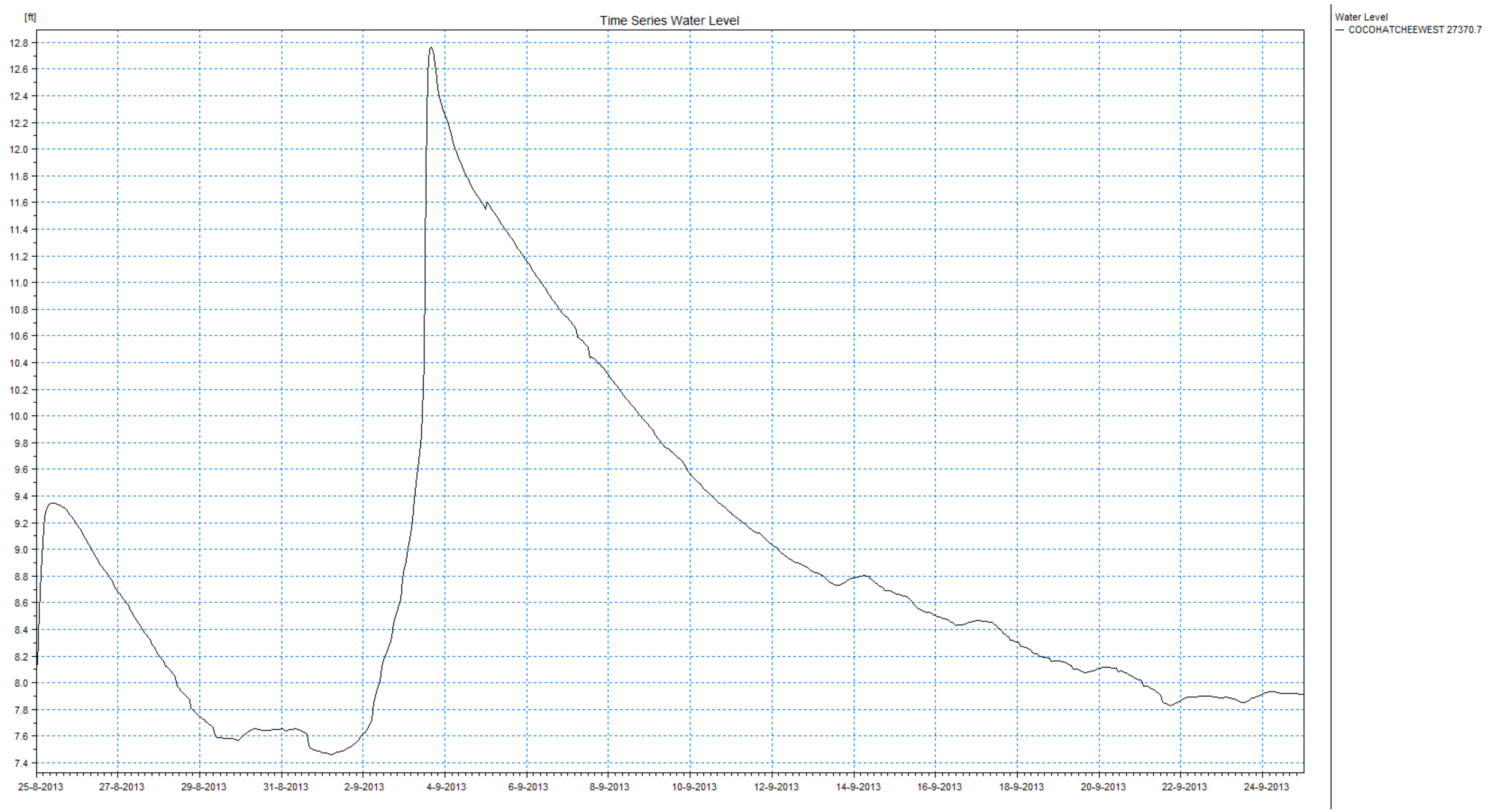

Figure 10. COCO3 Tailwater Stage (ft)

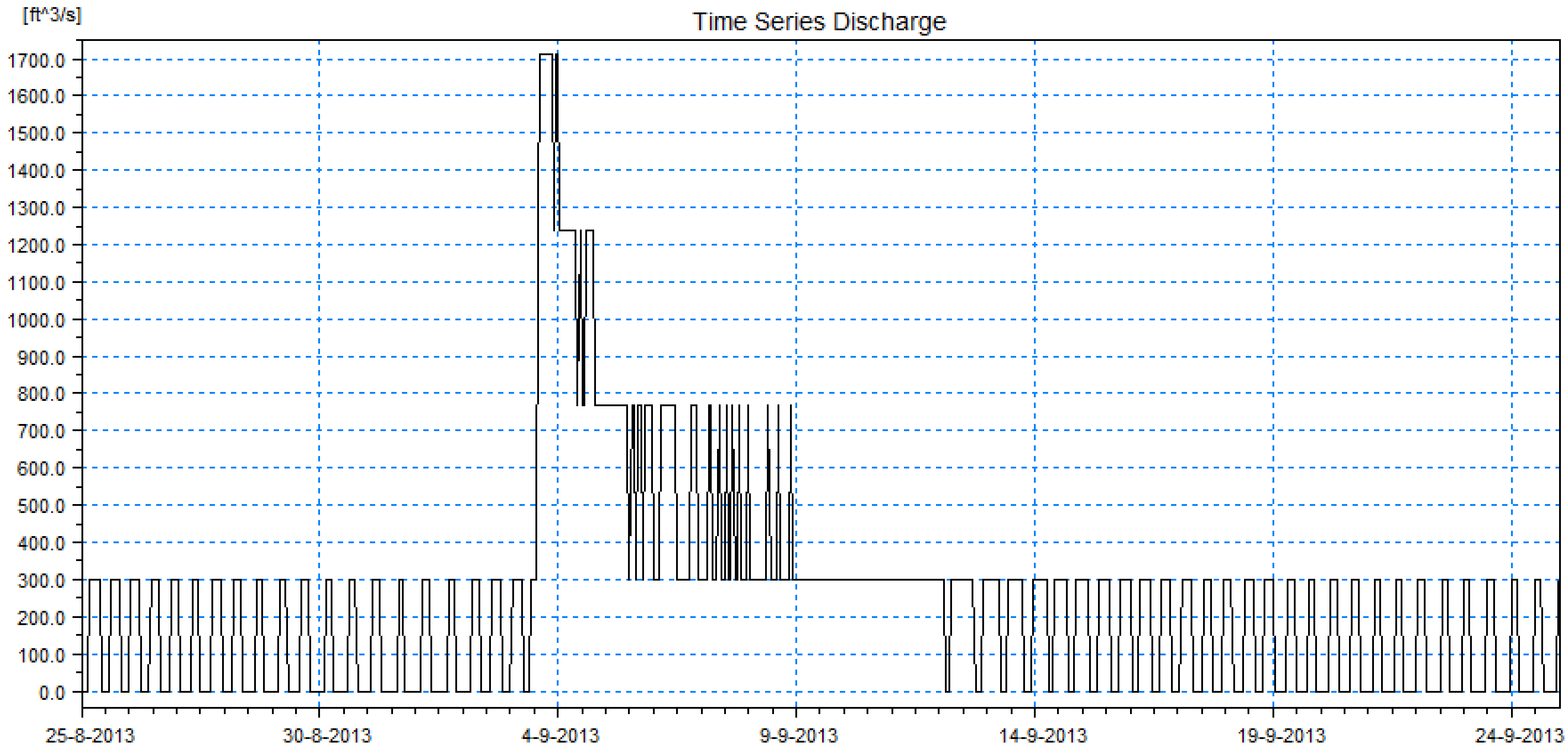

Figure 11. FU Pump Discharge (cfs)

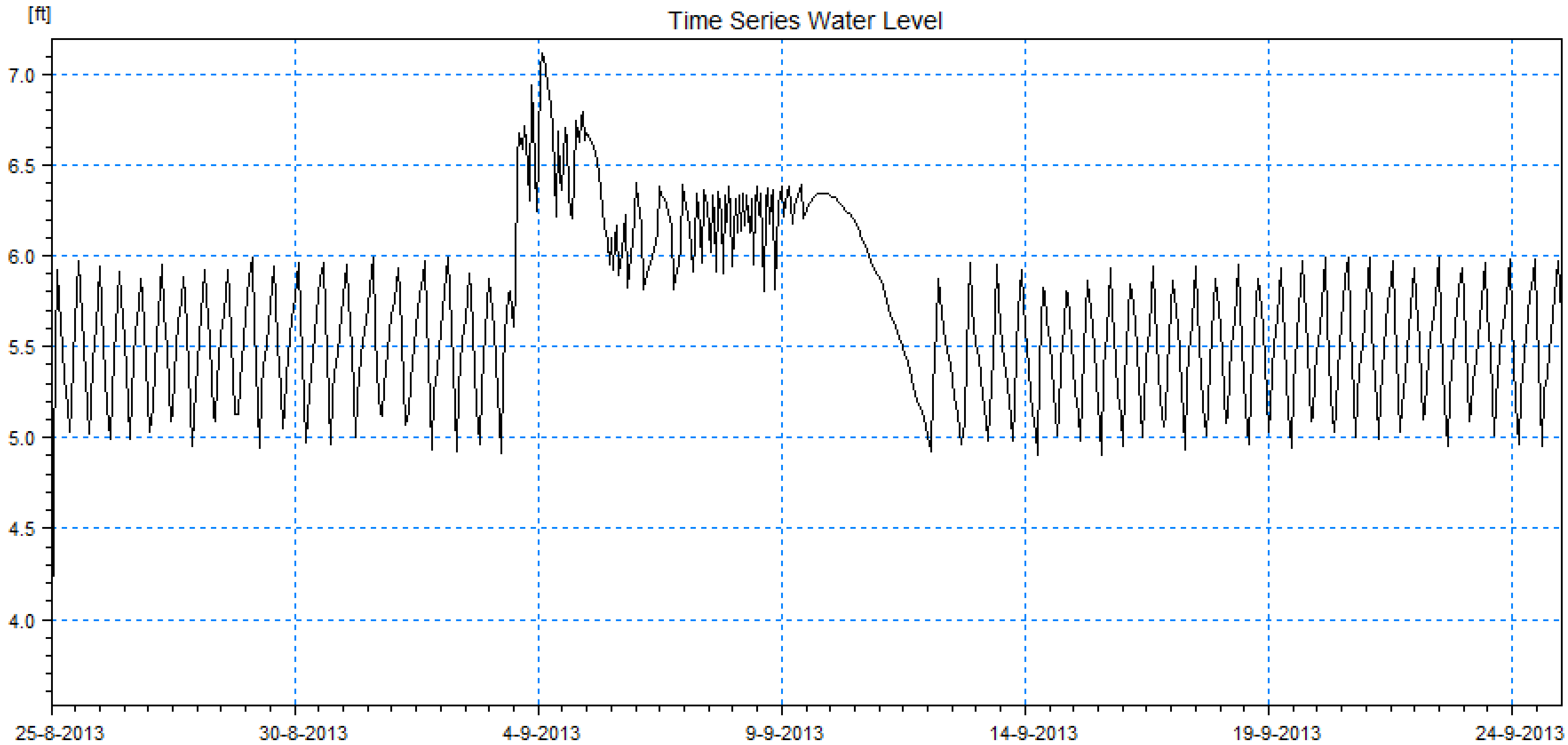

Figure 12. FU Pump Headwater Stage (ft)

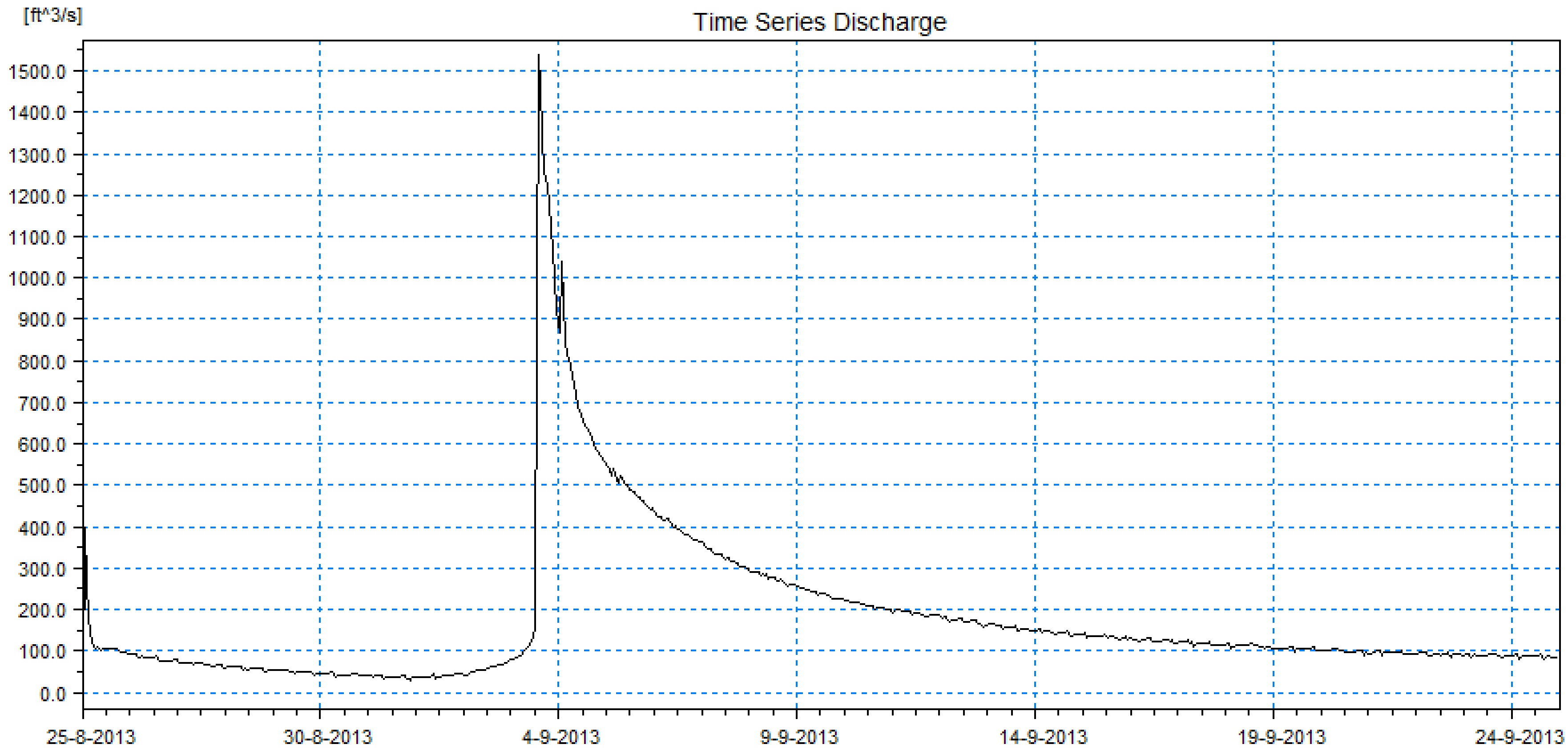

Figure 13. FU4 Discharge (cfs)

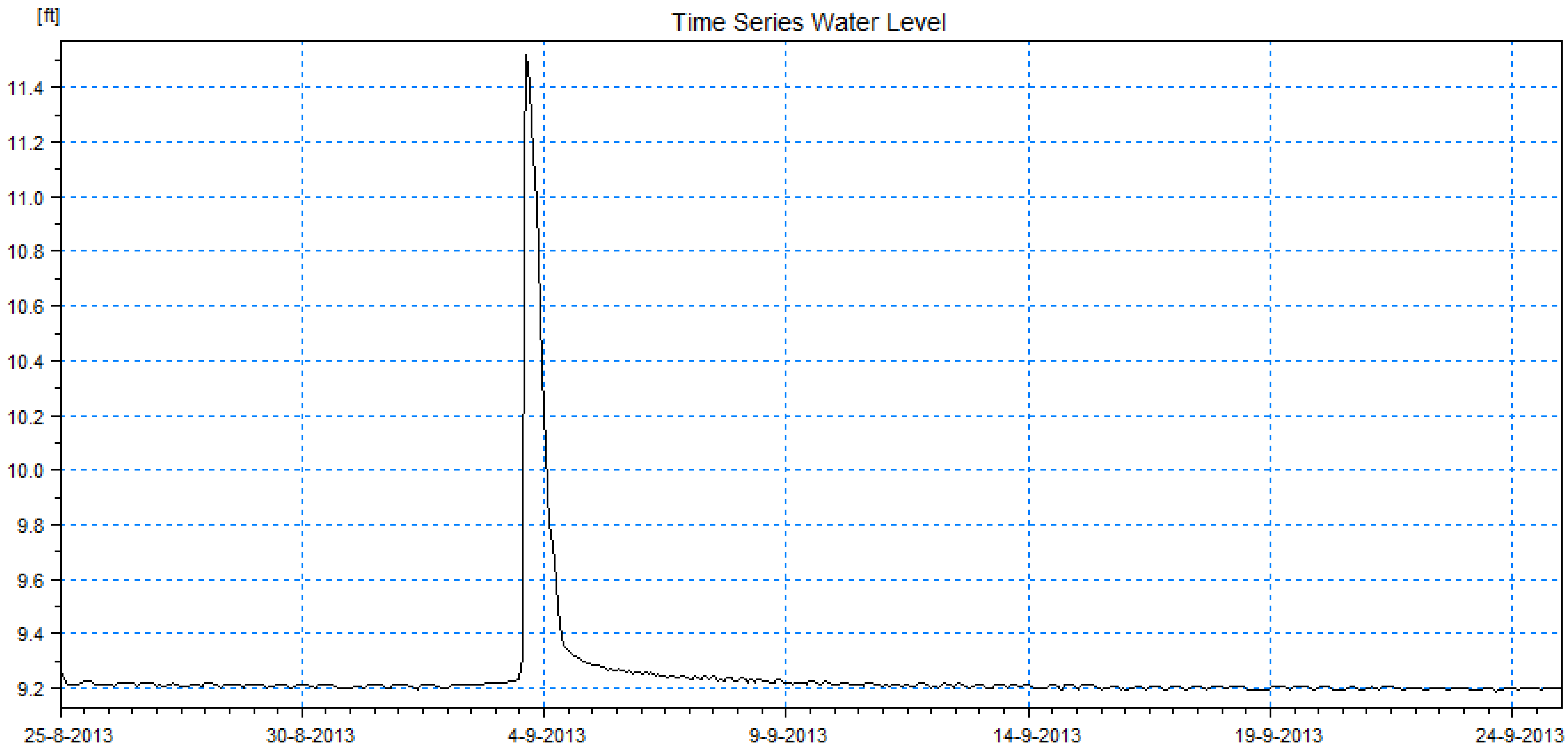

Figure 14. FU4 Headwater Stage (ft)

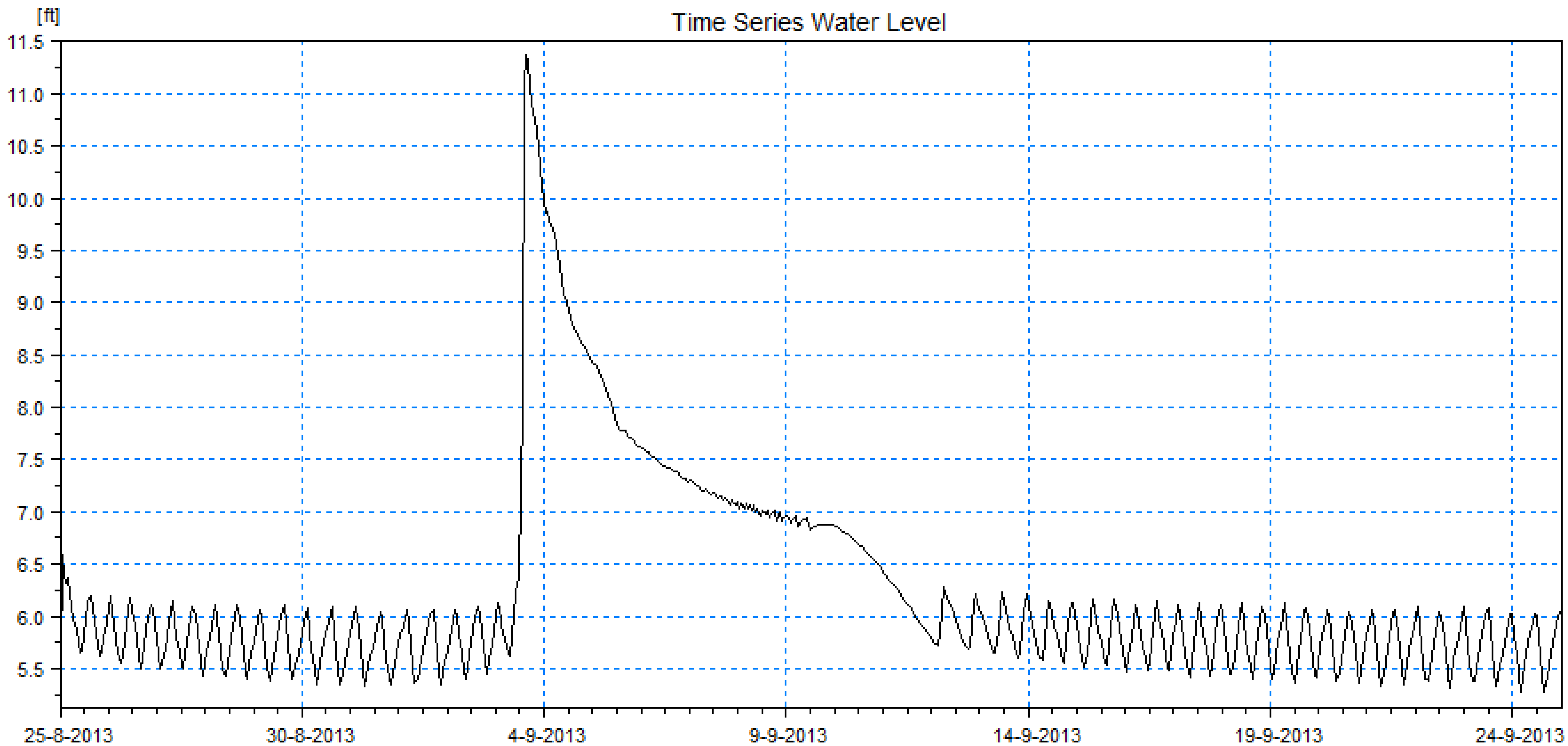

Figure 15. FU4 Tailwater Stage (ft)

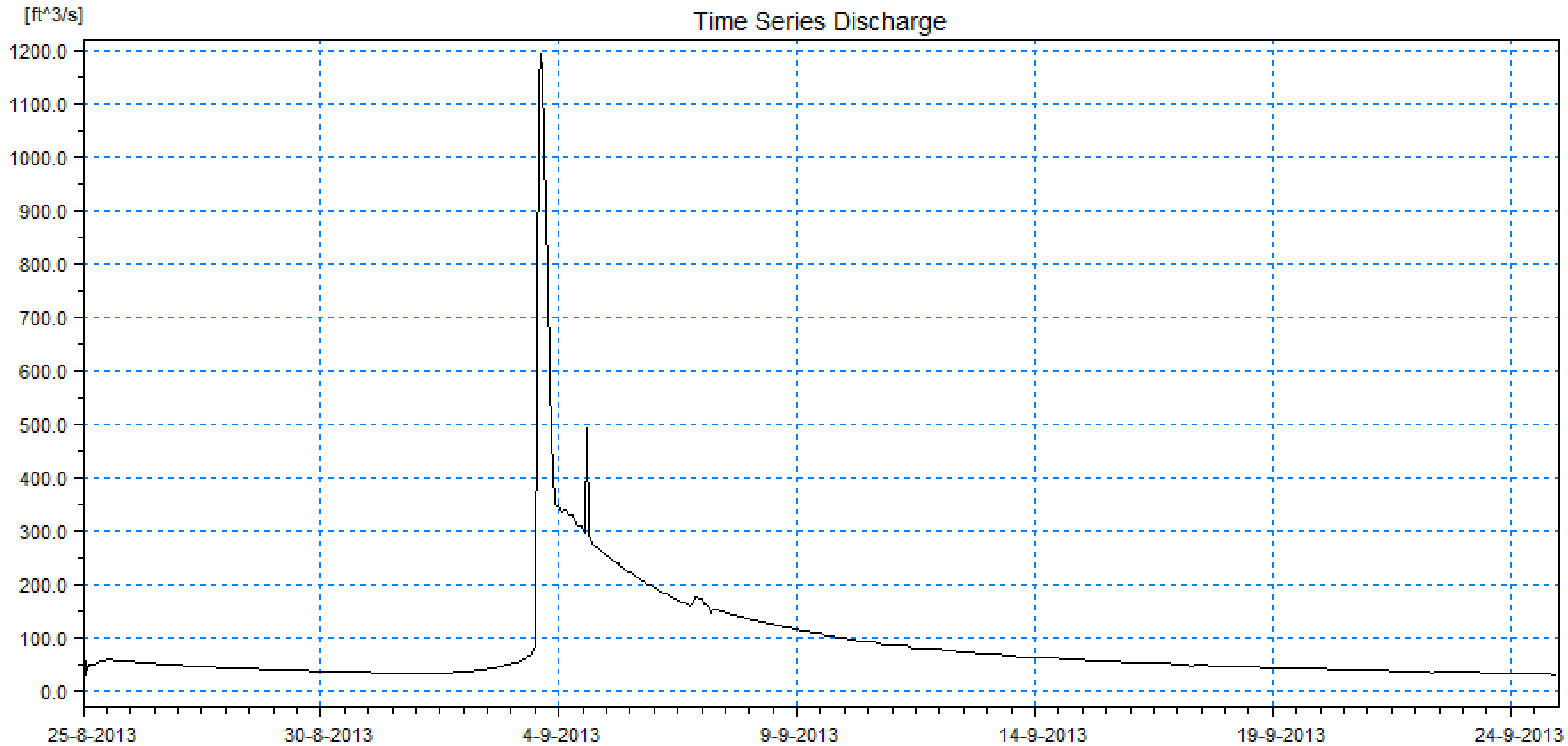

Figure 16. FU5 Discharge (cfs)

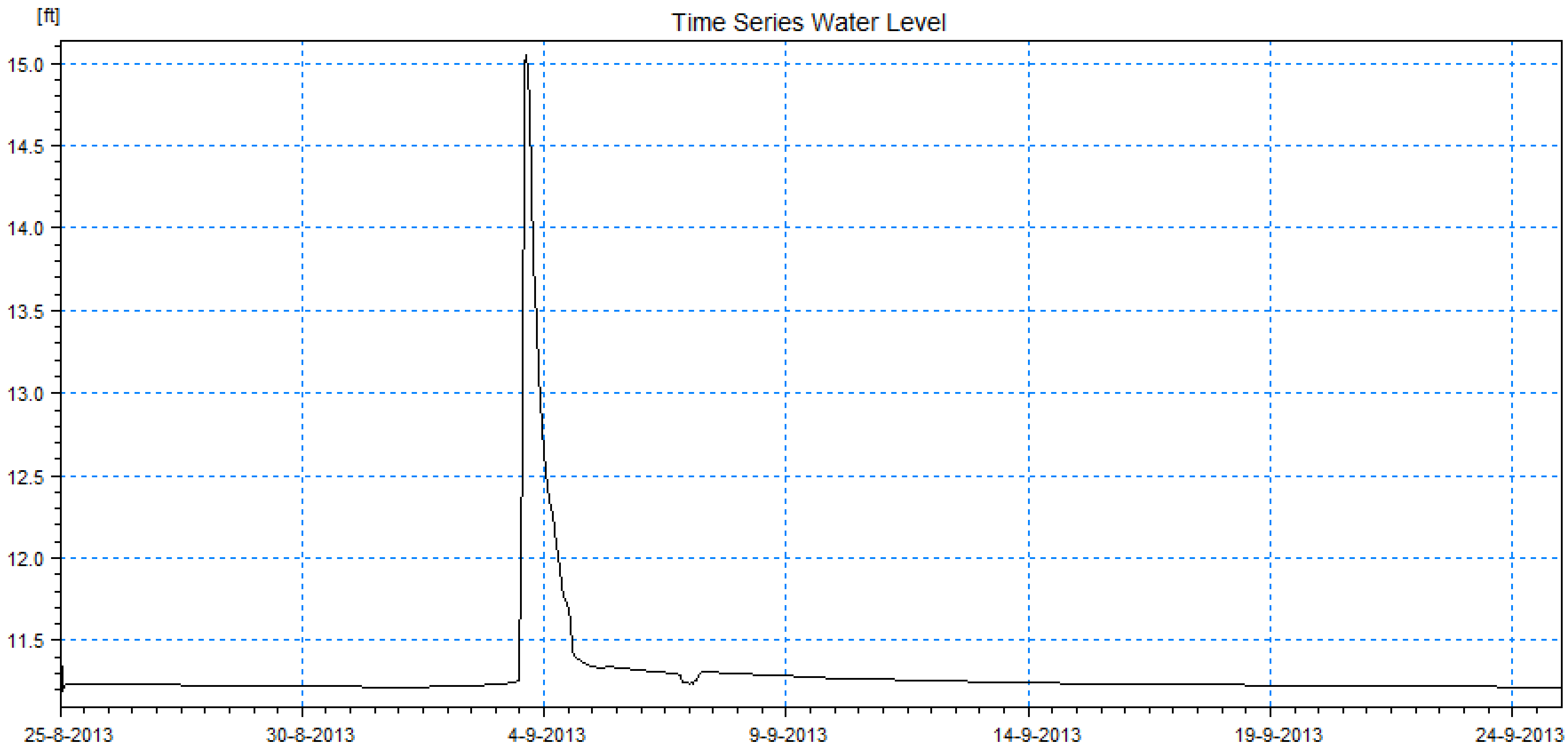

Figure 17. FU5 Headwater Stage (ft)

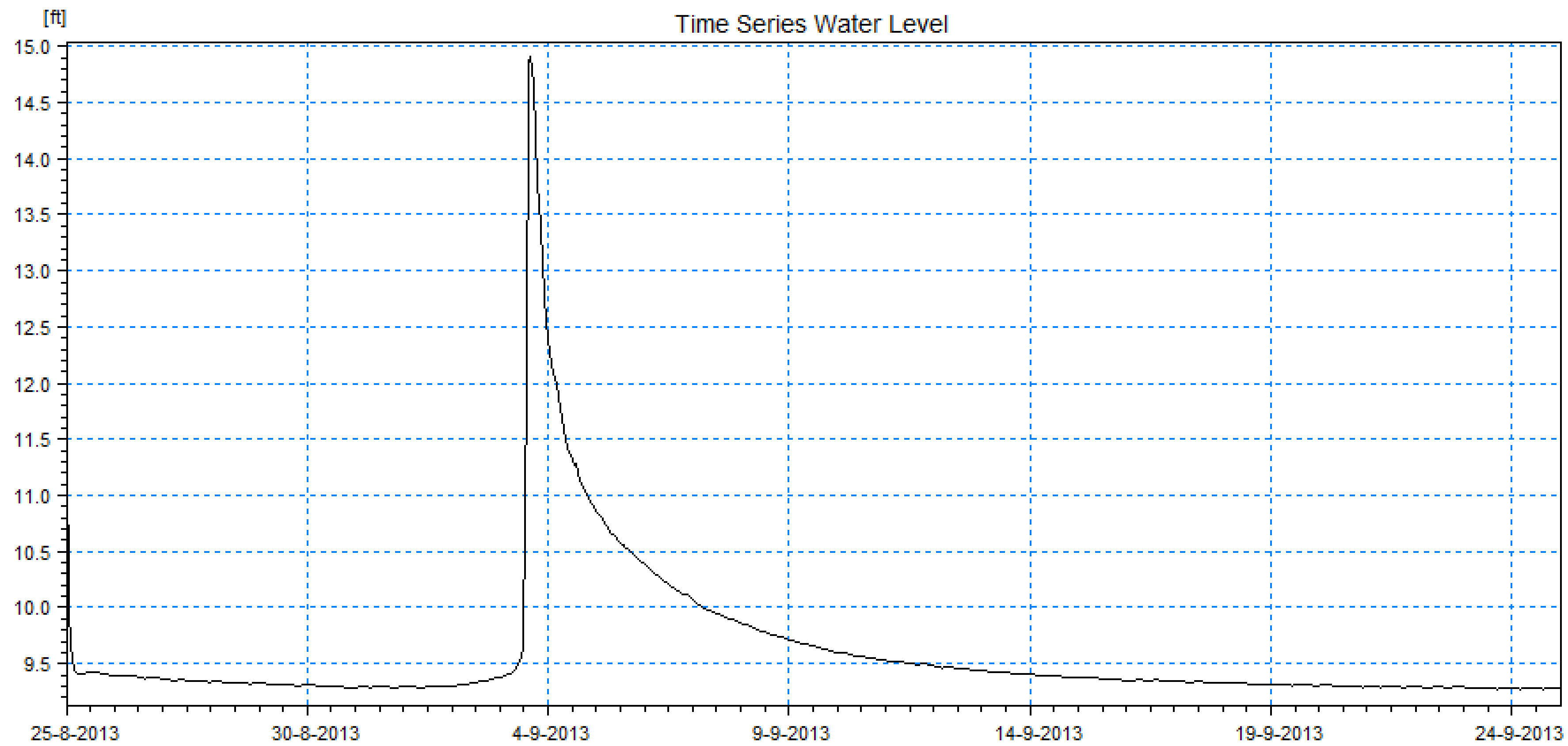

Figure 18. FU5 Tailwater Stage (ft)

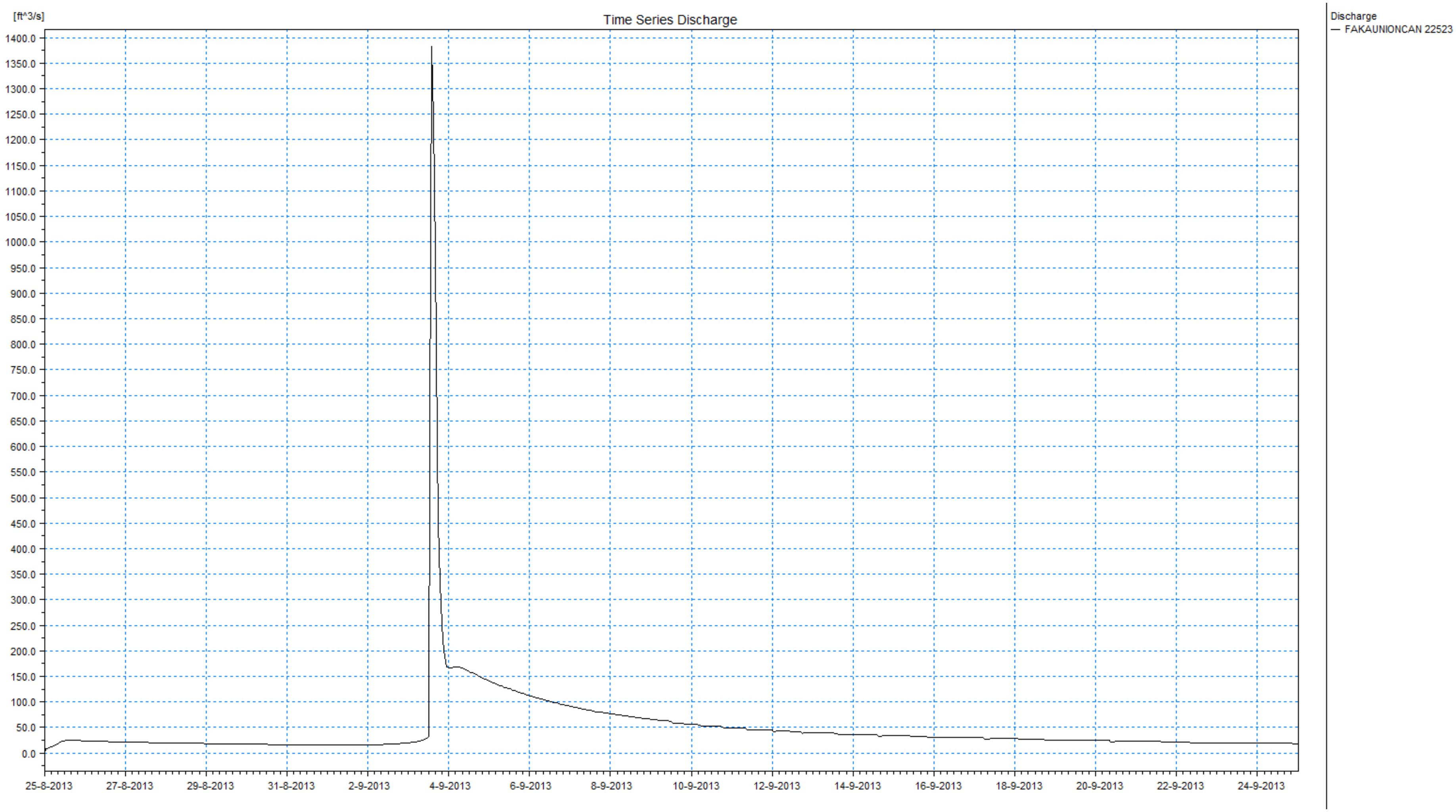

Figure 19. FU6 Discharge (cfs)

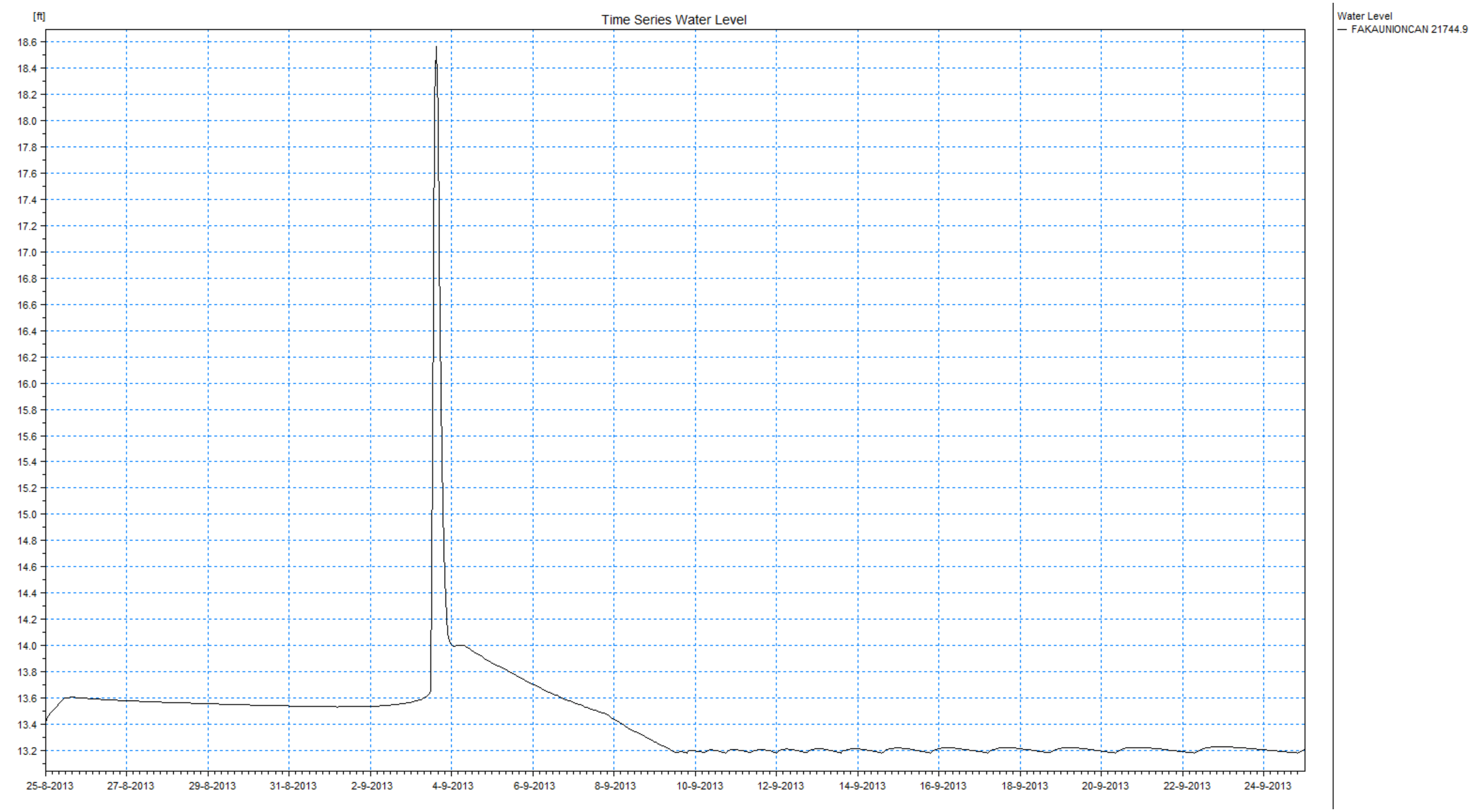

Figure 20. FU6 Headwater Stage (ft)

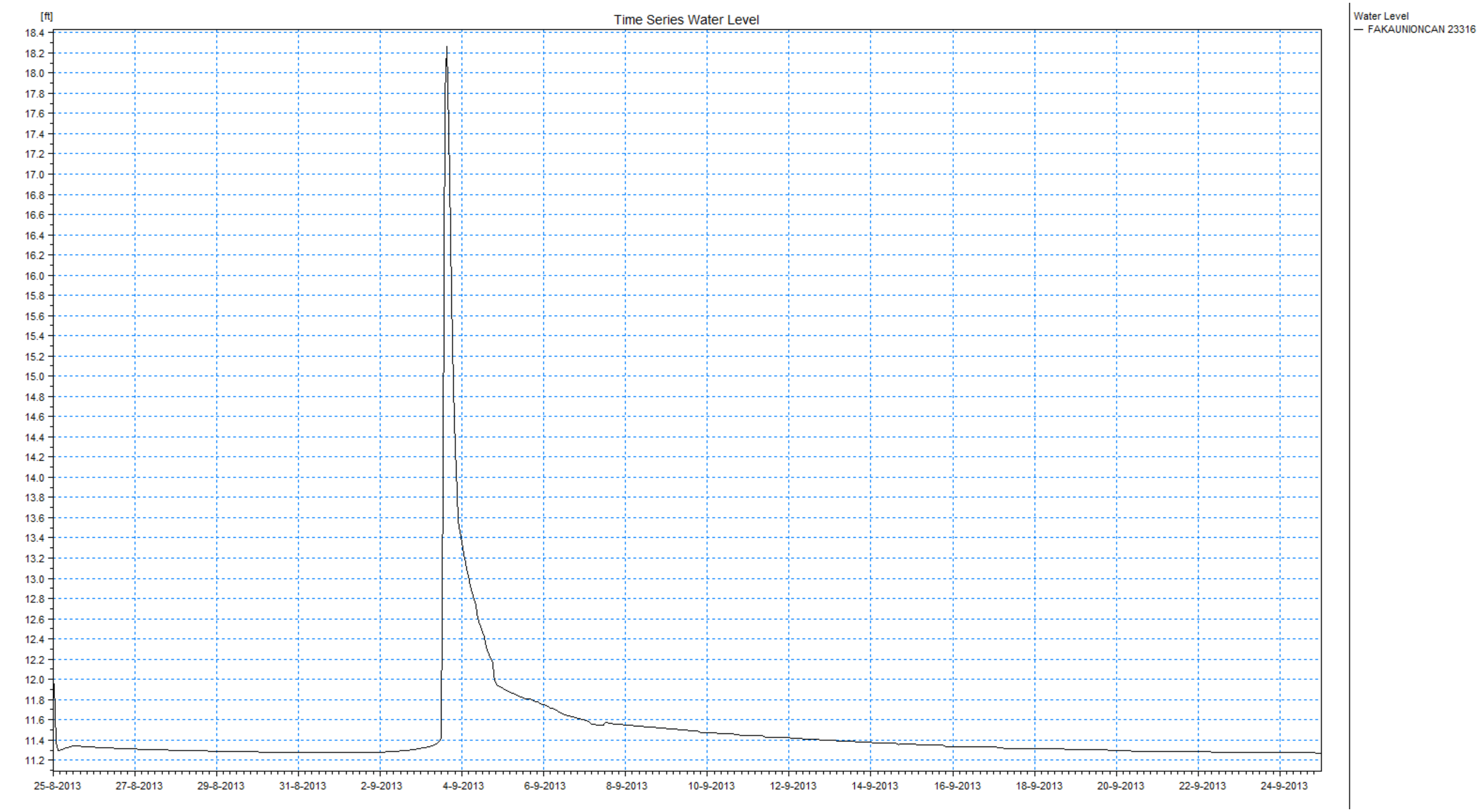

Figure 21. FU6 Tailwater Stage (ft)

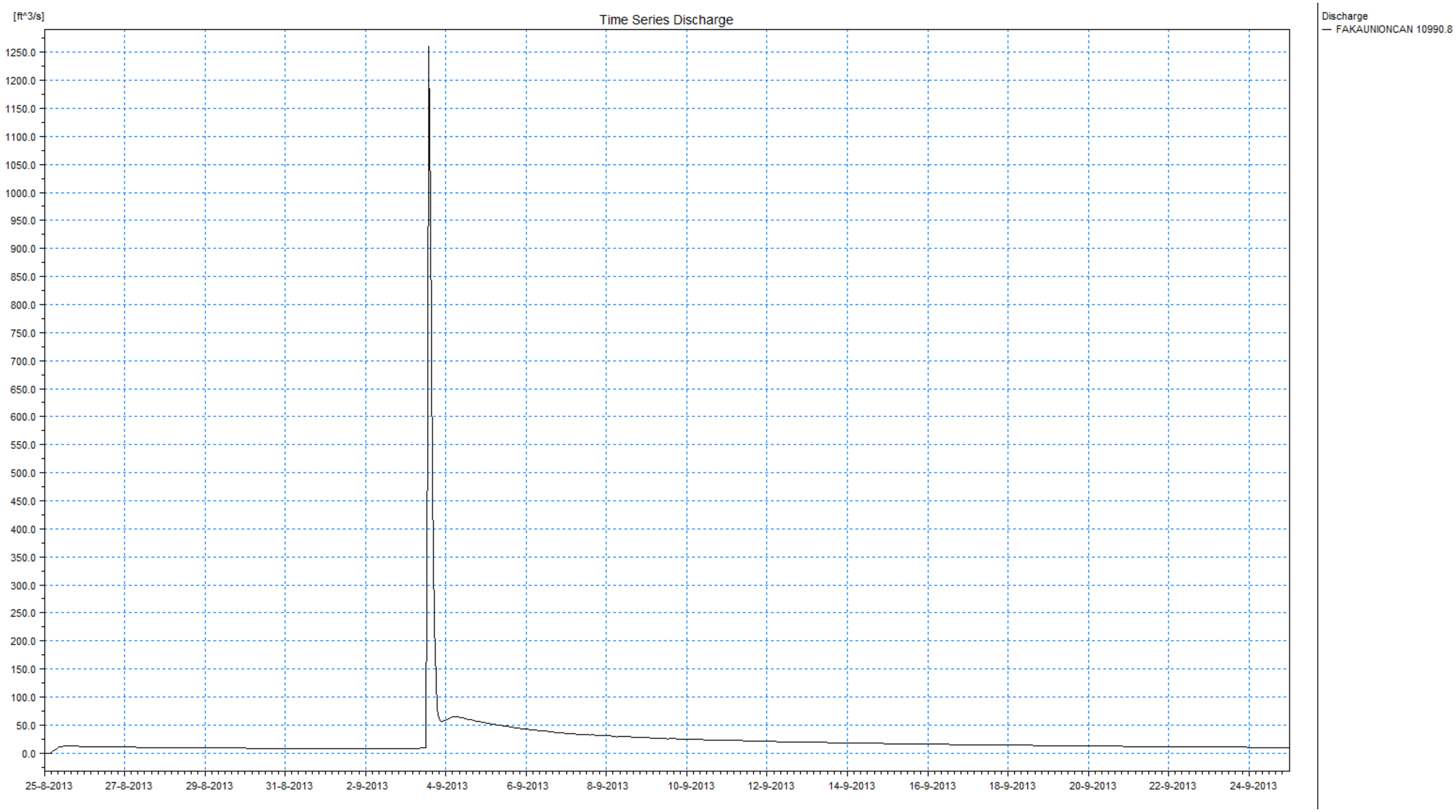

Figure 22. FU7 Discharge (cfs)

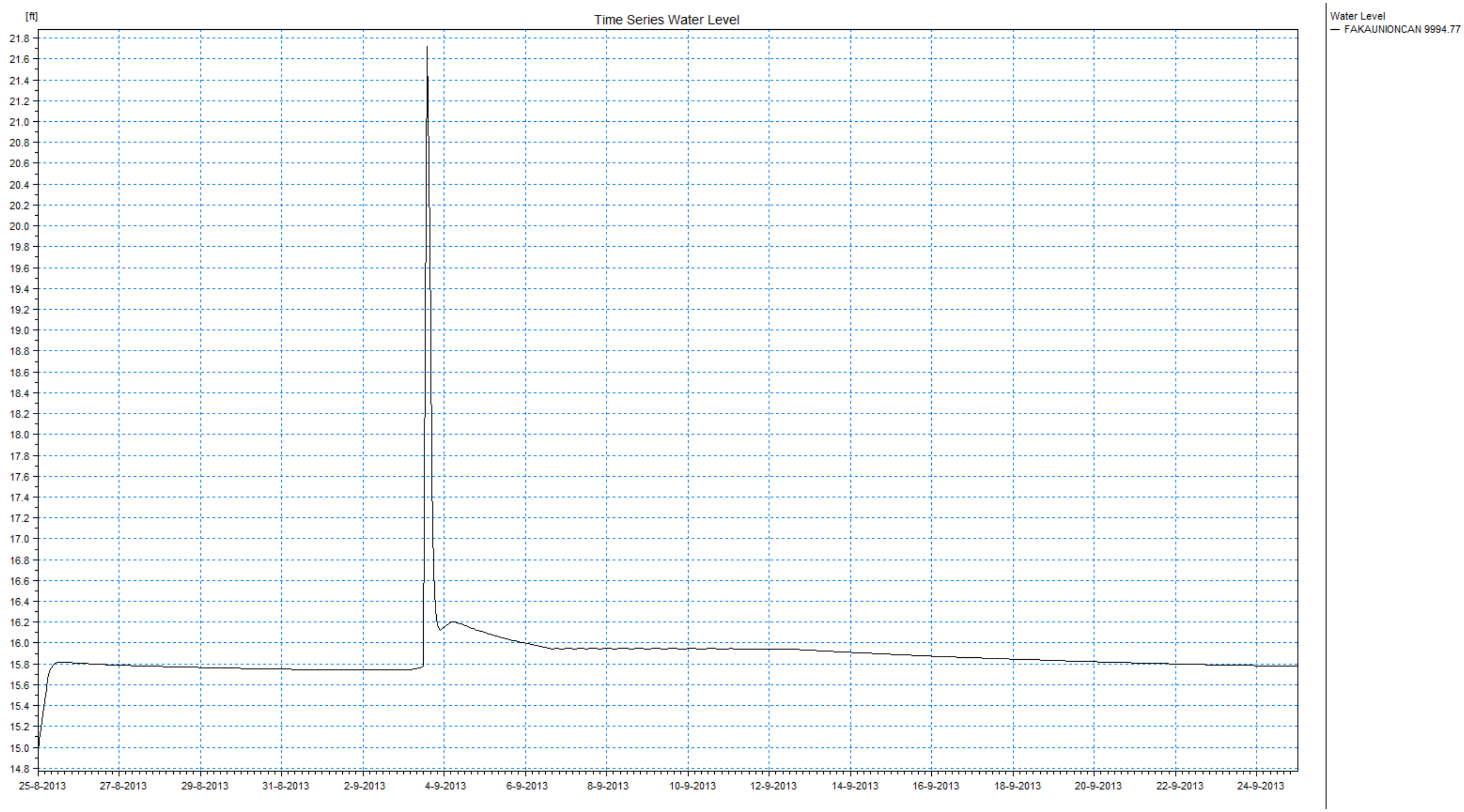

Figure 23. FU7 Headwater Stage (ft)

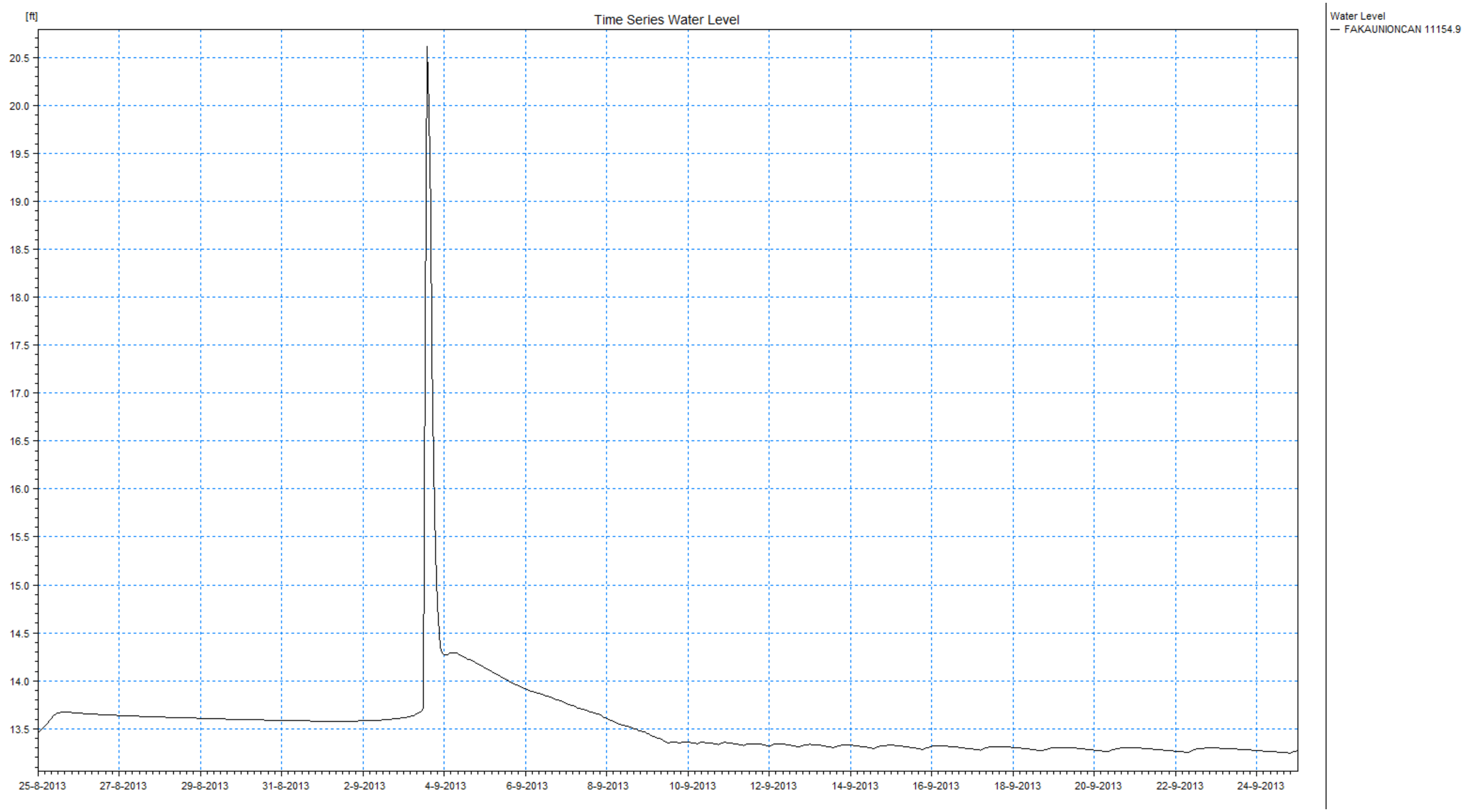

Figure 24. FU7 Tailwater Stage (ft)

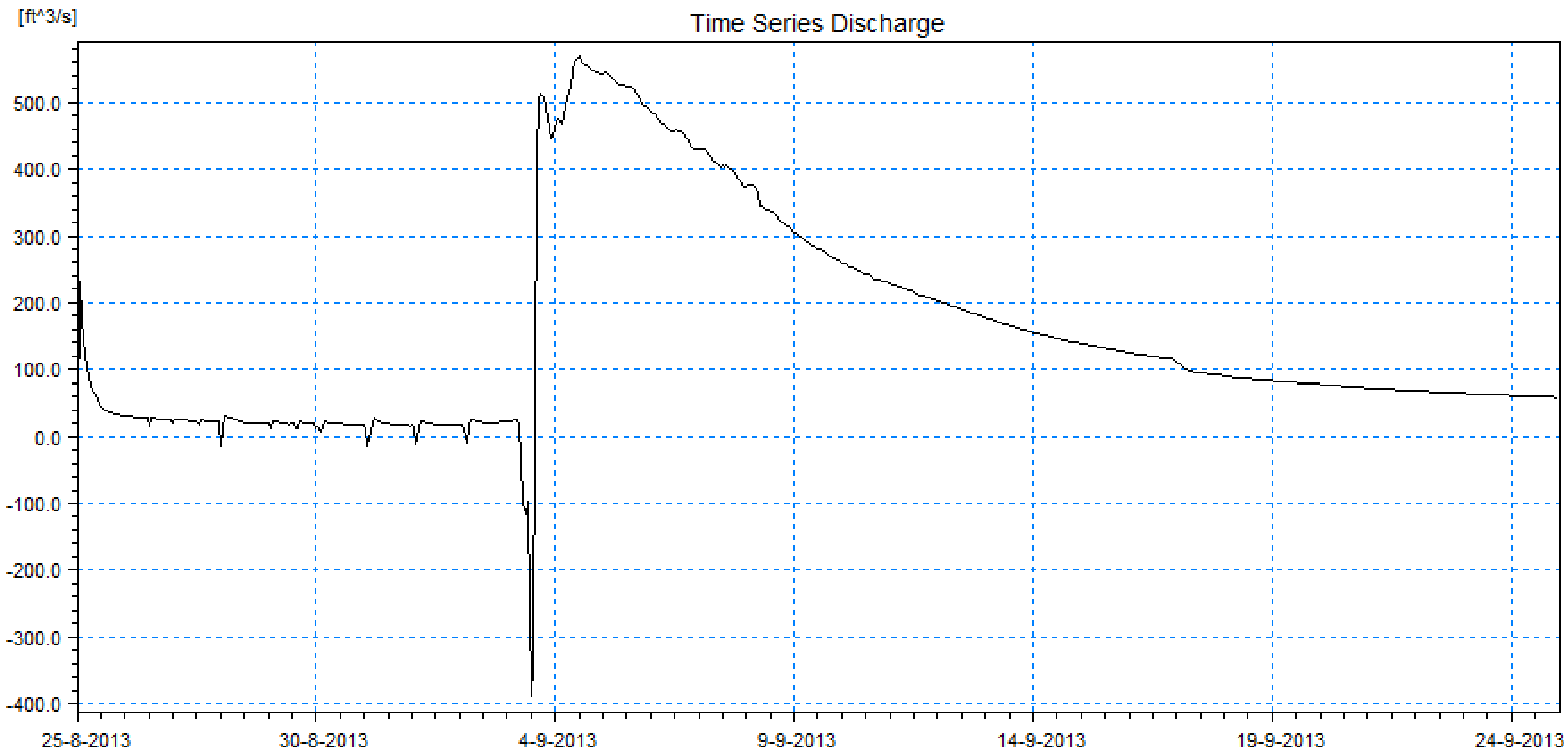

Figure 25. HC1 Discharge (cfs)

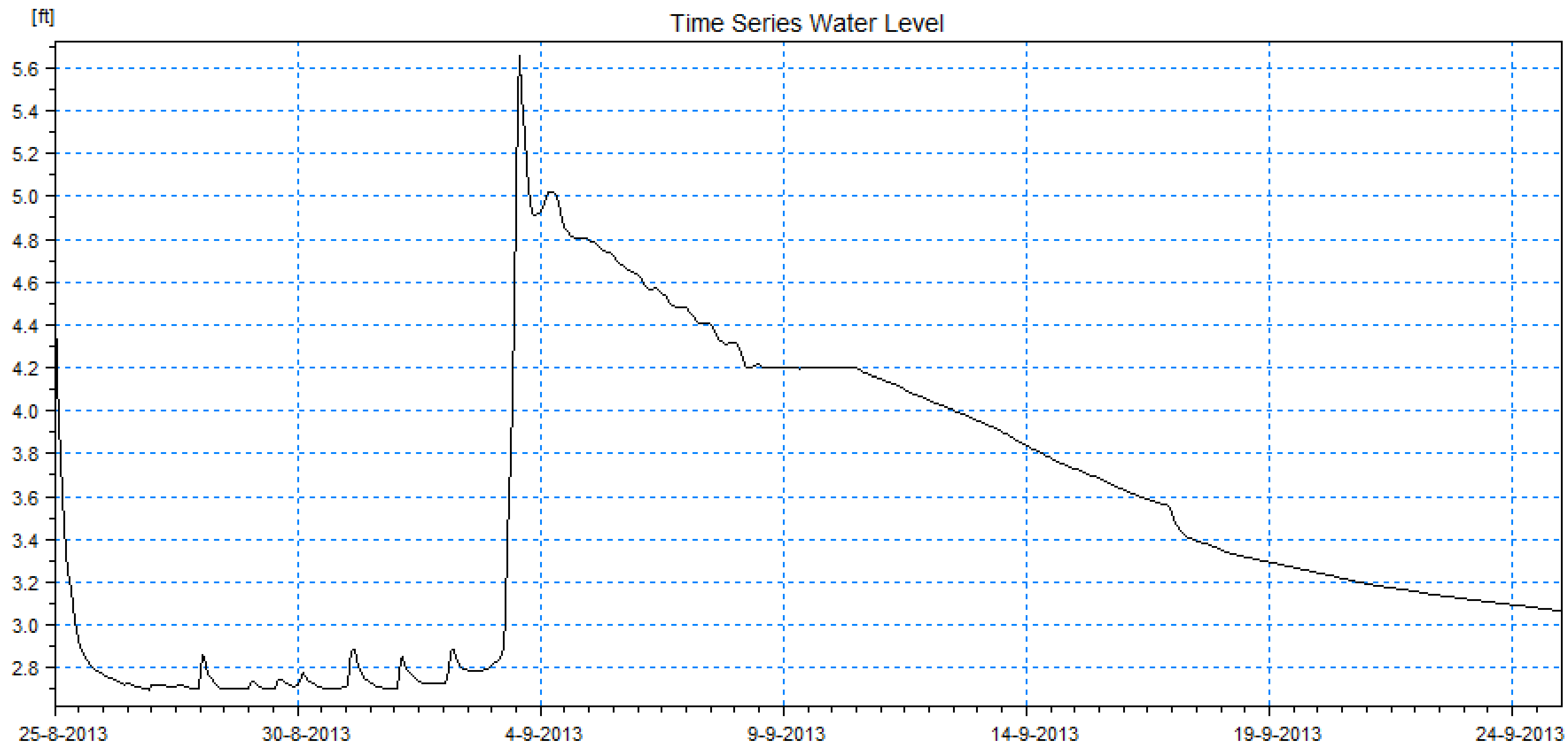

Figure 26. HC1 Headwater Stage (ft)

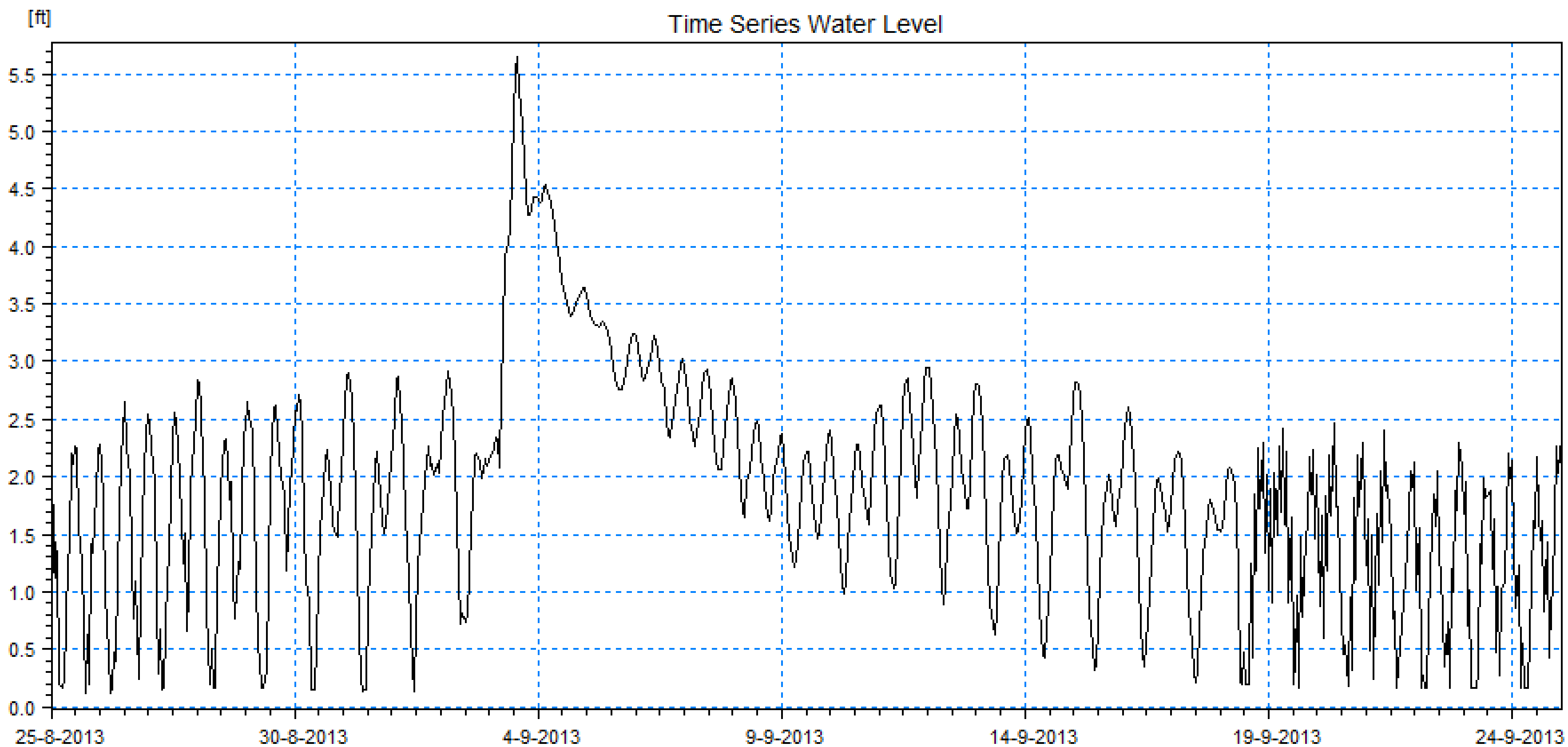

Figure 27. HC1 Tailwater Stage (ft)

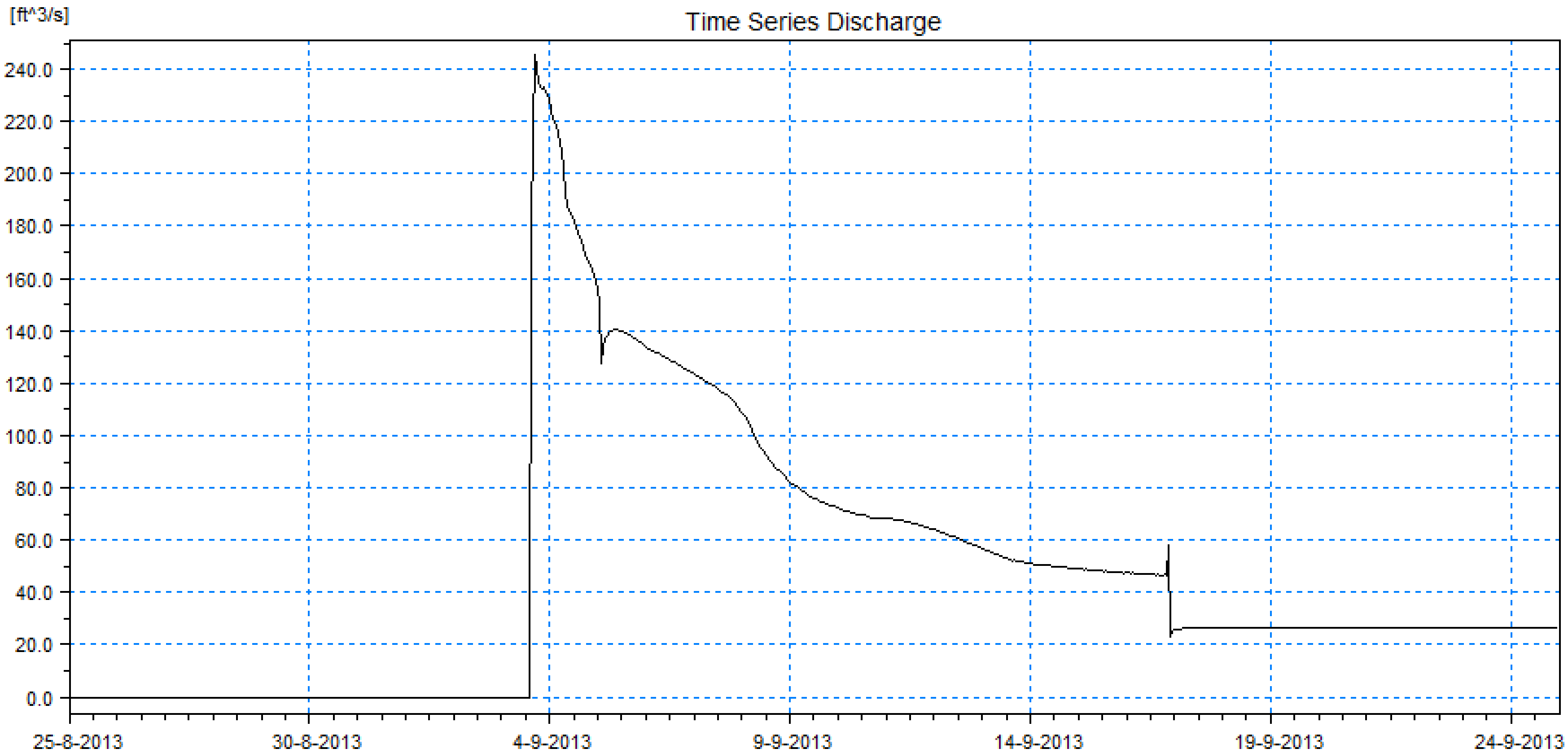

Figure 28. HC2 Discharge (cfs)

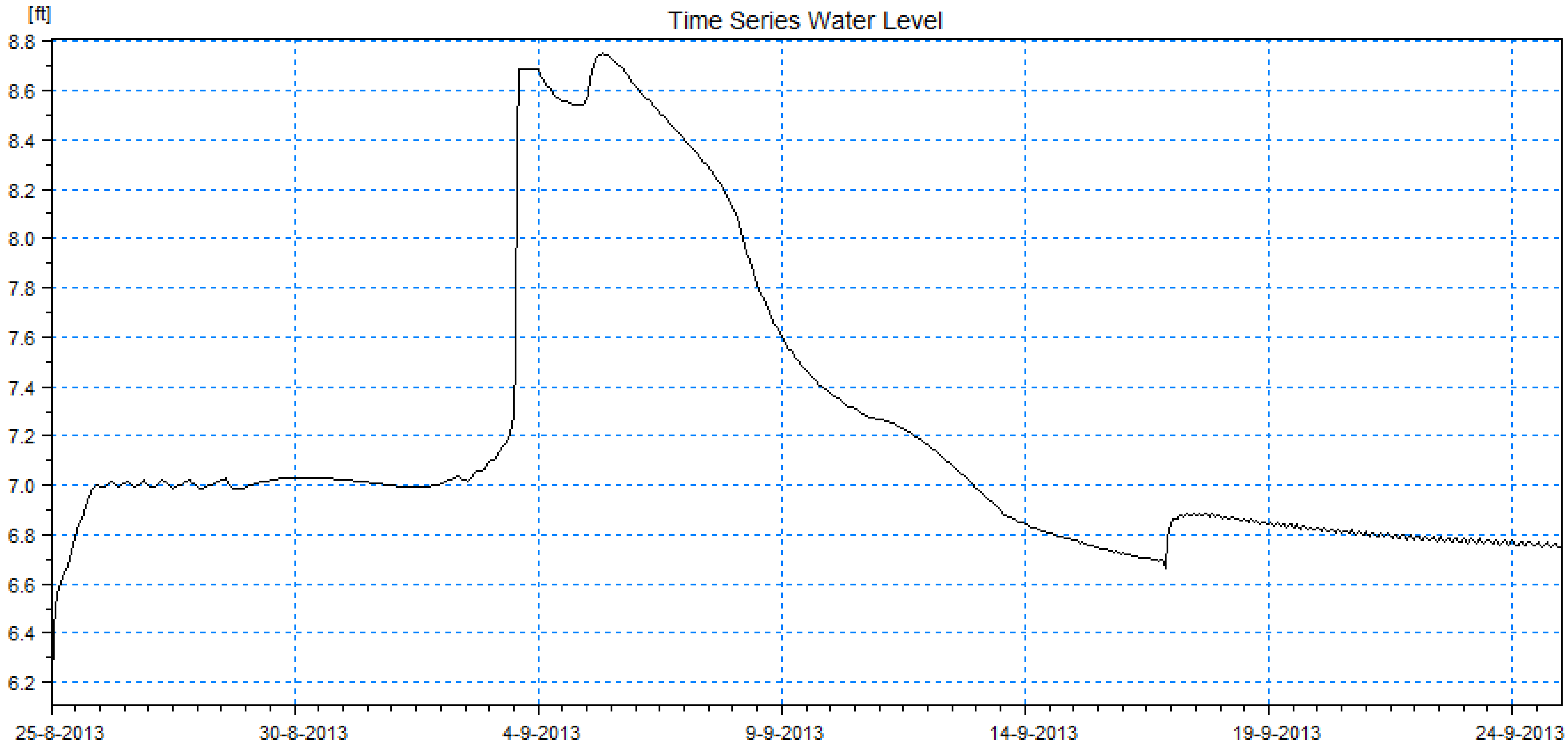

Figure 29. HC2 Headwater Stage (ft)

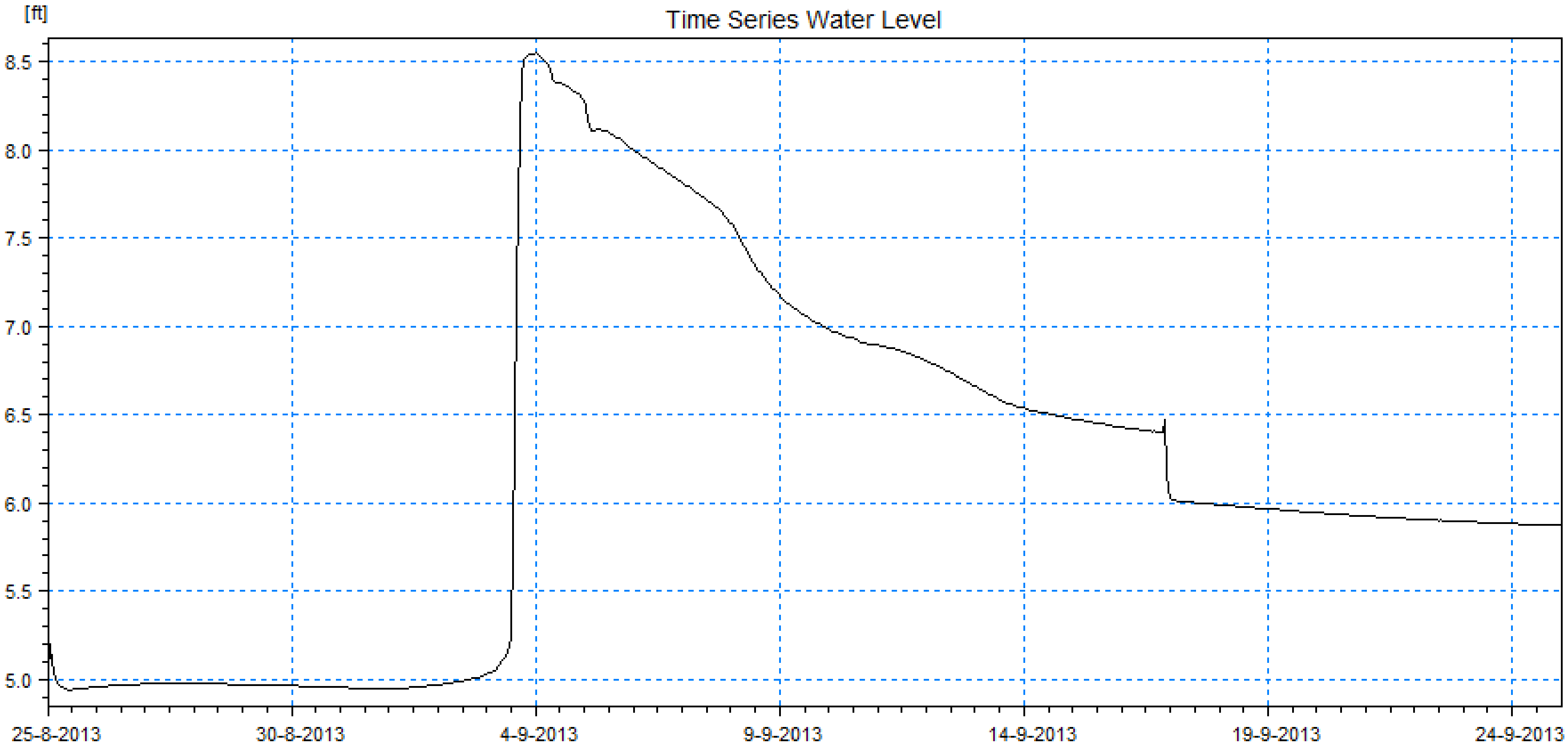

Figure 30. HC2 Tailwater Stage (ft)

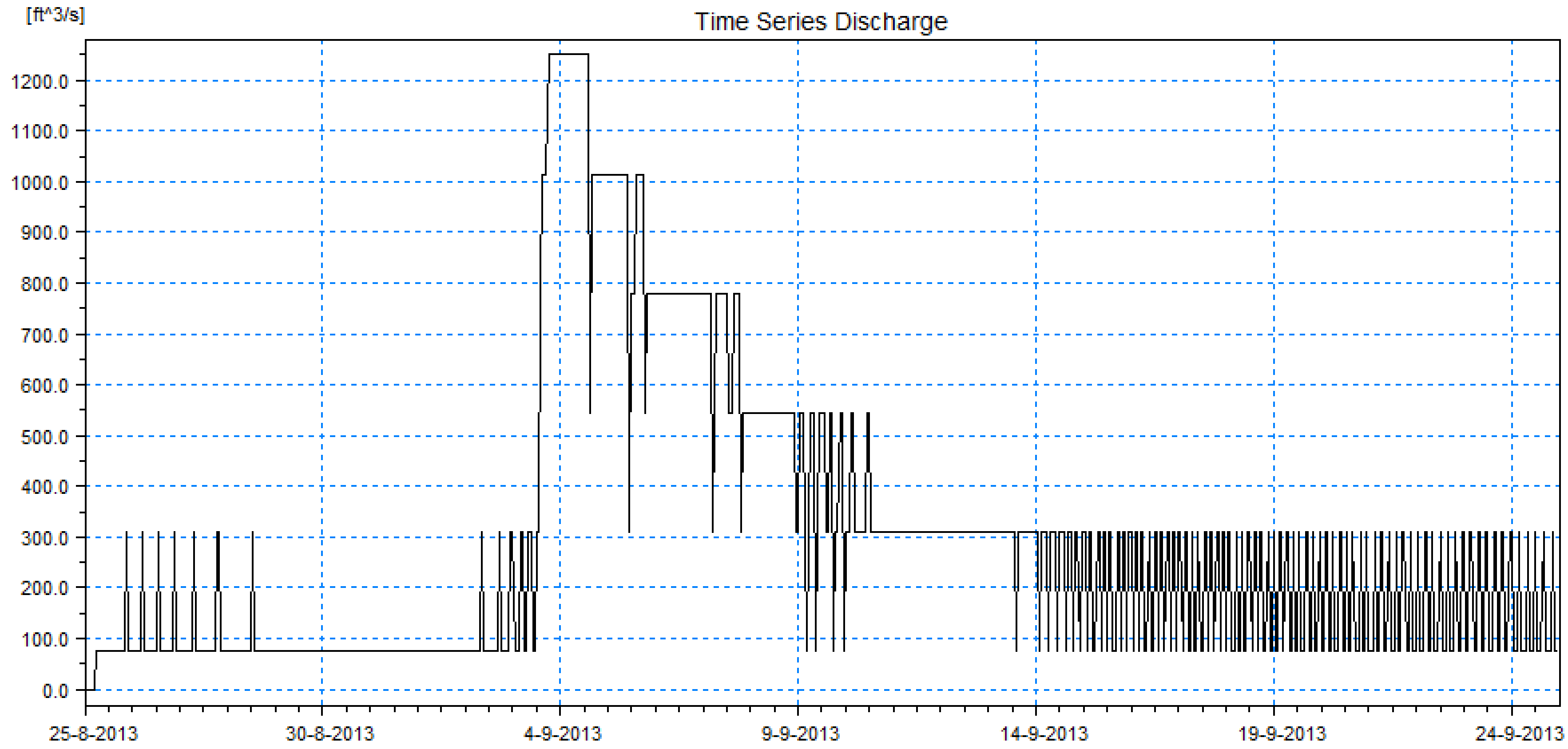

Figure 31. Miller Pump Discharge (cfs)

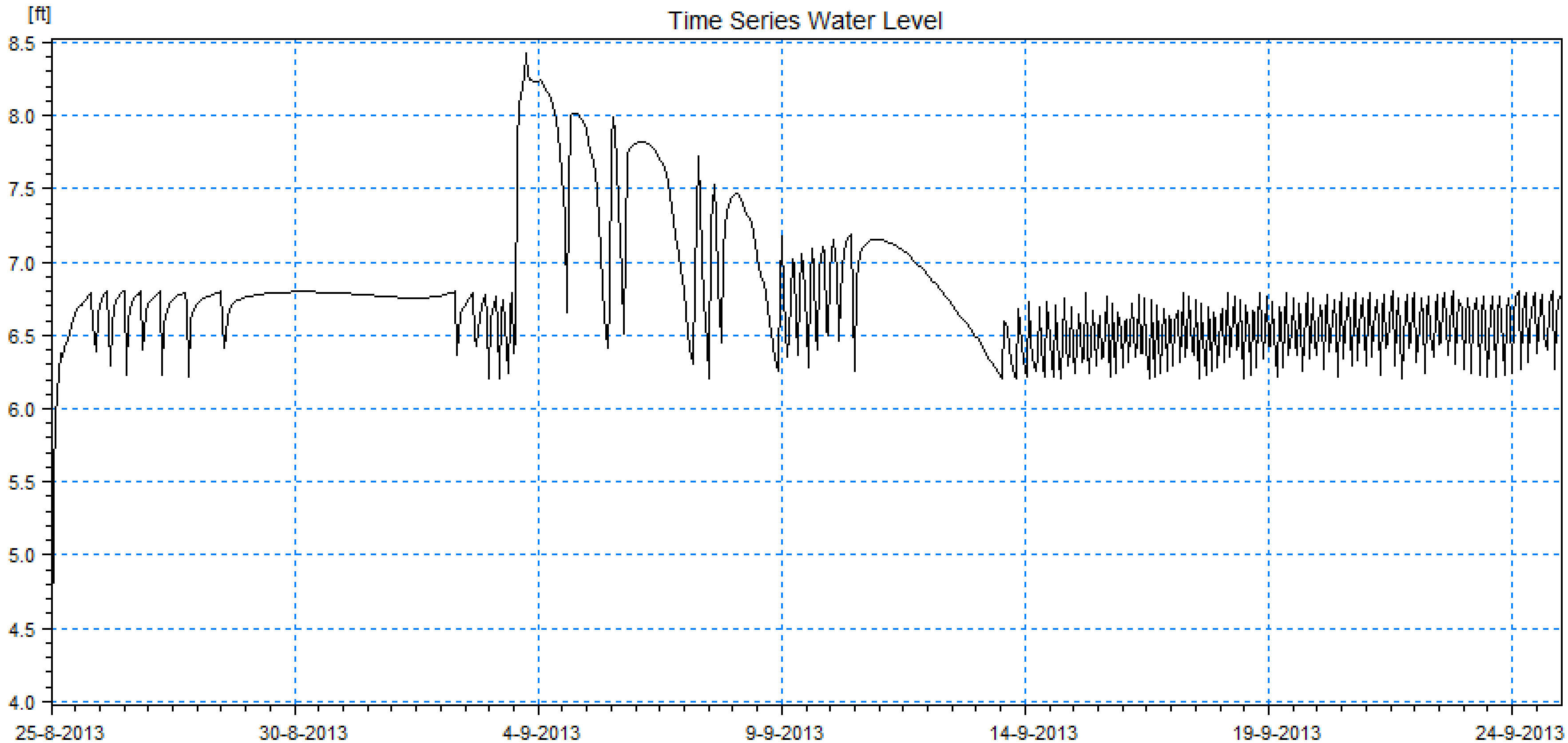

Figure 32. Miller Pump Headwater Stage (ft)

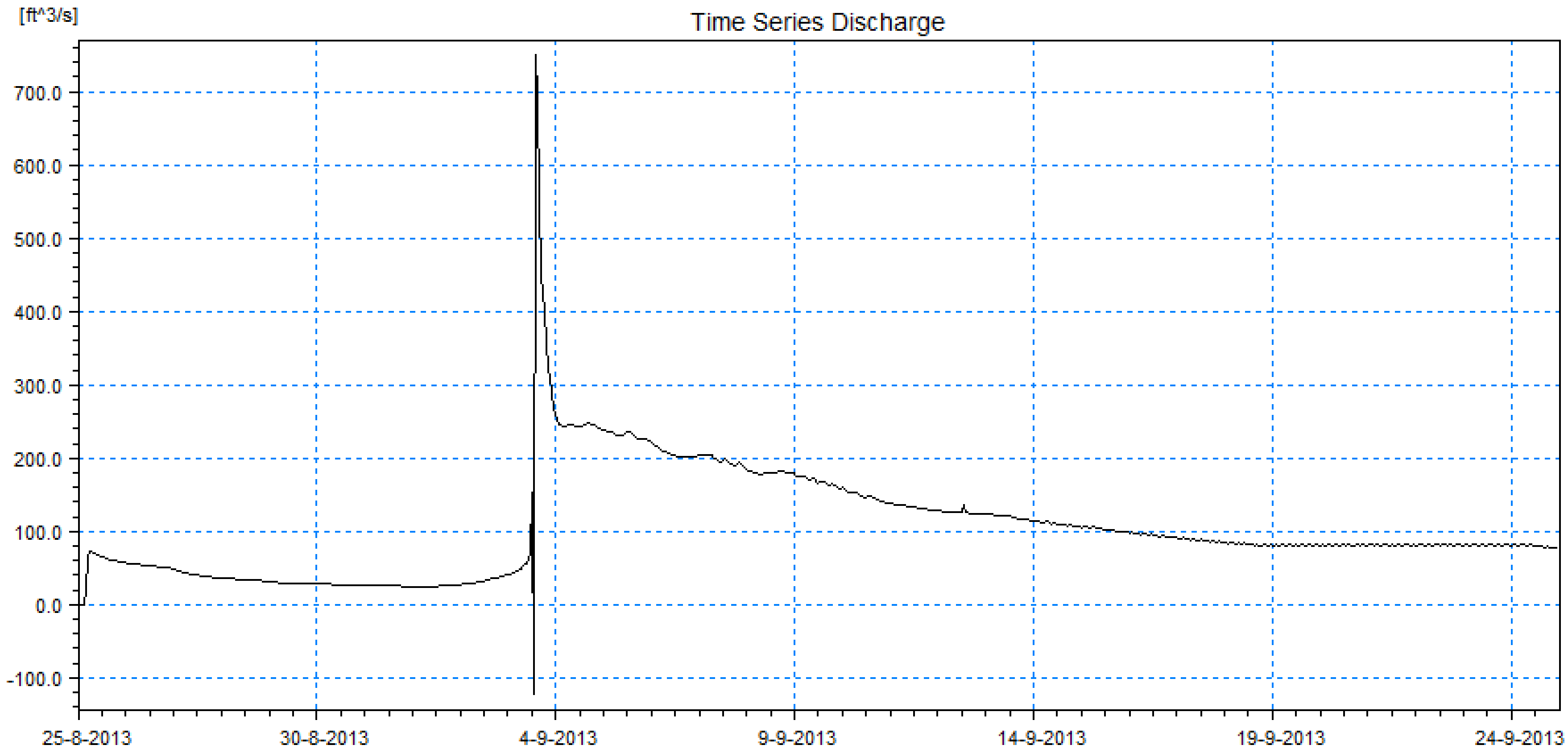

Figure 33. Miller3 Discharge (cfs)

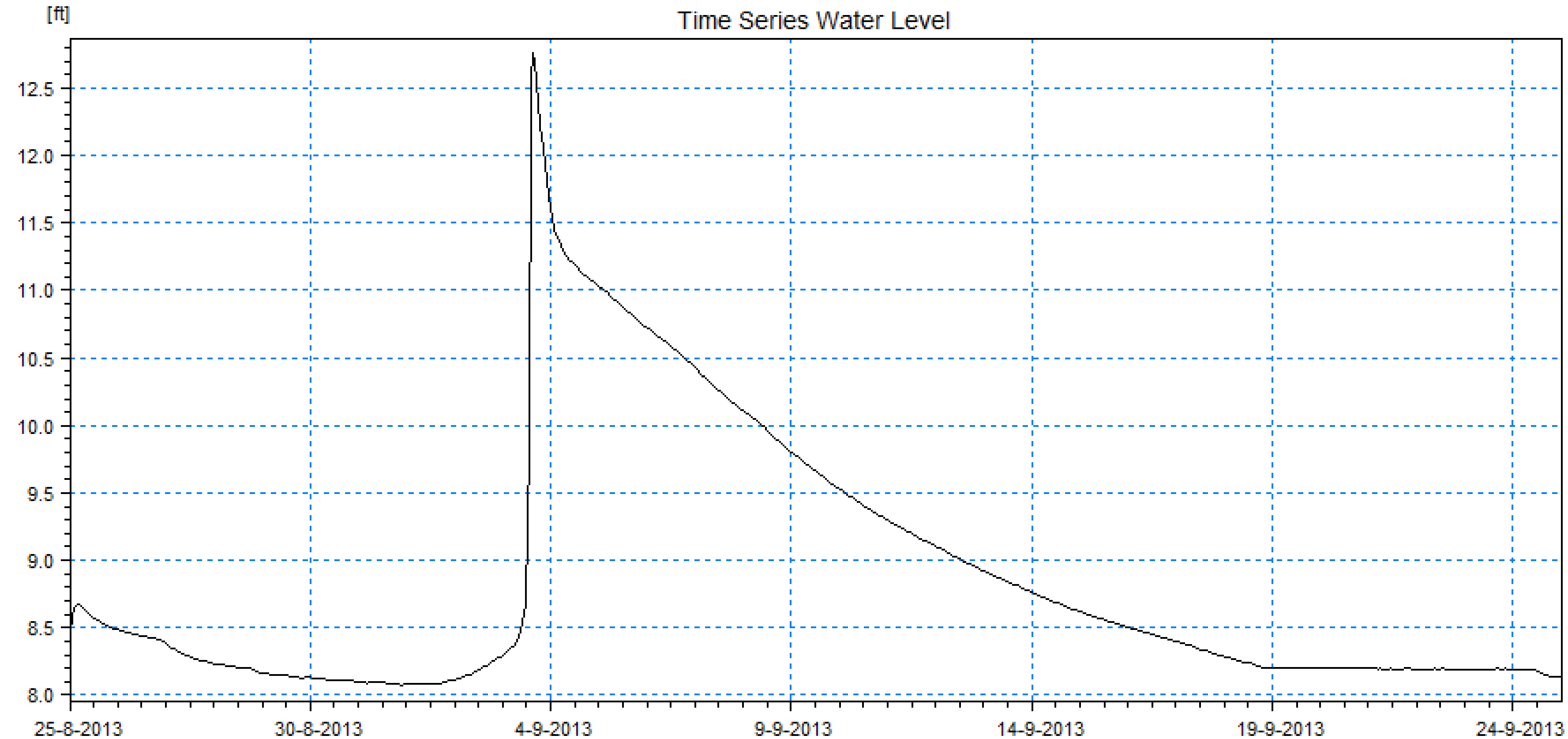

Figure 34. Miller3 Headwater Stage (ft)

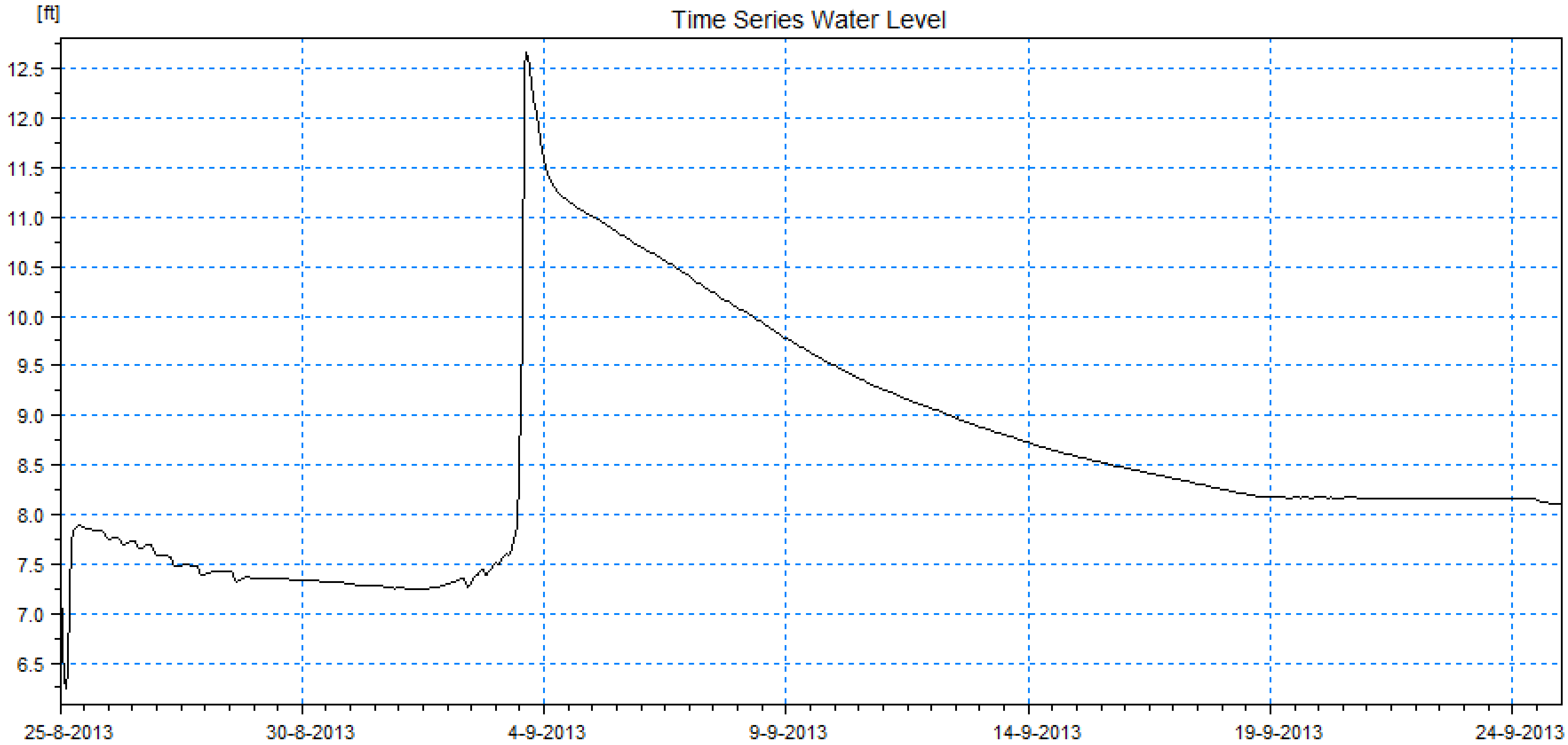

Figure 35. Miller3 Tailwater Stage (ft)

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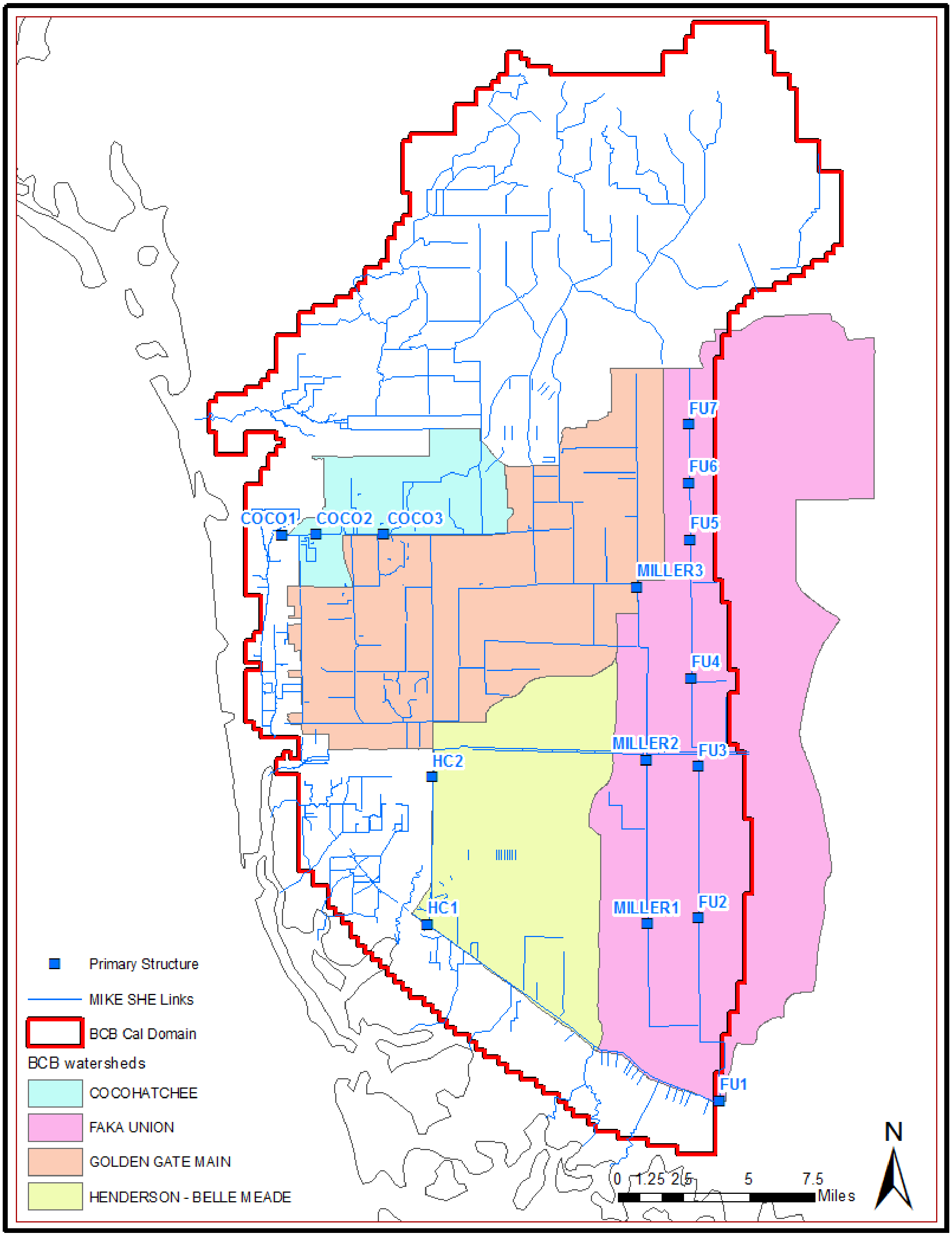

Figure 1 Mike SHE Stations within COCO, FAKA and HEND Watersheds

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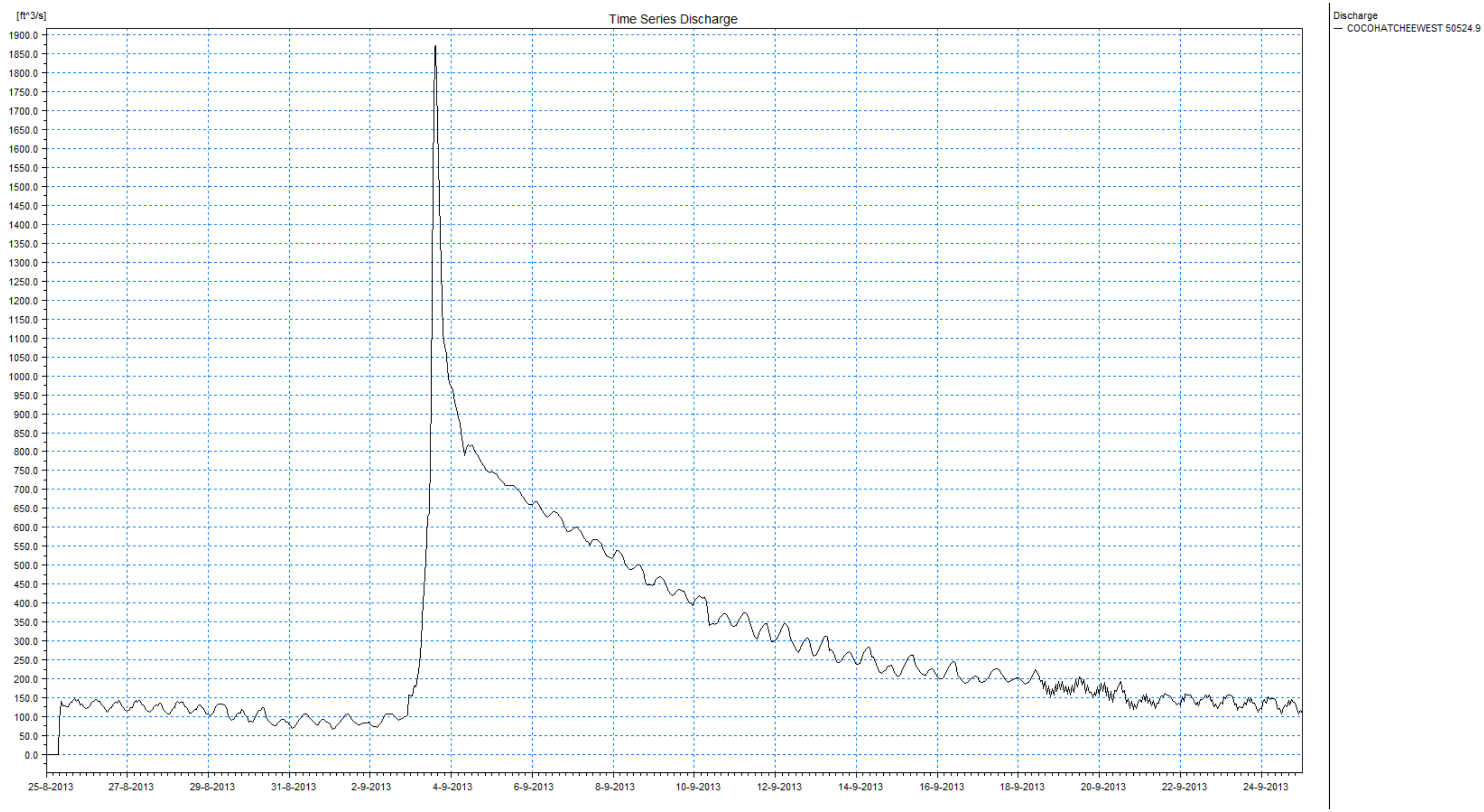
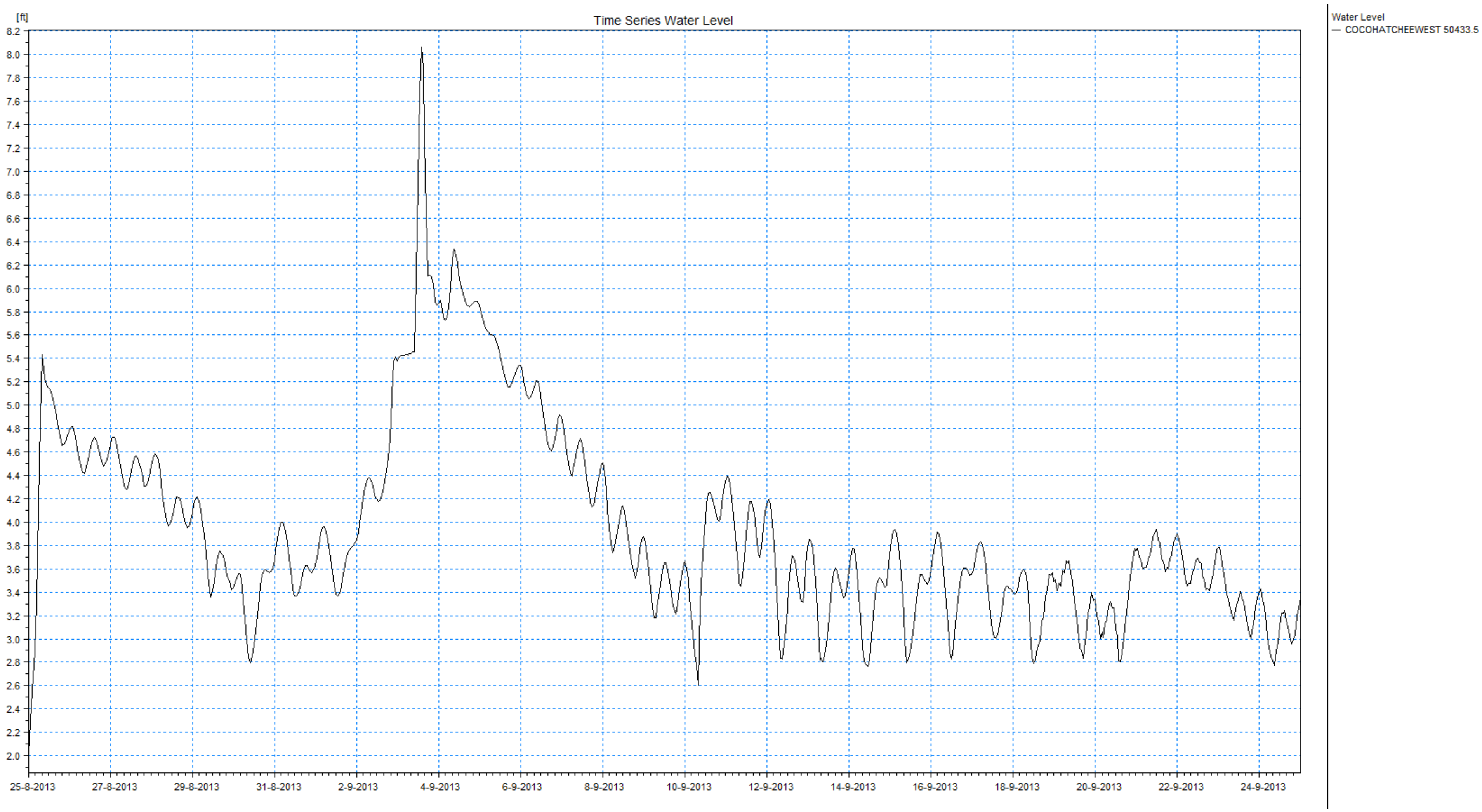


Figure 2. COCO1 Discharge (cfs)



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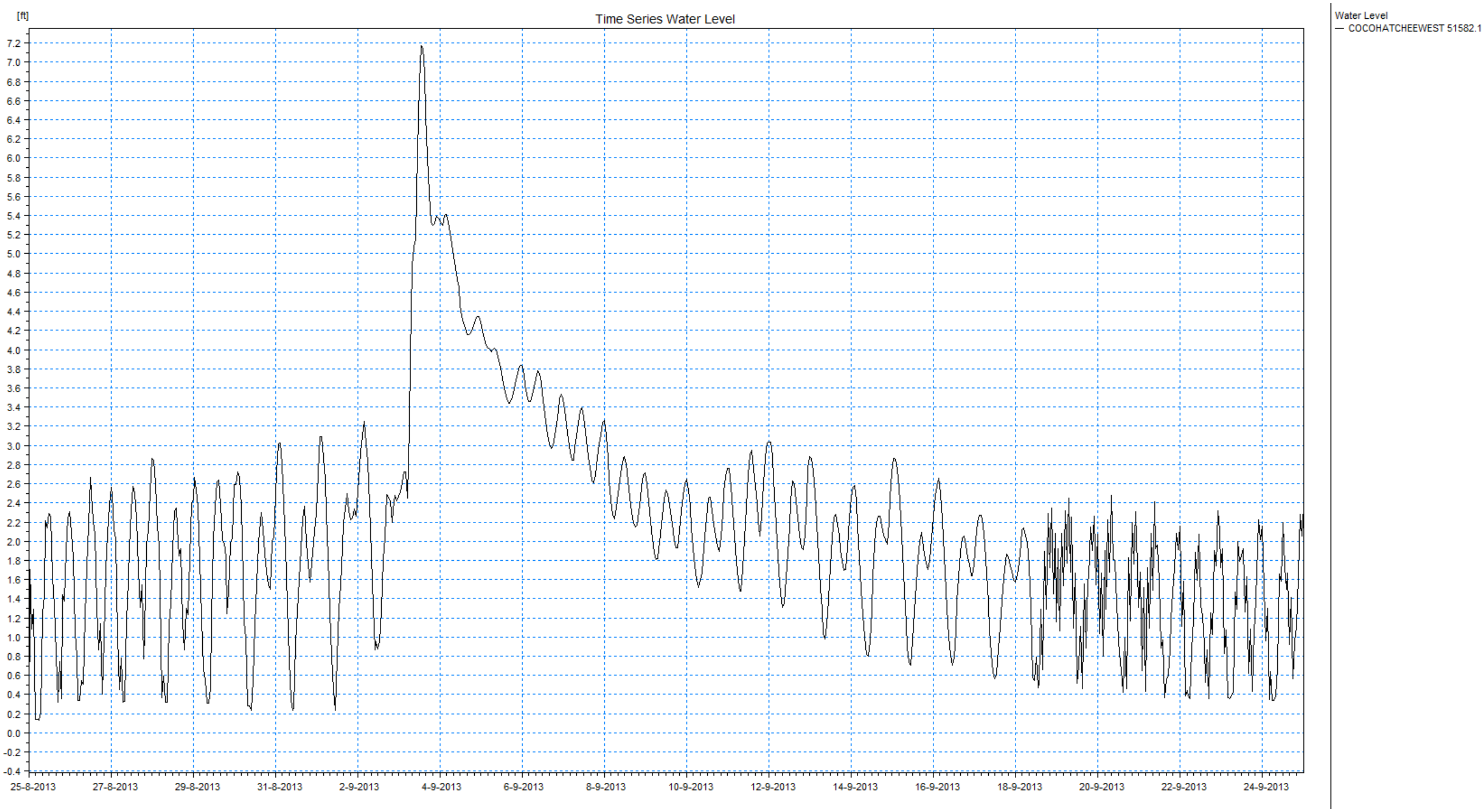


Figure 4. COCO1 Tailwater Stage (ft)

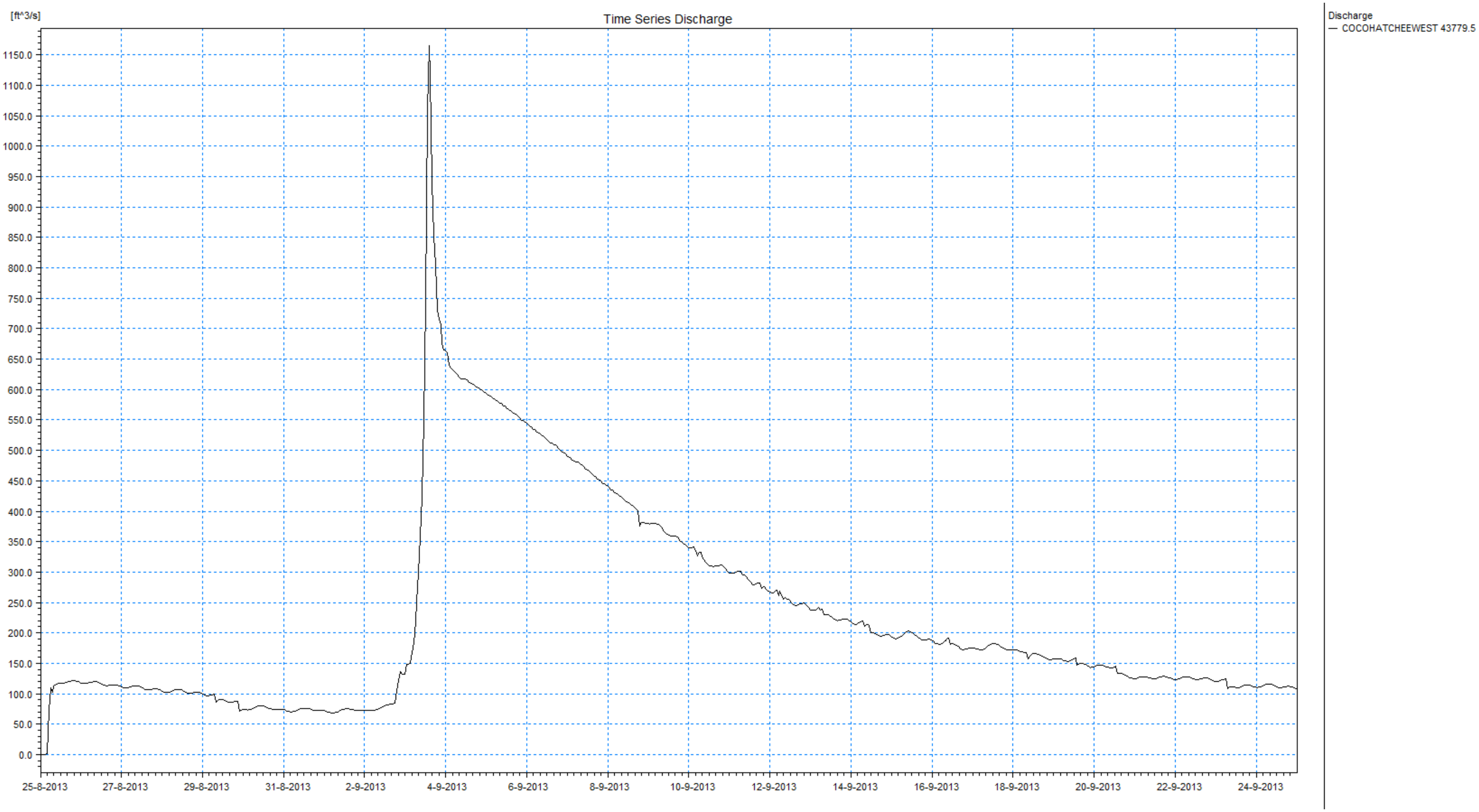


Figure 5. COCO2 Discharge (cfs)

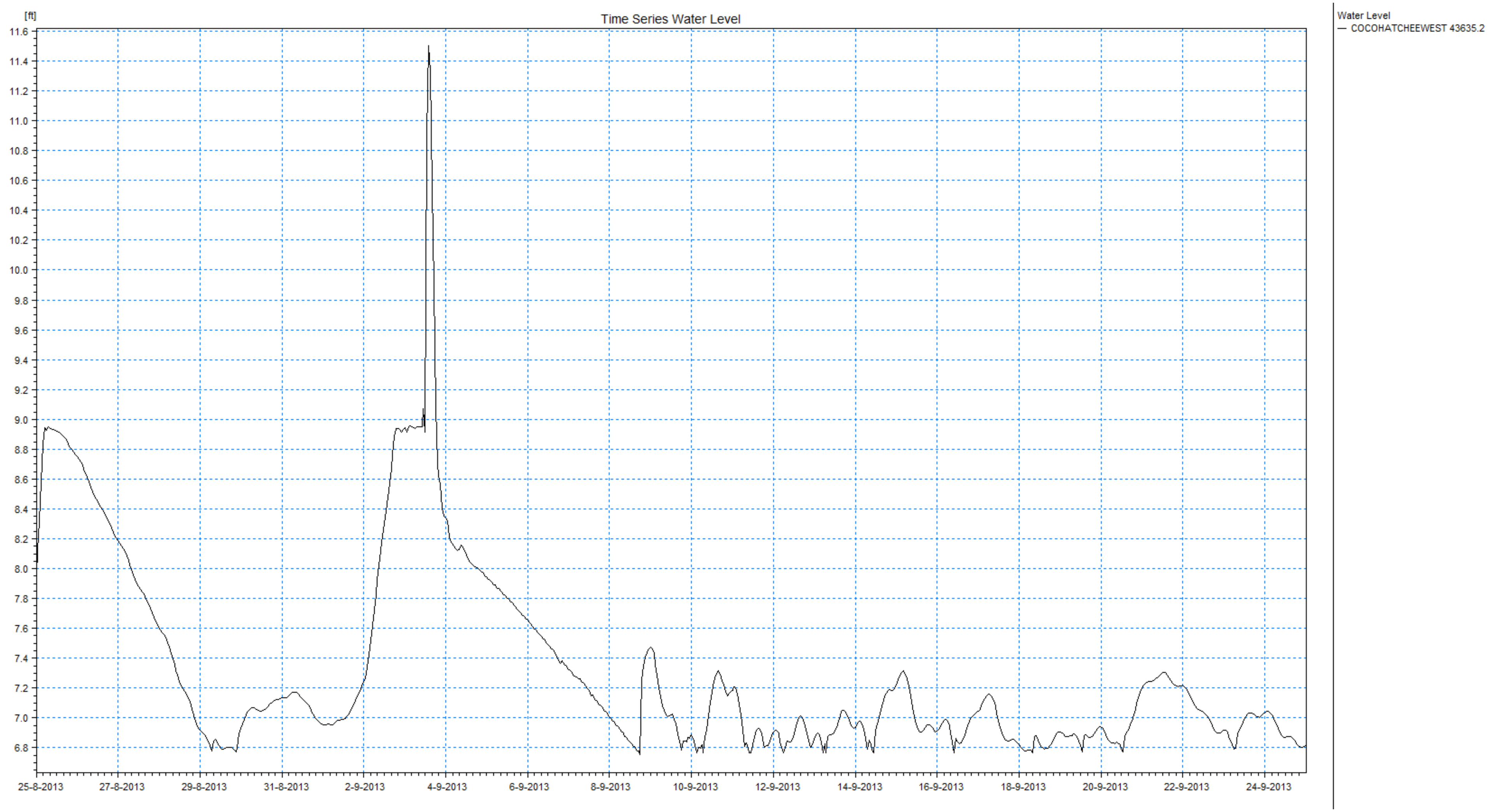


Figure 6. COCO2 Headwater Stage (ft)

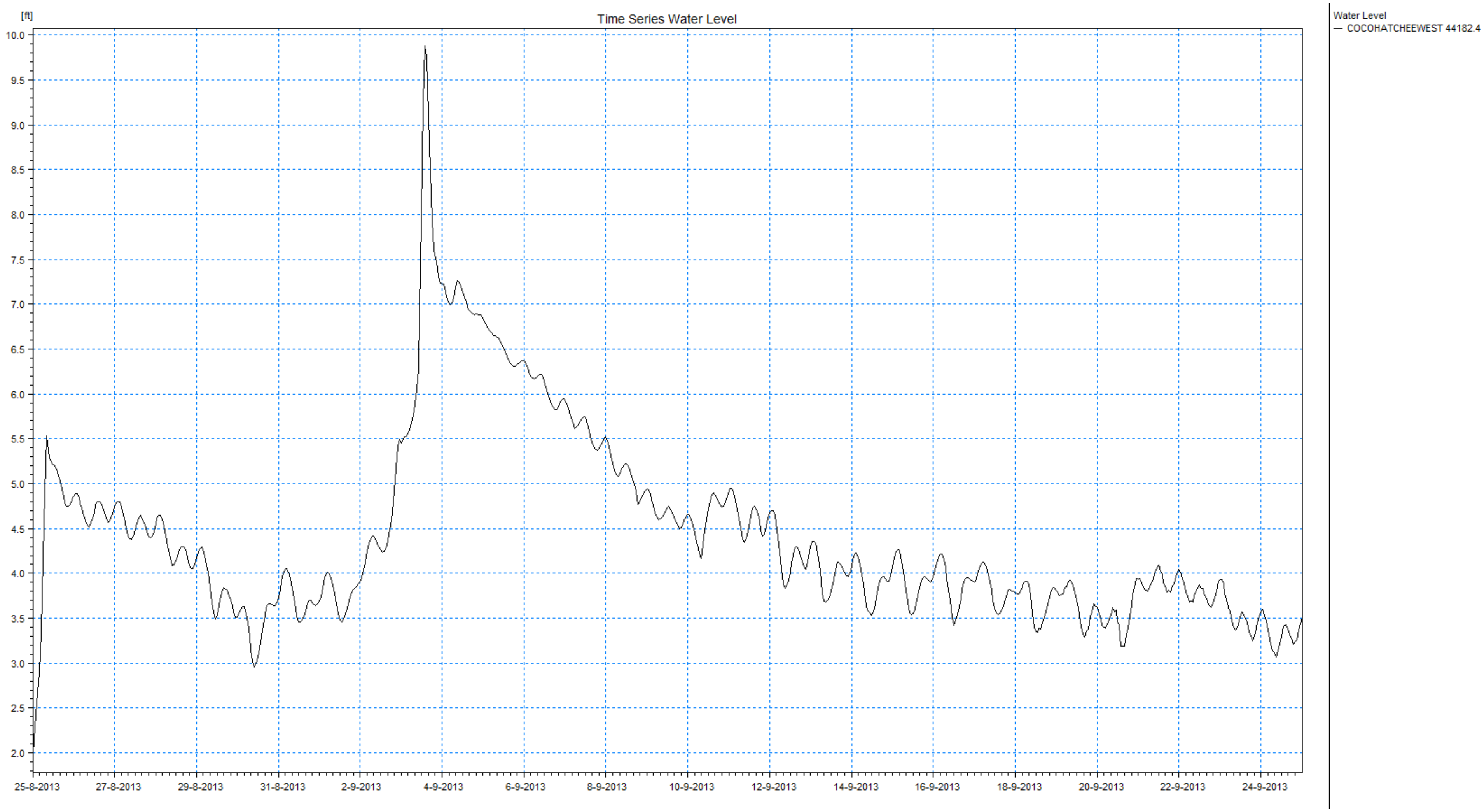


Figure 7. COCO2 Tailwater Stage (ft)

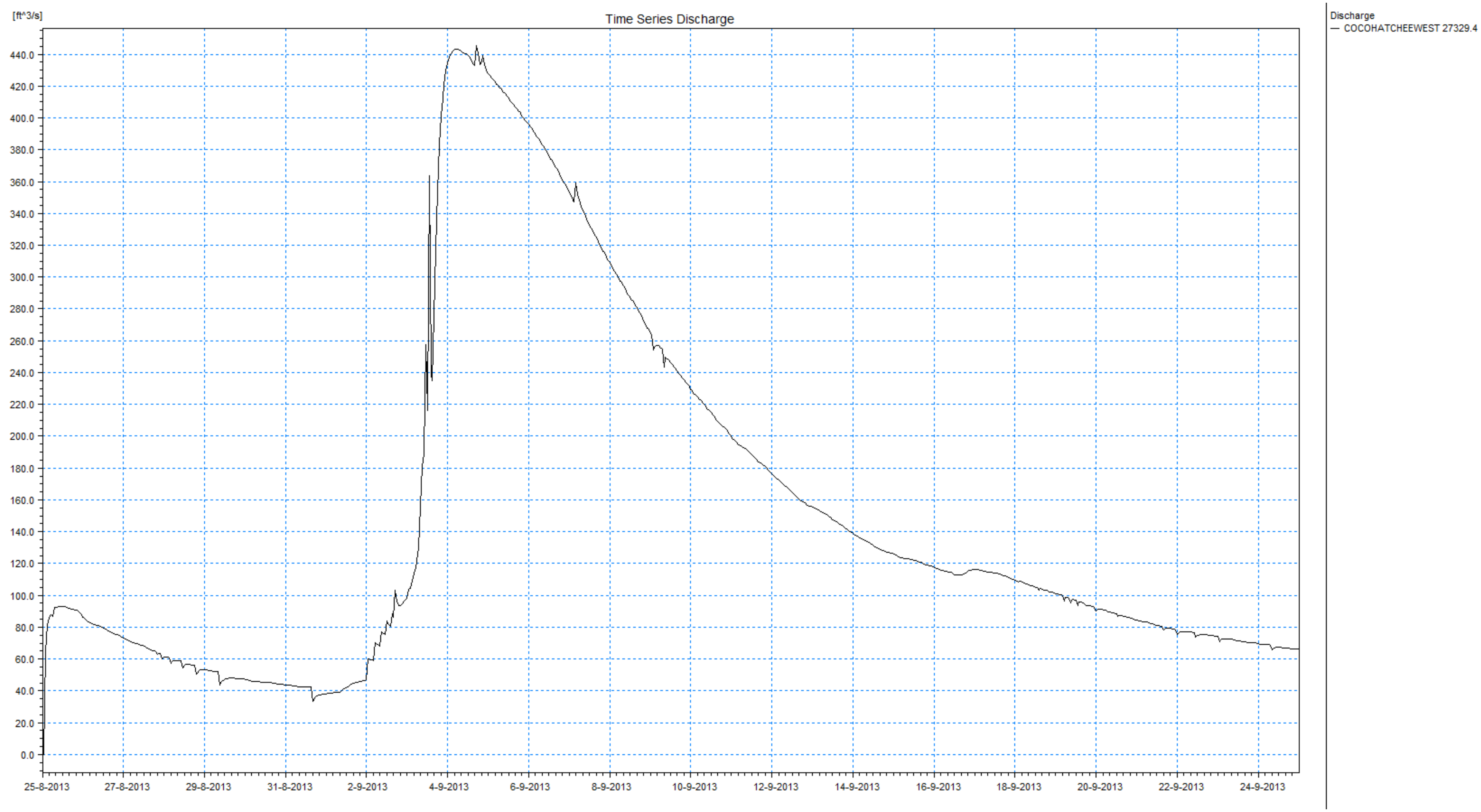


Figure 8. COCO3 Discharge (cfs)

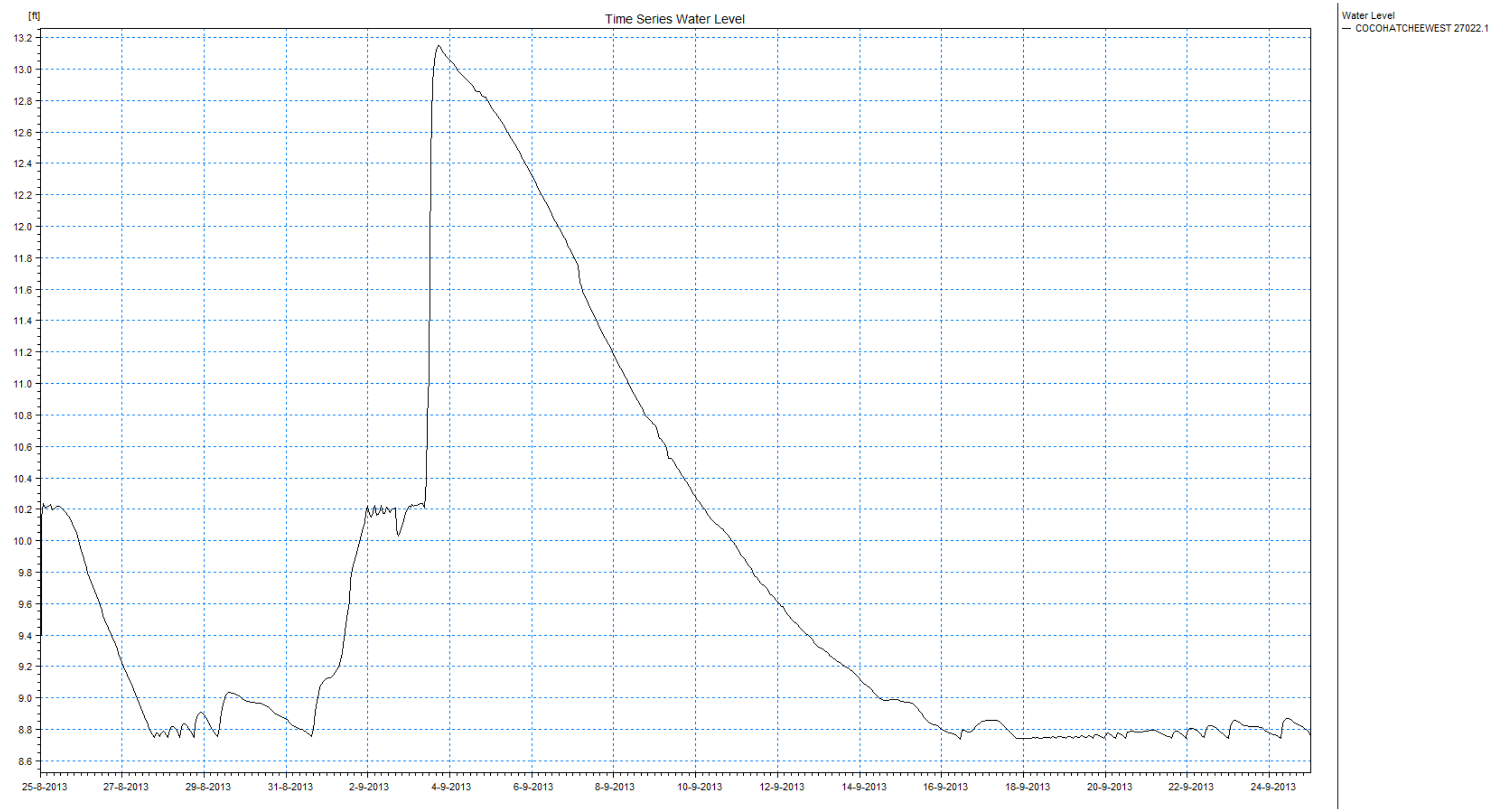


Figure 9. COCO3 Headwater Stage (ft)

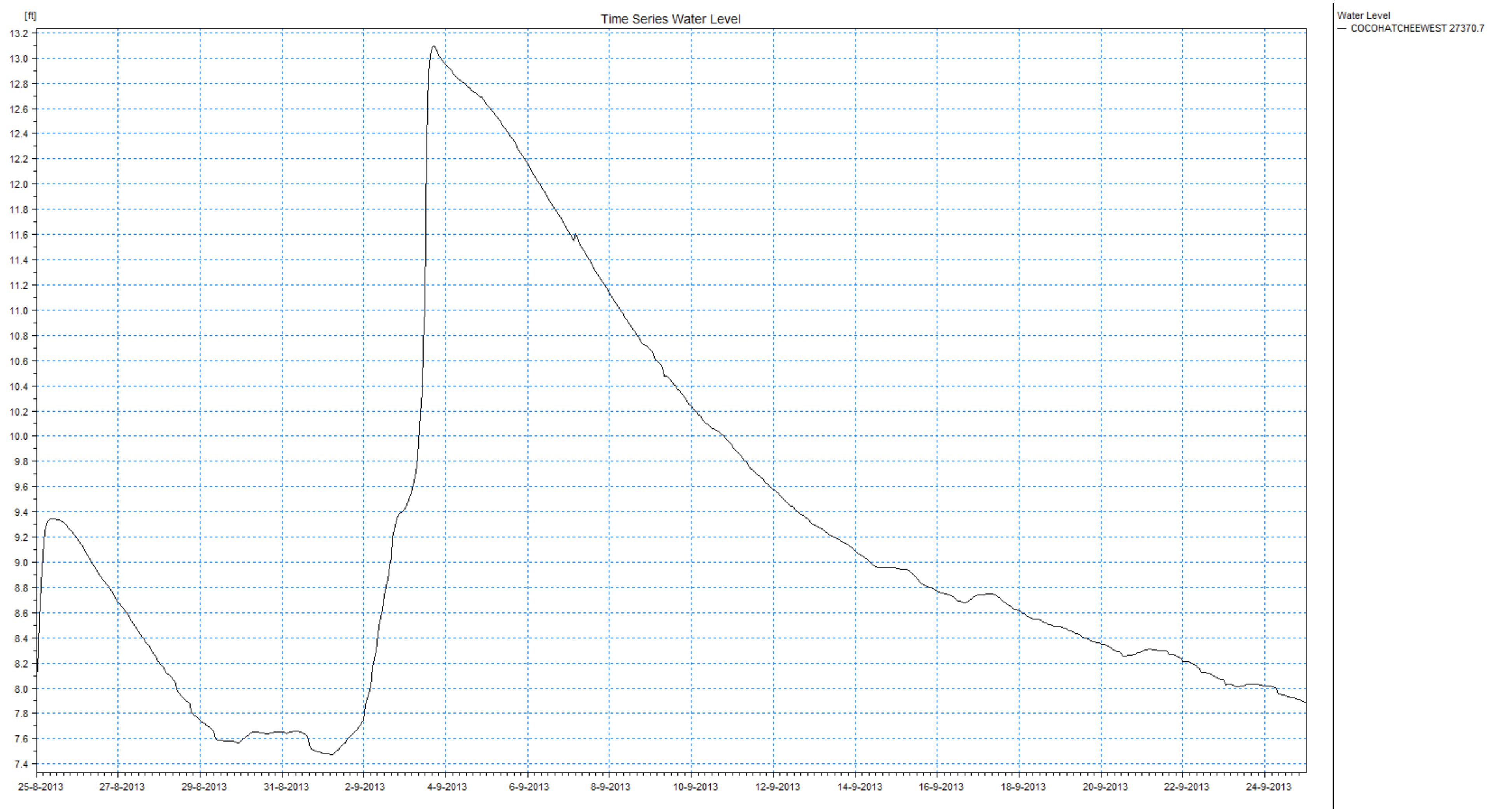


Figure 10. COCO3 Tailwater Stage (ft)

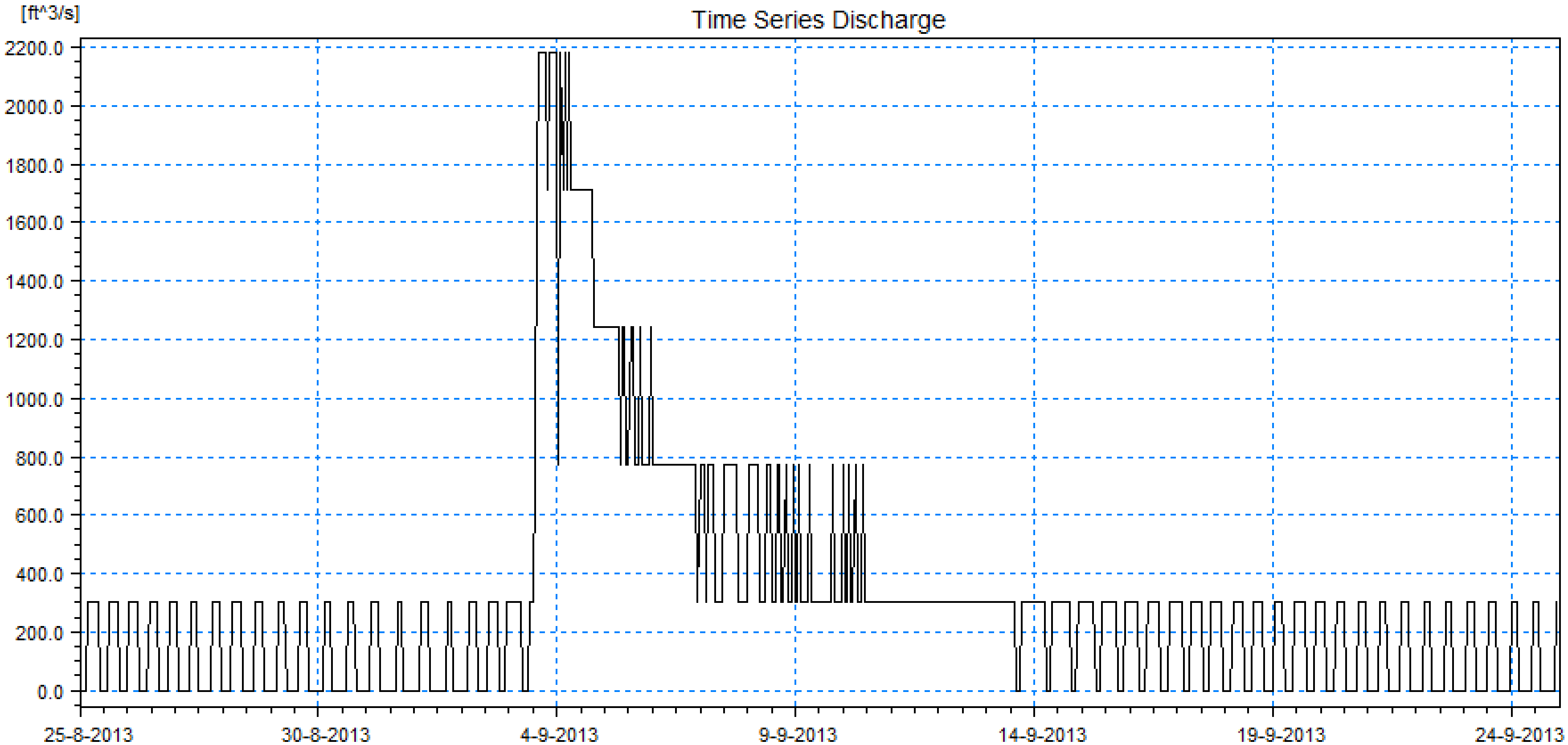


Figure 11. FU Pump Discharge (cfs)

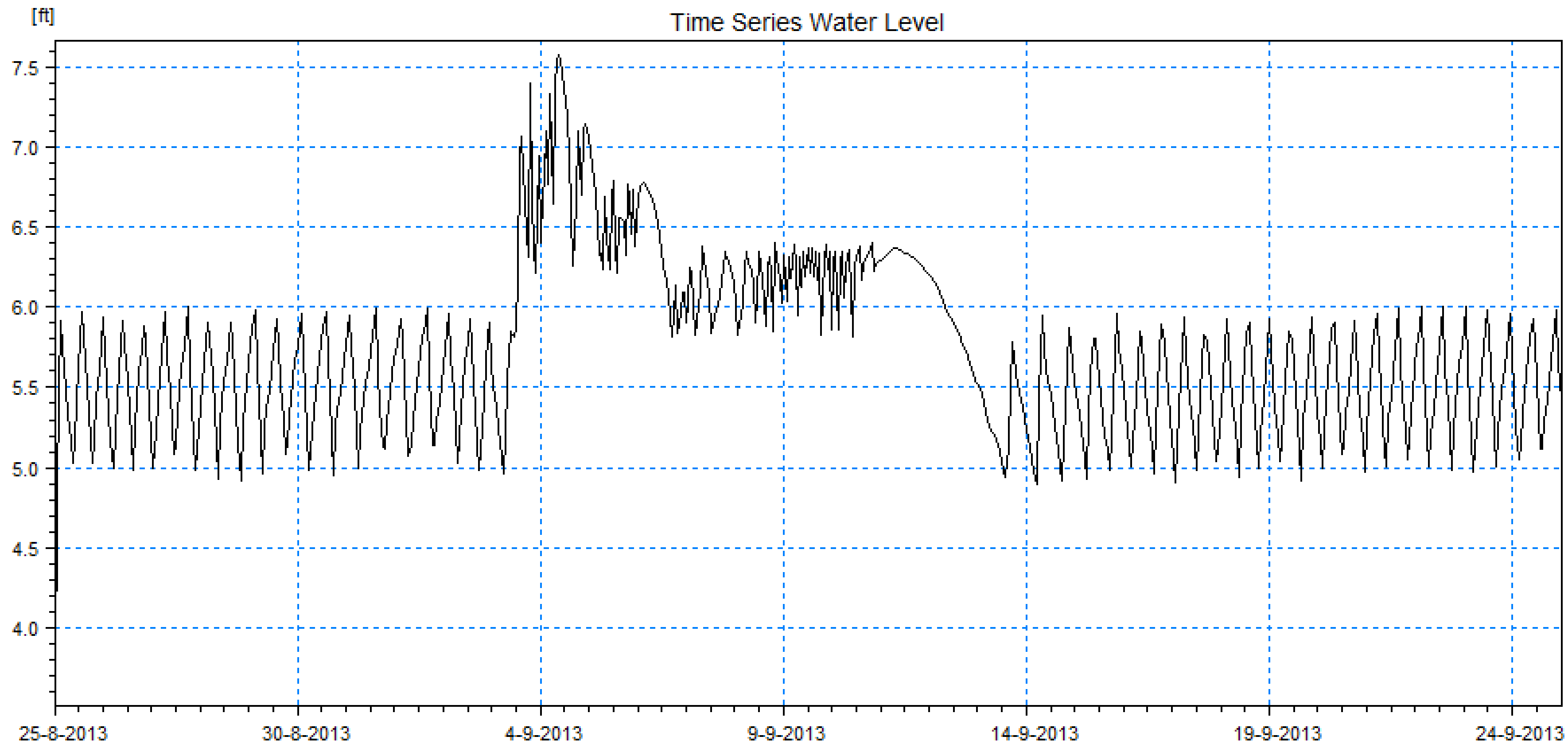


Figure 12. FU Pump Headwater Stage (ft)

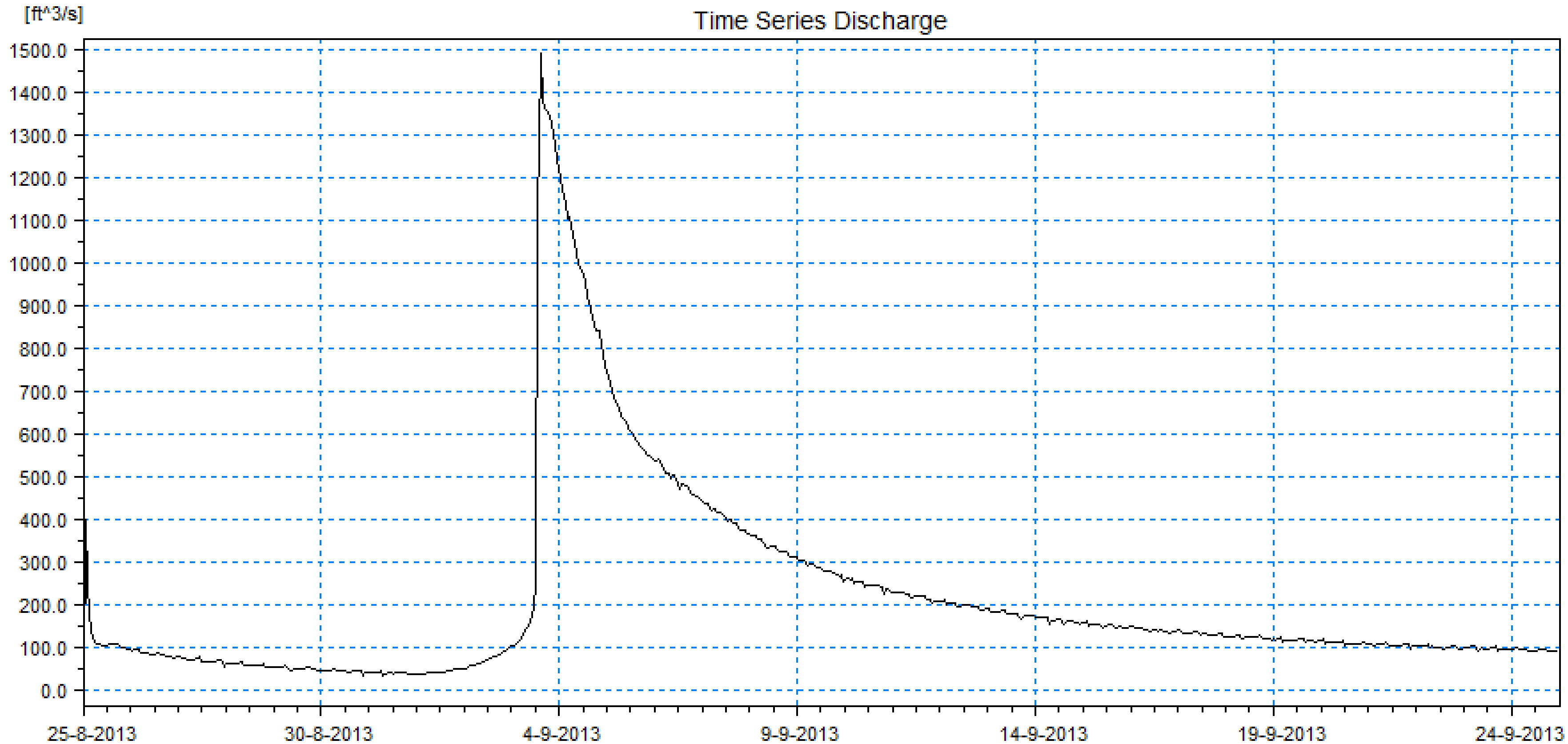


Figure 13. FU4 Discharge (cfs)

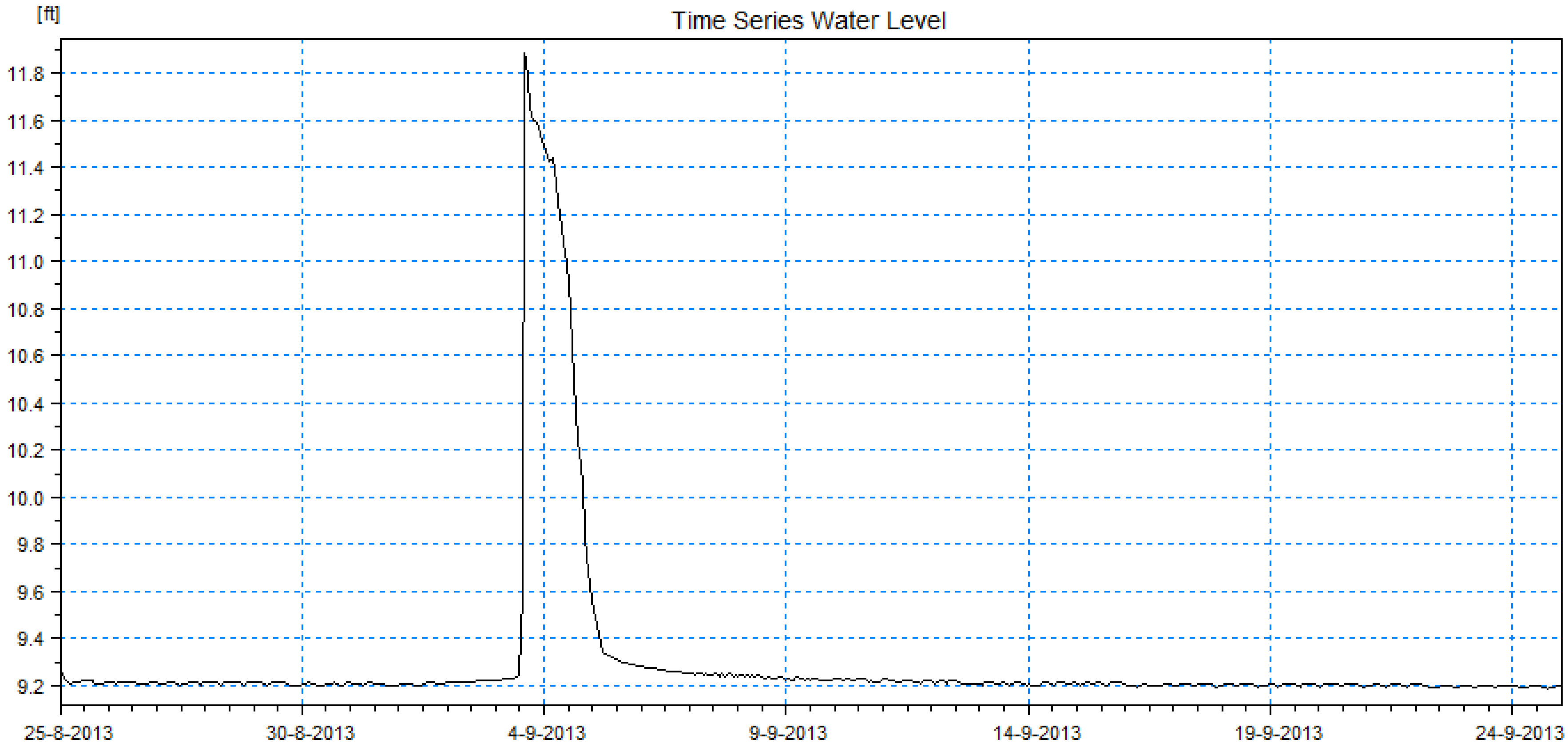


Figure 14. FU4 Headwater Stage (ft)

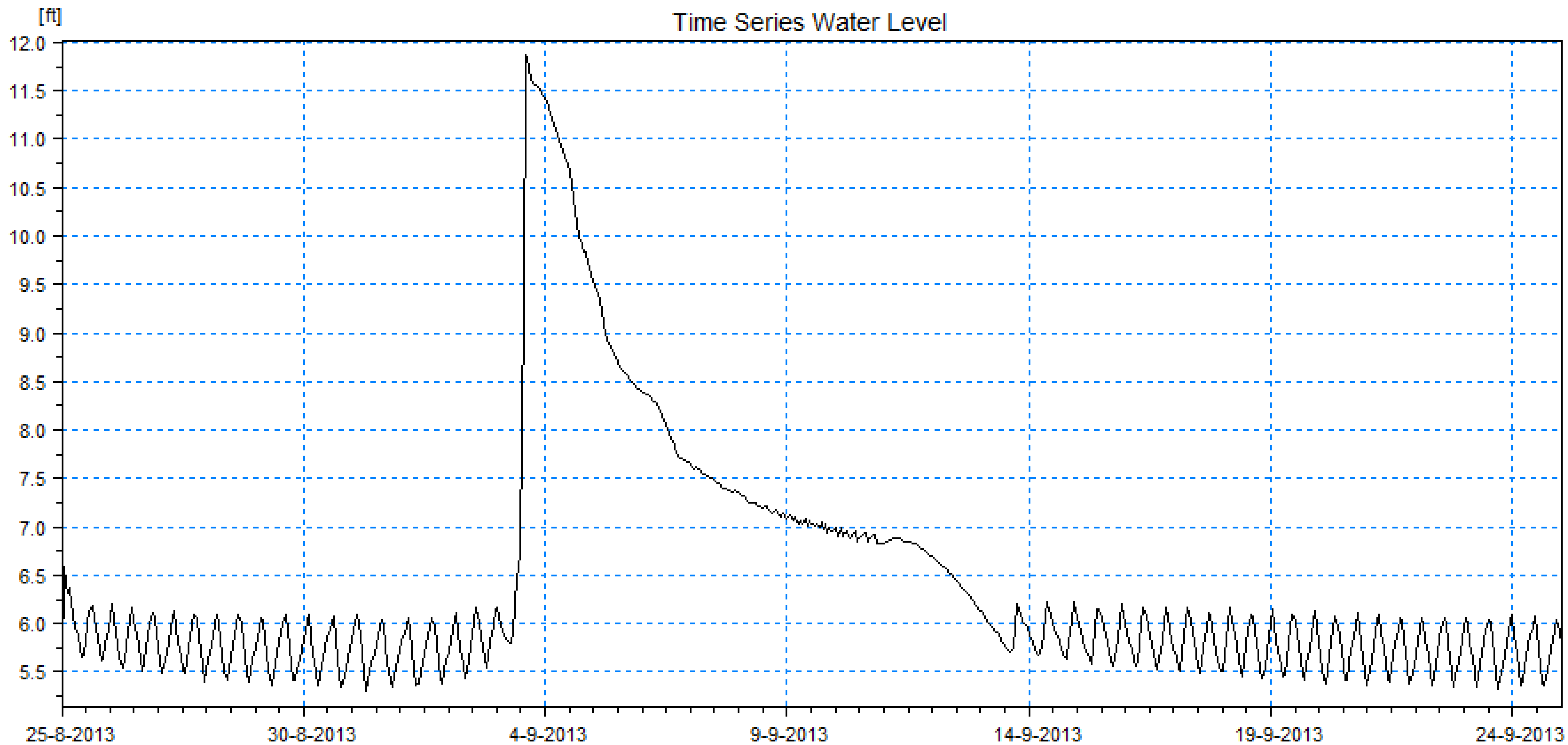


Figure 15. FU4 Tailwater Stage (ft)

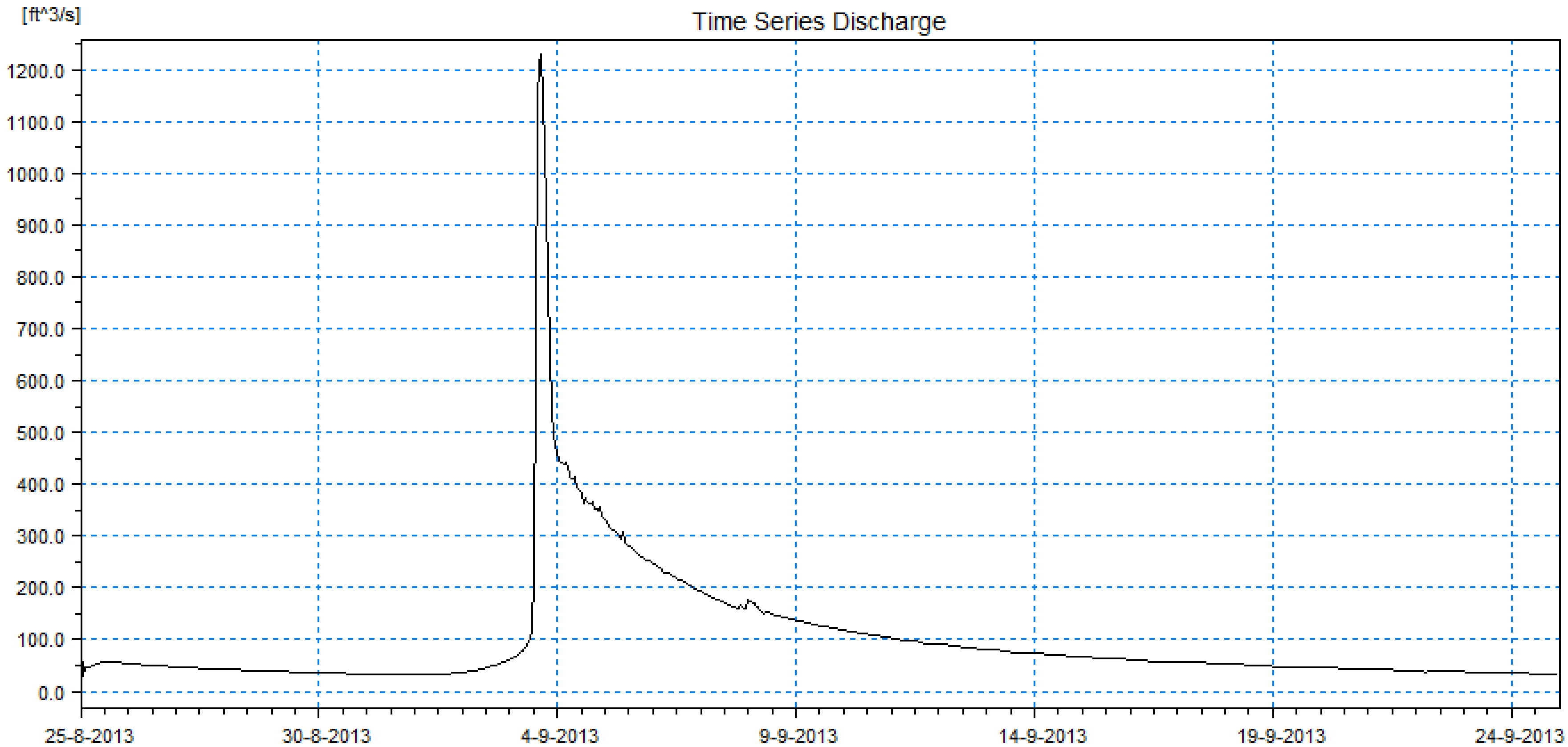


Figure 16. FU5 Discharge (cfs)

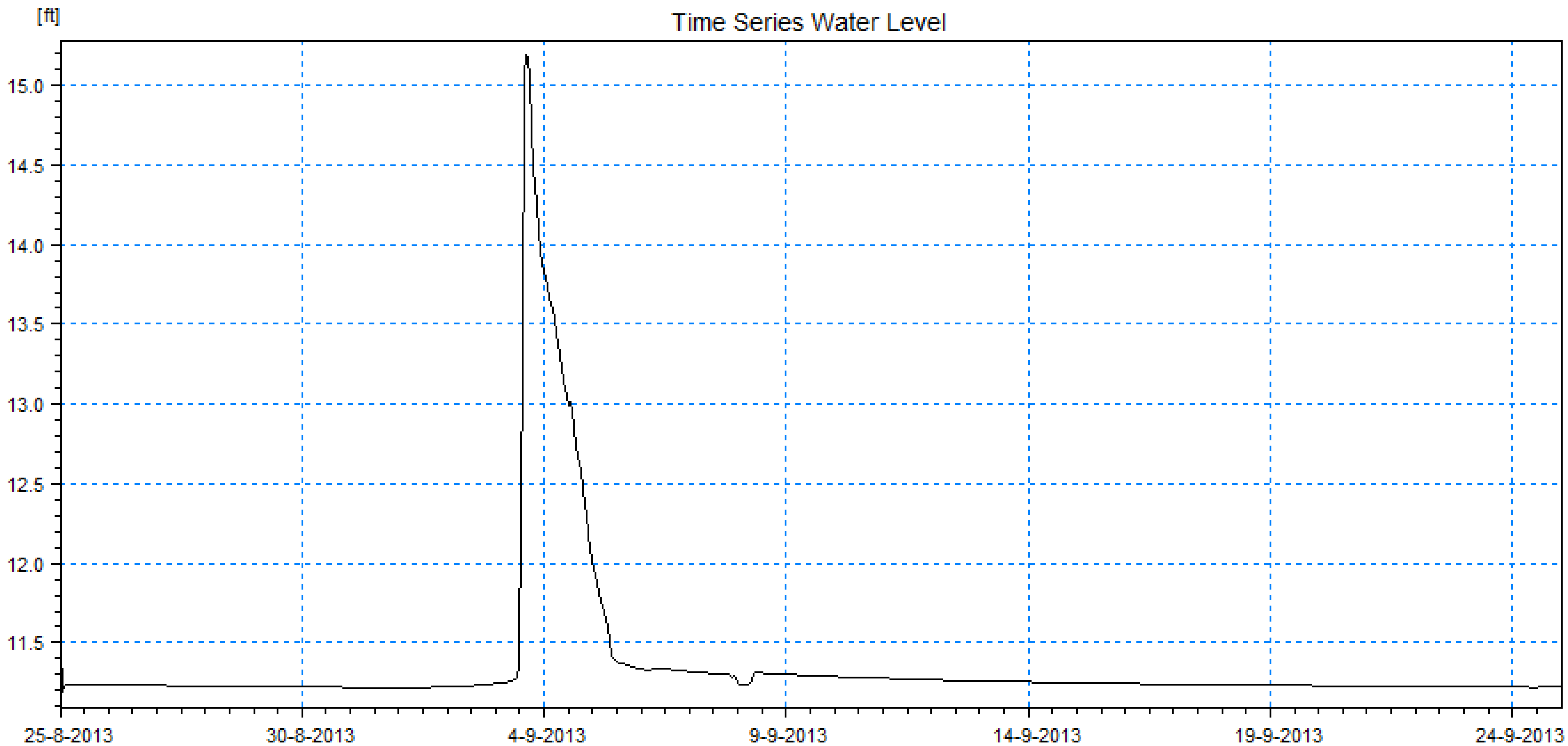


Figure 17. FU5 Headwater Stage (ft)

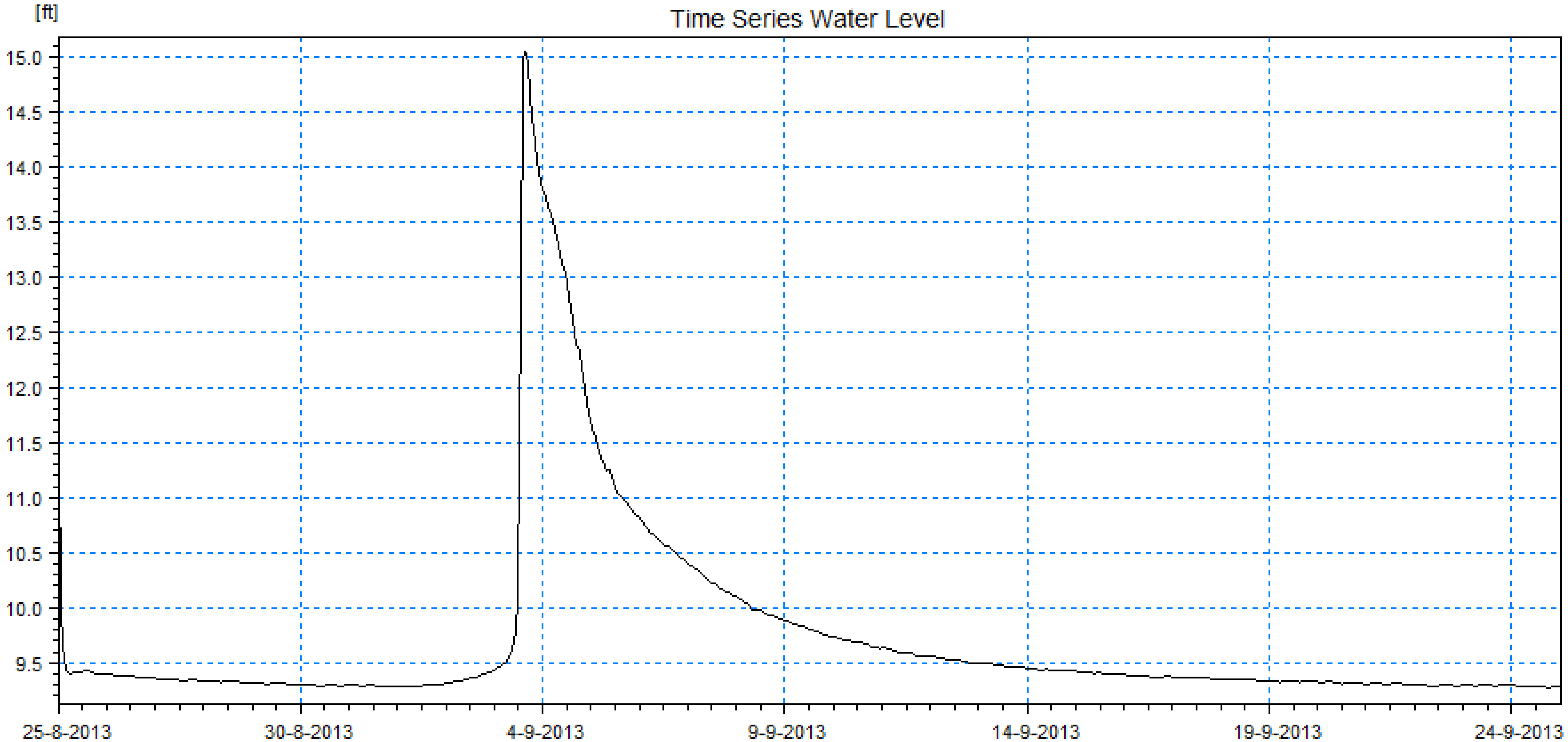


Figure 18. FU5 Tailwater Stage (ft)

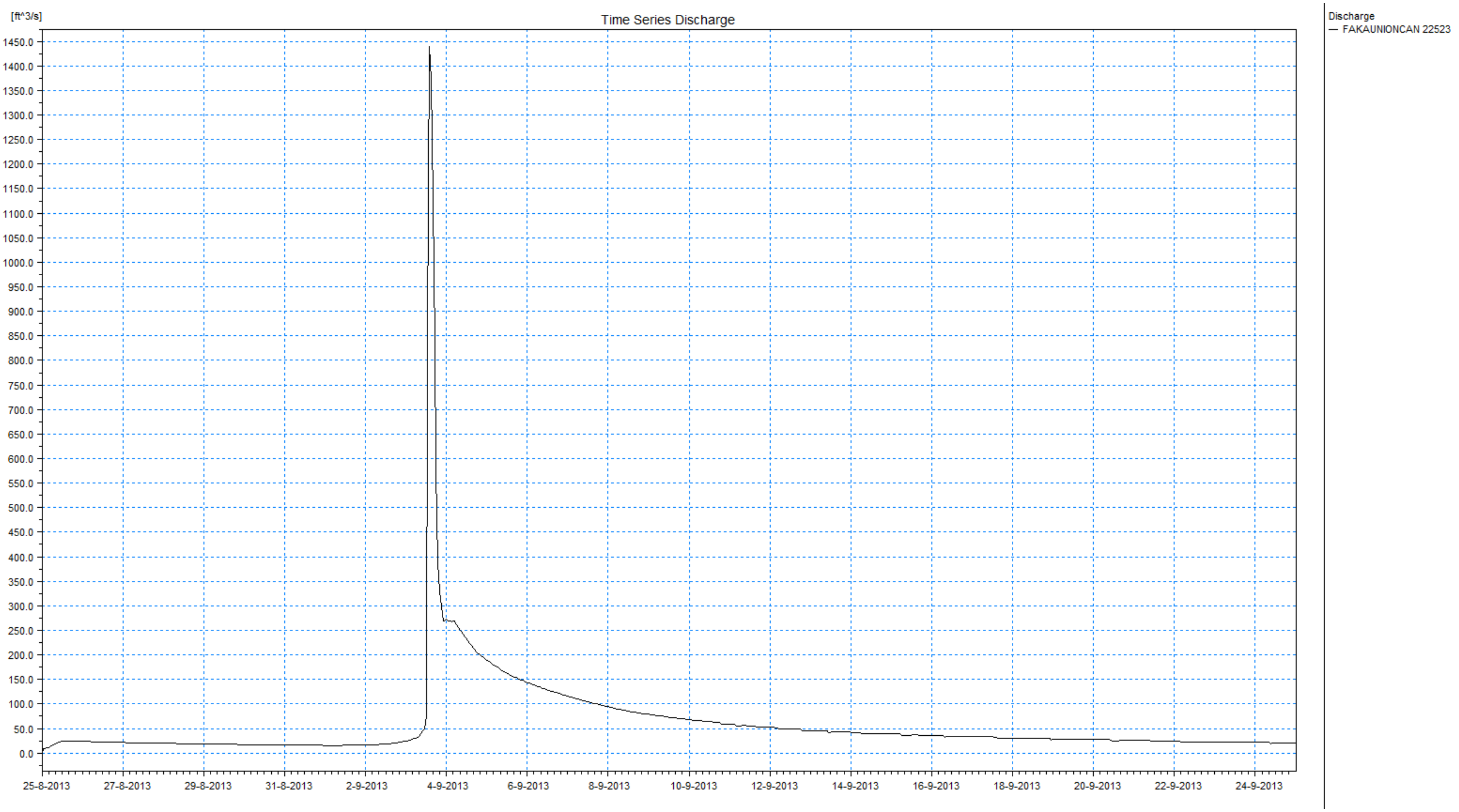


Figure 19. FU6 Discharge (cfs)

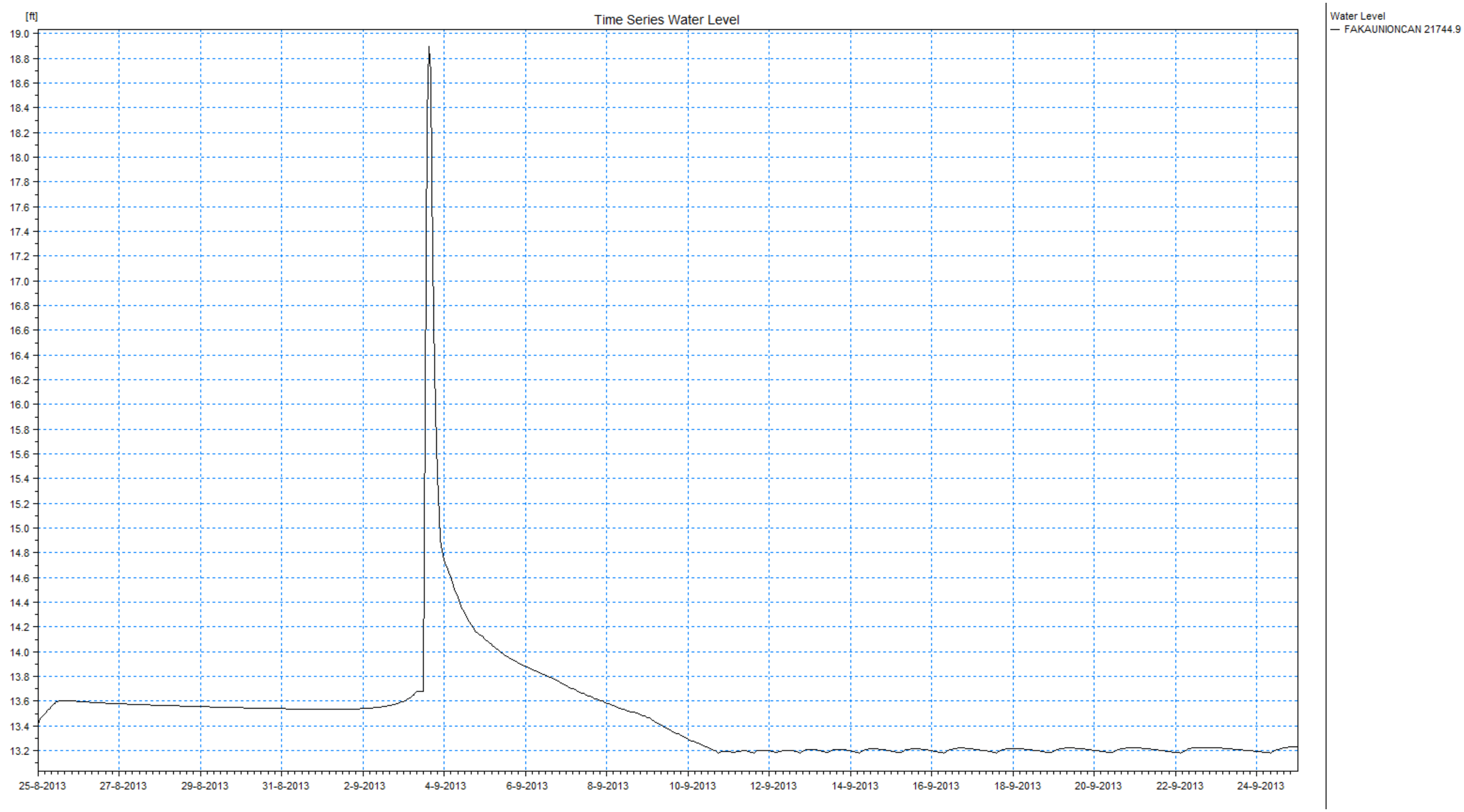


Figure 20. FU6 Headwater Stage (ft)

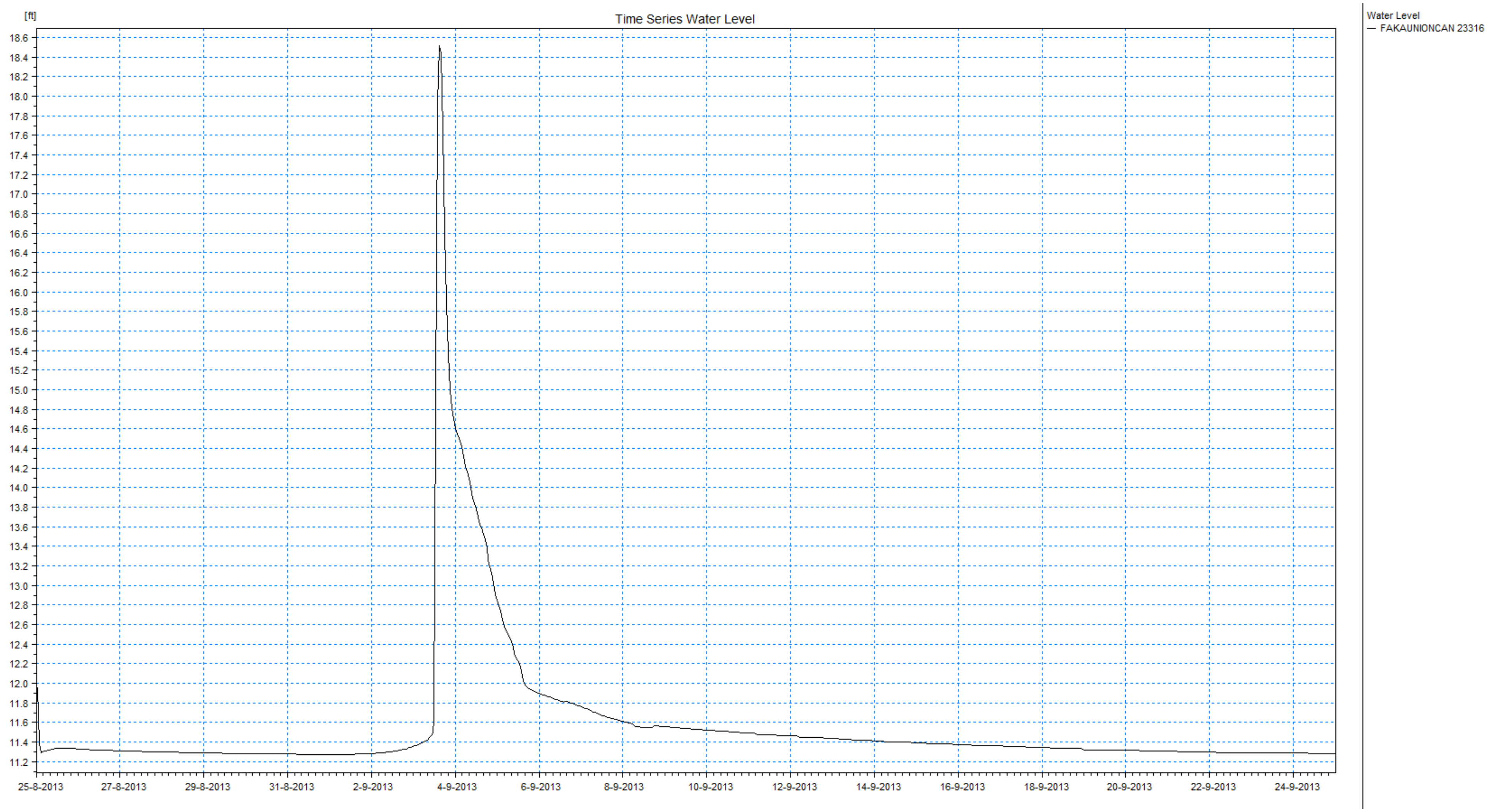


Figure 21. FU6 Tailwater Stage (ft)

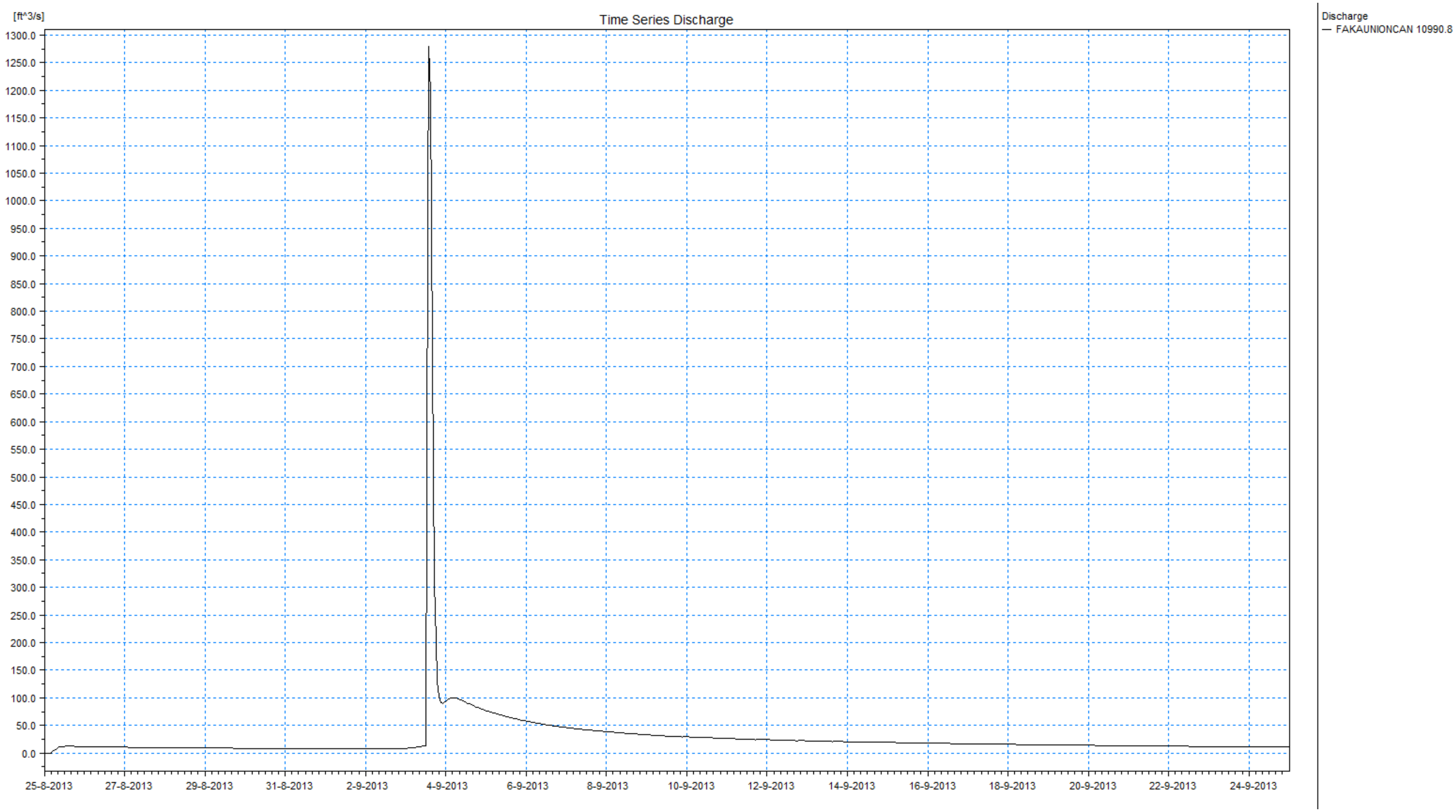
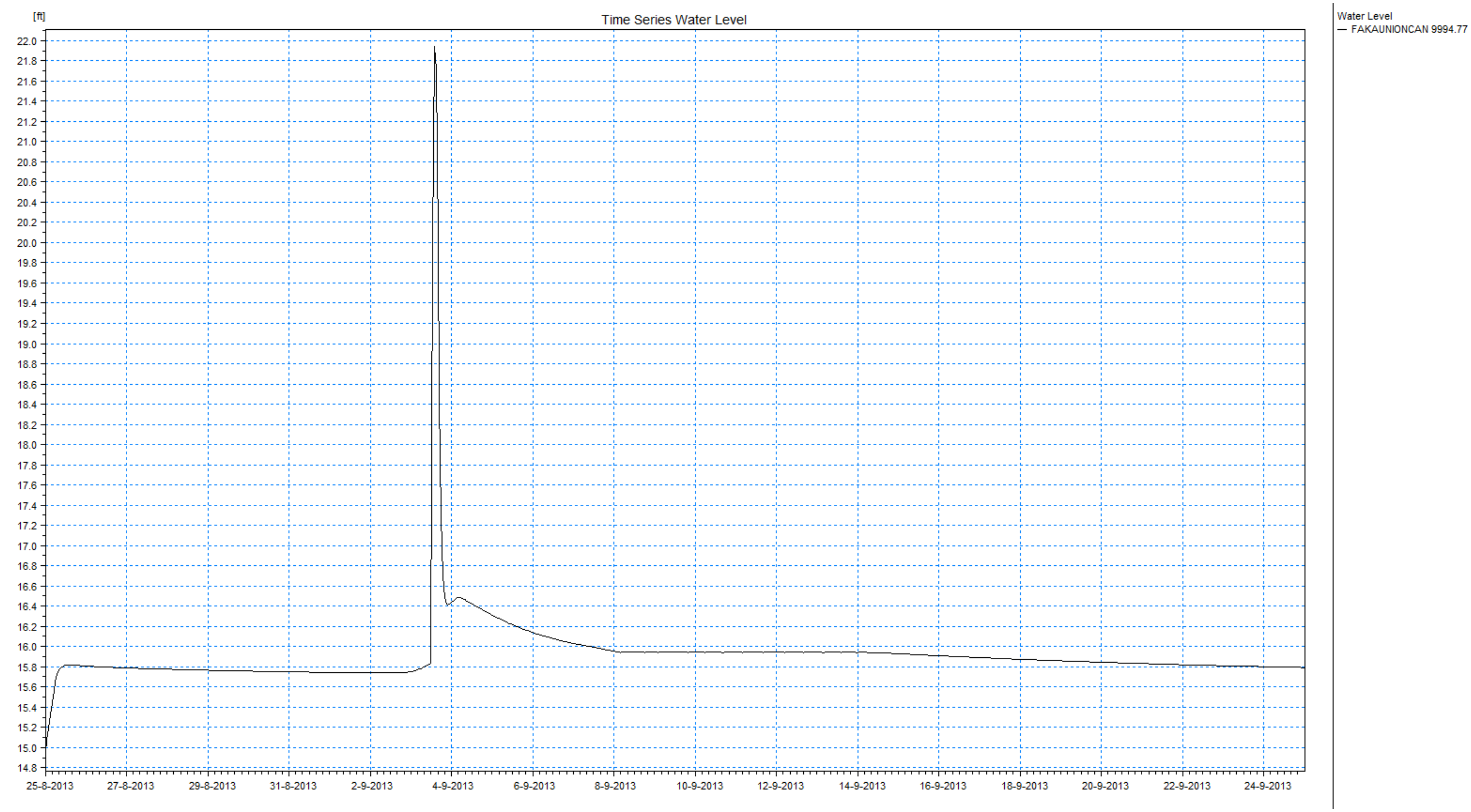


Figure 22. FU7 Discharge (cfs)



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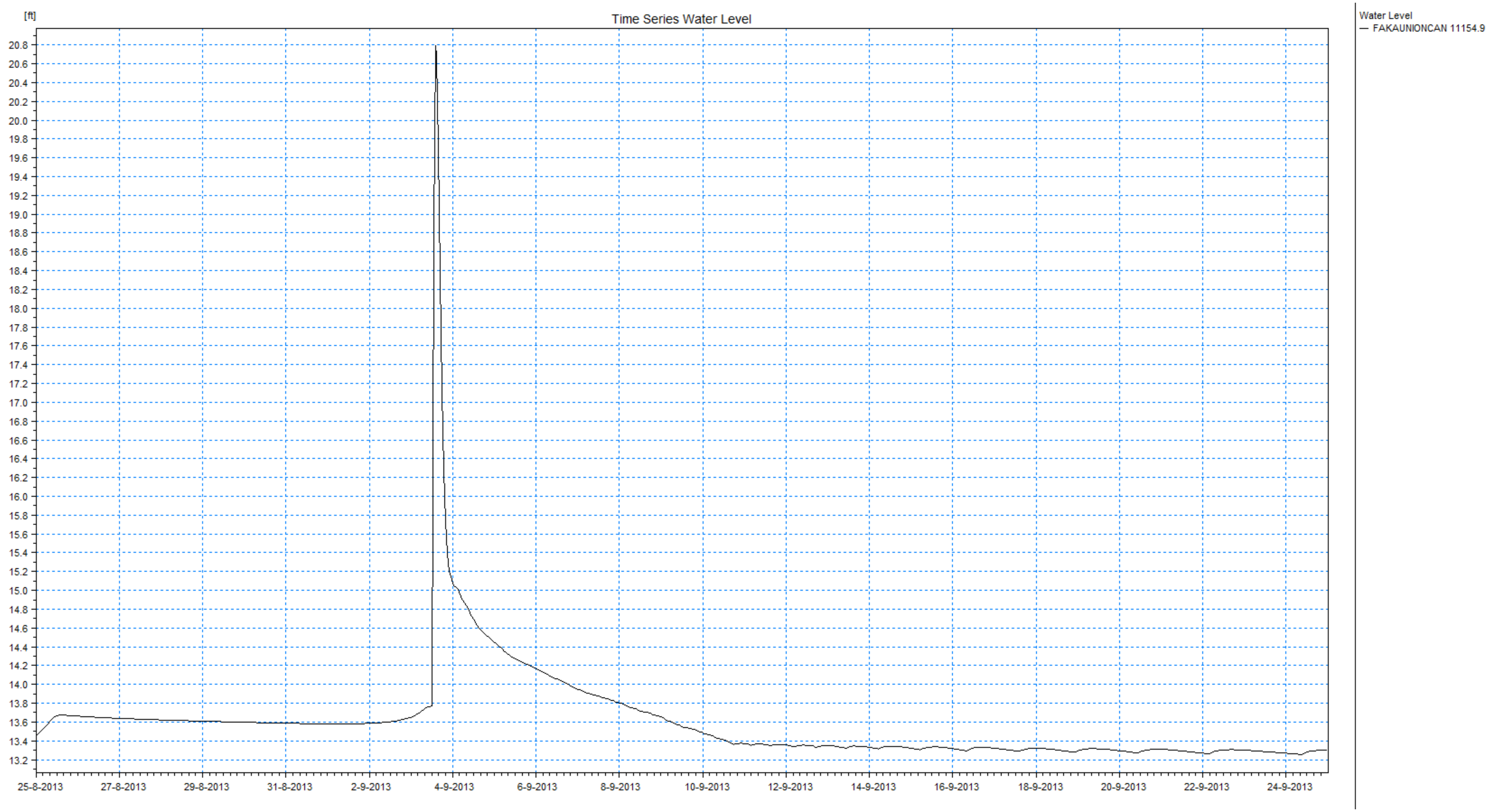

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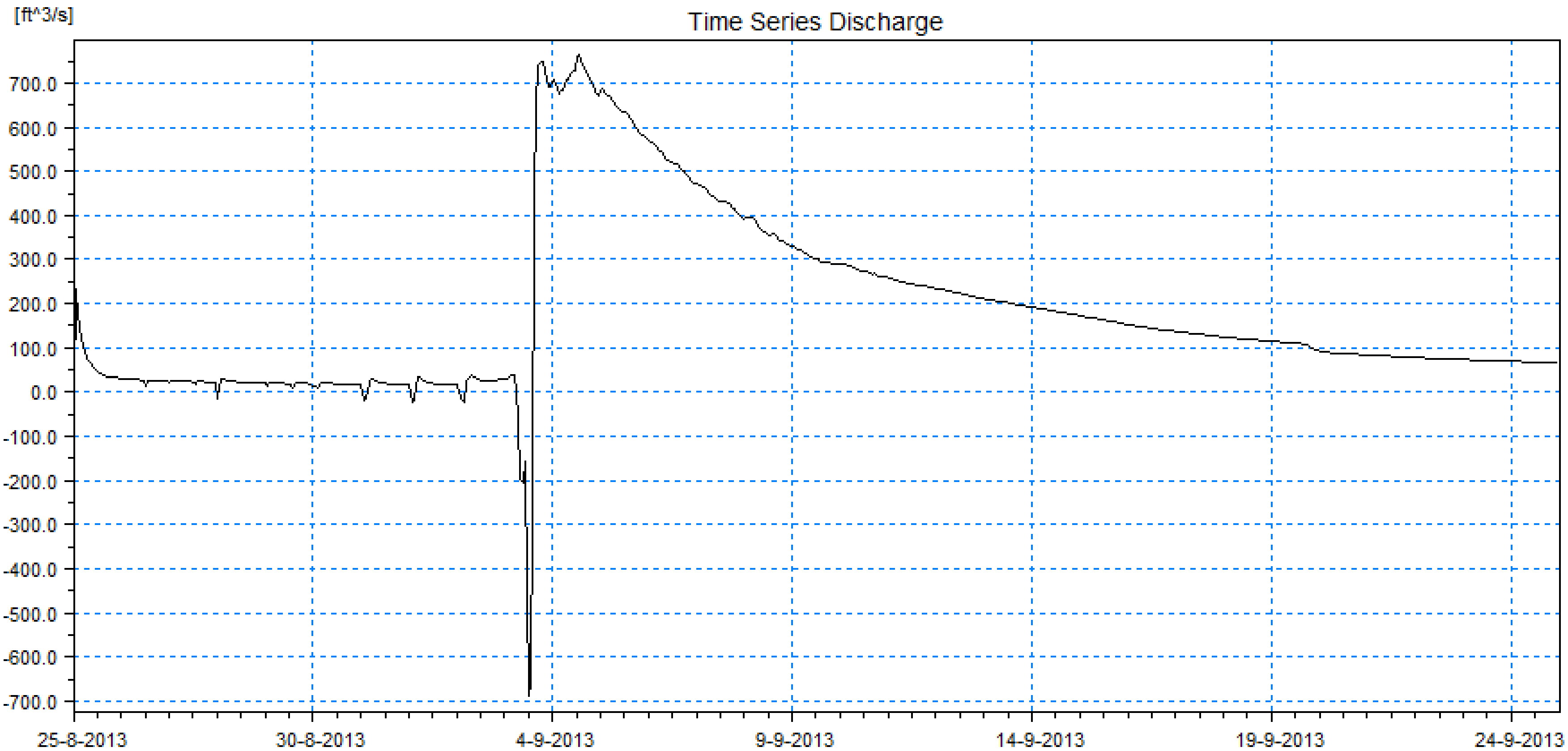


Figure 25. HC1 Discharge (cfs)

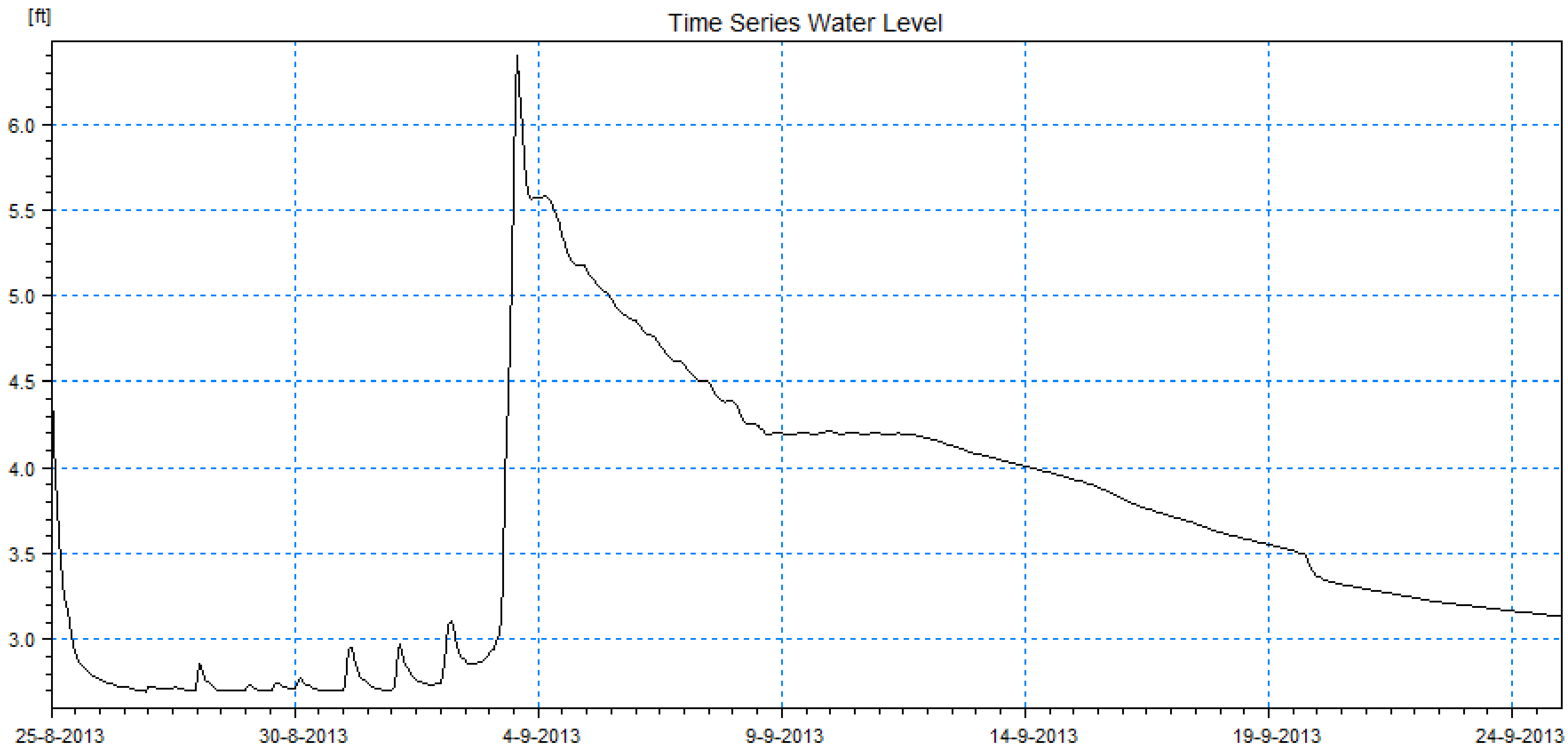


Figure 26. HC1 Headwater Stage (ft)

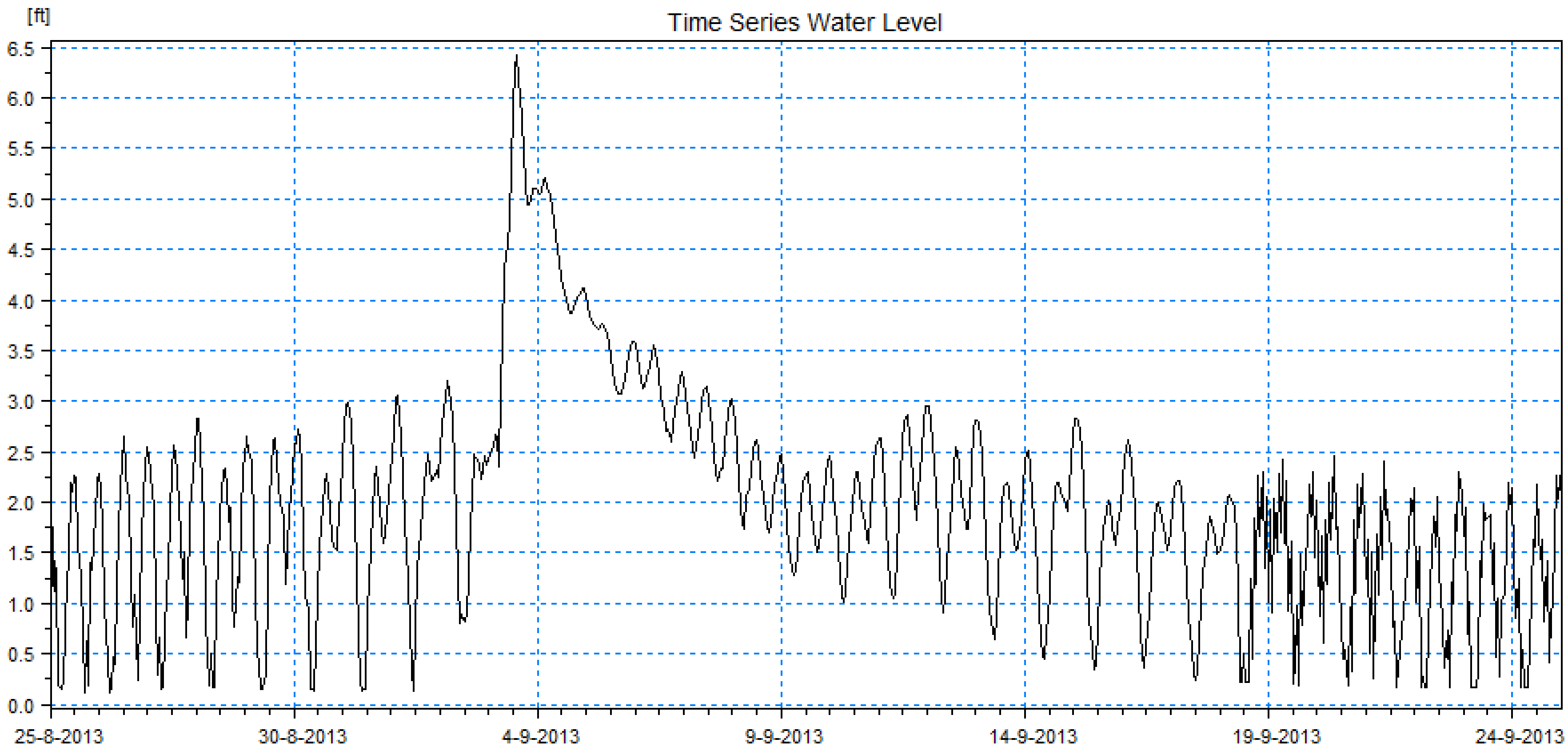


Figure 27. HC1 Tailwater Stage (ft)

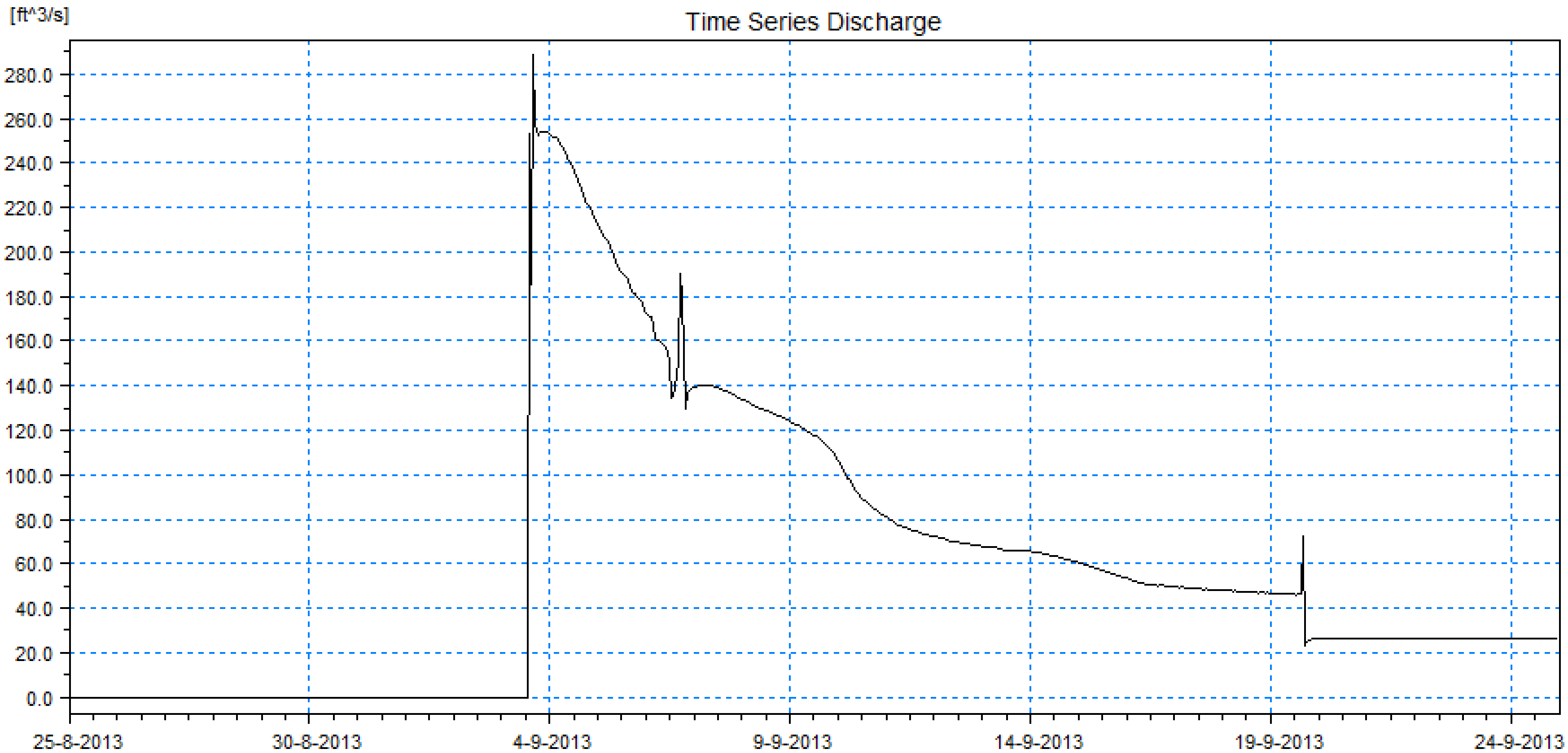
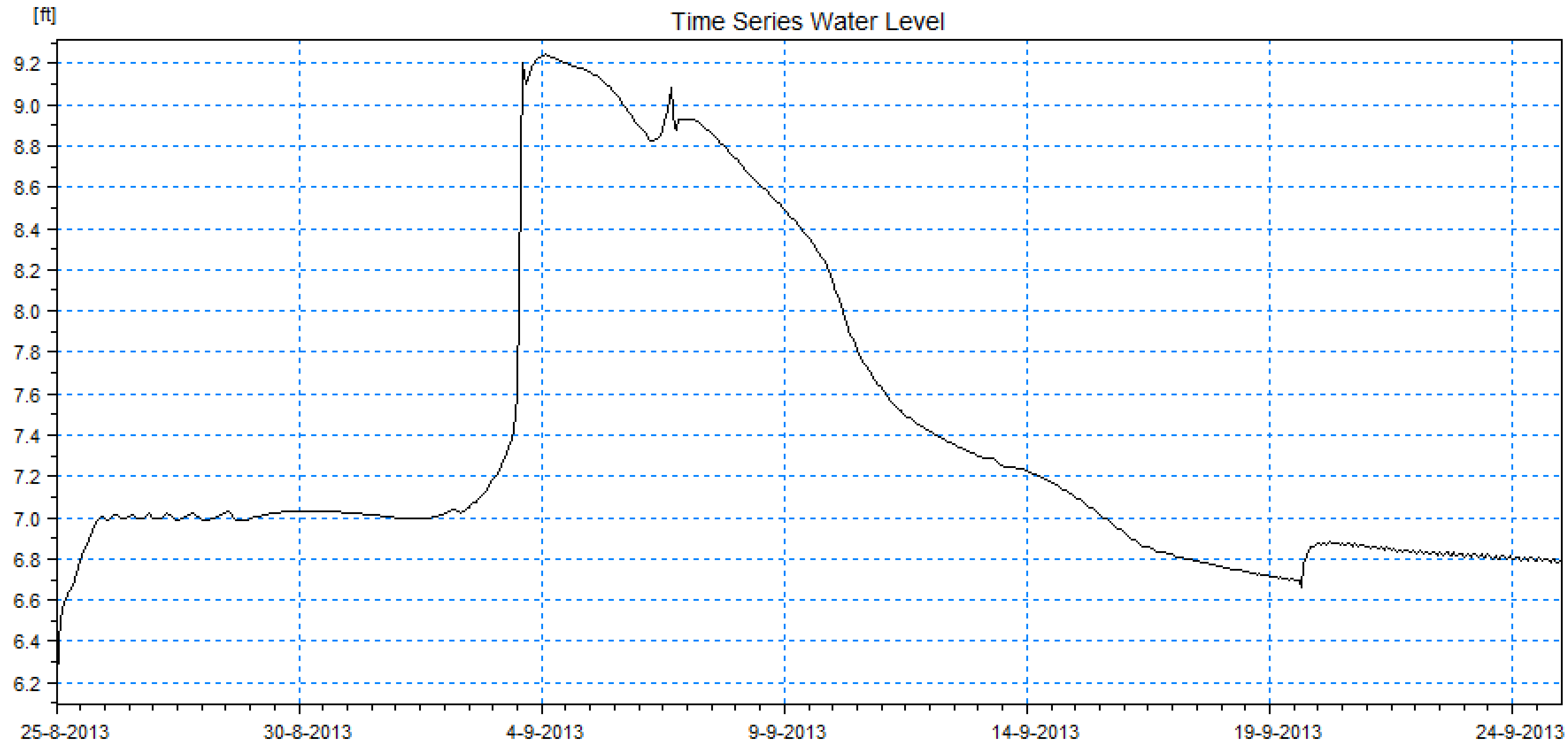


Figure 28. HC2 Discharge (cfs)



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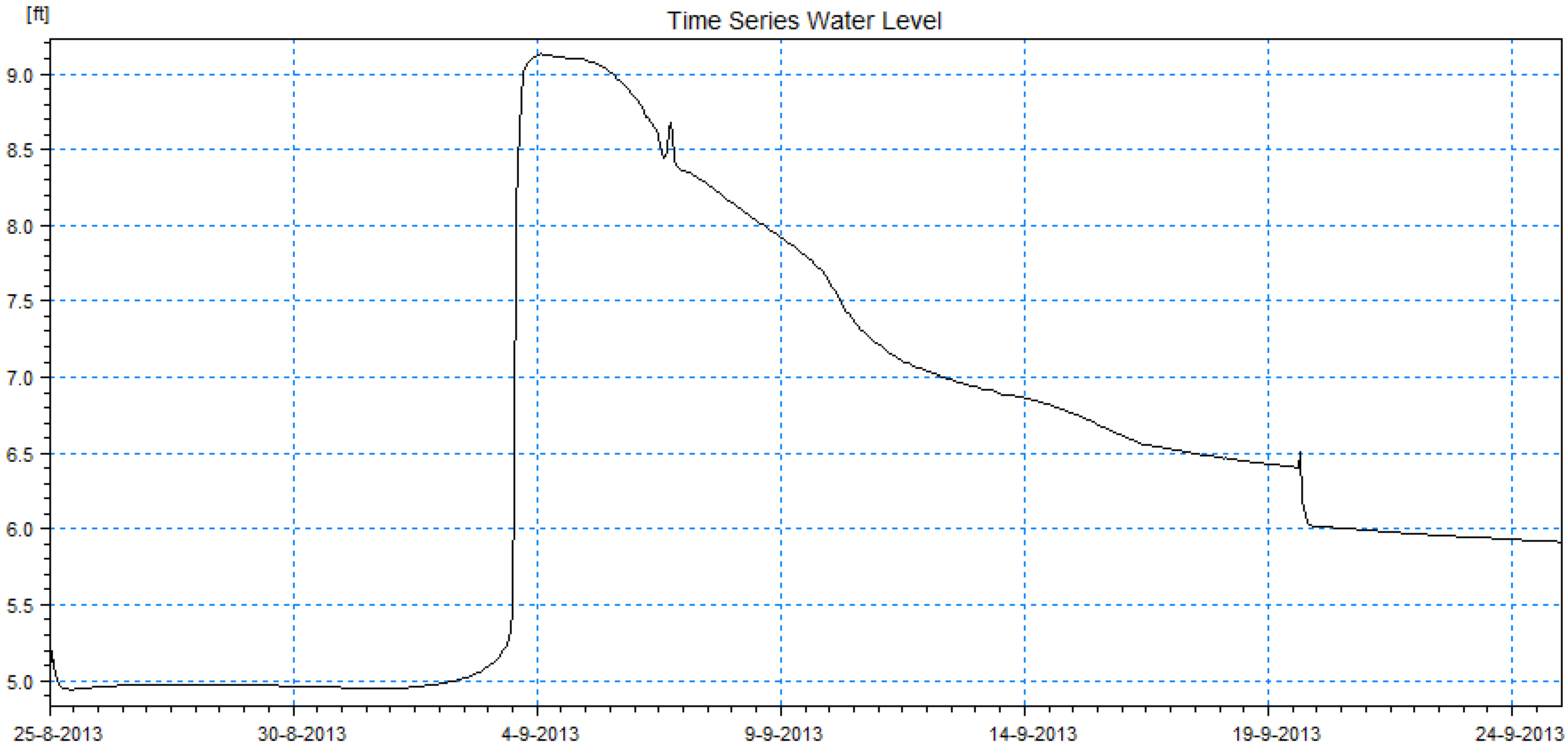

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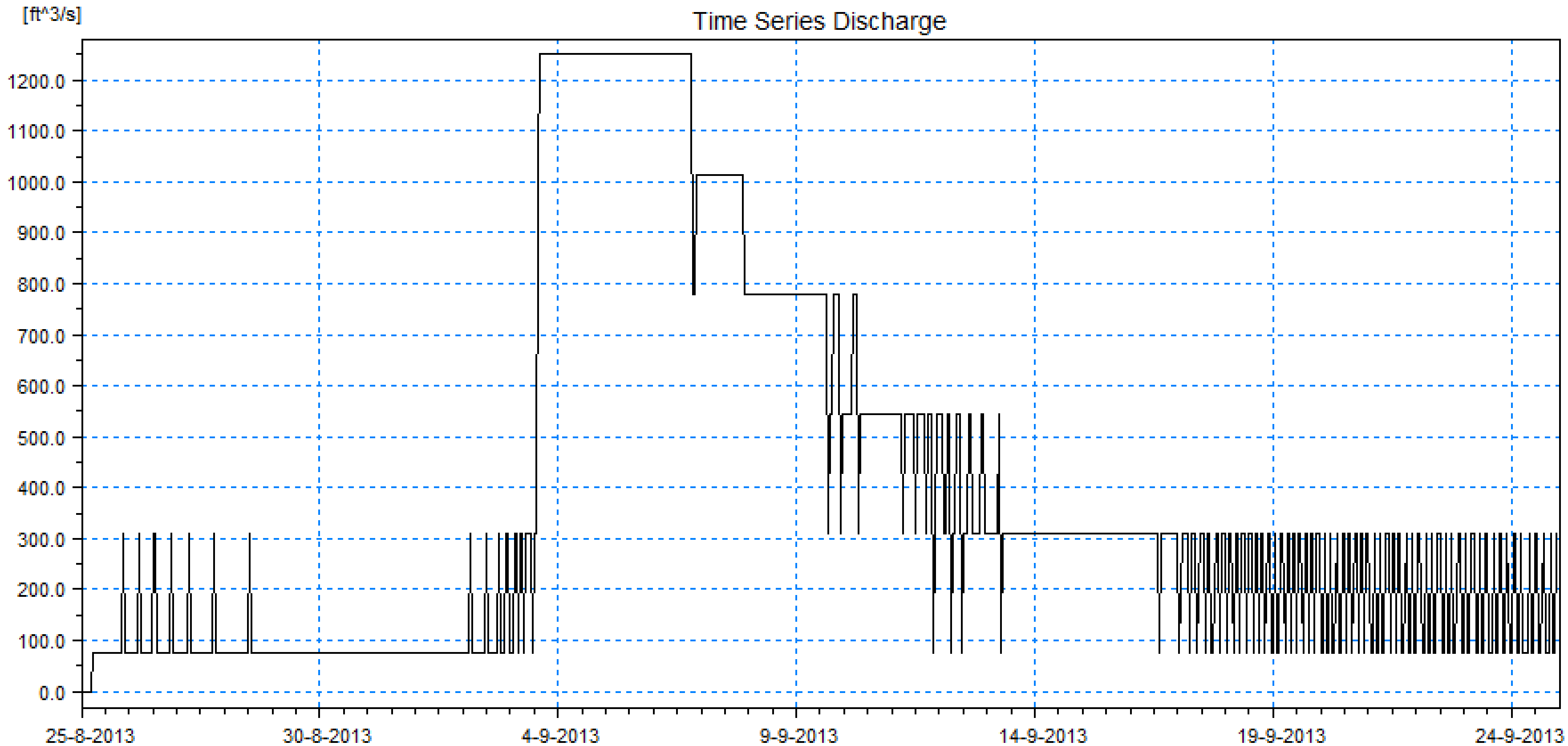


Figure 31. Miller Pump Discharge (cfs)

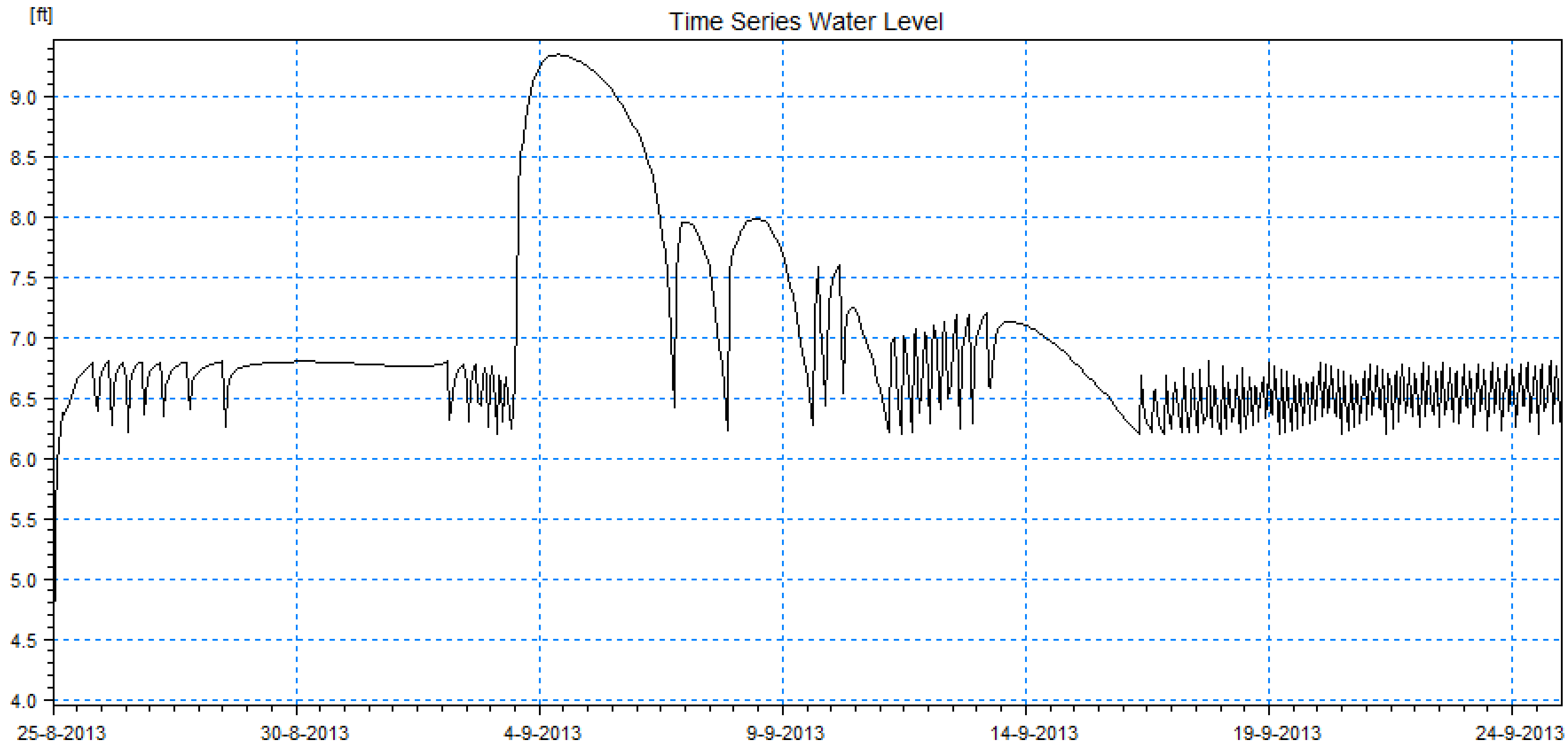


Figure 32. Miller Pump Headwater Stage (ft)

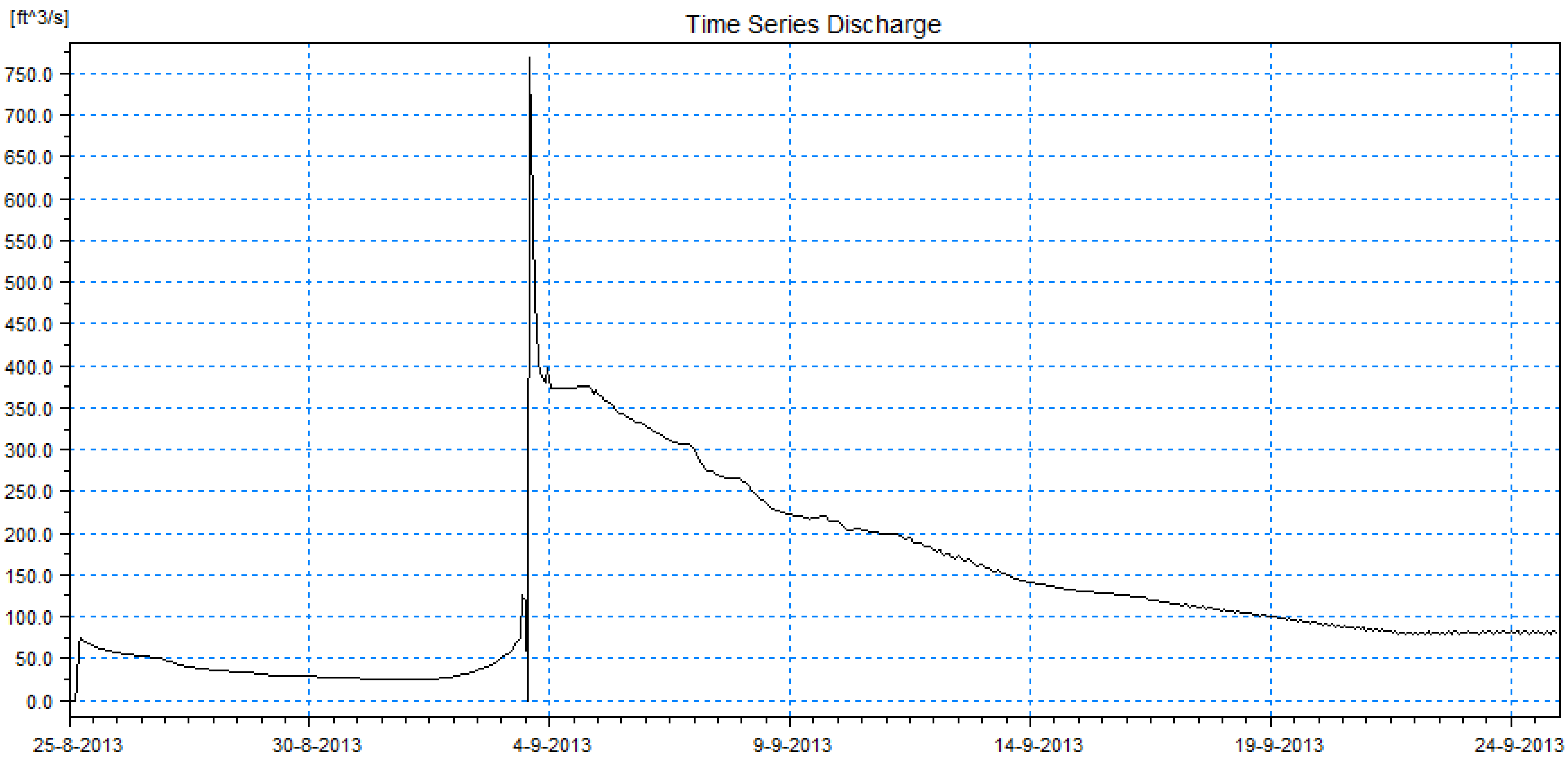
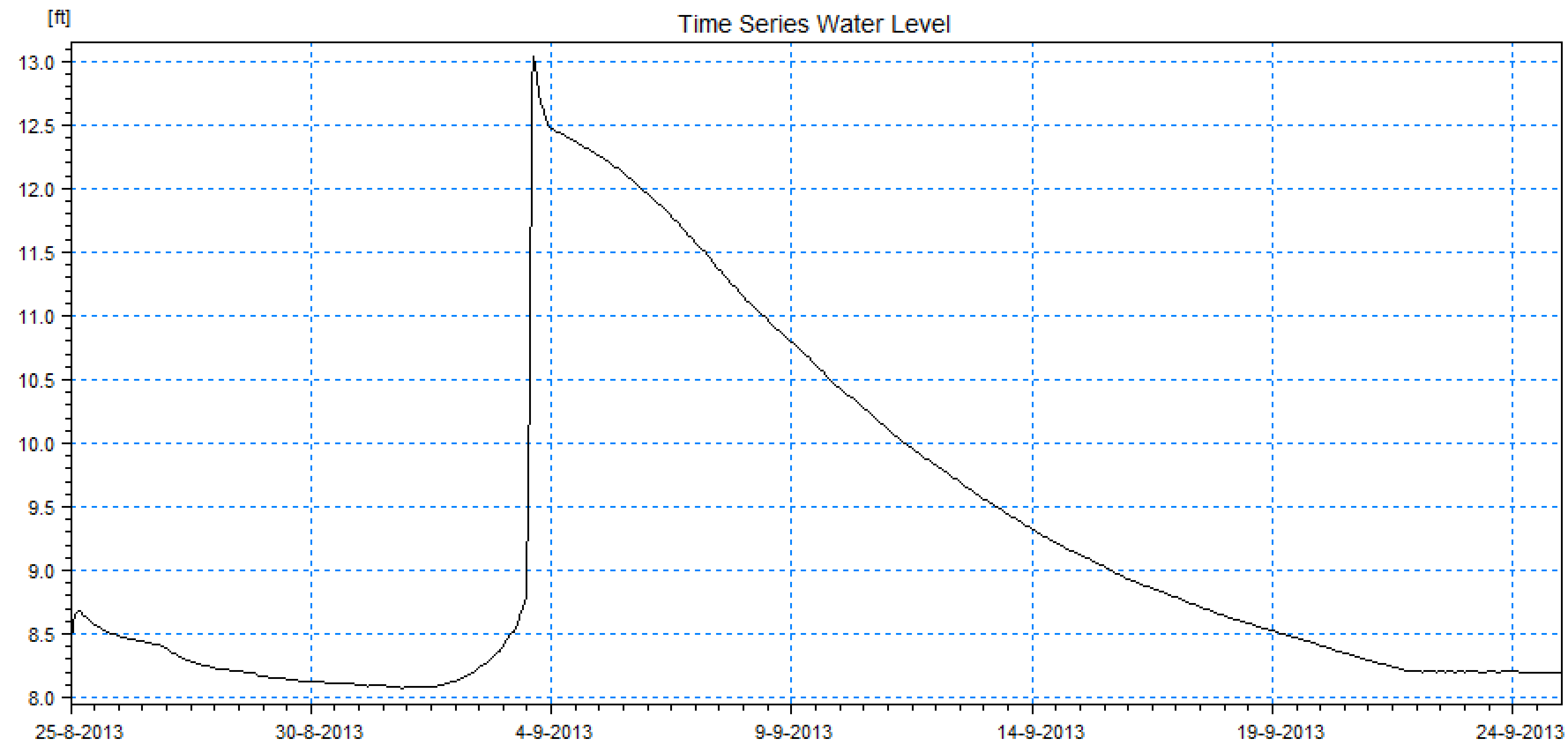


Figure 33. Miller3 Discharge (cfs)



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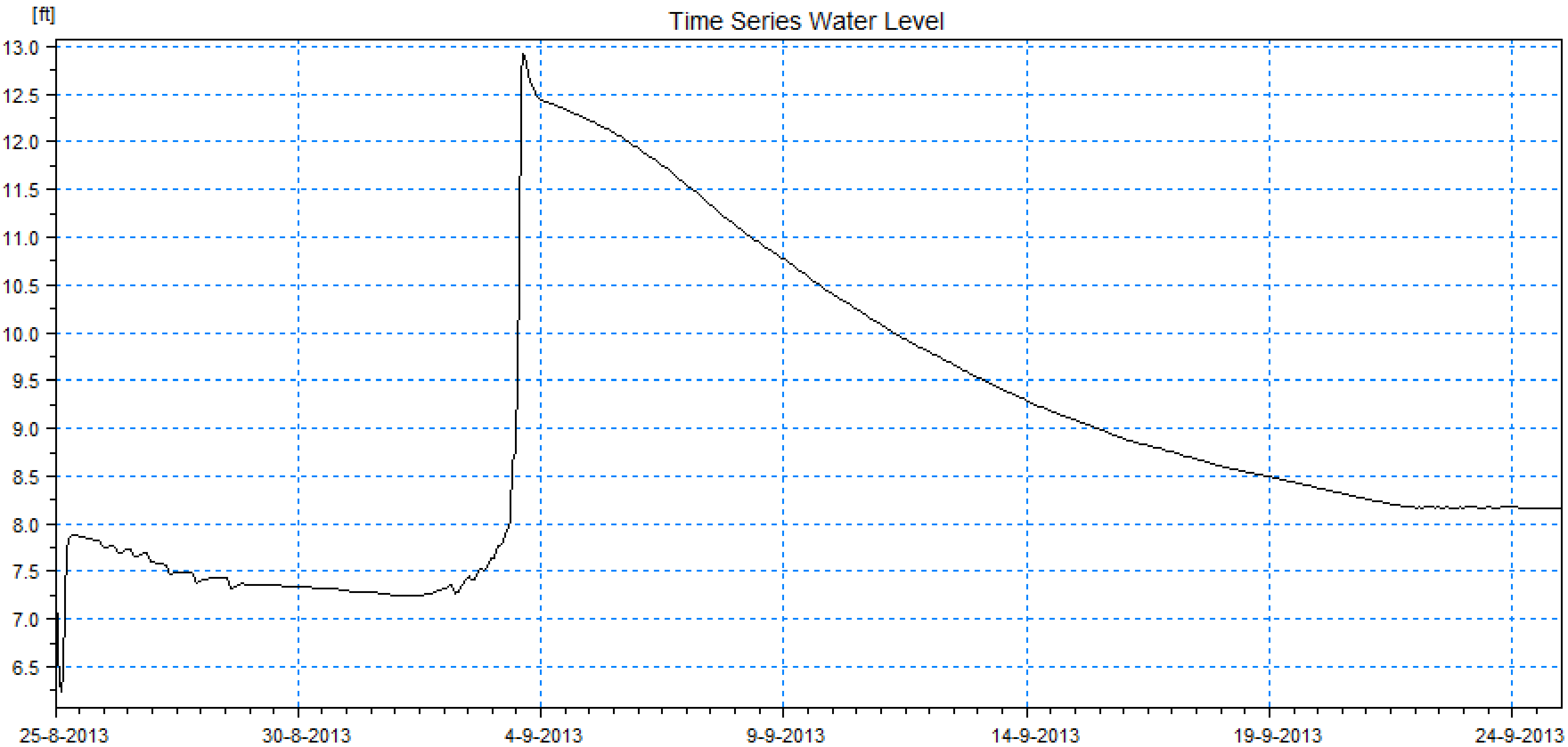

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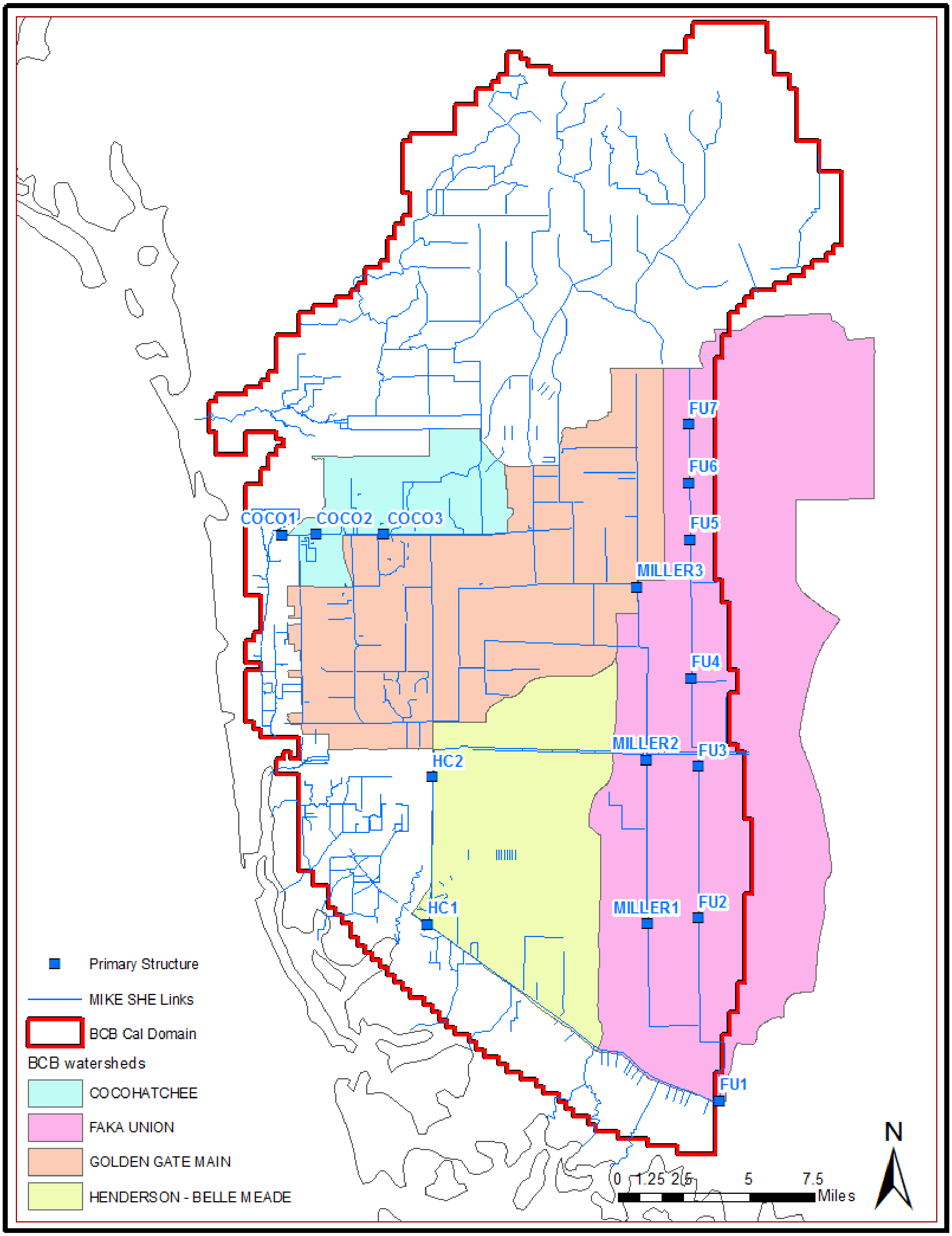

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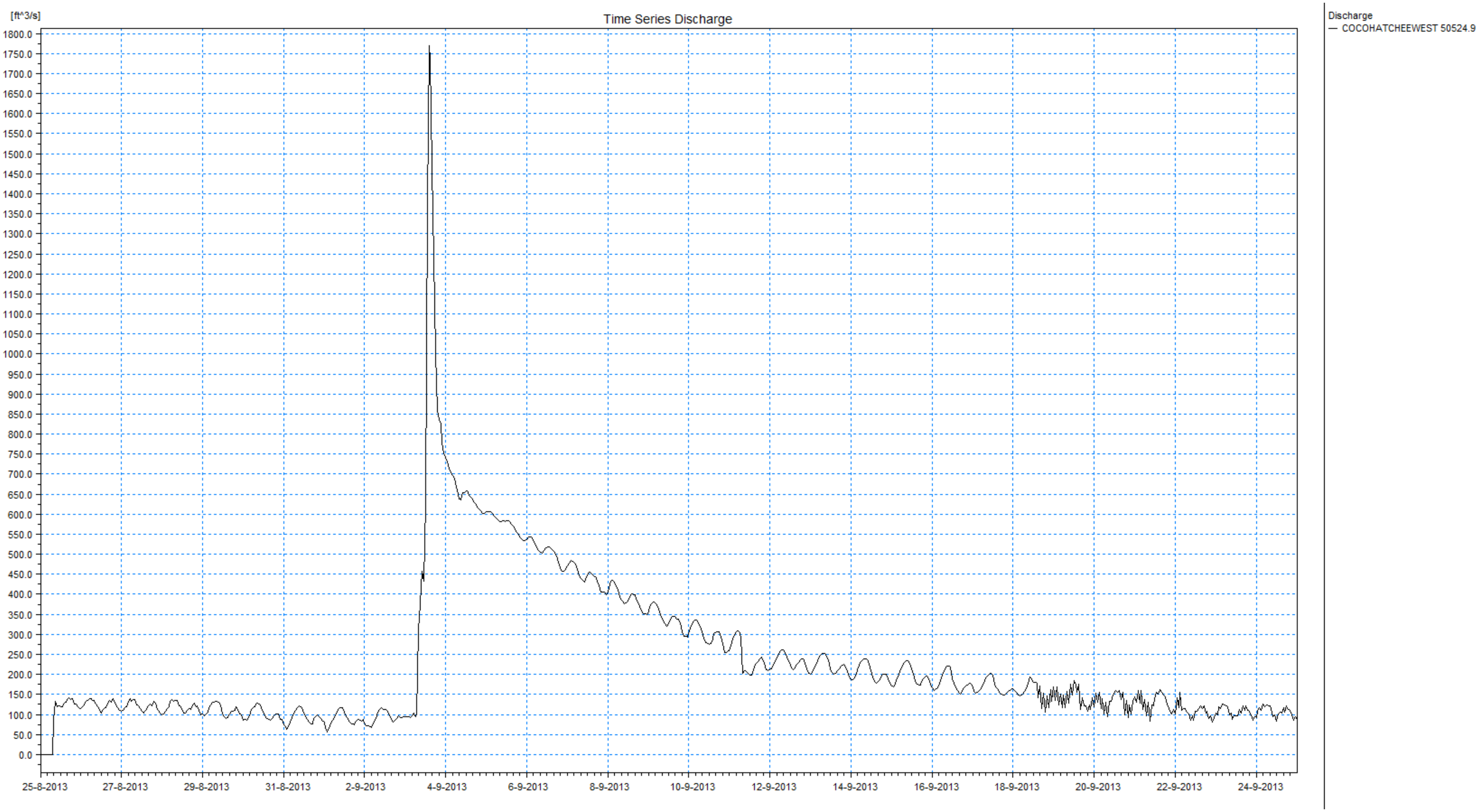


Figure 2. COCO1 Discharge (cfs)

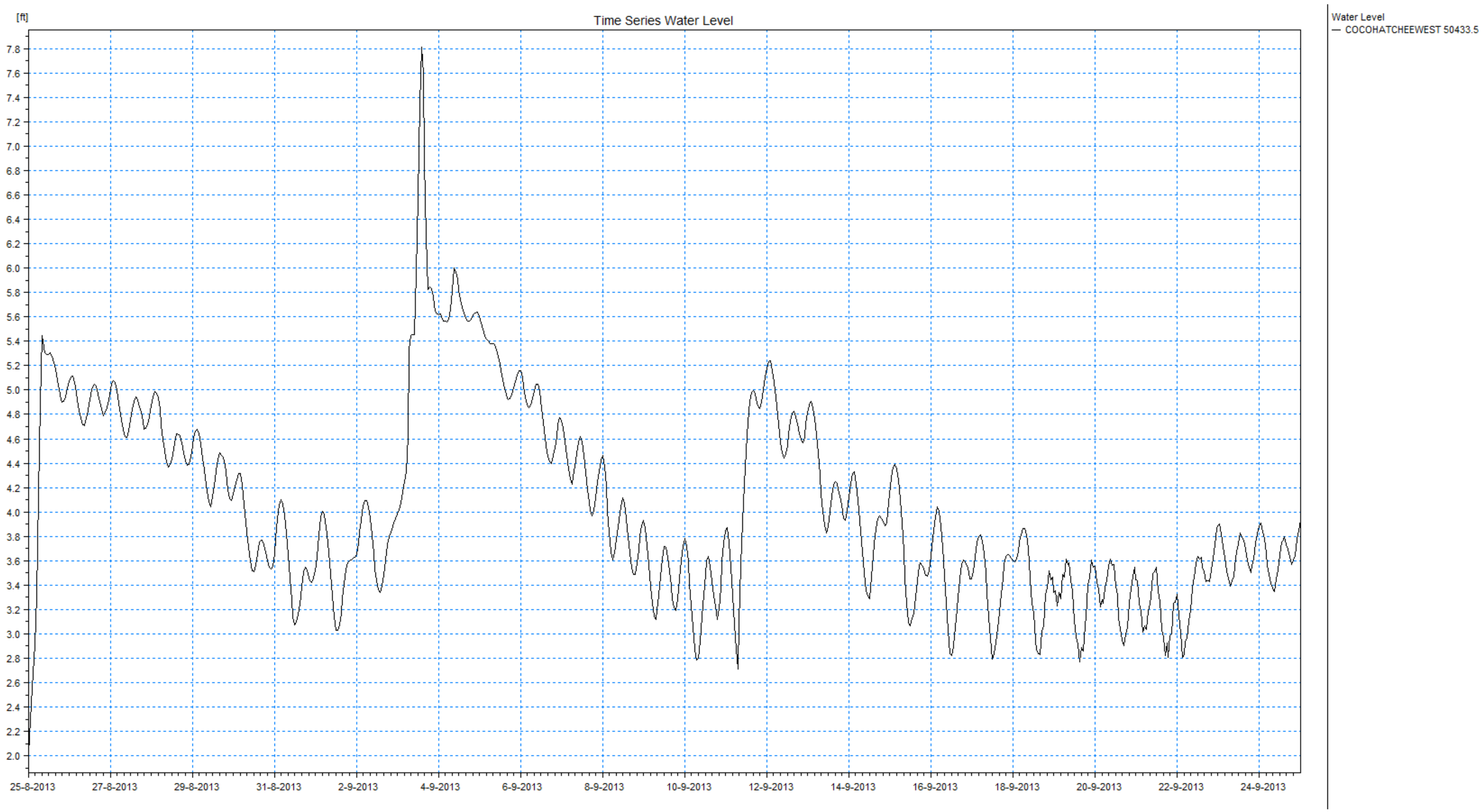


Figure 3. COCO1 Headwater Stage (ft)

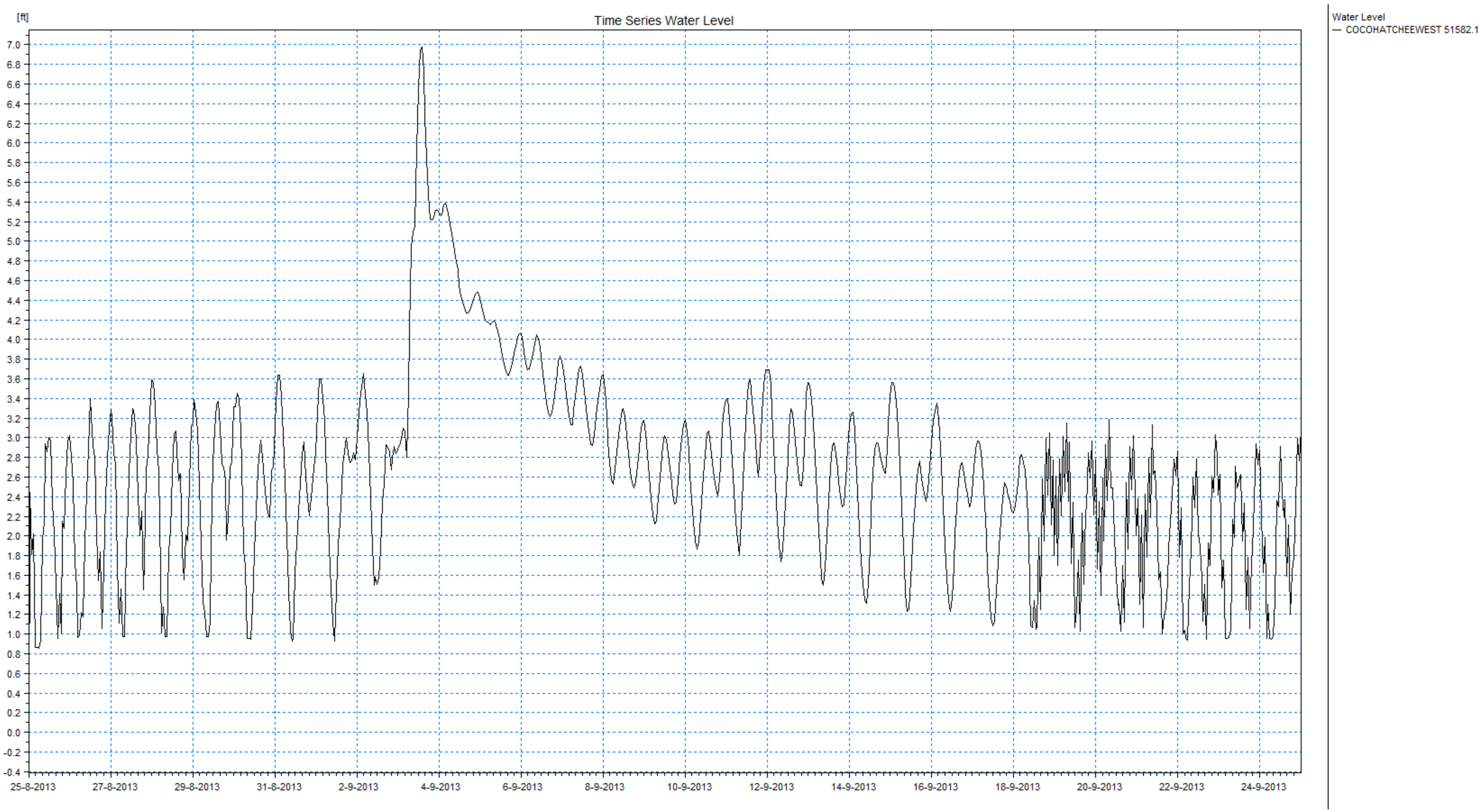


Figure 4. COCO1 Tailwater Stage (ft)

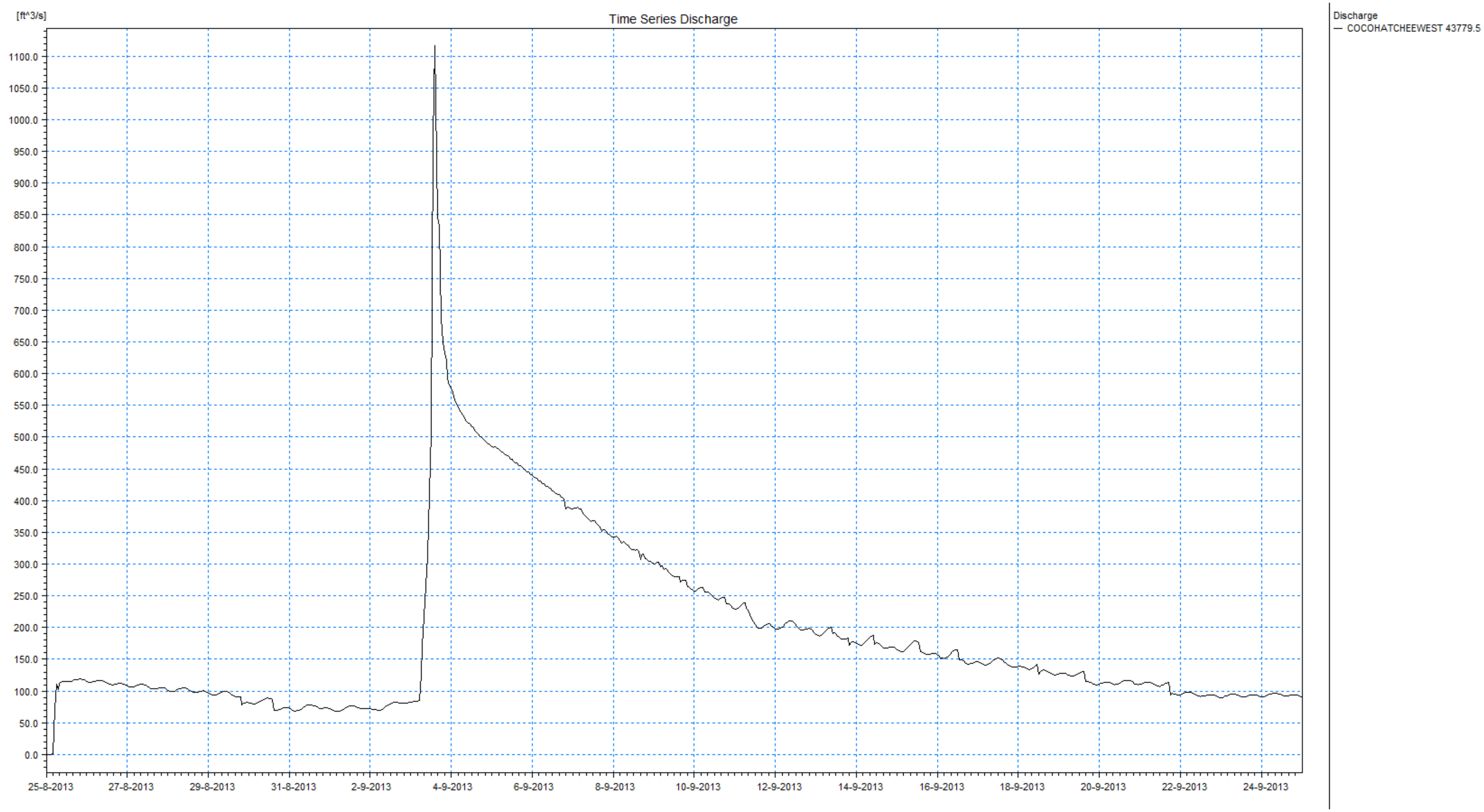


Figure 5. COCO2 Discharge (cfs)

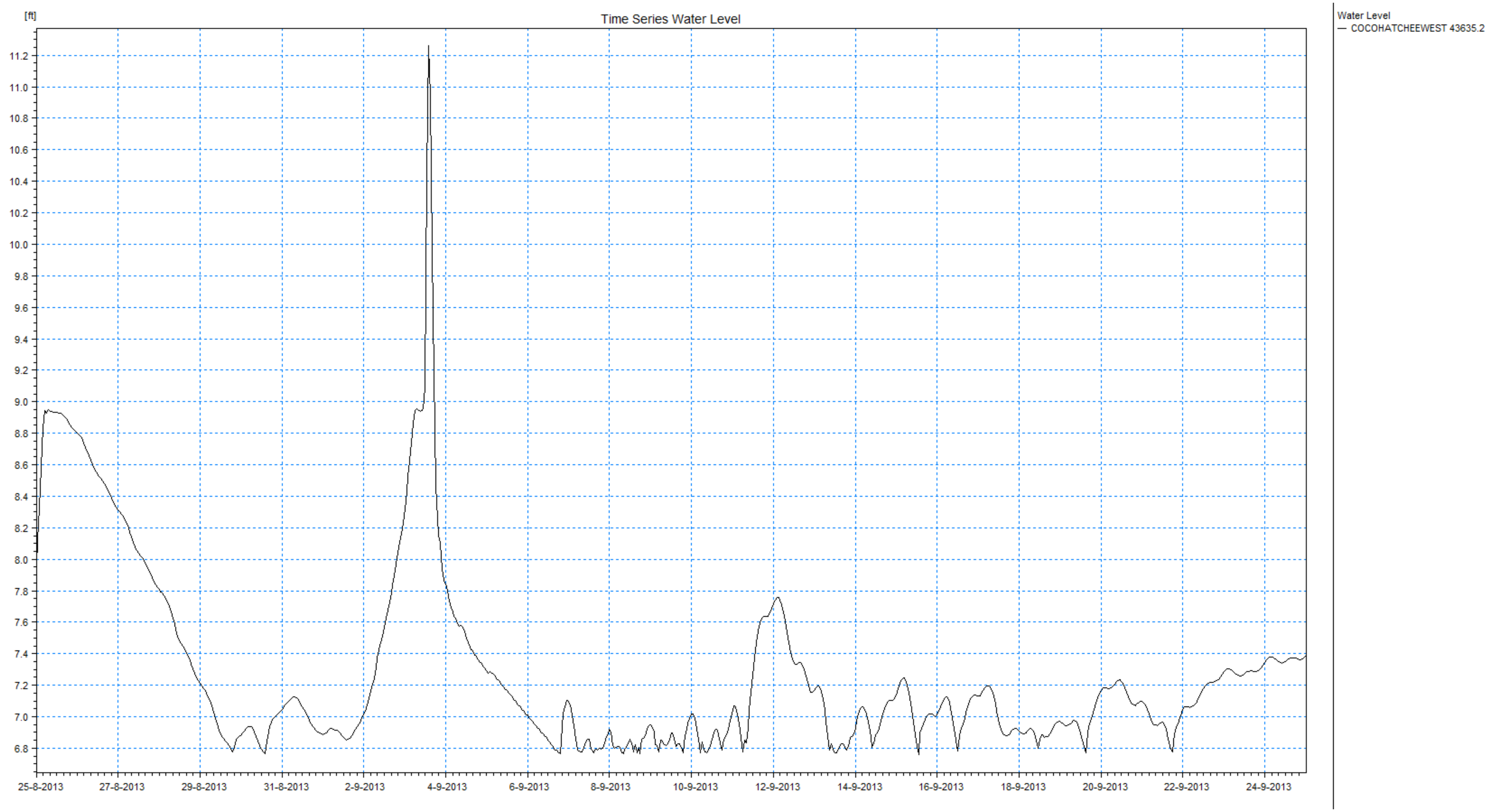


Figure 6. COCO2 Headwater Stage (ft)

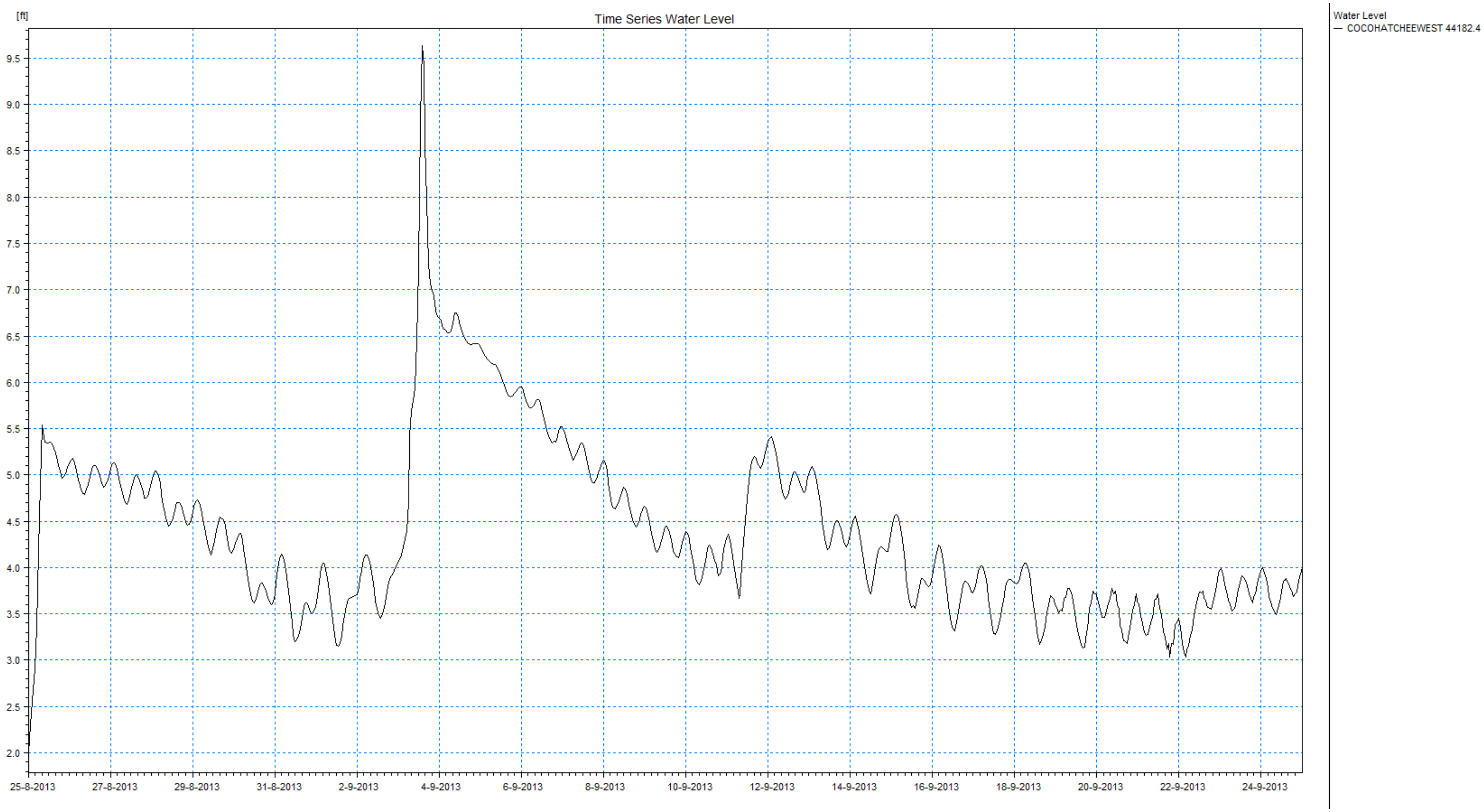


Figure 7. COCO2 Tailwater Stage (ft)

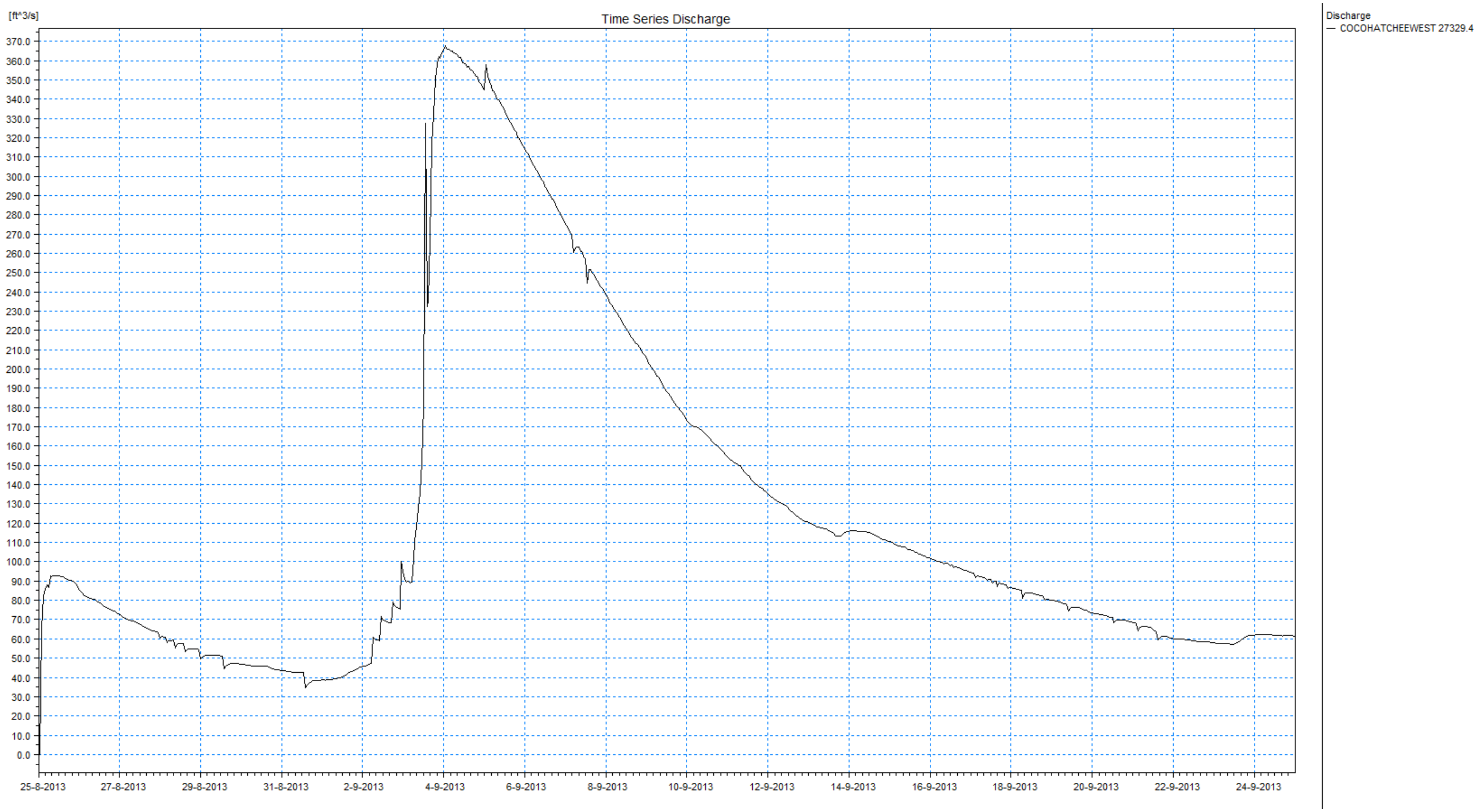
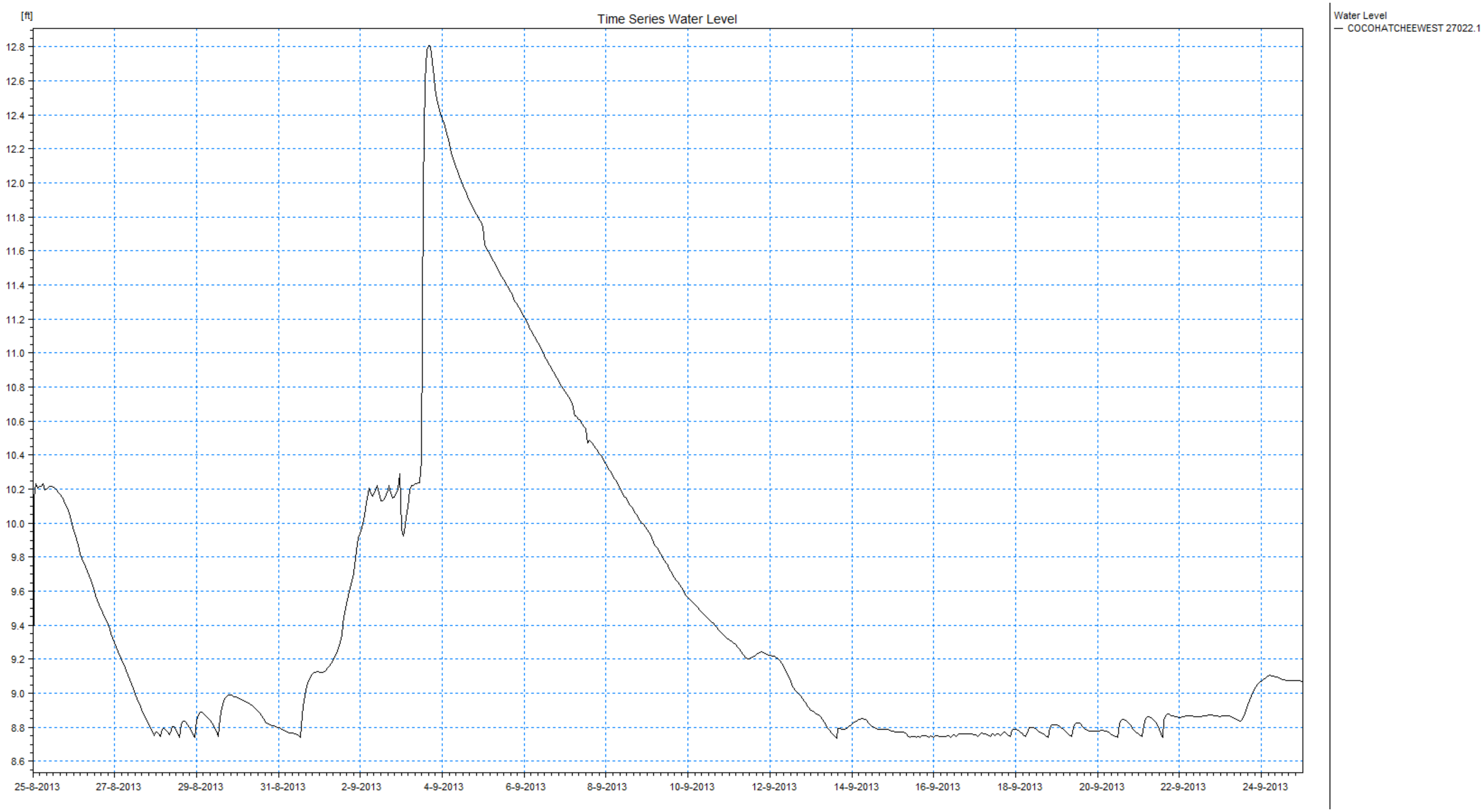


Figure 8. COCO3 Discharge (cfs)



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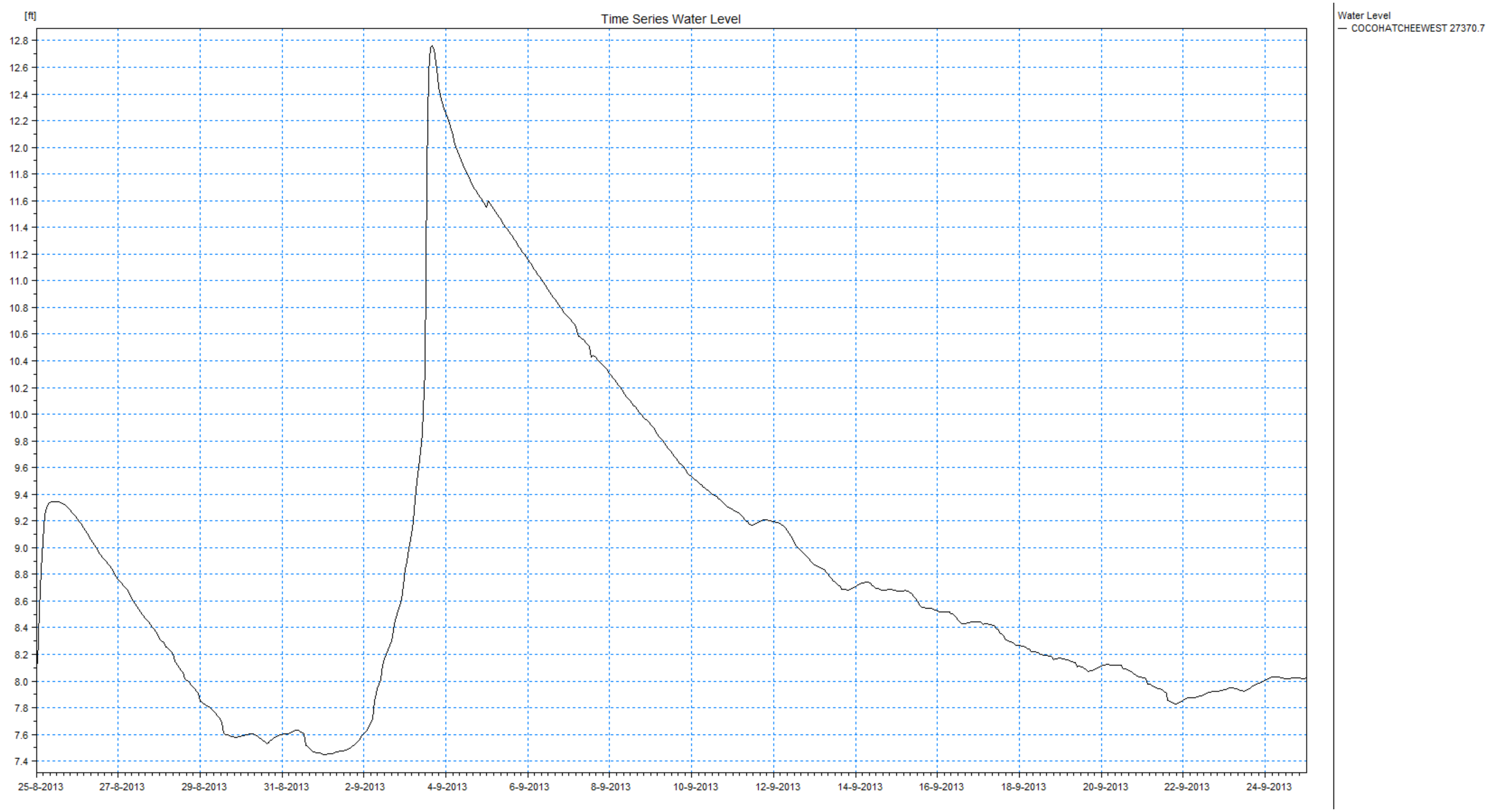
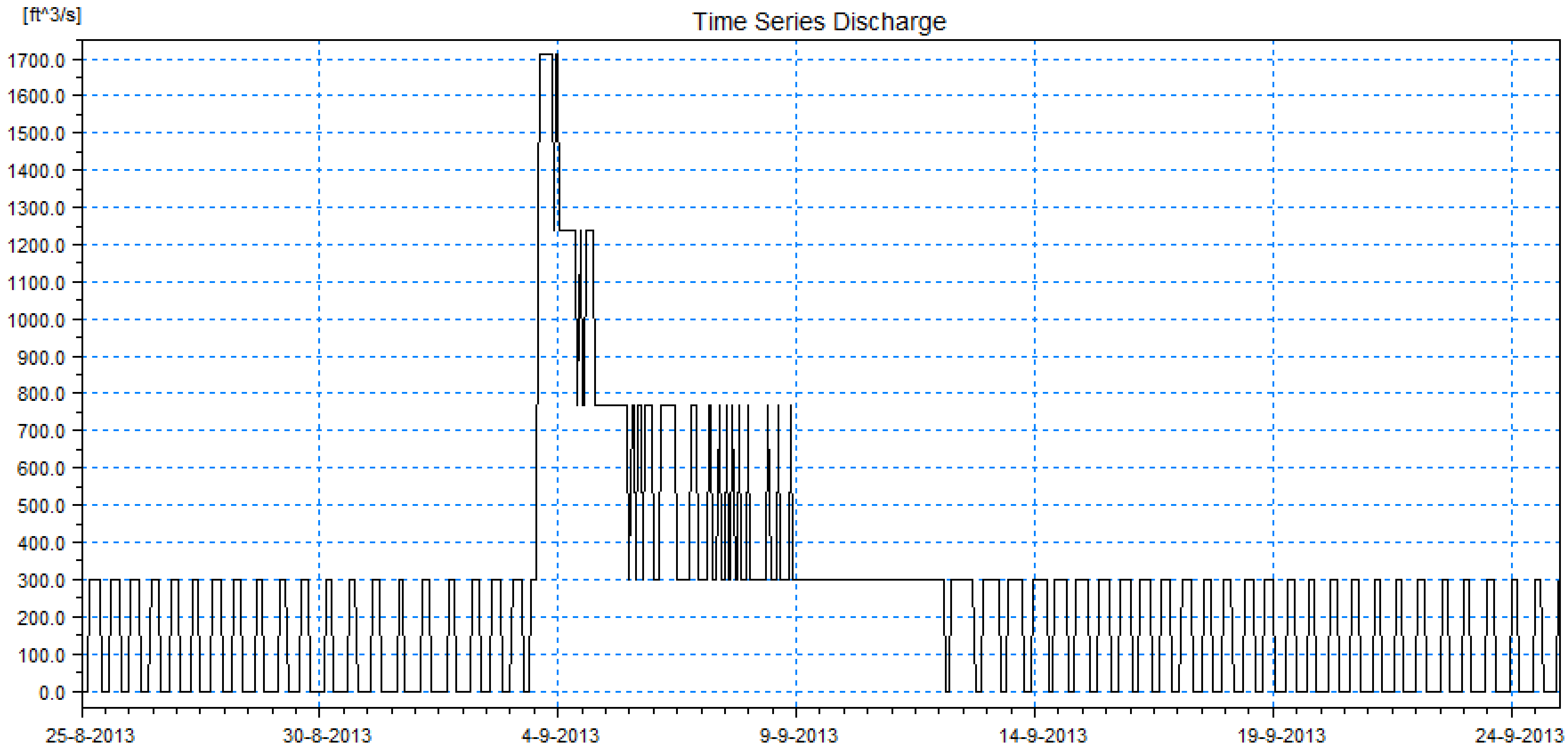


Figure 10. COCO3 Tailwater Stage (ft)



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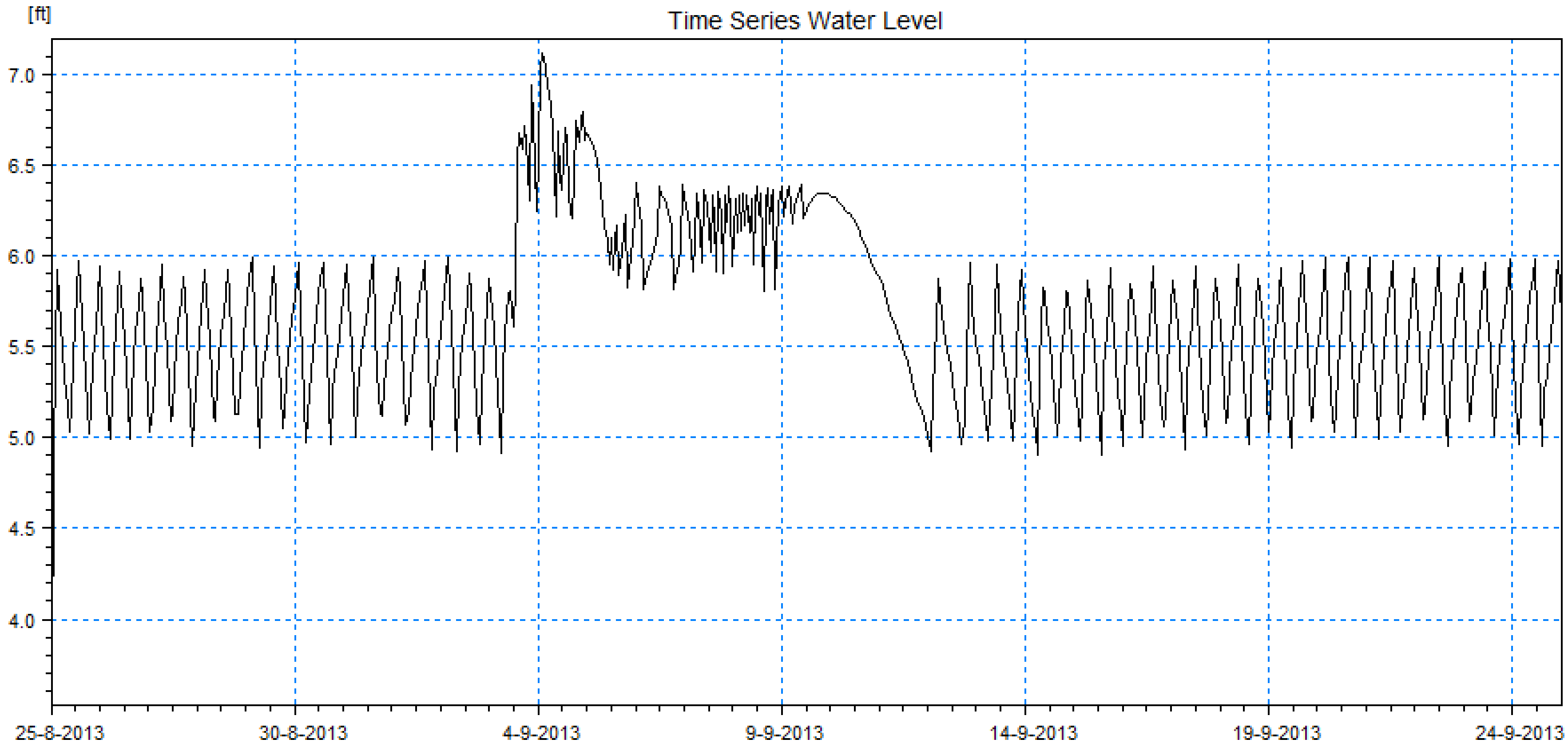


Figure 12. FU Pump Headwater Stage (ft)

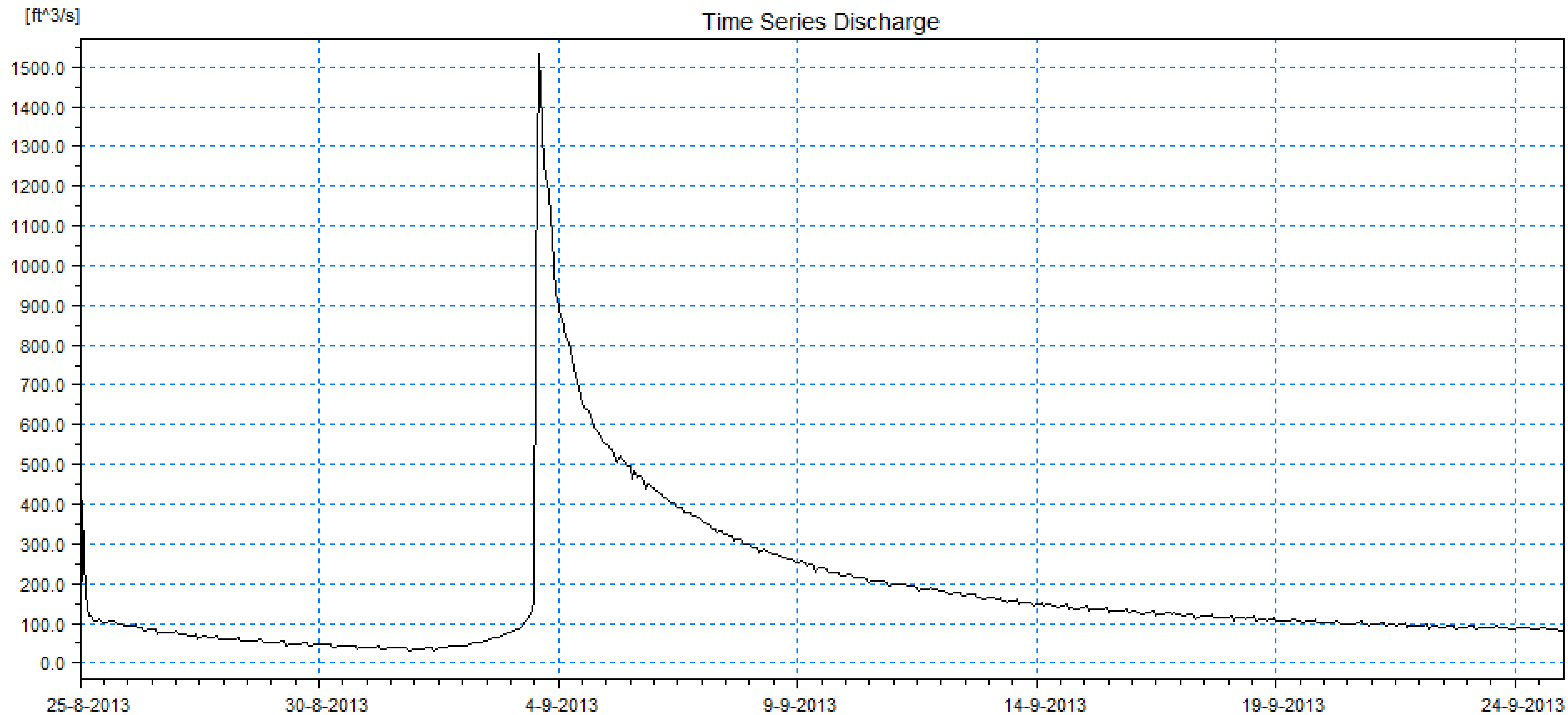
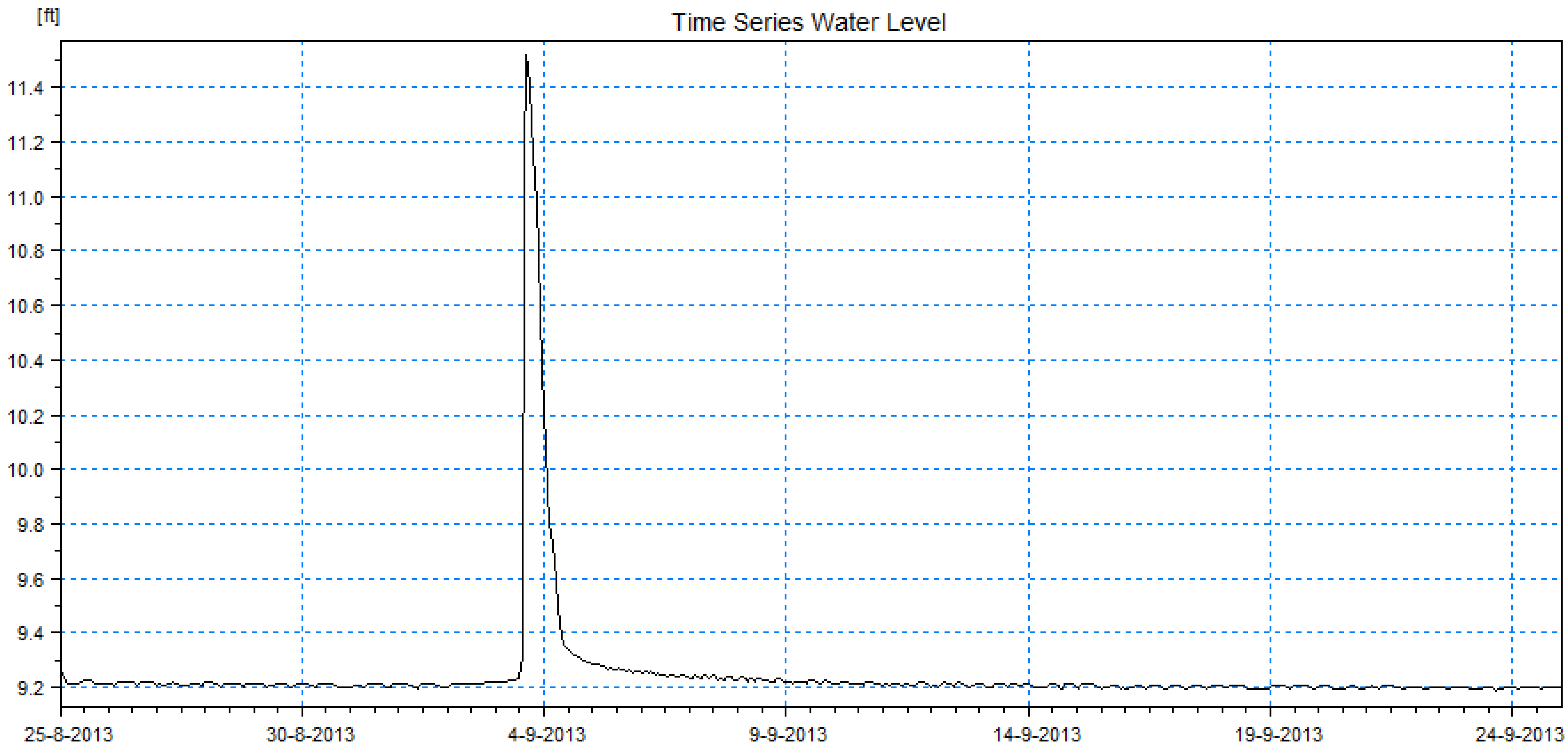


Figure 13. FU4 Discharge (cfs)



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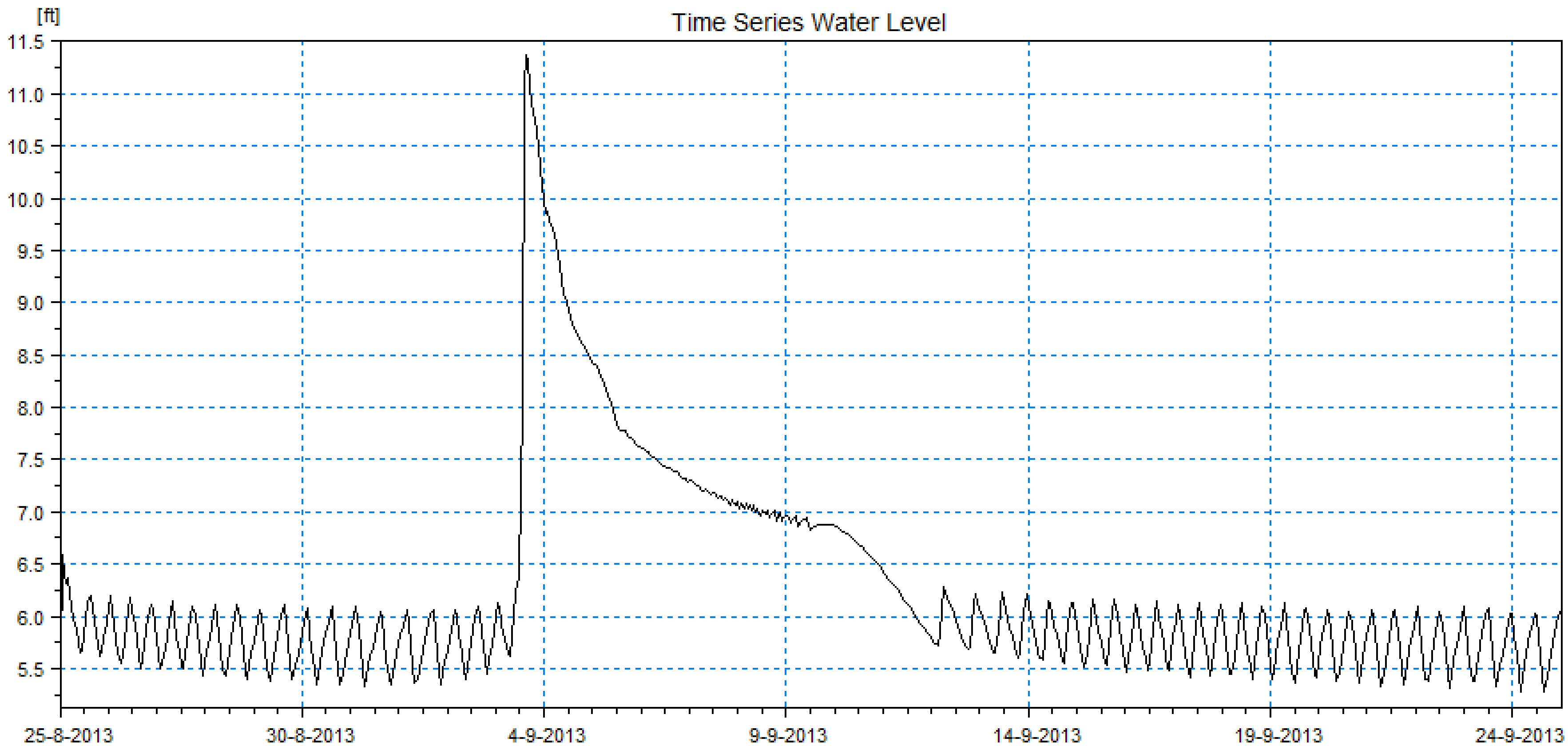


Figure 15. FU4 Tailwater Stage (ft)

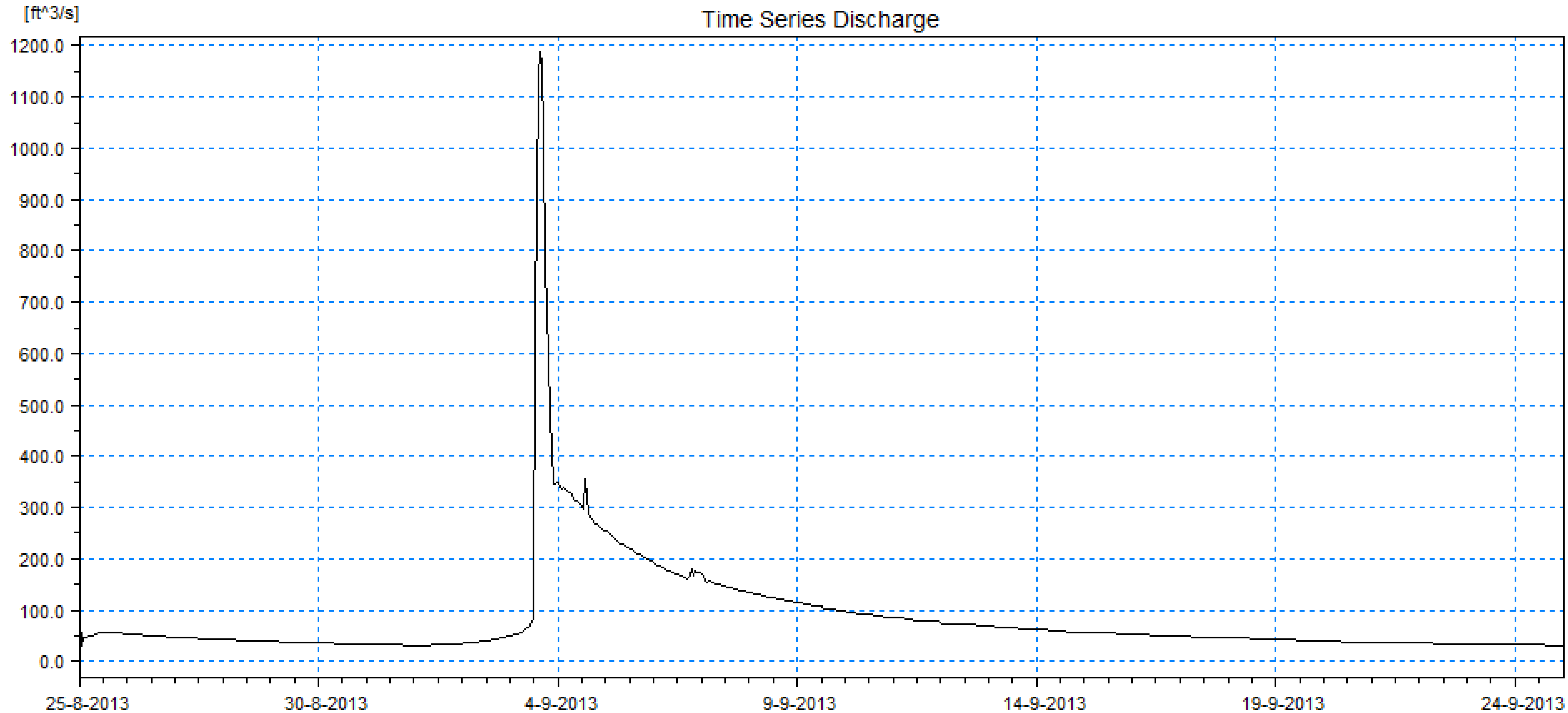


Figure 16. FU5 Discharge (cfs)

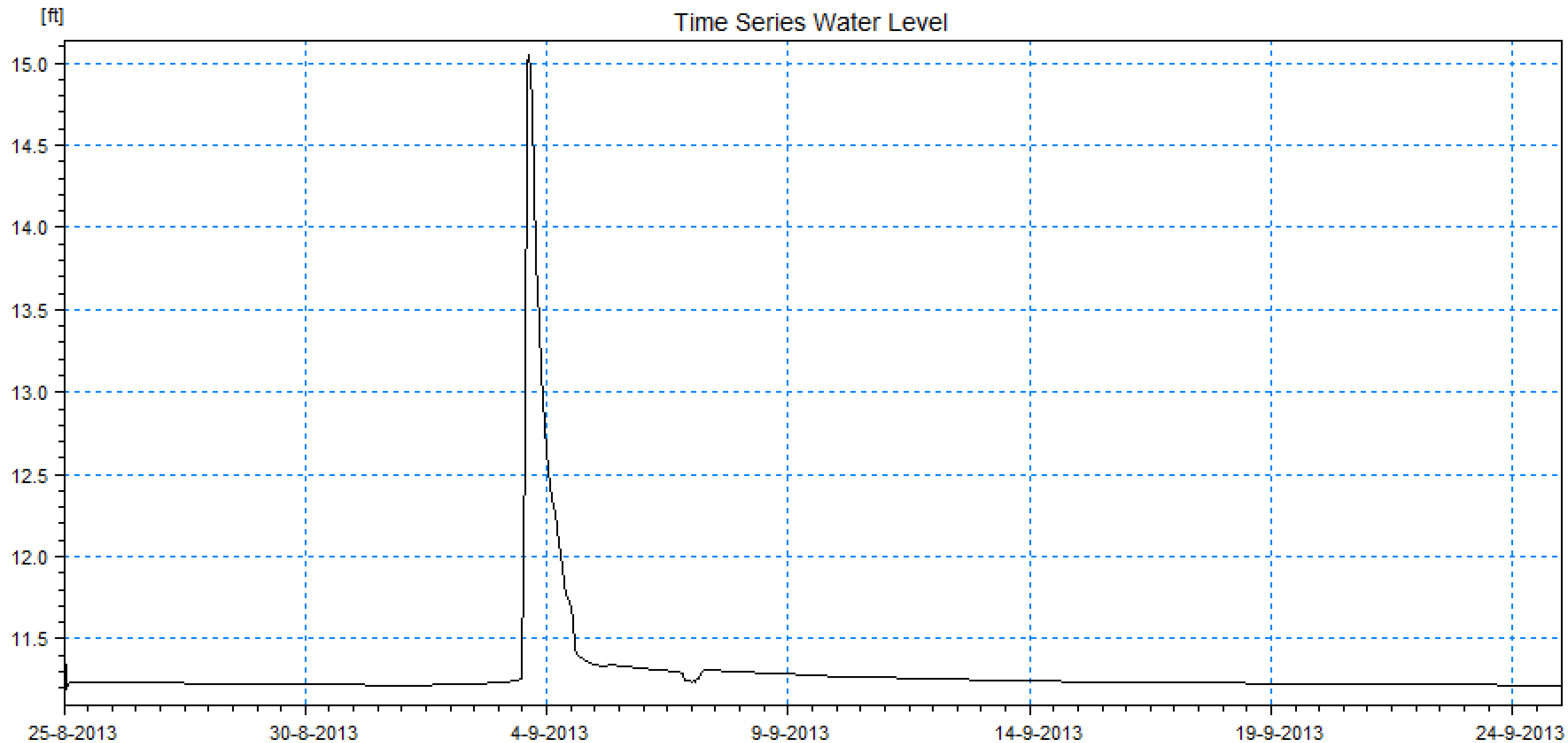
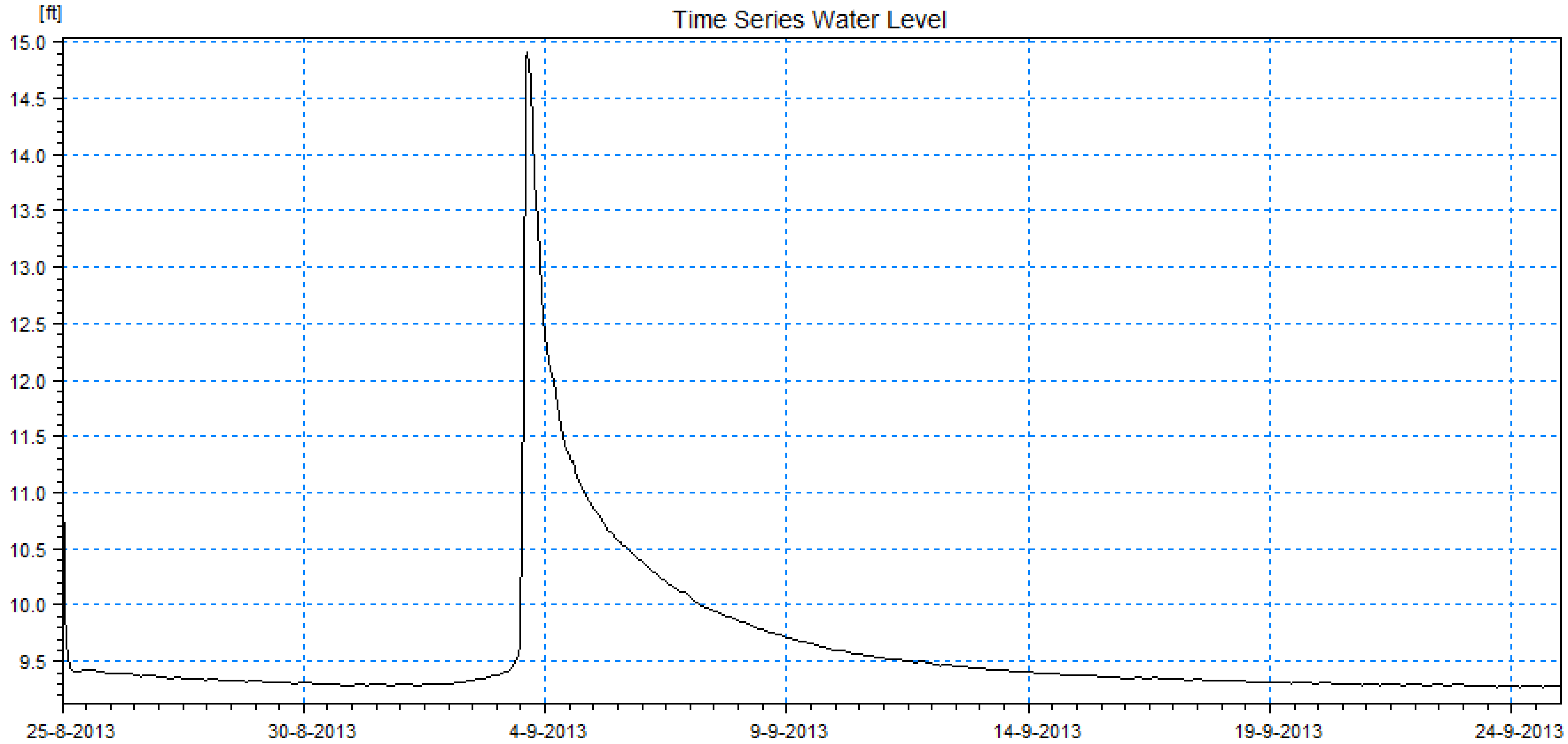


Figure 17. FU5 Headwater Stage (ft)



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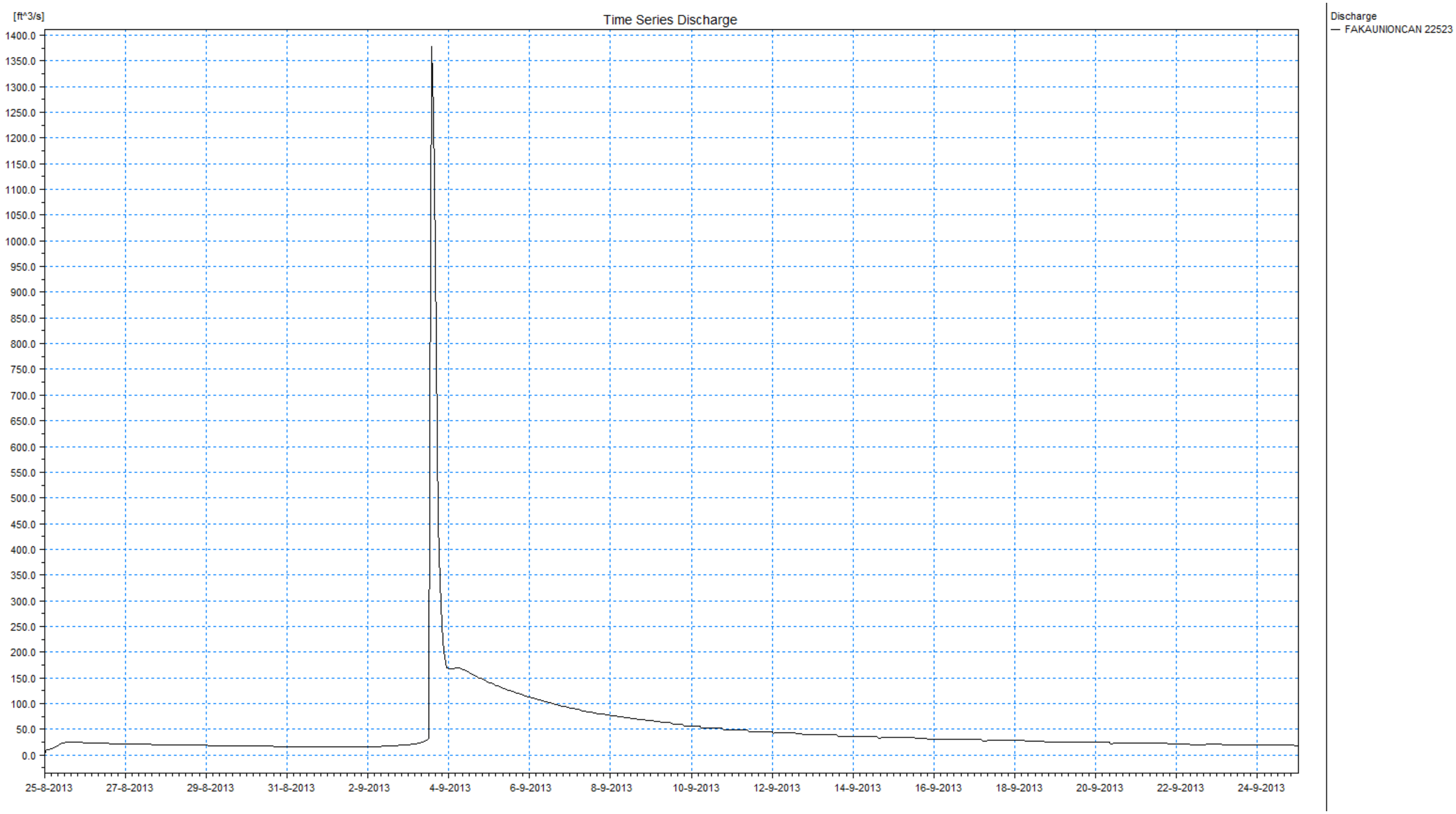

Figure 19. FU6 Discharge (cfs)

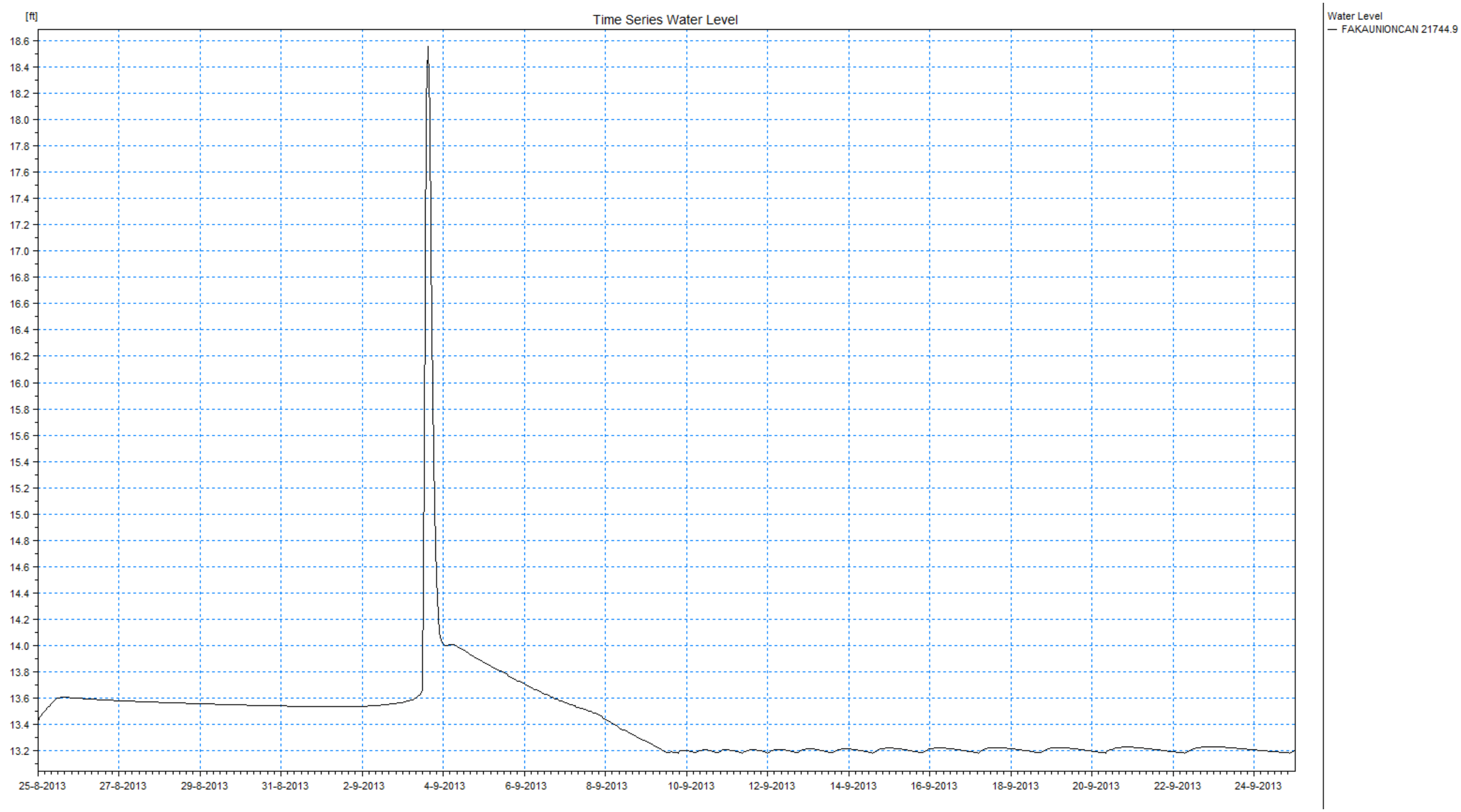

Figure 20. FU6 Headwater Stage (ft)

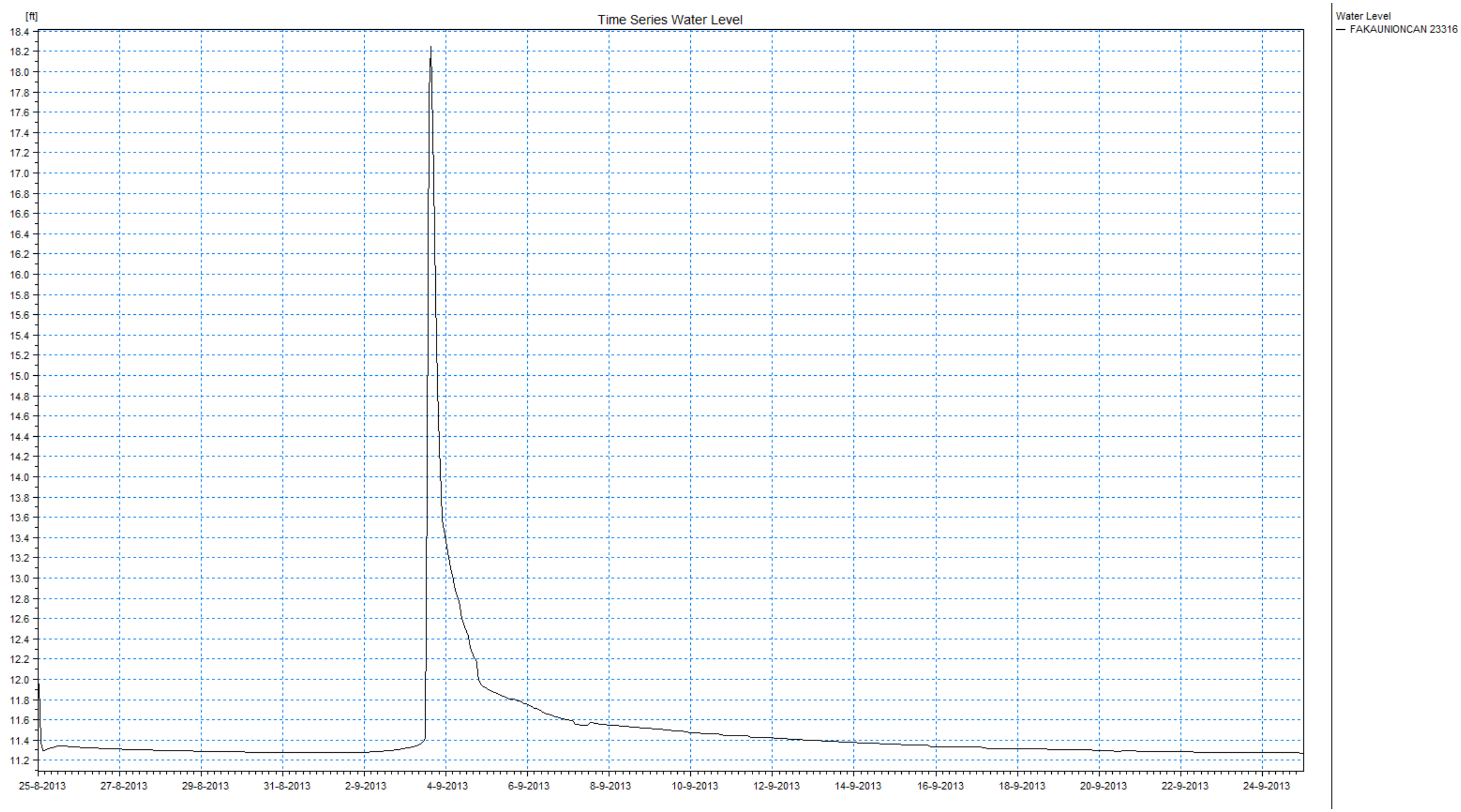

Figure 21. FU6 Tailwater Stage (ft)

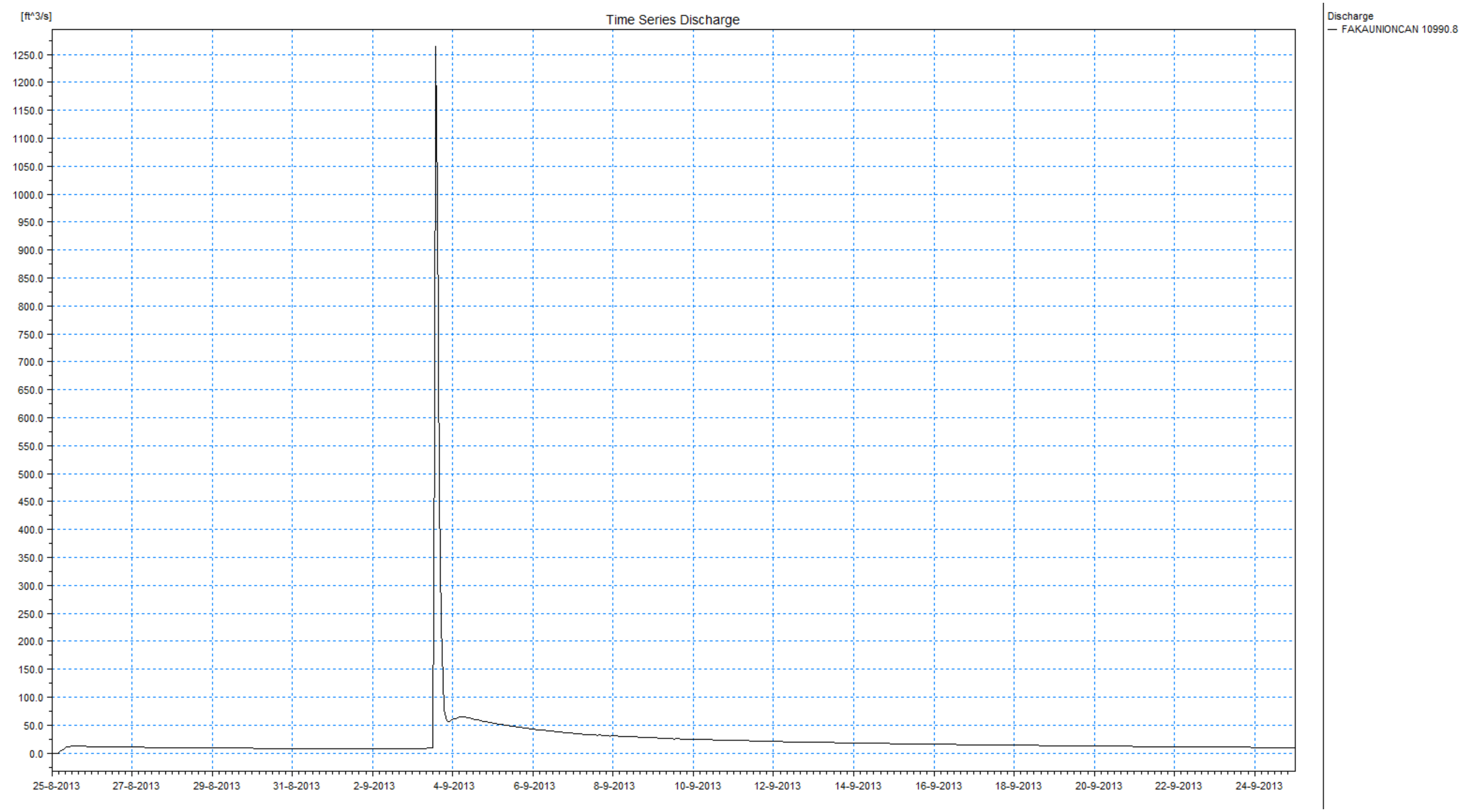

Figure 22. FU7 Discharge (cfs)

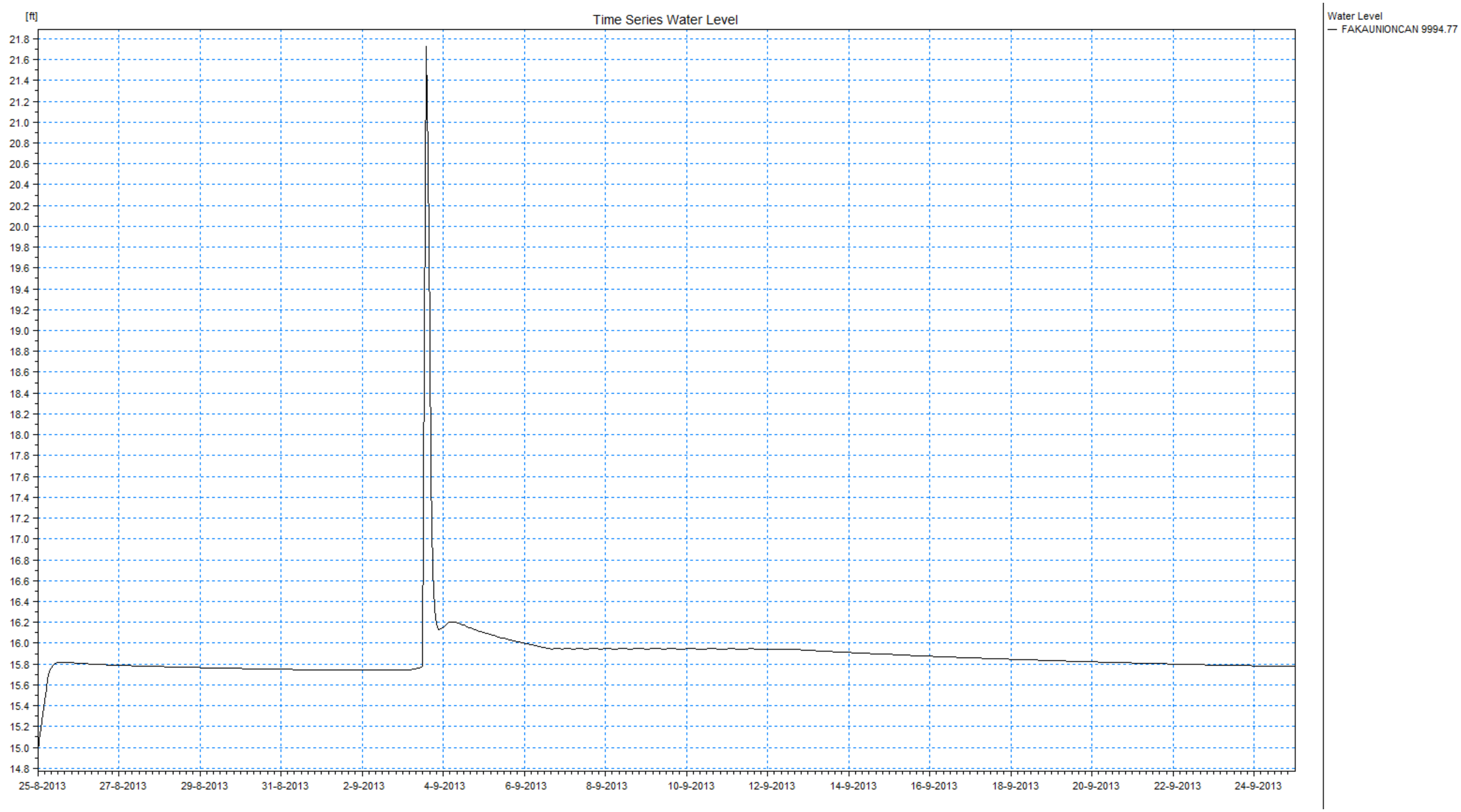

Figure 23. FU7 Headwater Stage (ft)

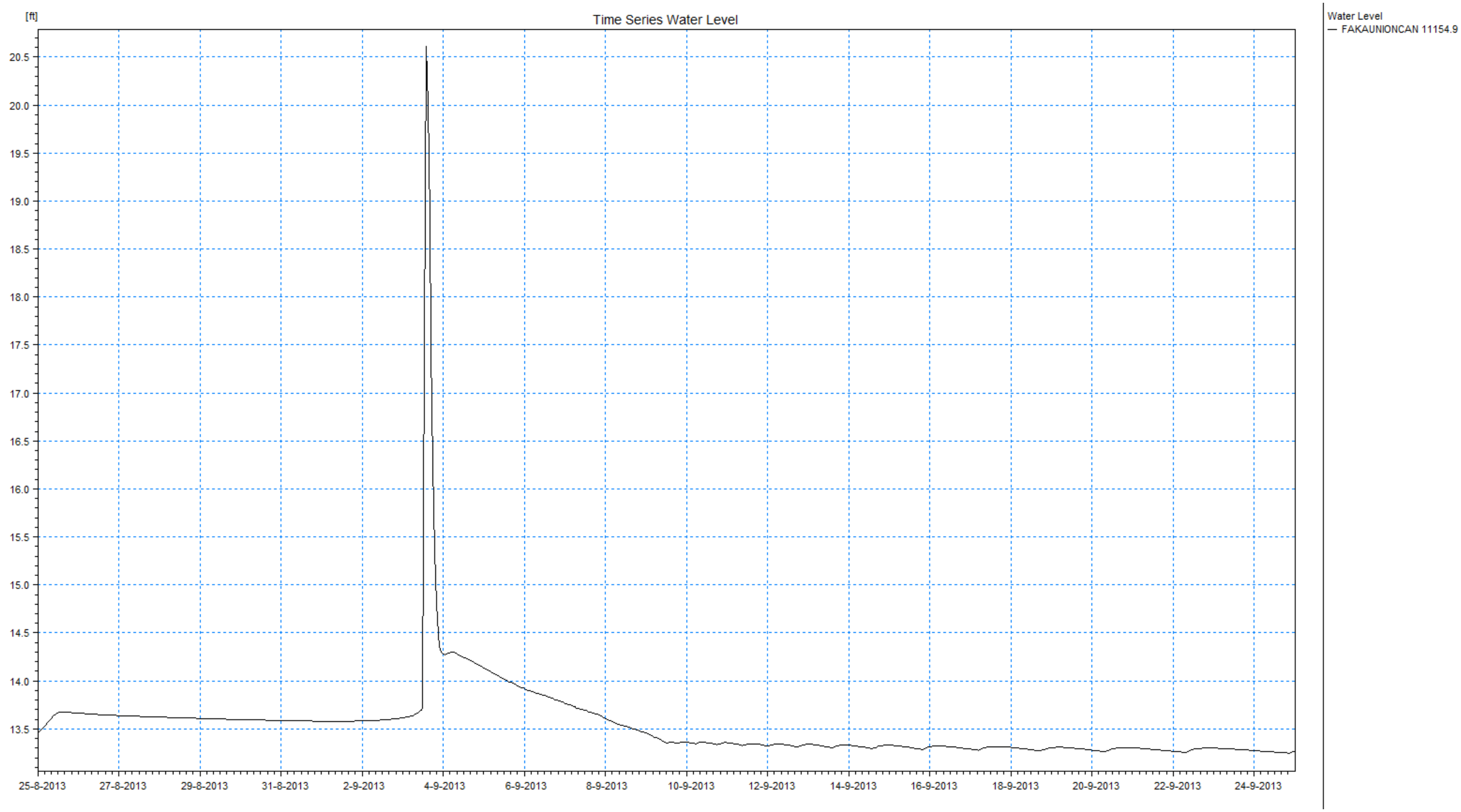

Figure 24. FU7 Tailwater Stage (ft)

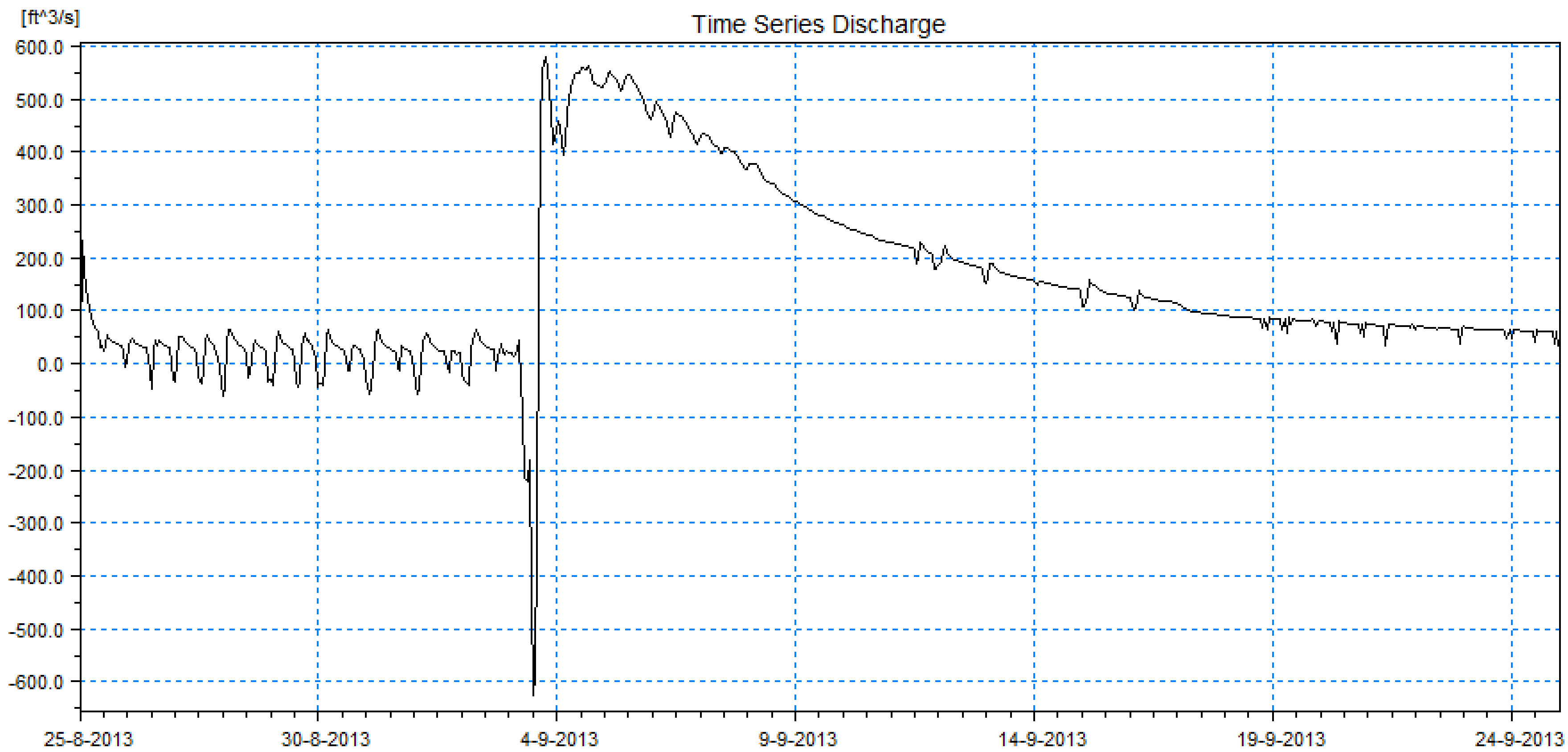

Figure 25. HC1 Discharge (cfs)

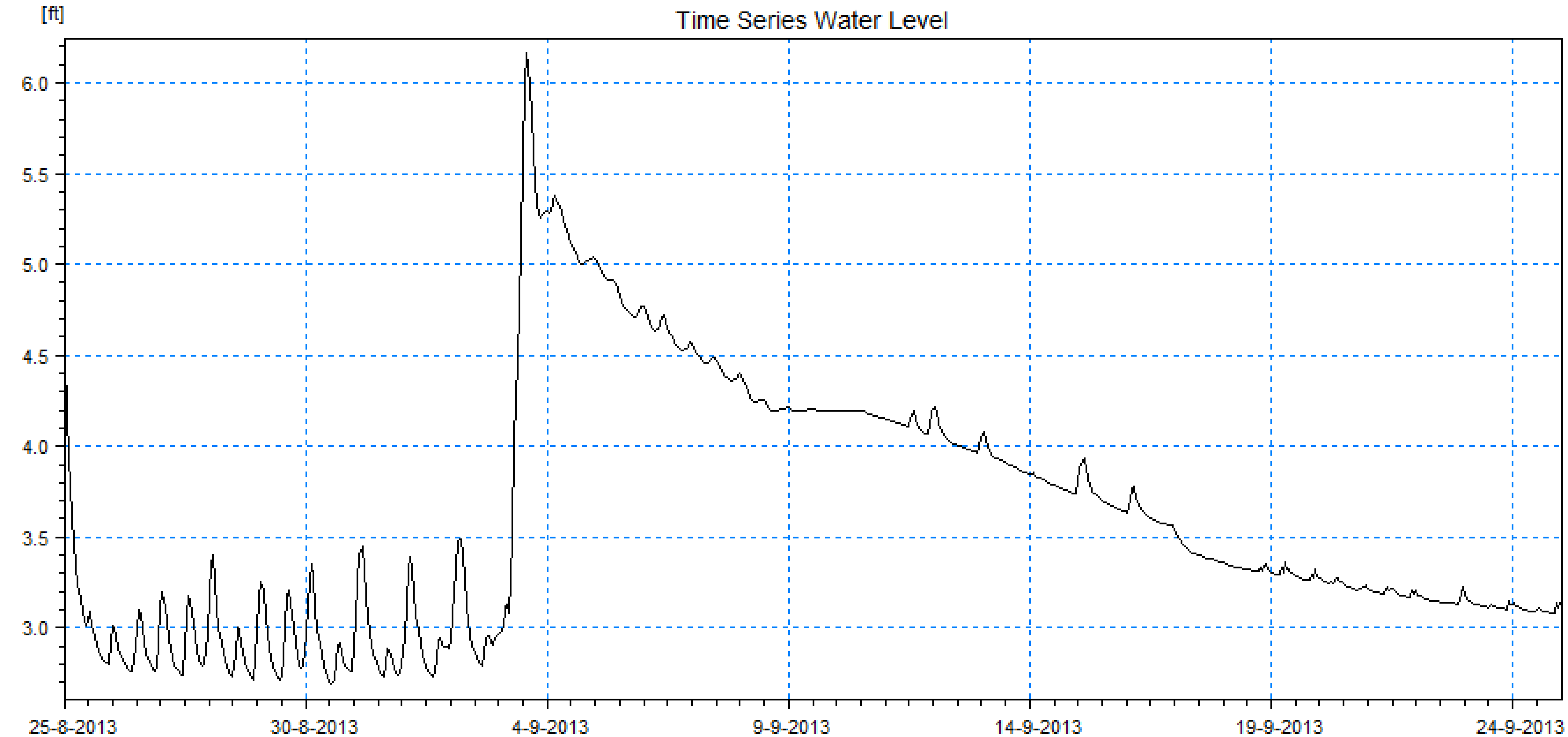

Figure 26. HC1 Headwater Stage (ft)

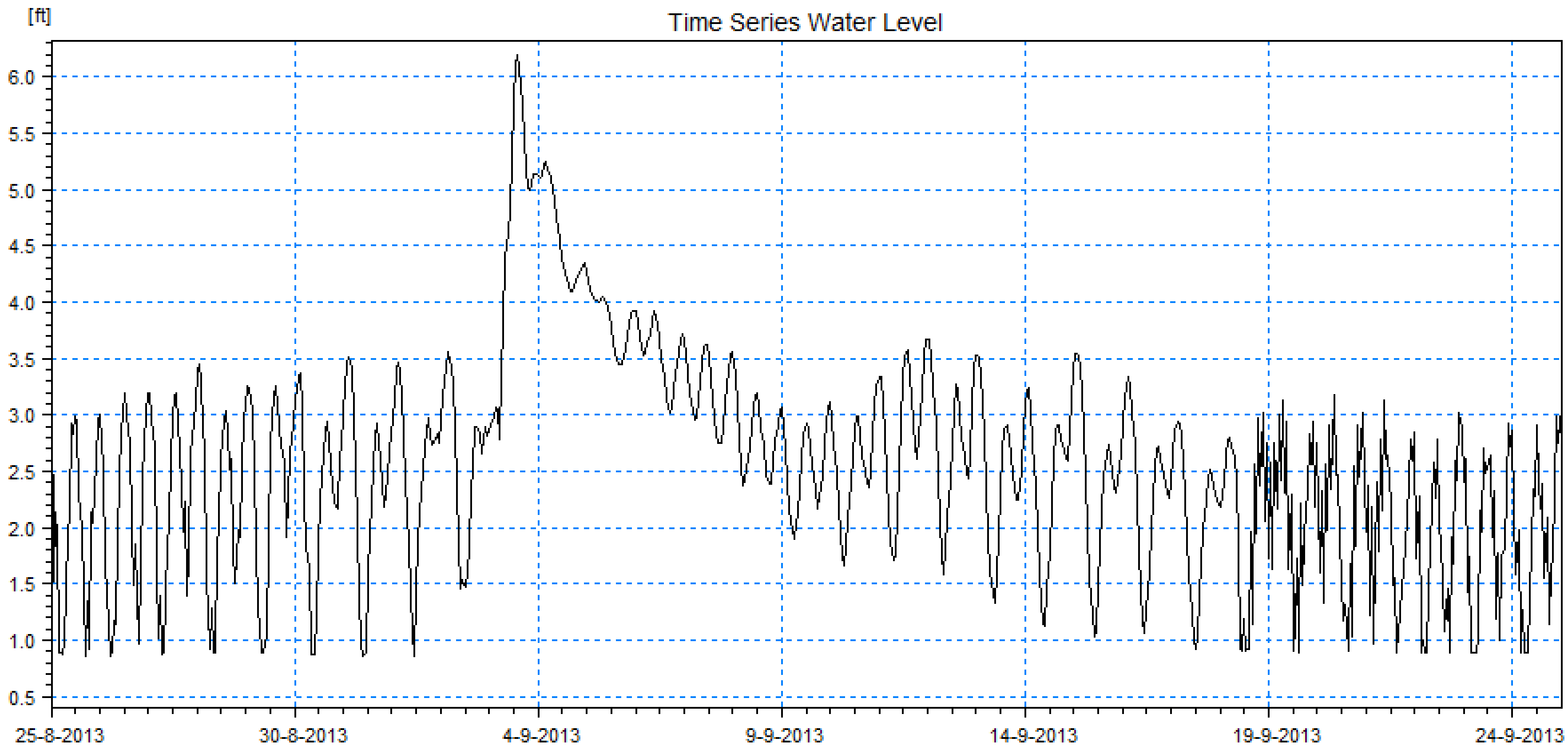

Figure 27. HC1 Tailwater Stage (ft)

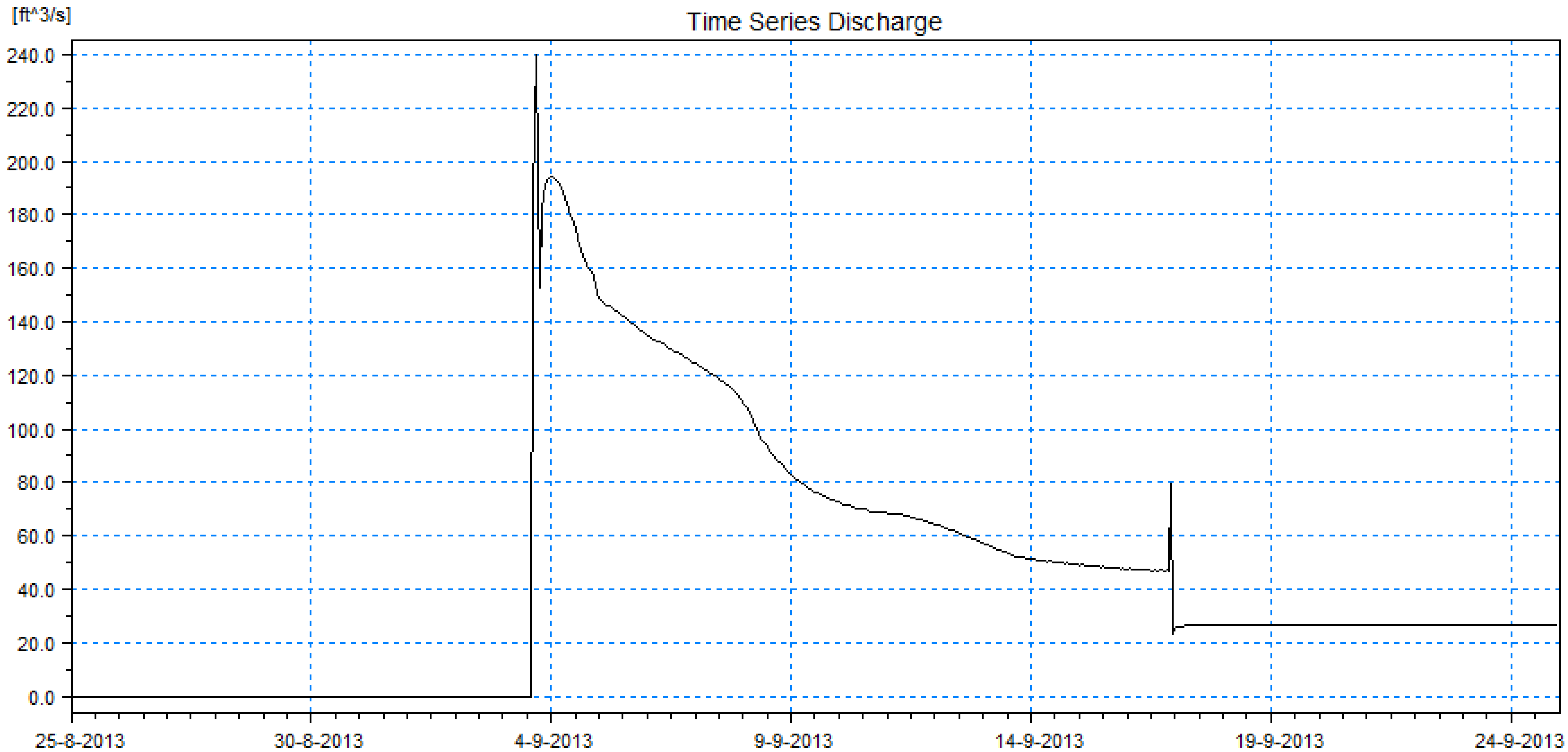

Figure 28. HC2 Discharge (cfs)

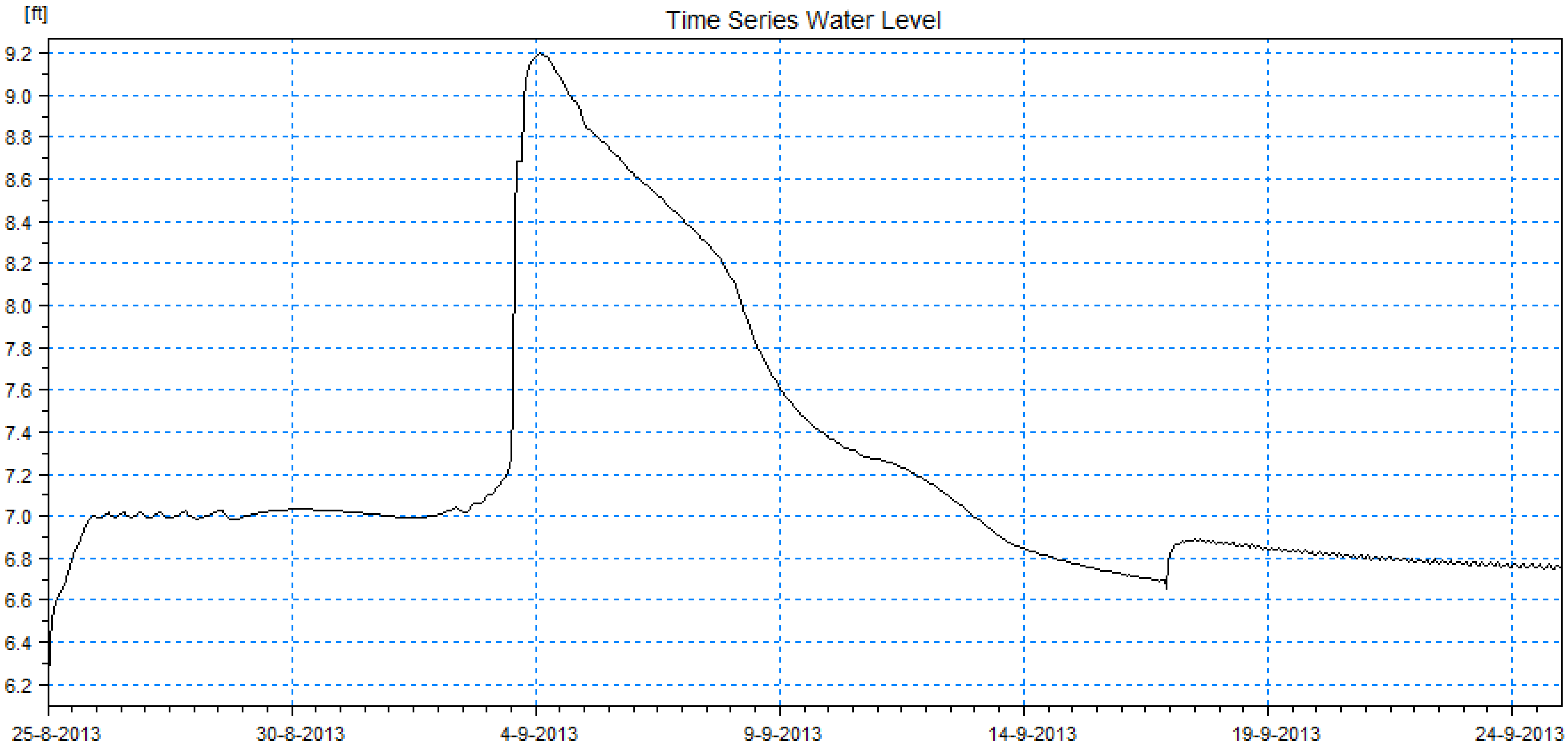

Figure 29. HC2 Headwater Stage (ft)

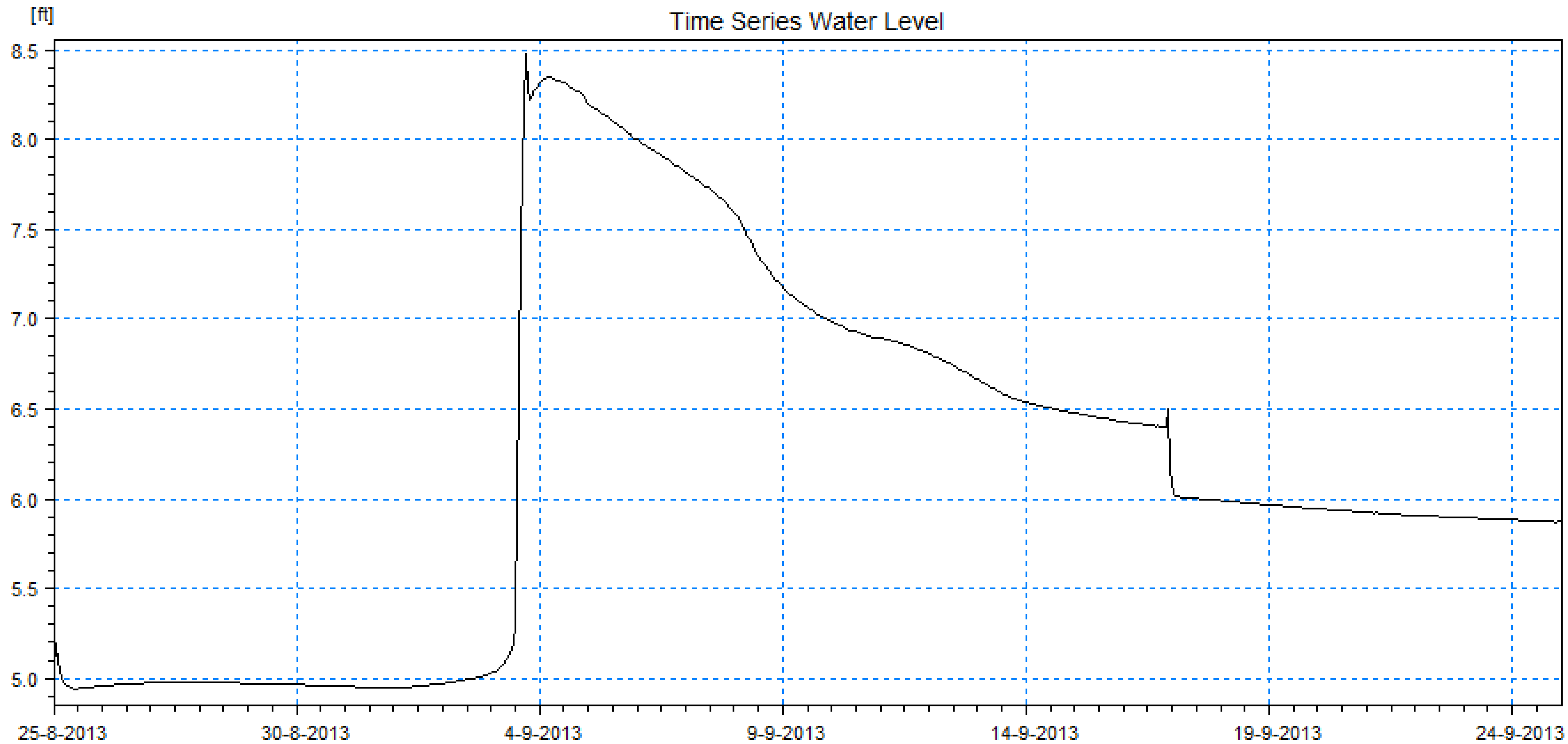

Figure 30. HC2 Tailwater Stage (ft)

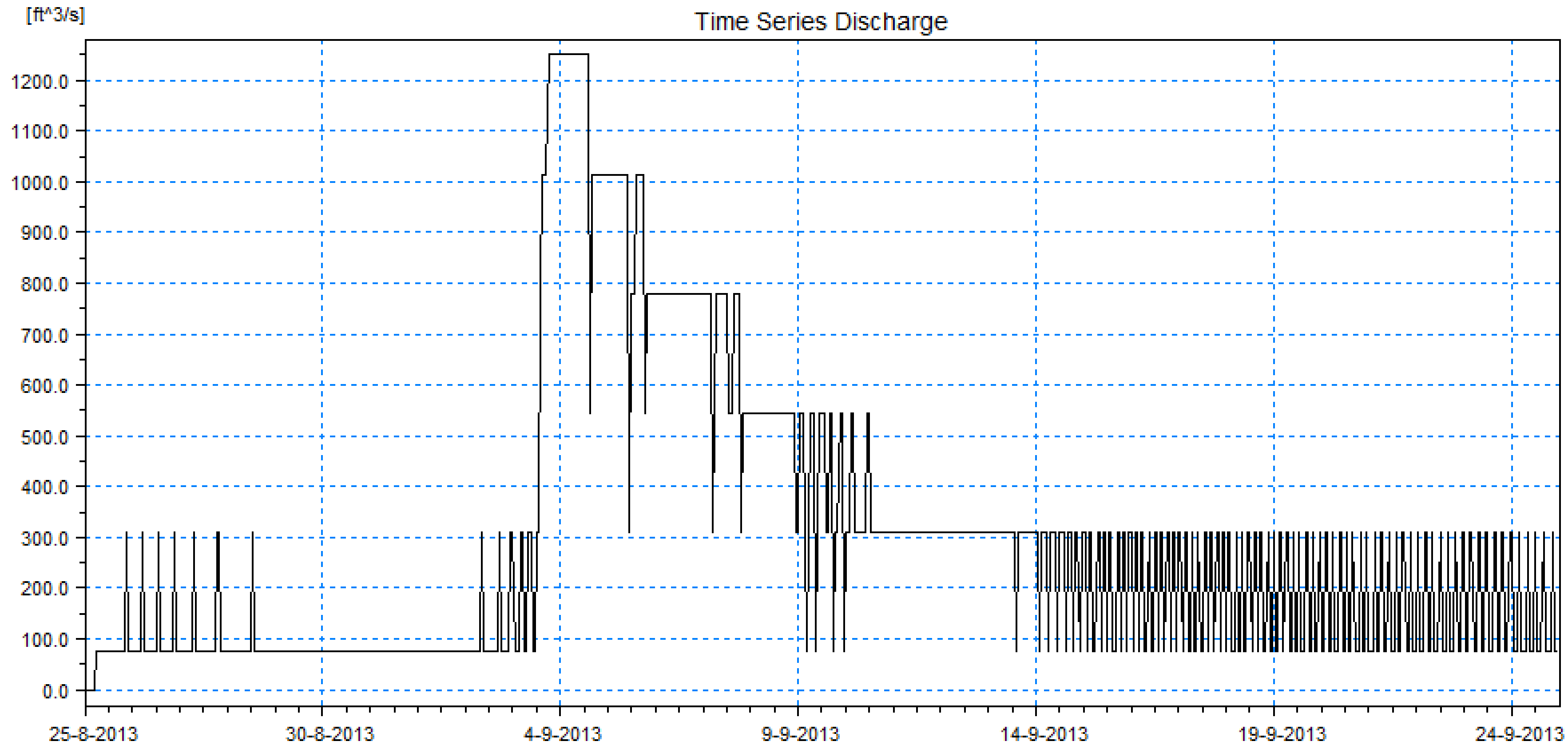

Figure 31. Miller Pump Discharge (cfs)

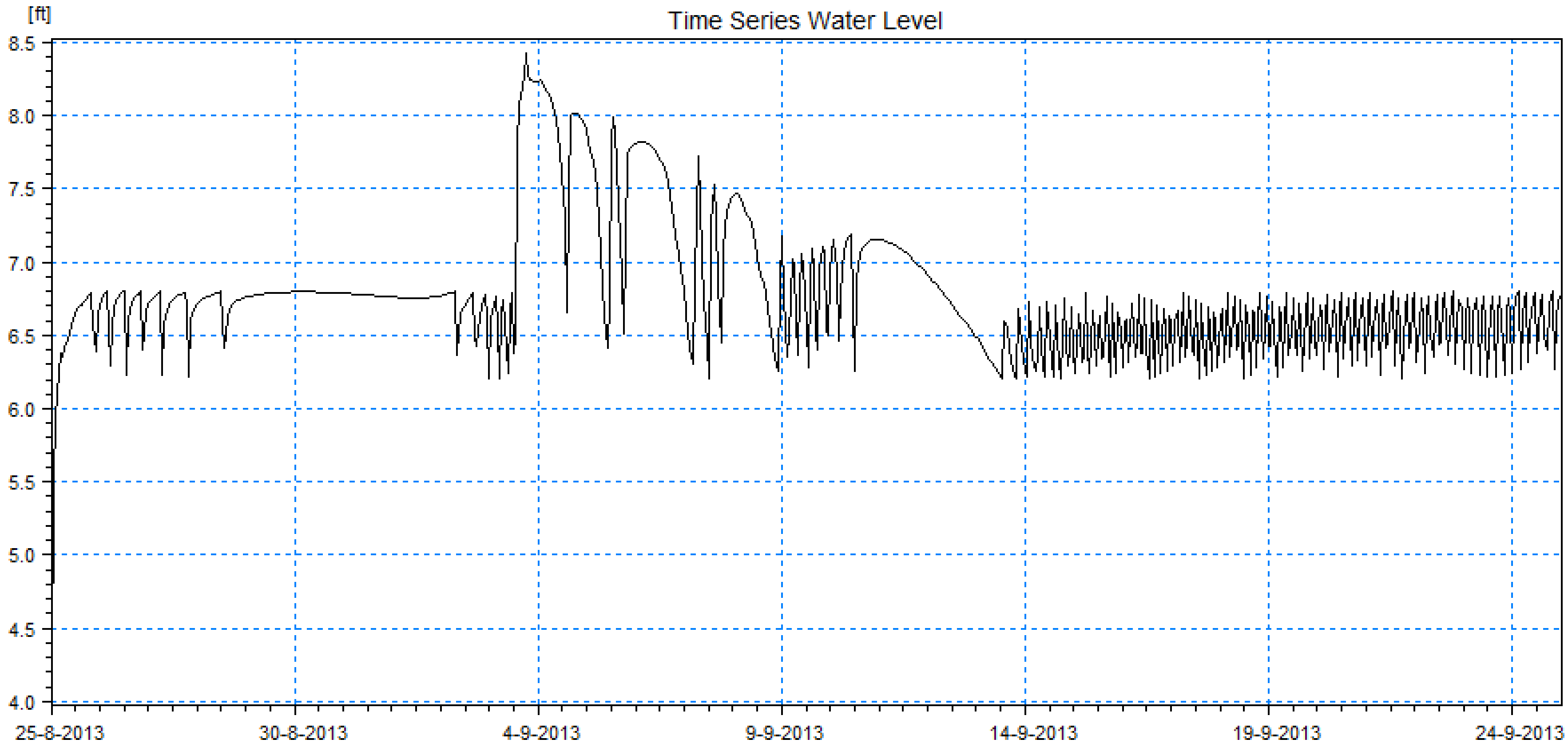

Figure 32. Miller Pump Headwater Stage (ft)

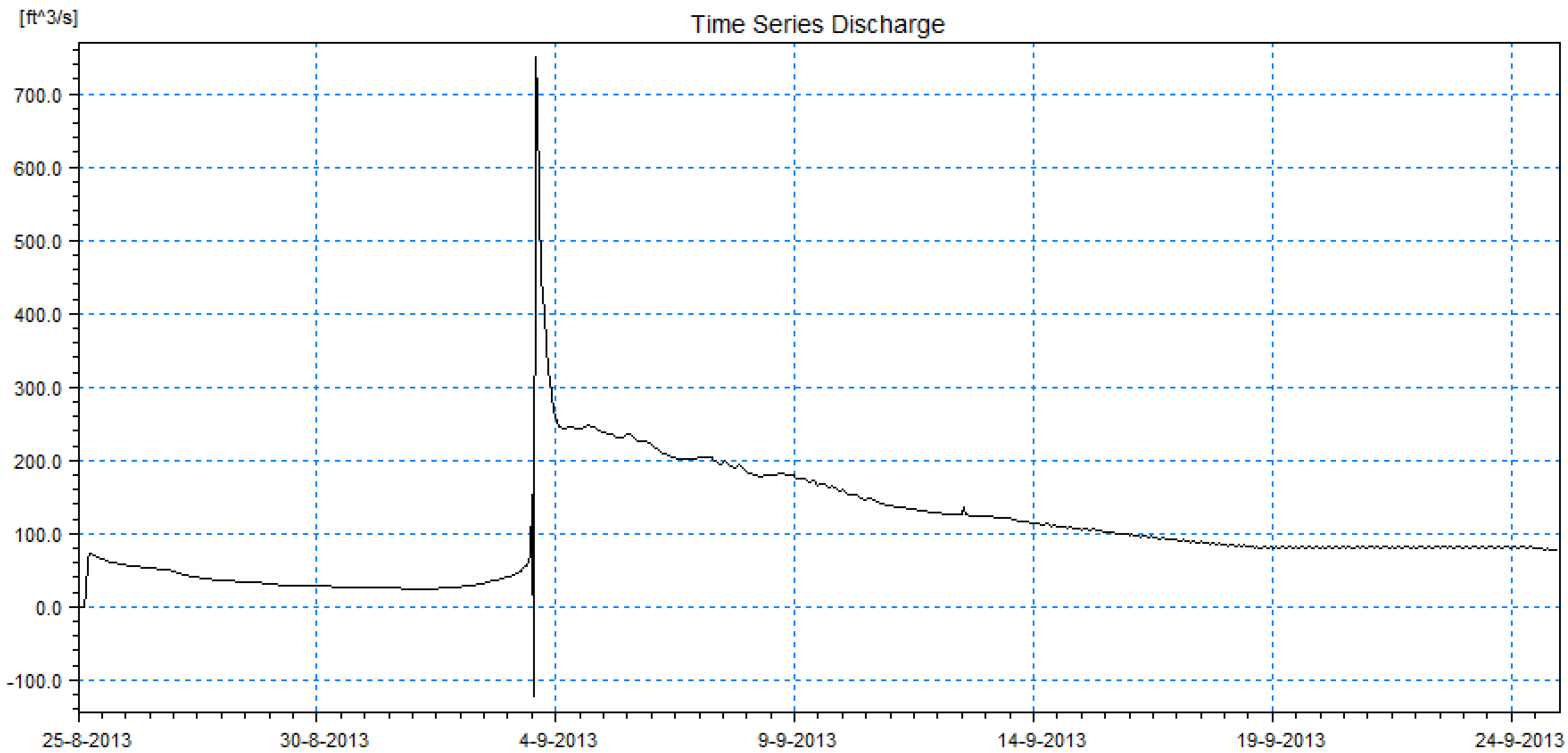

Figure 33. Miller3 Discharge (cfs)

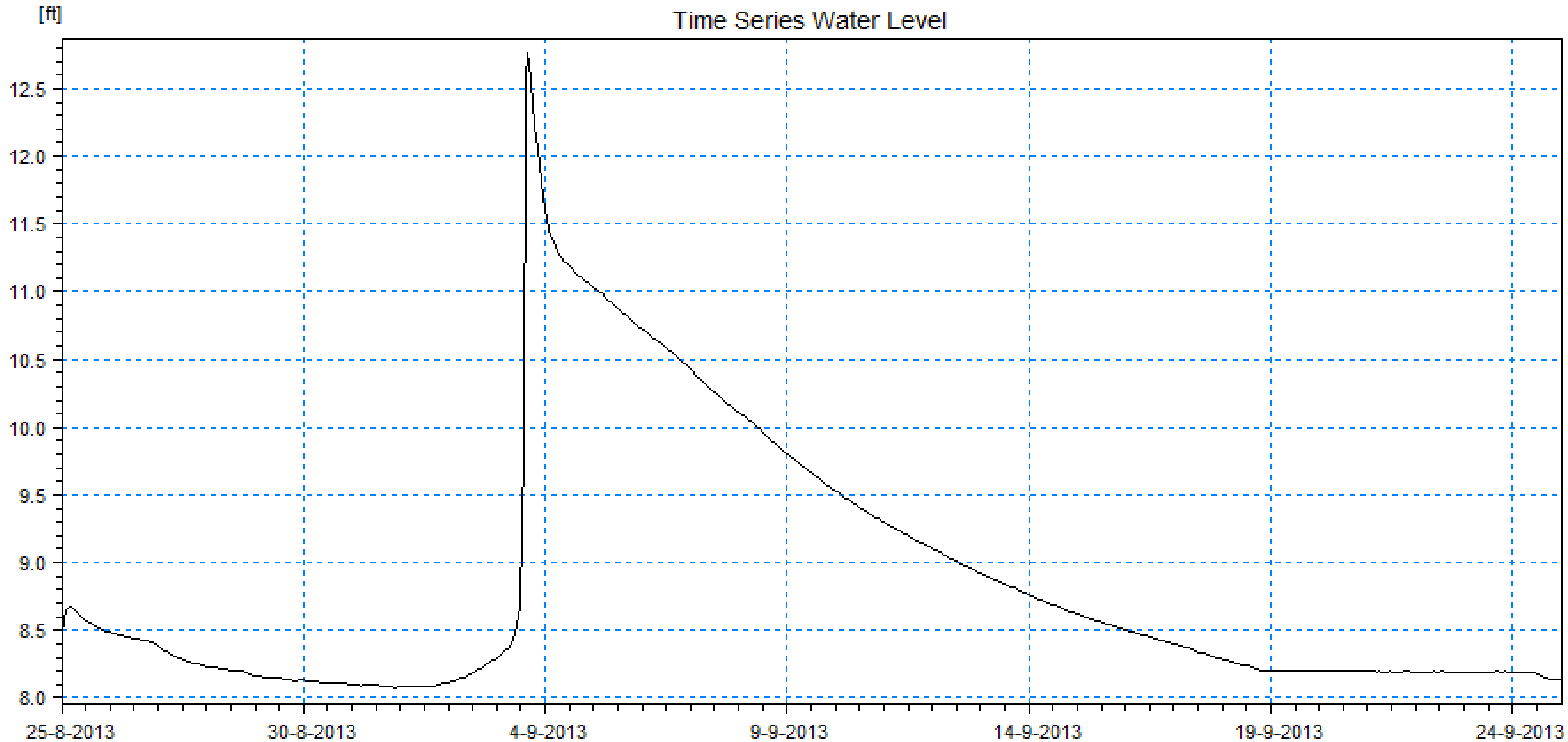

Figure 34. Miller3 Headwater Stage (ft)

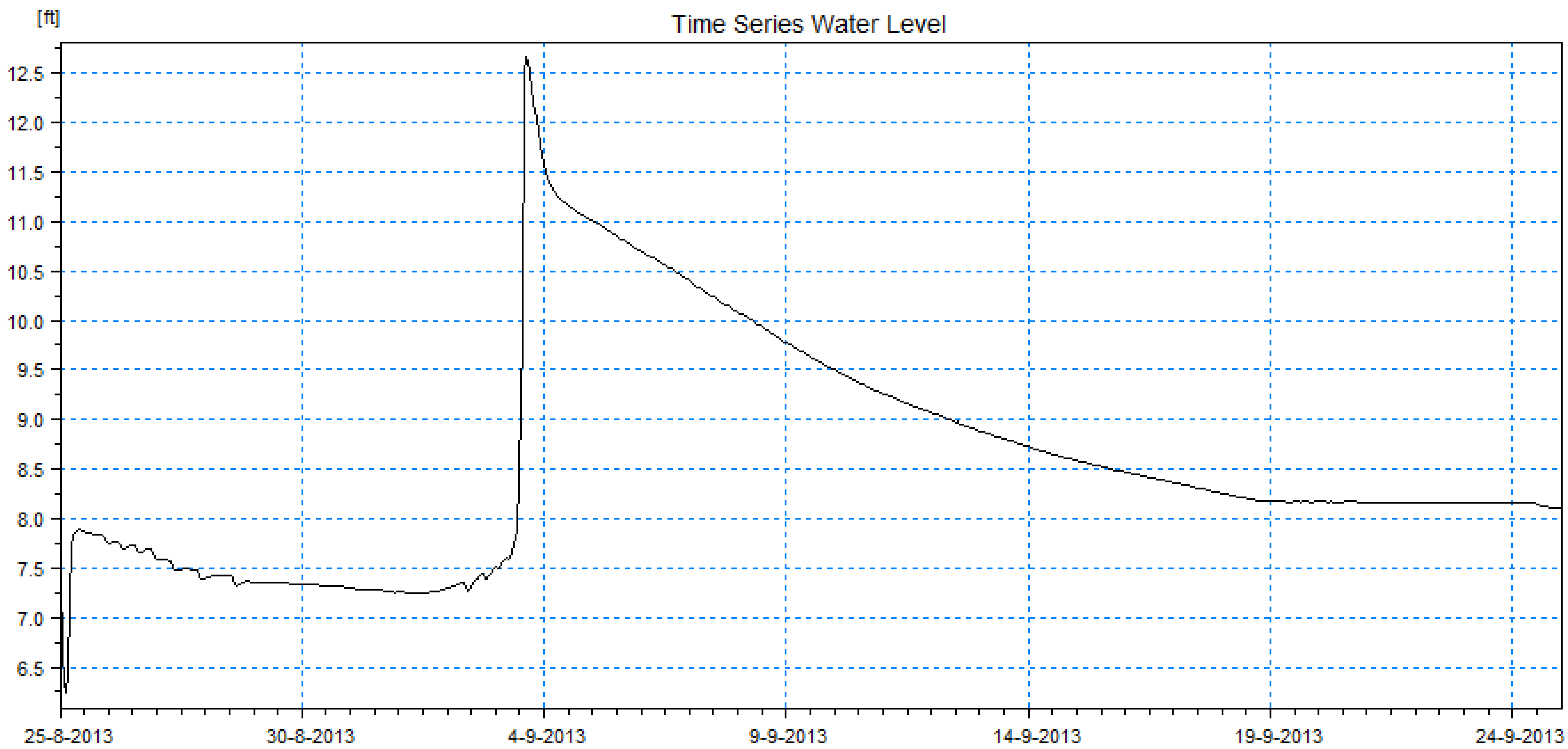

Figure 35. Miller3 Tailwater Stage (ft)

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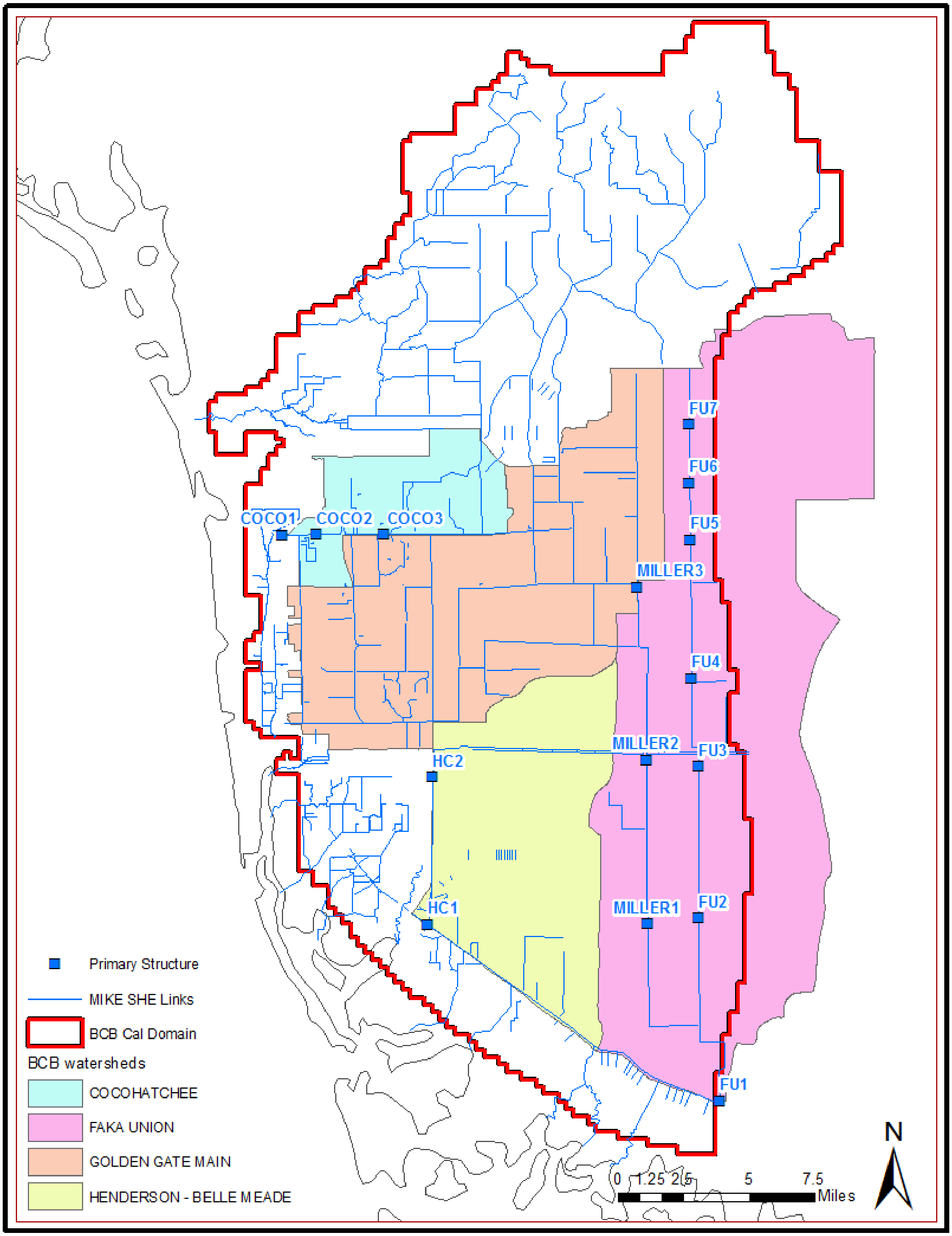

Figure 1 Mike SHE Stations within COCO, FAKA and HEND Watersheds

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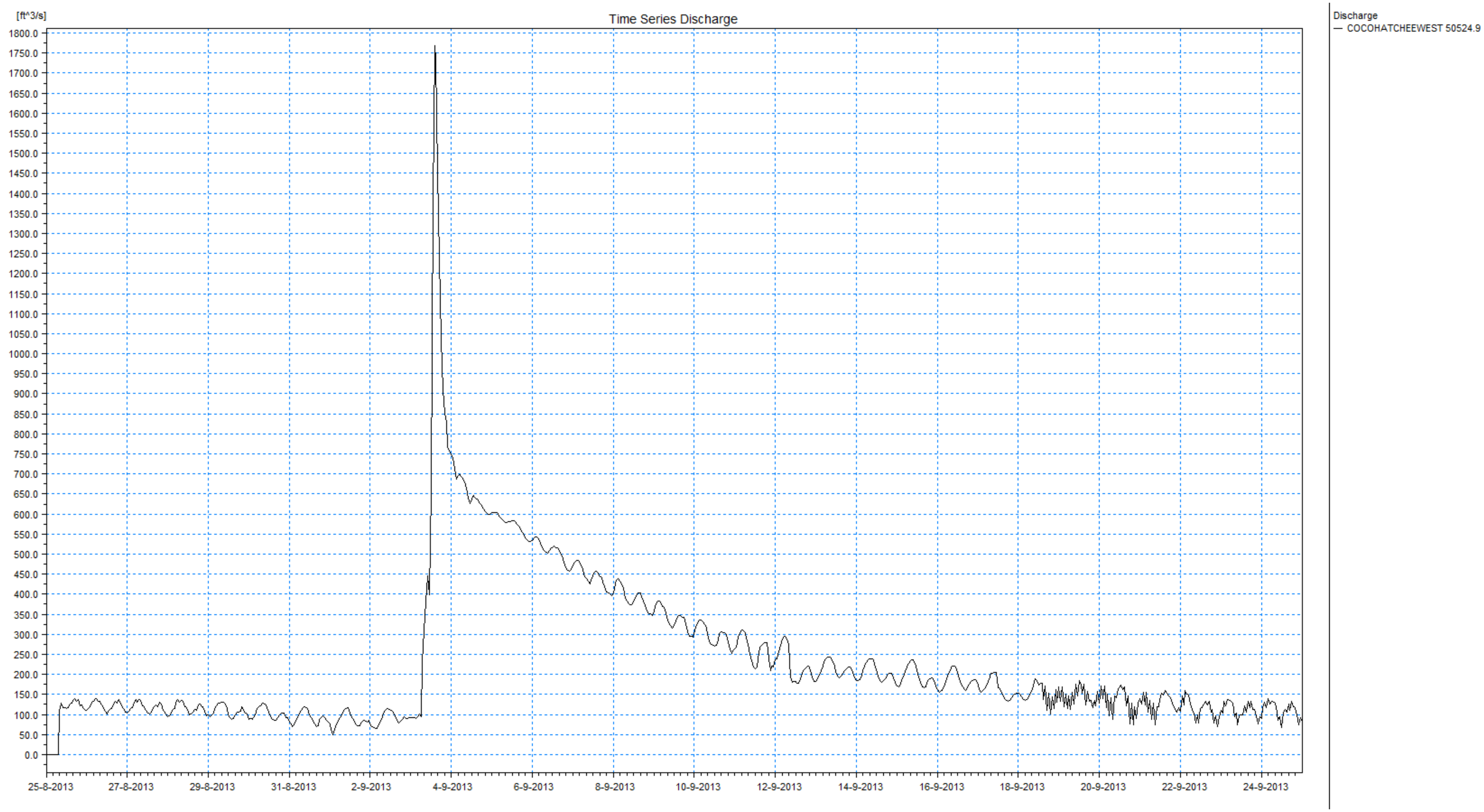


Figure 2. COCO1 Discharge (cfs)

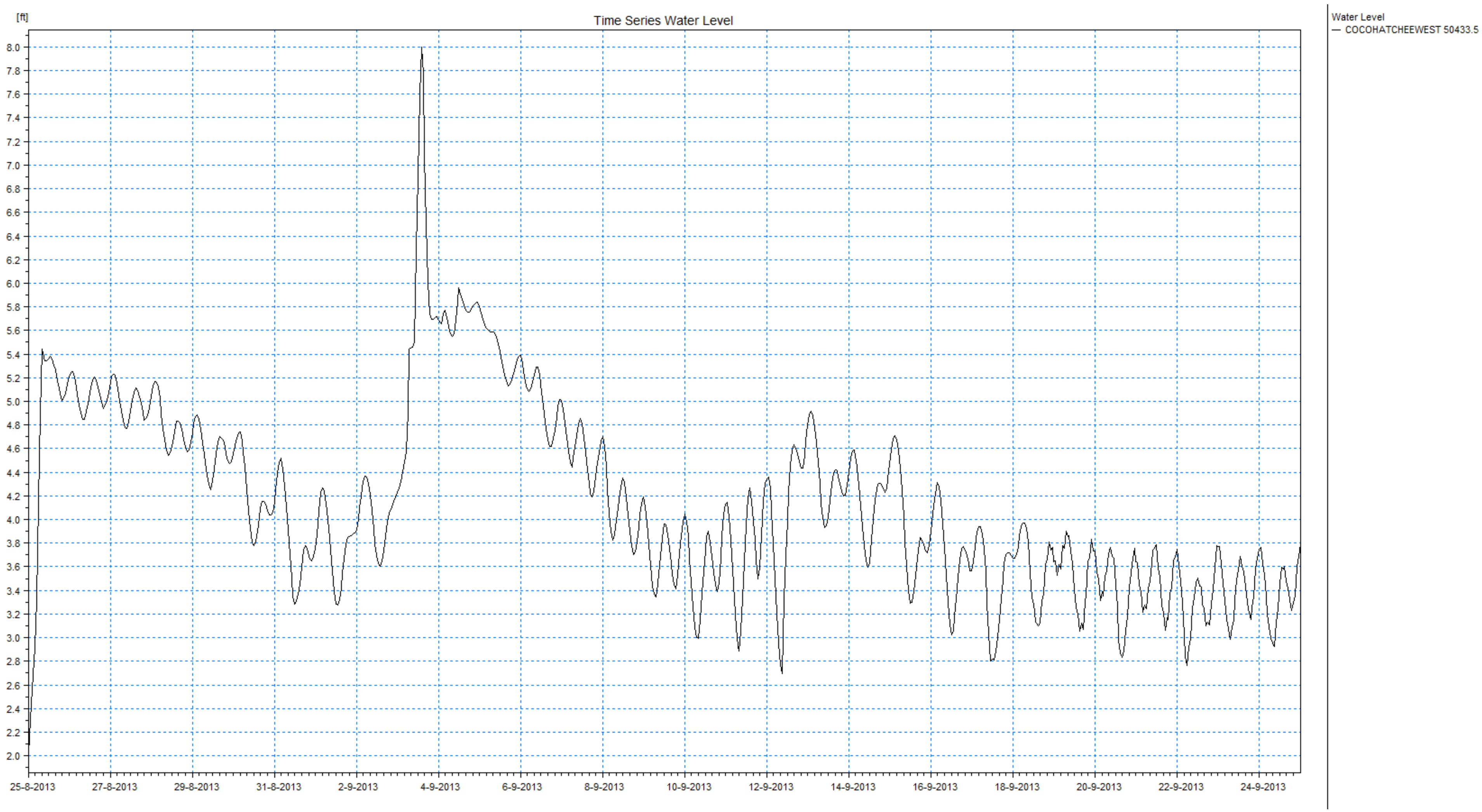


Figure 3. COCO1 Headwater Stage (ft)

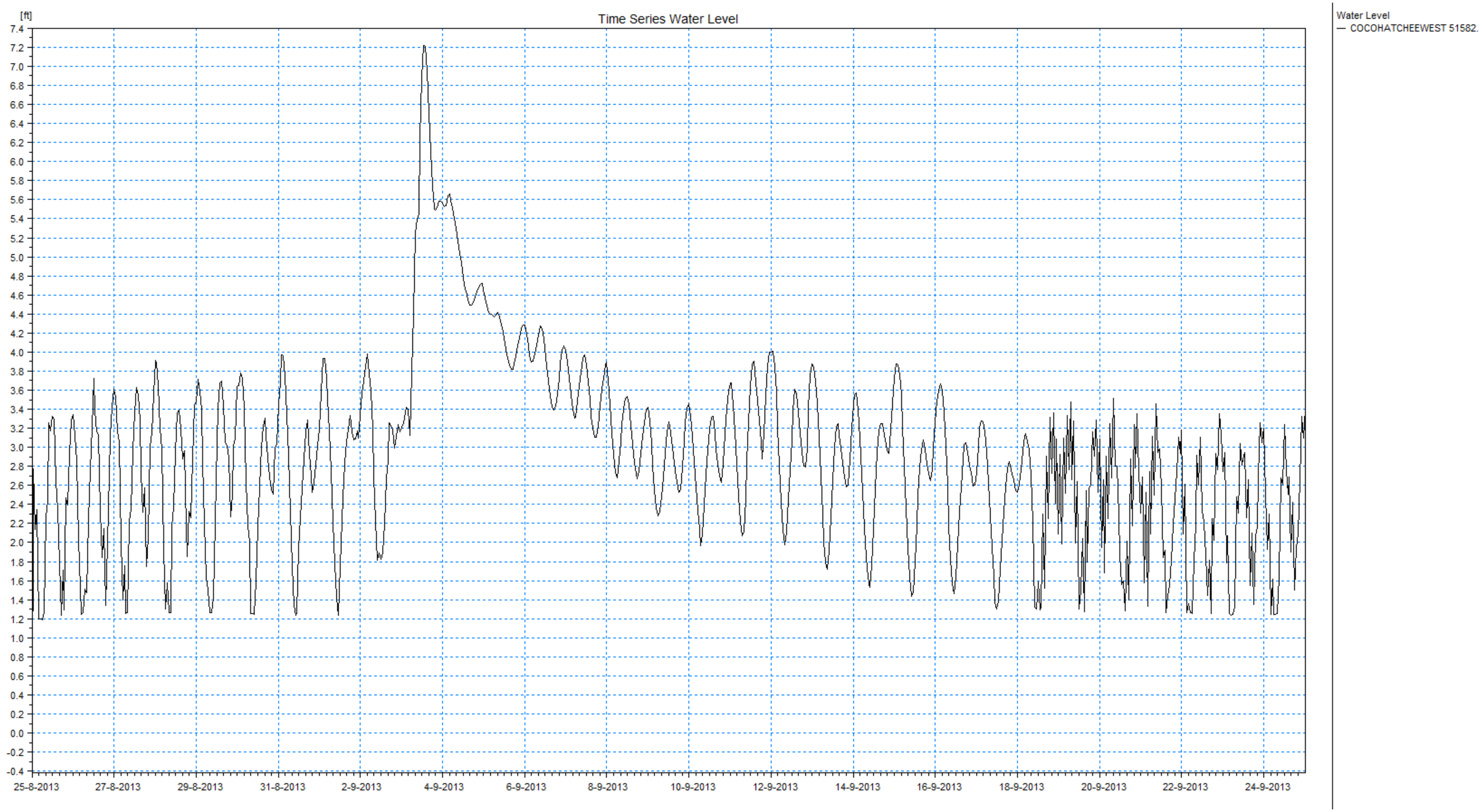


Figure 4. COCO1 Tailwater Stage (ft)

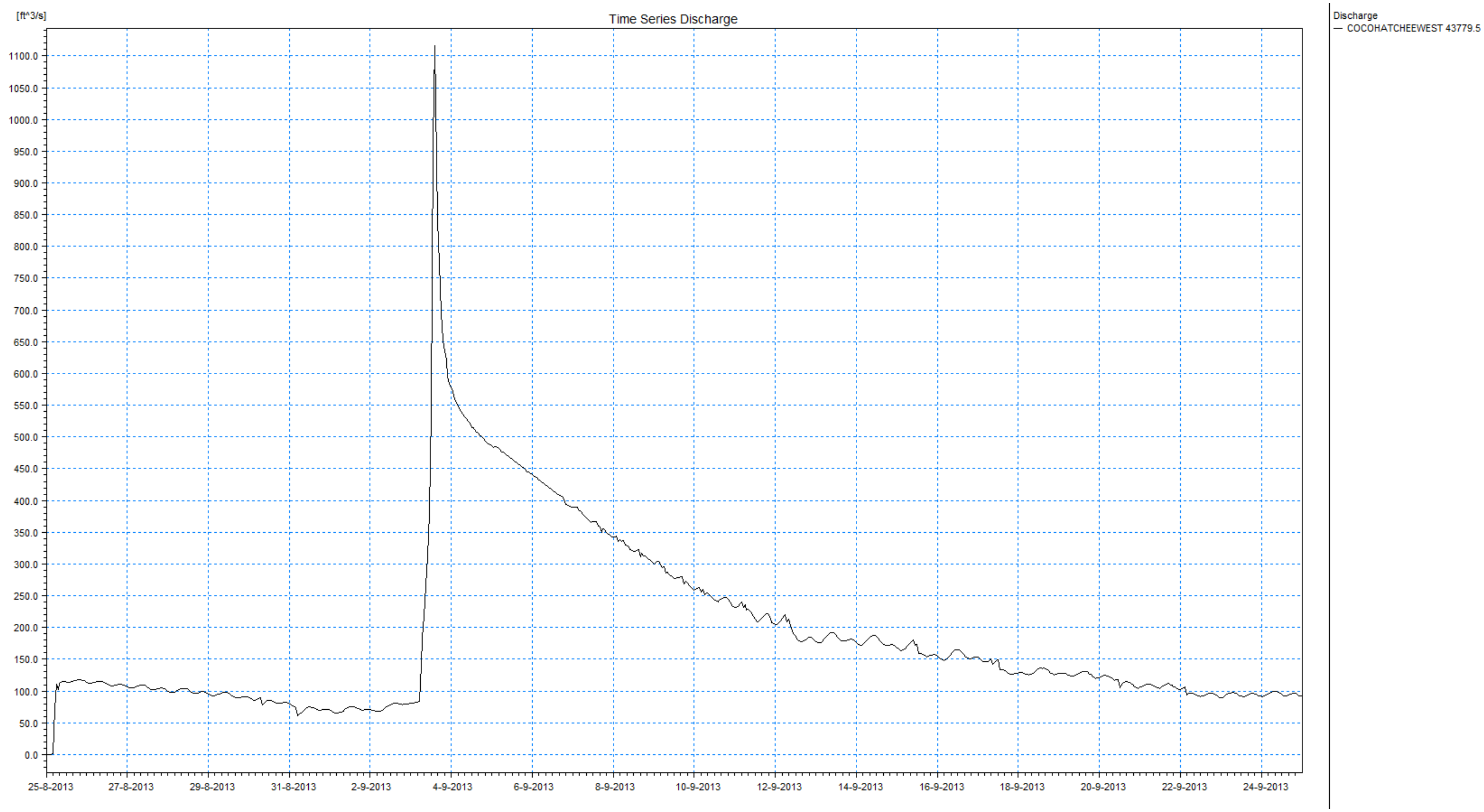


Figure 5. COCO2 Discharge (cfs)

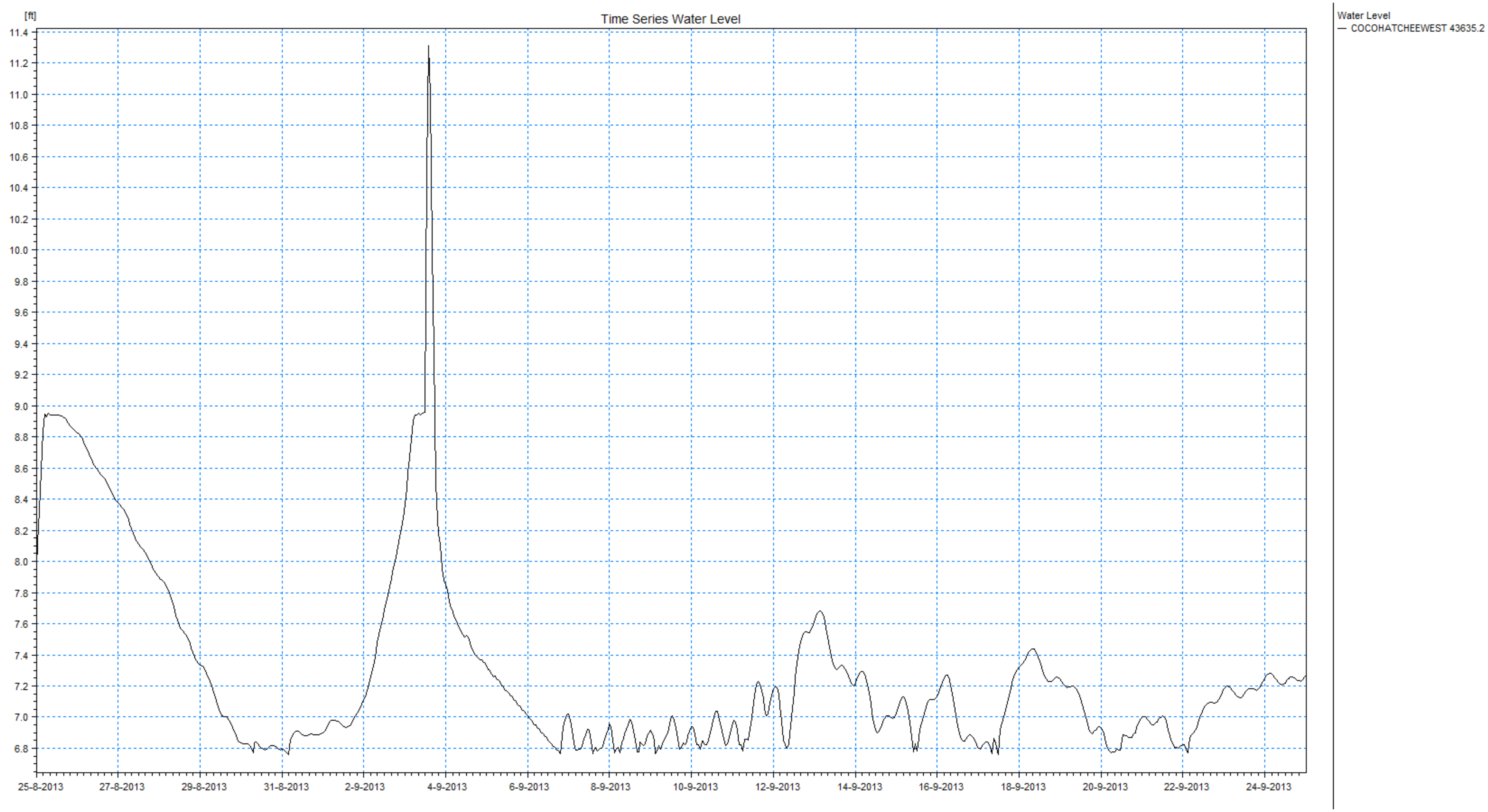


Figure 6. COCO2 Headwater Stage (ft)

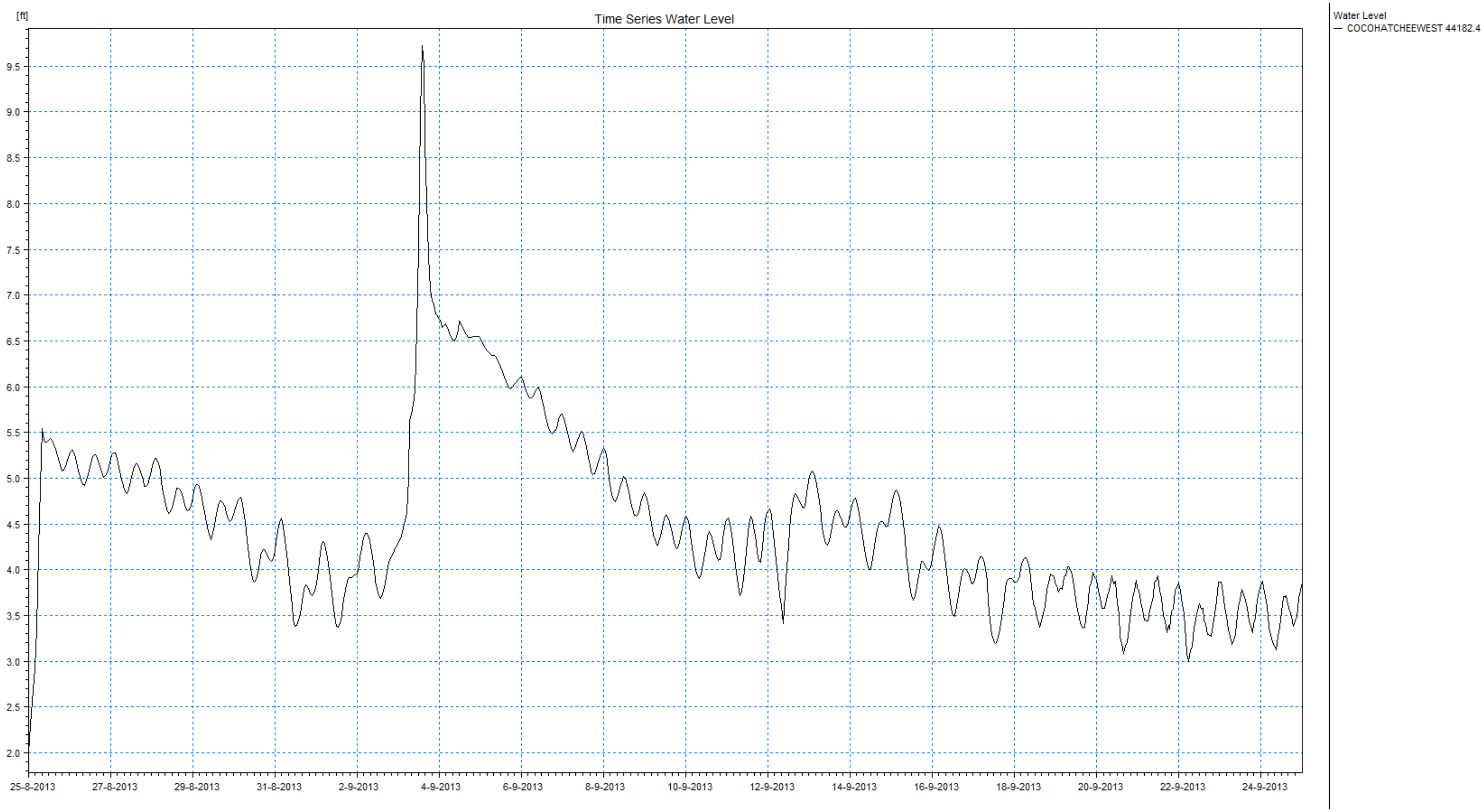


Figure 7. COCO2 Tailwater Stage (ft)

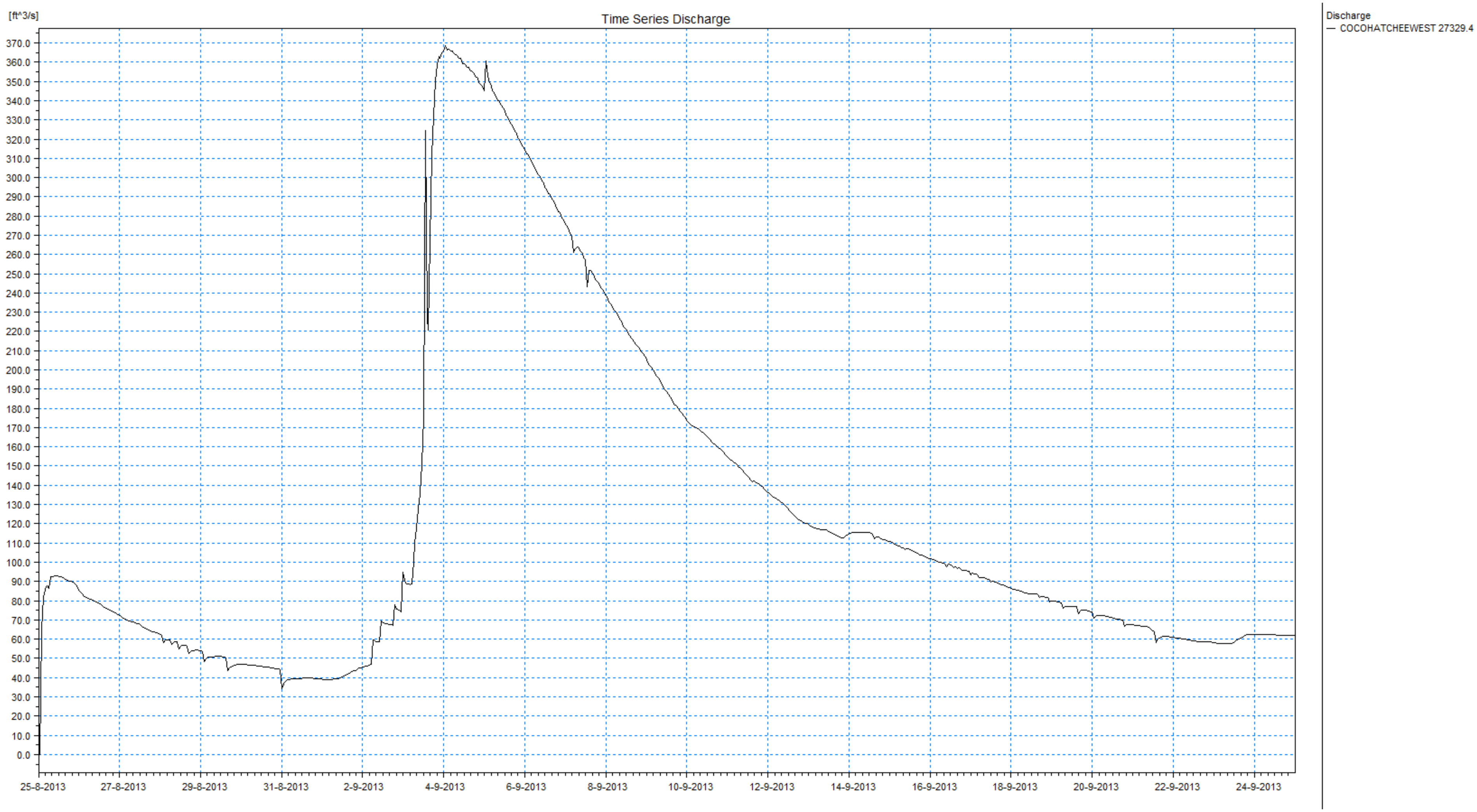


Figure 8. COCO3 Discharge (cfs)

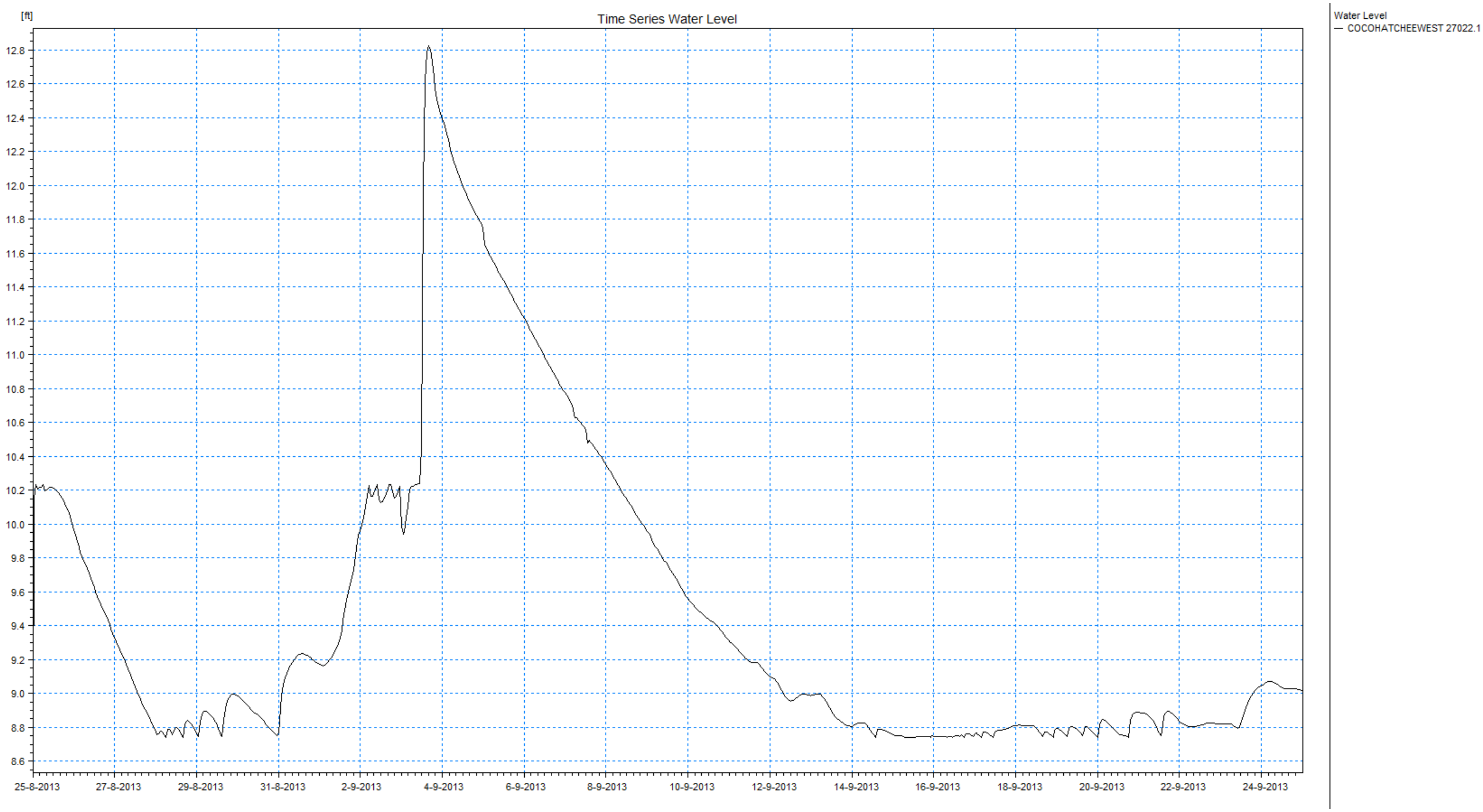


Figure 9. COCO3 Headwater Stage (ft)

Figure 10. COCO3 Tailwater Stage (ft)

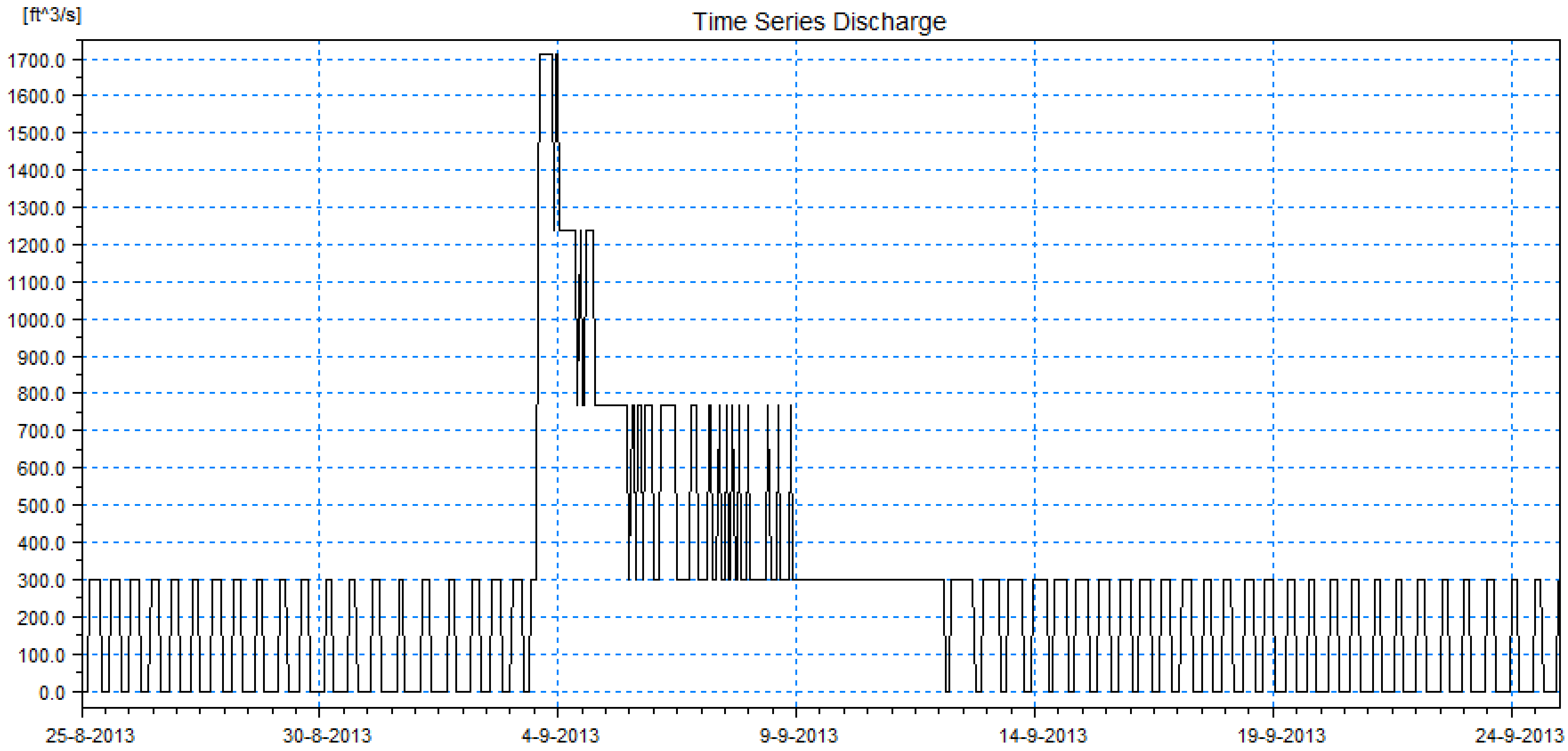


Figure 11. FU Pump Discharge (cfs)

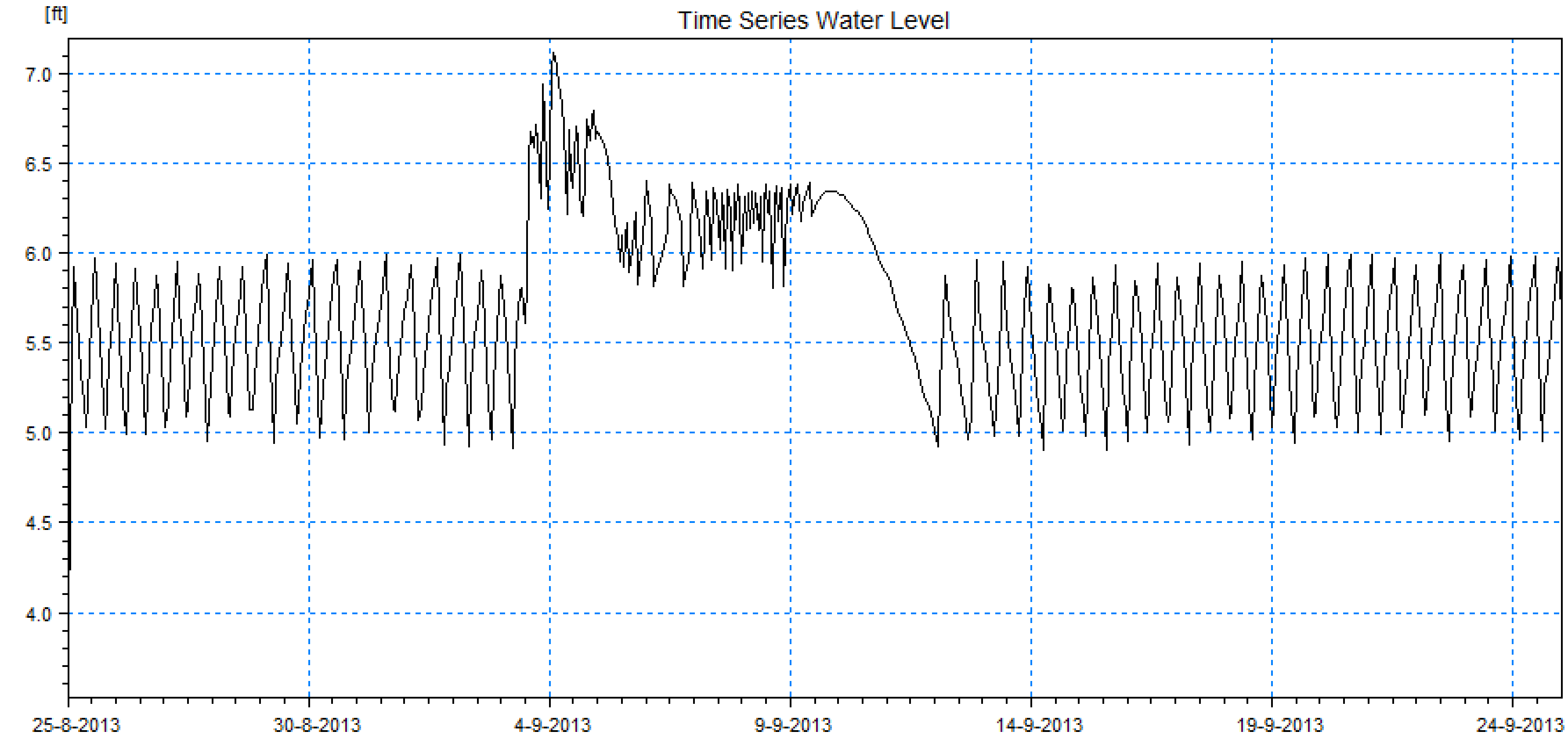


Figure 12. FU Pump Headwater Stage (ft)

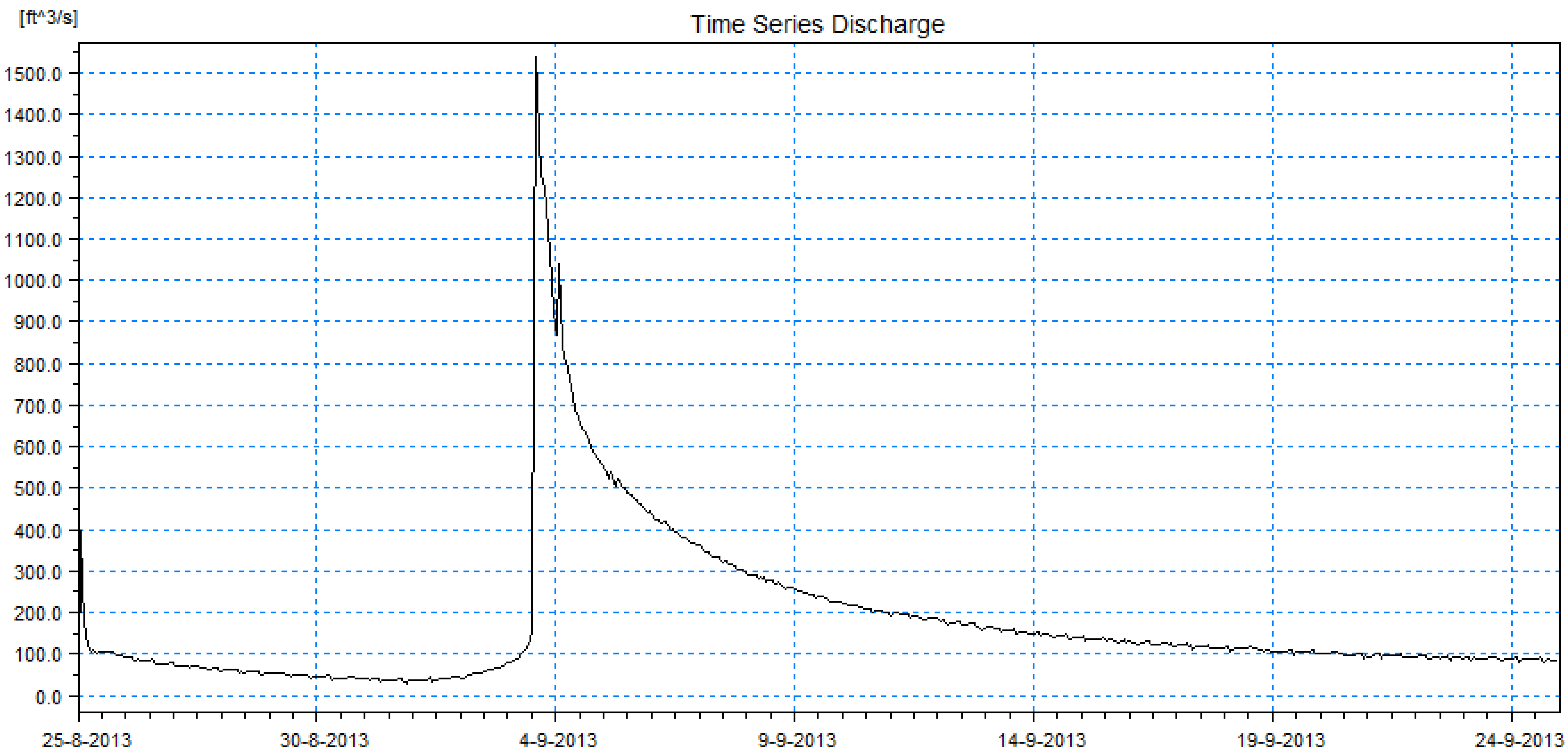


Figure 13. FU4 Discharge (cfs)

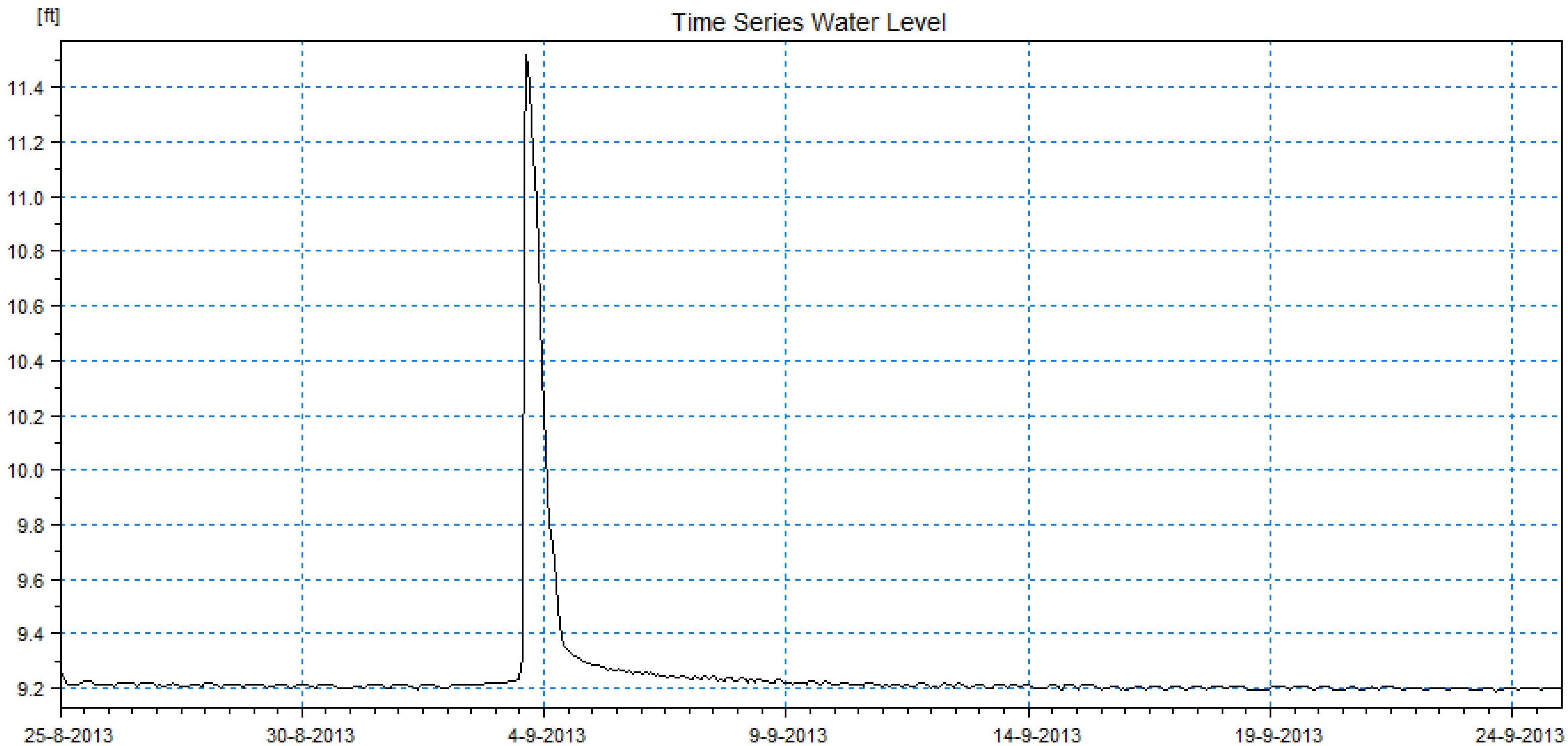


Figure 14. FU4 Headwater Stage (ft)

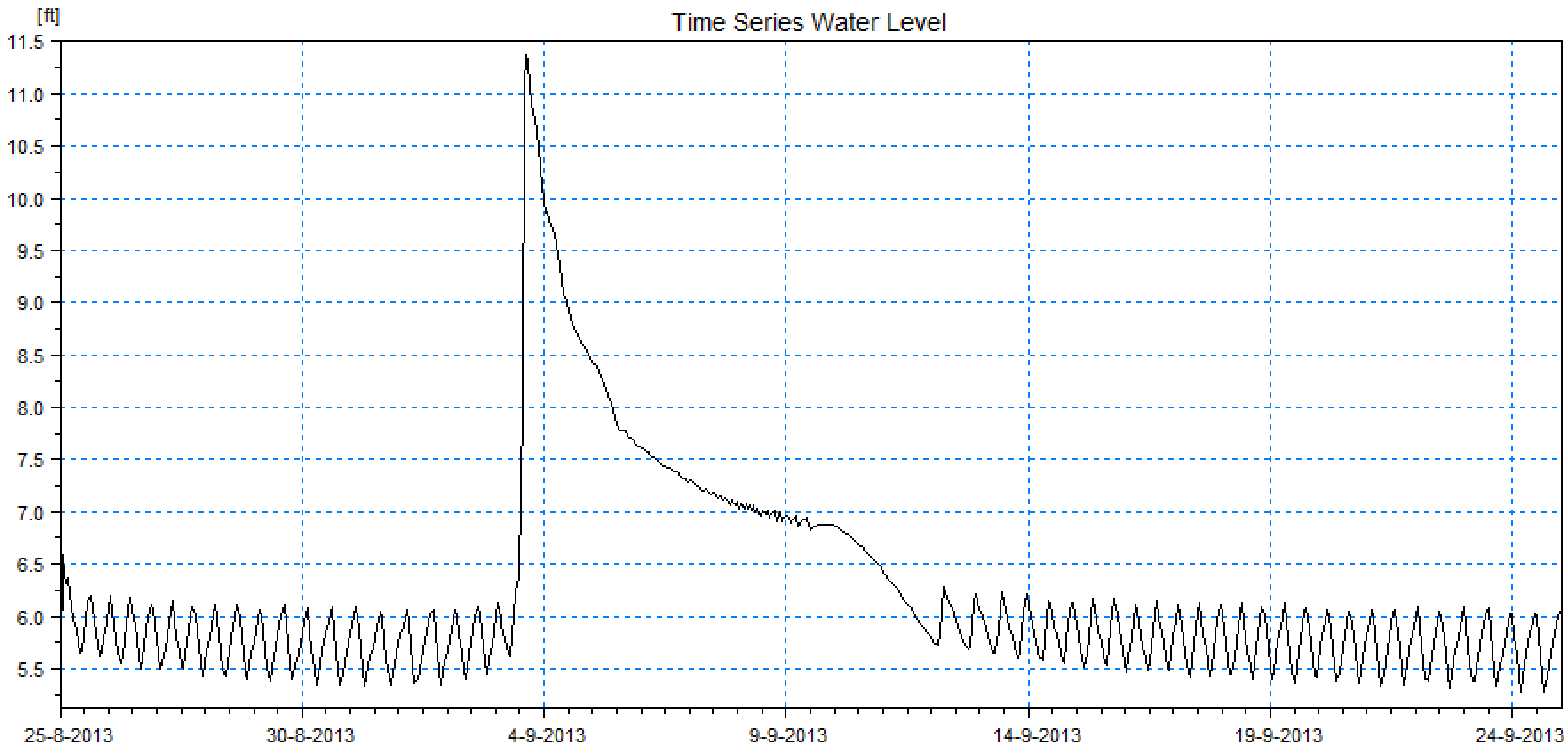


Figure 15. FU4 Tailwater Stage (ft)

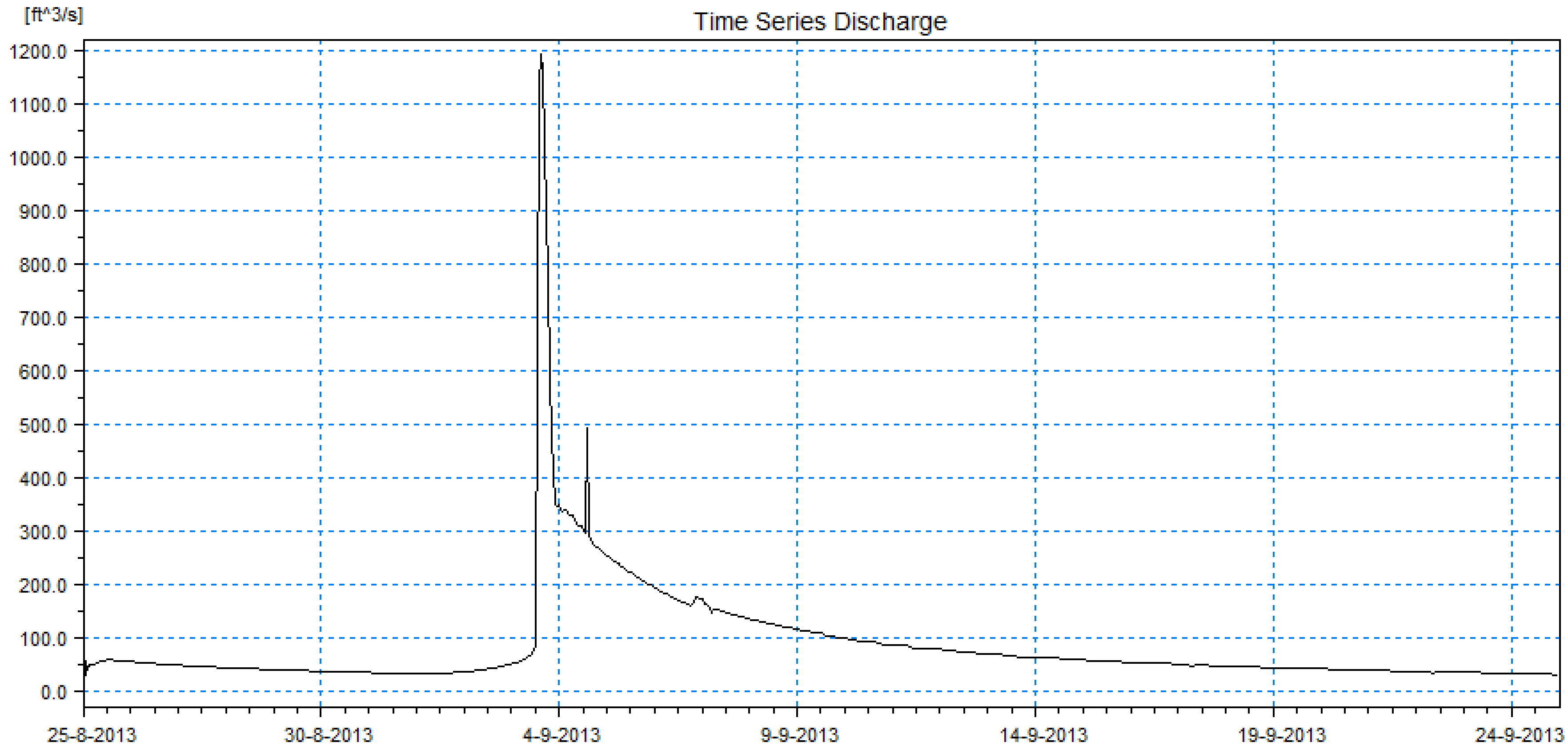
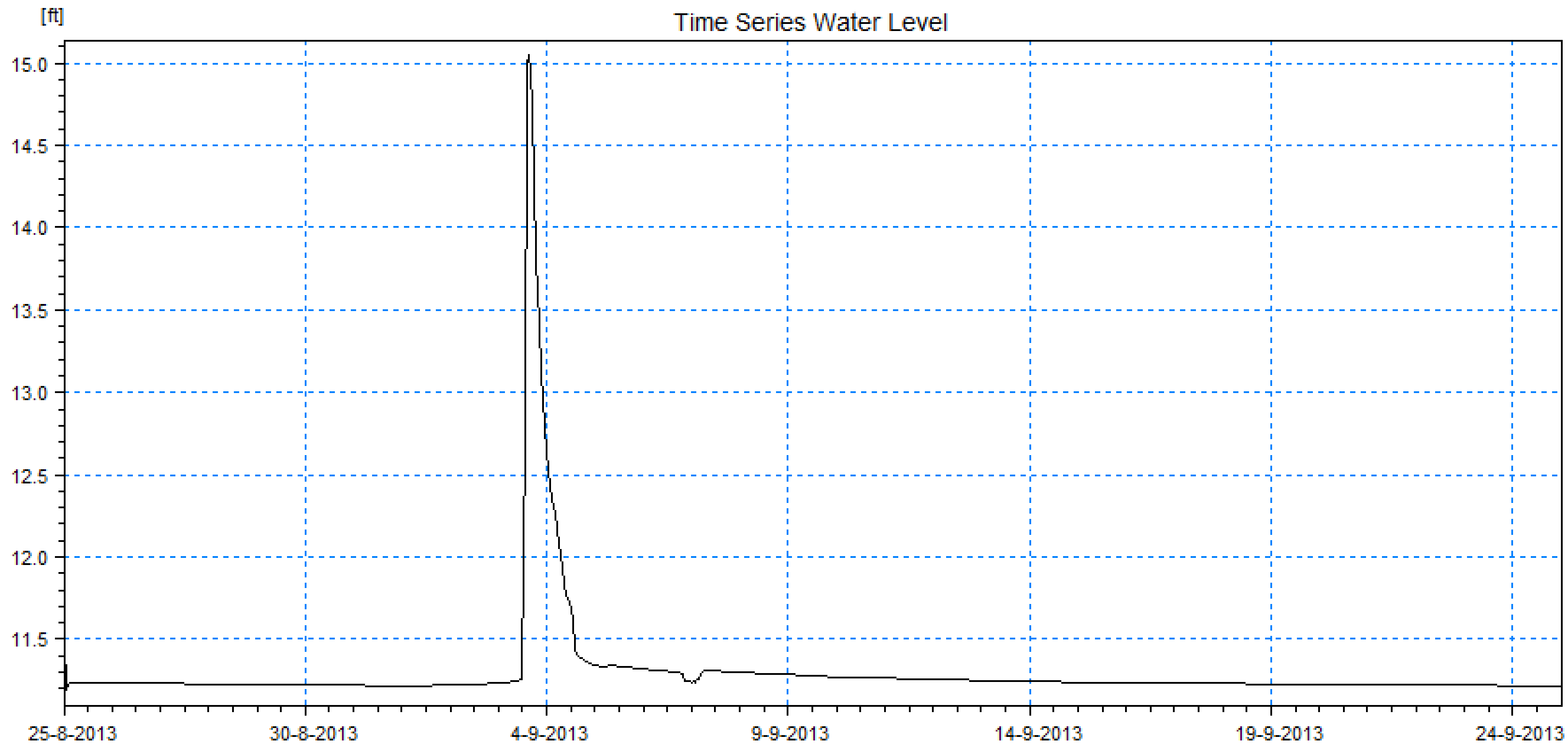


Figure 16. FU5 Discharge (cfs)



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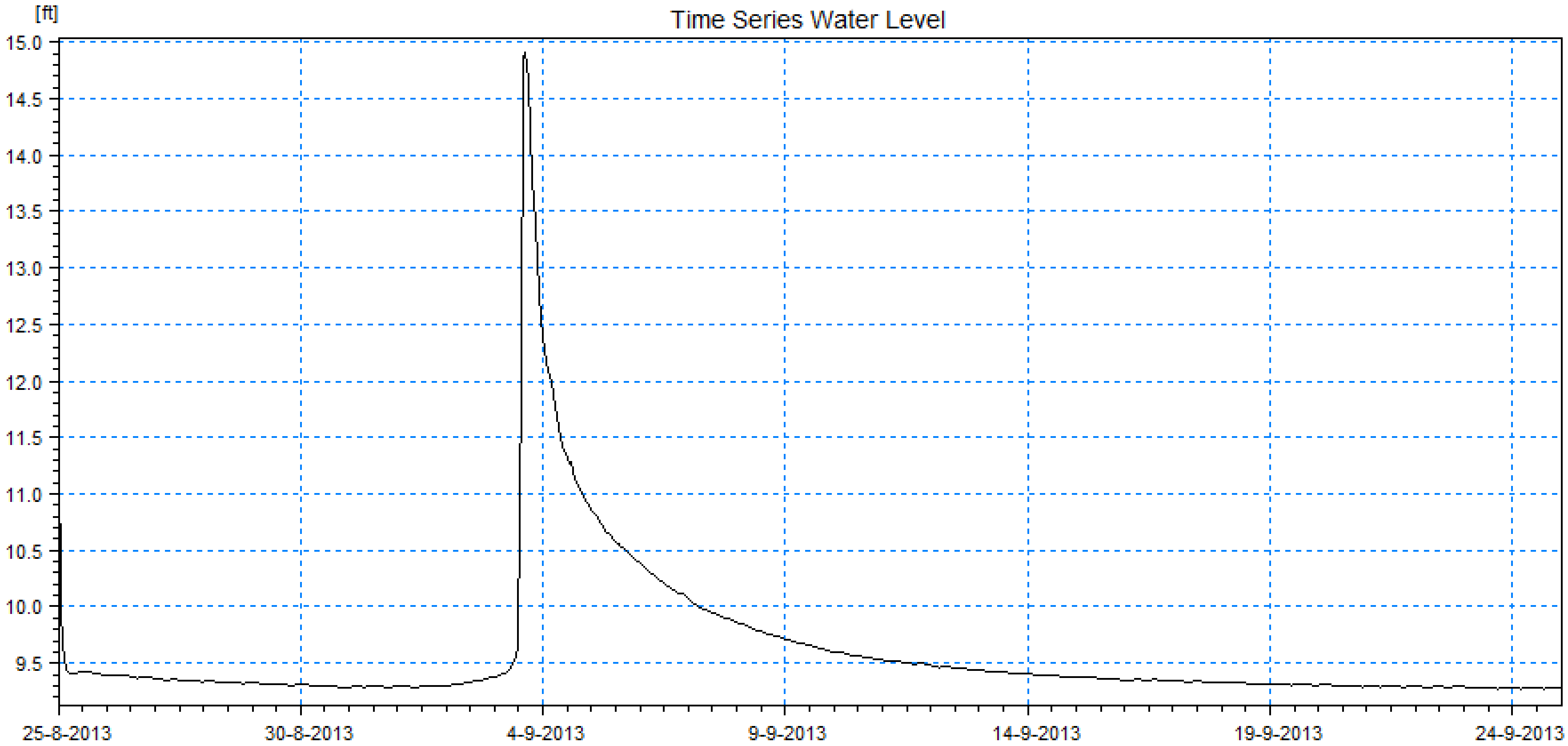


Figure 18. FU5 Tailwater Stage (ft)

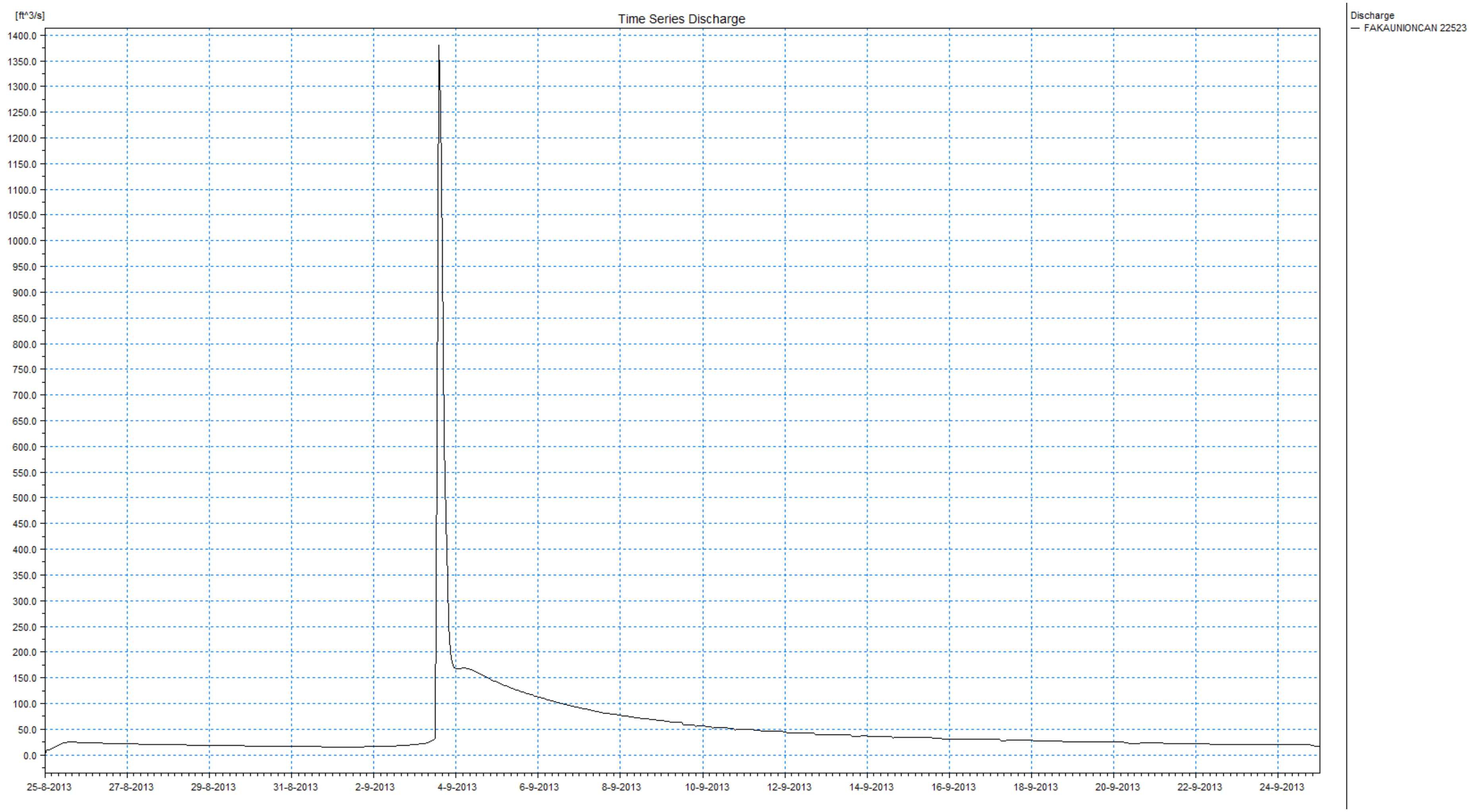


Figure 19. FU6 Discharge (cfs)

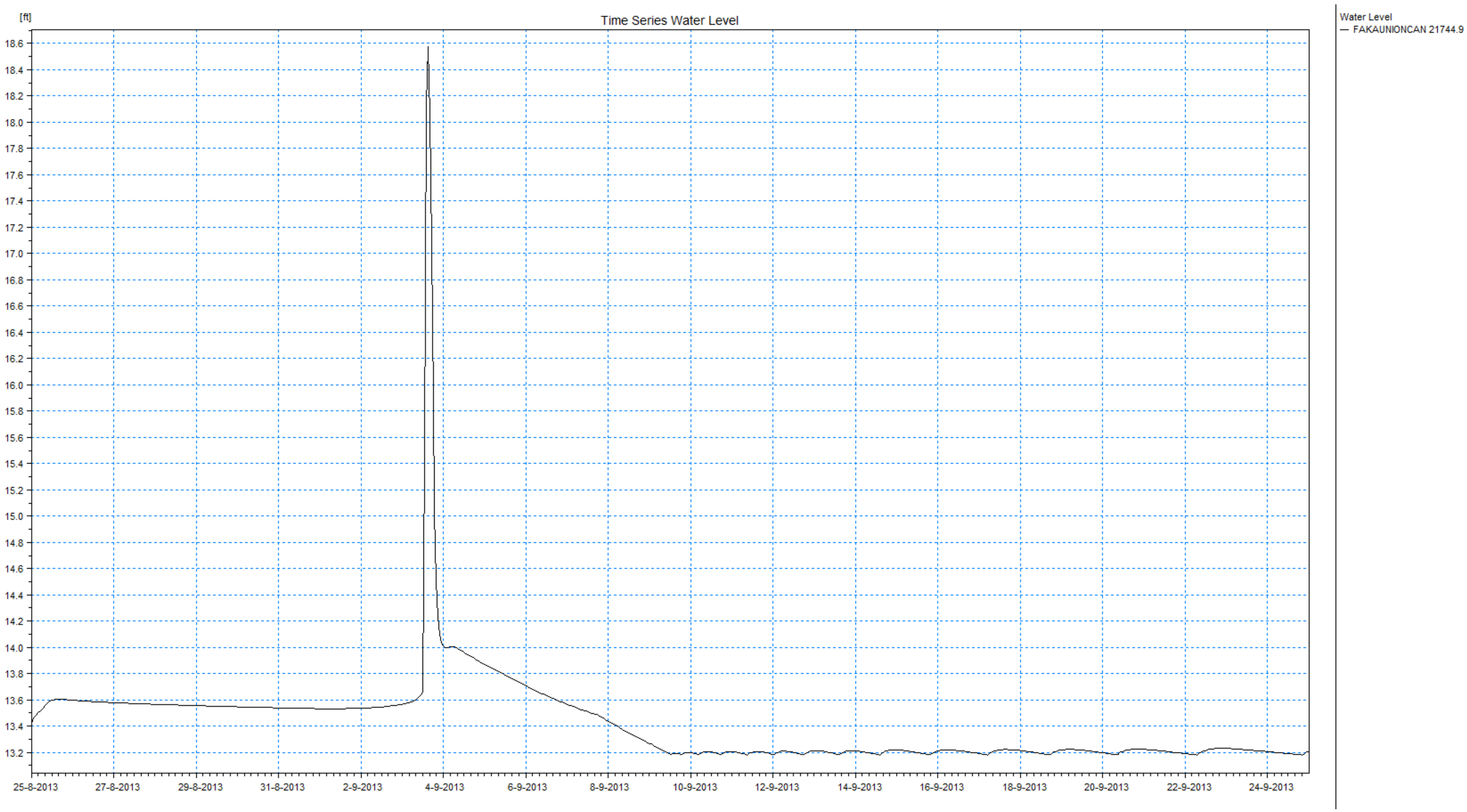


Figure 20. FU6 Headwater Stage (ft)

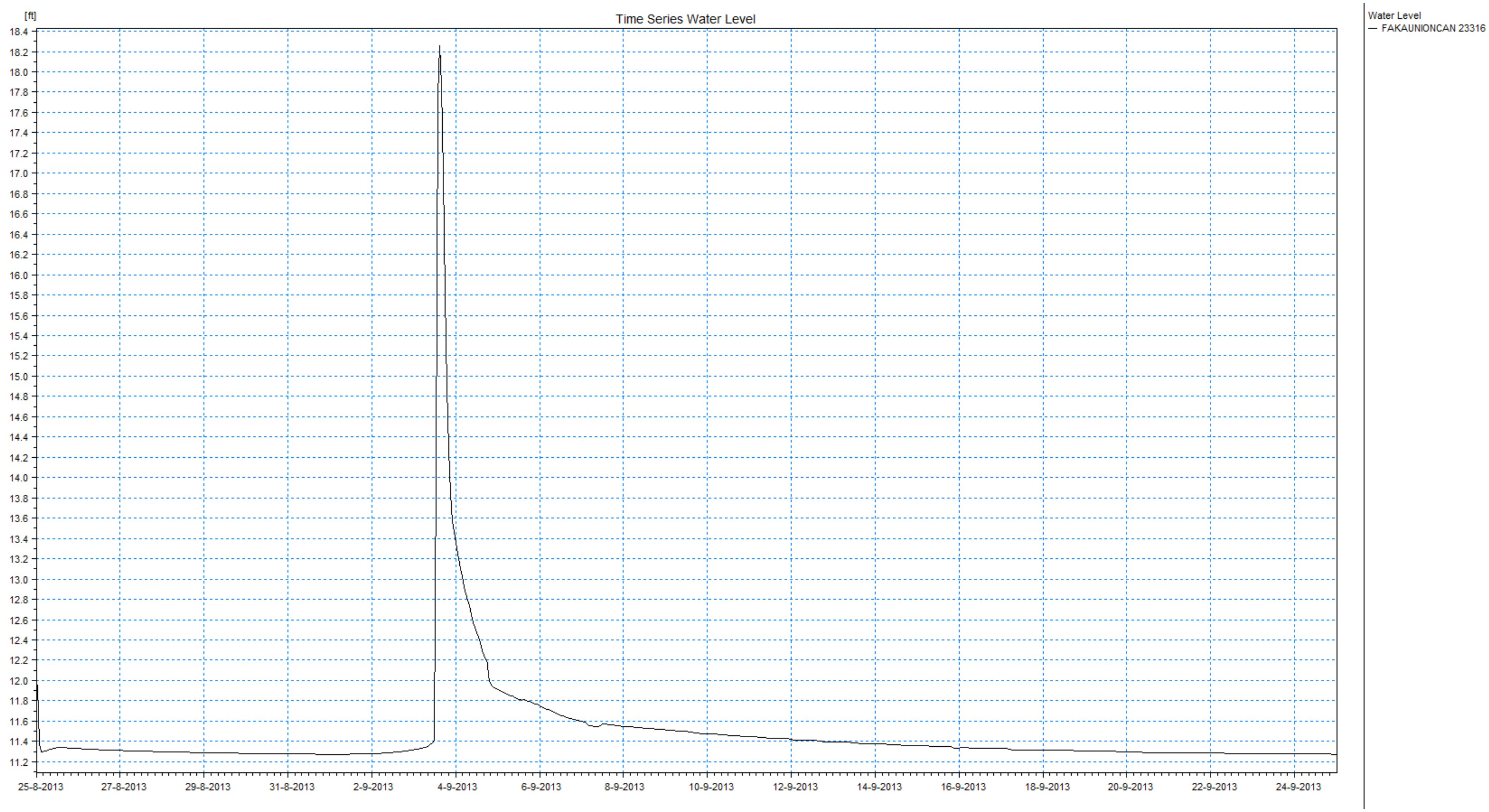


Figure 21. FU6 Tailwater Stage (ft)

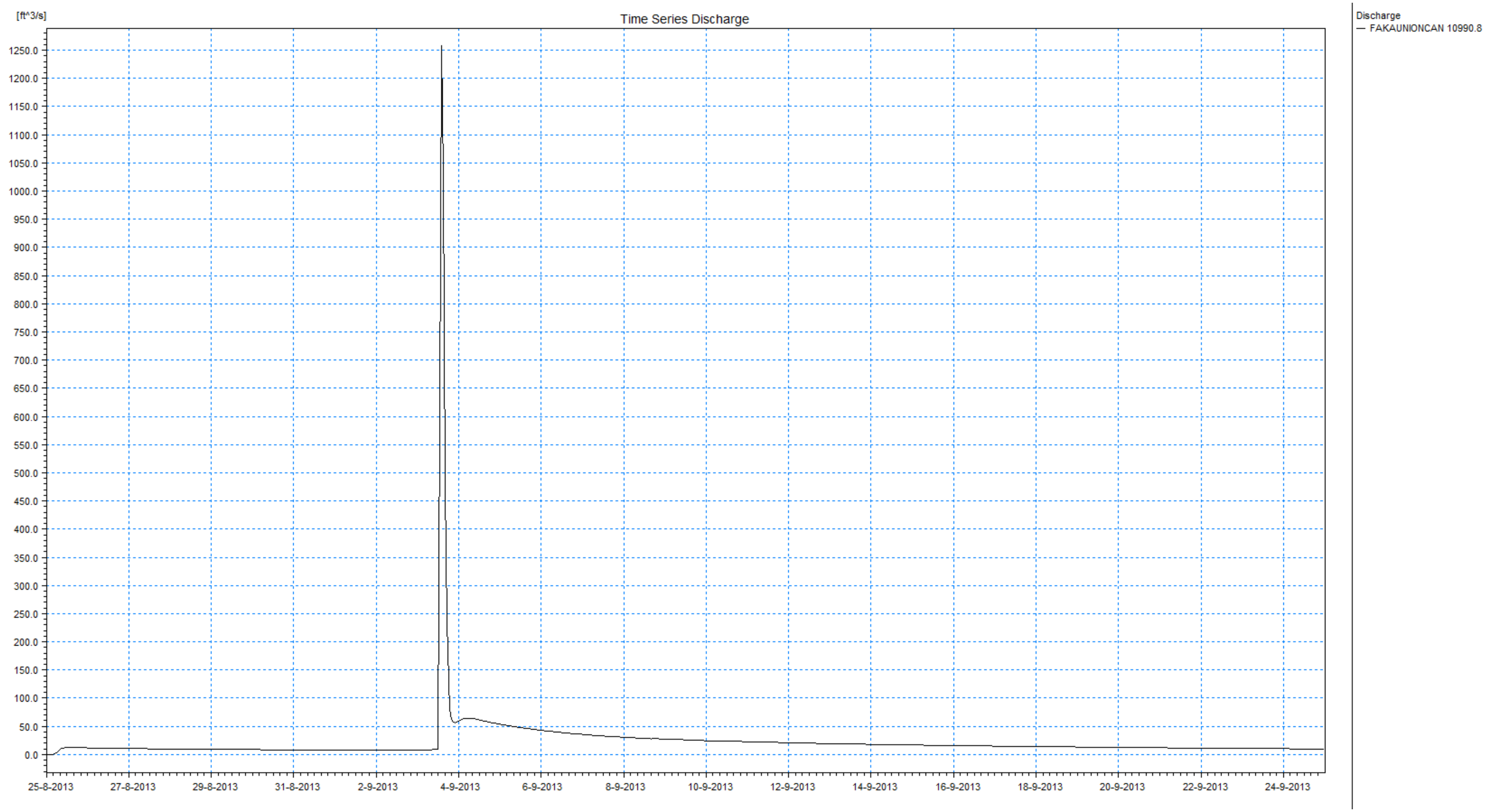


Figure 22. FU7 Discharge (cfs)

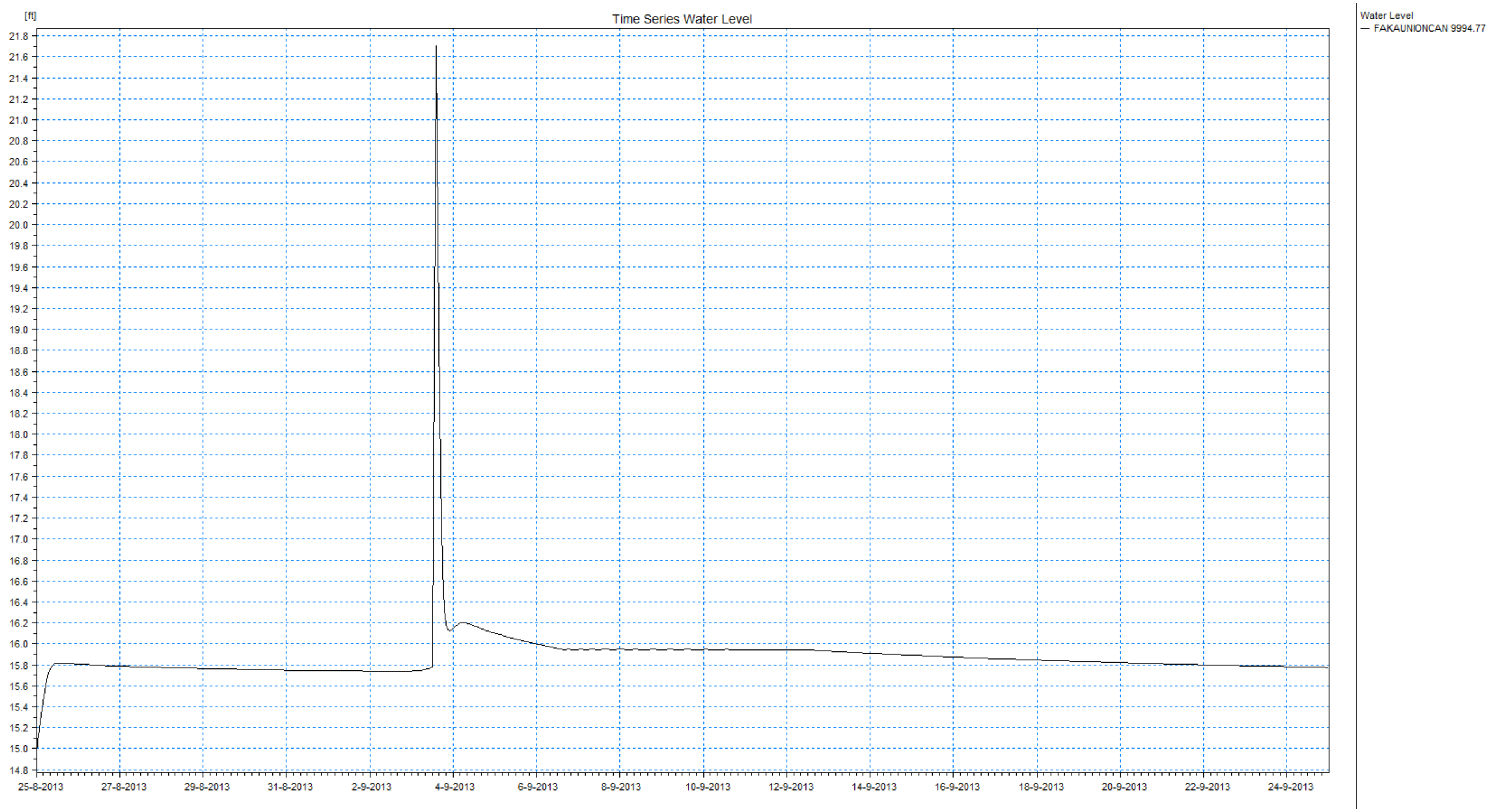


Figure 23. FU7 Headwater Stage (ft)

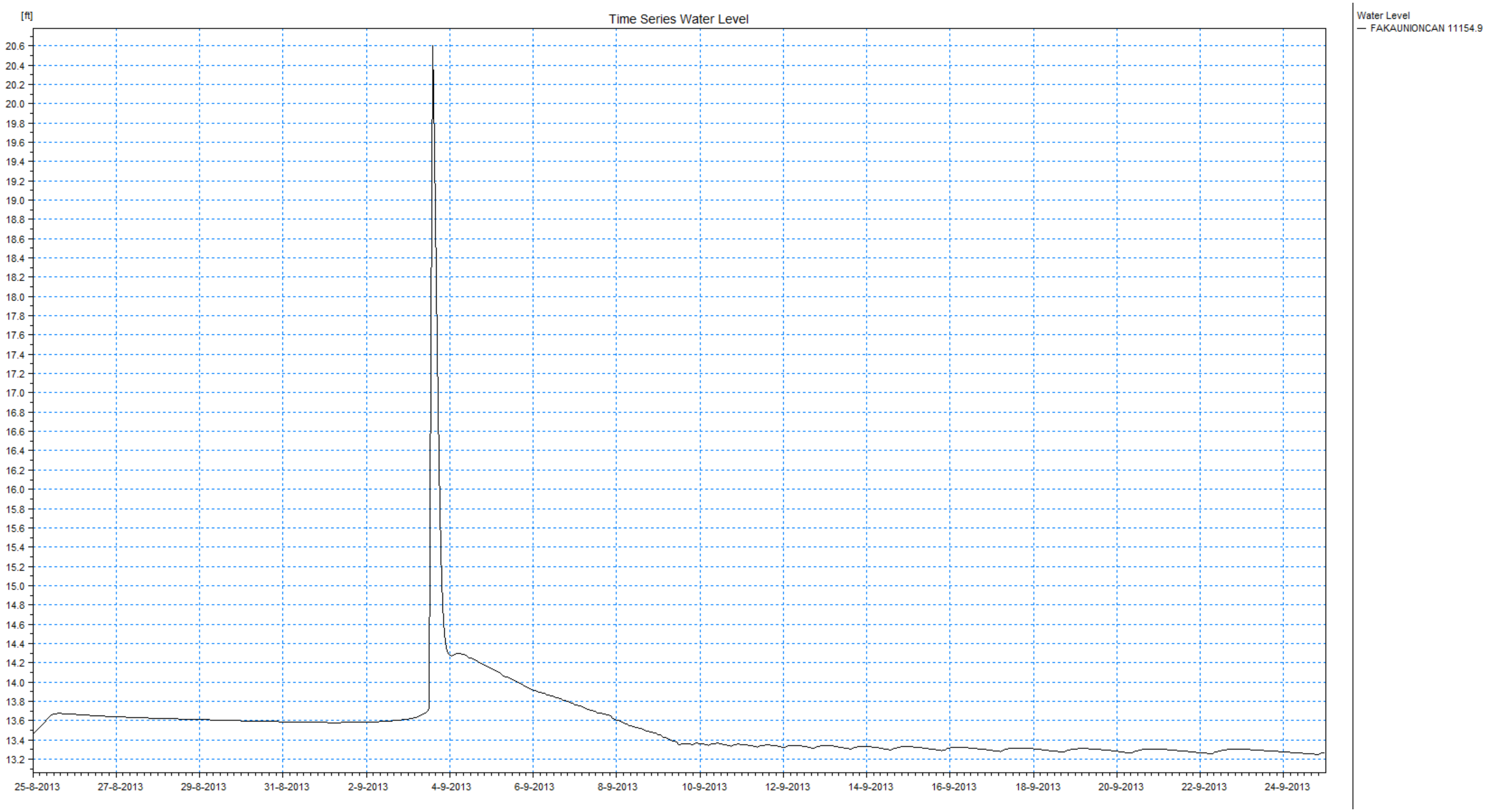


Figure 24. FU7 Tailwater Stage (ft)

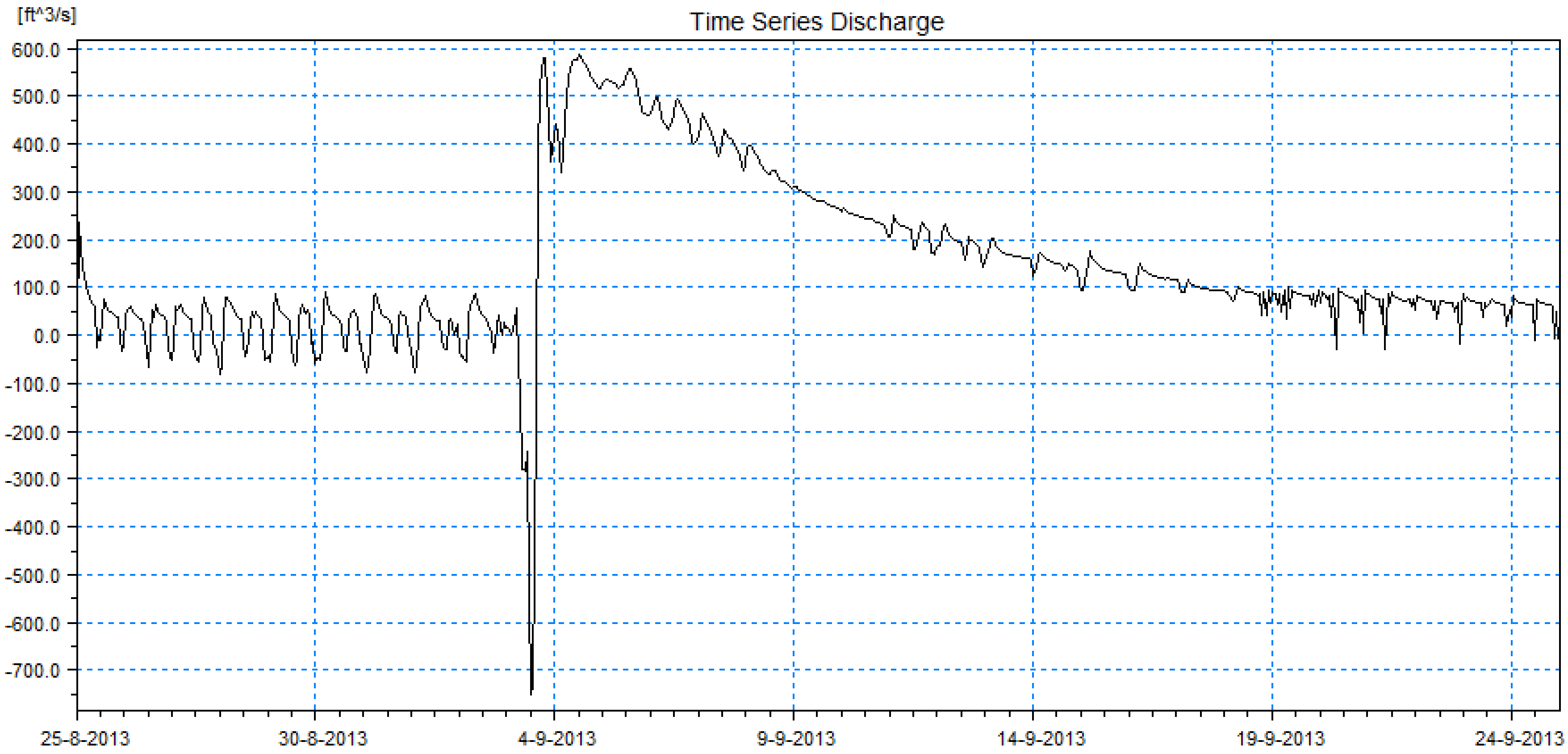


Figure 25. HC1 Discharge (cfs)

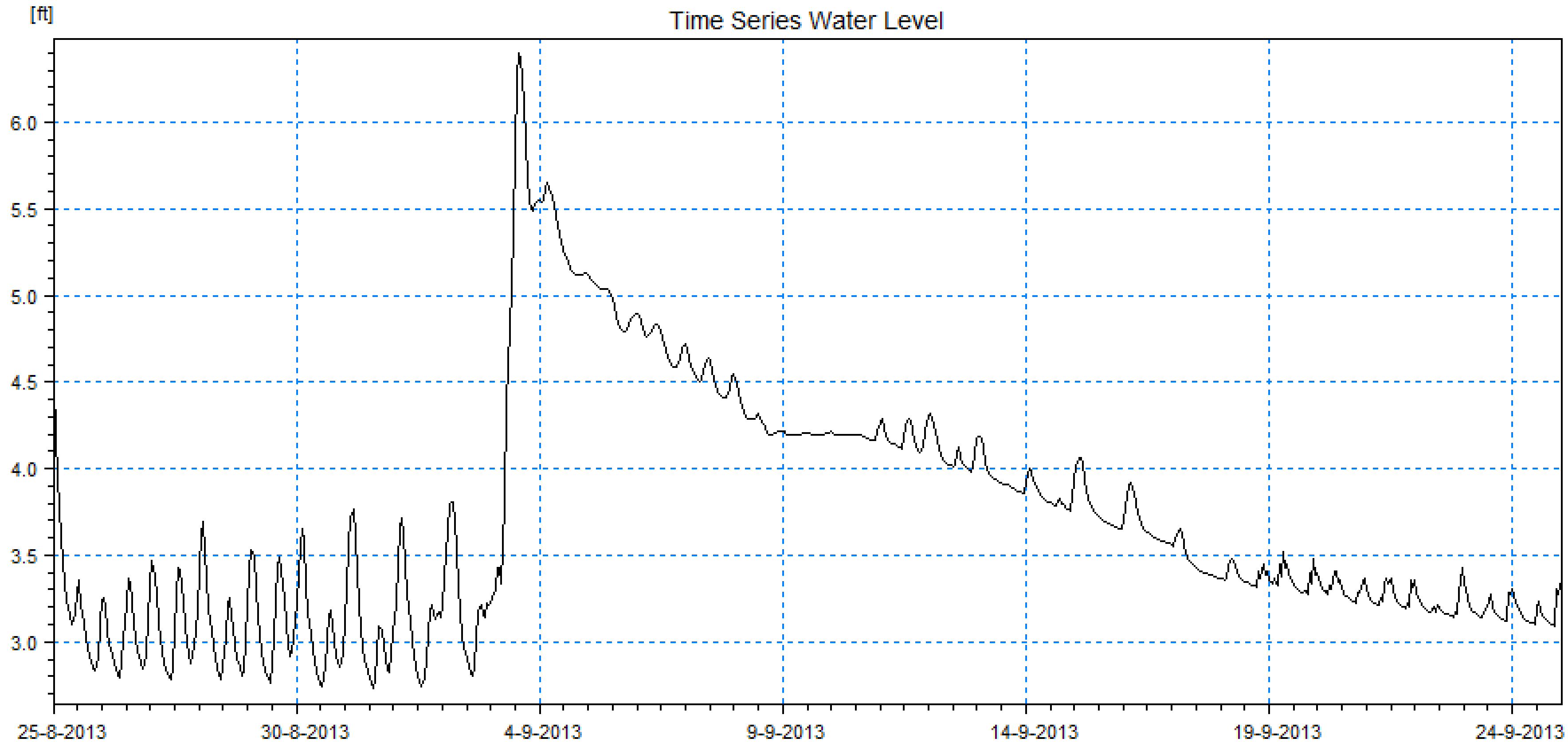
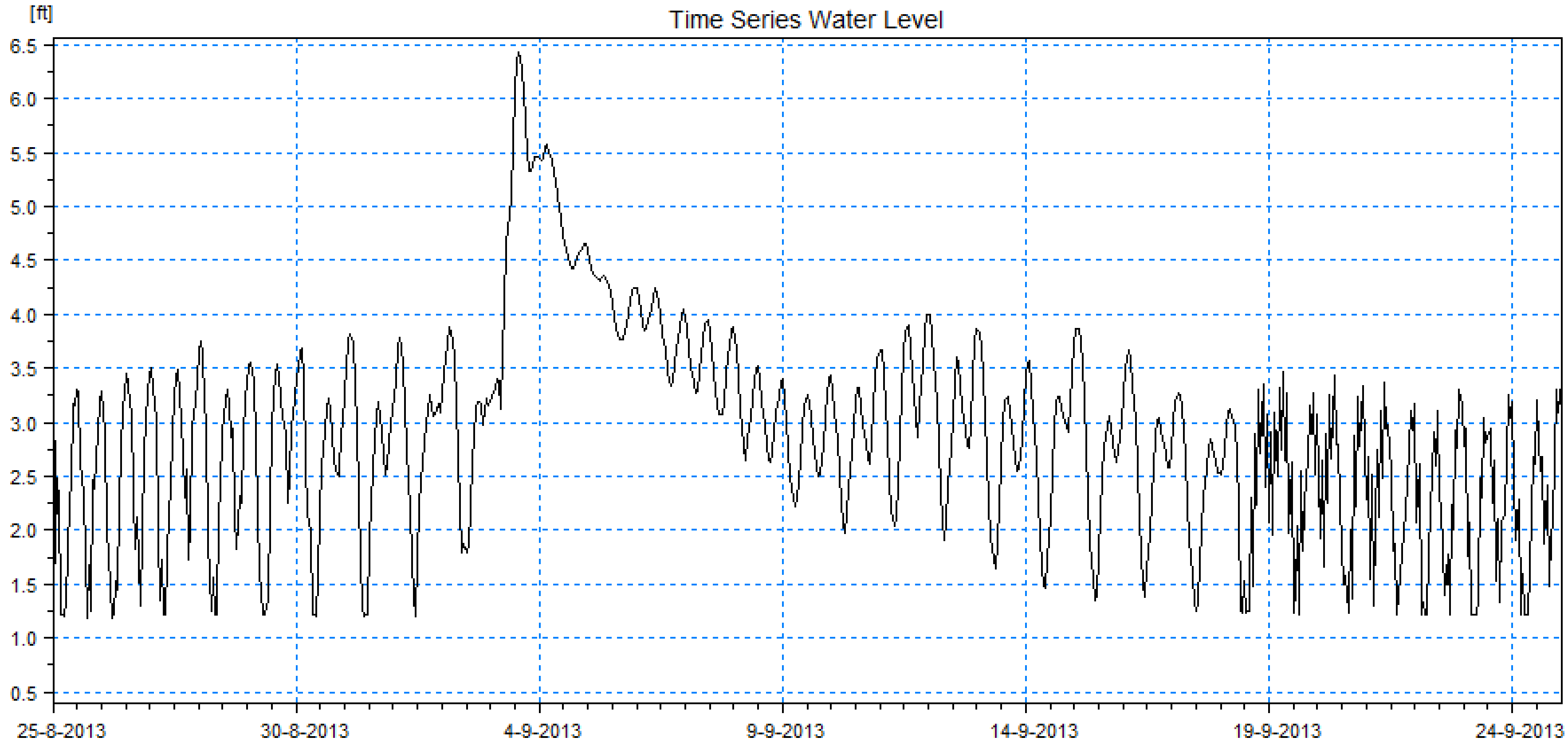


Figure 26. HC1 Headwater Stage (ft)



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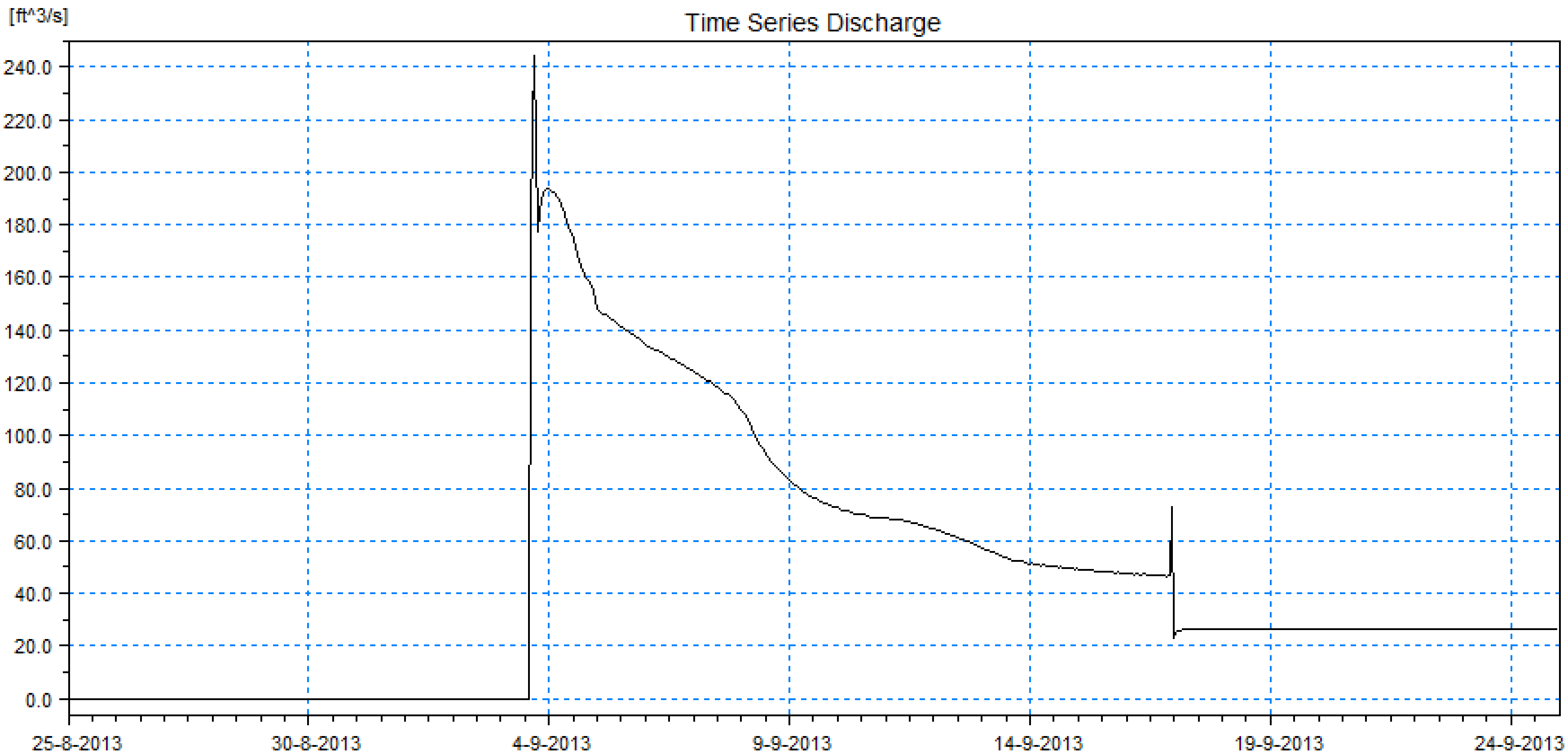


Figure 28. HC2 Discharge (cfs)

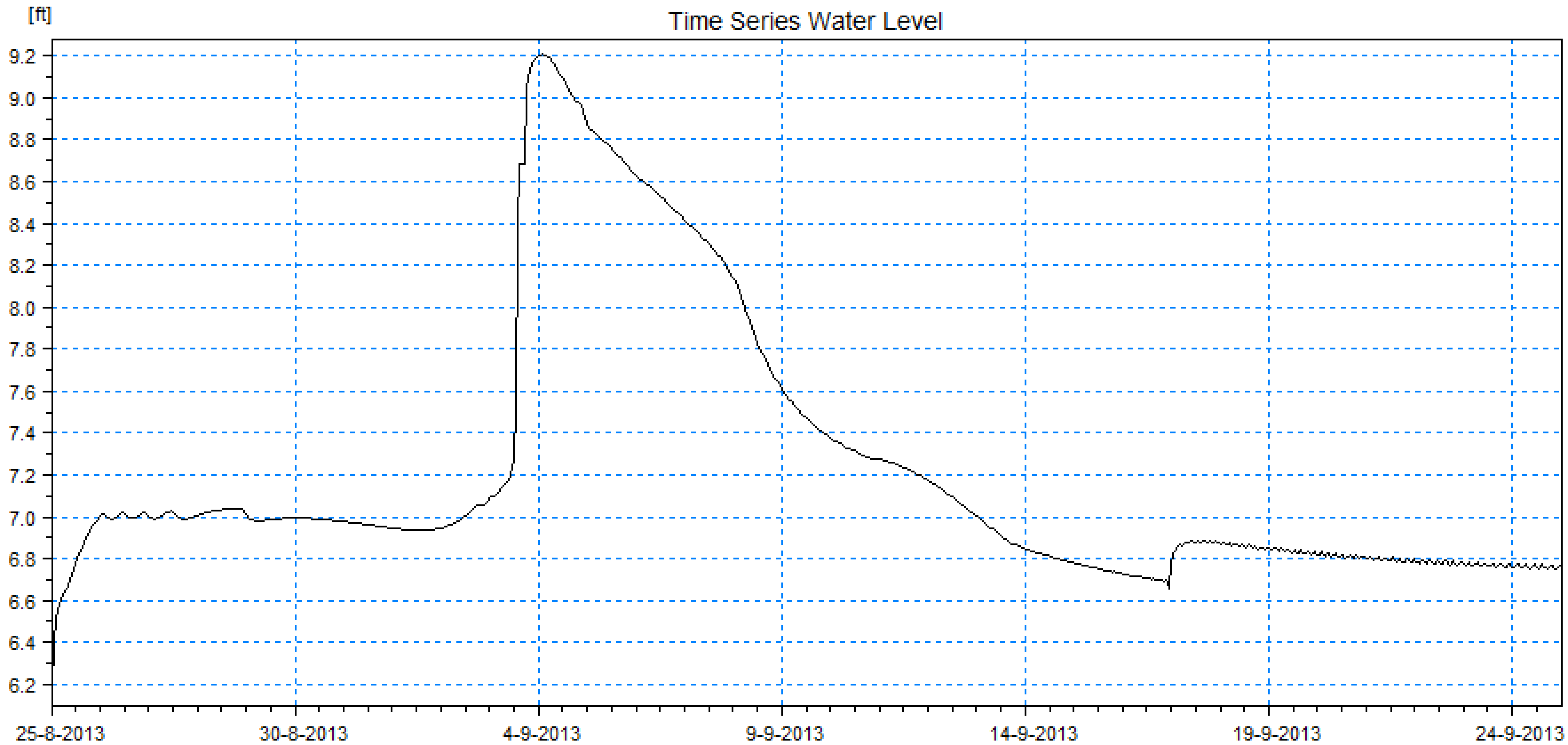


Figure 29. HC2 Headwater Stage (ft)

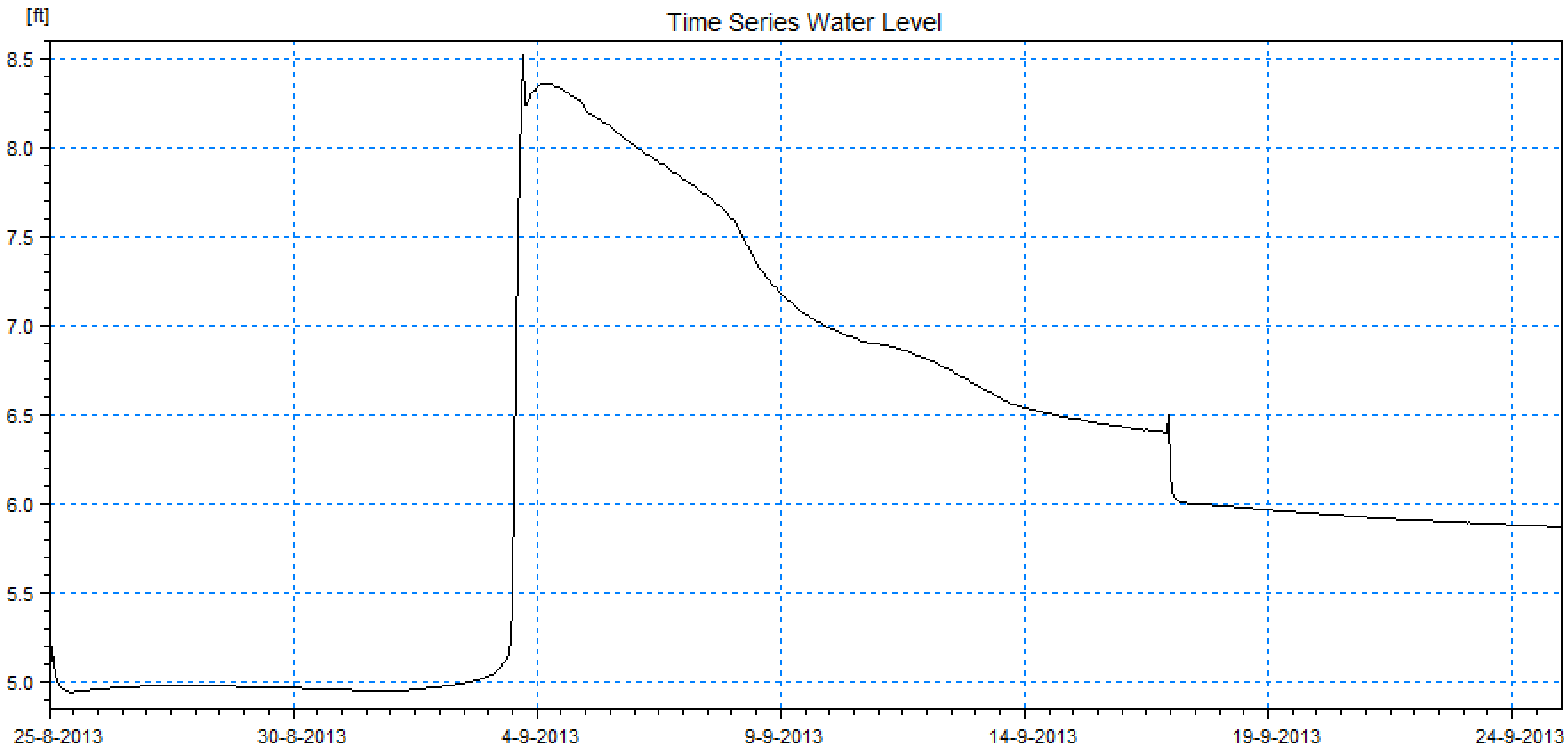


Figure 30. HC2 Tailwater Stage (ft)

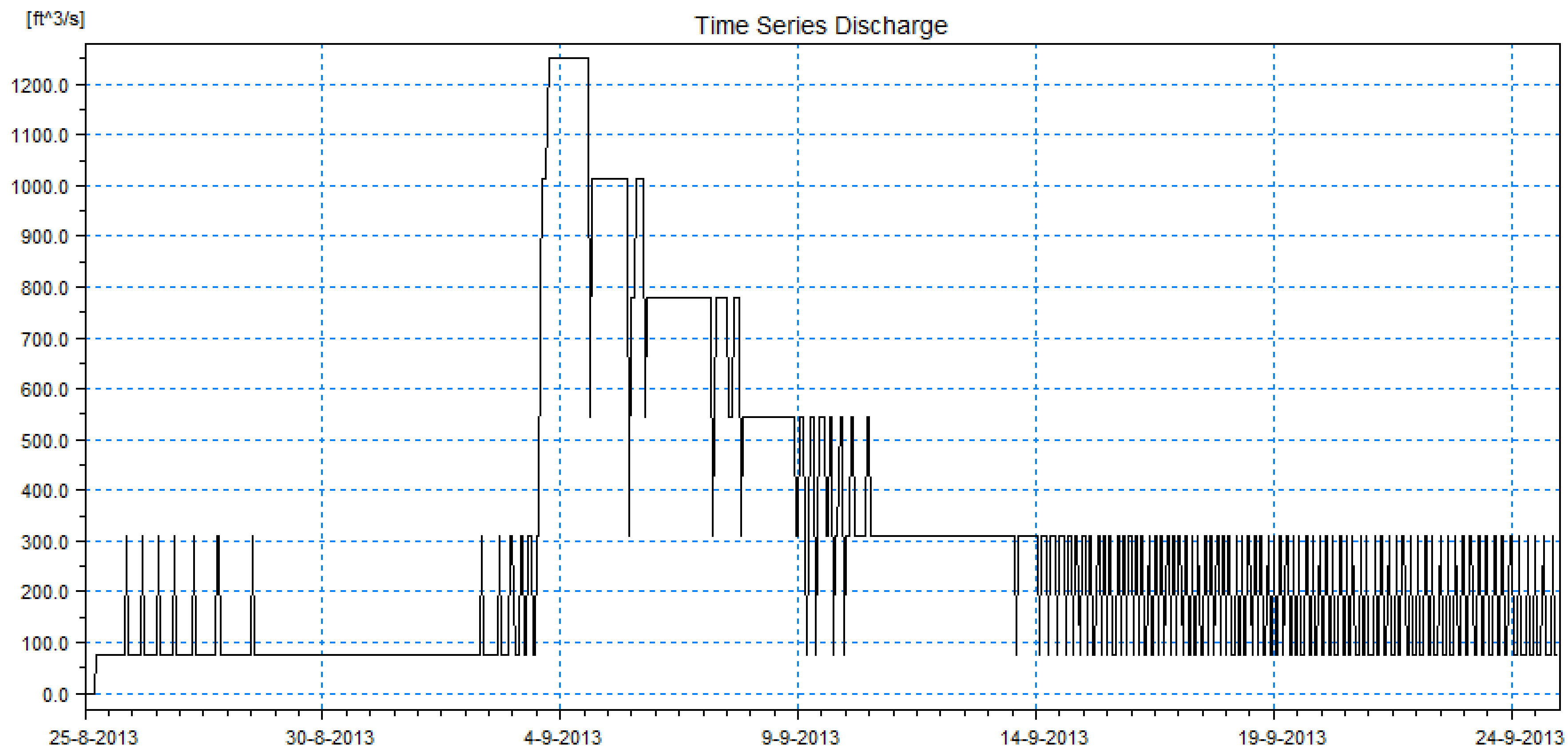


Figure 31. Miller Pump Discharge (cfs)

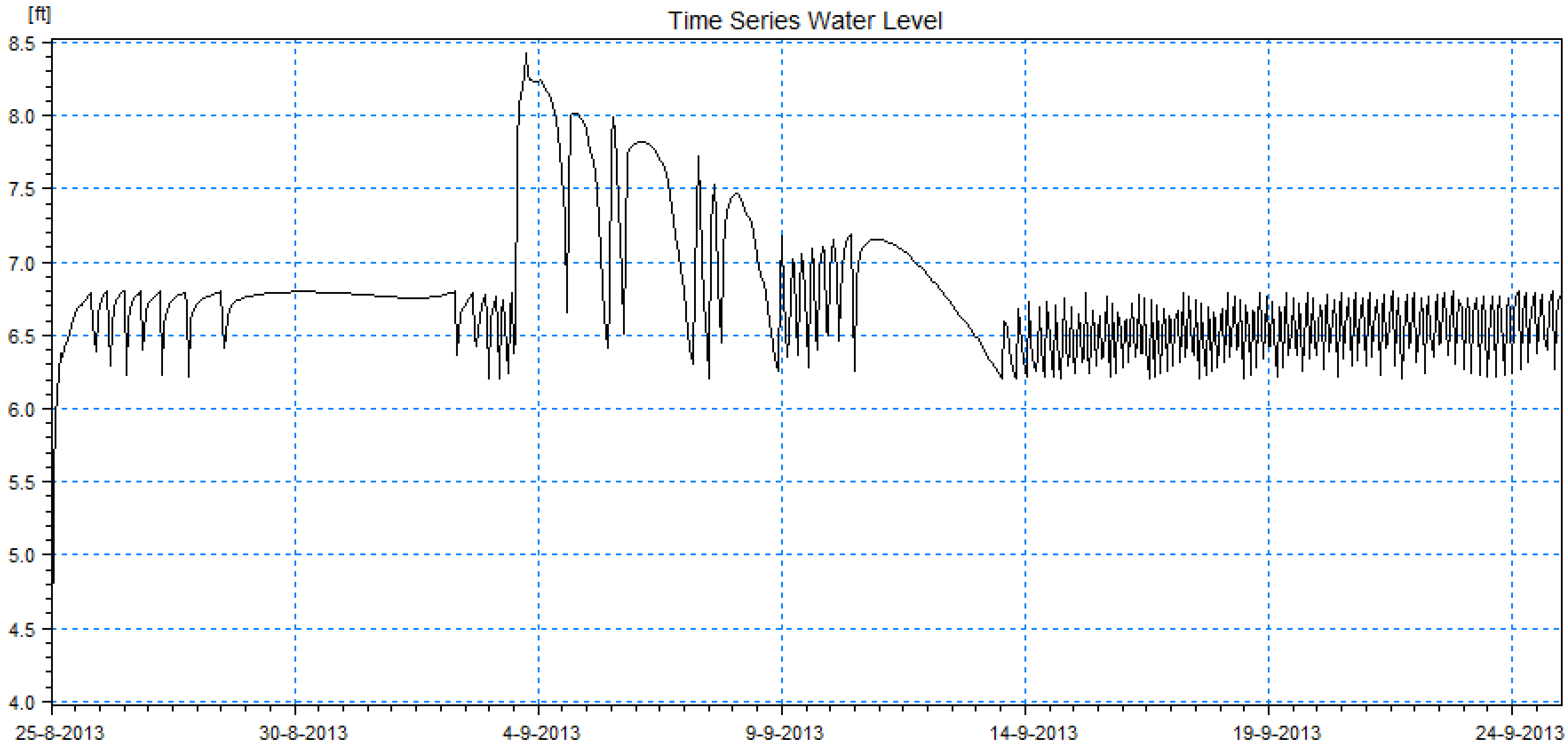


Figure 32. Miller Pump Headwater Stage (ft)

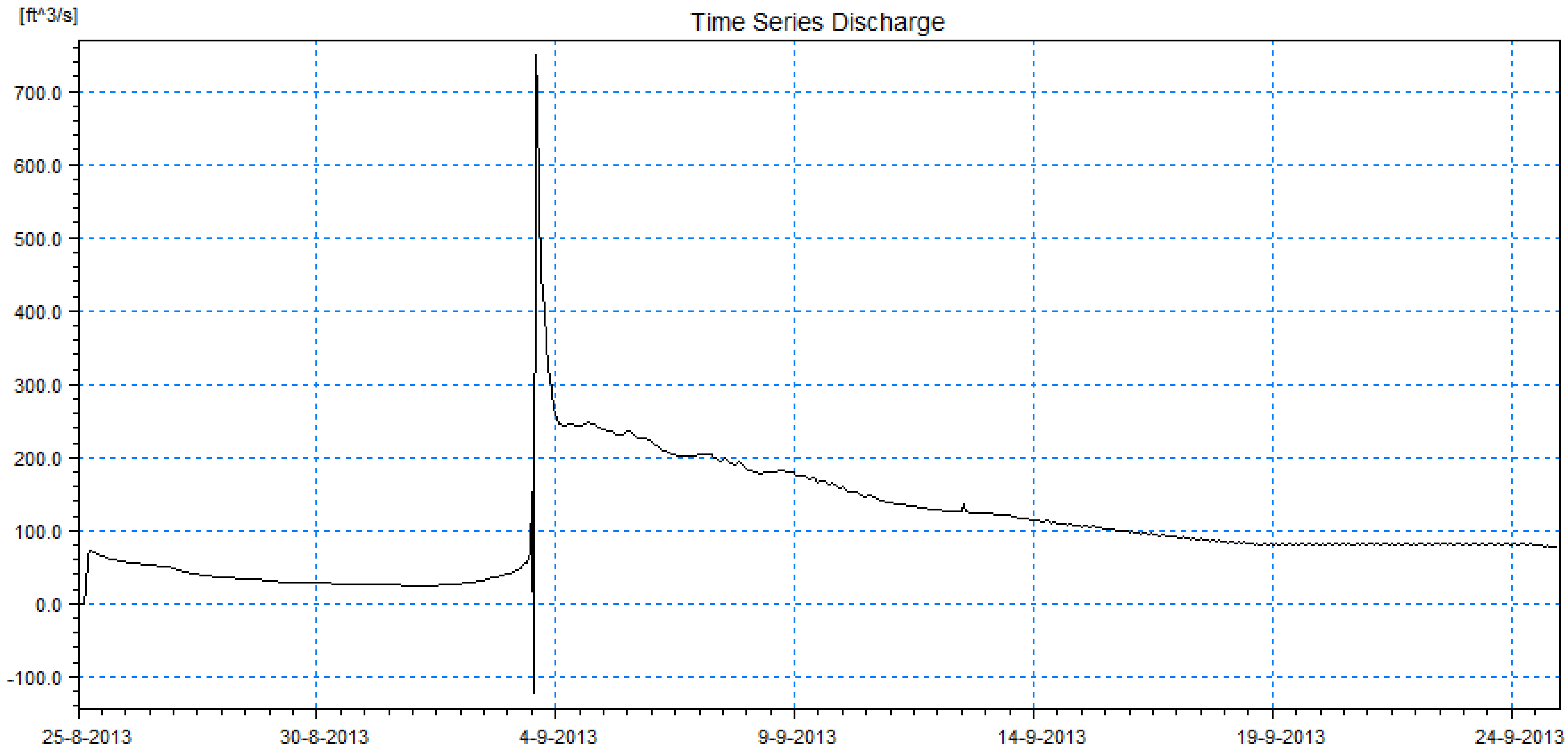


Figure 33. Miller3 Discharge (cfs)

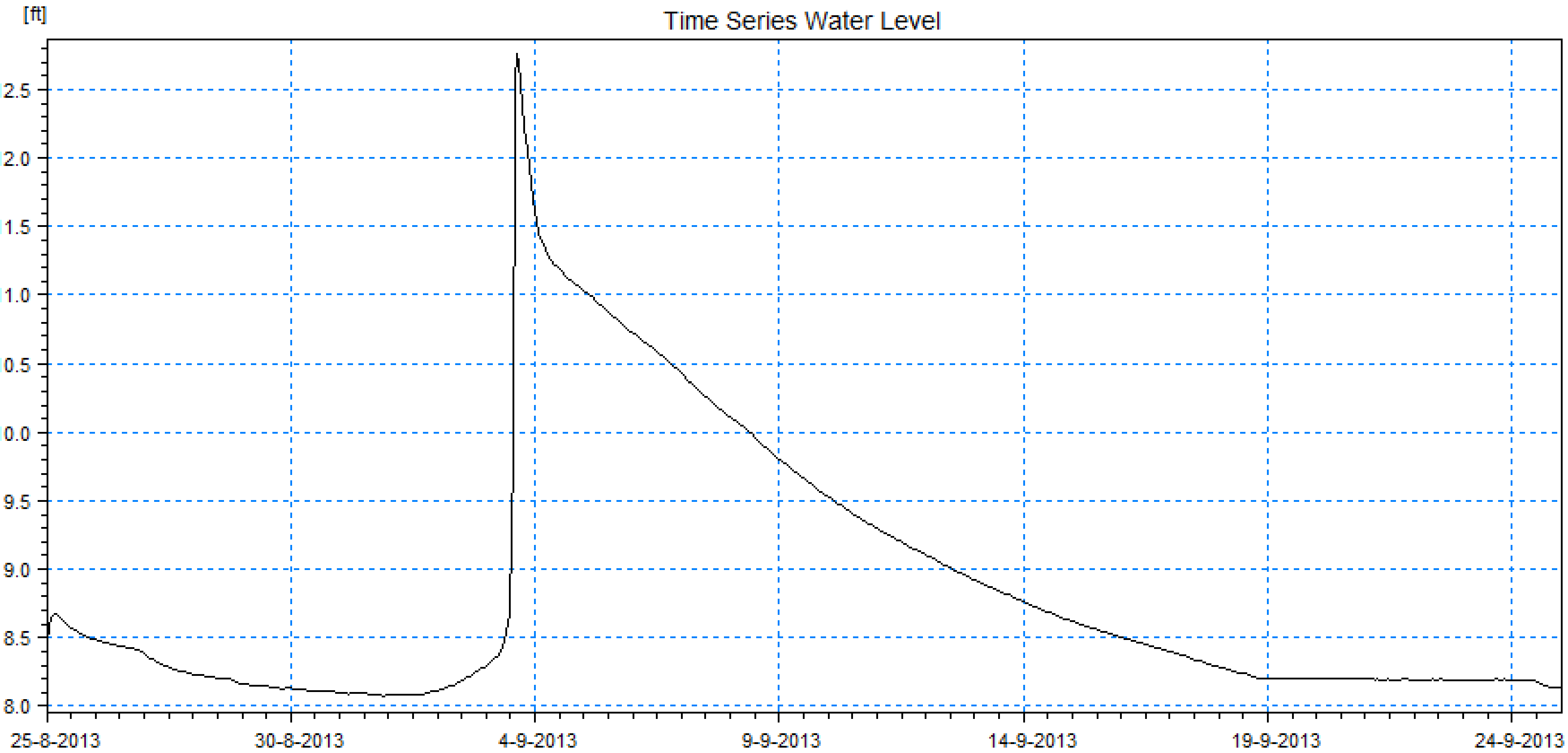
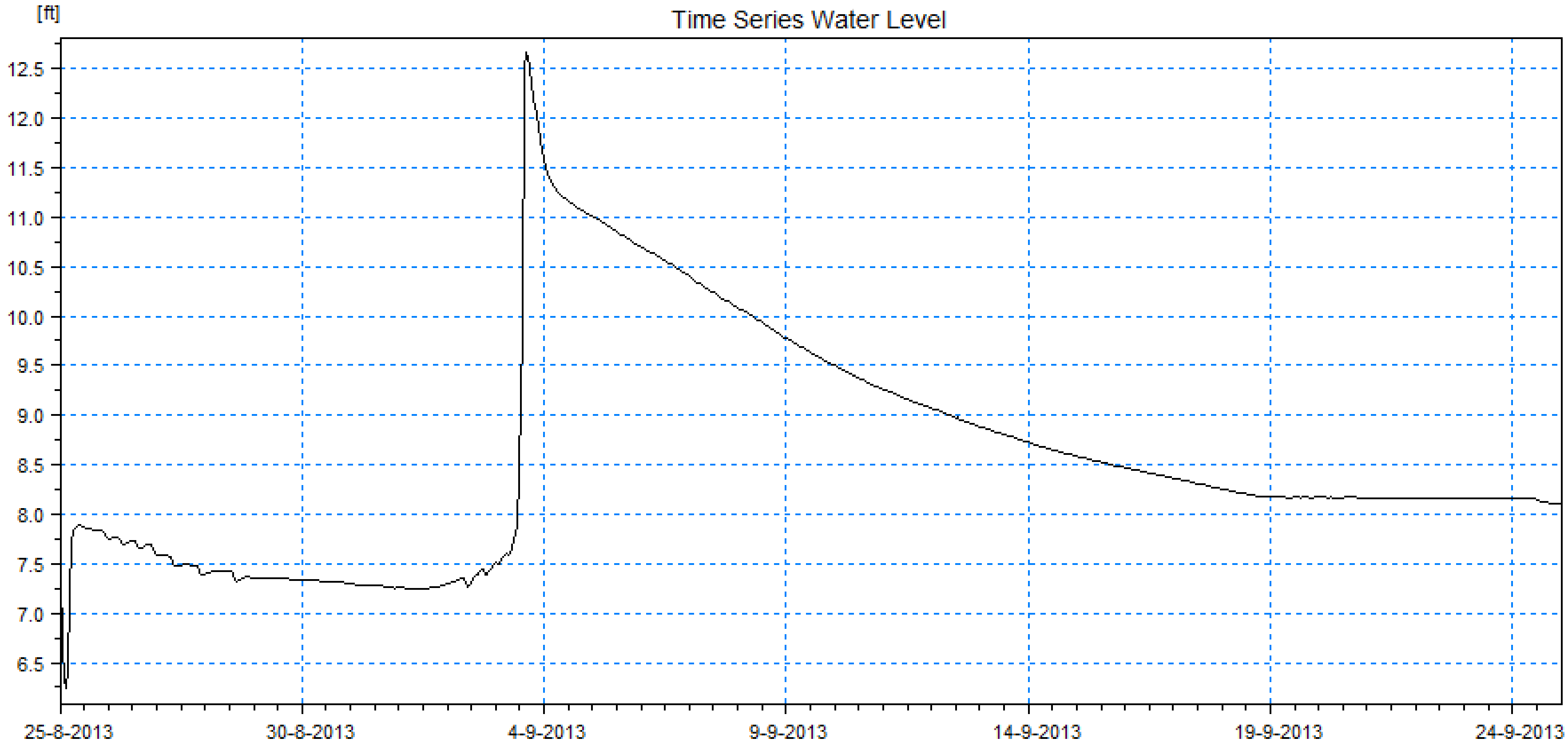


Figure 34. Miller3 Headwater Stage (ft)



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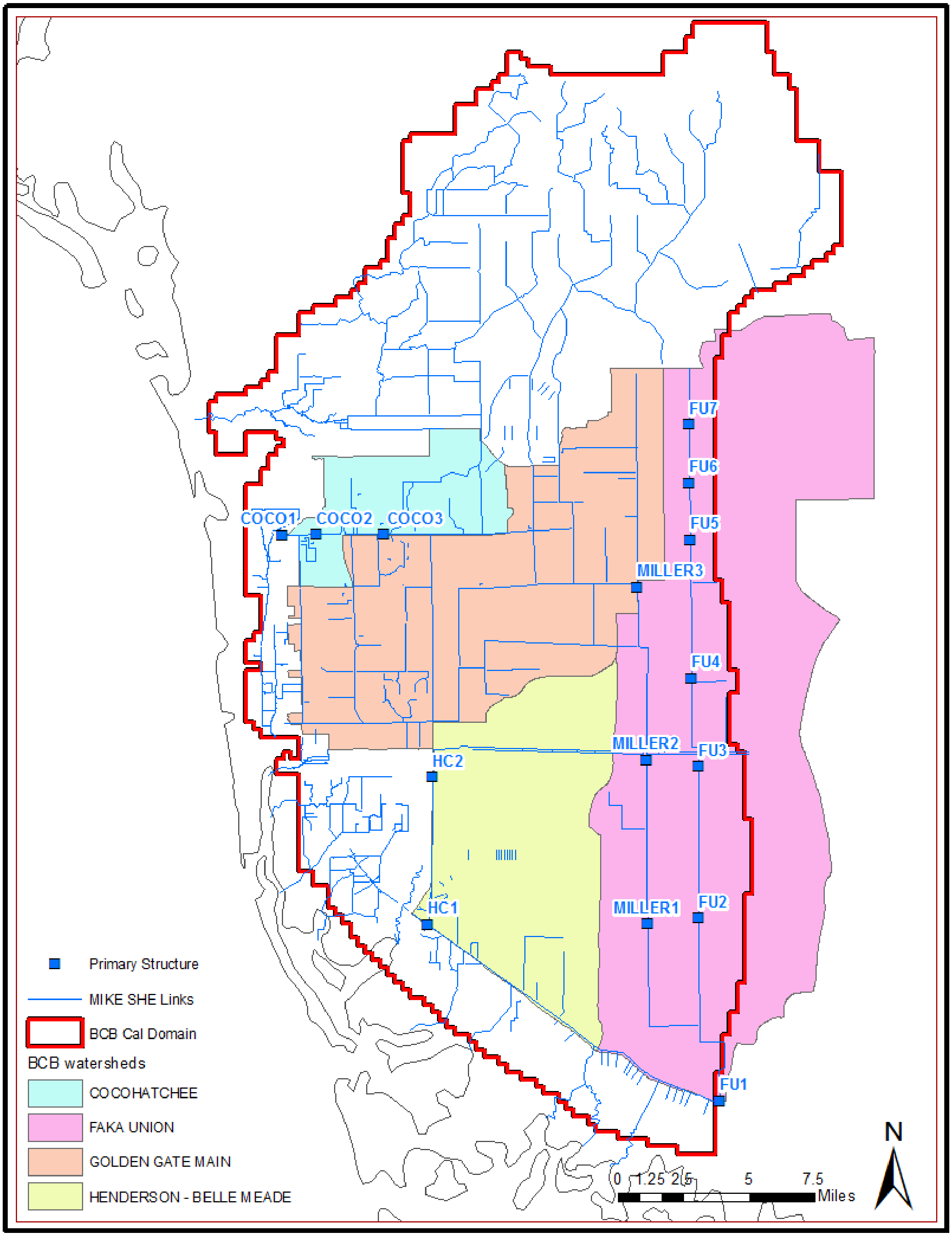

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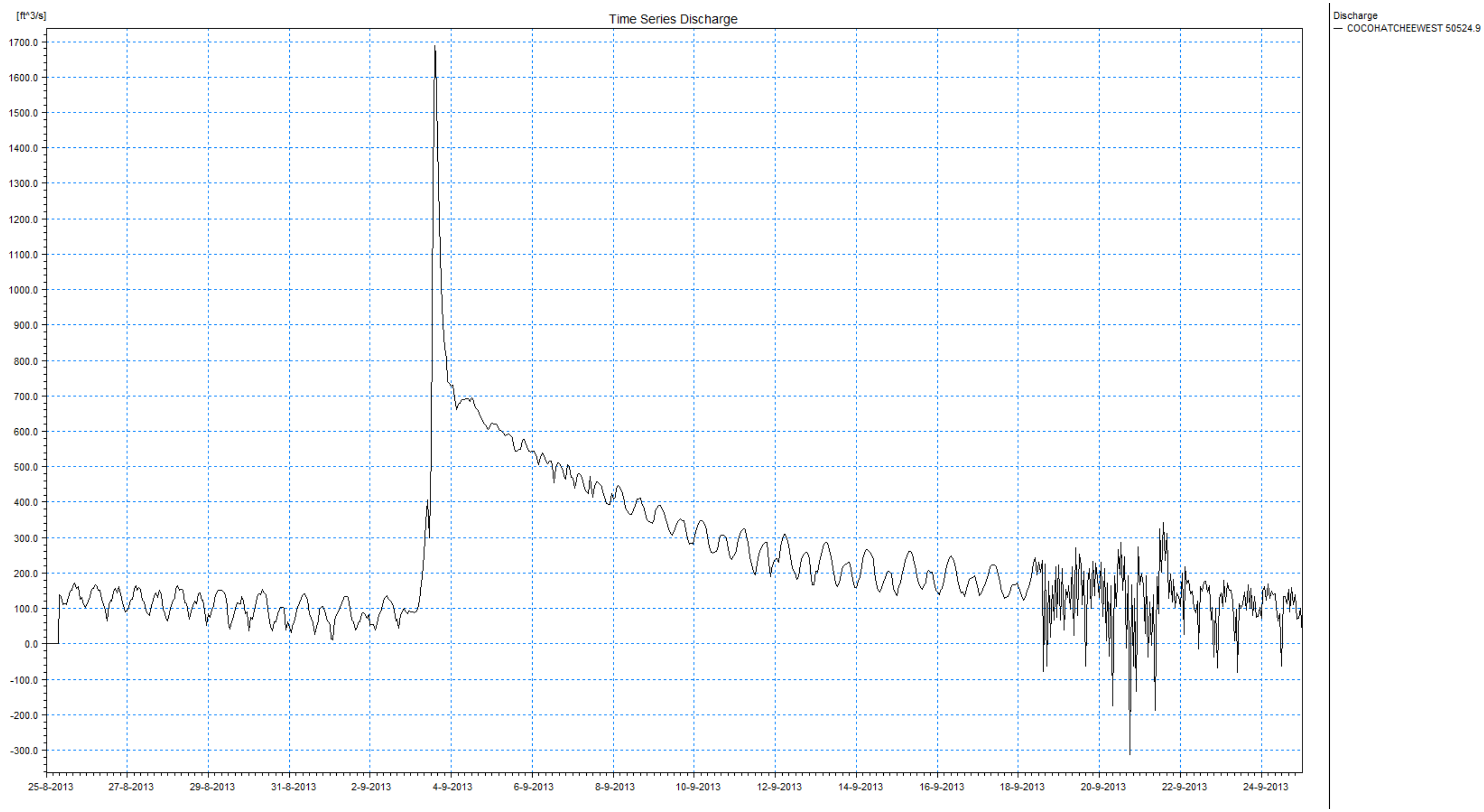


Figure 2. COCO1 Discharge (cfs)

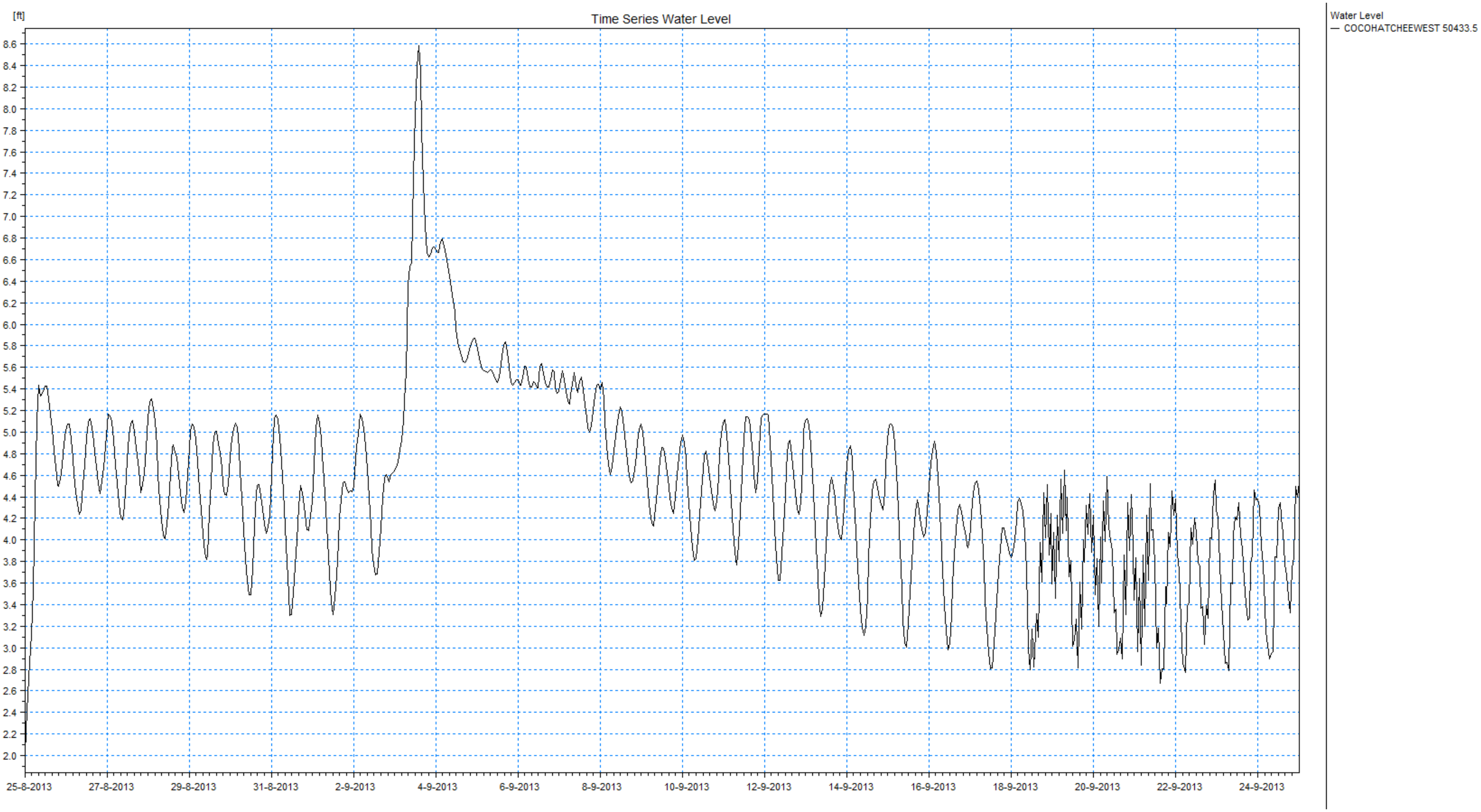


Figure 3. COCO1 Headwater Stage (ft)

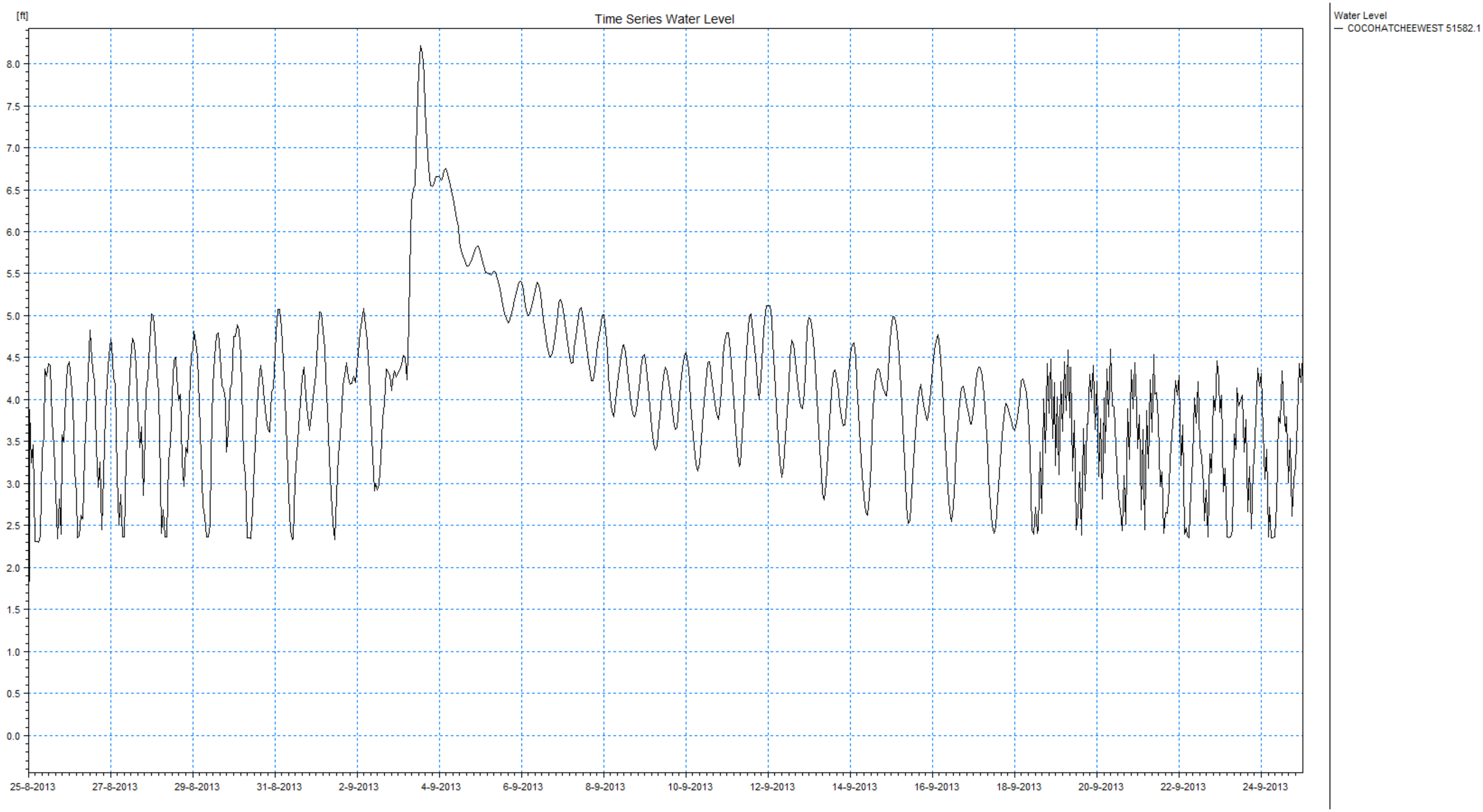


Figure 4. COCO1 Tailwater Stage (ft)

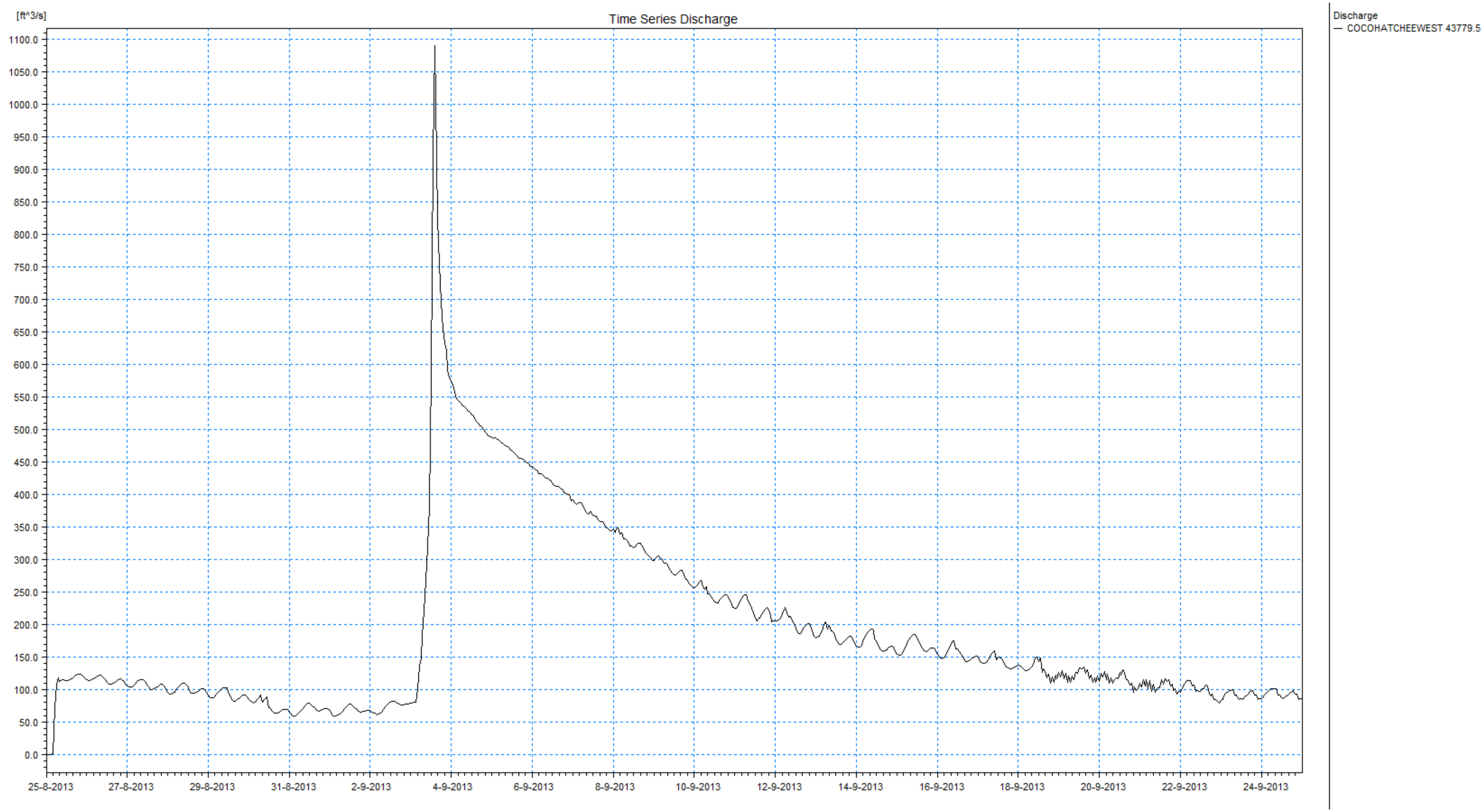


Figure 5. COCO2 Discharge (cfs)

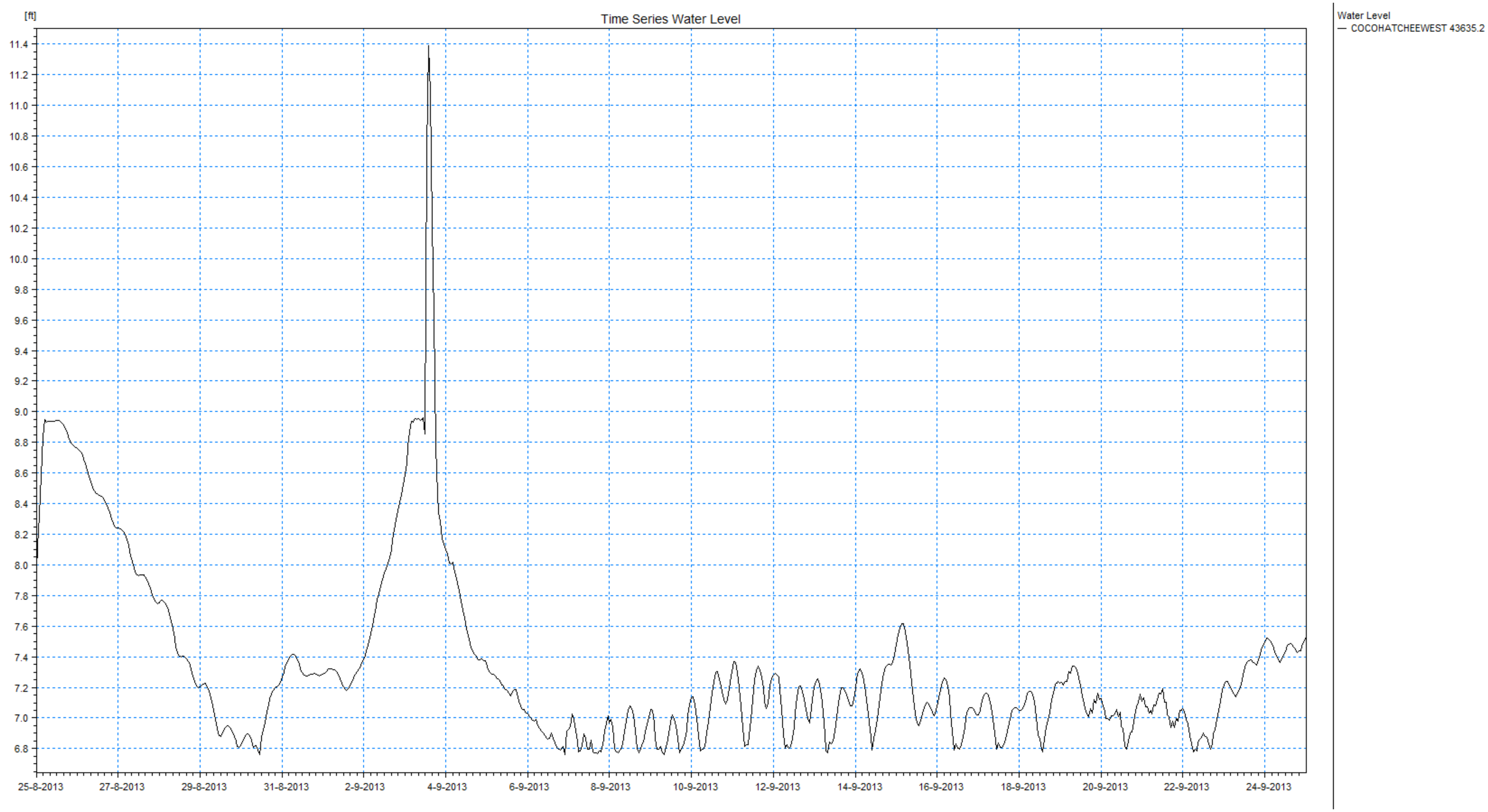


Figure 6. COCO2 Headwater Stage (ft)

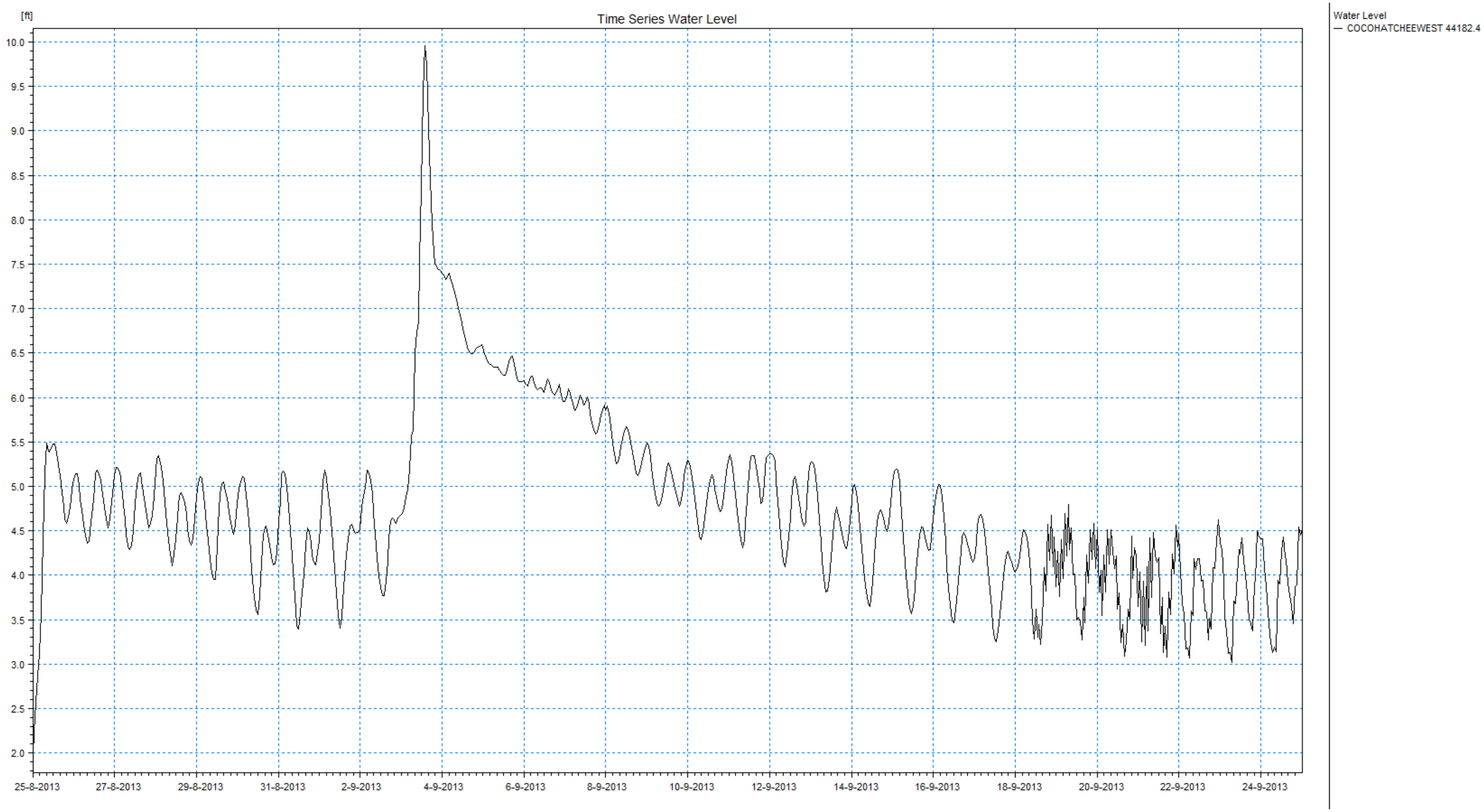


Figure 7. COCO2 Tailwater Stage (ft)

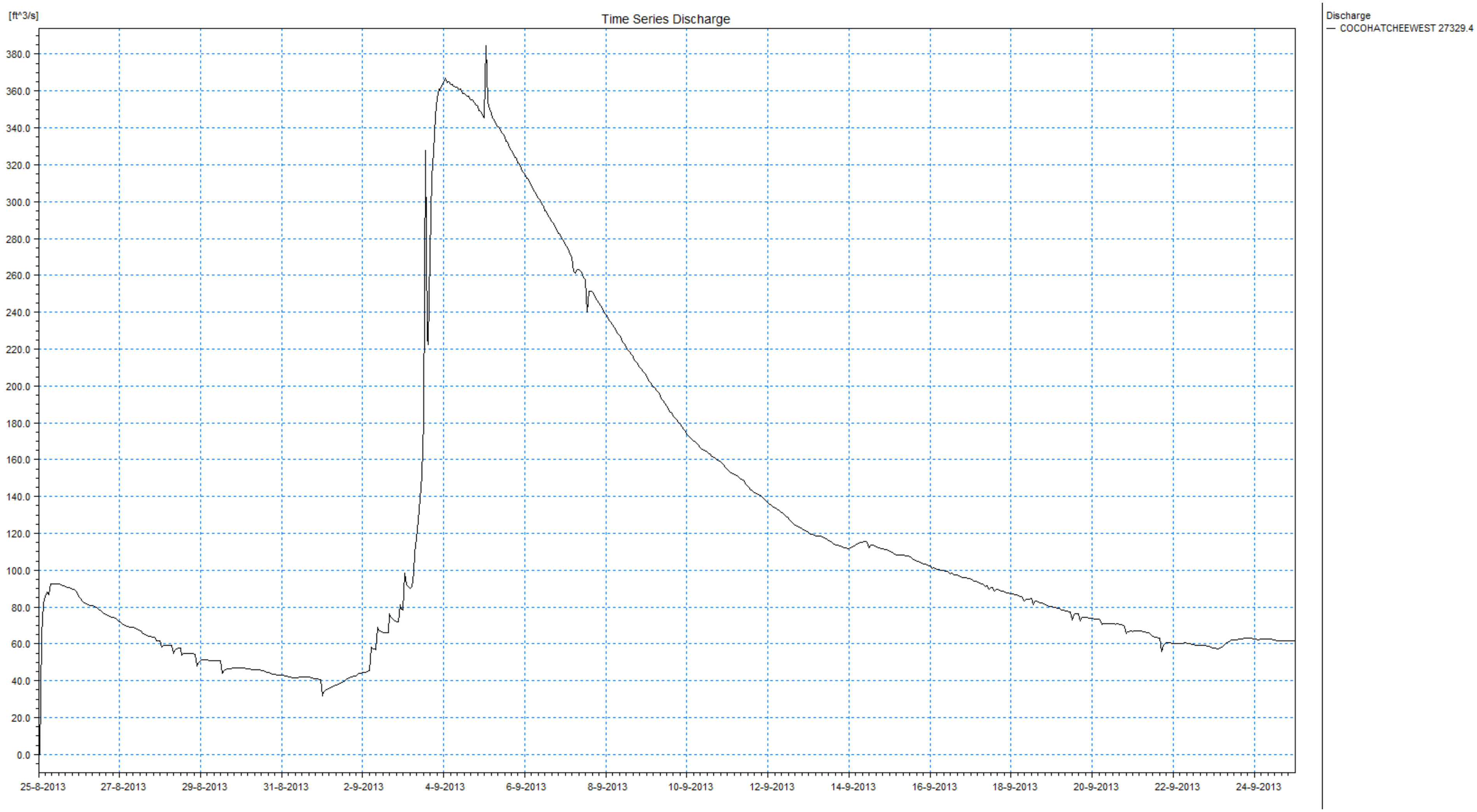
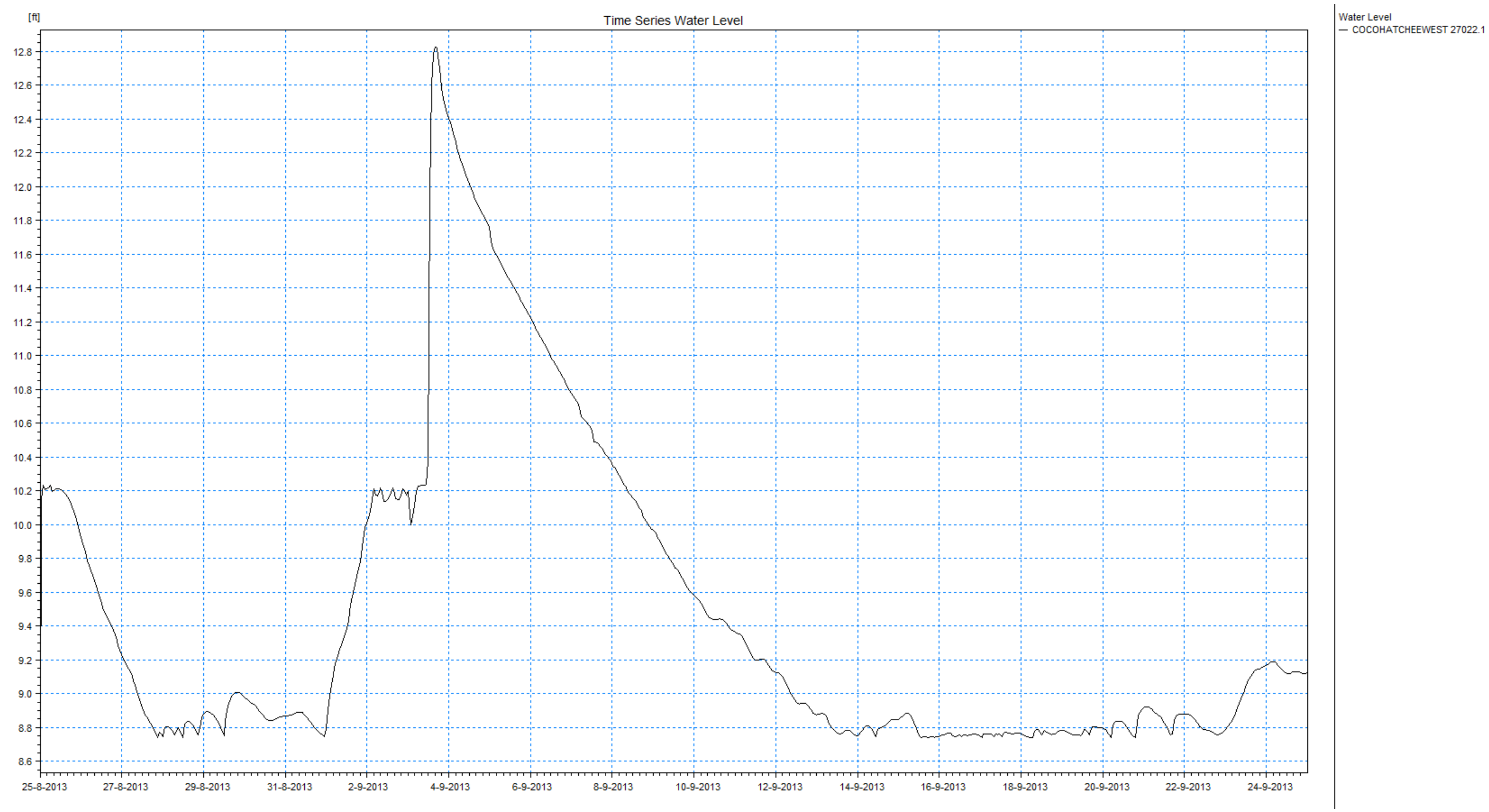


Figure 8. COCO3 Discharge (cfs)



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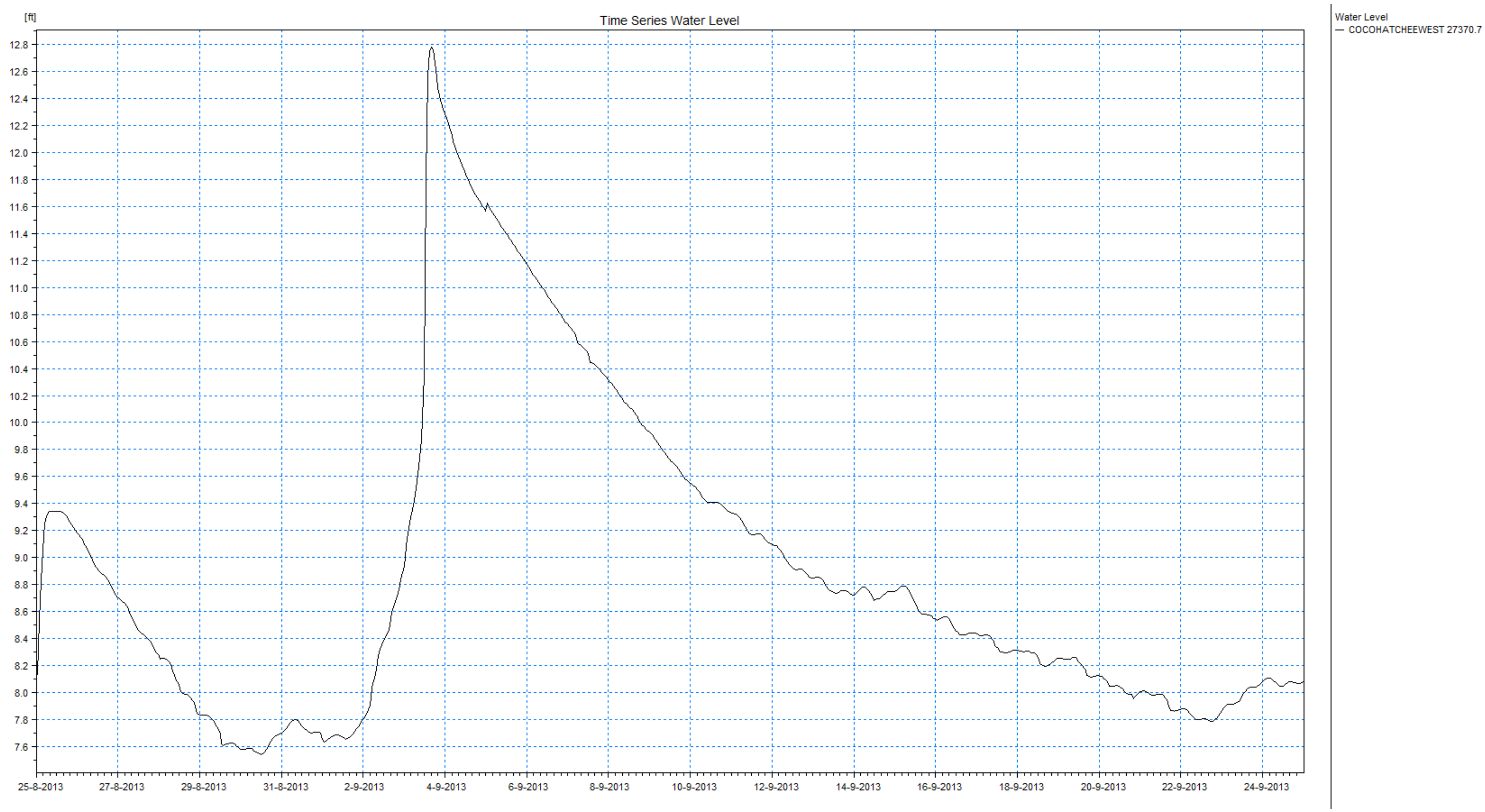

Figure 10. COCO3 Tailwater Stage (ft)

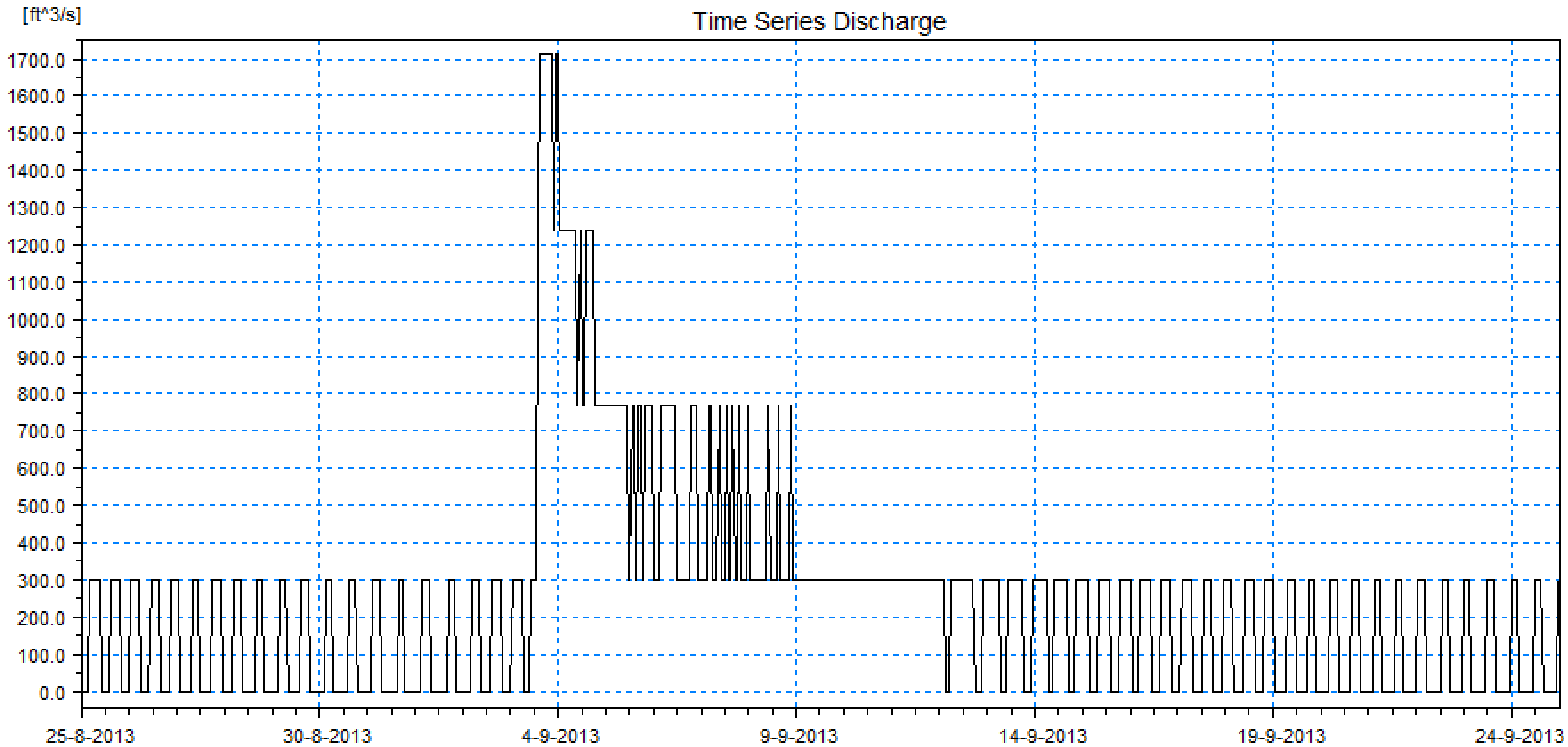

Figure 11. FU Pump Discharge (cfs)

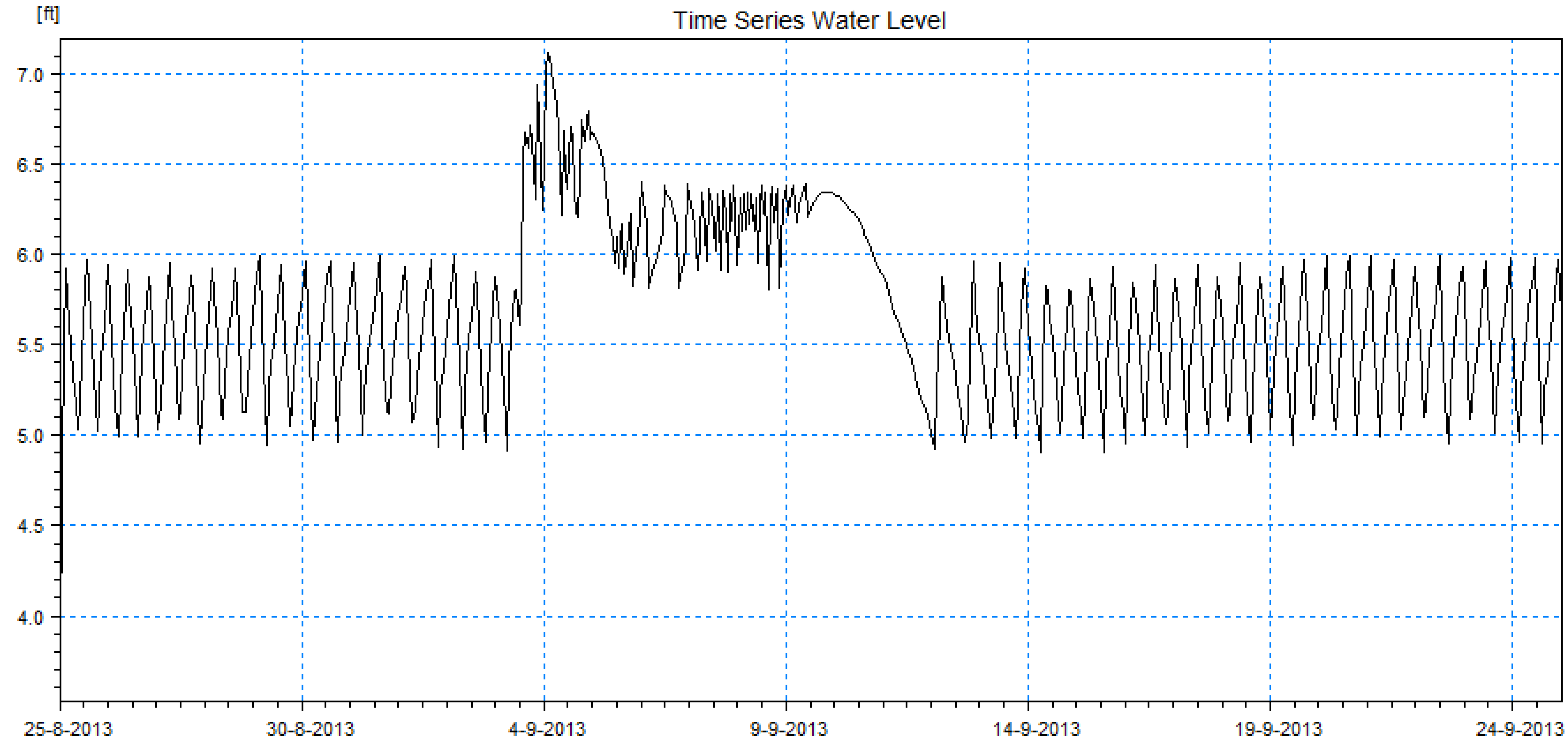

Figure 12. FU Pump Headwater Stage (ft)

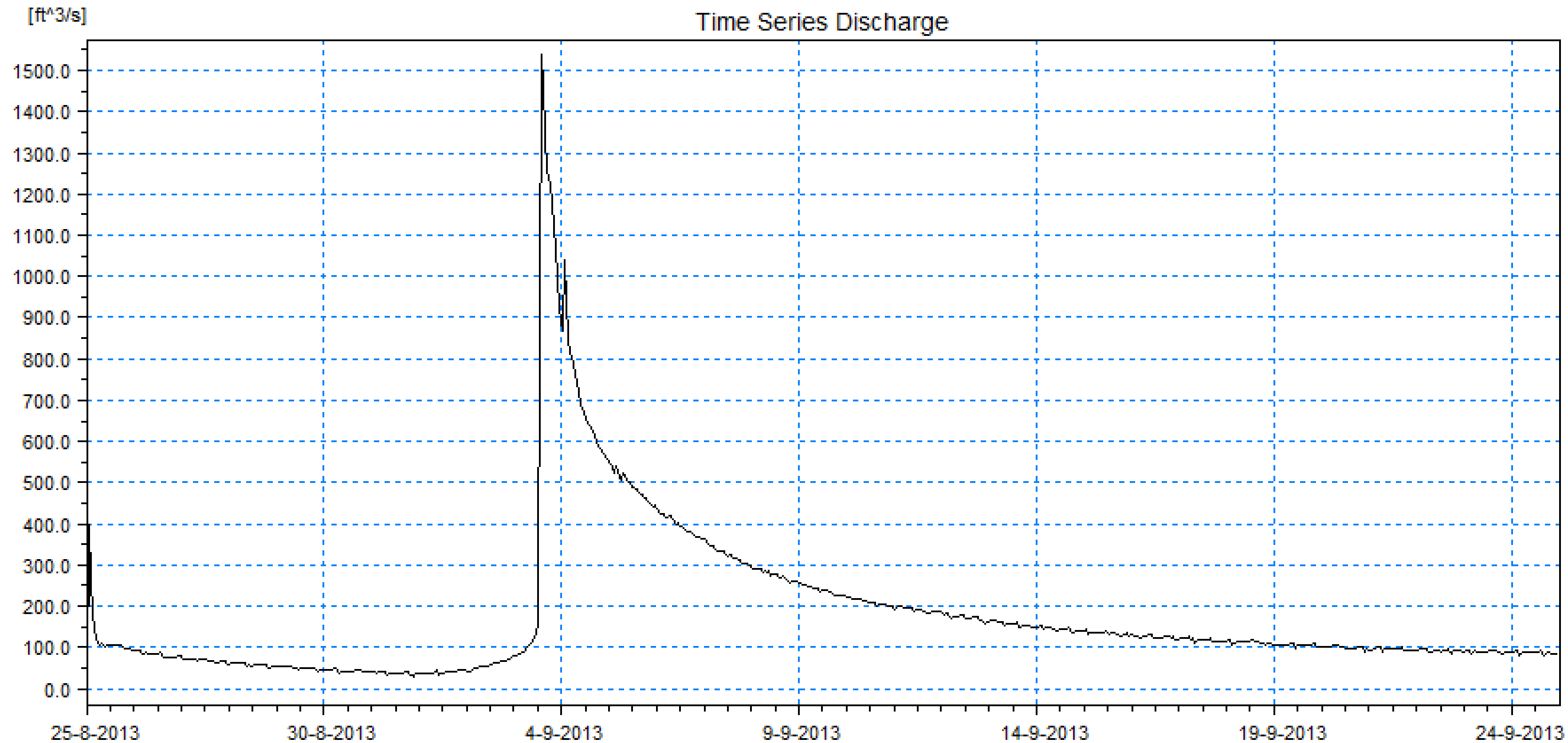

Figure 13. FU4 Discharge (cfs)

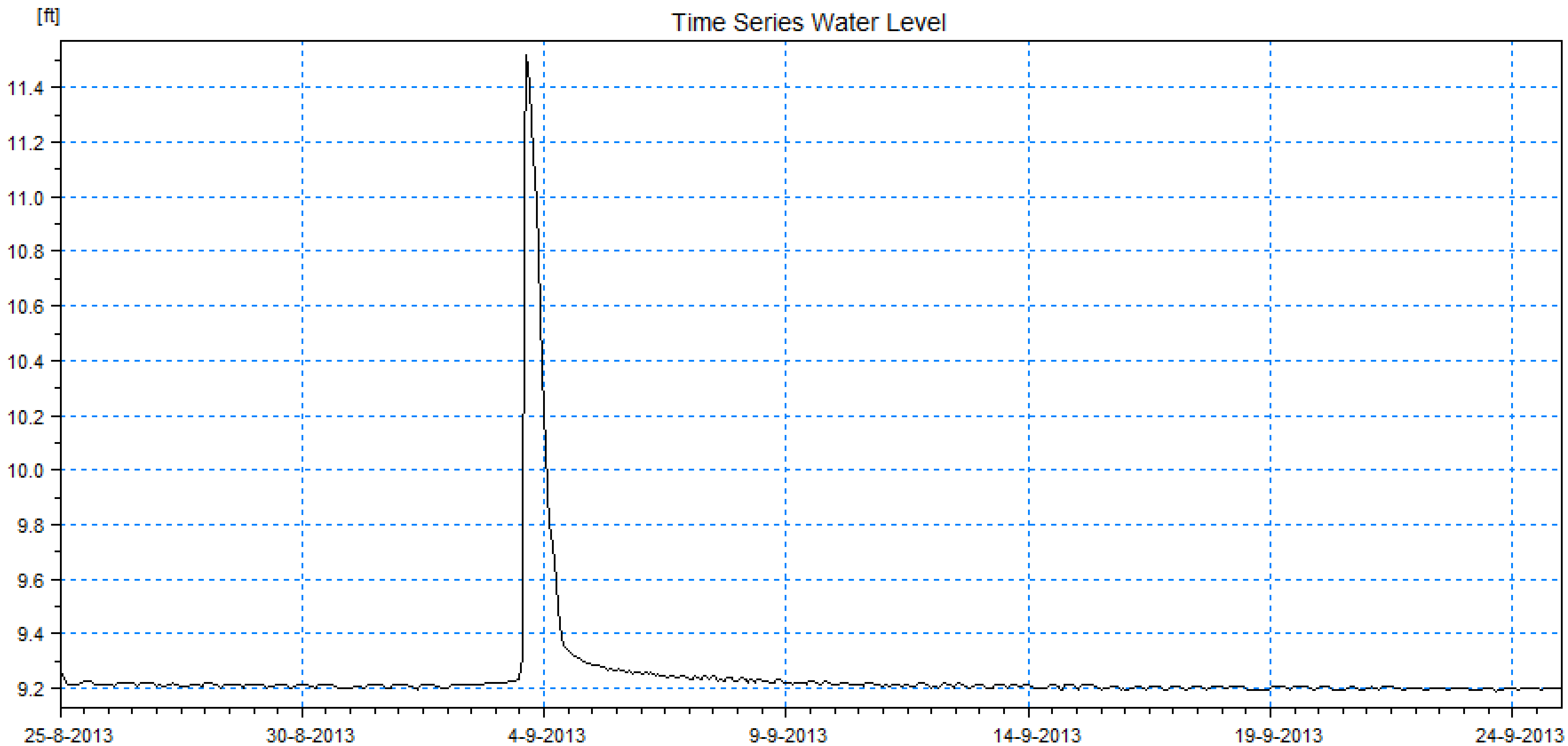

Figure 14. FU4 Headwater Stage (ft)

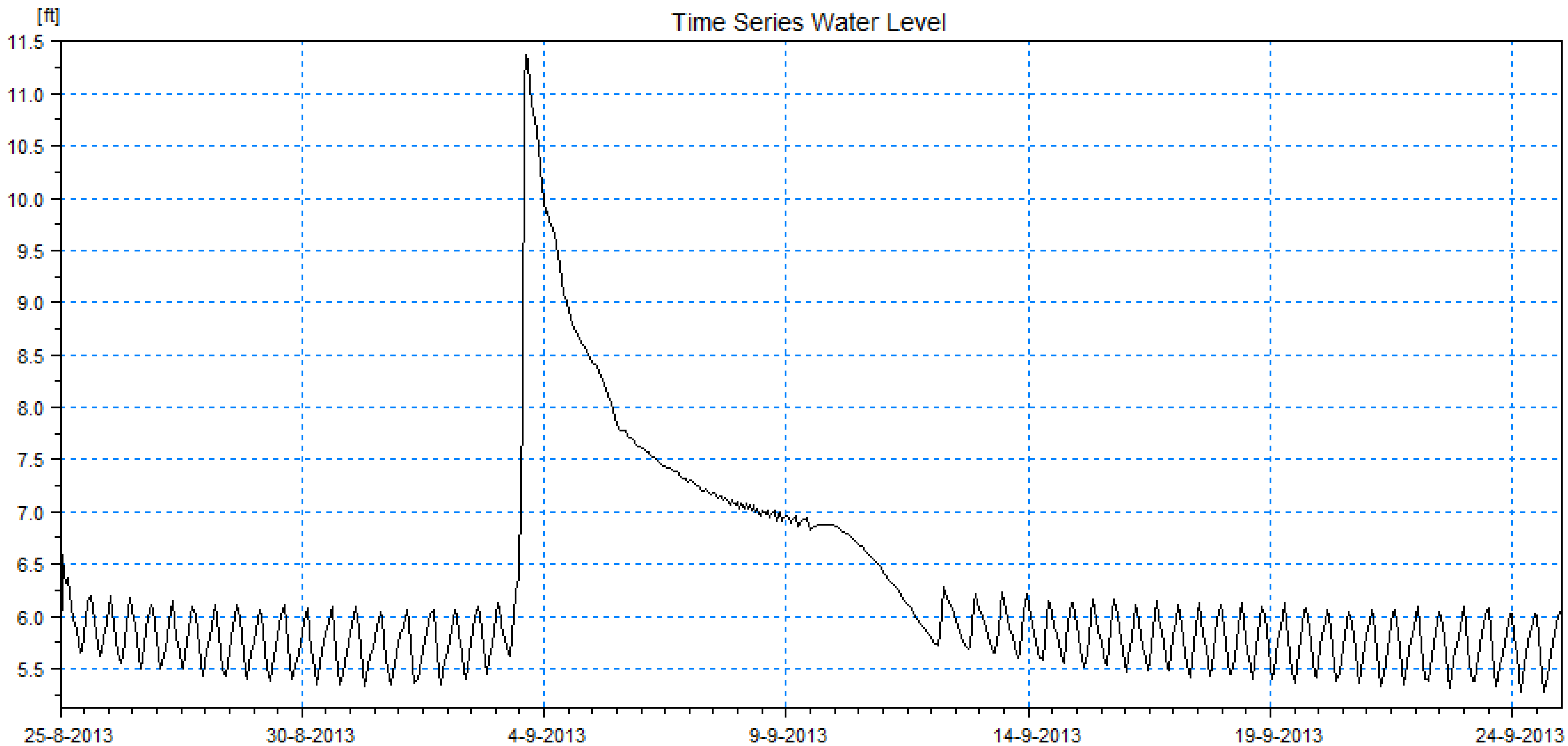

Figure 15. FU4 Tailwater Stage (ft)

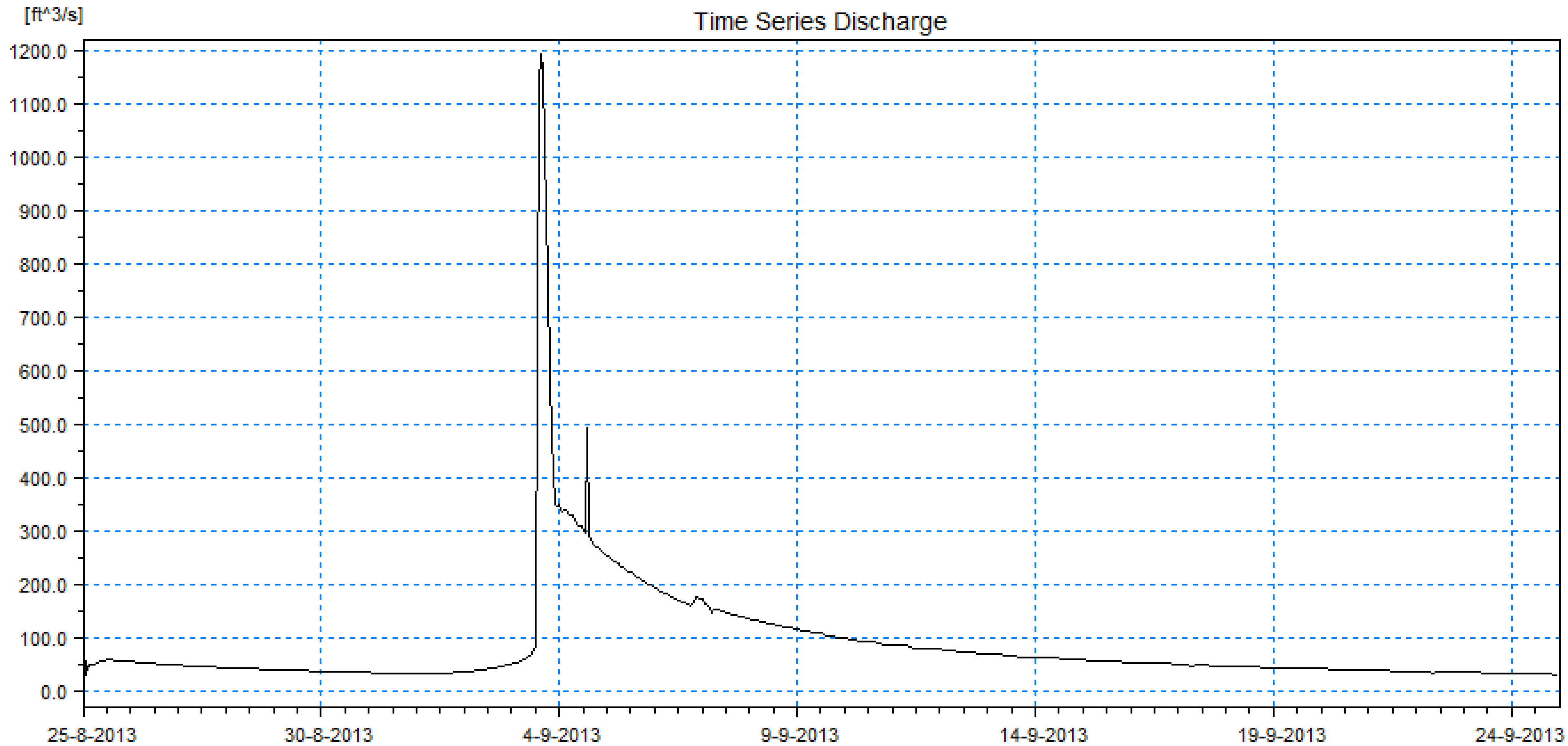

Figure 16. FU5 Discharge (cfs)

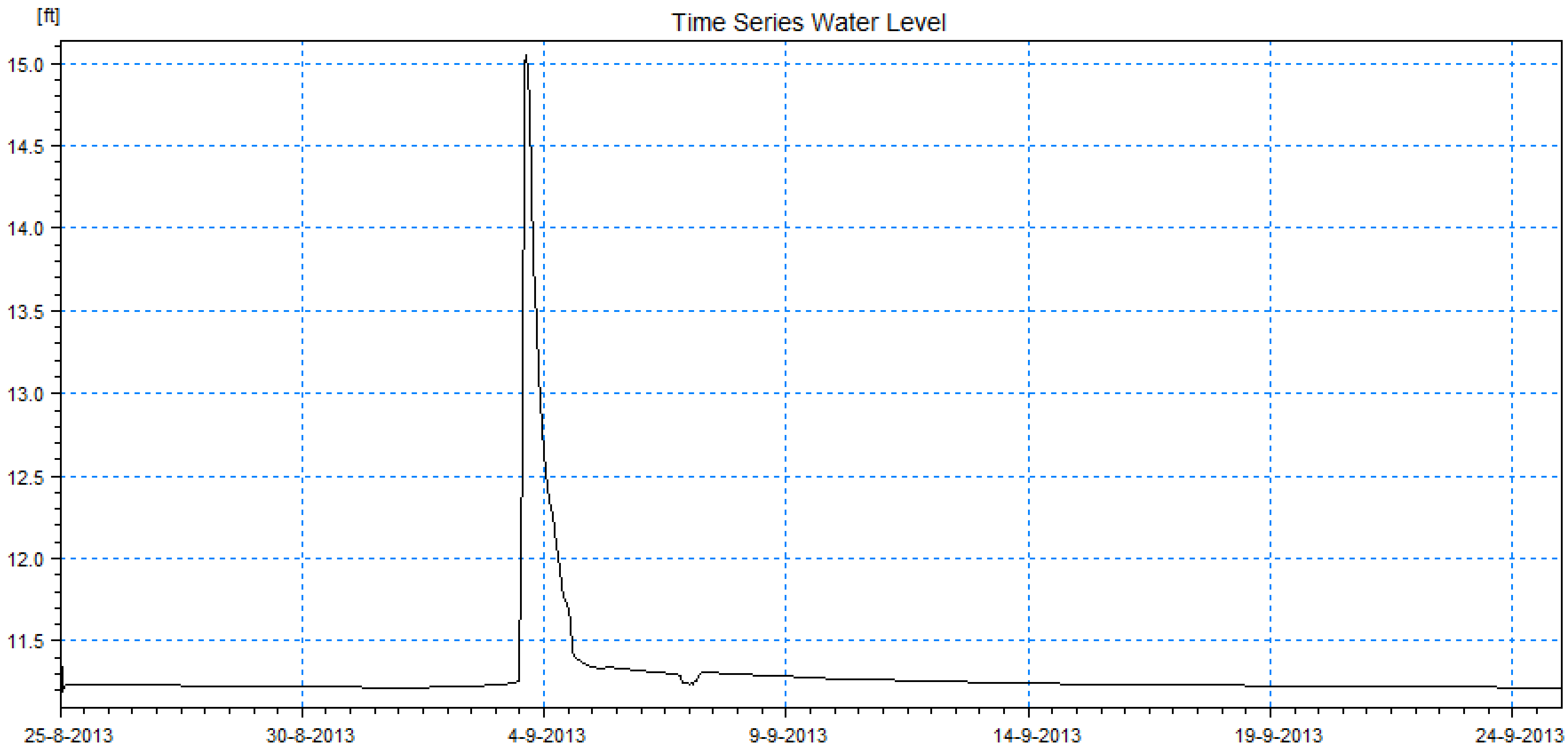

Figure 17. FU5 Headwater Stage (ft)

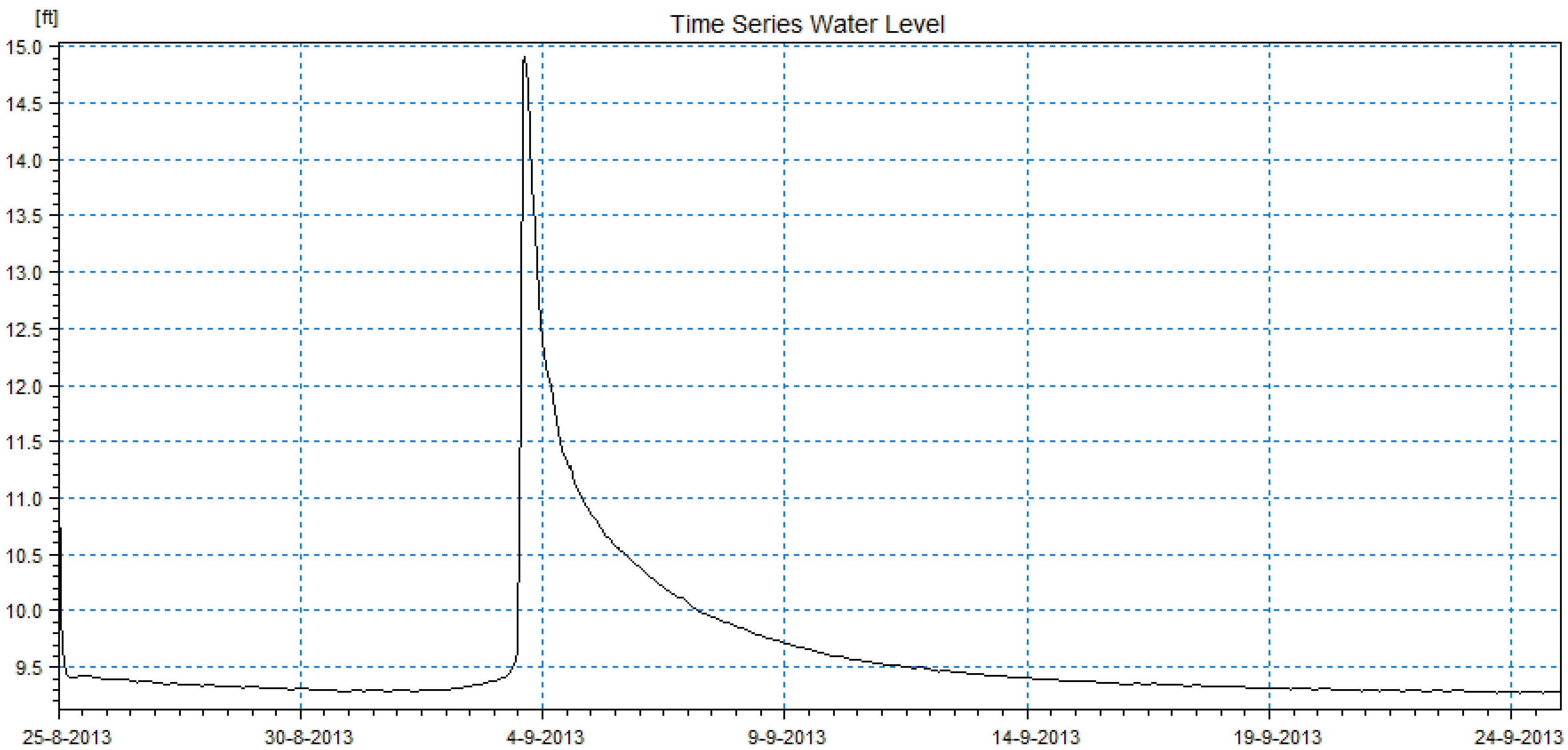

Figure 18. FU5 Tailwater Stage (ft)

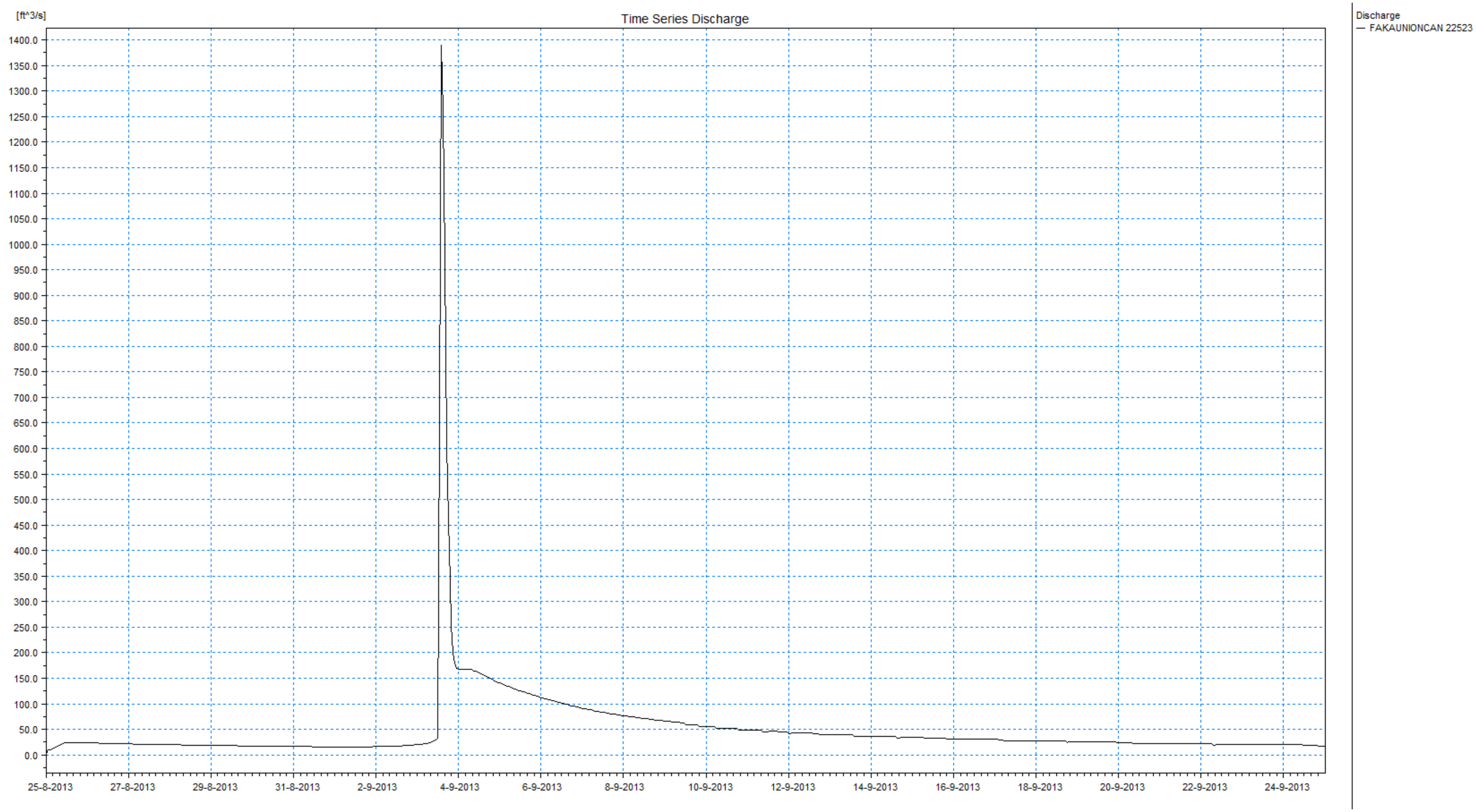

Figure 19. FU6 Discharge (cfs)

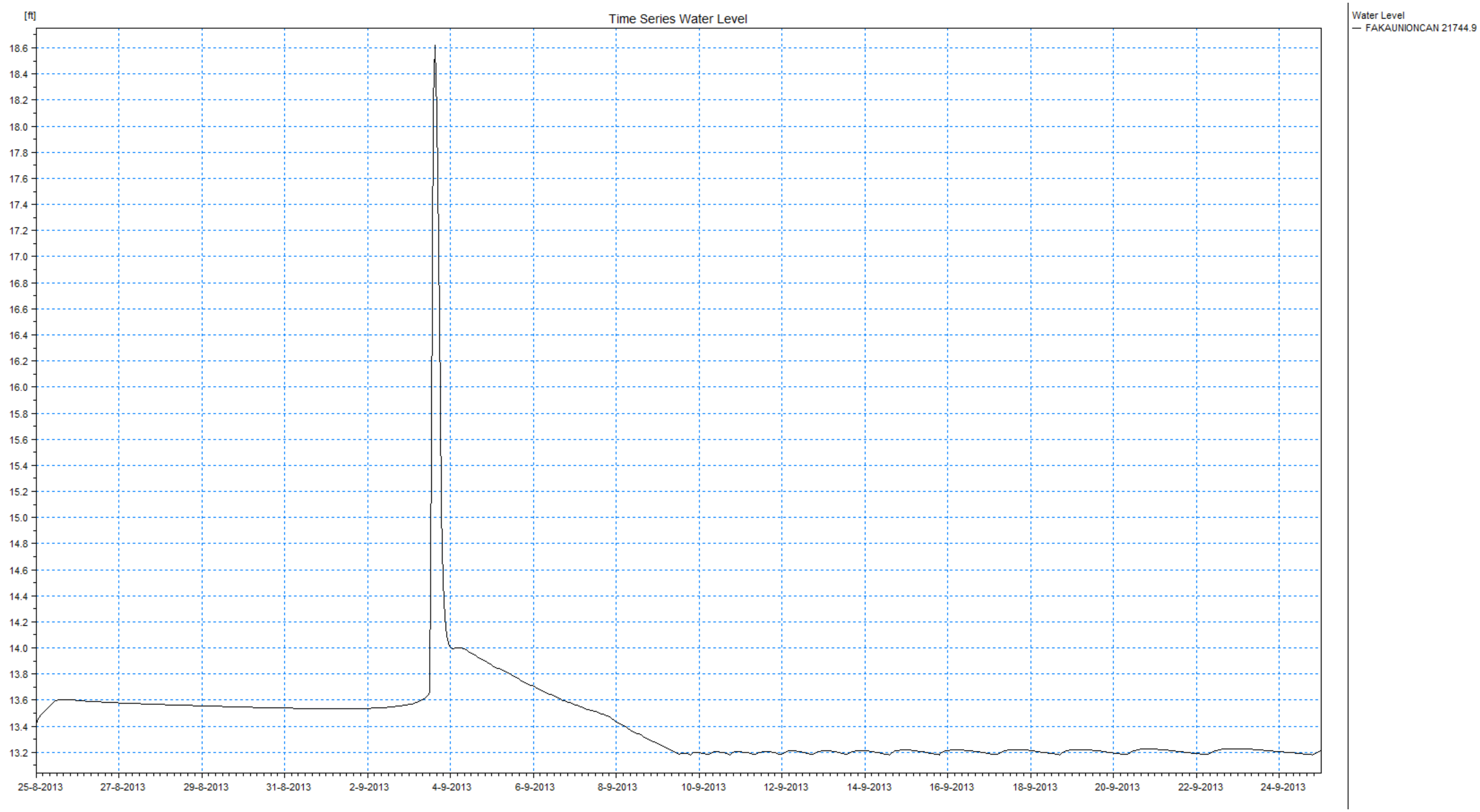

Figure 20. FU6 Headwater Stage (ft)

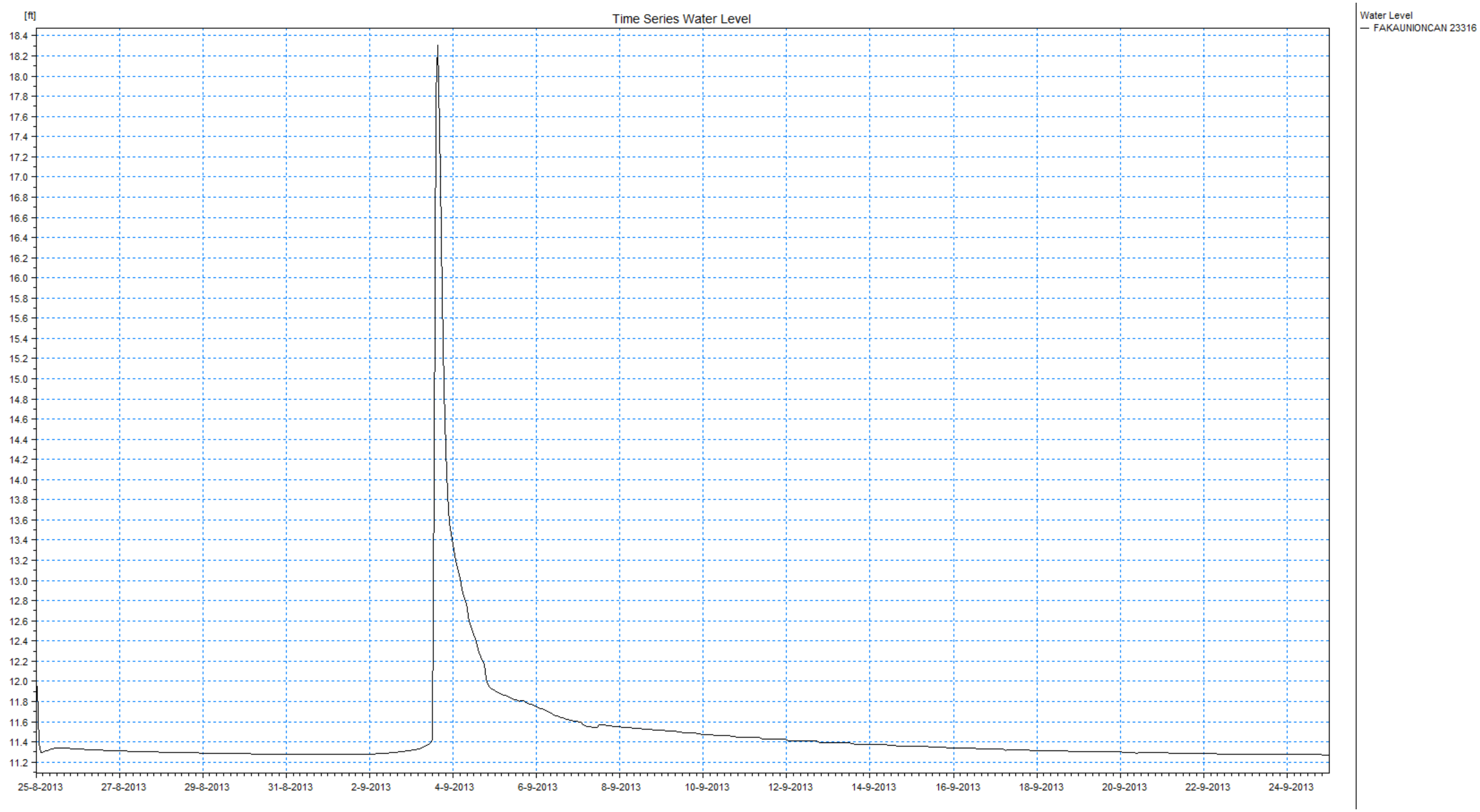

Figure 21. FU6 Tailwater Stage (ft)

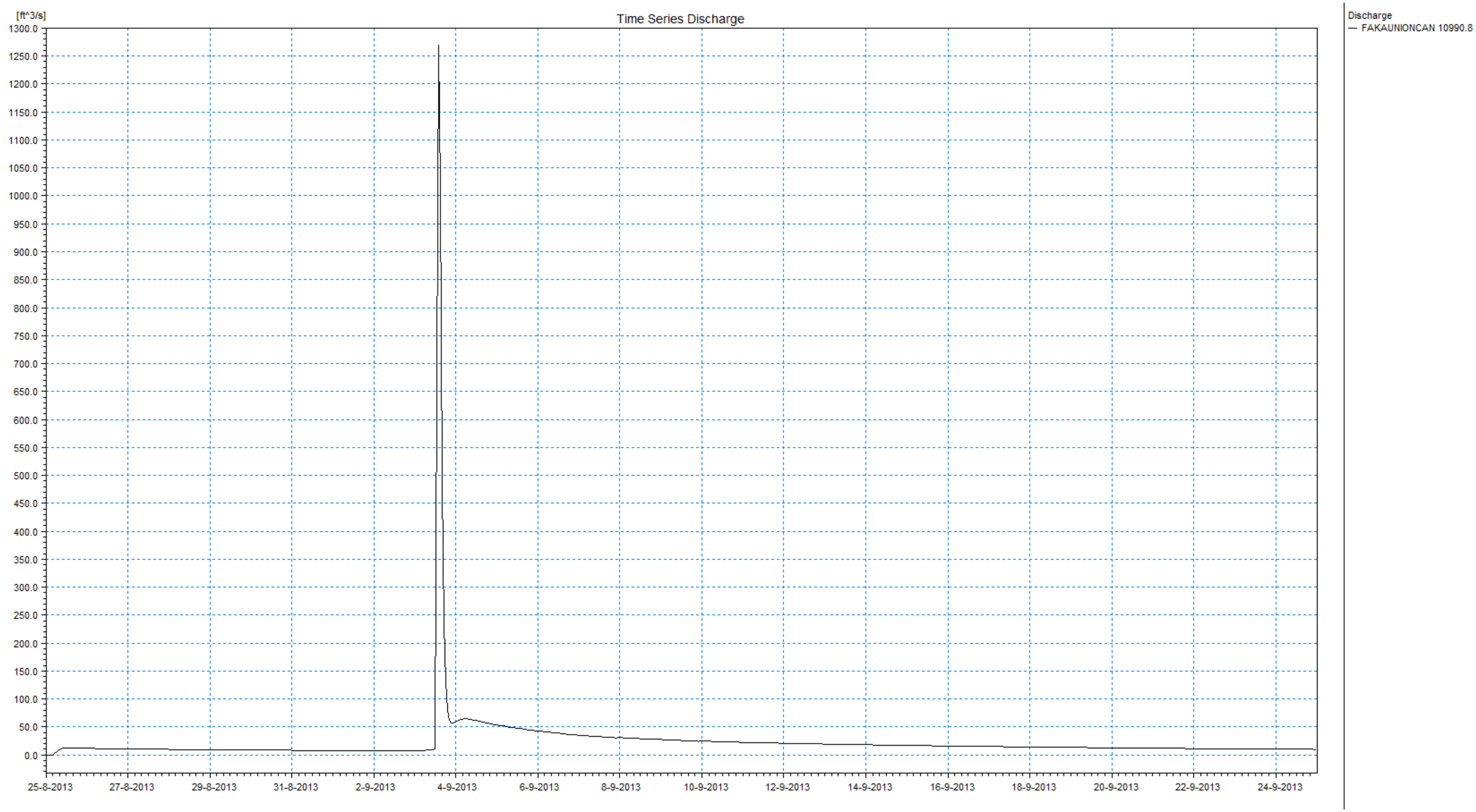

Figure 22. FU7 Discharge (cfs)

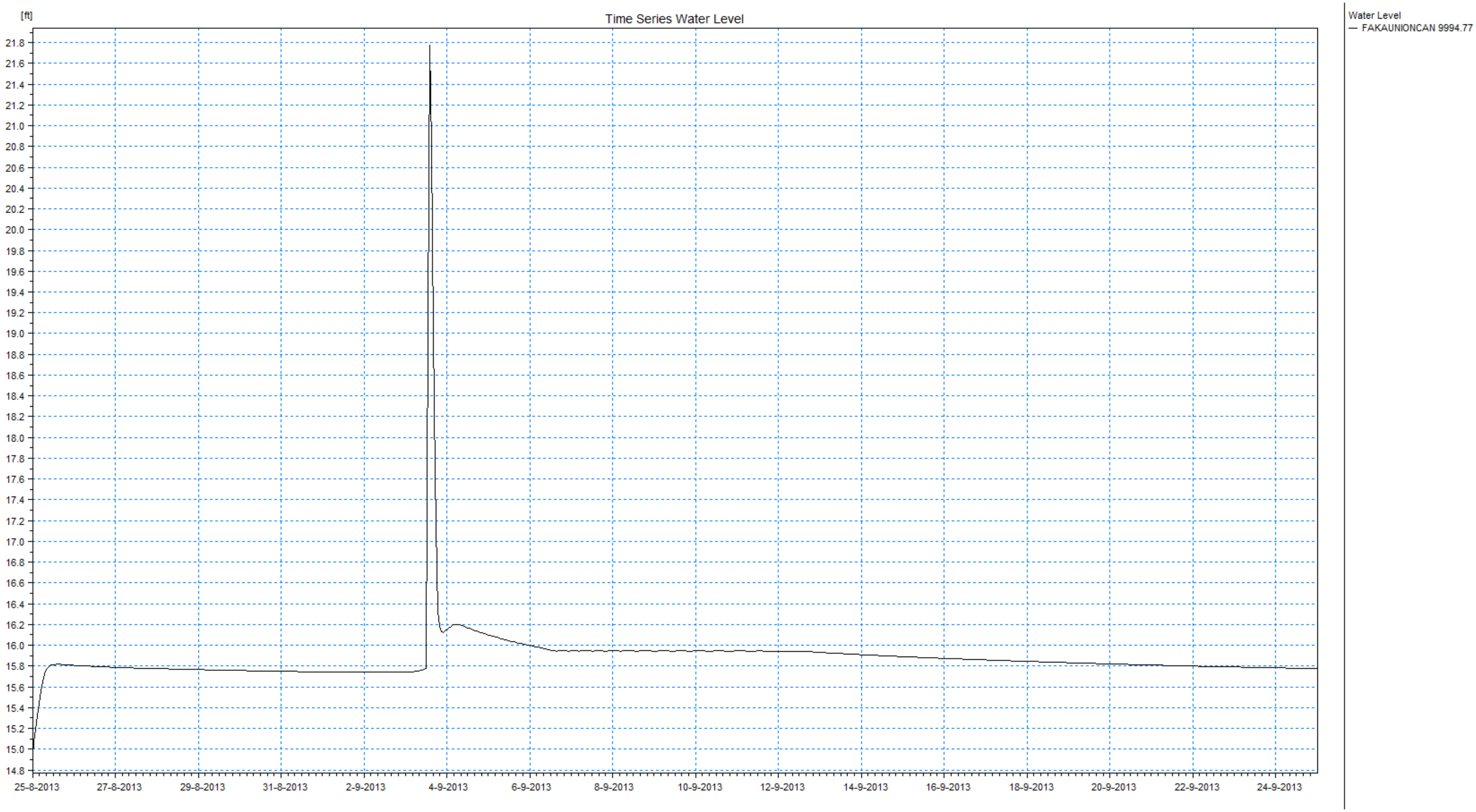

Figure 23. FU7 Headwater Stage (ft)

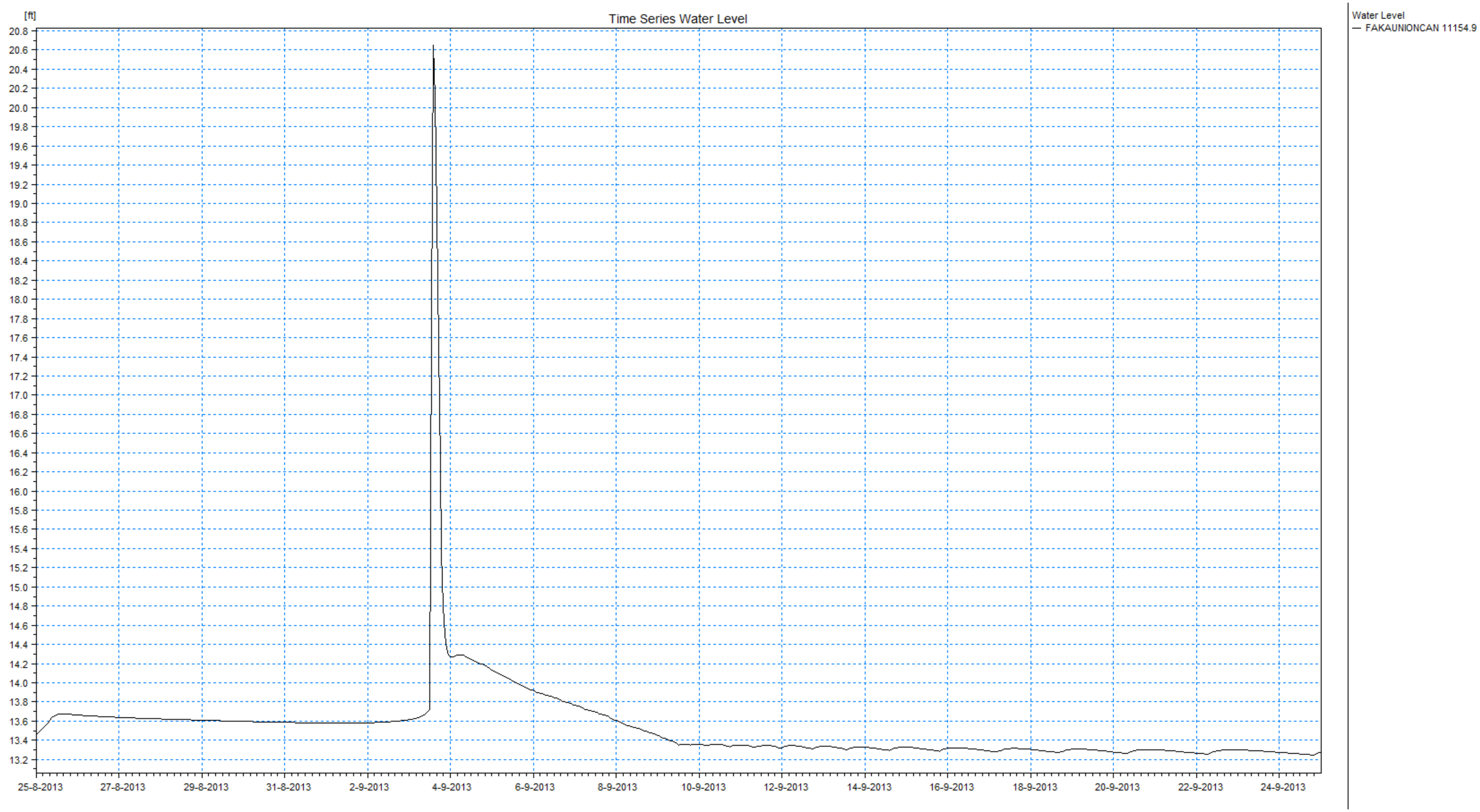

Figure 24. FU7 Tailwater Stage (ft)

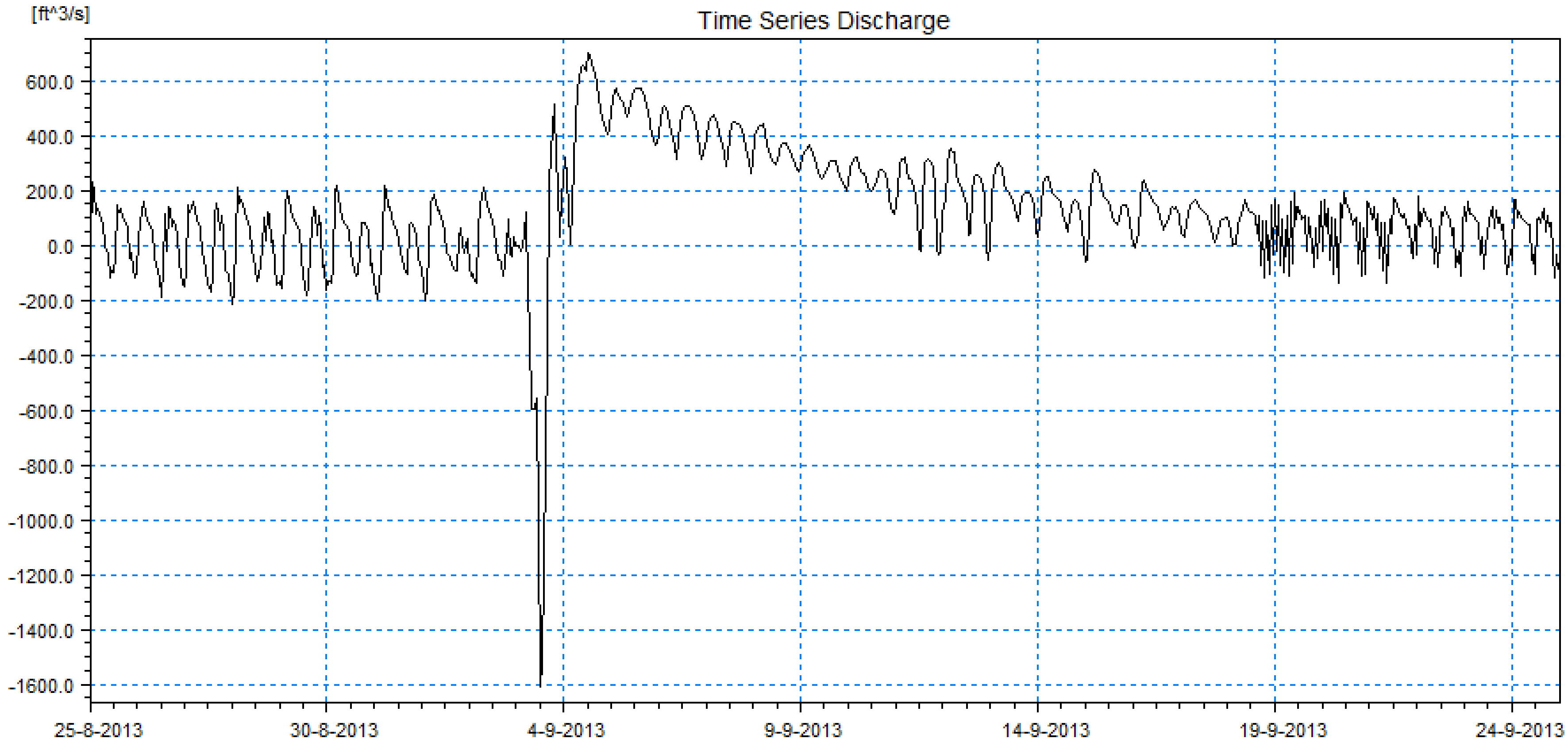

Figure 25. HC1 Discharge (cfs)

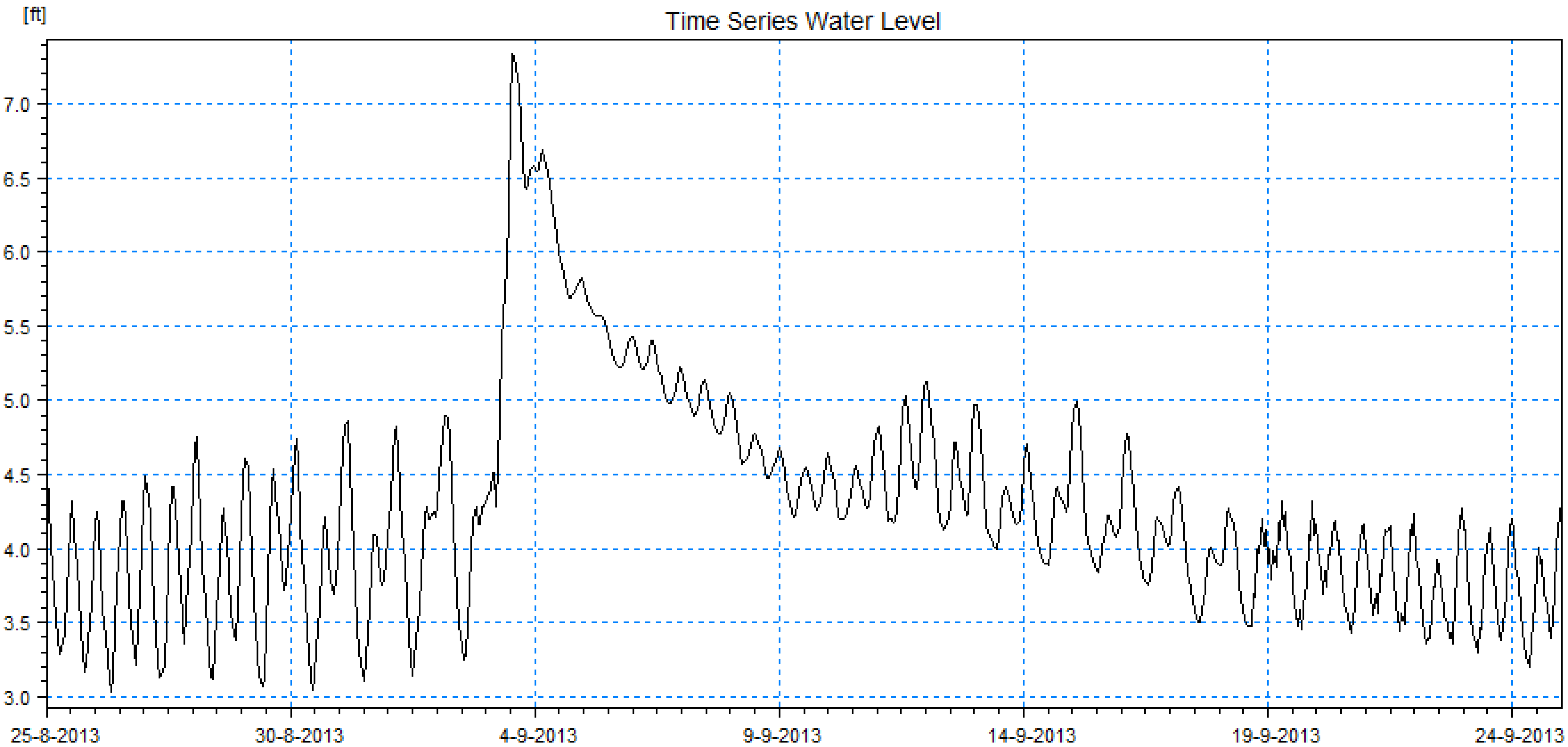

Figure 26. HC1 Headwater Stage (ft)

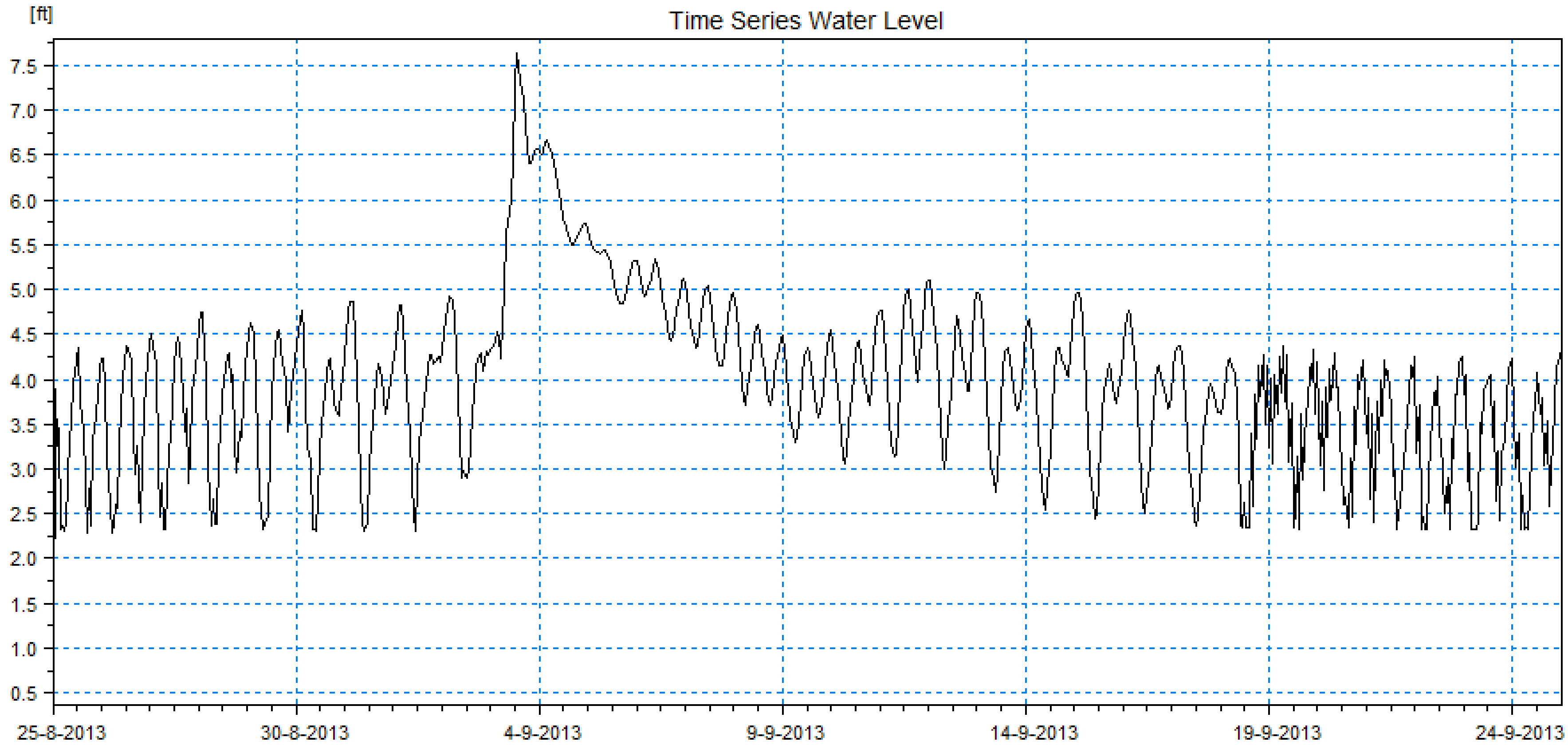

Figure 27. HC1 Tailwater Stage (ft)

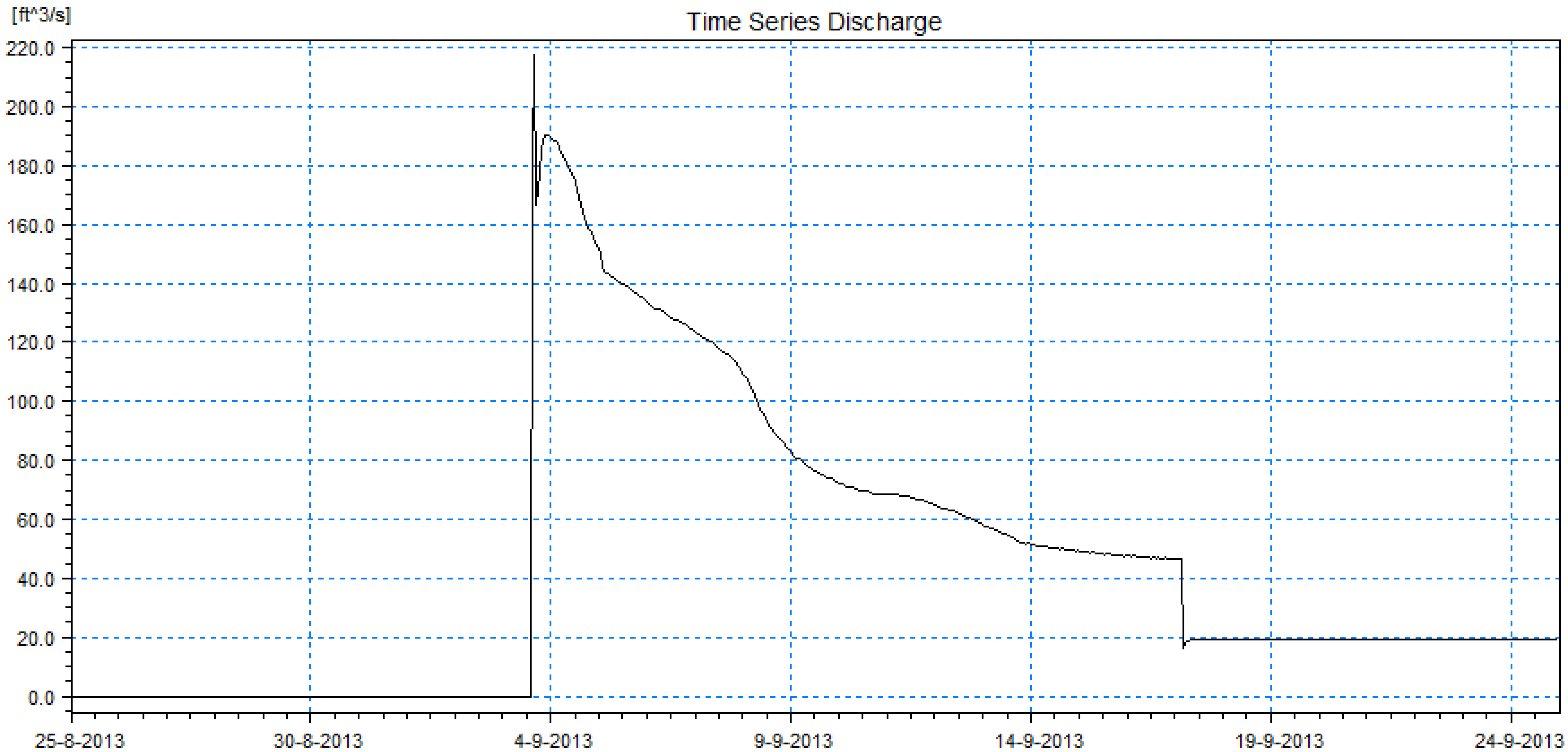

Figure 28. HC2 Discharge (cfs)

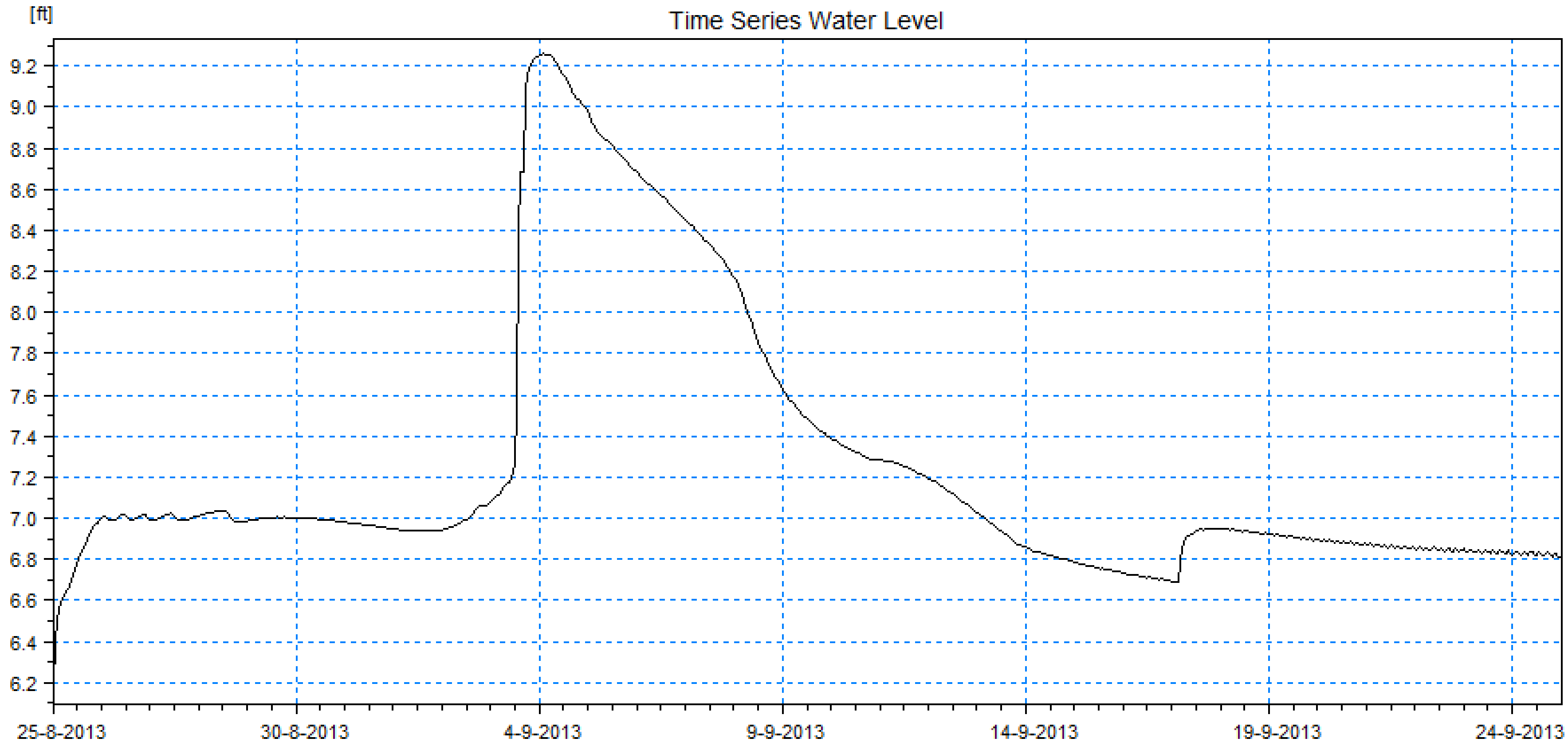

Figure 29. HC2 Headwater Stage (ft)

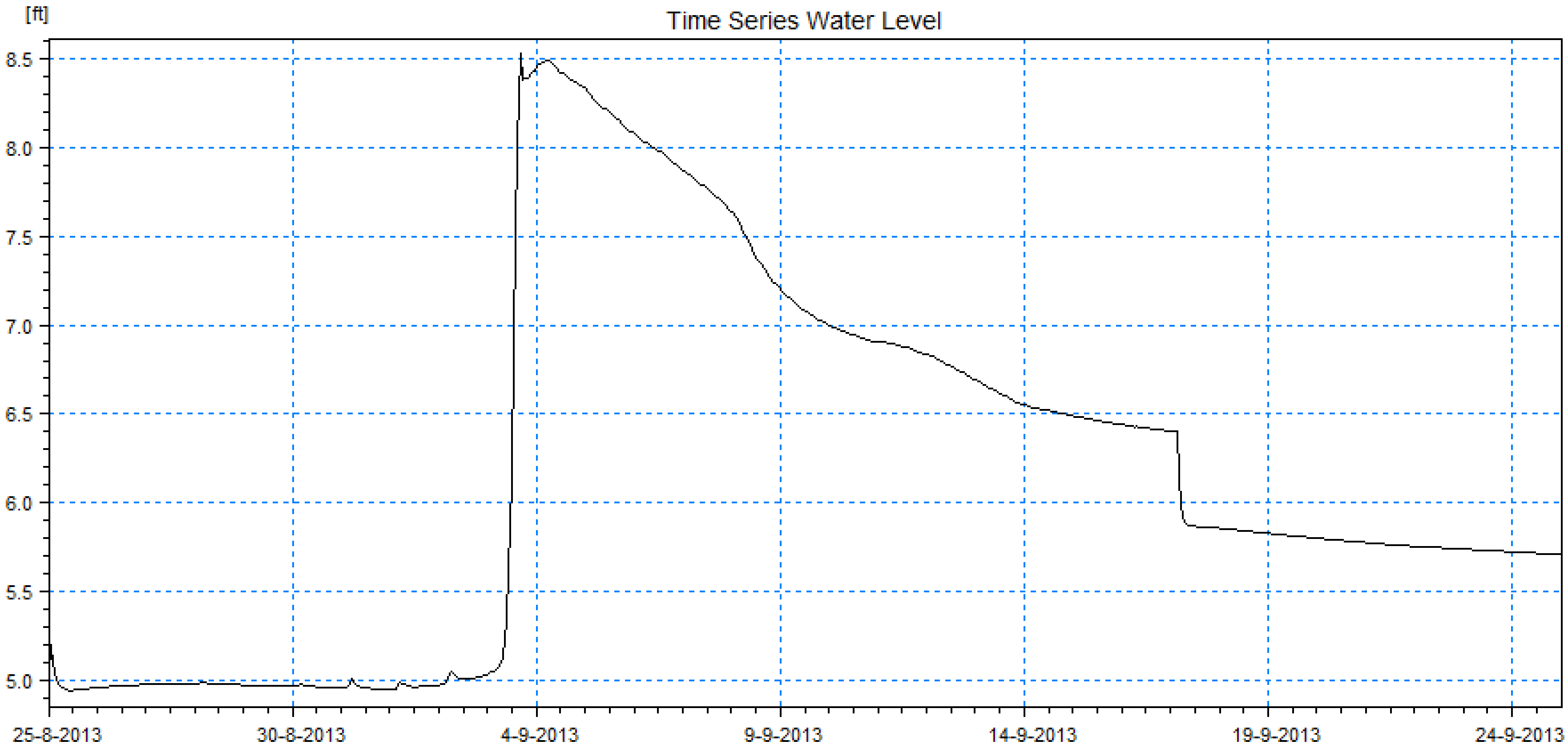

Figure 30. HC2 Tailwater Stage (ft)

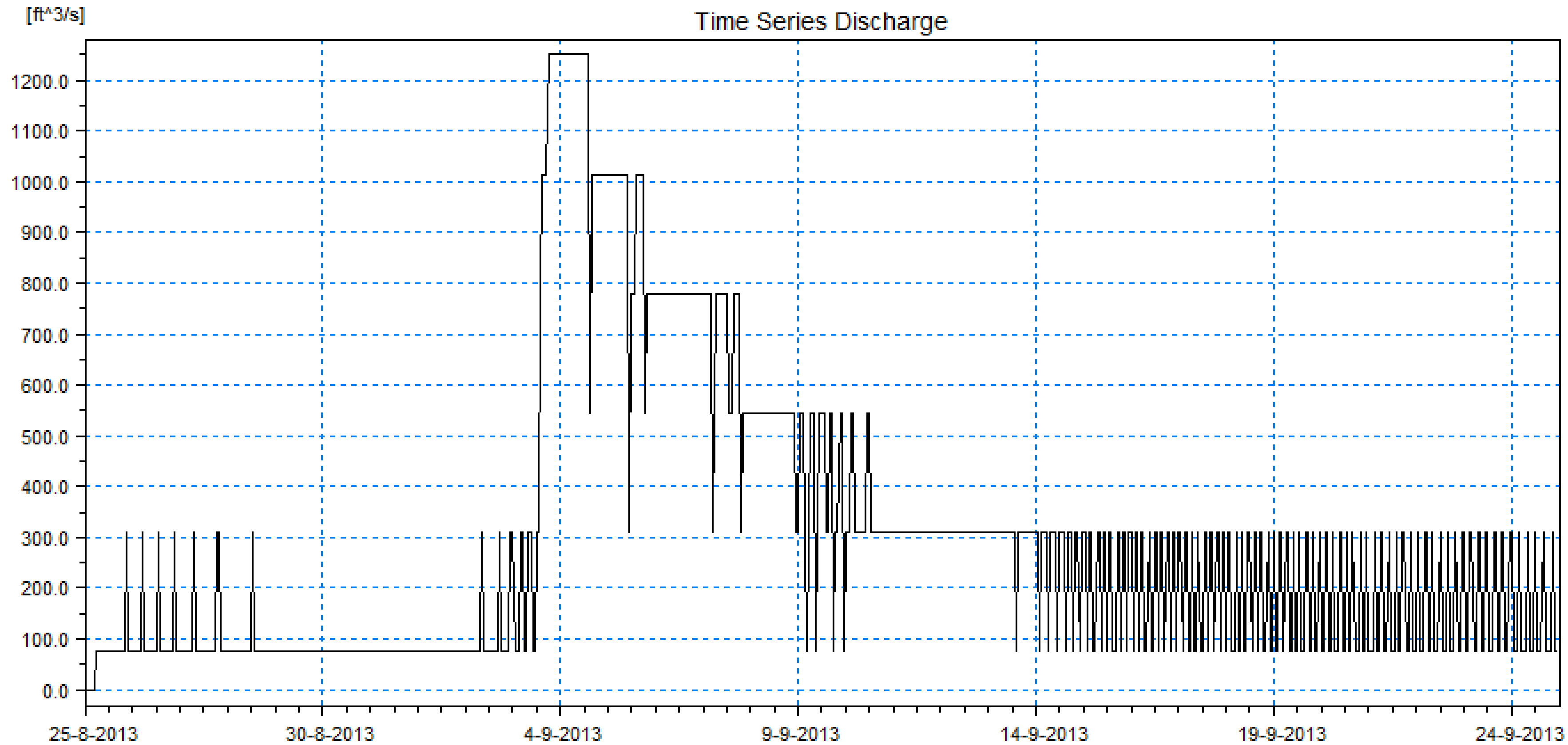

Figure 31. Miller Pump Discharge (cfs)

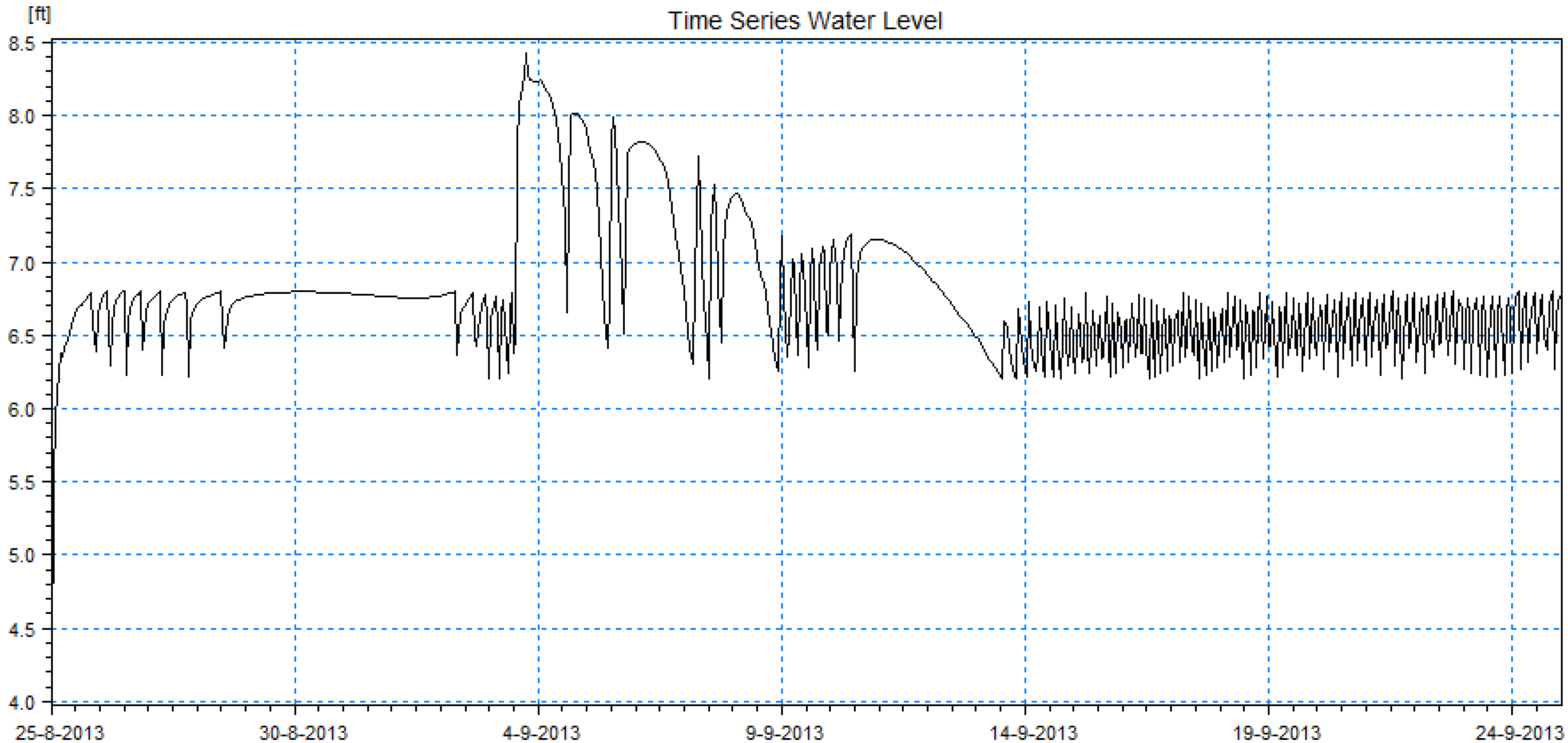

Figure 32. Miller Pump Headwater Stage (ft)

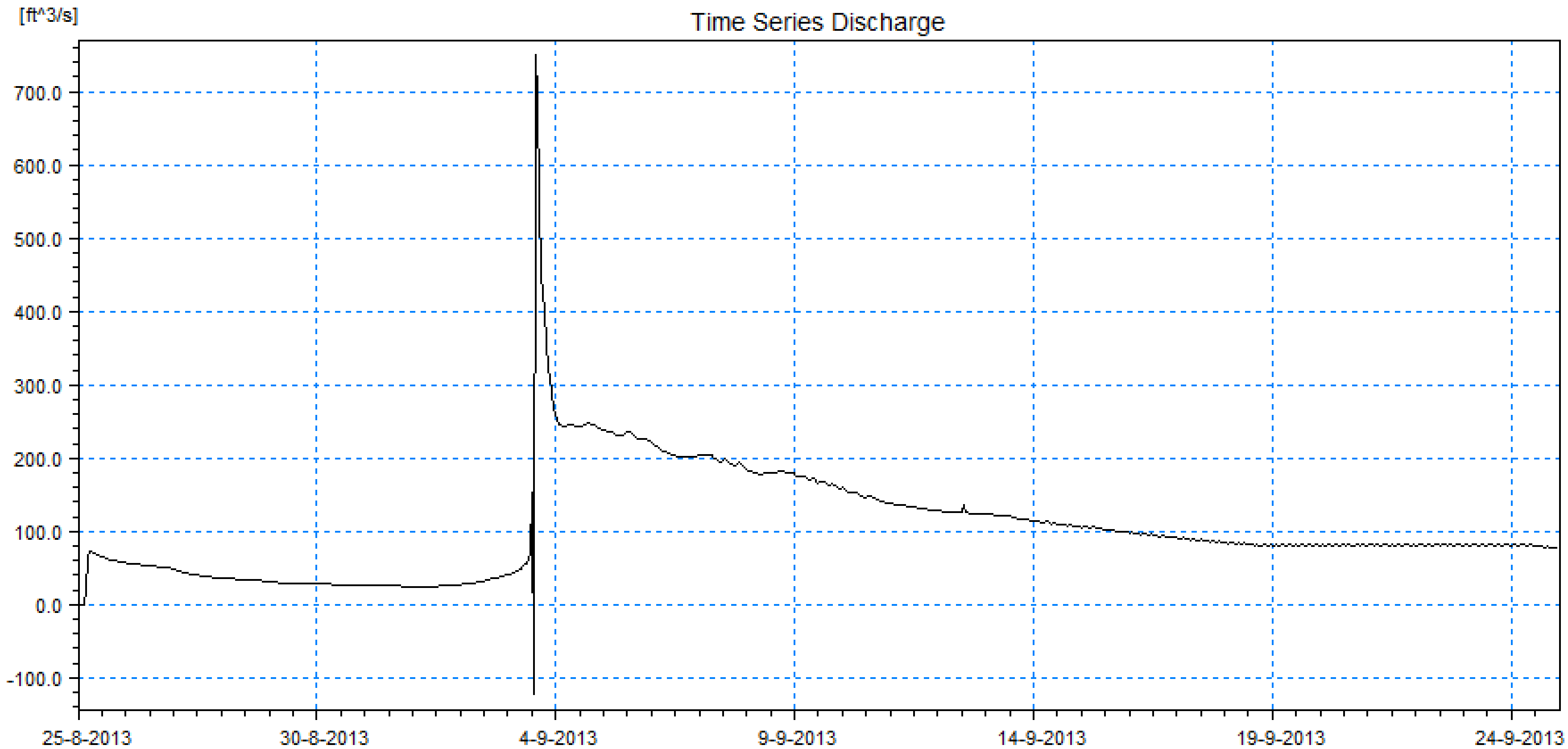

Figure 33. Miller3 Discharge (cfs)

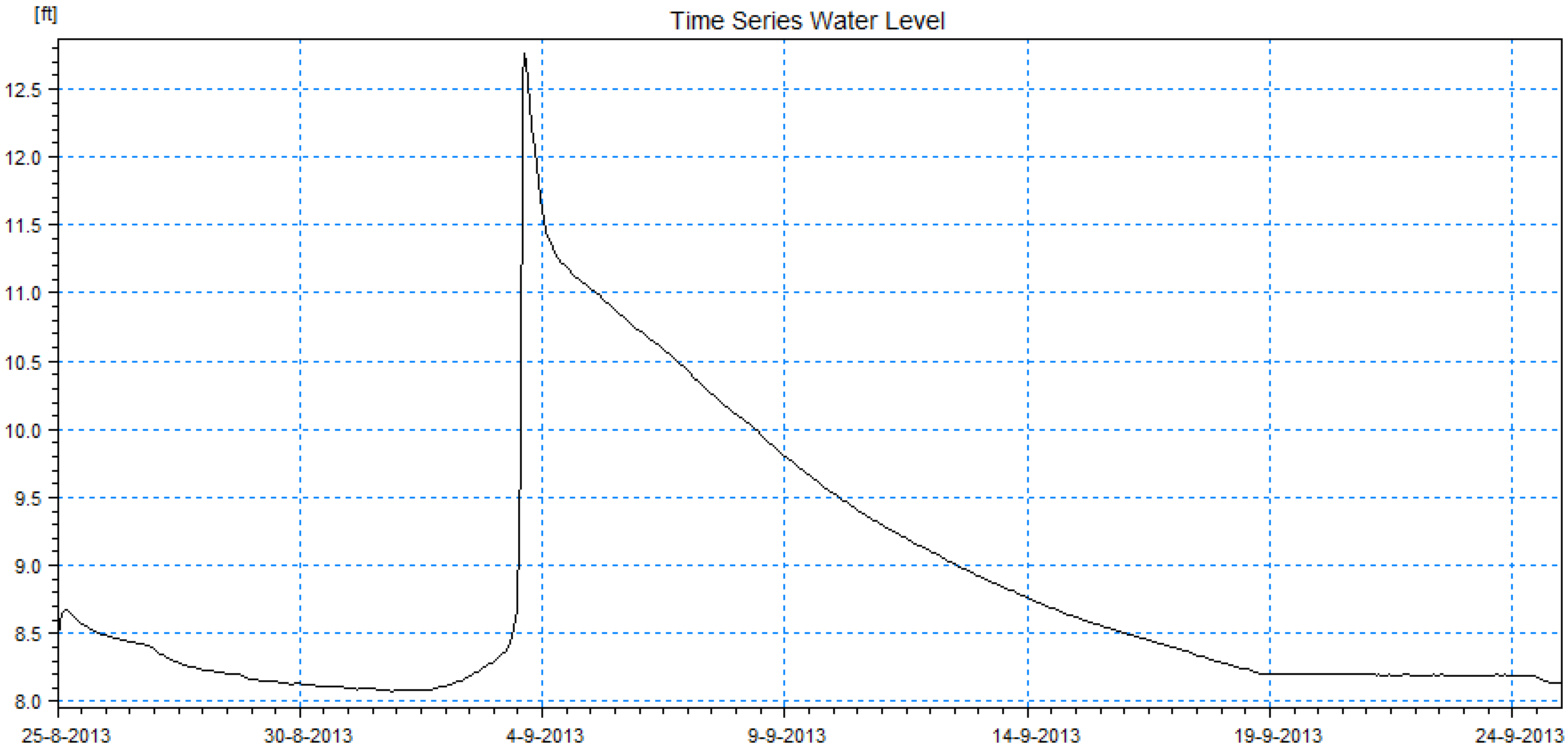

Figure 34. Miller3 Headwater Stage (ft)

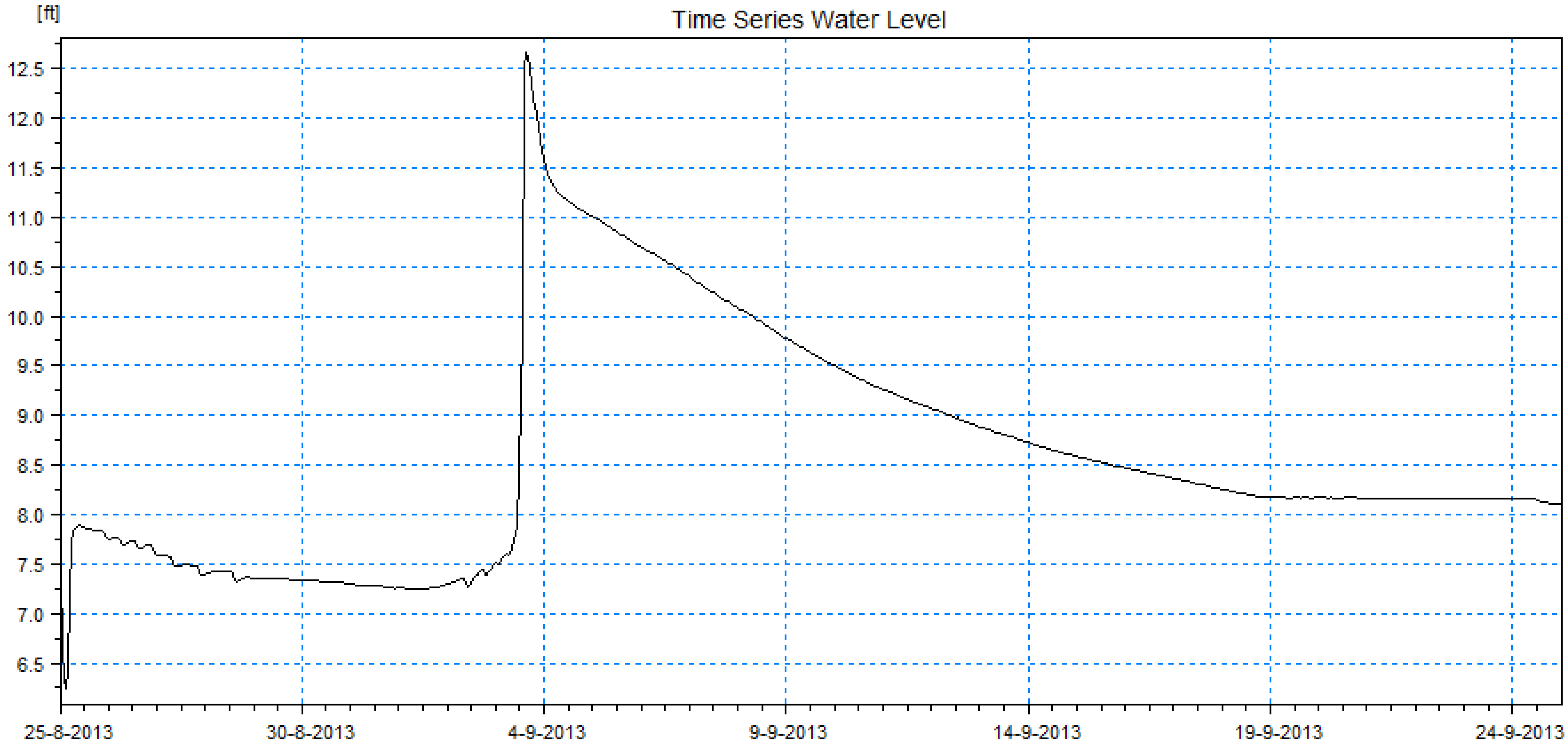

Figure 35. Miller3 Tailwater Stage (ft)