WATERTOWN CITY COUNCIL
WORK SESSION AGENDA
CITY HALL
23 SECOND STREET NORTHEAST
WATERTOWN, SOUTH DAKOTA

Monday April 18th, 2016 **5:30 PM**

1. Call to Order
2. Information on the 4th Annual Clean and Green Spring Litter Blitz
3. Update on the Stony Point Development Supporting Documentation
4. Future agenda items
5. Adjournment

Rochelle M. Ebbers, CPA
Finance Officer

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Back To Top

SEH

Building a Better World for All of Us℠

MEMORANDUM

TO: Shane Waterman, City Engineer - Watertown

FROM: Rocky J. Keehn, PE, CFM

DATE: April 8, 2016

RE: Review of Stony Point 3rd Addition H&H
SEH No. 14.00

SEH completed a comparison review of the Developer's hydrology and hydraulic computations and response memorandum completed by RESPEC. The purpose of the review was not to "redesign" the system, but to provide a discussion on whether or not the site meets the requirements of the Watertown Post-Construction Stormwater BMP Manual (BMP Manual) as it relates to the concerns raised in the RESPEC memorandum.

Information provided includes:

- Stony Point 3 Add – Hydraulic Analysis 160304
- Stony Point 3 Add – Preliminary Plat 160304
- Watertown Post Construction BMP Manual
- Stony Point March 2016 – RESPEC memo

The discussion will closely follow the main issues discussed in the RESPEC memorandum. These were:

- Pre-developed Hydrology
- Post-developed Hydrology
- BMP hydraulic routing
- BMP pond bottoms

A summary of the review proceeds a detailed discussion of each issue outlined above.

Summary

1. Agree with RESPEC that the C value for existing conditions should have been 0.30 not 0.45.
2. Did not see any concerns on the post-developed Hydrology and both parties were in agreement.
3. Agree with RESPEC that the hydrograph method used by the Developer does not appear to meet the volume requirements in the BMP Manual for the 2-year and 100-year rainfall events.
4. Agree with RESPEC that the Developer's model shows some abnormalities, but since it could be a function of how the model works, we do not agree it is a significant design issue that would impact the project design or Lake Kampeska.

Back To Top

5. The 2-year and 100-year, 24 hour events are for determination of rainfall amounts and not, as RESPEC implies, to determine the type of model to use, which they indicate should have a 24-hour rainfall distribution.
6. The BMP Manual has no discussions on the relationship between the lake elevations and the bottom elevation of the ponds.
7. Disagree with RESPEC where they imply the 10-year lake flood elevation should be used for the bottom elevation of the ponds since we would assume the rainfall impacts to the development and lake are independent events and the BMP Manual does not provide any guidance.
8. Since there is no direct specific guidance on what the pond bottom elevations should be in relationship to the lake, we recommend that the 2-year event might be the most appropriate base lake elevation since it is referenced in the BMP Manual as a storm event to be used for analysis of downstream impacts not related to the 100-year major flood event. The other event that is not referenced in the BMP Manual that could be appropriate is the Ordinary Highwater Elevation (OHW).
9. The volumetric impact of the project on the lake for the 100-year event is calculated to add 0.00046 feet given there is no discharge flow from the lake. The BMP Manual states: "providing storage for the 100-year storm is meant to reduce the possibility of damaging floods downstream". The increase in volume from the development would not cause damage downstream and SEH concluded that volume control from the development site is not a critical design component.
10. RESPEC did not appear to focus on the water quality design of the project which appears to meet the BMP Manual requirements and since the water quantity impacts are not critical to the design, thus the project should not "affect the ability of each BMP to provide consistent protection of Lake Kampeska's water resources."
11. If the pond bottom elevations are to be based on their relationship with either the Lake outlet weir or Lake OHW elevation, the elevations will need to be checked since it appears the Lake information is based on a datum that is 1 foot lower than the FIS and datum used for the site plan.
12. Water quality BMP's are the most critical design components for this project.

Pre-developed Hydrology

Developer: Used a rational C value of 0.45 for existing conditions

RESPEC: Indicates that the BMP Manual states the maximum value for the Rational Method C value is 0.30 for pre-developed conditions.

Comments: If City ordinances or the BMP Manual does not allow for any variances, the RESPEC analysis is correct in stating the Developer's engineer will need use an existing Rational Method C value of 0.30. This change should only impact the size of the retention/detention facility and with the small drainage areas there should be enough open space to increase the size of the ponds.

Post-developed Hydrology

Back To Top

Developer: Prepared a weighted Rational Method C value for the post-developed hydrology and methodology is accurate.

RESPEC: Agreed with Developer's values for Rational Method C value and values computed using the equations in Section 5.4 in the BMP manual.

Comments: Would agree that the post-developed Rational Method C values and computations are acceptable.

BMP Hydraulic Routing

Developer: Used a Rational Method based hydrograph model which creates a triangular inflow hydrograph based on a peak flow and assumed time of concentration. This hydrograph is then routed through the BMP using standard pond routing methods. This method is not specifically referenced in the BMP Manual as an accepted model so it falls under the category "Others as approved by the City Engineer".

RESPEC: Indicated that the model used by the Developer does not meet the requirements of Section 5.4.1. They then discuss in detail how the NRCS methodology meets the requirements of the BMP Manual and the model used by the Developer does not. Their conclusion is the ponds are undersized and would not meet the requirements outlined in BMP Manual. They also discuss that the output hydrographs have abnormal jumps that need to be explained further.

Comments: RESPEC makes the point that the ponds, based on Section 5.4.1 need to be designed for a 2- and 100- year 24-hour design flow and then goes into a detailed discussion which implies that NRCS methods must be used to meet this requirement. However, it appears this section reference is for the rainfall amounts. A further review of Section 5.4.4 supports this assumption since the equations use the 24-hour rainfall amounts in conjunction with the Rational Method.

The Developer's method of using a Rational Method hydrograph would be valid if it reproduces the volume of runoff and peak flows that could be expected from a 24-hour rainfall. Section 5.4.4 in the BMP Manual does show how the rainfall can be converted using the Rational Method to acceptable runoff volumes and pre-developed flow rates to determine the size of the ponds for a 2-year and 100-year event. What is not described in the BMP Manual is how you use the equations in Section 5.4.4 to actually determine the size of the retention/detention basin. Both the Developer and RESPEC assumed that a model was required to size the detention basins.

The model used by the Developer does not appear to accurately convert the Rational Method values in section 5.4.4 to 24-hour rainfall storms. RESPEC also reached this conclusion when they compared the runoff volumes calculated by the equations in Section 5.4.4 to the output from the models used by the Developer. I agree with RESPEC that the model used by the Developer greatly underestimates the required storage volumes required in the BMP Manual. If a Rational Method based hydrograph method is approved by the City Engineer, then the volume of runoff should be near the equations in Section 5.4.4. One option would be for the Developer to redo the model. The BMP Manual does not require a specific model, so this method used by the Developer could be an acceptable approach if the City Engineer approves it. Whatever model is used, it must use or reproduce a 24-hour rainfall.

Back To Top

RESPEC also mentioned that the output hydrographs from the Developer's model had abnormal jumps. Depending on the input hydrographs, volume of runoff and outlet configuration it is not unusual for models to show abnormalities. Since the routing method used was the storage-indication method, the time between points on the hydrograph is a critical component of the model and thus many models will allow for finer time increments if the hydrographs do not appear to look correct. Our experience is that if these abnormalities occur on rising and falling limbs, the peak flow results normally don't change even with a finer time increment. Since the model that was used may need to be redone, this abnormality may fix itself.

To aid in our review, SEH determined the pond volumes using the equations in 5.4.4 in the BPM Manual. This will provide us with a better understanding of how this hand calculation method compares to the hydrograph methods. The designer could use the equation in 5.4.4 to determine the required volumes for the 2-year and 100-year. That volume could then be incorporated into the pond design and the depth could be determined based on the grading. Once this depth is known, the outlet can be sized to release the pre-developed flow rate that was calculated using standard orifice, culvert or weir computation methods to limit the flow to the peak flow rate that was determined by the equations in the BMP Manual for the 2-year and 100-year pre-developed flows.

The BMP Manual does indicate this is a conservative method (equations in 5.4.4) and we would agree since essentially the pond is filled as if there is a gate and then the gate is opened when the pond reaches the design runoff volume. In the real world, the pond will discharge volume during the rainfall event, so if a hydrograph method is used, this can be part of the design and thus reduce the size of the pond. In our opinion, once the Developer uses a hydrograph method, section 5.4.4 is not used in the same way as described in the BMP Manual.

BMP Pond Bottoms

Developer: Assumes the basis for their ponds should be at the lake outlet weir elevation and/or the OHW elevation obtained from the City and/or DNR.

RESPEC: Assumes the FIS information should be used to determine the bottom of the BMP ponds. The event they focused on was the 10-year lake elevation.

Comments: The key discussion point not addressed by either party is whether or not the BMP Manual or some other source of information addresses what the assumed lake elevation should be during the design event for the development.

The closest section to providing any guidance in the BMP Manual would be Section 2.1, Item 7 which states:

> The planning and design of drainage systems shall be such that problems are not transferred from one location to another. Outfall points shall be designed in such a manner that they will not create flooding hazards.

and 5.1 Step 3 may also apply in this location which states:

Back To Top

> The use of onsite detention is required at those locations where storage for the 2- and 100-year storms is not provided by a regional facility.

As part of our review, SEH needed to determine if the lake and pond were acting independent of each other. Since the drainage area to the lake is 20,433 acres and the development area is already part of this acreage, the development site is already contributing some runoff to the lake. It is also known the lake area is about 5,250 acres. With this large lake area and the large contributing watershed, the lake's 2-year, 10-year and 100-year peak more than likely will not occur at the same time as the proposed development pond 2-year, 10-year and 100-year.

RESPEC includes a discussion on the relationship between the pond's lowest elevation and the 10-year flood elevation of the Lake. There is no specific guidance in the BMP Manual on what should be the pond bottom elevations as it relates to various Lake Elevations. To determine the appropriated lowest elevation, without any specific guidance, an assumption on the flood timing between the Lake and ponds will need to be determined. In our opinion, the likely scenario is that any rainfall event that would cause flooding around the Lake would first cause the ponds in the development area to peak before the Lake. The Lake will then rise and at a much later time will peak, well after the ponds high-water has receded. Flood elevations of the Lake are also linked to flood elevations in the Big Sioux River. The Big Sioux River has no direct relationship to the rainfall events that would occur on the development. The development appears to have two potential flood events. The direct flow on the site and backwater or high-water elevations in the Lake. When there is a possibility of two different frequency flood events occurring at the same time it is called a coincidental frequency.

This discussion is to provide background on why SEH does not agree with RESPEC that the 10-year event should be used to determine the bottom elevation of the ponds. A better reference elevation for the pond's lowest elevation would be to relate it to either the Lake weir elevation or the OHW, or if event based, the other storm referenced in the BMP Manual which is the 2-year. We recommend that the 2-year event is the most appropriate base lake elevation since it is referenced in the BMP Manual as a storm event to be used for analysis of downstream impacts. The 10-year event is not referenced in the BMP manual and the probability of a 100-year major flood event in the residential neighborhood during a 100-year event on the Lake would have a much higher flood frequency then a 100-year so should not be used. The other event based elevation that would be appropriate is the Ordinary High-water Elevation (OHW).

One conclusion reached by RESPEC is that if the ponds are not above the 10-yeaar elevation of the lake, the designed system "could affect the ability of each BMP to provide consistent protection of Lake Kampeska's water resources. " The extra volume of runoff from the development will reach the lake in a short amount of time whether or not there are detention ponds. The ponds will only delay the volume reaching the lake and depending on the time of peak of the detention ponds, they could actual have a negative impact since the peak outflow timing has been delayed and may be closer to the lake peak time.

To put the volume impacts of the development in perspective, if you assume the site is 20.25 Acres, the increase in runoff can be based on a "C" value change of 0.25 (0.55 C post-developed minus 0.30 pre-

Back To Top

developed) and if the 100-year rainfall is 5.76-inches, the increase in elevation to the Lake is 0.00046 feet. We would conclude that this increase in volume and function of the detention facilities has no impact on Lake Kampeska's water resources from a water quantity standpoint.

The BMP manual clearly discusses the purpose of providing onsite BMP's for the 2-year and 100-year events. From the BMP manual: "Providing storage for the 2-year storm is meant to reduce erosion downstream, while providing storage for the 100-year storm is meant to reduce the possibility of damaging floods downstream". The downstream system is a lake so erosion in a channel or creek is not a concern. As discussed in the previous paragraph, the additional volume of runoff is negligible and will not cause additional flooding around the lake so the mitigation of the 100-year flow volumes in a detention pond may not be required based on the language in the BMP Manual.

The other BMP reference related to on-site pond design is that it is not required if the there is a regional downstream detention facility. The final point that could be made is Lake Kampeska is based on the definitions in the BMP manual a "regional detention facility" since it has identified flood elevations based on storing water from a large contributing watershed. If that is the case, then the on-site pond is required for water quality purposes only. Again, detention ponding is not critical to protection of Lake Kampeska, so the bottom elevation of the ponds from a water quantity standpoint are not a basis for concern in our opinion and thus we do not agree with RESPEC's conclusion.

The potential impact on the water resources is water quality not water quantity. RESPEC really does not discuss how the high elevation of the pond bottom or how the modeling would impact how the site treats the runoff from a water quality standpoint. Their main focus appears to be on water quantity. If the Developer provides the necessary volume for water quality and an outlet structure to provide rate control such as a standpipe outlet, the water quality requirements in the BMP Manual are being meet.

The final point is the datum's need to be checked. Based on the information sent to us by Shane Waterman (see below), the weir height and OHW (or High Water Mark) used by the developer is NGVD 29 but the rest of the plans are in NAVD 88. The flood elevations and FIS information is based on NAVD 88. The conversion is to add 1.0 feet to the NGVD 29.

1717.8 Full Lake
1718.3 High Water Mark
1723.8 1997 Actual
(1929 Datum)

January 16, 2009
BFE=1724.8 (NAVD 1988)
Lowest floor and/or top
of foundation = 1724.8
+1.0
1725.8

rjk
c: Al Murra, SEH

Back To Top

RESPEC

3824 Jet Drive, Rapid City, SD 57703 **605.394.6400** 651.394.6456 (fax) **respec.com**

March 24, 2016

Mr. Paul Hinderaker
Lake Kampeska Water Project District
P.O Box 966
Watertown, SD 57201

RE: Stony Point Third Addition Plan Review – March 2016 Update

Dear Mr. Hinderaker:

We have conducted a review and analysis of the Stony Point Third Addition 2016 Preliminary Plans dated March 3, 2016. In comparison to the previous plans from November of 2015, significant changes include treatment of the water quality capture volume for a reasonable majority of the development, on-site storage of the 2- and 100-year stormwater runoff events, and effort to maintain pre-development peak flow rates. However, we have identified potential issues with the placement and errors in the sizing of proposed stormwater best management practices (BMPs). These issues may limit the intended functions of the BMPs, ultimately leaving the water resources of Lake Kampeska at elevated risk for adverse impacts, and leaving the City of Watertown with ownership of potentially maintenance-burdened infrastructure. Our findings are detailed in this memo.

CORRESPONDENCE & AVAILABLE DATA

RESPEC received PDF documents of the updated preliminary plans for Stony Point Third Addition on March 15, 2016. The documents and supplemental references used in performing this review included:

- Stony Point Third Addition 2016 Preliminary Plans, PDF by Aason Engineering Company (Dated February 17, 2016 and stamped March 3, 2016)
- Stony Point 3 Add – Hydraulic Analysis 160304, PDF by Aason Engineering Company (Hydrograph Report dates ranging from January 21 – February 18, 2016)
- City of Watertown Post-Construction Stormwater Best Management Practices Manual (2008)
- FEMA FIRM Map 46029C0315D (January 16, 2009)

Also, RESPEC contacted the U.S. Army Corps of Engineers (USACE) on March 21, 2016 to understand what new information may have been submitted and the status of the permit for this project, but only discussions have been exchanged between the USACE and the developer since the end of 2015. The USACE will ultimately need additional information before moving forward in the permitting process for this project.

APPLICABLE STORMWATER DESIGN GUIDANCE

As with our review of the previous plans, this review was guided in large part by the City of Watertown's Post-Construction Stormwater Best Management Practices Manual (Manual). Where the previous review had focused on applicability of the Manual to the Stony Point Third Addition project and on

Back To Top

changes to the site hydrology, this review focuses on the proposed BMPs and the specific hydrologic and hydraulic calculations used to size them.

Section 5 of the Manual provides BMP requirements for new development, and outlines them in three steps:

- Step 1 – Employ Runoff Reduction Practices
- Step 2 – Provide Water Quality Capture Volume
- Step 3 – Provide 2- and 100-Year Storage Volume

Section 5.4 of the Manual recommends that water quality capture volume (WQCV) facilities be incorporated into stormwater quantity detention facilities wherever possible. This section goes on to stipulate the following BMP design requirements:

- Stormwater quantity basins shall be designed for the 2- and 100-year 24-hour design flows
- Runoff captured from the 2- and 100-year 24-hour storm must be released at a rate less than the pre-development peak rate

Section 5.4 identifies simple, "conservative" methods for estimating both design volumes and maximum release rates for the 2- and 100-year 24-hour storm events. These methods are based on the Rational Method, which is generally acceptable for highly impervious sites less than 100 acres in size, and should be considered acceptable for this project. The equations are included as an attachment to this memo.

Section 5.6 of the Manual provides guidance for determining the required WQCV of a BMP based on the characteristics of the area draining to it and on desired BMP drain time. The equation and chart provided by the Manual are included as an attachment to this memo.

PRE-DEVELOPMENT HYDROLOGY

The updated plans for Stony Point Third Addition identify seven subcatchments in the post-development conditions, named Area A through Area G. The plans used the rational method according to the equations given in the Manual to determine the pre-development peak flow rates for the 2- and 100-year storm events for each of the subcatchments. The plans use runoff coefficients, C, ranging from 0.35 to 0.45, identified as representing turf meadows of varying slopes. The subcatchment characteristics and pre-development peaks are shown in Table 1.

Back To Top

Table 1. Subcatchment information for pre-development conditions.

Sub-catchment	Total Area (acres)	C	2-yr Peak Flow (cfs)	100-yr Peak Flow (cfs)
Area A	2.50	0.45	3.2	8.1
Area B	4.45	0.45	5.7	14.5
Area C	6.52	0.45	8.4	21.2
Area D	2.60	0.35	2.6	6.6
Area E	1.69	0.40	1.9	4.9
Area F	0.97	0.40	1.1	2.8
Area G	1.52	0.45	2.0	4.9

The peak flow values shown in Table 1 were calculated in the plans in order to determine pre-development peak flow rates. Ultimately, these represent the maximum rates at which runoff may be released under post-development conditions for each respective storm.

Section 2.1 of the Manual states that the pre-developed (or "historic") condition of a study area shall be described using an average runoff coefficient not more than 0.30 for the Rational Method. However, all runoff coefficients used for pre-development conditions exceed 0.30. Table 2 shows peak flow rates calculated for each subcatchment using a runoff coefficient of 0.30 and shows the resultant reduction in peak flow rates from those calculated in the plans.

Table 2. Pre-development peak rate comparison.

Sub-catchment	Total Area (acres)	C	2-yr Peak Flow (cfs)	100-yr Peak Flow (cfs)	Plans - Peak Rate Reduction
Area A	2.50	0.30	2.1	5.4	33%
Area B	4.45	0.30	3.8	9.6	33%
Area C	6.52	0.30	5.6	14.1	33%
Area D	2.60	0.30	2.2	5.6	14%
Area E	1.69	0.30	1.5	3.7	25%
Area F	0.97	0.30	0.8	2.1	25%
Area G	1.52	0.30	1.3	3.3	33%

Table 2 shows that pre-development peak flow rates should be significantly reduced from those calculated for the plans; up to 33% for some of the subcatchments. The result of lowering the maximum peak flow rates is that greater storage volume must be provided for each BMP.

POST-DEVELOPMENT HYDROLOGY

For post-development conditions, the rational method was again used according to the Manual to determine peak flow rates and volumes for the 2- and 100-year storm events in the plans. Runoff coefficients ranging from 0.51 to 0.55 were used for the post-development conditions, which are consistent with the proposed development. The subcatchment characteristics and post-development peaks are shown in Table 3.

Back To Top

Table 3. Subcatchment information for post-development conditions.

Sub-catchment	Total Area (acres)	C	2-yr Peak Flow (cfs)	2-yr Runoff Volume (ac-ft)	100-yr Peak Flow (cfs)	100-yr Runoff Volume (ac-ft)
Area A	2.50	0.55	3.9	0.28	9.9	0.66
Area B	4.45	0.55	7.0	0.49	17.7	1.17
Area C	6.52	0.53	9.9	0.69	24.9	1.66
Area D	2.60	0.51	3.8	0.27	9.6	0.64
Area E	1.69	0.55	2.7	0.19	6.7	0.45
Area F	0.97	0.55	1.5	0.11	3.9	0.26
Area G	1.52	0.53	2.3	0.16	5.8	0.39

The peak flow rates and volumes shown in Table 3 appear reasonable and the same values were calculated as part of this review. These values are inputs to each subcatchment's BMP, ultimately used for sizing each BMP and configuring its outlet to maintain pre-development peak flow rates.

The equations given in the manual were used for determining post-development WQCVs. The volumes were determined based on 40-hour drain times for each BMP. The WQCVs reported in the plans were found to be greater than those determined through calculation, likely due to BMP sizing needs. The WQCV values for each subcatchment, both calculated by the Manual equation and reported in the plans, are shown in Table 4.

Table 4. Water quality capture volume information for each subcatchment.

Sub-catchment	Total Area (acres)	Imp. Area	i	a	WQCV-Calc (cf)	WQCV-Plans (cf)	Plans – WQCV Oversize
Area A	2.50	1.14	46%	1.0	2140	2880	26%
Area B	4.45	2.01	45%	1.0	3780	4610	18%
Area C	6.52	2.71	42%	1.0	5270	6380	17%
Area D	2.60	1.10	42%	1.0	2120	2590	18%
Area E	1.69	0.34	20%	1.0	860	1260	31%
Area F	0.97	0.19	20%	1.0	490	700	30%
Area G	1.52	0.53	35%	1.0	1110	1510	27%

The comparison in Table 4 shows that WQCVs are oversized in the plans by amounts ranging from 17 percent to 31 percent. As stated before, these differences are likely due to specific BMP design considerations, and this should not be an issue.

BMP HYDRAULIC ROUTING

Although the rational method described in the Manual is adequate for determining approximate runoff volumes, BMPs must be sized and their outlets configured through consideration of hydraulic routing. With hydraulic routing through a BMP, the duration of runoff and the timing of the peak flow drive the ultimate design. The duration and timing of runoff is determined through application of a design storm. Section 2.3 of the Manual addresses design storm calculations and states:

Back To Top

"Unless a continuous simulation approach to drainage system hydrology is used, all design rainfall events will be based on the Soil Conservation Service (SCS, now called NRCS) Type II distribution. Computations of runoff hydrographs that do not rely on a continuous accounting of antecedent moisture conditions will assume a conservative wet antecedent moisture condition."

As indicated in Section 5.4 of the Manual, the design storm for both the 2- and 100-year events must be 24 hours in duration. The SCS Type II distribution places the maximum rainfall intensity halfway through the storm, at 12 hours in. Ultimately, the runoff hydrograph produced should also peak near the 12-hour mark, at approximately the same rate determined for post-development conditions using the Rational Method. The full 24-hour hydrograph should also represent the entire post-development runoff volume determined with the Rational Method. Figure 1 shows inflow and release hydrographs for the 100-year storm at subcatchment Area A, as calculated in the plans. The blue "Hyd No. 1" hydrograph is the inflow hydrograph and the red "Hyd No. 2" hydrograph is the BMP outflow hydrograph.

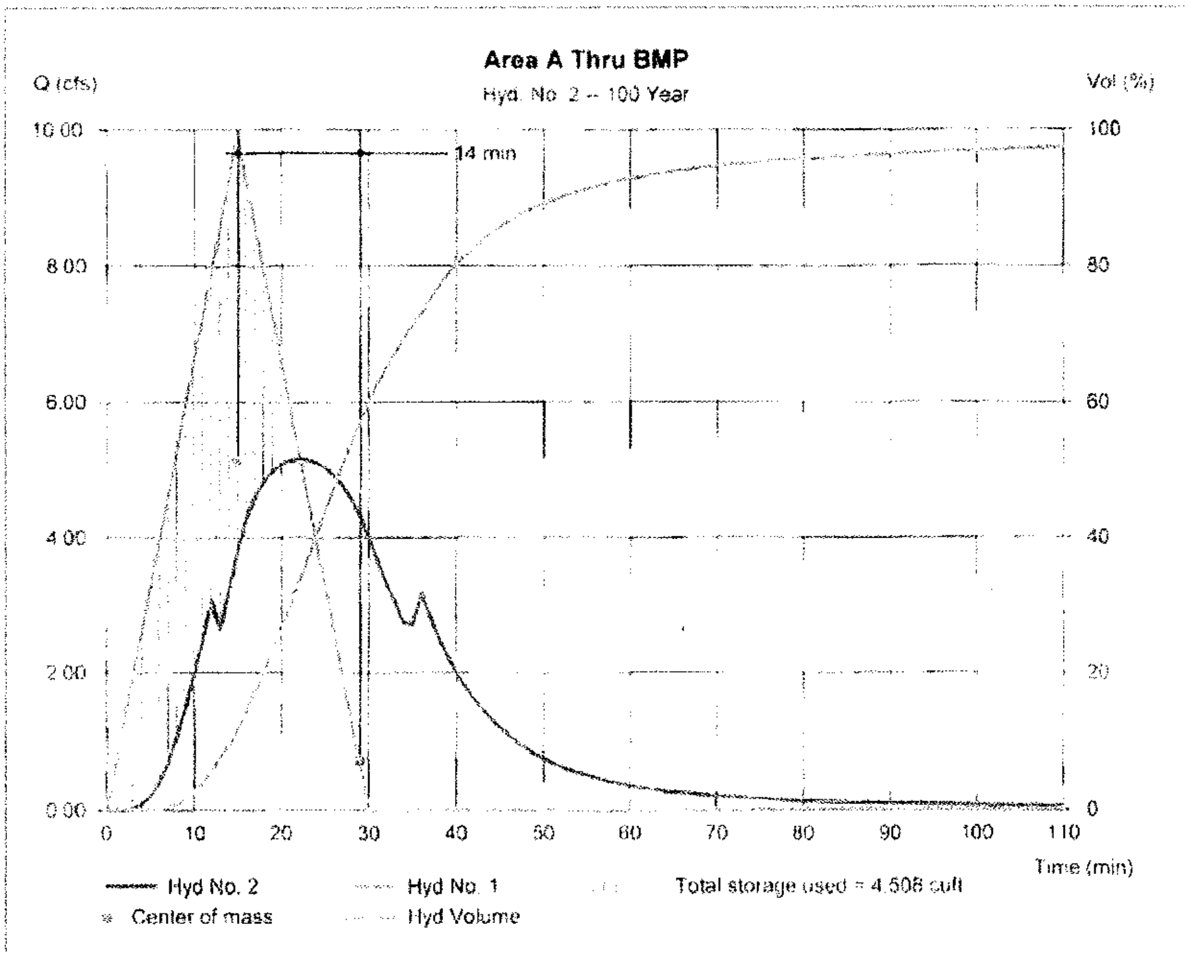

Figure 1. 100-year inflow hydrograph (blue Hyd No. 1) and BMP outflow hydrograph (red Hyd No. 2) for Area A.

Figure 1 shows that the outflow hydrograph has jumps in both the rising and falling limbs of the outflow hydrograph. Similar outflow hydrograph characteristics appear in in the hydrograph reports for the Area C BMP. It is our experience and opinion that the characteristics of these outflow hydrographs may

Back To Top

indicate errors in routing calculations. As a BMP fills with water from a steady inflow hydrograph and the ponded depth increases, the outflow hydrograph should only ever increase up to the peak flow rate; it should not decrease prior to the peak.

It can also be seen in Figure 1 that the inflow hydrograph has a peak of 9.9 cfs, time to peak of 14 minutes, and total duration of 30 minutes. At 9.9 cfs, the peak flow of this hydrograph matches the post-development 100-year peak flow calculated for Area A (shown in Table 3). However, this hydrograph does not represent runoff produced by the SCS Type II 24-hour design storm. This hydrograph has a total volume of 0.08 acre-feet, while Table 3 shows that the 100-year runoff volume for Area A should be around 0.66 acre-feet. Figure 2 shows a hydrograph produced with HydroCAD using the characteristics of Area A and the SCS Type II 24-hour storm at the 100-year rainfall total of 5.76 inches for Watertown, and compares it to the hydrograph used in the plans. The SCS Type II produces a hydrograph containing approximately 0.66 acre-feet, matching the total volume estimated in Table 3.

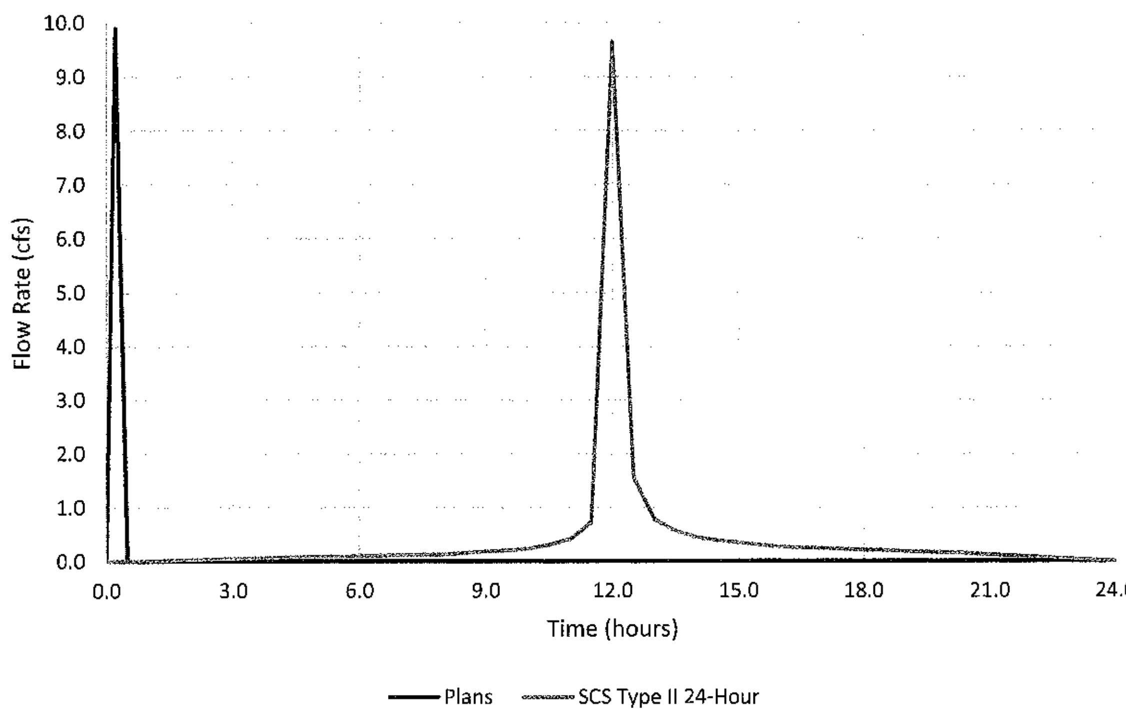

Figure 2. Comparison of the hydrograph used in the plans (blue) and the hydrograph produced by the SCS Type II 24-hour design storm for the 100-year event over Area A.

The timing characteristics of the hydrograph used in the plans for Area A are the same for all hydrographs used in the plans to size the proposed BMPs; each used the appropriate peak flow rate, but a time to peak of about 15 minutes and a total duration of 30 minutes. These characteristics result in significant underestimation of the storage volume required for each BMP. Table 5 compares the post-development runoff volumes estimated using the Rational Method as specified in the Manual with the hydrograph volumes used for BMP sizing and outlet configuration.

Back To Top

Table 5. Comparison of post-development runoff volumes used in the plans for BMP sizing and volumes calculated using the manual.

Sub-catchment	2-year Volume (acre-feet)		Plans Volume Underestimation	100-year Volume (acre-feet)		Plans Volume Underestimation
	Plans	Manual		Plans	Manual	
Area A	0.08	0.28	70%	0.21	0.66	69%
Area B	0.14	0.49	70%	0.37	1.17	69%
Area C	0.20	0.69	70%	0.52	1.66	69%
Area D	0.08	0.27	70%	0.20	0.64	69%
Area E	0.05	0.19	70%	0.14	0.45	69%
Area F	0.03	0.11	70%	0.08	0.26	69%
Area G	0.05	0.16	70%	0.12	0.39	69%

Table 5 shows that all hydrograph volumes used for BMP sizing and outlet configuration underestimate the post-development runoff volumes by about 70 percent. This results in undersizing of the BMP storage volumes. Table 6 compares the BMP design volumes with the post-development runoff volumes estimated with the manual.

Table 6. Comparison of BMP design volumes with post-development runoff volumes estimated using the manual.

Sub-catchment	Volume (acre-feet)		Difference versus Calculated
	Designed	Calculated	
Area A	Not Given	0.66	-
Area B	0.29	1.17	76%
Area C	0.24	1.66	85%
Area D	0.11	0.64	82%
Area E	0.08	0.45	81%
Area F	0.04	0.26	86%
Area G	0.06	0.39	84%

Table 6 shows that the BMP design volumes range from 76 percent to 86 percent less than the 100-year hydrograph volumes estimated using the Manual guidelines.

Back To Top

BMP ELEVATIONS

When designing BMPs, both for water quality and water quantity control, it is recommended that the minimum BMP elevation be set above any adjacent 100-year water surface elevation (WSEL). The Manual does not provide guidance on this issue, but other communities require that minimum BMP elevations be placed anywhere from 0.5 feet to 3 feet above the 100-year WSEL. As discussed in our previous review, the Flood Insurance Study for Codington County, effective 2009, lists the 10-, 50-, and 100-year WSELs for Lake Kampeska at 1721.2 feet, 1723.4 feet, and 1724.8 feet, respectively. Table 7 lists the bottom elevations of the proposed BMPs.

Table 7. Bottom elevations of the BMPs proposed in the plan.

Sub-catchment	BMP Bottom Elevation	Lake Kampeska 10-yr WSEL	Lake Kampeska 100-yr WSEL
Area A	1719.0	1721.2	1724.8
Area B	1719.8		
Area C	1724.2		
Area D	1720.9		
Area E	1720.8		
Area F	1721.0		
Area G	1720.9		

Table 7 shows that all six of the seven BMPs are below the 10-year WSEL of Lake Kampeska, and all BMPs are below the 100-year WSEL. At elevations below the 10-year water surface elevations, there is a 10 percent chance each year that six of the seven BMPs will undergo flooding. These BMPs will not protect the water resources of Lake Kampeska if they are flooded.

Back To Top

CONCLUSIONS

The Stony Point Third Addition 2016 Preliminary Plans still result in a significant increase of the site's area draining to Lake Kampeska, as discussed in our previous review. These revised plans do include water quality BMPs and make effort to provide on-site storage for the 2- and 100-year storm events, as required by the City's Post-Construction Stormwater Best Management Practices Manual. However, we have identified potential issues with the placement, and errors in the sizing of the proposed BMPs.

- The pre-development peak flow rates in the plans are calculated using higher runoff coefficients than allowed in the Manual
    - This results in underestimation of required BMP storage volumes
- The post-development runoff hydrographs used for BMP routing are incorrect and underestimated
    - This results in underestimation of required BMP storage volumes
- Outflow hydrographs from two BMPs appear to have abnormal jumps that should be explained further
- Six of the seven BMPs are located below the 10-year water surface elevation of Lake Kampeska, and all seven BMPs are located below the 100-year water surface elevation
    - This could affect the ability of each BMP to provide consistent protection of Lake Kampeska's water resources

Also, the plans state that all storm water conveyance and treatment facilities are intended to be turned over to the City upon acceptance of the final construction. Undersized BMPs may result in more-frequent overtopping, potentially creating increased maintenance issues and an increased chance of BMP failure. In order to correct the identified issues, significant changes to the designs and drainage easements proposed in the Stony Point Third Addition 2016 Preliminary Plans will likely be required.

Thank you for the continued opportunity to provide services to the Lake Kampeska Water Project District. If you have any questions or would like any additional information, please contact Pete Rausch by email (*peter.rausch@respec.com*) or phone (605.394.6400).

Sincerely,

Pete Rausch, P.E.
Staff Engineer

Lee Rosen, P.E., CFM
Staff Engineer

ATTACHMENT A

EQUATIONS FROM THE WATERTOWN POST-CONSTRUCTION STORMWATER BEST MANAGEMENT PRACTICES MANUAL

Back To Top

The following approach is suggested:

- Water quality: The full WQCV is to be provided according to the design procedures documented for the structural BMP.
- 2-year storm: The WQCV plus the full 2-year detention volume is to be provided.
- 100-year storm: The WQCV plus the full 100-year detention volume is to be provided.

5.4.1 Design Storm

Storm water quantity basins shall be designed for 2- and 100-year 24-hour design flows.

5.4.2 Release Methods

Careful consideration must be given to the discharge of the surface release as to the elimination of erosion potential and the capacity of the downstream surface water course. The release structure shall be designed to withstand the forces caused by the structure being overtopped during a larger-than-design storm.

The 100-year detention level is provided above the WQCV, and the outlet structure is designed to control two or more different releases. Standard Drawing 4 (Attachment 1) shows an example of a combined quality/quantity outlet structure.

5.4.3 Maximum Release Rate

The detention pond volumes and release rate shall be designed to accommodate runoff generated by the development and post-developed upstream properties. Runoff captured from the 2-year and the 100-year 24-hour storm must be released at a rate less than the pre-development peak rate.

5.4.4 Design Procedure

The following steps outline a calculation method that meets the minimum standards of the City of Watertown. Refer to Section 2.3, Design Storm Calculations, for additional approved calculation methods.

1. Step 1—Calculate Storage Volumes

   A conservative estimate of the design volume in acre-feet can be calculated by multiplying the 24-hour precipitation depth by the watershed area that is contributing runoff as follows:

$$Design\ Volume_{100\,year} = \left(\frac{5.76\ in}{12}\right) * C * Area \qquad \textbf{(Equation 4)}$$

   in which:

   Area = The watershed area tributary (acres)
   C = Post-development Rational Method Runoff Coefficient

$$Design\ Volume_{2\,year} = \left(\frac{2.40\ in}{12}\right) * C * Area \qquad \textbf{(Equation 5)}$$

in which:

Area = The watershed area tributary (acres)
C = Post-development Rational Method Runoff Coefficient

2. Step 2—Calculate Release Rates

   Pre-development peak runoff rates can be calculated using the Rational Method as follows:

$$Maximum\ \mathrm{Re}lease\ Rate_{100\,year} = 7.22\ {}^{in}\!/_{hr} * C * Area \qquad \textbf{(Equation 6)}$$

in which:

Area = The watershed area tributary (acres) Assumes 15-min tc
C = Pre-develoment Rational Method Runoff Coefficient

$$Maximum\ \mathrm{Re}lease\ Rate_{2\,year} = 2.86\ {}^{in}\!/_{hr} * C * Area \qquad \textbf{(Equation 7)}$$

in which:

Area = The watershed area tributary (acres)
C = Pre-develoment Rational Method Runoff Coefficient

3. Step 3—Incorporate Water Quantity Volume into WQCV Basin

   Using guidelines provided for the selected basin, size the basin to provide additional capacity for the 2- and 100-year storms.

4. Step 3—Outlet Design

   Design a multiple-stage outlet to control the WQCV, 2-year, and 100-year storm volumes to the appropriate release rate.

5.4.5 Adjacent Property Elevations

The property corner elevation of properties abutting a basin shall be 1 foot above the 100-year design storm. Recommended minimum ground elevations for homes abutting or affected by the basin shall be 2 feet above the overflow elevation. The recommended minimum ground elevation for homes abutting or affected by basins will be a minimum of 4 feet above the 100-year pond high water elevation if an overflow system is not available or at an elevation that provides an additional 50 percent storage.

5.4.6 Parking Lots

Parking lots that serve as detention storage ponds must not have a storage depth of more than 1 foot. It is recommended that notification signs be installed in parking lots that serve as detention ponds. The signs shall be permanent and high quality, meeting requirements of the Manual on Uniform Traffic Control Devices.

Back To Top

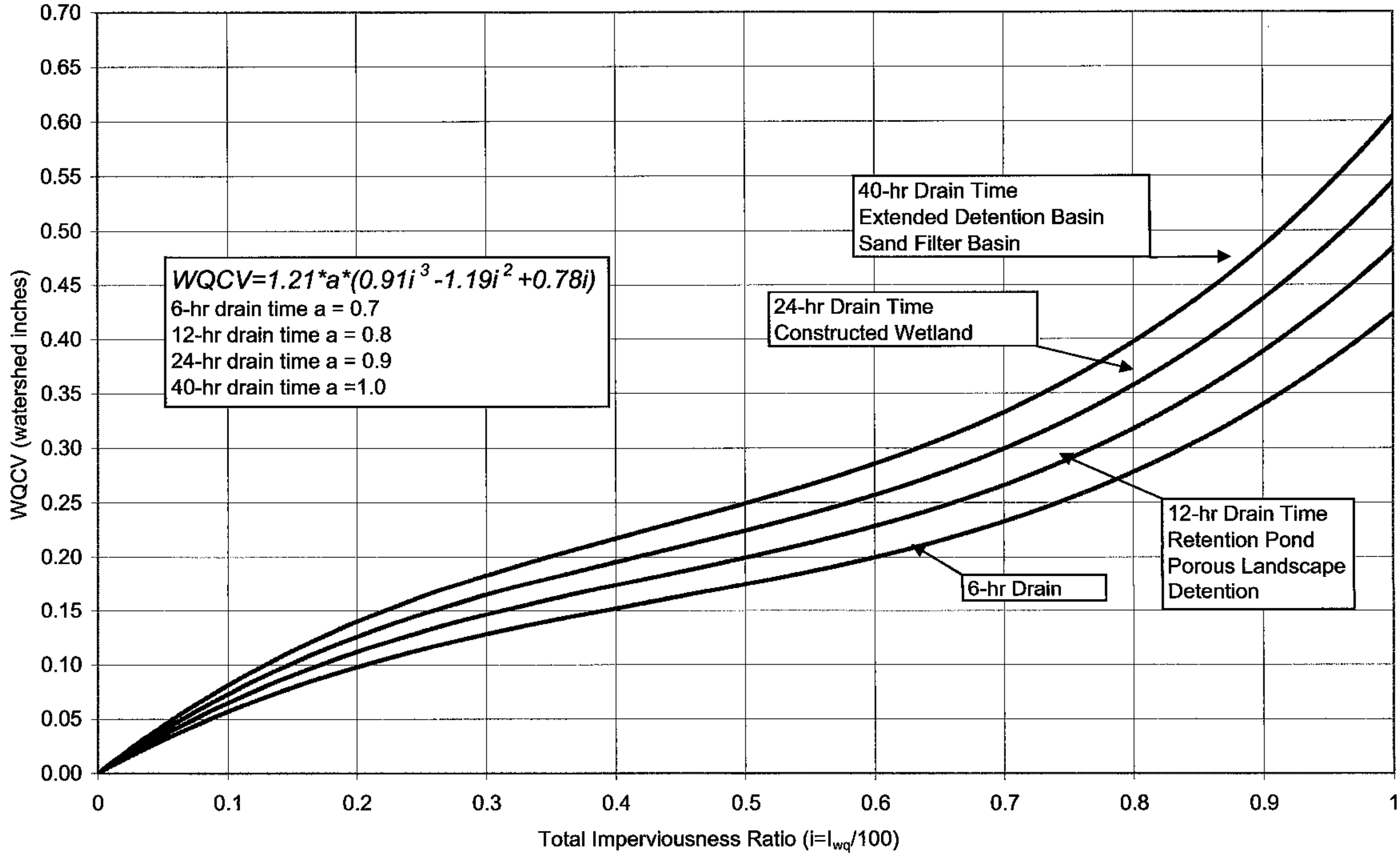

EXHIBIT 13
Water Quality Capture Volume,
80th Percentile Runoff Event
Watertown Post-Construction Manual