

EXIT 96 INDUSTRIAL PARK PRELIMINARY ENGINEERING REPORT AND MASTER PLAN (DRAFT)



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Executive Summary

The Waynesboro Economic Development Authority (EDA) is in the process of developing the 169+/- acre Exit 96 Industrial Park, located just off of S. Delphine Avenue adjacent to I-64 and the South River and has taken steps to develop the site as a Tier 4 or Tier 5 site per the TG Tier System for site readiness. As part of this development process, the Waynesboro EDA contracted Timmons Group to perform additional due diligence on the site and update the Master Plan and Preliminary Engineering Report accordingly such that the EDA can continue to advance the development of the site.

This Preliminary Engineering Report and Master Plan outline the potential development options, both from an infrastructure and site development perspective, and identifies improvements necessary to advance the site accordingly.

Site Constraints

There were several site constraints that we needed to take into consideration, especially when it came to evaluating transportation alternatives. The site generally consists of topography falling from S. Delphine Avenue to the South River, with an average grade of approximately 5-7% across the site. Site elevations range from EL 1309 at the South River to EL 1462 at S. Delphine Avenue, with the 100-yr floodplain as identified by FEMA mapping ranging from EL 1323 to EL 1325. The current rail elevation at the proposed road crossing is approximately EL 1370. Based upon previous environmental studies, there appear to be minimal environmental issues that would negatively affect the site development options for this property.

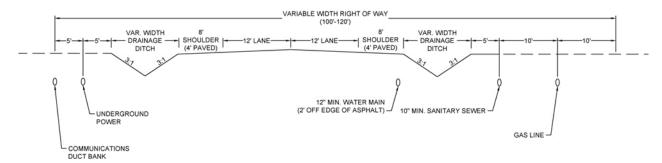
Transportation Alternatives

In order to evaluate the potential transportation (or connector road) alternatives, we took the following constraints into consideration:

- The tie-in at S. Oak Lane at EL 1332
- The at grade crossing of the Norfolk-Southern rail line in a tangent section at EL 1370
- Allowing adequate site distance for the tie-in to S. Delphine Avenue
- Minimize the road grade to 5% or less (to minimize access issues during extreme weather)
- Maximize lot sizes for marketing and development (at least one 50 acre lot size)
- Cut through traffic from S. Oak Lane to S. Delphine Avenue

Based upon a review of the potential uses for the site, combined with potential cut-through traffic anticipated, it was ultimately concluded that a two lane road would be more than adequate for the site and subsequent connection between S. Oak Lane and S. Delphine Avenue. Furthermore, it was concluded that this road would need to be 32' wide (two 12' lanes with a 4' paved shoulder on each side). After meeting with City Staff, it was noted they would be willing to make an exception for curb and gutter and sidewalk provided that the EDA would consider installing a paved asphalt walkway along or near the road.

Another consideration regarding the roads was installation of utilities along this corridor. It was concluded that the wet utilities (water main and sanitary sewer) and natural gas should be installed on the northeastern, or development side of the road and dry utilities (power and communications) should be installed on the southeastern, or non-development side of the road. Below is a potential road cross-section through the road:



Total estimated cost for the road as proposed is approximately \$5,138,000. The EDA has indicated they would like a nice landscaped entrance as well as entrance sign such that clients have a strong "sense of arrival", so for the purposes of this report, we recommend a budget of \$200,000 to \$300,000 for signage and a landscaped entrance. Since the City is going to potentially allow for an asphalt walkway in lieu of the curb & gutter and sidewalk along the connector road, we recommend a budget of \$60,000 (3,000 LF \times \$20 per LF for a 5' wide asphalt walkway).

Master Planning Process

In working through the Master Planning process, it was concluded that the Waynesboro EDA wanted to have a minimum 50 acre site with rail access that could be proactively marketed to potential prospects. It was possible to achieve this on the "south" side (or towards S. Delphine Avenue) of the rail line. Once we laid out the potential road and rail options, we were able to preserve a 60+/- acre site on the south side of the rail and a 40+/- acre site on the north side (towards S. Oak Lane and the South River) of the

rail that could be served by Norfolk-Southern. Furthermore, we were able to identify potential locations for a PGI offloading facility and preserve the integrity of the sites.

As a part of the overall study, preliminary concept layouts were prepared, with the goal to establish multiple options for development, capitalizing on the site's location and access to rail. The layouts were developed to allow for maximum build out, while also taking into consideration grade constraints, existing road connection points and the rail location. Following is a summary table of the options considered and the maximum building footprints:

	North of Rail	South of Rail	Build Out	Yield
Development Options	ment Options Total Total (SF) (SF)		Potential (SF)	(SF/Acre)
	· · · · · · · · · · · · · · · · · · ·	. ,	. ,	
Option 1	450,000	825,000	1,275,000	7,544
Option 2	450,000	700,000	1,150,000	6,805
Option 3	500,000	700,000	1,200,000	7,701
Option 4	500,000	600,000	1,100,000	6,509
Option 5	500,000	500,000	1,000,000	5,917
Option 6*	500,000	1,000,000	1,500,000	8,021

^{*} Please noted that Option 6 took into account the possibility of acquiring the parcel adjacent the Exit 96 Interchange which would provide an additional 18+/- acres of property.

We evaluated the potential costs to develop "pad ready" or Tier 5 sites for each of these options. The total costs to develop "pad ready" sites ranged from \$9,179,000 to \$11,764,000 (or \$91,790 to \$117,640 per developable acre), which appears to be consistent for sites located throughout this region. Below is a table summarizing those costs and the costs per developable acre:

	Total Costs for	Developable	Costs per
Grading Option*	"Pad Ready" Sites	Acreage	Developable Acre
Option 1	\$11,764,000	100	\$117,640
Option 2	\$10,462,000	100	\$104,620
Option 3	\$10,023,000	100	\$100,230
Option 4	\$9,833,000	100	\$98,330
Option 5	\$9,179,000	100	\$91,790
Option 6	\$13,268,000	120	\$110,567

^{*} Given the multiple variations as to how the site can be developed, we do not recommend that the EDA invest in "pad ready" or Tier 5 sites at this point in time.

Utility Infrastructure

In reviewing the potential water and sewer demands for the park, it was concluded that the EDA should plan around 250,000 GPD for water and sewer demands at build-out of the park. Currently the site has no water and sewer infrastructure in place and would need to be designed to accommodate this potential demand and an elevated water storage tank would need to be installed to accommodate potential fire flows for the park. The minimum fire demands required would be 2,000 gpm for 2 hours (or 120 min), thereby requiring a minimum storage for fire flow of 240,000 gallon. Based upon this potential demand and required fire flow storage, the City would need to install a minimum 500,000 gallon elevated storage tank on the sites.

As such, we determined the following utility infrastructure would be required for the park:

- 10" gravity sewer along the proposed connector road to a potential pump station location along the South River at a cost of approximately \$595,000;
- 500,000 gallon elevated storage tank along S. Delphine Avenue and a water booster pump station at a cost of approximately \$2,152,000 and \$317,000, respectively;
- 12" waterline along S. Delphine Avenue along the proposed connector road to S. Oak Lane at approximately \$595,000.

Summary of Project Costs

Following is a summary table identifying the potential high and low range for total development costs to achieve a Tier 4 and Tier 5 site status level:

Exit 96 - Summary of Total Costs	Low	High	
Infrastructure Costs (Tier 4)			
Connector Road (VDOT Standard)	\$5,138,000	\$5,138,000	
Landscaped Entrance / Signage	\$200,000	\$300,000	
5' Asphalt Walkway Along Road (3,000 LF x \$20/LF)	\$60,000	\$60,000	
5' Asphalt Fitness Trail in Park (7,500 LF x \$20/LF)	\$150,000	\$150,000	
10" Gravity Sewer	\$595,000	\$595,000	
500,000 Gal Elevated Storage Tank	\$2,152,000	\$2,152,000	
Water Booster Pump Station	\$317,000	\$317,000	
12" Water Line	\$665,000	\$665,000	
Total Infrastructure Costs (Tier 4 Status)	\$9,277,000	\$9,377,000	
Total "Pad Ready" Site Development Costs (Tier 5)	\$9,179,000	\$11,764,000	
Total Potential Development Costs (Tier 5 Status)	\$18,456,000	\$21,141,000	

Conclusions and Recommendations

Given our understanding that the City / EDA desires to achieve a Tier 4 or Tier 5 status for the site, we have drawn the following conclusions and are making the following recommendations:

- 1. The Master Planning process revealed that we could achieve a 40 acre rail served site on the north side of the rail, and a 60 acre rail served site on the south side of the rail, thereby giving the City / EDA a very marketable product.
- 2. We believe it is in the best interest of the City / EDA to install the necessary infrastructure (Roads, Water & Sewer) for the park to develop a Tier 4 site.
- 3. Given the multiple scenarios under which the sites can be developed and potential costs for achieving a "pad ready" or Tier 5 site, we do not believe it is worth the City / EDA making this investment as we believe it will reduce the overall flexibility with the site from a marketing perspective.

Acknowledgements

We would like to thank the City of Waynesboro and Waynesboro EDA for their time and assistance with this project. In particular we would like to thank the following individuals:

- Greg Hitchin, CEcD, Director of Economic Development and Tourism
- Jim Shaw, Assistant City Manager
- Mike Hamp, City Manager
- Michael Barnes, City Planner
- Todd Wood, City Engineer
- Jim Powers, Assistant City Engineer

Waynesboro EDA Members:

- Jim Hyson, Chairman
- Lorie Strother, Vice Chairman
- Tom Reider
- Perry Fridley
- Sharon Plemmons
- Robert Vailes
- Mary Sullivan

Participants & Valued Partners from June 16, 2015 Work Session:

• Bruce Allen, Waynesboro City Council

- Pete Marks, Waynesboro City Council
- Jon Slaunwhite, Columbia Gas
- Doug Bishop, Dominion Power
- Bryan Smith, Dominion Power
- Russell King, Lumos Networks
- Robert Thompson, Lumos Networks
- Henry Guess, Lumos Networks
- John Loftus, VEDP

It has been a pleasure working with everyone to date on this project to date and we look forward to implementing the recommendations in this report.

Section 1 - Project Purpose and Scope

In 2011, the Exit 96 Industrial Park property was purchased by the City of Waynesboro to be developed as an industrial park that would support their economic development vision. The search for the site, included up to a dozen sites, however, the vicinity of the site to Interstate 64 and the potential for rail access led to its eventual selection and acquisition.

The City and Waynesboro EDA has procured Timmons Group to provide professional services associated with updating the Master Plan and Preliminary Engineering Report, as well as additional due diligence work to help the City determine the most appropriate manner in which to develop the site and position it for future economic development success. Previously, the City of Waynesboro had engaged Thompson & Litton to prepare a Master Plan and Preliminary Engineering Report, along with the aid of EEE Consulting, Inc. and Schnabel Engineering to provide assistance with wetland mapping and geotechnical investigation. Timmons Group utilized information from these previous studies where applicable.

The purpose of this Preliminary Engineering Report and Master Plan is to evaluate and develop options for developing this site to a Tier 4 or Tier 5 status as well as identify the associated costs. This study included evaluating various road options, site development options and determining potential utility needs as well as verify the adequacy of fiber and power to serve the site. We also performed additional due diligence items for the site that had not yet been completed by the Waynesboro EDA.

Section 2 - Master Planning

While developing the program for the site, Timmons Group considered several factors. The predominant factor is a desire to capitalize on the Site's unique geographic location, straddling the edge of both the Shenandoah Valley and Central Virginia regions, while having access to the Interstate and onsite rail. Six options for the project were developed for maximizing the building footprint area onsite. The information is provided below in Table 1.

Phase 1 Development Options	Build Out Potential (SF)
Option 1	1,275,000
Option 2	1,150,000
Option 3	1,200,000
Option 4	1,100,000
Option 5	1,000,000
Option 6	1,500,000

Table 1: Gross Floor Area by Phase

The six alternatives are provided as exhibits in Appendix A. Another goal throughout the master planning process was to allow the opportunity to preserve 50 contiguous acres, as that appears to be the appropriate threshold to be desirable in the marketplace.

Section 2.1 - Property Description

The property is approximately 169 AC \pm , bounded on the north by the South River and S. Delphine Avenue on the south. The site is bisected by the Norfolk Southern Railway, dividing the site into a north end and a south end. The area to the north is approximately 66 AC \pm , while the south is approximately 103 AC \pm .

According to one-foot interval aerial topographic mapping, the north topography ranges from a low elevation of 1309' to a high elevation of 1393', while the south topography ranges from a low elevation of 1345' to a high elevation of 1462'. The topography is rolling, with the majority of slopes to be impacted under 25%. The steeper slopes (greater than 25%) are mostly located at the areas of stream and along the railroad, where the terrain was sloped to keep the rail level.

Site drainage is generally split to the east and west, with both draws draining to the South River, which parallels the northern property line. Water and Sanitary Sewer service will need to be

brought to the site, with this study suggesting water be brought from the South Delphine side, along with the addition of a 500,000 gallon water tank, while sanitary sewer is proposed to be gravity fed to the northern corner of the property and then pumped underneath the South River via a new pump station, allowing it to connect to the gravity system located at the intersection of North Oak Lane and Lyndhurst Road.

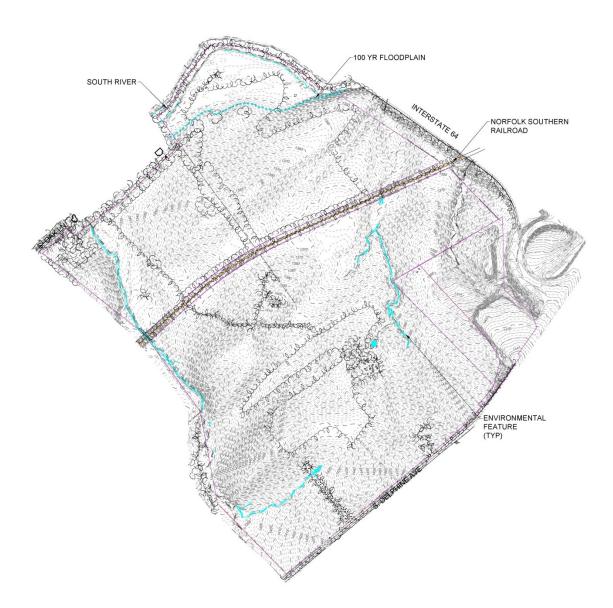


Figure 1: Existing Conditions of Exit 96 Industrial Park



Figure 2: Key Features of Surrounding Area

Section 2.2 - Property Constraints

The property features several constraints that were considered throughout the Master Plan and Preliminary Engineering Report. Constraints that were considered include the topography, flood plain, environmental features and potential for karst. The tie in to South Oak Lane also served as a fixed constraint, while the rail crossing and connection to South Delphine Avenue were other variable constraints that required consideration.

The topography throughout the site slopes consistently down from South Delphine Avenue to the north at the railroad. This slope influenced both the location of the access road as well as the need to tier the sites on the south side, relative to the rail. Due to the topography, retaining walls will be required to maximize the build out potential's square footage.

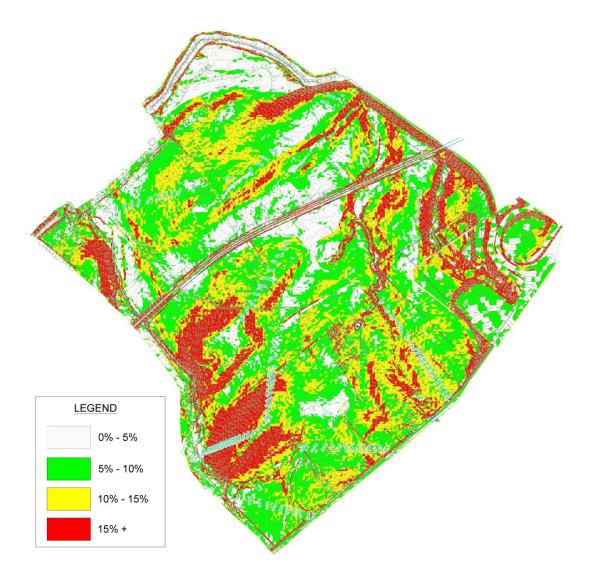


Figure 3: Slope Analysis Mapping

Floodplain constraints were also considered for the northern most point of the site, as the flood plain extends off of the South River, affecting approximately 8 acres of the site. The floodplain does not allow for fill to occur within it, therefore retaining walls may be required in the ultimate build out and were considered an option during the development of the Master Plans.

The environmental features, streams and wetlands, are fairly minimal on the site. Their location does limit the ability to develop the northeast corner of the site (south of the rail), as it would be difficult to traverse the streams in the area to gain access to this area. Additionally, the grading was minimized at the stream crossings, with 2:1 slopes considered.

The potential for karst was also considered, as there were a few locations identified on the site.

Resistivity testing and mitigation options were developed in a separate study, noting construction measures to be adhered to, thereby reducing the potential for subsurface failure on the site.

Section 2.3 - Transportation Alternatives

In evaluating the transportation alternatives, we took the following constraints into consideration:

- The tie-in at S. Oak Lane
- The at grade crossing of the Norfolk-Southern rail line in a tangent section
- Allowing adequate site distance for the tie-in to S. Delphine Avenue
- Minimize the road grade to 5% or less (to minimize access issues during extreme weather)
- Maximize lot sizes for marketing and development (at least one 50 acre lot size)
- Cut through traffic from S. Oak Lane to S. Delphine Avenue

Two transportation alternatives were considered, with the entrance location off of South Delphine Avenue the variable. The two options to be evaluated were the only locations where adequate site distance, 615' to the south and 530' to the north, was available onto South Delphine Avenue, without the requirement of road reconstruction, removing the hump, which limits site intersection distance. The option that was selected, as depicted below in the Development Area Options, was due to the ability to reduce the access road's slope to 5% or less (See Appendix F for Road Profile). The other alternative, locating the access road at the existing peak elevation along South Delphine Avenue would have yielded a steeper access road (7-8%), which was undesirable leading down to the rail crossing from a safety perspective, especially during harsher winter seasons and the potential for slippery roads in extreme weather.

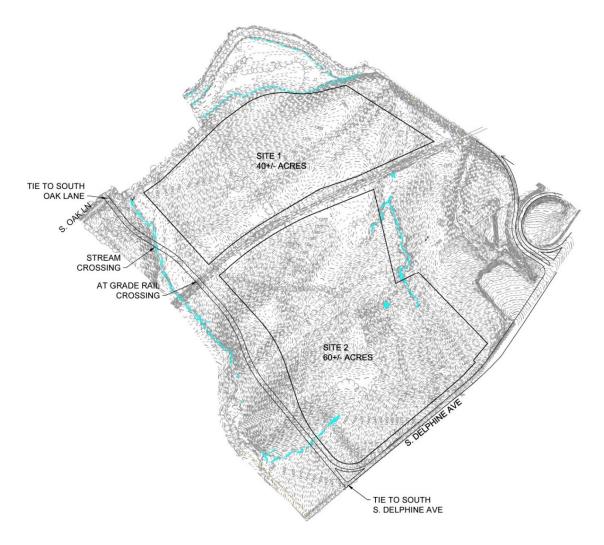


Figure 4: Road Option 1

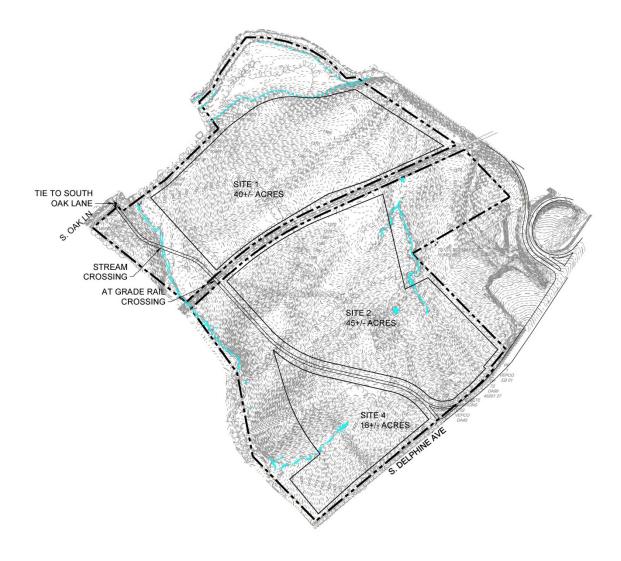


Figure 5: Road Option 2 (Alternative)

Section 2.4 - Rail Alternatives - PGI

Layouts for rail access has been prepared previously by Norfolk Southern. Those rail layouts along with some alternatives were evaluated considering grade and site access. Ultimately, a rail siding was decided to be the basis of the Master Plan, as the location meets a need of the current business community in Polymer Group Inc. (PGI).

PGI is located to the north of the Exit 96 Industrial Park on Shenandoah Village Drive. Currently, PGI uses a site to the north of Interstate 64, where they offload approximately 11 cars weekly. As such the area that was considered needed to be able to accommodate 10-12 rail cars at one time and

also allow for easy access along the rail for offloading, allowing trucks to move through the site without needing to back up.

Throughout the options detailed below, a rail offload site has been shown on either the north or south side of the existing Norfolk Southern rail. While the north side is located on the same side of the existing rail as the existing PGI manufacturing facility, the vicinity to residential units and the potential for noise associated with the offloading, it is recommended that the south end be used to accommodate the rail offload facility.



Figure 6: Existing PGI Offload Facility

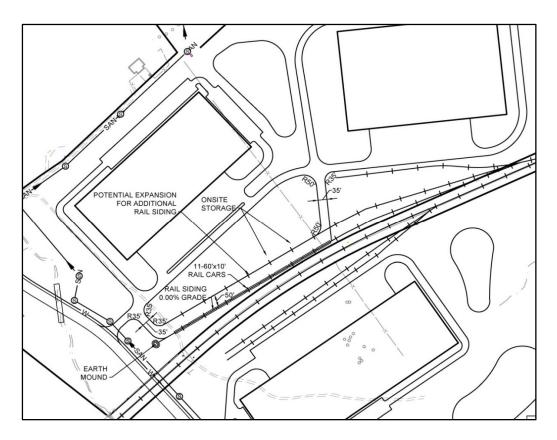


Figure 7: PGI Offload Facility North Siding

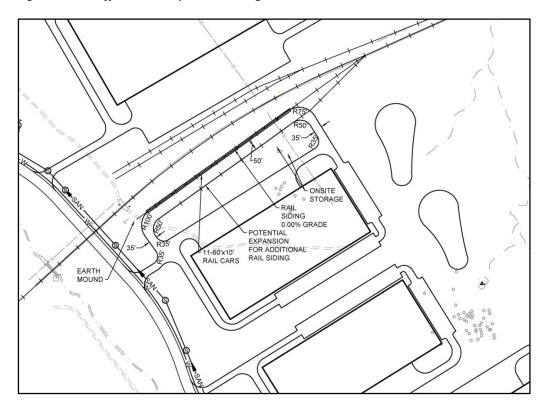


Figure 8: PGI Offload Facility South Siding

Section 2.5 - Development Area

Several options were developed through the Master Planning stage to demonstrate the different options available to the site. Throughout the development the constraints listed above were considered, with each Option below utilizing the same access road, tying to S. Delphine Avenue and South Oak Lane, as well as crossing the Norfolk Southern Rail at the same point.

Phase 1	North of Rail	South of Rail	Build Out Potential	Yield (SF/Acre)
Development Options	Total (SF)	Total (SF)	(SF)	(SF/ACIE)
Option 1	450,000	825,000	1,275,000	7,544
Option 2	450,000	700,000	1,150,000	6,805
Option 3	500,000	700,000	1,200,000	7,701
Option 4	500,000	600,000	1,100,000	6,509
Option 5	500,000	500,000	1,000,000	5,917
Option 6	500,000	1,000,000	1,500,000	8,108

Table 2: Summary Table of Options

Option 1 is comprised of one large scale 500,000 SF facility along S. Delphine Avenue and up to three facilities with rail access: one 200,000 SF user south of the railroad, with one 250,000 SF user to the north of the rail. Additionally, an area to allow for rail offloading has been included on the north side of the rail, with another 200,000 SF user adjacent with access to rail as well. The remaining area is made up of one 25,000 SF and one 50,000 SF flex space users.



Figure 9: Master Plan Option 1

Option 2 features a cluster of 200,000 - 300,000 SF facilities throughout the site. The north end of the rail is similar to Option 1. The south end of the railroad features two 200,000 SF facilities and one 300,000 SF facility aligned with the new road. The three facilities will be tiered following the existing terrain.



Figure 10: Master Plan Option 2

Option 3 includes a single 500,000 SF user on the north end of the rail. This location allows a 500,000 SF building to have direct access to the rail, which is not able to be directly accommodated on the south end of the rail in any of the options, due to grade constraints. The south end of the rail is shown similarly to that of Option 2.



Figure 11: Master Plan Option 3

Option 4 also includes the 500,000 SF user on the north side of the rail, however it also includes more small flex type space with a pair of 100,000 SF buildings. The south side of the rail also features two 200,000 SF buildings, one of which has rail access.



Figure 12: Master Plan Option 4

Option 5 includes a pair of 500,000 SF users, allowing the overall site to be marketed to two large users as large tracts (with the Southern one being greater than 50 acres). Additionally, these two tracts of land will have rail access available to them.



Figure 13: Master Plan Option 5

Option 6 includes a pair of 500,000 SF users, along with one 300,000 SF building and a 250,000 SF building. A 50,000 SF building is also shown. Option 6 is considered a max build out of the site and would include the acquisition of the adjacent 18-acre parcel to the northeast, along S. Delphine Avenue. The additional parcel allows the existing parcel to be developed to the max, as there are some environmental constraints that otherwise close off the northeast tip of the site.



Figure 14: Master Plan Option 6 (includes acquisition of adjacent 18 +/- acres)

Section 2.6 "Pad Ready" Development Costs

As part of this study we evaluated the potential costs to develop "pad ready" sites for each of these options. The costs to develop "pad ready" sites ranged from \$91,790 to \$117,640 per acre, which appears to be consistent for sites located throughout this region. Below is a table summarizing those costs and the costs per developable acres:

	Total Costs for	Developable	Costs per
Grading Option	"Pad Ready" Sites	Acreage	Developable Acres
Option 1	\$11,764,000	100	\$117,640
Option 2	\$10,462,000	100	\$104,620
Option 3	\$10,023,000	100	\$100,230
Option 4	\$9,833,000	100	\$98,330
Option 5	\$9,179,000	100	\$91,790
Option 6	\$13,268,000	120	\$110,567

Given the multiple variations as to how the site can be developed, we do not recommend that the EDA invest in "pad ready" sites at this point in time.

Section 3 - Stormwater Management Concept Plan

The following stormwater management concept plan has been prepared to provide baseline knowledge of existing conditions, governing stormwater regulations, preliminary stormwater compliance requirements for the six site layout options presented, a catalog of appropriate Best Management Practices (BMPs), and stormwater management strategies. The purpose of the following report is to allow developers to effectively budget for design and construction of the ultimate build out of the Exit 96 Industrial Park.

Section 3.1 - Existing Site Conditions Relative to Stormwater Management

Land Cover

The site boundary, which does not include the area where the rail bisects the property, covers approximately 169 acres and is best characterized as managed forest land. Three primary land cover categories are used for determining stormwater compliance: impervious, managed turf, and forest/open space. As shown below in Figure 7, forest/open space occupies 71.5% (121 acres) of the site, while open space accounts for the remaining 28.5%. The rail, which qualifies as impervious is not included in the parcel's site boundary. Note: wetlands are considered forested land cover, and open water is considered impervious cover.



Figure 15: Existing Land Cover

Soils

The Soil Survey, as prepared by the USDA Natural Resource Conservation Service, was used to determine the existing soils at the site as BMP design and performance is highly contingent upon hydrologic soil group (HSG). Soil survey maps are useful in providing a baseline of expected soil characteristics across large scale areas to help with planning for suitability of potential BMPs, but more detailed geotechnical soil borings will be required prior to the design stage to more accurately characterize local water tables, soil porosity, and infiltration rates. Illustrated below in Figure 8 is a map of soil survey data by hydrologic soil group for the site, where B soils contain a mixture of gravelly sand and clay with a moderate infiltration rate, and C soils contain a mixture of sand, clay, and loam and have low infiltration rates. The site is primarily covered by B soils (approximately 94 acres or 55.6%), with C soils comprising 47 acres or 27.8%. Type A soils and D soils make up the remaining 9 and 19 acres, respectively.

During the due diligence period and geotechnical investigation it was determined that there were areas that karst could be present. Further testing better identified these areas (report included in Appendix O, P, and Q) and generated recommendations that no infiltrating BMPs be used. Additionally, any ponds or surface stormwater storage or conveyance (i.e. ditches or swales) should feature clay liners to disallow the ability for infiltration.



Figure 16: Map of Soil Survey Data by Hydrologic Soil Group

Topography

Existing drainage patterns at the site are illustrated below in Figure 9 – Topography Map. Several drainage divides traverse the site, dividing runoff from the site into four major outfalls. In general the southern portion of the site is split, with approximately 41 acres draining to the west, with the remaining 62 acres drainage to the east. Both southern drainage areas cross the railroad tracks through a culvert entering and exiting through a defined channel. The northern portion of the site is also split into two main drainage areas, where the area draining to the west comprised of approximately 23 acres and the area that drains to the east made up of 43 acres. Unlike the southern area, the northern end is not collected in a concentrated manner (such as a ditch or swale) until discharging directly into the South River, with the majority of the site sheet flowing towards it.

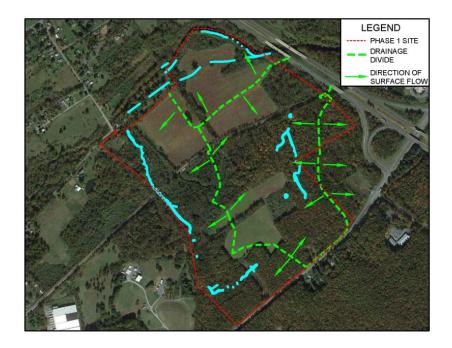


Figure 17: Drainage Map

Section 3.2 - Governing Stormwater Management Regulations

In July of 2014 Stormwater Management guidelines shifted, changing policies for meeting the criteria for improvements to water quality and water quantity on site. Based on the Virginia Stormwater Management Program Regulations, 9VAC25-870-48-B the Exit 96 Industrial Park qualifies for grandfathering under the previous set of stormwater regulations. This is due to an obligation of locality funding prior to 2012, which includes the following:

- Property Appraisal (April 2010)
- Preliminary Site Evaluation (June 2010)
- Second Property Appraisal (Spring 2011)
- Geotechnical Report and Phase I Evaluations (October 2011)
- Purchase Agreement (July 2011)
- Bond Closing and Purchase Closing (Late 2011 January 2012)

In addition to the above, in accordance with 9VAC25-870-48-D, the City of Waynesboro issued municipal bonds for the project, which were then sold to SunTrust for sale to public investors, thereby qualifying as public debt financing.

Based on the above, the Exit 96 Industrial Project was requested to be grandfathered, in perpetuity, and subject to stormwater technical criteria Part IIC. This was accepted by both the City of Waynesboro's City Engineer and City Attorney (See Appendix H).

Water Quality Criteria

Water quality compliance for this Stormwater Management Concept Plan is demonstrated using the Part IIC criteria and the Simple Method formula, as defined in the Virginia Stormwater Management Handbook. Under the Part IIC criteria, regional stormwater management methods were encouraged and have been considered as a part of this plan.

Water Quantity Criteria

Water quantity compliance is developed around preventing stream channel erosion and flooding. Concentrated stormwater flow from development sites must be reduced to predeveloped flow rates for both the 2 and the 10 year storms. Channel adequacy is to be identified to where the subject parcel is less than 1% of the contributing drainage area, which for the subject site will be at the South River. For the areas north of the rail it may be feasible to discharge directly to the South River. Other areas will need to analyze the natural channels that are used for stormwater discharge from any particular site.

Section 3.3 - Preliminary Stormwater Compliance Requirements

Proposed drainage patterns for Options 1-6 were determined based on the proposed location of building pads as they correspond to existing drainage patterns. Proposed grades were also taken into account, as to understand how the site will drain once built out.

The six layout options were analyzed to determine the preliminary and post development phosphorus load generated and the removal requirement for the new development, prior to determining appropriate BMPs and stormwater management strategies, as presented below in Table 2, with additional analysis available in Appendix J.

		Option 1	Option 2	Option 3	Option 4	Option 5	Option 6
Land Cover	Total Area (Acres)	169	169	169	169	169	187
	Impervious Post Developed						
	Surface Area (Acres)	57.70	56.05	54.35	54.10	51.80	68.80
Pre Developed Phosphorous Load (lb/yr)		74.75	74.75	74.75	74.75	74.75	82.71
Post Developed Phosphorous Load (lb/yr)		137.65	134.26	130.79	130.27	126.07	162.49
Load Reduction Required, (lb/yr)		62.90	59.51	56.04	55.52	51.32	79.78

Table 3: Preliminary Pollutant Load Reduction Summary

The phosphorous load reduction requirement ranged from 51 lb/yr to 63 lb/yr, with the max build out jumping up to approximately 80 pounds per year. Option 5 results in a greater preservation of forest/open space and less overall impervious area. Therefore, based on these preliminary site layouts, Option 5 will require less stormwater management to meet water quality requirements for the site.

Section 3.4 - Recommended Site Specific BMPs

Recommendation of specific BMPs for the development of the Exit 96 Industrial Site has two main considerations. The most important is that there is potential for karst to be present, which does not support any type of infiltration, directly or indirectly, such as a pond. Secondly, it is important that, due to the high rate of concentrated impervious area on the site, that BMPs be selected that provide high removal rates of phosphorous.

With those two criteria in mind, there are four BMPs that should be targeted, Sand Filters, Retention Basins, Biofilters or Proprietary BMPs. Each of these options has the ability to obtain a phosphorous removal rate of 65%, which under the Part IIC criteria is the maximum achievable.

It is imperative that if either a Biofilter or Retention Basin is selected that a double clay membrane be installed on the bottom, to prevent infiltration from occurring, based on the karst noted in Section 3.1 above.

Retention Basins

Retention Basins (wet pond) can better be described as a wet pond. As such, on this particular site, a double clay membrane would be recommended, due to the potential for karst in the region. Retention basins can not only serve as a means to acquire water quality credit, but can also be used to establish additional storage for reducing the flow off the site. Typically, safety benches and aquatic benches are associated with this BMP to gain the highest level of phosphorous removal (65%). The aquatic benches should be planted with species that are able to withstand 12" – 18" of water on a consistent basis.

Bioretention

Bioretention basins and/or cells are desirable BMPs due to their versatility in spatial configuration and exceptional phosphorous removal efficiency (65%). One limitation is the amount of area that is practical to drain to it, however smaller, more localized Bioretention basins can be a good solution. Bioretention cells require forebays in addition to a planting plan. Additionally, as noted in the section, regarding Retention Basins, the bottom of any bioretention should include a double membrane of clay to reduce infiltration.

Sand Filters

Sand Filters can be designed to gain up to 65% removal, same as Bioretention and Retention Basins. For this site, the application of sand filters would best be used by running the stormwater through a sand filled structure, such as a pipe, where the sand acts as a filter to pull the phosphorous out. Because of the debris that can get caught up in the sand filter, maintenance and observation is important and should be scheduled regularly.

Proprietary BMPs

Proprietary BMPs can be an effective measure to reduce the pollutant loads where other methods will not work. Proprietary BMPs, such as StormFilters®, utilize a replaceable cartridge system to filter pollutants from runoff from impervious areas. Such systems are housed in an underground vault and require routine maintenance to clean/replace the filter cartridges and remove debris. These BMPs are ideally suited for urban areas where available surface area is sparse; however they also can be quite affective where large areas to grade out surface BMPs (i.e. bioretention or ponds) are not practicable.

Section 3.5 - Stormwater Management Strategies

While on a site by site basis the above mentioned site specific BMPs can work, it is typically most cost effective to use a Retention Basin (see Appendix I for specifications), to both address water quality requirements and reduce the flow off of the site. Since the overall site is subject to the Part IIC stormwater requirements, it is recommended that regional basins be considered for both the north and south side of the railroad. While a regional stormwater strategy is recommended, it is not recommended that any basins be built prior to being needed, to remain flexible in placement and design for each individual user.

Two basins should be considered for the installation of the road, as water quality requirements will need to be addressed. A reduction of the flow will not be required to be addressed, as road projects are not require to adhere to this requirement.

The northern section of the site can be accommodated in a single pond, if the ultimate build out allows for a single user or multiple users to share in a joint outfall location. The single pond to accommodate the needs of Options 1-6 on the north end of the site would require a wet volume of 149,000 – 151,000 cubic feet of wet storage, to provide water quality treatment, while dry storage would require approximately 375,000 cubic feet of dry storage. Overall the pond would need to be excavated to accommodate 526,000 cubic feet (19,500 cubic yards) which to excavate would cost approximately \$156,000. Factoring in landscaping for the acquatic bench, riser and outfall pipes, and emergency outfall the total cost would be approximately \$250,000. If the ponds were developed separately the cost would increase. Preliminary routing calculations for the site are provided in Appendix I.

The southern section of the Exit 96 Industrial Park could potentially be developed to accommodate sharing a single stormwater pond. If that was the case Options 1 – 5 would require 245,000 – 270,000 cubic feet of wet storage, to provide water quality treatment, while an additional 850,000 cubic feet would be required to detain and reduce the outflow from the site. Overall the southern pond would need to be excavated 1,120,000 cubic feet (41,500 cubic yards), accommodating all post developed drainage towards the northeast, which to excavate would cost approximately \$380,000. Factoring in the additional items noted above for the northern pond, the total stormwater treatment and detention costs would range from \$500,000 - \$575,000.

However, due to the grades from S. Delphine Avenue to the rail, a shared facility for the south end of the development may not be feasible, depending on the users that decide to locate here. Based on the Master Plan layouts, there are approximately three levels across the south end of the Exit 96 Industrial Park and it is feasible that each level would need its own pond. Each additional stormwater management pond will raise the overall installation cost \$50,000 - \$100,000, to accommodate additional freeboard (space between peak water elevation and top of dam), forebays, outlet protection and control structures.

Section 4 - Environmental Mitigation Plan

The site drains to its north, making use of some onsite channels, ultimately draining to the South River, which serves as a portion of the northern boundary for the site. The South River is joined by Back Creek, just upstream of the Exit 96 Industrial Park along with other tributaries. Additionally, the River is stocked with trout and other fish downstream at Ridgeview Park.

Section 4.1 - Existing Wetlands and Streams Assessment

EEE Consulting Inc. performed a preliminary wetlands assessment for the subject area in 2012. The Waters of the U.S. provided in Table 7 and depicted in Figure 11 are subject to change and have not been confirmed by the U.S. Army Corps of Engineers (COE). The wetland delineation will need to be confirmed by the U.S. Army Corps of Engineers prior to applying for any clean water act permits.

Total Acreage	Wetlands	Streams & Ditches
(AC)	(AC)	(LF)
169 <u>+</u>	0.057 <u>+</u>	4,247 <u>+</u>

Table 4: Estimated Wetlands and Streams for Subject Areas



Figure 18: Stream and Wetland Mapping

Section 4.2 - Wetland and Stream Impacts

Each buildable area alternative was evaluated for stream and wetland impacts, along with the associated amount of mitigation, if required, based on permit thresholds. Wetlands were assumed to be mitigated at a ratio of 2:1 for all wetlands (forested, scrub shrub and emergent). Mitigation ratios for stream channel impacts will need to be determined by evaluating the stream impact sites using the Unified Stream Methodology (USM) protocol to access the quality of each stream at the impact locations. Stream channel impacts were assumed to be mitigated for at a ratio of 1.25 credits per linear foot for this study.

Wetland and streams can be impacted up to a tenth of an acre and 300 linear feet of stream under a General Permit prior to mitigation being required. It is assumed that due to the connective nature of the access road, adjoining South Oak Lane to S. Delphine Avenue, that this portion of construction will have independent utility and be seen as a stand-alone project from the rest of the development, with any road impacts to wetlands and streams not counting against the Exit 96 Industrial Park's impact accumulation. In the event that the impacts associated with the site's development are tied to the road impacts, then mitigation could be required for the site's impacts, as the impacts will be seen as cumulative and would exceed the mitigation thresholds.

The information provided below in Table 8 demonstrates that the 300 linear foot stream impacts or one-tenth of one acre wetland impacts are not exceeded, therefore for cost estimating purposes, no mitigation costs have been assumed. Stream impacts for the road installation total 288 linear feet. The minimum impacts for site development for wetlands are 0.03 acres (AC) and corresponding stream impacts are 199 linear feet (LF). The maximum wetland impacts are 0.03 acres (AC) and corresponding stream impacts are 299 linear feet (LF).

Development Phase	Wetland Impacts (AC)	Stream Impacts (LF)	Wetland Mitigation Required (AC)	Stream Mitigation Required (LF)
Access Road	0	288	0	0
Option 1	0.03	294	0	0
Option 2	0.03	199	0	0
Option 3	0.03	199	0	0
Option 4	0.03	262	0	0
Option 5	0.03	260	0	0
Option 6	0.03	299	0	0

Table 5: Estimated Wetland and Stream Impacts and Mitigation Requirements

Section 4.3 - Wetland and Stream Mitigation Options

There are several courses of action for the mitigation of unavoidable impacts to wetlands and Waters of the U.S.: construction of a Permittee Responsible Mitigation (PRM) Bank, construction of a commercial mitigation bank, purchase credits from an existing stream and/or wetland mitigation bank or payment into the in-lieu fee fund. However, due to the small amount of overall impacts on this site and the possibility that mitigation requirements may be able to be avoided, it is recommended that the only option considered be to purchase credits.

Purchasing credits from an existing mitigation bank is subject to market conditions. As of July 2015, stream credits were available for \$450 per credit within the watershed, from the Shenandoah Wetland and Stream Bank.

Section 5 - Water and Wastewater Infrastructure Plan

Assessment of a potential industrial development site, such as the Exit 96 Industrial Park, requires evaluation of existing and potential wastewater and potable water infrastructure. The proceeding content provides information about existing water and wastewater infrastructure as well as proposed water and wastewater infrastructure expansion and improvements that would be required for the development.

Overall, the information contained in the content below is for discussion and planning purposes only.

Section 5.1 - Potential Water Demands

Two main water infrastructure components are required for any proposed development lacking onsite potable water: a water source and a water distribution system. Before evaluating existing utilities and determining the extent of water infrastructure expansion required to serve a new development, an estimate of the overall future water demand must be developed.

Currently, the future water demand associated with the site is an unknown. Therefore, a conservative estimate must be developed based on conventional water service demands for similar development scenarios. In this case, an average demand of 2,000 gallons per day per acre developed is a reasonable assumption. With about 110 - 120 acres of developable land comprising the Exit 96 Industrial Park, the total estimated water demand may be 250,000 gallons per day. It should be noted that a large water user could use between 500,000 - 1,000,000 MGD alone.

Section 5.2 - Existing Water Infrastructure

The site would require potable water distribution throughout to serve the needs of each user. Currently, both the City and Augusta County Service Authority (ASA) have systems within the vicinity of the property, with the ASA owning a 12 inch waterline located in S. Delphine Avenue, while the closest City owned line is a 16 inch waterline located along Lyndhurst Road, north of the proposed Industrial Park. The City's line has been determined to be a more feasible option, due to capacity issues, currently being experienced by the ASA. The City's line is located in the Maupintown Tank pressure zone and is supplied by the City's 6.0 million gallons per day (MGD) water treatment plant (WTP), which is currently operating around 42% of its capacity, with an average daily flow of 2.52 MGD.

Section 5.3 - Existing Wastewater Infrastructure

The Waynesboro Wastewater Treatment Plant is currently sized to accommodate up to 6.0 MGD, however it is currently only operating at 42% of that, giving it the capacity to treat any additional flow that could be associated with the Exit 96 Industrial Park, including a large water user that could demand up to 1.0 MGD on its own. The nearest connection point to the City's sanitary sewer is near the intersection of North Oak Lane and Lyndhurst Road. The 8" line that is situated beyond the northwest corner of the site could be accessed by way of a pump station for the Exit 96 Industrial Park. The pump station would be sized to accommodate both the industrial park and the nearby residential areas. Currently, a plan is underway to take sewer and pump it across the South River up to its location in Lyndhurst Road.

Section 5.4 - Potential Water Infrastructure Improvements

A new elevated water storage tank, located along South Delphine Avenue, will likely be required to provide adequate fire suppression flow and pressure. At the normally require rate of 2,000 gallons per minute over a two-hour duration, the tank will need to be a minimum 240,000 gallons, therefore it is recommended a 500,000 gallon elevated storage tank be constructed. Additionally, the water tank will allow for the marketing of the park, as it also can serve as a giant billboard or welcome sign.

Beyond the new main and the elevated storage tank described above, ancillary water infrastructure components like fire hydrants, gate valves, and air release valves will also be required to ensure adequate water system operation. Overall, about 4,660 ft. of 12" water line will likely be required to expand and improve the onsite water infrastructure, respectively.

Section 5.5- Potential Wastewater Infrastructure Improvements

Similar to the waterline infrastructure, no sanitary collection system exists within the proposed industrial park property. Therefore, construction of a new collection system will be required in order to accommodate the industrial/commercial flows associated with the park's build out. The potential peak demand for the wastewater system, using a peaking factor of 2.5, would be 250,000 x 2.5 for 625,000 gallons per day system design.

As noted above, in Section 5.2, there are plans, currently under design to pump sewer up to Lyndhurst Road, across the South River. On site, an additional 4,500 linear feet of 10" gravity sewer line will be required to allow the site to gravity flow to the proposed pump station.

Section 5.6 - Water and Wastewater Infrastructure Plan Summary

During evaluation of a potential industrial site, future water and sewer demands must be assessed in order to adequately plan and budget for additional infrastructure. As described above, while infrastructure upgrades and expansions would be required to serve the future tenants of the Exit 96 Industrial Park, much of the local existing water and sewer infrastructure may be potentially utilized if demand increases, thereby limiting the extent of new construction in the form of treatment plants, infrastructure, water sources, etc.

In general, a new collection and distribution system within the park (along with the pump station and elevated water tank) will be necessary to accommodate the full site's build out. Additionally, it is worth noting that coordination with Norfolk Southern will be necessary to obtain permits for boring the utilities underneath the rail.

As such, we determined the following infrastructure would be required for the park with associated costs (Cost Estimates can be found in Appendix L):

- 10" gravity sewer along the proposed connector road to a potential pump station location along the South River at a cost of approximately \$595,000;
- 500,000 gallon elevated storage tank along S. Delphine Avenue and a water booster pump station at a cost of approximately \$2,152,000 and \$317,000, respectively;
- 12" waterline along S. Delphine Avenue along the proposed connector road to S. Oak Lane at approximately \$595,000.

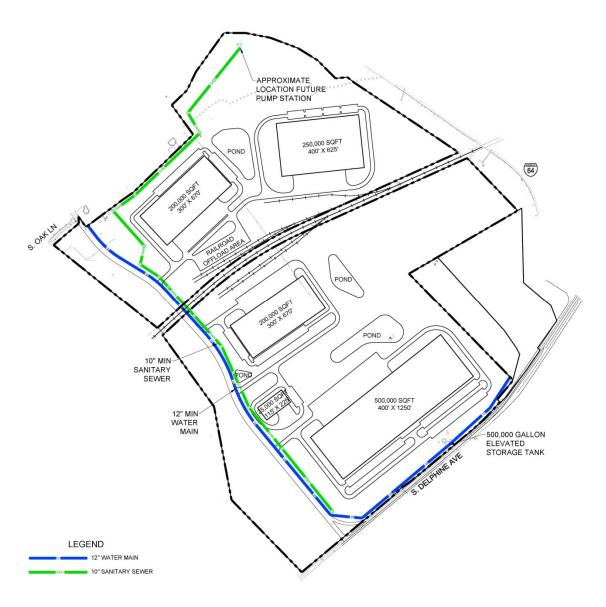


Figure 19: Potential Water and Wastewater Utilities Layout

Section 6 - Transportation Improvements

The Exit 96 Industrial Park is bound on the east by S. Delphine Avenue and Interstate 64 to its north. The northwest corner intersects with South Oak Lane. Additionally, the site is bisected by a Norfolk Southern rail line. The transportation network supporting the Park will require two primary entrances (S. Delphine Avenue and South Oak Lane), and a new at-grade railroad crossing. In addition to identifying the required access locations, additional criteria has been considered, including horizontal and vertical alignment of the spine road connecting these intersections, intersection sight distance and full access spacing requirements from the Exit 96 ramp.

The assessment of the transportation network will require three intersection points (S. Delphine Avenue, South Oak Lane and the crossing at the rail line). In addition to connectivity at each of these points, additional criteria has been considered, including proposed slopes, intersection sight distance and full access spacing requirements from the Exit 96 ramp.

This assessment of the transportation network in the vicinity of the Exit 96 Industrial Park considers the potential land uses and associated square footages in order to estimate the commercial and personal vehicle trips generated by the development, as the road will connect traffic from South Oak Lane, Lyndhurst Road and potentially Shenandoah Village Drive (Appendix Q). The 2014 annual average daily traffic (AADT) volumes published by the Virginia Department of Transportation (VDOT) were reviewed in the context of this work (see Appendix E); peak hour directional turning movement traffic counts were not collected at nearby intersections.

Traffic operations at intersections and along highway segments were considered but not analyzed since peak hour traffic counts were not available.

It is important to note that a detailed traffic engineering analysis, including highway capacity analyses of critical intersections, should be conducted to determine the extent of the necessary roadway improvements.

Section 6.1 - Anticipated Land Uses

The ultimate development of the Exit 96 Industrial Park is anticipated to occur in a single phase. The size of the development is anticipated to be between 1.175 million and 1.225 million square feet based on the Master Plan layouts above (See Figures 1-5)

A mix of industrial land uses were considered for the development. The land use categories were determined using the Institute of Transportation Engineers' (ITE's) *Trip Generation Manual* 9th Edition (2012). The land uses considered for the site are listed below along with the corresponding description from the *Trip Generation Manual*:

- General Light Industrial (ITE land use code 110) "Light industrial facilities are free standing facilities devoted to a single use. The facilities have an emphasis on activities other than manufacturing and typically have minimal office space. Typical light industrial activities include printing, material testing, and assembly of data processing equipment."
- Manufacturing (ITE land use code 140) "Manufacturing facilities are areas where the
 primary activity is the conversion of raw materials or parts into finished products. Size and
 type of activity may vary substantially from one facility to another. In addition to the actual
 production of goods, manufacturing facilities generally also have office, warehouse,
 research, and associated functions."
- Warehousing (ITE land use code 150) "Warehouses are primarily devoted to the storage of materials, but they may also include office and maintenance areas."
- High-Cube Warehouse/Distribution Center (ITE land use code 152) "High-cube warehouses/ distribution centers are used for the storage of materials, goods and merchandise prior to their distribution to retail outlets, distribution centers or other warehouses. These facilities are typically characterized by ceiling heights of at least 24 feet with small employment counts due to a high level of mechanization. High-cube warehouses/distribution centers generally consist of large steel or masonry shell buildings and may be occupied by single or multiple tenants. A small ancillary office use component may be included and some limited assembly and repackaging may occur within these facilities. High-cube warehouses/distribution centers may be located in industrial parks or be free-standing."

Section 6.2 - Site Access Alternatives

While entrance to the site can be accommodated by both South Oak Lane and S. Delphine Avenue, the majority of the industrial park's destination traffic will enter from S. Delphine Avenue, due to its vicinity to Interstate 64. Along S. Delphine Avenue two alternatives were studied for site access. They are as follows:

1. Alternative 1 entrance was located just off the crest vertical curve of S. Delphine Avenue. This locations provided adequate intersection site distance as well as full access spacing from the Interstate 64 off ramp. With consideration to fixed tie-in points of S. Delphine and the at-grade railroad crossing, the access road extending from this entrance required a maximum longitudinal slope of 8% and is able to be developed on both sides of the road. This alternative was not selected due to potential issues with high heavy truck utilization on an 8% grade.

2. Alternative 2 assumes the site's primary entrance is located off S. Delphine Avenue on the southern end of the site, which is also at a location that provides adequate intersection site distance and full access spacing from I-64 ramps. Other benefits of this access point are that the road's slope is able to be held at or below 5% throughout and the majority of the site's development occurs on one side, giving the appearance that you are driving by the industrial park, versus driving through it.

Section 6.3 - VDOT Minimum Spacing Standards

Timmons Group conducted a review of the adjacent roadways and how the industrial park access road will interact with those roads. South Delphine Avenue and South Oak Lane are classified by VDOT as a Major Urban Collector Road and Minor Urban Collector Road, respectively. These classifications allow us to understand the minimum spacing standards required by VDOT.

VDOT tables were used to verify that adequate sight distance is provided. Site distance was analyzed for a 4 lane divided, 45 mph highway. While the road is not yet a 4 lane divided road, in the immediate vicinity of the access road, the future traffic counts could eventually exceed 10,000 trips per day, which then may warrant the need for widening of the South Delphine Avenue. As such the minimum intersection site distance to the right (south) is 615', while the required intersection site distance to the left (north) is 530'. Site distance is measured with an eye and object height of 42 inches, simulating the seated eye height in a vehicle and passenger vehicle in the roadway.

Additionally, Table 5 and Figure 11 from Appendix F of the VDOT Road Design Manual shown below summarize VDOT's minimum spacing standards for full access spacing from an interchange off ramp. At 1,320' we are able to accommodate both entrance options that were described in Section 6.1.

Spacing Dimension							
X	Y M						
750'	1320'	990'					

Table 6: Minimum Spacing Standards for Commercial Entrances and Intersections Near Interchange Areas on Multilane Crossroads VDOT Road Design Manual, Appendix F, Table 2-3

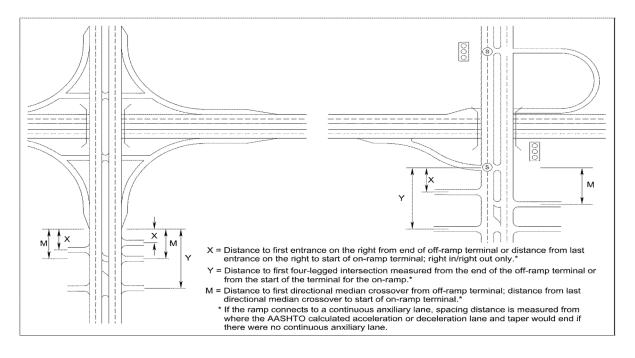


Figure 20: Access Control on Multi Lane Highways at Interchanges VDOT Road Design Manual, Appendix F, Figure 2-9

It is anticipated that the connection to South Oak Lane with operation as a stop controlled intersection with right turn slip ramp (with stop control) to accommodate large heavy vehicle turning movements into the park.

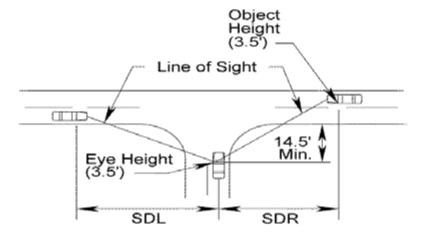


Figure 21: VDOT Sight Distance Standards

SDR = Sight Distance Right (For a vehicle making a left turn)

SDL = Sight Distance Left (For a vehicle making a right or left turn)

Height of Eye 3.5' Height of Object								3.5'				
Design Speed (mph)*	*	20	25	30	35	40	45	50	55	60	65	70
SDL=SDR: 2 Lane Major Road		225	280	335	390	445	500	555	610	665	720	775
SDR: 4 Lane Major Road (Undivided) or 3 Lane		250	315	375	440	500	565	625	690	750	815	875
SDL: 4 Lane Major Road (Undivided) or 3 Lane		240	295	355	415	475	530	590	650	710	765	825
SDR: 4 Lane Major Road (Divided – 18' Median)		275	340	410	480	545	615	680	750	820	885	955
SDL: 4 Lane Major Road (Divided – 18' Median)	Feet	240	295	355	415	475	530	590	650	710	765	825
SDR: 5 Lane Major Road (continuous two-way turn- lane)	In Fe	265	335	400	465	530	600	665	730	800	860	930
SDL: 5 Lane Major Road (continuous two-way turn- lane)		250	315	375	440	500	565	625	690	750	815	875
SDR: 6 Lane Major Road (Divided – 18' Median)		290	360	430	505	575	645	720	790	860	935	1005
SDL: 6 Lane Major Road (Divided – 18' Median)		250	315	375	440	500	565	625	690	750	815	875
SDL: (Where left turns are physically restricted)		210	260	310	365	415	465	515	566	620	670	725

TABLE 2-7 INTERSECTION SIGHT DISTANCE

Source: AASHTO Green Book, Chapter 9, Section 9.5.3, page 9-37 thru 9-52, * Table 9-5 thru 9-14

Table 7: VDOT's Minimum Spacing Standards for Commercial Entrances, Intersections, and Crossovers¹

Section 6.4 - Site Traffic Volume Estimates

Three (3) development scenarios were considered for the purpose of estimating potential site traffic:

^{**}For all tables, use design speed if available, if not use legal speed.

¹ Source: VDOT's *Road Design Manual* Appendix F "Access Management Design Standards for Entrances and Intersections" (July 2013) Table 2-2 (page F-23).

- <u>Scenario 1</u> assumes a General Light Industrial Land Use, where the buildings are focused
 on activities other than manufacturing and minimal office space is provided. This could
 include industries such as printing, material testing or the assembly of data processing
 equipment.
- Scenario 2 assumes development as an Industrial Park. Under this scenario, a mix of manufacturing, service and warehouse type facilities are included. This could either be highly diversified mix of smaller businesses or a limited number of larger industries.
- <u>Scenario 3</u> is similar to Scenario 2, however, an assumed development mix consisting of equal square footages of General Light Industrial, Manufacturing, Warehousing and High-Cube Warehousing/Distribution Center was considered.

Site traffic for each of the aforementioned three (3) scenarios was calculated using the of the Institute of Traffic Engineers (ITE) *Trip Generation Manual*, 9th edition and square footage was used as the independent variable.

Based on the available land and the preliminary layouts, it was determined that between 1,000,000 and 1,225,000 SF of development can be accommodated on the site. That being noted, an assumed build out of 1,225,000 SF was used for each scenario to provide a conservative and consistent basis for comparison.

The trip generation estimates for Scenarios 1, 2, and 3 are summarized in Table 12. The average daily trips (ADT) were estimated along with the AM and PM peak hour trips entering and exiting the site. The provided AM and PM peak hour trips are assumed to coincide with the typical morning and evening peak periods on the adjacent roadway network; 7:00 to 9:00 AM and 4:00 to 6:00 PM.

		WEEKDAY						
	AMOUNT		AM PEAK HOUR			PM PEAK HOUR		
LAND USE	(GROSS SF)	ADT	IN	OUT	TOTAL	IN	OUT	TOTAL
SCENARIO 1								
General Light Industral	1,225,000	8,538	1,193	163	1,356	191	1,403	1,594
SCENARIO 2								
Industrial Park	1,225,000	6,791	561	123	684	207	779	986
SCENARIO 3								
General Light Industral	325,000	2,265	259	35	294	37	271	307
Manufacturing	300,000	1,143	171	48	219	78	140	218
Warehousing	300,000	1,068	119	32	151	30	90	120
High-Cube Warehouse/Distribution Center	300,000	504	11	5	16	11	24	35
	Total	4,981	561	120	681	156	525	681

Table 8: Trip Generation Estimates Summary²

As indicated in Table 7, Scenarios 1 and 2 generate more traffic than Scenario 3 and will therefore have more of an impact on the proposed road and existing adjacent roadway network. Scenario 1 generates significantly more daily, AM peak hour, and PM peak hour trips than Scenarios 2 and 3.

Scenario 3 generates the fewest number of trips with respect to ADT and both peak hours. This is due primarily to the inclusion of lower traffic-generating uses such as Distribution Center and Warehousing. Ultimately this places site-generated traffic for Scenario at just under 5,000 ADT.

Overall, the industrial and manufacturing uses typically generate more employee trips during the AM and PM peak hours. Also, warehousing and distribution center uses typically generate a higher percentage of commercial vehicle trips outside the AM and PM peaks that are distributed over the course of the day, resulting in fewer trips during the critical AM and PM peak hours.

The estimated traffic volumes from Table 12 were assigned to the surrounding roadway network based on available 2014 AADT volumes from VDOT's website and the layout of the proposed site, including the anticipated connector between Shenandoah Village Drive and South Oak Lane (ultimately connecting to the Industrial Park's access road).

The potential site traffic volumes on the surrounding roadway network were calculated for each of the three development scenarios. Potential improvements to the existing roadway network were then identified based on the highest site traffic volumes.

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² Source: Institute of Transportation Engineers' *Trip Generation Manual* 9th Edition (2012)

Section 6.5 - VDOT Road Design Standards

The VDOT road design standards are found in Appendix A of VDOT's *Road Design Manual*. The geometric design standards are determined based on the functional classification of the roadway and the design year ADT volumes. Based on the connection of South Oak Lane to South Delphine Avenue, and the existing road classifications, Minor and Major Collector roadways, respectively, it is reasonable to assume the new access road would be classified as a Major Urban Collector road.

VDOT's geometric design standards for an Urban Collector Road (GS-7) with an ADT volume over 2,000 vpd with 5% or more of truck usage require the following:

- A minimum lane width of 12 feet;
- 4-foot wide paved shoulders;
- A minimum graded shoulder width of 8 feet (including the paved shoulder).

Assuming a 12-foot lane with an 8-foot shoulder the roadways listed above will require a pavement and shoulder section of 40' (8' + 12' + 12' + 8' = 40'). Ditches will be provided on both sides of the roadway to collect run-off and will be located within VDOT's right-of-way. With the additional width needed to accommodate expected ditches and an additional 60' utility corridor, the necessary right-of-way for this roadway section would fall between 120' and 140'.

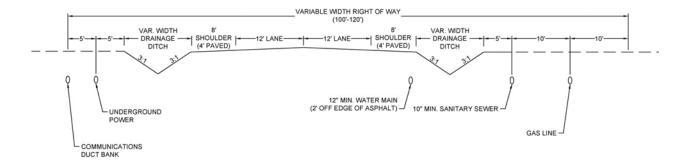


Figure 22: Typical Road Right of Way Section

Section 6.6 – Potential Transportation Network Improvements

The existing roadway network was assessed based on the existing 2014 AADT volumes published by VDOT and the anticipated amount daily site trips that will be added to the network. Traffic

operations at intersections, ramp junctions, and along two-lane highway segments were not analyzed since peak hour traffic counts were not available.

Based on the 2014 AADT volumes from VDOT and the projected daily site trips, the existing transportation network appears to be generally acceptable for increased traffic usage.

Given the location of the proposed road and adjacent uses, it is anticipated that a majority of traffic will enter from and exit toward the I-64 Interchange. The southbound right movement will likely exceed 40 vehicles per hour (and also consist of heavy vehicles). This will warrant the installation of a southbound right turn lane. A northbound left turn lane is also recommended, given the traffic potential a northbound auxiliary lane is recommended to increase safety and preserve the carrying capacity of South Delphine Avenue.

As the industrial park gets built out the access along South Delphine Avenue may need to be upgraded, extending the four lane portion of the road down to the Exit 96 Industrial Park's entrance.

Section 6.7 - Planning-Level Opinion of Probable Costs

Total estimated costs for the road as proposed is approximately \$5,138,000. The EDA has indicated they would like a nice landscaped entrance as well as entrance sign such that clients have a strong "sense of arrival", so for the purposes of this report, we recommend a budget of \$200,000 to \$300,000 for signage and a landscaped entrance. Since the City is going to potentially allow for an asphalt walkway in lieu of the curb & gutter and sidewalk along the connector road, we recommend a budget of \$60,000 (3,000 LF x \$20 per LF for a 5' wide asphalt walkway).

Section 7 - Summary of Overall Project Costs

Following is a summary table identifying the potential high and low range for total development costs to achieve a Tier 4 and Tier 5 site status level:

Exit 96 - Summary of Total Costs	Low	High		
Infrastructure Costs (Tier 4)				
Connector Road (VDOT Standard)	\$5,138,000	\$5,138,000		
Landscaped Entrance / Signage	\$200,000	\$300,000		
5' Asphalt Walkway Along Road (3,000 LF x \$20/LF)	\$60,000	\$60,000		
5' Asphalt Fitness Trail in Park (7,500 LF x \$20/LF)	\$150,000	\$150,000		
10" Gravity Sewer	\$595,000	\$595,000		
500,000 Gal Elevated Storage Tank	\$2,152,000	\$2,152,000		
Water Booster Pump Station	\$317,000	\$317,000		
12" Water Line	\$665,000	\$665,000		
Total Infrastructure Costs (Tier 4 Status)	\$9,277,000	\$9,377,000		
Total "Pad Ready" Site Development Costs (Tier 5)	\$9,179,000	\$11,764,000		
Total Potential Development Costs (Tier 5 Status)	\$18,456,000	\$21,141,000		

Section 8 - Conclusions and Recommendations

Given our understanding that the City desires to achieve a Tier 4 or Tier 5 status for the site, we have drawn the following conclusions and are making the following recommendations:

- 1. The Master Planning process revealed that we could achieve a 40 acre rail served site on the north side of the rail and a 60 acre rail served site on the south side of the rail, thereby giving the City / EDA a very marketable product.
- 2. We believe it is in the best interest of the City / EDA to install the necessary infrastructure (Roads, Water & Sewer) for the park to develop a Tier 4 site.
- 3. Given the multiple scenarios under which the sites can be developed and potential costs for achieving a "pad ready" or Tier 5 site, we do not believe it is worth the City / EDA making this investment as we believe it will reduce the overall flexibility with the site from a marketing perspective.

Waynesboro EDA – Exit 96 Preliminary Engineering Report & Master Plan September 2015

Appendices

A – Master Plan Layouts

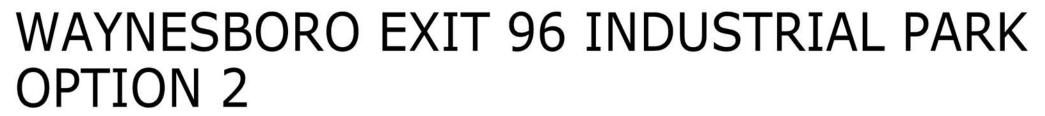


WAYNESBORO EXIT 96 INDUSTRIAL PARK OPTION 1



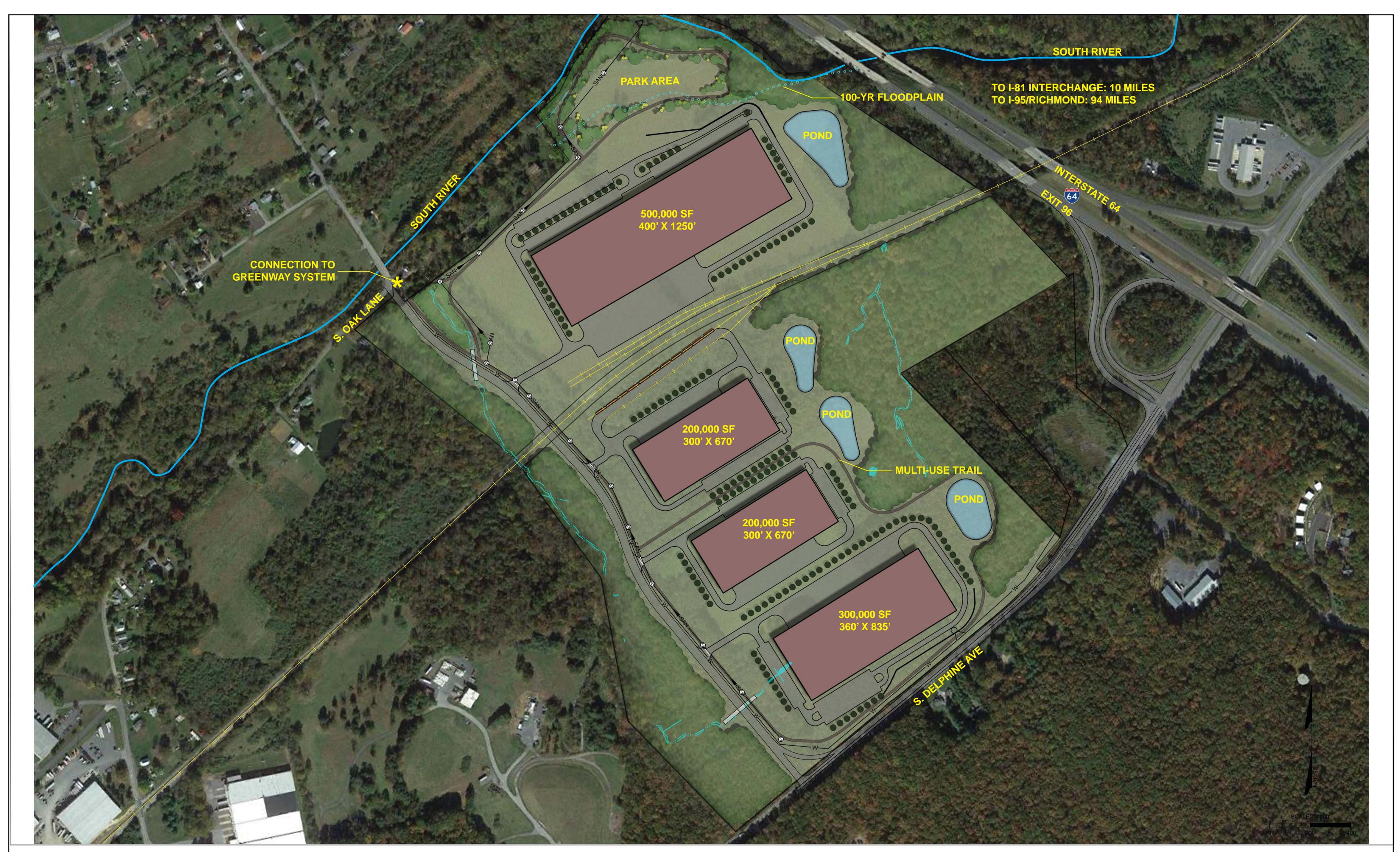








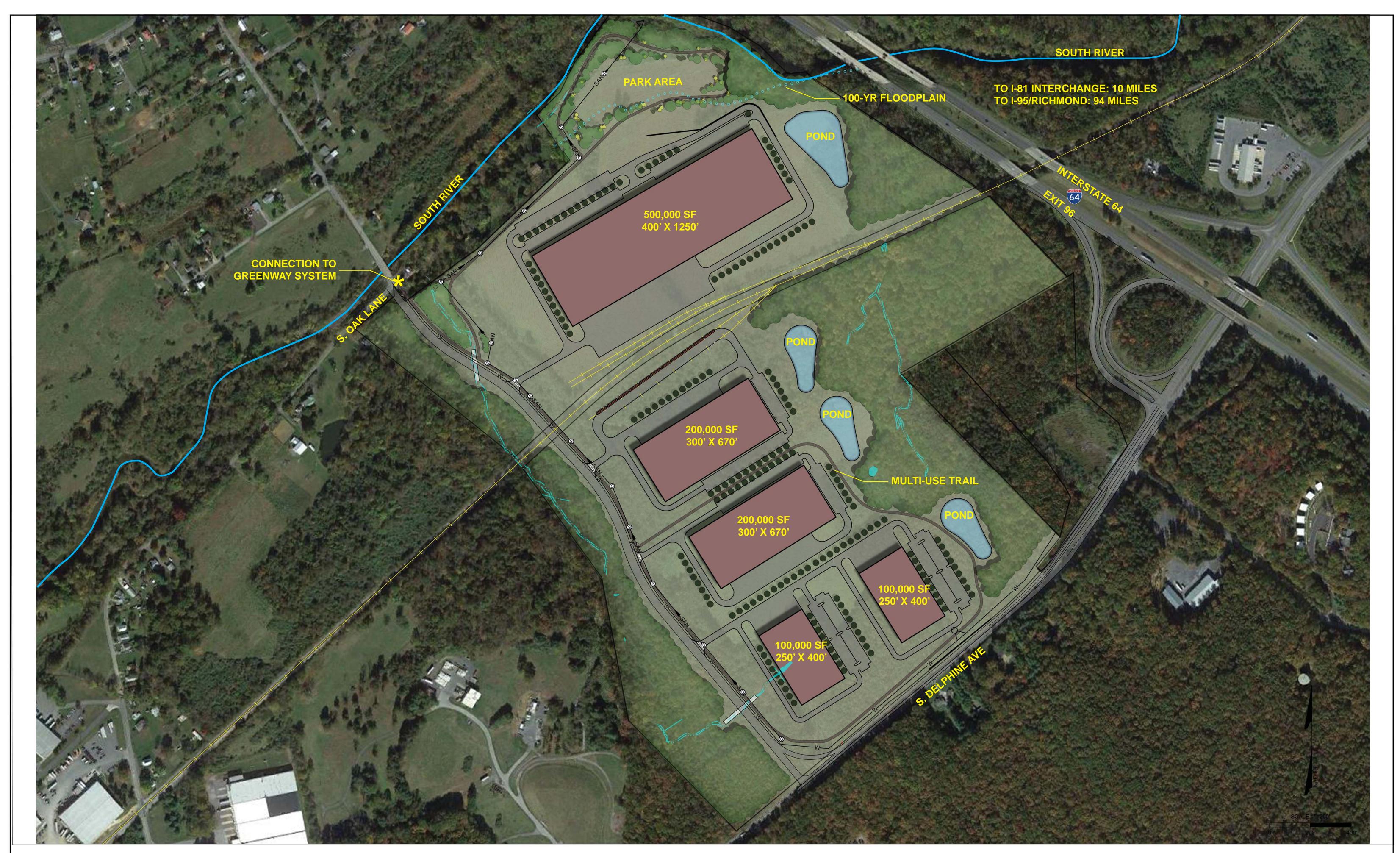




WAYNESBORO EXIT 96 INDUSTRIAL PARK OPTION 3



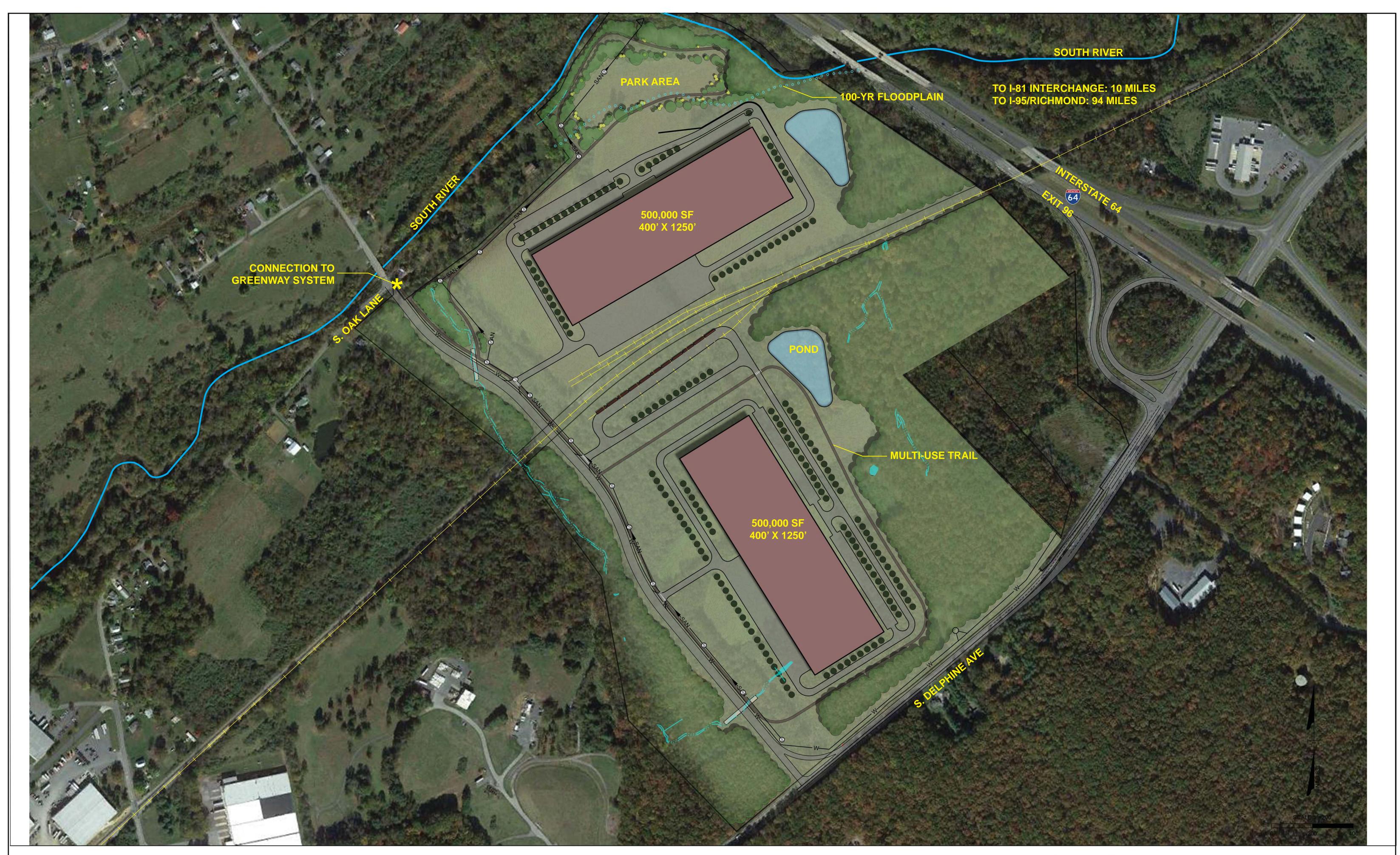








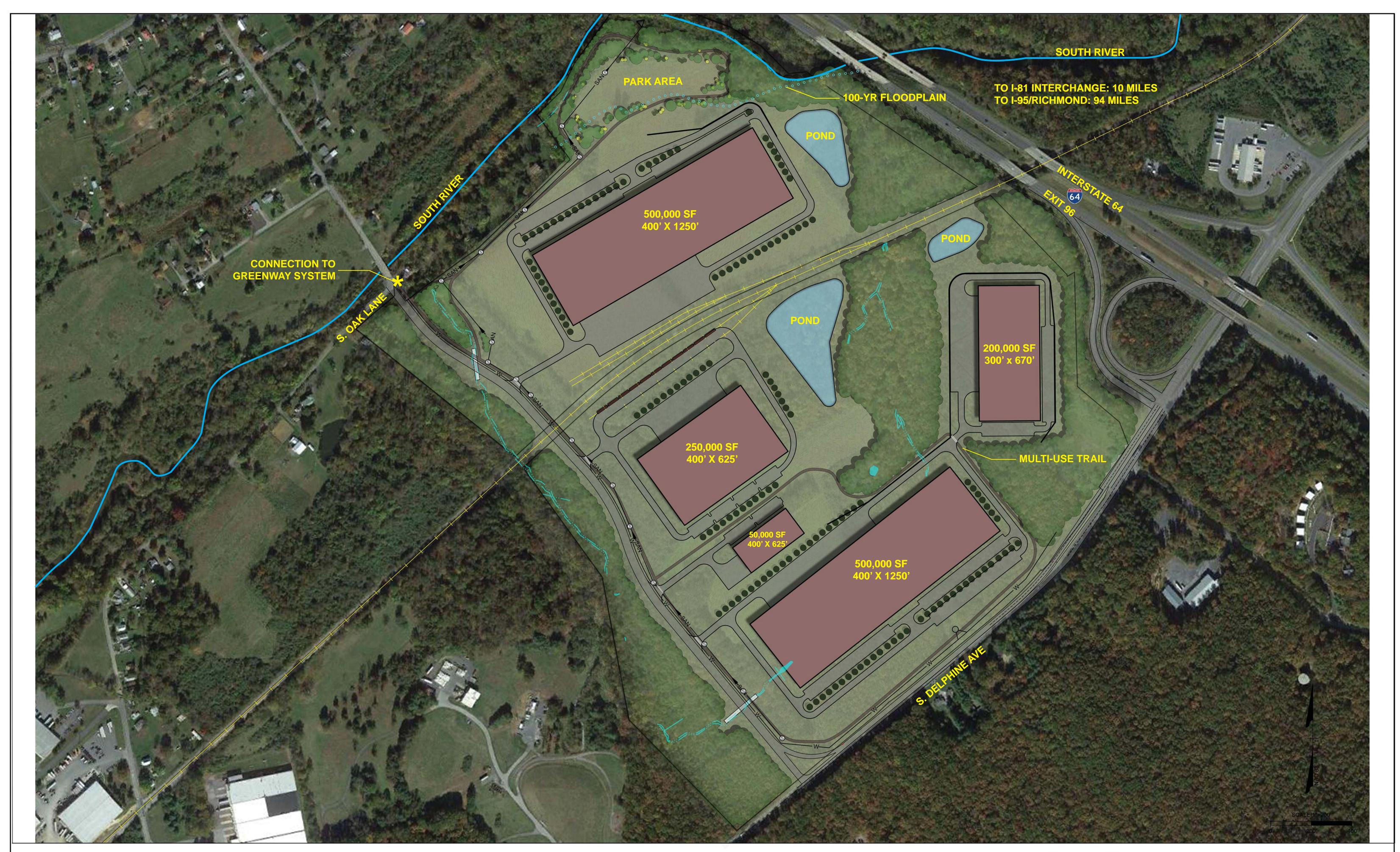


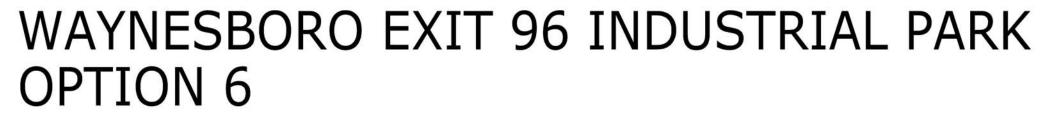








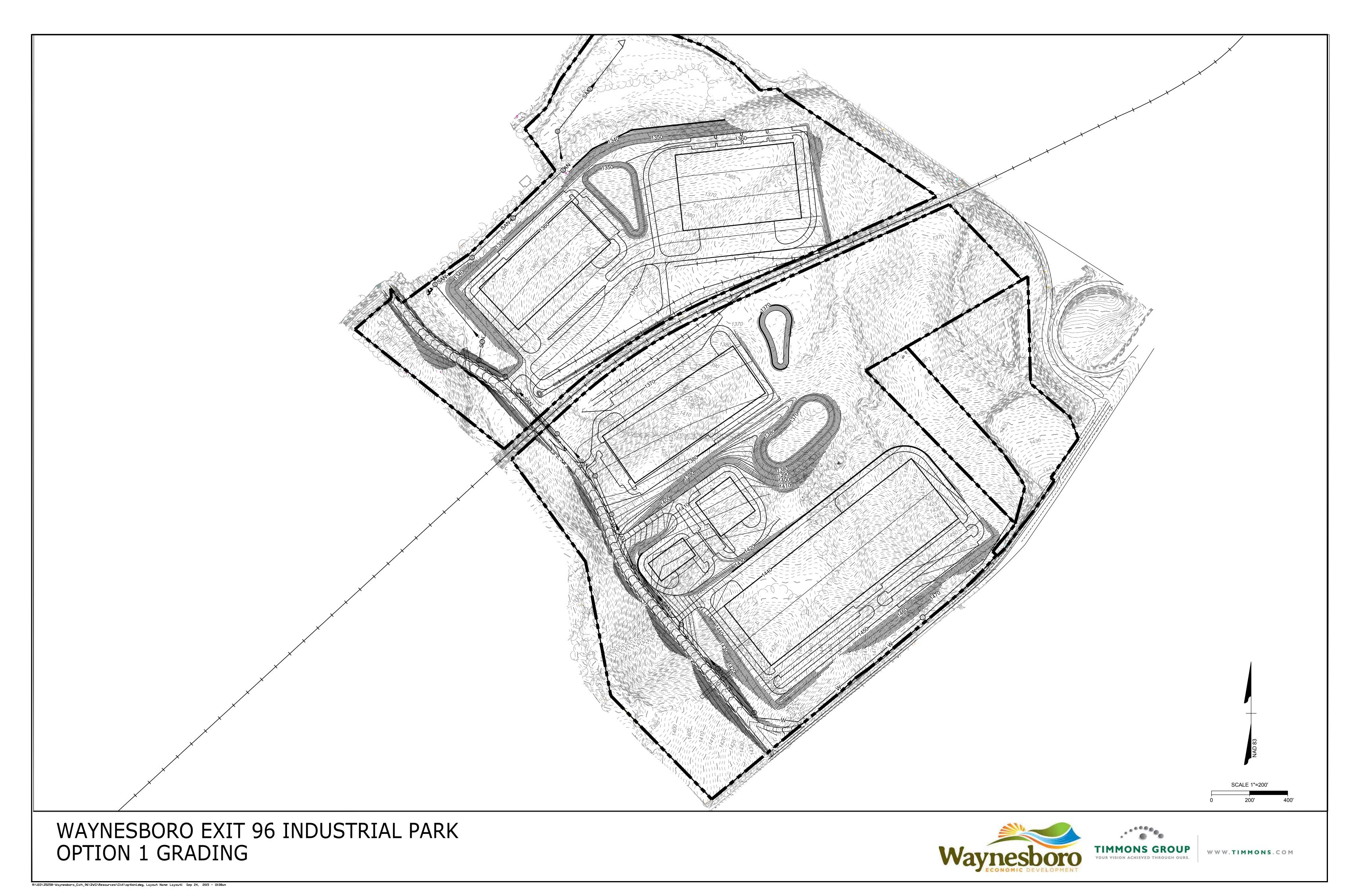








B – Master Plan Grading Exhibits





WAYNESBORO EXIT 96 INDUSTRIAL PARK OPTION 2 GRADING





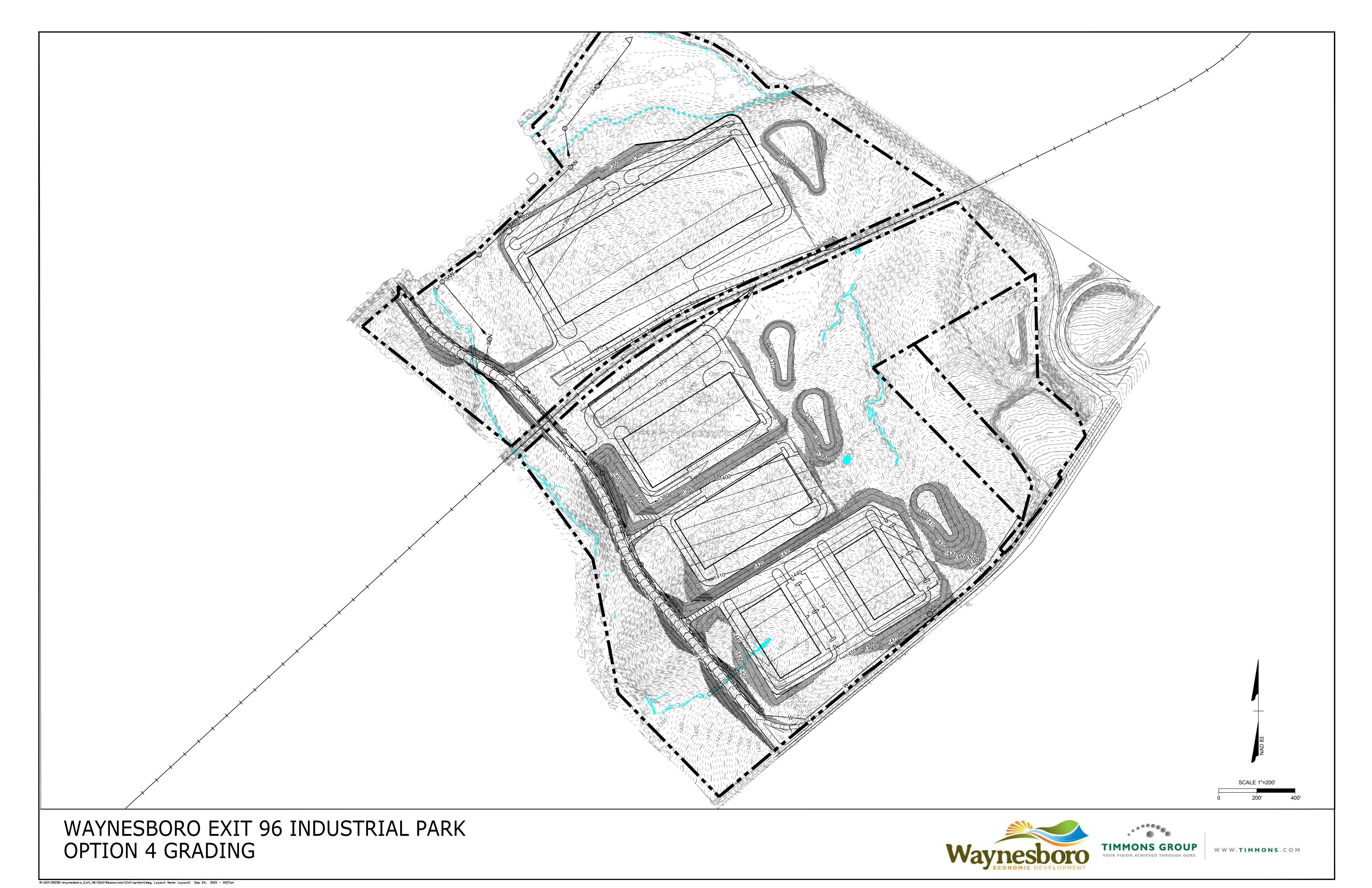




OPTION 3 GRADING





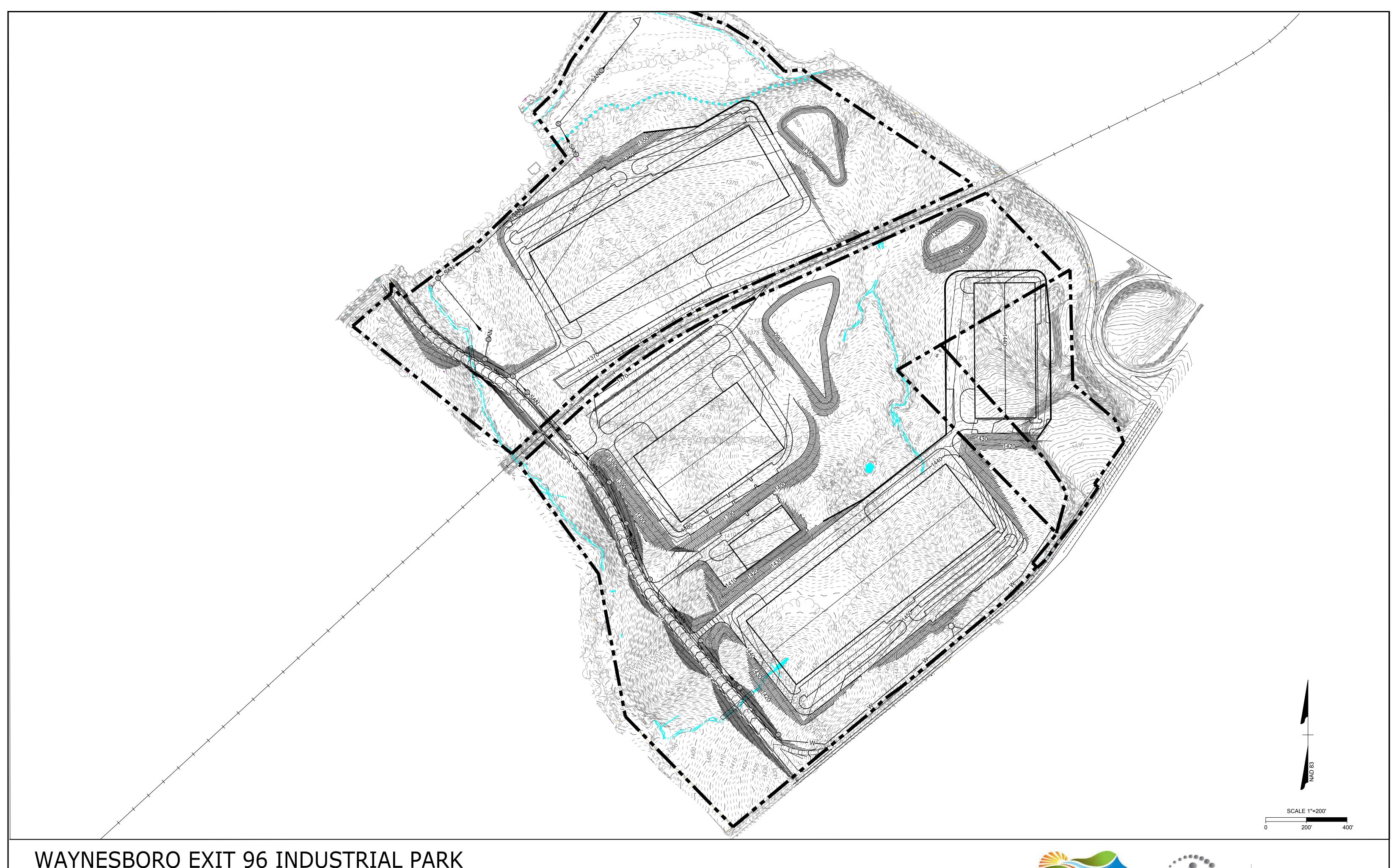




WAYNESBORO EXIT 96 INDUSTRIAL PARK OPTION 5 GRADING







WAYNESBORO EXIT 96 INDUSTRIAL PARK OPTION 6 GRADING





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C – Soil Mapping Exhibit

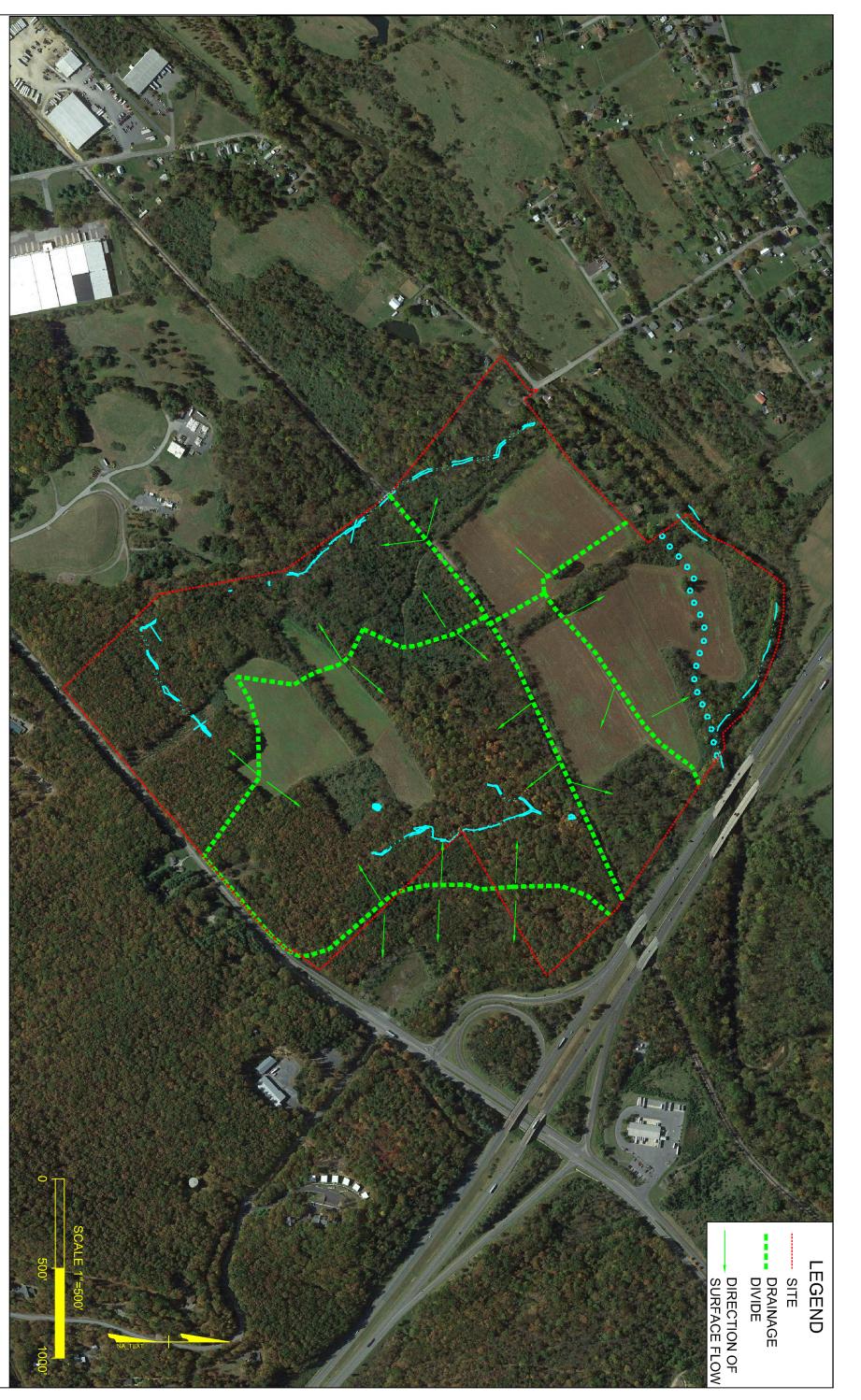


WAYNESBORO EXIT 96 INDUSTRIAL PARK SOIL MAPPING





D – Drainage Divides Exhibit



WAYNESBORO EXIT 96 INDUSTRIAL PARK DRAINAGE DIVIDES





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E – Road Network and Traffic Data Exhibit



F – Road Profiles

G – Water and Wastewater Utility Layout Exhibit







H – Stormwater Grandfathering Letter and Acceptance

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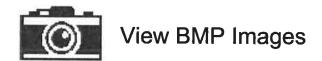
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I – Retention Basin Specifications

MINIMUM STANDARD 3.06

RETENTION BASIN



LIST OF ILLUSTRATIONS

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MINIMUM STANDARD 3.06

RETENTION BASIN



A retention basin is a stormwater facility which includes a permanent impoundment, or pool of water, and, therefore, is normally wet, even during non-rainfall periods. Inflows from stormwater runoff may be temporarily stored above this permanent pool.



A retention basin provides for long-term water quality enhancement of stormwater runoff. Stormwater inflows may also be temporarily stored above the permanent pool for downstream flood control and channel erosion control. A retention basin is considered one of the most reliable and versatile BMPs available.

Water Quality Enhancement

High removal rates of particulate and soluble pollutants (nutrients) can be achieved in retention basins through *gravitational settling*, *biological uptake* and *decomposition*. When an even higher degree of pollutant removal efficiency is required, the basin can be *enhanced* by using various modifications relating to the size and design of the permanent pool.

Monitoring studies have shown sediment removal efficiencies to range from 50-90%, total phosphorus removal efficiencies to range from 30-90% and soluble nutrient removal efficiencies to range from 40-80%. (MWCOG, 1992). The design elements, physical characteristics, and monitoring techniques varied for each basin studied, which explains the wide range of efficiencies. The target pollutant removal efficiencies assigned to the different design options are presented in **Table 3.06-1**.

FIGURE 3.06 - 1
Retention Basin - Plan & Section

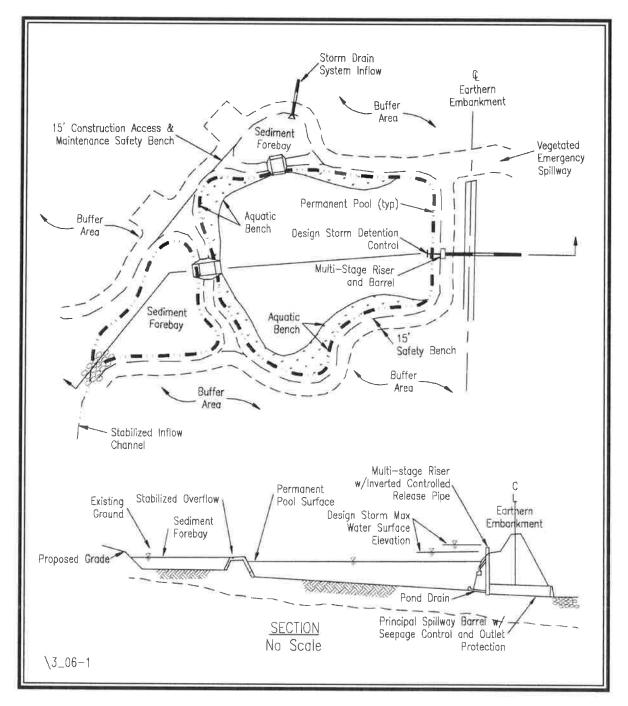


TABLE 3.06 - 1
Pollutant Removal Efficiencies for Retention Basins

Туре	Sizing Rule	Target Phosphorus Removal Efficiency	Impervious Cover
Retention Basin I	3.0 x WQ Volume	40%	22-37%
Retention Basin II	4.0 x WQ Volume	50%	38-66%
Retention Basin III	4.0 x WQ Volume with Aquatic Bench	65%	67-100%

Flood Control

Retention basins which provide flood control are designed with "dry" storage above the permanent pool. This dry storage works in concert with a riser or control structure to reduce the peak rate of runoff from a drainage area. Typically, the design storms selected for flood control (i.e., 2-year, 10-year frequency, etc.) are specified by state and local ordinances, or are based on specific watershed conditions. In either case, the required volume to be stored above the permanent pool can be readily determined using the hydrologic methods discussed in **Chapter 4**. Similarly, a control or spillway structure can be designed using the engineering calculation procedures presented in **Chapter 5**.

Channel Erosion Control

The storage volume above the permanent pool can also be used to control or reduce channel erosion. Channel erosion protection can be accomplished by reducing the peak rate of discharge, similar to flood control, or by controlling the time over which the peak volume of discharge is released (extended detention), similar to water quality enhancement. **Chapter 5-11** provides a discussion on the design criteria for channel erosion control.

Conditions Where Practice Applies

Drainage Area

A contributing watershed of at least 10 acres and/or a good source of baseflow should be present for a retention basin to be feasible. Even with 10 acres of contributing watershed, the permanent pool may be susceptible to dry weather drawdowns due to infiltration and evaporation.

(Refer to Chapter 5, Appendix 5C for water balance calculation procedures.) Dry weather stagnation may result in aesthetic and odor problems for adjacent property owners. Therefore, for residential or high visibility applications, a minimum of 15 to 20 acres of contributing watershed may be more appropriate. Infiltration basins, trenches or extended-detention basins are more suitable for smaller sites.

A retention basin is recommended for use as a regional or watershed-wide stormwater management facility since its cost per acre treated is inversely proportional to the watershed size. Studies confirm that the most cost-effective application of a retention basin is on larger, more intensely developed sites (Schueler, et. al., 1985).

Note that excavated retention basins in areas of high groundwater, such as in Tidewater, Virginia, may be feasible with very small drainage areas. The groundwater elevation should be carefully monitored, however, to verify the design permanent pool elevation.

Development Conditions

Retention basins have the potential for removing high levels of soluble and particulate pollutants which makes them suitable for most types of development. They are appropriate for both high- and low- visibility sites. However, for high-visibility sites, care must be taken to avoid the aesthetic problems associated with stagnation or excessive infiltration of the permanent pool. Maintenance of the permanent pool is not necessarily critical to the retention basin's ability to remove pollutants, but maintenance is critical to ensure the BMP's acceptance by adjacent landowners. If adequate space is available, retention basins may also be used for both high and low density residential or commercial developments.

A minimum 20-foot wide vegetated buffer should be provided around a retention basin to help filter out pollutants before they enter the basin. This requirement results in the need for more land, especially for those basins that may already be oversized to enhance their pollutant removal capabilities. It is for this reason that the use of large retention basins may not be a feasible option in developing watersheds where land is at a premium. This strengthens the argument for a regional or watershed approach to stormwater management. A regional retention or extended-detention basin is not only more cost-effective, it is also more likely to be installed on land that is not suitable for development. (It should be noted, however, that the environmental impacts and appropriate permits must still be considered for such an application.)

Planning Considerations

The success of a retention basin is dependent on the designer's ability to identify any site or downstream conditions that may affect the design and function of the basin. Above all, the facility should be compatible with both upstream and downstream stormwater systems, thus promoting a watershed approach in providing stormwater management.

Site Conditions

Existing site conditions should be considered in the design and location of a retention basin. Features such as topography, wetlands, structures, utilities, property lines, easements, etc., may impose constraints on the location or construction of the basin. Local government land use and zoning ordinances may also designate certain requirements.

All retention basins should be a minimum of 20 feet from any structure or property line (as required by local ordinances), and 100 feet from any septic tank/drainfield. (The designer should be aware that an impoundment of water may elevate the local water table which could adversely effect drainfields and structures.) Retention basins should be a minimum of 50 feet from any steep slope (greater than 15%). Alternatively, a geotechnical report must address the potential impact of any retention basin that is to be constructed on or near such a slope.

Additional considerations are as follows:

1. Soils -

In the past, many designs were accepted based upon soils information compiled from available data, such as SCS soil surveys. While such a source may be appropriate for a pre-engineering feasibility study, final design and acceptance **should be based on an actual subsurface analysis and a permeability test**, accompanied by appropriate engineering recommendations. The references listed at the end of this standard and at the end of **Minimum Standard 3.10**, **Infiltration Practices** provide more detailed information regarding the feasibility analysis of subsurface conditions for various soil types. Due to its complexity, this topic is not covered here. Note that the geotechnical study required for the embankment design (reference **Minimum Standard 3.01**, **Earthen Embankment**) will often provide adequate data to verify the soil's suitability for a retention basin.

The goal of a subsurface analysis is to determine if the soils are suitable for a retention basin. The textural character of the soil horizons and/or strata units within the subsoil profile should be identified to at least 3 feet below the facility bottom. This information is used to verify the infiltration rate or permeability of the soil. For a retention basin, water inflow (base flow and groundwater) must be greater than water losses (infiltration and evaporation). If the infiltration rate of the soil is too high, then a retention basin may not be an appropriate BMP.

Permeable soils are not suited for retention basins. The depth of the permanent pool can influence the rate at which water will infiltrate through the existing soil. The soil permeability may be such

that the basin can support a shallow marsh or constructed wetland. However, as the depth of the permanent pool increases, the increased head or pressure on the soil may increase the infiltration rate. If necessary, a liner of clay, geosynthetic fabric, or other suitable material may be used in the basin (as specified by a geotechnical engineer). Refer to the design criteria for basin liners.

2. Rock -

A subsurface investigation should also identify the presence of rock or bedrock. Excavation of rock may be too expensive or difficult with conventional earth moving equipment, precluding the use of a basin. Blasting the rock for removal may be possible, but blasting may open seams or create cracks in the underlying rock, resulting in an unwanted drawdown of the permanent pool. Blasting of rock is not recommended unless a liner, as described above, is installed.

3. Karst -

In regions where Karst topography is prevalent, projects may require thorough soils investigations and specialized design and construction techniques. The presence of karst should be determined **during the planning phase of the project** since it may affect BMP selection, design, and cost.

4. Existing Utilities—

Most utility companies will not allow a permanent or temporary pool to be installed over their underground utility lines or right-of-ways. However, if such a site must be used, the designer should obtain permission from the utility company **before designing** the basin. The relocation of any existing utilities should be researched and the costs included in the overall basin cost estimate.

Environmental Impacts

1. Wetlands –

Large facilities and/or regional facilities naturally lend themselves to being placed in low lying, and usually environmentally sensitive, areas. Such locations often contain wetlands, shallow marshes, perennial streams, wildlife habitat, etc., and may be protected by state or federal laws. The owner or designer should investigate regional wetland maps and contact appropriate local, state, and federal agencies to verify the presence of wetlands, their protected status, and suitability for a retention basin at the location in question.

With careful planning, it may be possible to incorporate wetland mitigation into a retention basin design. This assumes that the functional value of the existing or impacted wetland can be identified and included, reconstructed, or mitigated for, in the basin. The Virginia Department of Environmental Quality should be contacted for more information regarding wetland mitigation.

2. Downstream Impacts –

A retention basin may have an adverse impact on downstream water quality by altering the biological oxygen demand (BOD), dissolved oxygen (DO), temperature, etc., of the water body. This is of special concern in cold water trout streams. The release depth of the control structure, overall pond depth, hydraulic residence time, and other design features can be manipulated to help meet the site specific needs of the downstream channel.

Urban detention and retention basin design should be coordinated with a watershed or regional plan for managing stormwater runoff, if available. In a localized situation, an individual basin can provide effective stream protection for the downstream property if no other areas contribute runoff in a detrimental way to that property. However, an uncontrolled increase in the number of impoundments within a watershed can severely alter natural flow conditions, causing combined flow peaks or increased flow duration. This can ultimately lead to downstream flooding and degradation.

3. Upstream Impacts –

The upstream channel must also be considered, especially when the retention basin is to be used to control downstream channel erosion. Erosive upstream flows will not only degrade the upstream channel, but will also significantly increase the maintenance requirements in the basin by depositing large amounts of sediment eroded from the channel bottom.

Water Quality Enhancement

A retention basin is typically selected for its water quality enhancement abilities and/or aesthetic value. The flexibility of providing for additional control components (channel erosion control, flood control, habitat, etc.) increases their value. The permanent pool of a retention basin serves to enhance the quality of the stormwater within it. Studies show that providing a larger permanent pool, and/or adding modifications such as an aquatic bench, sediment forebay, etc., will provide greater and more consistent pollutant removal benefits (refer to the **Design Criteria** section in this standard). Currently, no credit is given for any additional pollutant removal efficiency that may occur with an extended-detention volume stacked on top of the permanent pool of a retention basin. However, significant improvements in channel erosion control have been reported using extended-detention for the 1-year frequency design storm (Galli, MWCOG, 1992). Refer to **Minimum Standard 3.07, Extended Detention Basins**.

A concern in specifying a retention basin is how much land it will occupy. The size of the permanent pool will be based on the desired pollutant removal efficiency. The "dry" storage volume above the permanent pool will be sized for downstream channel erosion and/or flood control. The size of these two components together will determine the size of the basin.

Preliminary sizing estimates for the permanent pool and "dry" storage volume are recommended during the planning stages to evaluate the feasibility of using a retention basin.

If a retention basin is used to remove pollutants, the water quality within the basin will be lowered, thus possibly reducing its desirability for water supply, recreation, and aesthetic purposes. Therefore, the engineer should be aware of the site's specific runoff components and understand their possible effects on the quality of the stored water. Runoff from highways and streets can be expected to carry significant concentrations of heavy metals such as lead, zinc, and copper. These and other heavy metals may accumulate in the bottom of a facility, creating a potential health and environmental hazard. If a basin is in a watershed where a significant portion of the runoff is from highways, streets or parking areas, then access to the facility should be limited and warning signs should be posted. Proper disposal of the bottom sediments from these basins may require that they be hauled to an approved facility.

Further, retention basins in residential areas are subject to nutrients from lawn fertilizers and other urban sources. Excess nutrients can lead to algae and other undesirable vegetation which can diminish the aesthetic and recreational value of the basin.

Flooding and Channel Erosion Control

Flood control and downstream channel erosion are managed by providing additional storage volume, referred to as dry storage, above the permanent pool, and properly sizing a discharge opening in the riser structure.

When a retention basin is designed for channel erosion control and/or flood control, but <u>not</u> water quality enhancement, the permanent pool volume should be sized to address maintenance, aesthetic, and feasibility concerns (adequate drainage area, etc.).

Sediment Control

A stormwater retention basin may initially serve as a sediment control basin during the project's construction. A sediment basin is designed for the maximum drainage area expected to contribute to the basin during the construction process, while a permanent stormwater basin is designed based on post-developed land use conditions. When designing a facility to do both, the basin should be sized using the most stringent criteria, sediment control or stormwater management, which will result in the largest storage volume. The design elevations should be set with final clean out and conversion in mind. The bottom elevation of the permanent SWM basin should be lower than the design bottom of the temporary E&S basin. This allows for the establishment of a solid permanent bottom after sediment is removed from the facility.

The riser and barrel hydraulics and materials should be designed as the permanent stormwater control structure. However, the permanent riser may be temporarily modified to provide a sediment basin with wet and dry storage as required by the <u>Virginia Erosion and Sediment Control Handbook</u>, (VESCH), 1992 edition.

Safety

Basins that are readily accessible to populated areas should include all possible safety precautions. Steep side slopes (steeper than 3H:1V) at the perimeter should be avoided and dangerous outlet structures should be protected by enclosures. Warning signs for deep water and potential health risks should be used wherever appropriate. Signs should be placed so that at least one is clearly visible and legible from all adjacent streets, sidewalks or paths. A notice should be posted warning residents of potential waterborne disease that may be contracted by swimming or diving in these facilities.

If the basin's surface area exceeds 20,000 square feet, an aquatic bench should be provided. (Refer to the **Design Criteria for Aquatic Bench**.)

A fence is **required** at or above the maximum water surface elevation when a basin slope is a vertical wall. Local governments and homeowner associations may also require appropriate fencing without regard for the steepness of the basin side slopes.

Maintenance

Retention basins have shown an ability to function as designed for long periods without routine maintenance. However, some maintenance is essential to protect the aesthetic and wildlife properties of these facilities.

Vehicular access to the permanent pool area and release structure must be provided to allow for long-term maintenance operations (such as sediment removal) and repairs, as needed. The incorporation of a *sediment forebay* at the inflow points into the basin will help to localize disturbance during sediment removal operations. An onsite area designated for sediment dewatering and disposal should also be included in the design. Care must be taken in the disposal of sediment that may contain an accumulation of heavy metals. Sediment testing is recommended prior to sediment removal to assure proper disposal.

A sign should be posted near the basin that clearly identifies the person or organization responsible for basin maintenance. Allowing participation by adjacent landowners or visitors is very helpful, especially if the facility serves as a recreational facility. Maintenance needs that are observed and addressed early will help to lower the overall maintenance costs. Routine maintenance inspections, however, should be conducted by authorized personnel. In all cases, access easements should be provided to facilitate inspection and maintenance operation.

Design Criteria

This section provides recommendations and minimum criteria for the design of stormwater retention basins intended to comply with the Virginia Stormwater Management program. It is the designer's responsibility to decide which aspects of the program apply to the particular facility being designed and if any additional design elements are required. The designer should also consider the long-term functioning of the facility in the selection of materials for the structural components.

Hydrology and Hydraulics

Chapter 4, Hydrologic Methods and Chapter 5, Engineering Calculations should be used to develop the pre- and post-developed hydrology for a basin's contributing watershed, to design and analyze the hydraulics of the riser and barrel system, and to design the emergency spillway.

The design of the riser and barrel system should take into account any additional storage provided above the permanent pool for peak discharge control. Generally, the 2-year storm should be used in *receiving channel adequacy* calculations and the 10-year storm should be used for *flood control* calculations. Alternative requirements such as 1-year extended detention for channel erosion control may be imposed by local ordinances.

The contributing drainage area should be a minimum of 10 acres with an adequate base flow. Fifteen to 20 acres is more appropriate to sustain a healthy permanent pool. Note that this requirement may preclude the use of the Modified Rational Method for the basin's design.

Embankment

The design of the earthen embankment for a retention basin should comply with **Minimum Standard 3.01**, **Earthen Embankment**. The requirements for geotechnical analysis, seepage control, maximum slopes and freeboard are particularly appropriate.

Principal Spillways

The design of the principal spillway and barrel system, anti-vortex device, and trash racks should comply with **Minimum Standard 3.02**, **Principal Spillway**.

Emergency Spillway

An emergency spillway that complies with Minimum Standard 3.03, Vegetated Emergency Spillway should be provided when possible, or appropriate.

Sediment Basin Conversion

When a proposed stormwater facility is used as a temporary sediment basin, the conversion to the permanent facility should be completed after final stabilization and approval from the appropriate erosion and sediment control authority.

In most cases the design criteria for the temporary sediment basin will require more storage volume (combined wet and dry) than that of a stormwater basin. In such cases, the extra volume should be allocated to the component of the facility that would derive the greatest benefit from the increased storage. This will depend on the primary function of the facility (i.e., water quality enhancement, flood control, or channel erosion control).

If modifications to the riser structure are required as part of the conversion to a permanent stormwater facility, they should be designed so that a) the structural integrity of the riser is not threatened, and b) large construction equipment is not needed within the basin. Any heavy construction work required on the riser should be completed during its initial installation. It is **NOT** recommended to install a temporary riser structure in the sediment basin and then replace it with a permanent riser after final stabilization. This may affect the structural integrity of the existing embankment and barrel.

The following additional criteria should be considered for a conversion:

- 1. Final elevations and a complete description of any modifications to the riser structure's geometry should be shown in the approved plans.
- 2. The wet storage area must be dewatered following the methods outlined in the <u>VESCH</u>, 1992 edition.
- 3. Sediment and other debris should be removed to a contained spoil area. Regrading of the basin may be necessary to achieve the final design grades and to provide an adequate topsoil layer to promote final stabilization.
- 4. Final modifications to the riser structure should be carefully inspected for watertight connections and compliance with the approved plans.
- 5. Final landscaping and stabilization should be per the <u>VESCH</u>, 1992 edition, and **Minimum Standard 3.05, Landscaping** in this handbook.

Permanent Pool

When designing a permanent pool for water quality benefits, certain physical and hydraulic factors can be manipulated to achieve a desired *pollutant removal efficiency*. These factors, which also influence the downstream water quality, include the permanent pool's *volume*, *depth*, *geometry*, *hydraulic residence time*, and *release depth*.

1. Volume -

Increasing the *volume* of the permanent pool increases the *residence time*, resulting in an increase in the pollutant removal efficiency of the permanent pool. **Table 3.06-1** provides the target pollutant removal efficiencies associated with different sizing rules.

2. **Depth** –

The depth of the permanent pool will affect several features of a retention basin including a) aquatic plant selection, b) fish and wildlife habitat selection, and c) the rate at which nutrients are cycled. Retention basins and artificial marshes built too shallow will not support fish populations year round. Basins built too deep may stratify, creating anaerobic conditions that may result in the resolubilizing of pollutants that are normally bound in the sediment. The release of such pollutants back into the water column can seriously reduce the effectivenes of the BMP and may cause nuisance conditions.

The depth of a stormwater management basin should vary to include as much diversity as possible, with an average depth of 3 to 6 feet. Approximately 15% of the basin area should be less than 18 inches deep. (Schueler, 1987). This can be accomplished by using an aquatic bench along the perimeter of the permanent pool as shown in **Figure 3.06-2**. **Table 3.06-2** below provides recommended surface area - pool depth relationships.

TABLE 3.06 - 2
Recommended Surface Area - Pool Depth Relationships for Retention Basins

ВМР	Pool Depth (ft.)	Surface Area (as % of total BMP surface area)
Retention Basin	0 - 1.5 1.5 - 2	15% 15%
	2 - 6	70%

Source: Washington State D.O.E.

3. **Geometry** –

The geometry of a stormwater basin and the associated drainage patterns are usually dictated by site topography and development conditions. However, the alignment of the incoming pipes should be manipulated relative to the release structure to the greatest extent possible to avoid *short-circuiting* of the incoming runoff. *Short-circuiting* is the condition where incoming runoff passes through the basin without displacing the old water. This can be avoided by maximizing the distance between the inlet and outlet structures. It can also be

avoided by designing a *meandering* flow path through the basin, rather than a straight line flow path. In either case, a length-to-width ratio of 2:1 should be maintained. If site conditions prevent using the proper ratio, then baffles made from gabion baskets, earthen berms or other suitable materials may be used to lengthen the flow path (see **Figure 3.06-3**).

A retention basin should be multi-celled with at least two cells and preferably three. The first cell can be used as a sediment forebay to trap coarse sediments and reduce turbulence that may cause resuspension of sediments. This first cell should be easily accessible for maintenance purposes. The second (and third) cell provides for the further settling of pollutants and any biological processes.

4. Hydraulic Residence Time -

Hydraulic residence time is the permanent pool volume divided by the average outflow discharge rate. The longer the residence time, the higher the pollutant removal efficiency (Driscoll, 1983, Kulzer, 1989). A retention basin used for channel erosion control and flood control will usually achieve higher pollutant removal rates. This is due to the increased residence time associated with the peak discharge control above the permanent pool. The hydraulic residence time would be a factor in the design of a retention basin with a permanent pool volume based on an impervious area which is relatively small when compared to the contributory drainage area. In this case, the total drainage area discharge will turn over, or replace, the volume of the "undersized" pool volume before it has achieved an adequate residence time. Optimal pollutant removal efficiency is generally associated with a mean annual hydraulic residence time of 14 to 30 days (Driscoll, 1988; Kulzer, 1989; Schueler, 1987).

5. Release Depth –

The best water quality in a retention basin's permanent pool is usually at or near the surface (Galli, 1988; Redfield, 1983). Under normal dry weather conditions, the concentrations of total dissolved solids, phosphorus, and nitrogen generally decrease in the upper portions of the water column due to physical settling and algal and biological assimilation (Galli, 1992). This suggests that <u>subsurface</u> releases have high levels of nutrients and suspended solids. In addition, deeper basins usually have very low levels of dissolved oxygen in the bottom portions of the water column.

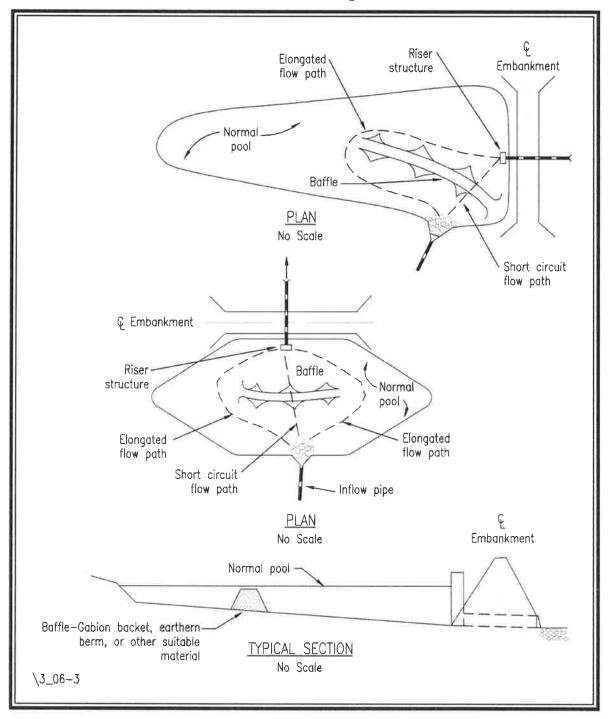
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SURFACE AREA
POOL DEPTH RELATIONSHIP Pool Depth (ft) 0-1.5 1.5-2.0 2.0-6.0 Surface Area (% of total BMP surface area) 9000 ft² (14.7%) 8250 ft² (13.5%) 43,750 ft² (71.7%) Sediment Forebay (2'-6") Avg. Depth (3.2') Total Pool Area = $61,000 \text{ ft}^2$ (0-1.5')(15-20) Deep Pool Area (2'-6') Sediment Forebay (2'-6') (0-1.5')Permanent Pool Surface Aquatic Bench (0-1.5') Safety Bench & Maintenance Access Deep Pool Area (2.5') (1.5'-2')

FIGURE 3.06 - 2
Varying Depth of Permanent Pool

SECTION NO SCALE

FIGURE 3.06 - 3
Short-Circuiting



In contrast, the water at or near the surface of a retention basin is warmer because of solar heating of the basin and heated stormwater inflow. This resembles the cycling process of water in natural lakes and water bodies. However, the proximity of a retention basin to development (i.e., impervious surfaces) may lead to an excessive heat buildup from the incoming runoff during the warmer months. Therefore, a release depth of approximately 18 inches from the water surface is recommended (Galli, 1992) to avoid extremes in temperature, nutrient levels, and dissolved oxygen (see **Figure 3.06-4**).

It should be noted that inexpensive design modifications can be incorporated into the design of a retention facility to mitigate downstream impacts such as: a) oversizing the barrel and adding surgestone or rip rap to the invert to help re-aerate the basin discharge (Schueler, 1987), and b) providing shade by planting (or saving) trees around the perimeter of the basin to help lower surface water temperature.

If the receiving stream supports a trout population, the designer should contact the Department of Game and Inland Fisheries for additional measures to protect the downstream habitat.

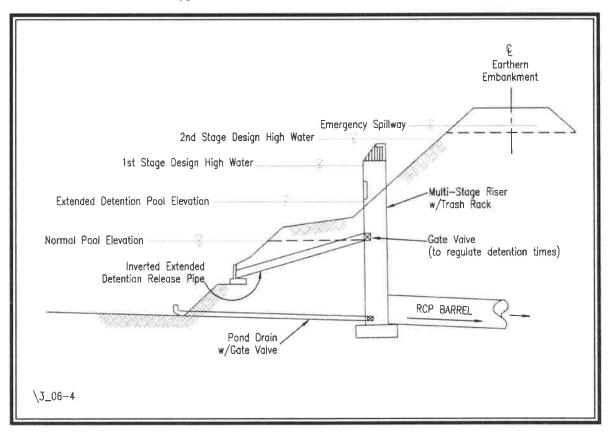


FIGURE 3.06 - 4
Typical Retention Basin Control Structure

Aquatic Bench

The pollutant removal efficiency of a retention basin can be further enhanced by adding an *aquatic bench*. An aquatic bench is a 10 to 15 foot wide area that slopes from zero inches at the shoreline to between 12 and 18 inches deep in the basin (see **Figure 3.06-5**). This bench provides suitable conditions for a variety of aquatic plants and emergent vegetation. Specific landscaping requirements for an aquatic bench should be provided on the landscaping plan per **Minimum Standard 3.05**, Landscaping.

Most important, an aquatic bench augments the pollutant removal capabilities of a retention basin by providing an environment for aquatic vegetation and associated algae, bacteria and other microorganisms that reduce organic matter and nutrients (Schueler, 1987). In addition, aquatic bench vegetation provides an ideal habitat for wildlife, such as waterfowl and fish, and for predator insects that feed on mosquitoes and other nuisance insects.

An aquatic bench also serves to stabilize and protect the shoreline from erosion resulting from fluctuating water levels, and provides a safety feature by eliminating the presence of a steep submerged slope next to the shoreline.

The increase in pollutant removal efficiency associated with the establishment of an aquatic bench is **approximated** based on available information. Note that discharge monitoring may indicate much higher or lower values since many variables exist in any given stormwater basin design and the efficiencies are estimated.

Sediment Forebay

A sediment forebay will help to postpone overall basin maintenance by trapping incoming sediments at a specified location. The forebay should be situated and designed per **Minimum Standard 3.04**, **Sediment Forebays**. Usually, a sediment forebay is placed at the outfall of the incoming storm drain pipes or channels directed toward the basin and is situated to provide access for maintenance equipment.

A sediment forebay enhances the pollutant removal efficiency of a basin by trapping the incoming sediment load in one area, where it can be easily monitored and removed. The *target pollutant removal efficiency* of a retention basin, as listed in **Table 3.06-1**, is predicated on the use of sediment forebays at the inflow points to the basin.

Liner to Prevent Infiltration

A retention basin should have negligible infiltration through its bottom. Infiltration may impair the proper functioning of the basin and may contaminate groundwater. Where infiltration is anticipated, or in areas underlain by karst topography then a retention or detention facility should **not** be used unless an impervious liner is installed. When using a liner, the specifications provided in **Table**

3.06-3 for clay liners and the following recommendations apply:

- 1. A clay liner should have a minimum thickness of 12 inches.
- A layer of compacted topsoil (minimum thickness 6 to 12 inches) should be placed over the liner before seeding with an appropriate seed mixture (refer to the <u>VESCH</u>, 1992 edition.)
- 3. Other liners may be used provided the engineer can supply supporting documentation that the material will achieve the required performance.

In many cases, the fine particulates and suspended solids in the water column of a new retention basin will settle out and quickly clog the the pores of the bottom soil. However, a geotechnical analysis should address the potential for infiltration and, if needed, specify liner materials.

Safety

The side slopes of a retention basin should be no steeper than 3H:1V and should be stabilized with permanent vegetation. If the basin surface exceeds 20,000 square feet, an aquatic bench should be provided to serve as a safety feature. Fencing may also be required by local ordinance.

Access

A 10 to 12-foot-wide access road with a maximum grade of 12% should be provided to allow vehicular access to both the outlet structure area and at least one side of the basin. The road's surface material should be selected to support the anticipated frequency of use and the anticipated vehicular load without excessive erosion or damage.

TABLE 3.06 - 3
Clay Liner Specifications

Property	Test Method (or equal)	Unit	Specification
Permeability	ASTM D-2434	cm/sec	1 x 10 ⁻⁶
Plasticity Index of Clay	ASTM D-423 & D-424	%	Not less than 15
Liquid Limit of Clay	ASTM D-2216	%	Not less than 30
Clay Particles Passing	ASTM D-422	%	Not less than 30
Clay Compaction	ASTM D-2216	%	95% of Standard Proctor Density

Source: City of Austin, 1988

Landscaping

A qualified individual should prepare the landscape plan for a retention basin. Appropriate *shoreline* fringe, riparian fringe and floodplain terrace vegetation must be selected to correspond with the expected frequency and duration of inundation. Selection and installation guidelines should be per **Minimum Standard 3.05**, **Landscaping**.

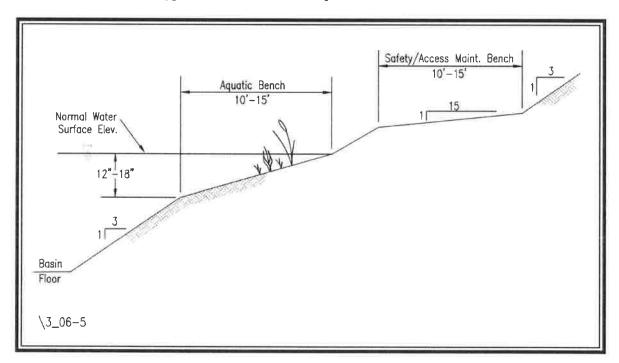
Vegetation should be planted in soil that is appropriate for the plants selected. Soil tests showing the adequacy of the soil or a soil enhancement plan should be submitted with the overall basin design.

The soil substrate must be soft enough to permit easy installation of the plants. If the basin soil has been compacted or vegetation has formed a dense root mat, the upper 6 inches of soil should be disked before planting. If soil is imported, it should be laid at least 6 inches deep to provide sufficient depth for plant rooting to occur.

Buffer Zones

A vegetated buffer strip should be maintained beside the basin. The strip should be a minimum of 20 feet wide, as measured from the maximum water surface elevation. Refer to **Minimum Standard 3.05**, **Landscaping**.

FIGURE 3.06 - 5
Typical Retention Basin Aquatic Bench - Section



Construction Specifications

The construction specifications for stormwater retention basins outlined below should be considered minimum guidelines. More stringent or additional specifications may be required based on individual site conditions.

Overall, widely accepted construction standards and specifications for embankment ponds and reservoirs, such as those developed by the USDA Soil Conservation Service or the U.S. Army Corps of Engineers, should be followed to build an impoundment.

Further guidance can be found in Chapter 17 of the Soil Conservation Service's Engineering Field Manual. Specifications for the work should conform to methods and procedures indicated for installing earthwork, concrete, reinforcing steel, pipe, water gates, metal work, woodwork and masonry and any other items that are apply to the site and the purpose of the structure. The specifications should also satisfy any requirements of the local plan approving authority.

The following minimum standards contain guidance and construction specifications for various components of retention basins: 3.01, Earthen Embankment; 3.02, Principal Spillway; 3.03, Vegetated Emergency Spillway; 3.04, Sediment Forebay; and 3.05, Landscaping.

Maintenance and Inspections

The following maintenance and inspection guidelines are not intended to be all-inclusive. Specific facilities may require other measures not discussed here. The engineer is responsible for determining if any additional items are necessary.

Inspecting and maintaining the structures and the impoundment area should be the responsibility of either the local government, a designated group such as a homeowner's association or an individual. A specific maintenance plan should be formulated outlining the schedule and scope of maintenance operations.

Any standing water pumped during the maintenance operation must be disposed of per the <u>VESCH</u>, 1992 edition and any local requirements.

General Maintenance

Maintenance and inspection guidelines found in the following minimum standards apply: 3.01, Earthen Embankment; 3.02, Principal Spillway; 3.03, Vegetated Emergency Spillway; 3.04, Sediment Forebay; and 3.05: Landscaping.

Vegetation

The basin's side slopes, embankment and emergency spillway should be mowed at least twice a year to discourage woody growth. For aesthetic purposes, more frequent mowing may be necessary in residential areas

Specific plant communities may require different levels of maintenance. Upland and floodplain terrace areas, grown as meadows or forests, require very little maintenance, while aquatic or emergent vegetation may need periodic thinning or reinforcement plantings. Note that after the first growing season, it should be obvious if reinforcement plantings are needed. If they are, they should be installed at the onset of the second growing season after construction.

Research indicates that for most aquatic plants the uptake of pollutants is stored in the roots, not the stems and leaves (Lepp 1981). Therefore, aquatic plants should not require harvesting before winter plant die-back. There are still many unanswered questions about the long term pollutant storage capacity of plants. It is possible that aquatic and emergent plant maintenance recommendations may be presented in the future.

Debris and Litter Removal

Debris and litter will accumulate near the inflow points and around the outlet control structure. Such material should be removed periodically. Also, as the water level rises during storm events, floatables accumulate around the grate or trash rack of the control structure. If a flat horizontal trash rack is used, floating debris will become lodged on the trash rack, which will remain clogged until it is manually cleaned. A significant accumulation can clog the riser structure. The use of an angled trash rack is recommended to allow any accumulated debris to slide off as the water level drops.

Sediment Removal

Sediment deposition should be continually monitored in the basin. Removal of any accumulated sediment, in the sediment forebay or elsewhere, is extremely important. A significant accumulation of sediment impairs the pollutant removal capabilities of the basin by reducing the permanent pool volume. The deposited sediment also becomes prone to resuspension during heavy flow periods. Unless unusual conditions exist, accumulated sediment should be removed from the sediment forebay and possibly other deep areas within the permanent pool every 5 to 10 years. The use of a sediment forebay with access for heavy equipment will greatly simplify the removal process. **During maintenance procedures, ensure that any pumping of standing water or dewatering of dredged sediments complies with the VESCH, 1992 edition, and any local requirements**.

Owners, operators, and maintenance authorities should be aware that significant concentrations of heavy metals (e.g., lead, zinc and cadmium) and some organics, such as pesticides, may be expected to accumulate at the bottom of a retention basin. Testing of sediment, especially near points of inflow, should be conducted regularly and **before disposal** to establish the leaching potential and

level of accumulation of hazardous materials. Disposal methods must comply with applicable state and local regulations (e.g., for special waste).

Inspections

A retention basin and its components should be inspected annually, at a minimum, to ensure that they operate in the manner originally intended. Items in need of repair should be addressed promptly and as specified in the comprehensive maintenance program. Detailed inspections by qualified person(s) should address the following areas/concerns:

- · Dam settling, woody growth, and signs of piping
- Signs of seepage on the downstream face of the embankment
- · Condition of grass cover on the embankment, basin floor and perimeter
- Riprap displacement or failure
- Principal and emergency spillway meet design plans for operation
- Outlet controls, debris racks and mechanical and electrical equipment
- Outlet channel conditions
- Inlet pipe conditions
- Safety features of the facility
- Access for maintenance equipment
- Sediment accumulation
- Debris and trash accumulation
- Erosion of the embankment or side slopes

Design Procedures

- 1. Determine if the anticipated development conditions and drainage area are appropriate for a stormwater retention basin BMP.
 - •• Minimum drainage area of 10 acres and/or base flow
- 2. Determine if the soils (permeability, bedrock, Karst, embankment foundation, etc.) and topographic conditions (slopes, existing utilities, environmental restrictions) are appropriate for a stormwater retention basin BMP.
- 3. Determine any additional stormwater management requirements (channel erosion, flooding) for the project.
- 4. Locate the stormwater retention basin on the site.

- 5. Determine the hydrology and peak discharges of the contributory drainage area for each of the required design storms (**Chapter 4, Hydrologic Methods**).
- 6. Calculate the permanent pool volume and approximate storage volume requirements (**Chapter 5**, **Engineering Calculations**).
- 7. Design the embankment (Min. Std. 3.01), principal spillway (Min. Std. 3.02), emergency spillway (Min. Std. 3.03), sediment forebay (Min. Std. 3.04), landscaping plan (Min. Std. 3.05), and the permanent pool and other components of a stormwater retention basin BMP (Min. Std. 3.06) using Chapter 5, Engineering Calculations, and the Minimum Standards listed.
 - •• permanent pool depth
 - Permanent pool geometry
 - •• release depth
 - •• aquatic bench
 - •• pond drain
- 8. Design final grading of basin.
 - •• landscape plan
 - •• 20-foot buffer area
 - •• safety (3:1 slopes with bench)
 - · · access
- 9. Establish specifications for sediment control and sediment basin conversion (if required).
- 10. Establish construction sequence and construction specifications.
- 11. Establish maintenance and inspection requirements.



Refer to Appendix-3A for Design and Plan Review, Construction Inspection, and Operation and Maintenance Checklists.

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Retention basin with small island.



Retention basin in ultra-urban setting (under construction).

Retention Basin



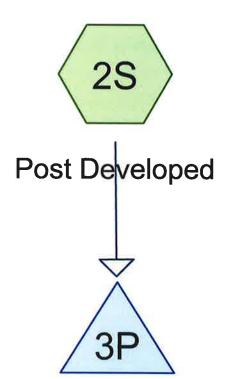
Retention basin – Note flat slopes with "rough" edge and aquatic bench provided as safety and pollutant removal features.

Retention Basin

J – Preliminary Pond Routings



PreDeveloped



Pond









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Area Listing (all nodes)

Area	CN	Description
(acres)		(subcatchment-numbers)
24.000	98	(2S)
17.000	68	(2S)
41.000	65	Fallow, crop residue, Good, HSG B (1S)
82.000	75	TOTAL AREA

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Soil Listing (all nodes)

Area (acres)	Soil Group	Subcatchment Numbers
0.000	HSG A	
41.000	HSG B	1S
0.000	HSG C	
0.000	HSG D	
41.000	Other	2 S
82.000		TOTAL AREA

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Ground Covers (all nodes)

	HSG-A (acres)	HSG-B (acres)	HSG-C (acres)	HSG-D (acres)	Other (acres)	Total (acres)	Ground Cover	Subcatchment Numbers
_	0.000	0.000	0.000	0.000	41.000	41.000		2S
	0.000	41.000	0.000	0.000	0.000	41.000	Fallow, crop residue, Good	1S
	0.000	41.000	0.000	0.000	41.000	82.000	TOTAL AREA	

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Pipe Listing (all nodes)

Line#	Node	In-Invert	Out-Invert	Length	Slope	n	Diam/Width	Height	Inside-Fill
	Number	(feet)	(feet)	(feet)	(ft/ft)		(inches)	(inches)	(inches)
1	3P	100.00	95.00	150.0	0.0333	0.050	48.0	0.0	0.0

Type II 24-hr 2-Year Rainfall=3.68"

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Time span=5.00-20.00 hrs, dt=0.05 hrs, 301 points
Runoff by SCS TR-20 method, UH=SCS, Weighted-CN
Reach routing by Stor-Ind+Trans method - Pond routing by Stor-Ind method

Subcatchment 1S: PreDeveloped

Runoff Area=41.000 ac 0.00% Impervious Runoff Depth>0.74"

Tc=25.0 min CN=65 Runoff=28.64 cfs 2.543 af

Subcatchment 2S: Post Developed

Runoff Area=41.000 ac 58.54% Impervious Runoff Depth>2.09"

Tc=15.0 min CN=86 Runoff=118.34 cfs 7.149 af

Pond 3P: Pond

Peak Elev=102.64' Storage=3.446 af Inflow=118.34 cfs 7.149 af

Outflow=25.19 cfs 6.273 af

Total Runoff Area = 82.000 ac Runoff Volume = 9.692 af Average Runoff Depth = 1.42" 70.73% Pervious = 58.000 ac 29.27% Impervious = 24.000 ac

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Summary for Subcatchment 1S: PreDeveloped

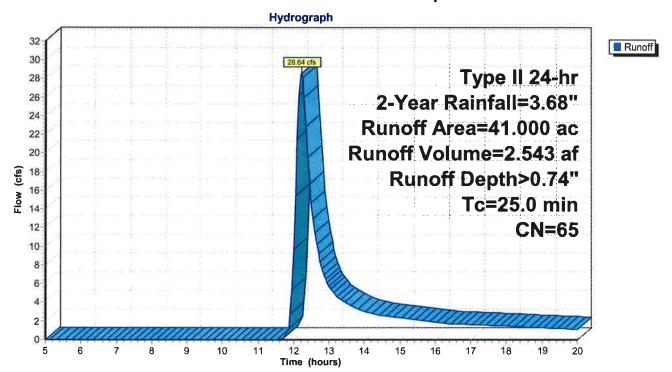
Runoff = 28.64 cfs @ 12.22 hrs, Volume=

2.543 af, Depth> 0.74"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 5.00-20.00 hrs, dt= 0.05 hrs Type II 24-hr 2-Year Rainfall=3.68"

	Area	(ac)	CN	Desc	ription		
4	41.	000	65	Fallo	w, crop re	sidue, Goo	od, HSG B
	41.	.000		100.0	00% Pervi	ous Area	
	Tc (min)	Length (feet		ope ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
-	25.0	(1001	/	1011)	(10000)	(010)	Direct Entry,

Subcatchment 1S: PreDeveloped



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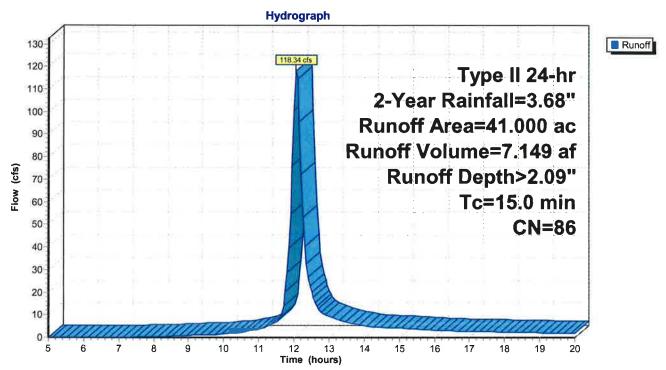
Summary for Subcatchment 2S: Post Developed

Runoff = 118.34 cfs @ 12.07 hrs, Volume= 7.149 af, Depth> 2.09"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 5.00-20.00 hrs, dt= 0.05 hrs Type II 24-hr 2-Year Rainfall=3.68"

	Area	(ac)	CN	Desc	cription			
*	24.	000	98					
*	17.	000	68					
-	41.	000	86	Weig	hted Aver	age		
	17.	000			6% Pervio			
	24.	000		58.5	4% Imperv	vious Area		
	Tc (min)	Leng		Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description	
	15.0						Direct Entry,	

Subcatchment 2S: Post Developed



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Summary for Pond 3P: Pond

Inflow Area = 41.000 ac, 58.54% Impervious, Inflow Depth > 2.09" for 2-Year event

Inflow = 118.34 cfs @ 12.07 hrs, Volume= 7.149 af

Outflow = 25.19 cfs @ 12.42 hrs, Volume= 6.273 af, Atten= 79%, Lag= 21.2 min

Primary = 25.19 cfs @ 12.42 hrs, Volume= 6.273 af

Routing by Stor-Ind method, Time Span= 5.00-20.00 hrs, dt= 0.05 hrs Peak Elev= 102.64' @ 12.42 hrs Surf.Area= 1.398 ac Storage= 3.446 af

Plug-Flow detention time= 114.3 min calculated for 6.273 af (88% of inflow)

Center-of-Mass det. time= 75.3 min (857.3 - 782.0)

Volume	Invert A	Avail.Stora	ge Stora	age Description
#1	100.00'	8.560	af Cust	tom Stage Data (Prismatic) Listed below (Recalc)
Elevation		ı Ind	c.Store	Cum.Store
(fee	t) (acres)) (acr	e-feet)	(acre-feet)
100.0	00 1.210		0.000	0.000
102.0	00 1.350)	2.560	2.560
104.0	00 1.500)	2.850	5.410
106.0	00 1.650)	3.150	8.560
Device	Routing	Invert	Outlet De	evices
#1	Primary	100.00'	48.0" Ro	ound Culvert L= 150.0' Ke= 0.050
	•		Inlet / Ou	utlet Invert= 100.00' / 95.00' S= 0.0333 '/' Cc= 0.900
			n = 0.050), Flow Area= 12.57 sf
#2	Device 1	100.00'	15.0" Ver	ert. Orifice/Grate X 3.00 C= 0.600
#3	Device 1	103.00'	72.0" Ho	oriz. Orifice/Grate C= 0.600
			Limited to	to weir flow at low heads

Primary OutFlow Max=25.18 cfs @ 12.42 hrs HW=102.64' (Free Discharge)

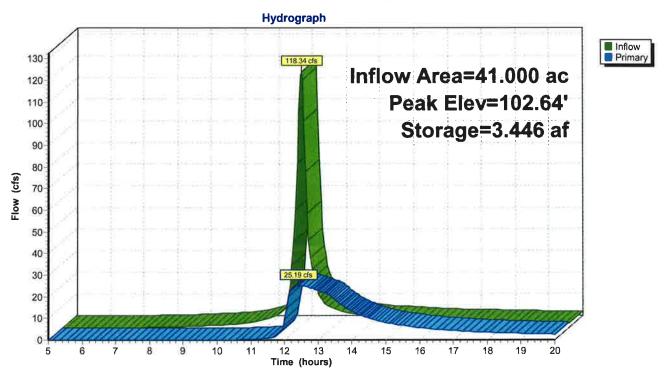
-1=Culvert (Passes 25.18 cfs of 34.08 cfs potential flow)

2=Orifice/Grate (Orifice Controls 25.18 cfs @ 6.84 fps)

-3=Orifice/Grate (Controls 0.00 cfs)

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Pond 3P: Pond



Type II 24-hr 10-Year Rainfall=5.56"

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Time span=5.00-20.00 hrs, dt=0.05 hrs, 301 points
Runoff by SCS TR-20 method, UH=SCS, Weighted-CN
Reach routing by Stor-Ind+Trans method - Pond routing by Stor-Ind method

Subcatchment 1S: PreDeveloped

Runoff Area=41.000 ac 0.00% Impervious Runoff Depth>1.83"

Tc=25.0 min CN=65 Runoff=77.92 cfs 6.262 af

Subcatchment 2S: Post Developed

Runoff Area=41.000 ac 58.54% Impervious Runoff Depth>3.73"

Tc=15.0 min CN=86 Runoff=205.35 cfs 12.734 af

Pond 3P: Pond

Peak Elev=104.15' Storage=5.628 af Inflow=205.35 cfs 12.734 af

Outflow=66.93 cfs 11.664 af

Total Runoff Area = 82.000 ac Runoff Volume = 18.996 af Average Runoff Depth = 2.78" 70.73% Pervious = 58.000 ac 29.27% Impervious = 24.000 ac

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Summary for Subcatchment 1S: PreDeveloped

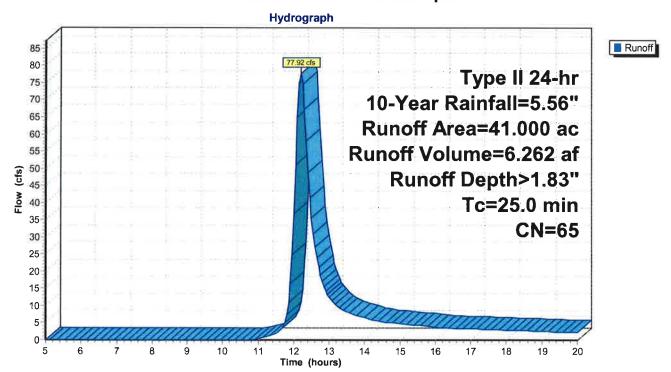
Runoff = 77.92 cfs @ 12.20 hrs, Volume=

6.262 af, Depth> 1.83"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 5.00-20.00 hrs, dt= 0.05 hrs Type II 24-hr 10-Year Rainfall=5.56"

-	Area	(ac)	CN	Desc	cription		
4	41.	000	65	Fallo	w, crop re	sidue, Goo	od, HSG B
	41.	000		100.	00% Pervi	ous Area	
	Тс	Lengt	h S	Slope	Velocity	Capacity	Description
	_(min)	(fee	t)	(ft/ft)	(ft/sec)	(cfs)	
	25.0						Direct Entry

Subcatchment 1S: PreDeveloped



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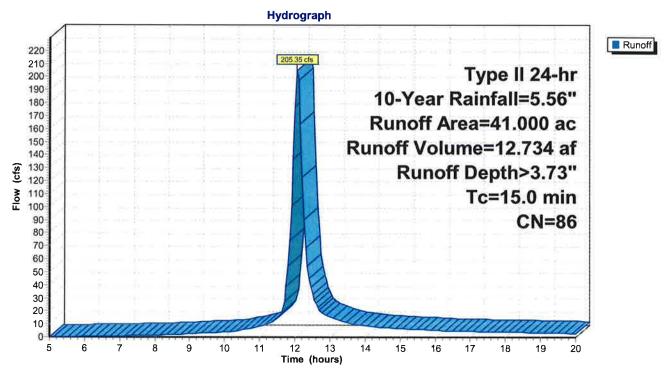
Summary for Subcatchment 2S: Post Developed

Runoff = 205.35 cfs @ 12.07 hrs, Volume= 12.734 af, Depth> 3.73"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 5.00-20.00 hrs, dt= 0.05 hrs Type II 24-hr 10-Year Rainfall=5.56"

_	Area	(ac)	CN	Desc	cription		
*	24.	000	98		100		
*	17.	000	68				
	41.	000	86	Weig	ghted Aver	age	
	17.	000		41.4	6% Pervio	us Area	
	24.	000		58.5	4% Imperv	vious Area	
	Tc (min)	Length (feet		Slope (ft/ft)	Velocity (ft/sec)	Capacity (cfs)	Description
-		Tieer		(IVIL)	(IUSEC)	(015)	
	15.0						Direct Entry,

Subcatchment 2S: Post Developed



Volume

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Summary for Pond 3P: Pond

Inflow Area = 41.000 ac, 58.54% Impervious, Inflow Depth > 3.73" for 10-Year event

Inflow = 205.35 cfs @ 12.07 hrs, Volume= 12.734 af

Outflow = 66.93 cfs @ 12.31 hrs, Volume= 11.664 af, Atten= 67%, Lag= 14.7 min

Primary = 66.93 cfs @ 12.31 hrs, Volume= 11.664 af

Routing by Stor-Ind method, Time Span= 5.00-20.00 hrs, dt= 0.05 hrs Peak Elev= 104.15' @ 12.31 hrs Surf.Area= 1.511 ac Storage= 5.628 af

Plug-Flow detention time= 96.9 min calculated for 11.664 af (92% of inflow)

Avail.Storage Storage Description

Center-of-Mass det. time= 67.4 min (836.2 - 768.7)

Invert

VOIGITIC	IIIVOIC /	rvan.Otorag	ge Clora	ge Description		
#1	100.00'	8.560	af Custo	om Stage Data	(Prismatic) Listed below ((Recalc)
Elevatior (feet			c.Store e-feet)	Cum.Store (acre-feet)		
100.00	1.210		0.000	0.000		
102.00	1.350		2.560	2.560		
104.00	1,500		2.850	5.410		
106.00	1.650		3.150	8.560		
Device	Routing	Invert	Outlet De	vices		
#1	Primary		Inlet / Out	und Culvert Litel Invert = 100.4 Flow Area = 12	00' / 95.00' S= 0.0333 '/'	Cc= 0.900
#2	Device 1	100.00'	15.0" Ver	t. Orifice/Grate	X 3.00 C= 0.600	
#3	Device 1	103.00'	72.0" Hor	iz. Orifice/Grate	e C= 0.600	

Limited to weir flow at low heads

Primary OutFlow Max=66.86 cfs @ 12.31 hrs HW=104.14' (Free Discharge)

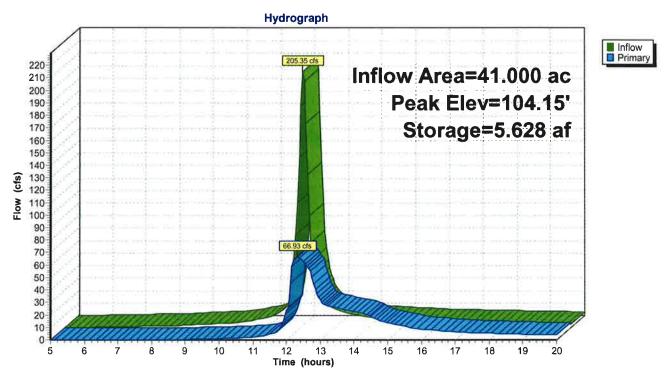
1=Culvert (Barrel Controls 66.86 cfs @ 6.39 fps)

2=Orifice/Grate (Passes < 33.24 cfs potential flow)

-3=Orifice/Grate (Passes < 75.13 cfs potential flow)

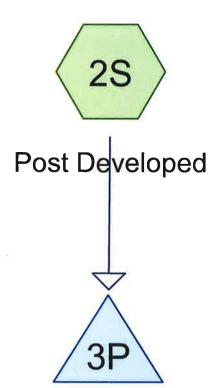
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Pond 3P: Pond





PreDeveloped



Pond









Routing Diagram for Southern Area
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Area Listing (all nodes)

Area	CN	Description
(acres)		(subcatchment-numbers)
84.000	65	(1S, 2S)
30.000	98	(2S)
114.000	74	TOTAL AREA

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Soil Listing (all nodes)

Area	Soil	Subcatchment
(acres)	Group	Numbers
0.000	HSG A	
0.000	HSG B	
0.000	HSG C	
0.000	HSG D	
114.000	Other	1S, 2S
114.000		TOTAL AREA

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Ground Covers (all nodes)

HS	G-A	HSG-B	HSG-C	HSG-D	Other (acres)	Total	Ground	Subcatchment
(ac	res)	(acres)	(acres)	(acres)		(acres)	Cover	Numbers
	000 000	0.000 0.000	0.000 0.000	0.000 0.000	114.000 114.000	114.000 114.000	TOTAL AREA	1S, 2S

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Pipe Listing (all nodes)

Line#	Node	In-Invert	Out-Invert	Length	Slope	n	Diam/Width	Height	Inside-Fill
	Number	(feet)	(feet)	(feet)	(ft/ft)		(inches)	(inches)	(inches)
1	3P	100.00	95.00	200.0	0.0250	0.013	48.0	0.0	0.0

Type II 24-hr 2-Year Rainfall=3.50" Printed 8/8/2015

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ı aye

Time span=0.00-24.00 hrs, dt=0.01 hrs, 2401 points
Runoff by SCS TR-20 method, UH=SCS, Weighted-CN
Reach routing by Stor-Ind+Trans method - Pond routing by Stor-Ind method

Subcatchment 1S: PreDeveloped

Runoff Area=49.000 ac 0.00% Impervious Runoff Depth>0.74"

Tc=25.0 min CN=65 Runoff=29.52 cfs 3.039 af

Subcatchment 2S: Post Developed

Runoff Area=65.000 ac 46.15% Impervious Runoff Depth>1.63"

Tc=12.0 min CN=80 Runoff=152.76 cfs 8.834 af

Pond 3P: Pond

Peak Elev=103.15' Storage=3.519 af Inflow=152.76 cfs 8.834 af

Outflow=28.15 cfs 8.387 af

Total Runoff Area = 114.000 ac Runoff Volume = 11.873 af Average Runoff Depth = 1.25" 73.68% Pervious = 84.000 ac 26.32% Impervious = 30.000 ac

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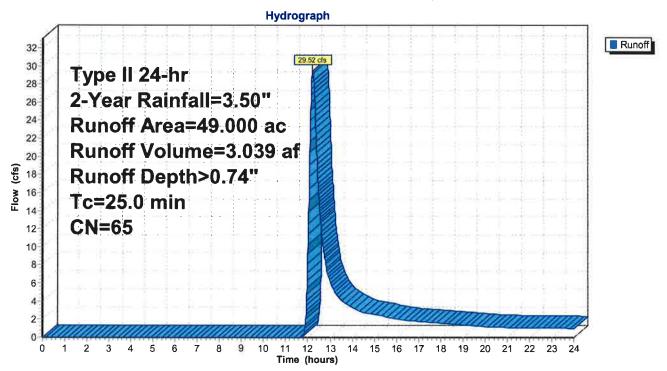
Summary for Subcatchment 1S: PreDeveloped

Runoff = 29.52 cfs @ 12.22 hrs, Volume= 3.039 af, Depth> 0.74"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-24.00 hrs, dt= 0.01 hrs Type II 24-hr 2-Year Rainfall=3.50"

	Area	(ac)	CN	Desc	cription		
*	49.	000	65				
	49.000			100.	00% Pervi	ous Area	
	Тс			Slope	•	Capacity	Description
-	(min)	(fee	et)	(ft/ft)	(ft/sec)	(cfs)	
	25.0						Direct Entry

Subcatchment 1S: PreDeveloped



Summary for Subcatchment 2S: Post Developed

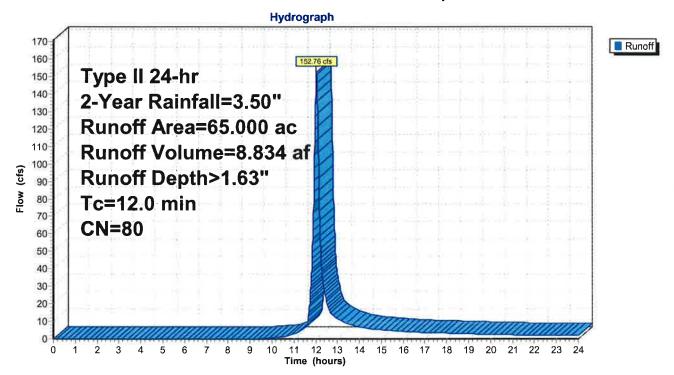
Runoff = 152.76 cfs @ 12.04 hrs, Volume=

8.834 af, Depth> 1.63"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-24.00 hrs, dt= 0.01 hrs Type II 24-hr 2-Year Rainfall=3.50"

	Area	(ac)	CN	Desc	cription			
*	30.	000	98					
*	35.	000	65					
2	65.	000	80	Weig	ghted Aver	age		
	35.000 53.85% Pervious Area							
	30.000			46.15% Impervious Area				
	Тс	Leng	th :	Slope	Velocity	Capacity	Description	
_	(min)	(fee	et)	(ft/ft)	(ft/sec)	(cfs)		
-	12.0						Direct Entry.	

Subcatchment 2S: Post Developed



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Summary for Pond 3P: Pond

Inflow Area = 65.000 ac, 46.15% Impervious, Inflow Depth > 1.63" for 2-Year event

Inflow = 152.76 cfs @ 12.04 hrs, Volume= 8.834 af

Outflow = 28.15 cfs @ 12.38 hrs, Volume= 8.387 af, Atten= 82%, Lag= 20.4 min

Primary = 28.15 cfs @ 12.38 hrs, Volume= 8.387 af

Routing by Stor-Ind method, Time Span= 0.00-24.00 hrs, dt= 0.01 hrs Peak Elev= 103.15' @ 12.38 hrs Surf.Area= 1.236 ac Storage= 3.519 af

Plug-Flow detention time= 88.1 min calculated for 8.383 af (95% of inflow)

Center-of-Mass det. time= 60.2 min (898.7 - 838.5)

Volume	Inv	ert A	vail.Stora	ige S	Storage Description					
#1	100.0	00'	14.250	af C	Custom Stage Data (Prismatic) Listed below (Recalc)					
				_						
Elevation	on Su	ırf.Area	In	c.Store	e Cum.Store					
(fee	et)	(acres)	(ac	re-feet)	t) (acre-feet)					
100.0	00	1.000		0.000	0.000					
102.0	00	1.150		2.150	0 2.150					
104.0	0	1.300		2.450	0 4.600					
106.0	00	1.500		2.800	0 7.400					
108.0	0	1,700		3.200	0 10.600					
110.0	00	1.950		3.650	0 14.250					
Device	Routing		Invert	Outlet	et Devices					
#1	Primary		100.00'	48.0"	Round Culvert					
	•			L= 20	00.0' RCP, groove end projecting, Ke= 0.200					
				Inlet /	/ Outlet Invert= 100.00' / 95.00' S= 0.0250 '/' Cc= 0.900					
				n = 0.0	013, Flow Area= 12.57 sf					
#2	Device 1	evice 1 100.00'		15.0"	15.0" Vert. Orifice/Grate X 3.00 C= 0.600					

72.0" Horiz. Orifice/Grate C= 0.600

Limited to weir flow at low heads

Primary OutFlow Max=28.16 cfs @ 12.38 hrs HW=103.15' (Free Discharge)

-1=Culvert (Passes 28.16 cfs of 80.10 cfs potential flow)

105.00

2=Orifice/Grate (Orifice Controls 28.16 cfs @ 7.65 fps)

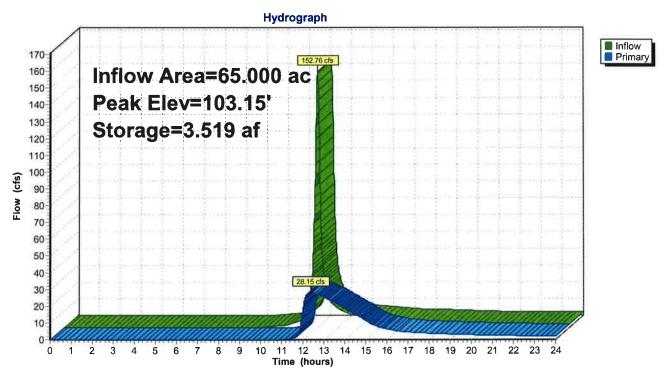
-3=Orifice/Grate (Controls 0.00 cfs)

#3

Device 1

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Pond 3P: Pond



Type II 24-hr 10-Year Rainfall=5.50" Printed 8/8/2015

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Time span=0.00-24.00 hrs, dt=0.01 hrs, 2401 points Runoff by SCS TR-20 method, UH=SCS, Weighted-CN Reach routing by Stor-Ind+Trans method - Pond routing by Stor-Ind method

Subcatchment 1S: PreDeveloped

Runoff Area=49.000 ac 0.00% Impervious Runoff Depth>1.98"

Tc=25.0 min CN=65 Runoff=91.51 cfs 8.079 af

Subcatchment 2S: Post Developed

Runoff Area=65.000 ac 46.15% Impervious Runoff Depth>3.32"

Tc=12.0 min CN=80 Runoff=309.01 cfs 18.003 af

Pond 3P: Pond

Peak Elev=105.86' Storage=7.192 af Inflow=309.01 cfs 18.003 af

Outflow=89.82 cfs 17.405 af

Total Runoff Area = 114.000 ac Runoff Volume = 26.081 af Average Runoff Depth = 2.75" 73.68% Pervious = 84.000 ac 26.32% Impervious = 30.000 ac

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Summary for Subcatchment 1S: PreDeveloped

Runoff

=

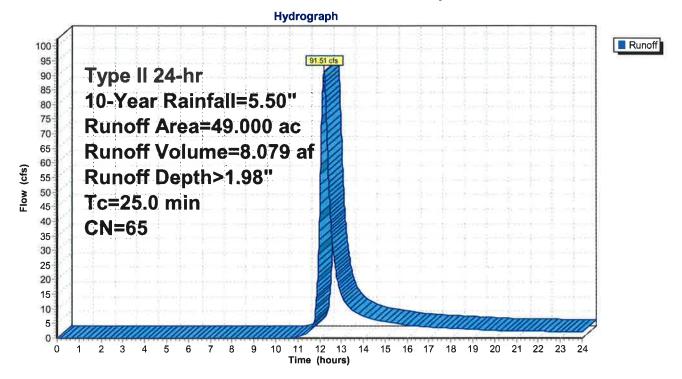
91.51 cfs @ 12.20 hrs, Volume=

8.079 af, Depth> 1.98"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-24.00 hrs, dt= 0.01 hrs Type II 24-hr 10-Year Rainfall=5.50"

	Area	(ac)	CN	Desc	cription		
*	49.	000	65				
8	49.000			100.	00% Pervi	ous Area	
	Tc			Slope	Velocity	Capacity	Description
-	(min) 25.0	(fee	∋τ)	(ft/ft)	(ft/sec)	(cfs)	Direct Entry

Subcatchment 1S: PreDeveloped



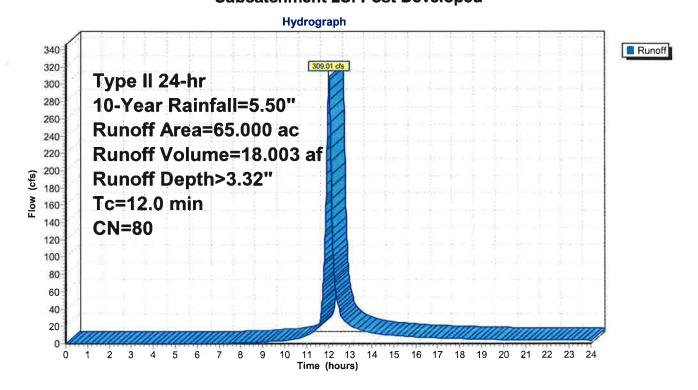
Summary for Subcatchment 2S: Post Developed

Runoff = 309.01 cfs @ 12.04 hrs, Volume= 18.003 af, Depth> 3.32"

Runoff by SCS TR-20 method, UH=SCS, Weighted-CN, Time Span= 0.00-24.00 hrs, dt= 0.01 hrs Type II 24-hr 10-Year Rainfall=5.50"

	Area	(ac)	CN	Desc	cription			
*	30.	000	98					
*	35.	000	65					
	65.	000	80	Weig	hted Aver	age		
	35.000 53.85% Pervious Area					us Area		
	30.000			46.15% Impervious Area				
	Тс	Lengt	h S	Slope	Velocity	Capacity	Description	
_	(min)	(feet	r)(r	(ft/ft)	(ft/sec)	(cfs)		_
	12.0						Direct Entry,	

Subcatchment 2S: Post Developed



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Summary for Pond 3P: Pond

65.000 ac, 46.15% Impervious, Inflow Depth > 3.32" for 10-Year event Inflow Area =

Inflow 309.01 cfs @ 12.04 hrs, Volume= 18.003 af

Outflow 17.405 af, Atten= 71%, Lag= 12.8 min 89.82 cfs @ 12.25 hrs, Volume=

Primary 89.82 cfs @ 12.25 hrs, Volume= 17.405 af

Routing by Stor-Ind method, Time Span= 0.00-24.00 hrs, dt= 0.01 hrs Peak Elev= 105.86' @ 12.25 hrs Surf.Area= 1.486 ac Storage= 7.192 af

Plug-Flow detention time= 88.1 min calculated for 17.398 af (97% of inflow)

Center-of-Mass det. time= 68.8 min (887.2 - 818.4)

Volume	Invert	Avail.Stora	ge Stor	age Description	
#1	100.00'	14.250	af Cus	tom Stage Data	(Prismatic) Listed below (Recalc)
Elevatio	n Surf.Ar	rea In	c.Store	Cum.Store	
(fee	t) (acre	es) (ac	re-feet)	(acre-feet)	
100.0	0 1.0	00	0.000	0.000	
102.0	0 1.1	50	2.150	2.150	
104.0	0 1.3	00	2.450	4.600	
106.0	0 1.5	00	2.800	7.400	
108.0	0 1.7	00	3.200	10.600	
110.0	0 1.9	50	3.650	14.250	
Device	Routing	Invert	Outlet D	evices	
#1	Primary	100.00'	48.0" R	ound Culvert	
	-		L= 200.0)' RCP, groove	end projecting, Ke= 0.200
			Inlet / Ou	utlet Invert= 100	.00' / 95.00' S= 0.0250 '/' Cc= 0.900
			n = 0.013	B, Flow Area= 1:	2.57 sf
#2	Device 1	100.00'	15.0" Ve	rt. Orifice/Grate	X 3.00 C= 0.600
#3	Device 1	105.00'	72.0" Ho	riz. Orifice/Gra	te C= 0.600
			Limited t	o weir flow at lo	w heads

Primary OutFlow Max=89.75 cfs @ 12.25 hrs HW=105.86' (Free Discharge)

-1=Culvert (Passes 89.75 cfs of 148.60 cfs potential flow)

2=Orifice/Grate (Orifice Controls 40.56 cfs @ 11.02 fps)

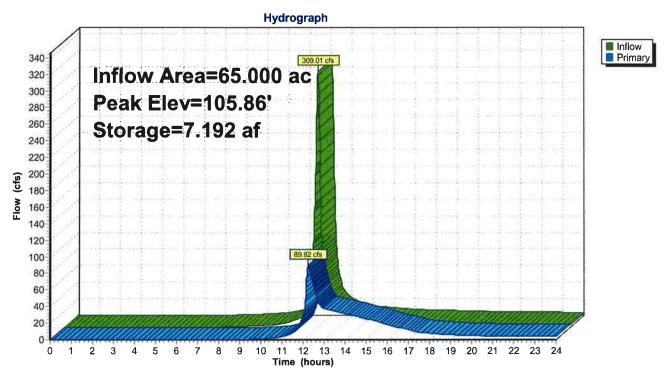
-3=Orifice/Grate (Weir Controls 49.19 cfs @ 3.03 fps)

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Pond 3P: Pond



K – Pollutant Load Calculations

Option 1

Worksheet 2: Situation 2

Page 1 of 4

Summary of Situation 2 criteria: from calculation procedure STEP 1 thru STEP 3, Worksheet 1:

Applicable area
$$(A)^* = 169$$
 acres

 I_{post} = (total post-development impervious cover ÷ A) × 100 = 34.14 %

$$I_{\text{watershed}} = \frac{\%}{\text{or}} \text{ or } I_{\text{watershed}} = 16\%$$

 $I_{\text{existing}} = (\text{total existing impervious cover} \div A^*) \times 100 = \underline{\hspace{1cm}}$

$$I_{\text{existing}} = 0 \% \le I_{\text{watershed}} = 6 \%$$
; and

STEP 4 Determine the relative pre-development pollutant load (L_{pre}).

$$\mathbf{L}_{\text{pre(watershed)}} = [0.05 + (0.009 \times I_{\text{watershed}})] \times A \times 2.28 \quad \text{(Equation 5-16)}$$

where: $L_{pre(watershed)}$ = relative pre-development total phosphorous load (pounds per year)

average land cover condition for specific watershed or locality or the Chesapeake Bay default value of 16% (percent expressed in whole numbers)

A = applicable area (acres)

$$L_{\text{pre(watershed)}} = [0.05 + (0.009 \times 16)] \times 169 \times 2.28$$

= 74.75 pounds per year

Worksheet 2: Situation 2

Page 2 of 4

STEP 5 Determine the relative post-development pollutant load (L_{post}).

$$L_{post} = [0.05 + (0.009 \times I_{post})] \times A \times 2.28$$
 (Equation 5-21)

where:

 L_{post} = relative post-development total phosphorous load (pounds per

year)

I_{post} = post-development percent impervious cover (percent expressed in

whole numbers)
A = applicable area (acres)

 $L_{\text{nost}} = [0.05 + (0.009 \times 34.14)] \times 169 \times 2.28$

= <u>137.66</u> pounds per year

STEP 6 Determine the relative pollutant removal requirement (RR).

$$\mathbf{RR} = \mathbf{L}_{\text{post}} - \mathbf{L}_{\text{pre(watershed)}}$$

= <u>62.91</u> pounds per year

STEP 7 Identify best management practice (BMP) for the site.

1. Determine the required pollutant removal efficiency for the site:

$$EFF = (RR \div L_{post}) \times 100$$

(Equation 5-22)

where:

EFF = required pollutant removal efficiency (percent expressed in whole numbers)

RR = pollutant removal requirement (pounds per year)

 L_{post} = relative post-development total phosphorous load (pounds per vear)

= _____%

Option a

Worksheet 2: Situation 2

Page 1 of 4

Summary of Situation 2 criteria: from calculation procedure STEP 1 thru STEP 3, Worksheet 1:

Applicable area $(A)^* = 169$ acres

 I_{post} = (total post-development impervious cover ÷ A) × 100 = 33.17 %

 $I_{watershed} =$ or $I_{watershed} = 16\%$

I_{existing} $0\% \le I_{watershed}$ 16 %; and

 I_{post} 33.17 % $> I_{watershed}$ 16 %

STEP 4 Determine the relative pre-development pollutant load (L_{pre}).

 $\mathbf{L}_{\text{pre(watershed)}} = [0.05 + (0.009 \times I_{\text{watershed}})] \times A \times 2.28 \quad \text{(Equation 5-16)}$

where: $L_{pre(watershed)}$ = relative pre-development total phosphorous load (pounds per year)

atershed = average land cover condition for specific watershed or locality or the Chesapeake Bay default value of 16% (percent expressed in whole numbers)

A = applicable area (acres)

 $\mathbf{L}_{\text{pre(watershed)}} = [0.05 + (0.009 \times 16)] \times 169 \times 2.28$

= $\frac{74.75}{}$ pounds per year

Worksheet 2 : Situation 2 Page 2 of 4

STEP 5 Determine the relative post-development pollutant load (L_{post}).

$$L_{post}$$
 = $[0.05 + (0.009 \times I_{post})] \times A \times 2.28$ (Equation 5-21)

where:

L_{post} = relative post-development total phosphorous load (pounds per

I_{post} = post-development percent impervious cover (percent expressed in whole numbers)

A = applicable area (acres)

$$L_{post} = [0.05 + (0.009 \times 33.17)] \times 169 \times 2.28$$

= 134.30 pounds per year

STEP 6 Determine the relative pollutant removal requirement (RR).

$$RR = L_{post} - L_{pre(watershed)}$$

$$RR = 134.30 - 74.75$$

$$= \underline{59.55} \text{ pounds per year}$$

STEP 7 Identify best management practice (BMP) for the site.

1. Determine the required pollutant removal efficiency for the site:

EFF =
$$(RR \div L_{post}) \times 100$$
 (Equation 5-22)

where: EFF = required pollutant removal efficiency (percent expressed in whole numbers)

RR = pollutant removal requirement (pounds per year)

 L_{post} = relative post-development total phosphorous load (pounds per year)

EFF = $(\div) \times 100$

Option 3

Worksheet 2: Situation 2

Page 1 of 4

Summary of Situation 2 criteria: from calculation procedure STEP 1 thru STEP 3, Worksheet 1:

Applicable area $(A)^* = 169$ acres

 I_{post} = (total post-development impervious cover ÷ A) × 100 = 32.16 %

 $I_{watershed} = _____ % or I_{watershed} = 16\%$

 $I_{\text{existing}} = (\text{total existing impervious cover} \div A^*) \times 100 = 6$

 I_{existing} \bigcirc $\% \le I_{\text{watershed}}$ \bigcirc %; and

Ipost 32.16% > Iwatershed 16 %

STEP 4 Determine the relative pre-development pollutant load (L_{pre}).

 $\mathbf{L}_{\text{pre(watershed)}} = [0.05 + (0.009 \times I_{\text{watershed}})] \times A \times 2.28 \quad \text{(Equation 5-16)}$

where: L_{pre(watershed)} = relative pre-development total phosphorous load (pounds per year)

 average land cover condition for specific watershed or locality or the Chesapeake Bay default value of 16% (percent expressed in whole numbers)

A = applicable area (acres)

 $\mathbf{L}_{\text{pre(watershed)}} = [0.05 + (0.009 \times \mathbf{\bot G})] \times \mathbf{\bot G} \times 2.28$

= <u>74.75</u> pounds per year

Worksheet 2: Situation 2

Page 2 of 4

STEP 5 Determine the relative post-development pollutant load (L_{post}).

$$L_{post} = [0.05 + (0.009 \times I_{post})] \times A \times 2.28$$
 (Equation 5-21)

where:

 L_{post} = relative post-development total phosphorous load (pounds per

year)

 I_{post} = post-development percent impervious cover (percent expressed in

whole numbers)

A = applicable area (acres)

$$L_{post} = [0.05 + (0.009 \times 33.16)] \times 169 \times 2.28$$

= 130.79 pounds per year

STEP 6 Determine the relative pollutant removal requirement (RR).

$$RR = L_{post} - L_{pre(watershed)}$$

= 56.04 pounds per year

STEP 7 Identify best management practice (BMP) for the site.

1. Determine the required pollutant removal efficiency for the site:

$$\mathbf{EFF} = (RR \div L_{post}) \times 100$$

(Equation 5-22)

where:

EFF = required pollutant removal efficiency (percent expressed in whole numbers)

RR = pollutant removal requirement (pounds per year)

 L_{post} = relative post-development total phosphorous load (pounds per year)

Option 4

Worksheet 2: Situation 2

Page 1 of 4

Summary of Situation 2 criteria: from calculation procedure STEP 1 thru STEP 3, Worksheet 1:

Applicable area $(A)^* = 169$ acres

 I_{post} = (total post-development impervious cover ÷ A) × 100 = 32.01 %

 $I_{\text{watershed}} = \frac{\%}{\text{or}} I_{\text{watershed}} = 16\%$

 $I_{\text{existing}} = (\text{total existing impervious cover} \div A^*) \times 100 = 0$

 $I_{\text{existing}} \bigcirc \% \le I_{\text{watershed}} \bigcirc \%$; and

 I_{post} 32.01% > $I_{watershed}$ / 6 %

STEP 4 Determine the relative pre-development pollutant load (L_{pre}).

 $\mathbf{L}_{\text{pre(watershed)}} = [0.05 + (0.009 \times I_{\text{watershed}})] \times A \times 2.28 \quad \text{(Equation 5-16)}$

where: L_{pre(watershed)} = relative pre-development total phosphorous load (pounds per year)

average land cover condition for specific watershed or locality or the Chesapeake Bay default value of 16% (percent expressed in

whole numbers)

A = applicable area (acres)

 $L_{pre(watershed)} = [0.05 + (0.009 \times 16)] \times 169 \times 2.28$

= 74.75 pounds per year

Worksheet 2: Situation 2

Page 2 of 4

STEP 5 Determine the relative post-development pollutant load (L_{post}).

$$L_{post} = [0.05 + (0.009 \times I_{post})] \times A \times 2.28$$
 (Equation 5-21)

where:

L_{post} = relative post-development total phosphorous load (pounds per

I_{post} = post-development percent impervious cover (percent expressed in whole numbers)

A = applicable area (acres)

$$L_{post} = [0.05 + (0.009 \times 32.01)] \times 169 \times 2.28$$

= 130.31 pounds per year

STEP 6 Determine the relative pollutant removal requirement (RR).

$$RR = L_{post} - L_{pre(watershed)}$$

= 55.52 pounds per year

STEP 7 Identify best management practice (BMP) for the site.

1. Determine the required pollutant removal efficiency for the site:

$$\mathbf{EFF} = (RR \div L_{post}) \times 100$$

(Equation 5-22)

where:

EFF = required pollutant removal efficiency (percent expressed in whole numbers)

RR = pollutant removal requirement (pounds per year)

 L_{post} = relative post-development total phosphorous load (pounds per year)

OPTION 5

Worksheet 2: Situation 2

Page 1 of 4

Summary of Situation 2 criteria: from calculation procedure STEP 1 thru STEP 3, Worksheet 1:

Applicable area (A)* = $\frac{1}{2}$ acres

 I_{post} = (total post-development impervious cover ÷ A) × 100 = 34.14%

 $I_{\text{watershed}} = \frac{\%}{}$ or $I_{\text{watershed}} = 16\%$

I_{existing} % ≤ I_{watershed} %; and

Ipost 30.8 % > Iwatershed 16 %

STEP 4 Determine the relative pre-development pollutant load (L_{pre}).

 $\mathbf{L}_{\text{pre(watershed)}} = [0.05 + (0.009 \times \mathbf{I}_{\text{watershed}})] \times \mathbf{A} \times 2.28 \quad \text{(Equation 5-16)}$

where: $L_{pre(watershed)}$ = relative pre-development total phosphorous load (pounds per year) $I_{watershed}$ = average land cover condition for specific watershed or locality or

the Chesapeake Bay default value of 16% (percent expressed in

whole numbers)

A = applicable area (acres)

 $\mathbf{L}_{\text{pre(watershed)}} = [0.05 + (0.009 \times 1/2)] \times 1/29 \times 2.28$

= 74.75 pounds per year

Worksheet 2: Situation 2

Page 2 of 4

STEP 5 Determine the relative post-development pollutant load (L_{post}).

$$L_{post} = [0.05 + (0.009 \times I_{post})] \times A \times 2.28$$
 (Equation 5-21)

where:

L_{post} = relative post-development total phosphorous load (pounds per

I_{post} = post-development percent impervious cover (percent expressed in whole numbers)

A = applicable area (acres)

$$\mathbf{L}_{post} = [0.05 + (0.009 \times 30.82)] \times 169 \times 2.28$$

$$= 126.07 \text{ pounds per year}$$

<u>STEP 6</u> Determine the relative pollutant removal requirement (RR).

$$\mathbf{RR} = \mathbf{L}_{\text{post}} - \mathbf{L}_{\text{pre(watershed)}}$$

= 51.32 pounds per year

STEP 7 Identify best management practice (BMP) for the site.

1. Determine the required pollutant removal efficiency for the site:

$$EFF = (RR \div L_{post}) \times 100$$
 (Equation 5-22)

where:

EFF = required pollutant removal efficiency (percent expressed in whole numbers)

RR = pollutant removal requirement (pounds per year)

 L_{post} = relative post-development total phosphorous load (pounds per year)

Option 6

Worksheet 2: Situation 2

Page 1 of 4

Summary of Situation 2 criteria: from calculation procedure STEP 1 thru STEP 3, Worksheet 1:

Applicable area (A)* = $\frac{167}{2}$ acres

 I_{post} = (total post-development impervious cover ÷ A) × 100 = $\frac{32.79\%}{100}$

 $I_{\text{watershed}} = \frac{\%}{100} \text{ or } I_{\text{watershed}} = 16\%$

I_{existing} % \leq I_{watershed} / 6 %; and

Ipost 3C.7Cl% > Iwatershed 16 %

STEP 4 Determine the relative pre-development pollutant load (L_{pr}).

 $\mathbf{L}_{\text{pre(watershed)}} = [0.05 + (0.009 \times I_{\text{watershed}})] \times A \times 2.28 \quad \text{(Equation 5-16)}$

where: $L_{pre(watershed)}$ = relative pre-development total phosphorous load (pounds per year) $I_{watershed}$ = average land cover condition for specific watershed or locality or

the Chesapeake Bay default value of 16% (percent expressed in

whole numbers)

A = applicable area (acres)

 $L_{\text{pre(watershed)}} = [0.05 + (0.009 \times 16)] \times 187 \times 2.28$ = 82.71 pounds per year

Worksheet 2: Situation 2 Page 2 of 4

STEP 5 Determine the relative post-development pollutant load (L_{post}).

$$L_{post} = [0.05 + (0.009 \times I_{post})] \times A \times 2.28$$
 (Equation 5-21)

where:

 L_{post} = relative post-development total phosphorous load (pounds per

year)

 I_{post} = post-development percent impervious cover (percent expressed in

whole numbers)

A = applicable area (acres)

$$L_{post} = [0.05 + (0.009 \times 36.79)] \times 187 \times 2.28$$

= 162.49 pounds per year

STEP 6 Determine the relative pollutant removal requirement (RR).

$$\mathbf{RR} = \mathbf{L}_{post} - \mathbf{L}_{pre(watershed)}$$

= $\frac{79.78}{}$ pounds per year

STEP 7 Identify best management practice (BMP) for the site.

1. Determine the required pollutant removal efficiency for the site:

EFF =
$$(RR \div L_{post}) \times 100$$
 (Equation 5-22)

where:

EFF = required pollutant removal efficiency (percent expressed in whole numbers)

RR = pollutant removal requirement (pounds per year)

 L_{post} = relative post-development total phosphorous load (pounds per year)

L – Engineer's Opinion of Probably Cost

Waynesboro EDA Exit 96 - Summary of Total Costs September 2015

Engineer's Opinion of Probable Costs

Exit 96 - Summary of Total Costs	Low	High
Infrastructure Costs (Tier 4)		
Connector Road (VDOT Standard)	\$5,138,000	\$5,138,000
Landscaped Entrance / Signage	\$200,000	\$300,000
5' Asphalt Trail Along Road (3,000 LF x \$20/LF)	\$60,000	\$60,000
5' Asphalt Fitness Trail in Park (7,500 LF x \$20/LF)	\$150,000	\$150,000
10" Gravity Sewer	\$595,000	\$595,000
500,000 Gal Elevated Storage Tank	\$2,152,000	\$2,152,000
Water Booster Pump Station	\$317,000	\$317,000
12" Water Line	\$665,000	\$665,000
Total Infrastructure Costs (Tier 4 Status)	\$9,277,000	\$9,377,000
Total "Pad Ready" Site Development Costs (Tier 5)	\$9,179,000	\$11,764,000
Total Potential Costs (Tier 5 Status)	\$18,456,000	\$21,141,000

Waynesboro EDA Exit 96 - Summary of "Pad Ready" Costs for Master Plan Options September 2015

Engineer's Opinion of Probable Cost

	Total Costs for	Developable	Costs per
Grading Option	"Pad Ready" Sites	Acreage	Developable Acres
Option 1	\$11,764,000	100	\$117,640
Option 2	\$10,462,000	100	\$104,620
Option 3	\$10,023,000	100	\$100,230
Option 4	\$9,833,000	100	\$98,330
Option 5	\$9,179,000	100	\$91,790
Option 6	\$13,268,000	120	\$110,567

PROJECT BUDGET

Exit 96 Industrial Park Connector Road Waynesboro Economic Development Authority September 2015

Engineer's Opinion of Probable Costs

CONSTRUCTION COSTS			Category
Demolition and Earthwork	\$	1,064,000	CN
Storm Sewer and Hydraulics	\$ \$	271,150	CN
Pavement	\$	692,000	CN
Geometric, Guardrail, Pavement Markings, Misc.	\$	112,220	CN
Maintenance of Traffic	\$	26,394	CN
Erosion and Sediment Control / Seeding	\$	131,969	CN
At-Grade Railroad Crossing	\$	250,000	CN
12' x 12' Box Culvert	\$ \$ \$ \$ \$ \$ \$	250,000	CN
2 - SWM Basins (water quantity only)		300,000	CN
Sub-Total A:	\$	3,097,733	CN
OTHER BID COSTS			
Mobilization for Sub-Total A (Calculated per VDOT formulas)	\$	184,887	CN
Construction Surveying (1%)	\$	30,977	CN
Materials Testing (2%)	\$ \$ \$	61,955	CN
Sub-Total B:	\$	277,819	CN
Total Roadway Items (A + B):	\$	3,375,552	CN
Other Project/Administrative Costs			
Construction Engineering & Inspection (15%)	\$	506,333	CN
Contingency for Total Roadway Bid Items (20%)	\$	675,110	CN
Survey, Design, & Permitting (15% OF CONSTRUCTION)	\$	506,333	PE
VDOT Administration Costs (BUDGET)	\$	25,000	PE
Sub-Total C:	\$	1,712,776	
GRAND TOTAL (A+B+C) (ROUNDED)	\$	5,088,000	
Summary of Budget in VDOT 3 phased system]
Preliminary Engineering (PE)	\$	531,000	
Right of Way (RW)	\$	50,000	
Construction (CN)	\$	4,557,000	
TOTAL RECOMMENDED PROJECT BUDGET	\$	5,138,000	

Notes:

- 1. Cost estimate assumes the following:
- a. No RW acquisition or utility relocation costs required other than tie-in to Oak Lane & S. Delphine Road
- b. wetland mitigation is not required.
- C. LID/Infiltration BMP's are not required
- d. No federal funding included.

PROJECT BUDGET - DETAILED BREAKDOWN Exit 96 Industrial Park Connector Road Waynesboro EDA, VA

Itom		ON OF PROBABLE		Unit Price		Total
Item		Quantity	Unit	Unit Price		TOTAL
	ion and Earthwork			T 4 4 2 2 2 2 2 2 2	_	50,000,00
00112	Clearing and Grubbing	15	AC	\$ 4,000.00		60,000.00
51910	Saw Cut Existing Pavement	500	LF	\$ 8.00		4,000.00
00120	Regular Excavation	100000	CY	\$ 10.00		1,000,000.00
		Sub-Total for D	Demolition	and Earthwork:	\$	1,064,000.00
	ewer and Hydraulics					
00525	Concrete Class A3 Misc.	15	CY	\$ 700.00		10,500.00
01152	15" Conc. Pipe	50	LF	\$ 70.00		3,500.00
01180	18" Pipe	50	LF	\$ 80.00		4,000.00
01360	36" Pipe	450	LF	\$ 100.00		45,000.00
06361	36" End Section ES-1	3	EA	\$ 2,100.00		6,300.00
27552	NS No. 57 Stone	350	TON	\$ 30.00		10,500.00
27500	Geotextile Fabric	550	SY	\$ 7.00	\$	3,850.00
09185	Paved Ditch PG-2A	1200	SY	\$ 75.00	\$	90,000.00
00588	Underdrain UD-4	6500	LF	\$ 15.00	\$	97,500.00
		Sub-Total for Sto	orm Sewer	and Hydraulics:	\$	271,150.00
Paveme	ent					
16335	Asphalt Concrete Type SM-12.5D	1100	TON	\$ 125.00		137,500.00
16373	Asphalt Concrete Type IM-19.0A	1500	TON	\$ 115.00	\$	172,500.00
16397	Asphalt Concrete Type BM-25.0D	1500	TON	\$ 100.00	\$	150,000.00
16242	Aggregate Base Material Type 1, No. 21B	7200	TON	\$ 30.00	\$	216,000.00
16516	Flexible Pavement Planing Tie-In (0"-2")	4000	SY	\$ 4.00	\$	16,000.00
			Sub-Tota	al for Pavement:	: \$	692,000.00
Geomet	ric, Guardrail, Pavement Markings, Misc.					
12020	Std. Curb CG-2	650	LF	\$ 16.00	\$	10,400.00
13108	CG-12 Detectable Warning Surface	30	SY	\$ 225.00	\$	6,750.00
13315	Guardrail Terminal GR-11	6	EA	\$ 1,200.00	\$	7,200.00
13320	Guardrail GR-2	2500	LF	\$ 22.00	\$	55,000.00
13345	Alt. Breakaway Cable Term. (GR-9)	6	EA	\$ 2,800.00	\$	16,800.00
54032	Type B Class I Pavement Line Marking 4"	14000	LF	\$ 0.65	\$	9,100.00
54042	Type B Class I Pavement Line Marking 24"	15	ĹF	\$ 8.00	\$	120.00
54300	Pavement Message Marking Elongated Arrow Single	3	EA	\$ 150.00	\$	450.00
50108	Sign Panel	8	EA	\$ 800.00	Ś	6,400.00
	Sub-Total for Geor	netric, Guardrail,	Pavement	Markings, Misc.	\$	112,220.00
Mainter	nance of Traffic			U.		
	MOT - Lump Sum 1% of Project Total)	1.0%	-	\$ 26,393.70	\$	26,394.00
			for Mainte	nance of Traffic		26,394.00
Erosion	and Sediment Control / Seeding				7,,,	20,23 1100
	E&S - Lump Sum (5% of Project Total)	5.00%	-	\$ 131,968,50	Ś	131,969.00

Exit 96 Industrial Park
Waynesboro, VA
PAD GRADING COST ESTIMATE OPTION 1

OPTION 1			
Item	Quantity Unit	Unit Price	
Mobilization	1 LS	\$50,000	\$50,000
Construction Entrance	2 EA	\$2,500	\$5,000
Construction Road	3,223 LF	\$30	\$96,690
Diversion Dike	46,860 LF	\$10	\$468,600
Silt Fence	1,800 LF	\$5	\$9,000
Sediment Traps & Basins	17,290 CY	\$15	\$259,350
Clear and Grub	58 Ac	\$7,500	\$435,000
Top Soil Removal and Stockpile	73,400 CY	\$4	\$293,600
Excavation (Cut to Fill)	1,191,900 CY	\$6	\$7,151,400
Seeding and Site Stabilization	91 Ac	\$1,500	\$136,500
Subtotal			\$8,905,140
Construction Contingency		15%	\$1,335,771
Subtotal - Construction Costs			\$10,240,911
Engineering		5.0%	\$512,046
Construction Administration, Legal, E	Bidding, Etc.	2.0%	\$204,818
Construction Inspections and Materi	als testing	2.5%	\$256,023
Construction Staking			\$50,000
Bad Soils/ Site Stabilization Allowar	oce		\$500,000
Total - Site Development Costs			\$11,764,000
Costs per Developable Acre		100 acres	\$117,640

Exit 96 Industrial Park
Waynesboro, VA
PAD GRADING COST ESTIMATE OPTION 2

OPTION 2			
Item	Quantity Unit	Unit Price	
Mobilization	1 LS	\$50,000	\$50,000
Construction Entrance	2 EA	\$2,500	\$5,000
Construction Road	3,223 LF	\$30	\$96,690
Diversion Dike	45,390 LF	\$10	\$453,900
Silt Fence	2,800 LF	\$5	\$14,000
Sediment Traps & Basins	16,850 CY	\$15	\$252,750
Clear and Grub	57 Ac	\$7,500	\$427,500
Top Soil Removal and Stockpile	73,400 CY	\$4	\$293,600
Excavation (Cut to Fill)	1,038,600 CY	\$6	\$6,231,600
Seeding and Site Stabilization	91 Ac	\$1,500	\$136,500
Subtotal			\$7,961,540
Construction Contingency		15%	\$1,194,231
Subtotal - Construction Costs			\$9,155,771
Engineering		5.0%	\$398,077
Construction Administration, Lega	l, Bidding, Etc.	2.0%	\$159,231
Construction Inspections and Mate	erials testing	2.5%	\$199,039
Construction Staking			\$50,000
Bad Soils / Site Stabilization Allov	vance		\$500,000
Total - Site Development Costs			\$10,462,000
Cost per Developable Acre		100 acres	\$104,620

Exit 96 Industrial Park Waynesboro, VA PAD GRADING COST ESTIMATE OPTION 3

OPTION 3				
Item	Quantity	Unit	Unit Price	
Mobilization	1	LS	\$50,000	\$50,000
Construction Entrance	2	EA	\$2,500	\$5,000
Construction Road	3,223	LF	\$30	\$96,690
Diversion Dike	17,000	LF	\$10	\$170,000
Silt Fence	1,870	LF	\$5	\$9,350
Sediment Traps & Basins	16,650	CY	\$15	\$249,750
Clear and Grub	52	Ac	\$7,500	\$390,000
Top Soil Removal and Stockpile	69,370	CY	\$4	\$277,480
Excavation (Cut to Fill)	1,038,600	CY	\$6	\$6,231,600
Seeding and Site Stabilization	86	Ac	\$1,500	\$129,000
Subtotal				\$7,608,870
Construction Contingency			15%	\$1,141,331
Subtotal - Construction Costs				\$8,750,201
Engineering			5.0%	\$380,444
Construction Administration, Legal, Bio	dding, Etc.		2.0%	\$152,177
Construction Inspections and Material	s testing		2.5%	\$190,222
Construction Staking				\$50,000
Bad Soils / Site Stabilization Allowand	e			\$500,000
Total - Site Development Costs				\$10,023,000
Costs per Developable Acre			100 acres	\$100,230

Exit 96 Industrial Park Waynesboro, VA PAD GRADING COST ESTIMATE OPTION 4

OPTION 4			
ltem	Quantity Unit	t Unit Price	
Mobilization	1 LS	\$50,000	\$50,000
Construction Entrance	2 EA	\$2,500	\$5,000
Construction Road	3,223 LF	\$30	\$96,690
Diversion Dike	12,750 LF	\$10	\$127,500
Silt Fence	1,880 LF	\$5	\$9,400
Sediment Traps & Basins	11,000 CY	\$15	\$165,000
Clear and Grub	51 Ac	\$7,500	\$382,500
Top Soil Removal and Stockpile	68,570 CY	\$4	\$274,280
Excavation (Cut to Fill)	1,036,400 CY	\$6	\$6,218,400
Seeding and Site Stabilization	85 Ac	\$1,500	\$127,500
Subtotal			\$7,456,270
Construction Contingency		15%	\$1,118,441
Subtotal - Construction Costs			\$8,574,711
Engineering		5.0%	\$372,814
Construction Administration, Legal	l, Bidding, Etc.	2.0%	\$149,125
Construction Inspections and Mate	erials testing	2.5%	\$186,407
Construction Staking			\$50,000
Bad Soils Allowance			\$500,000
Total - Site Development Costs			\$9,833,000
Costs per Developable Acre		100 acres	\$98,330

Exit 96 Industrial Park Waynesboro, VA PAD GRADING COST ESTIMATE OPTION 5

OPTION 6				
Item	Quantity	Unit	Unit Price	
Mobilization	1	LS	\$50,000	\$50,000
Construction Entrance	2	EA	\$2,500	\$5,000
Construction Road	3,223	LF	\$30	\$96,690
Diversion Dike	11,590	LF	\$10	\$115,900
Silt Fence	1,430	LF	\$5	\$7,150
Sediment Traps & Basins	12,685	CY	\$15	\$190,275
Clear and Grub	39	Ac	\$7,500	\$292,500
Top Soil Removal and Stockpile	57,275	CY	\$4	\$229,100
Excavation (Cut to Fill)	959,836	CY	\$6	\$5,759,016
Seeding and Site Stabilization	71	Ac	\$1,500	\$106,500
Subtotal				\$6,852,131
Construction Contingency			15%	\$1,027,820
Subtotal - Construction Costs				\$7,879,951
Engineering			5.0%	\$393,998
Construction Administration, Lega	l, Bidding, Etc.		2.0%	
Construction Inspections and Mate			2.5%	\$196,999
Construction Staking	J			\$50,000
Bad Soils / Site Stabilization Allow	vance			\$500,000
Total - Site Development Costs				\$9,179,000
Costs per Developable Acres			100 acres	\$91,790

Exit 96 Industrial Park Waynesboro, VA PAD GRADING COST ESTIMATE OPTION 6 WITH ADDITIONAL PARCEL

OPTION 5				
Item	Quantity	Unit	Unit Price	
Mobilization	1	LS	\$50,000	\$50,000
Construction Entrance	2	EA	\$2,500	\$5,000
Construction Road	3,223	LF	\$30	\$96,690
Diversion Dike	16,390	LF	\$10	\$163,900
Silt Fence	2,090	LF	\$5	\$10,450
Sediment Traps & Basins	19,185	CY	\$15	\$287,775
Clear and Grub	66	Ac	\$7,500	\$495,000
Top Soil Removal and Stockpile	80,670	CY	\$4	\$322,680
Excavation (Cut to Fill)	1,438,900	CY	\$6	\$8,633,400
Seeding and Site Stabilization	100	Ac	\$1,500	\$150,000
Subtotal				\$10,214,895
Construction Contingency			15%	\$1,532,234
Subtotal - Construction Costs				\$11,747,129
Engineering			5.0%	\$510,745
Construction Administration, Lega	al, Bidding, Etc	ş	2.0%	\$204,298
Construction Inspections and Mar	terials testing		2.5%	\$255,372
Construction Staking				\$50,000
Bad Soils / Site Stabilization Allo	wance			\$500,000
Total - Site Development Costs				\$13,268,000
Costs per Developable Acre			120 acres	\$110,566.67

Waynesboro Exit 96 Industrial Park Conceptual Utility Plan Gravity Sewer Collection

Probable	Construction Cost					
Item	Description	Quantity	Units	Unit \$		Total Cost
1	10" Gravity Sewer, PVC	4,500	LF	\$80		\$360,000
2	Manhole	17	EA	\$3,500		\$59,500
3	Manhole Frame & Cover	17	EA	\$350		\$5,950
4A	Jack and Bore Road Crossing	0	EA			
4B	Jack and Bore Road Crossing	0	LF	\$350		\$0
5A	Railroad Crossing	1	EA			
5B	Railroad Crossing	100	LF	\$350		\$35,000
6A	Creek Crossing (HDD)	0	EA			
6B	Creek Crossing (HDD)	0	LF	\$225		\$0
					Sub-total	\$460,450
	Construction Contingency					\$46,045
			Total Pr	obable Const	ruction Cost	\$506,495
Professio	onal Services Prior to Construction			- 0	625	
			Design B	Engineering	8.0%	\$40,520
		Utility	Location	and Survey	3.0%	\$15,195
	Wetland Delineation/ Environment	onmental Ass	sessment.	/ Permitting	1.0%	\$5,065
	Easeme	nt Acquisition	and Ease	ement Plats	0.0%	\$0
		Leg	al and Ad	ministration	0.0%	\$0
					Sub-total	\$60,779
Profession	onal Services During Construction					
	Construction Admir	nistration / Co	nstruction	Inspection	3.5%	\$17,727
	Materials Testing					\$5,065
	Construction Survey					\$5,065
					Sub-total	\$27,857
	Total Probable Project Cost \$595,00					

^{*}Assuming 100 LF per Railroad Crossing

Waynesboro Exit 96 Industrial Park Conceptual Utility Plan Water Distribution

Probable	Construction Cost					
Item	Description	Quantity	Units	Unit \$		Total Cost
1	12" Water Main, PVC	4,660	LF	\$85		\$396,100
2	12" Gate Valves	5	EA	\$4,000		\$20,000
3	16" Water Main, PVC	0	LF	\$125		\$0
4	16" Gate Valves	0	EA	\$6,500		\$0
5	Air Release Valves	7	EA	\$4,800		\$33,600
6	Connection to Existing Water Main	2	EA	\$5,000		\$10,000
7	Fire Hydrant	5	EA	\$4,000		\$20,000
8A	Directional Drill Creek Crossing	0	EA			\$0
8B	Directional Drill Creek Crossing	0	LF	\$150		\$0
9A	Jack and Bore Road Crossing	0	EA			\$0
9B	Jack and Bore Road Crossing	0	LF	\$350		\$0
10A	Railroad Crossing	1	EA		l.	\$0
10B	Railroad Crossing	100	LF	\$350		\$35,000
11	Master Meter	0	EA	\$50,000		\$0
12	Control Valve	0	EA	\$20,000		\$0
13	Service Reconnections	0	EA	\$1,750		\$0
14	New Service Connections	0	EA	\$4,000		\$0
					Sub-total	\$514,700
		Con	struction C	Contingency	10%	\$51,470
			Total Pr	obable Cons	truction Cost	\$566,170
Profession	onal Services Prior to Construction					
				Engineering	8.0%	\$45,294
		Utility	/ Location	and Survey	3.0%	\$16,985
	Wetland Delineation/ Envir	onmental Ass	sessment	/ Permitting	1.0%	\$5,662
		Leg	gal and Ad	ministration	0.0%	\$0
					Sub-total	\$67,940
Profession	onal Services During Construction			w	190	
	Construction Admir	nistration / Co	onstruction	n Inspection	3.5%	\$19,816
				ials Testing	1.0%	\$5,662
			Construc	tion Survey	1.0%	\$5,662
					Sub-total	\$31,139
		То	tal Pro	bable Pro	ject Cost	\$665,000

^{*}Assuming 100 LF per Railroad Crossing

Waynesboro Exit 96 Industrial Park Conceptual Utility Plan Water Booster Pump Station

Co-located with Storage Tank

Probable	Construction Cost					
Item	Description	Quantity	Units	Unit \$		Total Cost
1	Mobilization	1	LS	\$15,000		\$15,000
2	Building	1	LS	\$50,000		\$50,000
3	Duplex Skid Mounted Pumps					
4	Pump Controls	1	LS	\$60,000		\$60,000
5	Piping and Valves					
6	SCADA and Instrumentation	1	LS	\$5,000		\$5,000
7	Electrical Service	1	LS	\$25,000		\$25,000
8	Electrical & Mechanical	1	LS	\$30,000		\$30,000
9	Standby Power - Generator	1	LS	\$0		\$0
10	Site Work	1	LS	\$35,000		\$35,000
11	Site Piping and Valves	1	LS	\$25,000		\$25,000
			Sub-total	\$245,000		
		Contingency	10%	\$24,500		
	Total Probable Con					\$269,500
Profession	onal Services Prior to Construction			91	10.0%	
	Design Engineering					\$26,950
		and Survey	1.0% 1.0%	\$2,695		
l	Wetland Delineation/ Environmental Assessment / Permitting					\$2,695
Easement Acquisition and Easement Plats					0.0%	\$0
Legal and Administration					0.0% Sub-total	\$0
						\$32,340
Profession	onal Services During Construction				3.5%	
l	Construction Administration / Construction Inspection					\$9,433
l	Materials Testing					\$2,695
	Construction Survey					\$2,695
					Sub-total	\$14,823
	Total Probable Project Cost \$317,0					

Waynesboro Exit 96 Industrial Park Conceptual Utility Plan Elevated Water Storage Tank (500,000 Gal)

Probable	Construction Cost						
Item	Description	Quantity	Units	Unit \$		Total Cost	
1	0.5 Million Gallon, Waterspheroid	1	LS	\$1,400,000		\$1,400,000	
	(Spread Footing)	1			1		
2	Site Work	1	LS	\$150,000		\$150,000	
3	Site Piping and Valves	1	LS	\$75,000		\$75,000	
4	SCADA and Instrumentation	1	LS	\$25,000		\$25,000	
5	Tideflex Mixing Tree	1	LS	\$15,000		\$15,000	
	*				Sub-total	\$1,665,000	
		10%	\$166,500				
			Total F	Probable Const	ruction Cost	\$1,831,500	
Profession	Professional Services Prior to Construction						
	Design Engineering					\$183,150	
l	Utility Location and Survey					\$18,315	
	Wetland Delineation/ Environmental Assessment / Permitting					\$18,315	
	Easement Acquisition and Easement Plats					\$0	
		0.0%	\$0				
					Sub-total	\$219,780	
Profession	onal Services During Construction						
	Construction Administration / Construction Inspection					\$64,103	
	Materials Testing					\$18,315	
			Constru	uction Survey	1.0%	\$18,315	
	Su					\$100,733	
	Total Probable Project Cost \$2						

M – Technical Memorandum – Analysis of Southern Corridor



TECHNICAL MEMORANDUM

Subject: Southern Corridor

City of Waynesboro

Design & Estimate Memo

October 25, 2013

This technical memorandum has been prepared to serve as an engineering analysis justifying the estimated costs associated with the Southern Corridor. The planning-level cost estimate was developed to a degree good enough to allow the City to add these projects to its Capital Improvements Plan and the VDOT Six-Year Plan. The schematic was developed using City GIS and 2011 aerial imagery.

PE PHASE		\$1,400,000
RW PHASE		\$1,000,000
CN PHASE		\$10,530,468
PROJECT TOTAL		\$12,930,468

Note: See technical memo for assumptions and exclusions. Unit prices are based off of 2013 VDOT bid tabulations and the Project Total does not account for eventual construction year inflation.



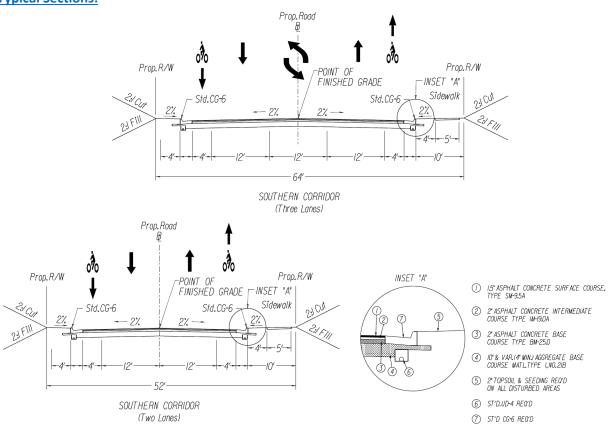


Alignment:

The Southern Corridor is positioned south of I-64 between Exits 94 & 96. The proposed alignment is approximately 1.6 miles and connects Shenandoah Village Drive to S. Delphine Avenue. The Southern Corridor starts at the end of existing Shenandoah Village Drive and runs eastward towards Lyndhurst Road. Once the Southern Corridor crosses Lyndhurst Road it runs southeast on the existing N. Oak lane and crosses the existing bridge over the South River. The road then continues on new alignment, crossing the Norfolk Southern Railroad and continuing southeast towards S. Delphine Avenue. It is anticipated that Lyndhurst Road and S. Delphine Avenue will also need to be widened to accommodate additional turning lanes.

The majority of the roadway will consist of three lanes, bike lanes in both directions, and a sidewalk on the southern side as seen in the typical section below. As an alternative to bike lanes and sidewalk, a shared use path could be utilized with minimal cost difference. The roadway narrows to a two lane typical section from the N. Oak Lane bridge spanning the South River to just beyond the proposed Norfolk Southern Railroad crossing. The estimate considers an at-grade railroad crossing; however, should a grade-separated railroad crossing be needed, we have provided a conceptual cost estimate (see "Assumptions").

Typical Sections:





Assumptions:

I. Roadway:

- 1. Design Criteria
 - a. Roadway Classification = Urban Collector (GS-7)
 - b. Design speed = 40mph ULS
 - c. Minimum Radius = 593' (ULS)
 - d. Lane width = 12' (11' plus 1' to accommodate anticipated truck traffic)
 - e. Superelevation = Normal Crown (ULS)
 - f. Pavement Section: 1.5" SM, 2" IM, 2" BM, 10" Subbase
- 2. The proposed alignment will connect Shenandoah Village Drive to S. Delphine Ave.
- 3. A three lane typical section will be utilized along the length of the corridor except from the existing bridge over the South River to the Norfolk Southern Railroad.
- 4. Pedestrian accommodations: For cost estimating purposes a 4' bike lane is proposed on both sides of the road with a 5' sidewalk on the southern side only.
- 5. Lyndhurst Rd. will be widened to accommodate left turns from each direction on to the Southern Corridor.
- 6. N. Oak Lane will be reconstructed/widened as a part of this project, since the existing road is a narrow two-lane road posted at 25 mph and is not suited for heavy trucks/higher volumes.
- 7. S. Delphine was widening to accommodate NB lefts and SB rights to the Southern Corridor.
- 8. The existing N. Oak Lane bridge over the South River will be utilized and the proposed road section will taper down to fit within the existing lane structure at this point.
- 9. Pipe (18") is assumed to run the length of one side with crossing every 350 LF. Inlets will be assumed every 350 LF down both sides.
- 10. Grading for potential stormwater management basins included in the lump sum price for Earthwork. We assumed four basins for this project.
- 11. Three 4' x 4' box culverts were assumed at a total length of 235 LF.

II. Traffic:

- 1. Two new signals are anticipated at Lyndhurst Rd. and S. Delphine Ave. at approximately \$250,000/signal. Additional traffic analysis will be necessary during preliminary engineering.
- 2. Signing and pavement markings are estimated at \$90,000/mile.
- 3. Maintenance of Traffic costs are estimated based on projects of similar magnitude. The majority of this is new alignment but there will be coordination needed of Lyndhurst Rd./N. Oak Ln./Norfolk Southern Railroad/S. Delphine Ave.
- 4. A new at-grade rail crossing is estimated at \$250,000 based on some planning-level cost estimates prepared for other localities within the past 6 months.

III. Environmental:

- 1. Stream impacts may require Compensatory Mitigation. For estimating purposes we assumed compensatory mitigation costs of \$400 per credit for stream impacts and \$70,000 per credit for wetland impacts. Currently in VA, the standard approved method to compensate for stream and wetland impacts is to purchase "credits" at a compensatory mitigation bank. For this estimate, we are assuming one linear foot of impact will require one credit and emergent wetlands will be compensated for at a 1:1 impact to compensatory mitigation ratio.
- 2. South River considered a Trout Stream (downriver) Time of Year Construction limitation may be required



- 3. South River considered Impaired by VDEQ downstream of project (through downtown Waynesboro) Water Quality Protection Measures may be required
- 4. VDCR database shows the Brook Floater, a state listed endangered species, as being present in the South River Survey for presence of species and Time of Year Construction limitation may be required
- Multiple threatened and endangered species listed in VDGIF and VDCR's databases are
 potentially present within 2 mile project area buffer Species and habitat surveys may be
 required
- 6. Portion of project in FEMA 100 year Floodplain (area surrounding South River)
- 7. Historical Archaeology Resources (VDHR ID 44AU0753 (Eligible for NHRP) and 44AU0754 (Not Eligible for NHRP)) on either side of South River Crossing
- 8. Karst topography present–sinkholes may be in area; survey may be required, especially if caves or other habitat for threatened/endangered bat species could be present

IV. ROW & Utilities:

- 1. Right of way is 10% of the total construction cost. This number may be modified depending on what the City owns or if they have any pre-arranged agreements with the developers to donate the land needed to install the Southern Corridor.
- 2. We anticipate at least 4 lots being significantly impacted along N. Oak Lane due to the widening.

V. General:

- 1. A grade-separated railroad cost estimate is being considered for planning purposes even though we recommend the at-grade crossing in our design. The grade-separated structure, including approach grades, would cost approximately \$1,800,000. The assumption is a 65' span that spans the railroad right of way and a width of 40'. The cost used is \$350/SF. A total cost for the grade-separated structure that accounts for contingencies, design, inspection, etc. is approximately \$3,600,000.
- 2. Additional roadway reconstruction efforts may be necessary for S. Delphine Ave. based on the vertical curvature and sight distance from the Southern Corridor.
- 3. The following items are not included in this estimate: Roadway lighting, landscaping, water, sanitary sewer and retaining walls.
- 4. A geotechnical report was not provided and therefore we did not consider elements that could affect the cost such as rock or unsuitable materials.
- 5. The "Environmental" bullet points have not been confirmed with the regulatory agencies. Information is based on preliminary analysis of GIS data. Federal Government Agency databases were not included in this preliminary analysis because websites were not operational as a result of the government shutdown.
- 6. Acronyms
 - a. ULS Urban Low Speed
 - b. FEMA Federal Emergency Management Agency
 - c. VDCR Virginia Department of Conservation and Recreation
 - d. VDEQ Virginia Department of Environmental Quality
 - e. VDGIF Virginia Department of Game and Inland Fisheries
 - f. VDHR Virginia Department of Historic Resources
- 7. CEI services are estimated at 15%



APPENDIX A: Estimate

SOUTHERN CORRIDOR					
PLANNING ESTIMATE				10/23/2013	
DESCRIPTION	UNIT	EST.	UNIT	AMOUNT	
CLEARING AND GRUBBING	LS	QUANTITY 1	PRICE (\$) \$50,000	(\$) \$50,000	
UNDERDRAIN UD-3	LF	18926	\$15	\$283,890	
UNDERDRAIN UD-4	LF	8200	\$15	\$123,000	
OUTLET PIPE	LF	500	\$25	\$12,500	
18" PIPE	LF	9352	\$80	\$748,160	
18" END SECTION ES-1 OR 2	EA	10	\$700	\$7,000	
MANHOLE MH-1 OR 2	LF	32	\$600	\$19,200	
FRAME AND COVER MH-1	EA	8	\$750	\$6,000	
DROP INLET, DI-3B, L=12'	EA	48	\$4,000	\$192,000	
SWM RISER (TO BE DESIGNED)	LF	20	\$1,500	\$30,000	
AGGR. BASE MAT'L TY. I NO. 21A	TON	1066	\$30	\$30,000	
AGGR. BASE MAT'L TY. I NO. 21B	TON	26039	\$30		
				\$781,156	
ASPH. CONC. BASE CR. TY. BM-25.0	TON	4410	\$115	\$507,192	
ASPH. CONC. TY. IM-19.0A	TON	4410	\$115	\$507,192	
FLEXIBLE PAVEMENT PLANING	SY	4891	\$6	\$29,345	
ASPHALT CONCRETE TY, SM-9.5A	TON	3846	\$125	\$480,721	
ST'D COMB. CURB & GUTTER CG-6	LF	18926	\$18	\$340,668	
CG-12 DETECTABLE WARNING SURFACE	SY	7.0	\$300	\$2,100	
HYDR. CEMENT CONCRETE 4"	SY	4555	\$40	\$182,200	
SAW CUT CURB, GUTTER AND ENTRANCES	LF	4000	\$3.00	\$12,000	
DEMOLITION OF PAVEMENT (FLEXIBLE)	SY	800	\$8.00	\$6,400	
BOX CULVERT (3 STREAM CROSSINGS)	LS	1	\$240,000	\$240,000	
PEDESTRIAN BRIDGE OVER SOUTH RIVER	LS	1	\$100,000	\$100,000	
EARTHWORK	LS	1	\$1,250,000	\$1,250,000	
EROSION AND SEDIMENT CONTROL	LS	1	\$100,000	\$100,000	
MOT	LS	1	\$100,000	\$100,000	
RAILROAD CROSSING (AT-GRADE)	LS	1	\$250,000	\$250,000	
TRAFFIC SIGNAL - LYNDHURST RD.	LS	1	\$250,000	\$250,000	
TRAFFIC SIGNAL - S. DELPHINE AVE.	LS	1	\$250,000	\$250,000	
SIGNING AND PAVEMENT MARKINGS	LS	1	\$140,000	\$140,000	
COMPENSATORY MITIGATION	LS	1	\$106,600	\$106,600	
SUBTOTAL				\$7,139,300	

PROJECT TOTAL (WITHOUT CONTINGENCY)	\$7,139,300
MOBILIZATION (10%)	\$713,930
CONSTR. SURVEY(2.5%)	\$178,483
CONTINGENCY (20%)	\$1,427,860
CEI (15%)	\$1,070,895
CN TOTAL (WITH CONTINGENCY)	\$10,530,468
PE PHASE	\$1,400,000
RW PHASE	\$1,000,000
CN PHASE	\$10,530,468
PROJECT TOTAL	\$12,930,468