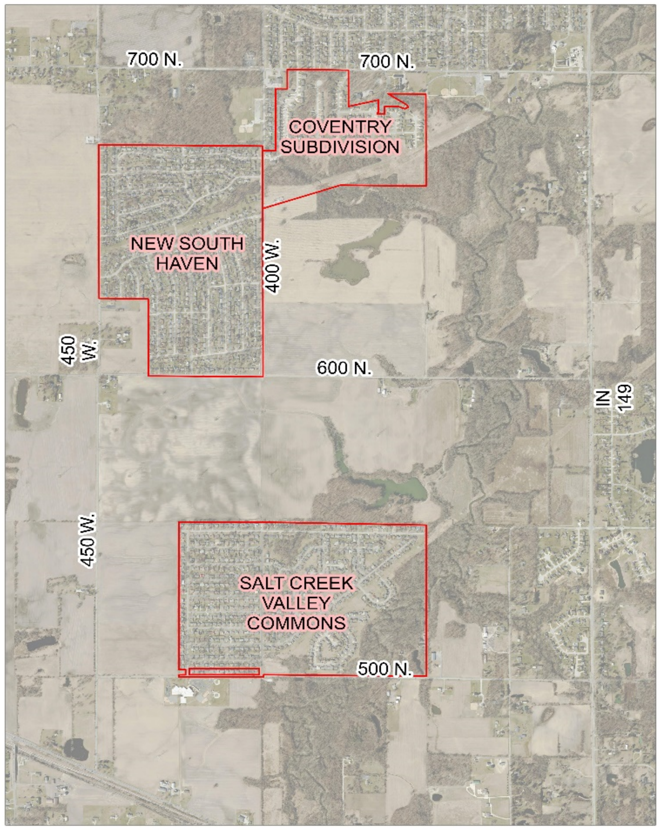
GREATER SOUTH HAVEN DRAINAGE STUDY-
COVENTRY, NEW SOUTH HAVEN AND SALT CREEK VALLEY COMMONS

Porter County, Indiana



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EXECUTIVE SUMMARY

The Porter County Department of Development and Storm Water Management (PCDDSWM) has undertaken a study of the stormwater infrastructure within the Coventry, New South Haven and Salt Creek Valley Commons subdivisions. Coventry, New South Haven and Salt Creek Valley Commons were constructed in the 1960's through the early 1990's and the age of the infrastructure is approaching 50 to 60 years. The primary pipe material of construction in New South Haven and Salt Creek Valley Commons subdivisions is galvanized corrugated metal pipe (CMP) and has a life expectancy of approximately 50 years (depending on specific site conditions). The majority of the pipes in the Coventry subdivision is high density polyethylene (HDPE). The areas have reported drainage issues, infrastructure failures (e.g. sinkholes) and were identified in 2010 County-wide Comprehensive Drainage Plan as areas of concern.

The study included video inspection of approximately 100,500 feet of storm sewer (approximately 19 miles) to assess the condition of the pipes. The video inspection confirmed the CMP pipe had visible corrosion and even areas of exposed soil. Standing water and sediment depth prevented video inspection in some instances. One pipe was almost completely corroded and was forming a sinkhole. The video inspection confirmed the majority of the CMP pipe was nearing the end of its service life.

The HDPE pipe in the Coventry subdivision displayed significant deformities, some so extreme the pipe was fractured. Inconsistent slopes were also frequently observed.

Hydrologic and hydraulic computer modeling was performed using the XPSWMM software to assess the potential flow capacity. Assuming the pipes were clean and in good shape geometrically, various storm events were run. The results found most of the pipes in all three subdivisions could convey the 10% annual exceedance probability (AEP) storm event ("the 10-year event) with the hydraulic gradeline (HGL) below the casting. The model was re-run with slightly reduced pipe diameters to account for possible lining. The most modified pipes continued to convey the 10% AEP event with the HGL below the casting validating possible lining activities.

Using both the video inspection and modeling results, multiple rehabilitation construction projects and costs were estimated within each subdivision for segments of the storm sewers to manage the overall budgeting. The following tables summarize the opinion of probable construction costs for each subdivision.

Summary of Opinions of Probable Cost – Coventry Subdivision	
Project ID	**Probable Cost**
Coventry	$4,700,000
New South Haven	$6,930,000
Salt Creek Valley Commons	$11,360,000
Total =	$22,990,000

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1.0 Introduction

The Porter County Department of Development and Storm Water Management (PCDDSWM) has undertaken a study of the stormwater infrastructure within areas of the greater South Haven area. The study area specifically includes the Coventry, New South Haven and Salt Creek Valley Commons subdivisions (see **Appendix I** for a map of the study areas).

The areas have reported drainage issues, infrastructure failures (e.g., sinkholes) and were identified in 2010 County-wide Comprehensive Drainage Plan as areas of concern.

2.0 Background and Overall Study Procedures Outline

An overview of the subdivisions history and the study procedure is provided in the following sections.

2.1 Background

Coventry, New South Haven and Salt Creek Valley Commons were constructed in the early 1960’s through the early 1990’s. During this period, stormwater conveyance, storage and design requirements were not as specific as current standards. Design storms, pipe capacity requirements, inlet casting requirements and detention design requirements have evolved and, in most cases, become more stringent. In addition to more demanding design requirements, materials of construction and installation standards have also improved. Inspection during construction is performed by the County or an independent contractor for current infrastructure construction projects but no records of inspection during construction activities for these subdivisions have been noted.

Currently, the subdivisions are included in the Twin Creeks Conservancy District and the District is responsible for flood control and drainage. Prior to the formation of the District on December 15, 1995, maintenance of the stormwater infrastructure was minimal. Maintenance after formation of the District included periodic repair of sinkholes occurring from failing pipe.

The age of the infrastructure in the New South Haven and Salt Creek Valley Commons subdivisions is approaching 50 to 60 years. The primary pipe material of construction is galvanized corrugated metal pipe (CMP) and has a life expectancy of approximately 50 years (depending on specific site conditions).

The primary material of construction in the Coventry subdivision storm sewer pipes is high density polyethylene pipe (HDPE) and the pipes are not as old as New South Haven and Salt Creek Valley Commons. However, HDPE pipe installation has more specific requirements relative to CMP and reinforced concrete pipe (RCP).

2.2 Video Investigation

Based on the age of the pipes and periodic need to repair sinkholes, an assessment of the current conditions of the storm sewers in all subdivisions was warranted. The current conditions of the storm sewers would indicate if replacement or lining was possible.

During the November and December of 2020 and January of 2021, approximately 100,500 feet (approximately 19 miles) of storm sewer were videoed in the three subdivisions.

Pipe material, size, geometry, sediment depth, standing water and condition of the pipes were among the observations noted. Video and pictures were part of the final work documentation. The specific results of the video inspection for each subdivision are discussed in Section 3.0, 4.0 and 5.0 below.

2.3 Hydrologic / Hydraulic Investigation

After assessing the pipe conditions, the next step in the investigation was to evaluate the potential runoff entering the storm sewer at each inlet watershed and the potential capacity of the existing storm sewers (assuming the pipes were in good condition). This information would also be a consideration in determining if lining or replacement would be chosen.

This stage of the study required a review of topographic data, watershed delineation, surface characterization, estimation of the time-of-concentration, pipe material and size. This information was then used to create a computer model of each storm sewer system and evaluate the hydraulic capacity of the existing pipes for each subdivision.

2.3.1 Topographic Data

Previous studies of the subdivisions including the 2010 County-wide Comprehensive Drainage Plan utilized the Porter County 2010 contour data. Since development within and adjacent to the subdivisions has been minimal since 2010 and to stay consistent with the 2010 County-wide Comprehensive Drainage Plan, the 2010 contour was used again for this investigation. As a quality control measure, contours generated from 2013 digital elevation maps (DEMs) were periodically compared. No significant differences were noted.

2.3.2 Watershed Delineation

Using the 2010 Porter County contour data, the off-site and on-site watersheds as well as the inlet subwatersheds were delineated for each subdivision. Watersheds for any contributing off-site depressional area were also delineated. Watershed maps for each subdivision are provided in **Appendices III A** (Coventry)**, IV A** (New South Haven) and **V A** (Salt Creek Valley Commons)**.**

2.3.3 Soil Types

The NCRS web soil survey was used to review the soil types within the subdivision boundaries. C soils are the predominate type with some C/D listings. With all subdivisions served by storm sewers and off-site agricultural fields served by field tiles, soils were assumed to be in the "drained" condition and subsequently C soils were assumed for all contributing runoff basin areas both on- and off-site.

2.3.4 Surface Characterization / Curve number Determination

The watershed curve numbers (CNs) for the lots within the subdivision were estimated using the aerial maps and *Table V-6* from the Porter County Stormwater Design Manual. The lots sizes are approximately ¼ acre for the majority of all three subdivisions. The east side of Salt Creek Valley Commons includes open area while there are still open areas and undeveloped areas within Coventry. These open areas could lower the curve number, but as a conservative measure, all watershed areas within the subdivisions were assumed to have the ¼ acre curve number for C soils (83).

The off-site areas contributing to New South Haven and Salt Creek Valley Commons were assumed to be cultivated, C soils with a CN of 88.

2.3.5 Times of Concentration

Times-of-concentration (Tc's) for the watersheds within the subdivision assumed to have a 10-minute Tc. This was considered conservative since runoff generally must pass from the rear-yards to the street. Off-site watershed Tc's were also estimated to be 10-mminutes since these areas were agricultural and the Tc's would vary depending on the time of the year and the surface conditions (bare earth or crop cover).

2.3.6 Stormwater Infrastructure – GIS Database

The pipe length and invert model input data were derived using the GIS based information from two sources, a Porter County Porter database provided to DLZ and supplemental field survey by DLZ for the structures missing from the database. DLZ understands the County data was collected by summer interns and inverts were obtained by measure-downs where the interns were able to access the inside of the structures.

Some existing structures included concrete lids and were inaccessible without equipment. During this study, the County changed the lids on these structures to cast iron, making them accessible with a manhole hook. DLZ then surveyed these structures and obtained inverts and pipe sizes.

DLZ created new pipe and structure databases, checking the data for inconsistencies, adjusting, and combining the County data and the DLZ field data into a single database with consistent formatting for each subdivision. A numbering system for each structure was created starting with each outfall and using a 4- or 5-digit number. This allowed each upstream structure to have the same starting digit(s) in the series, for example 1000's, 2000's, 3000's, etc. Pipes were then identified by the up- and downstream structures separated by a dash (e.g., 1001-1000). This nomenclature was repeated for each subdivision.

2.3.7 Computer Modeling

Using the CN, Tc and GIS structure and pipe databases discussed above, an XPSWMM model was created for each subdivision using the NRCS (SCS) methodology. This model was used with various storm events to evaluate the performance of the existing storm sewer system assuming uniform slopes, no sediment, and pipes in good condition.

Rainfall depths were obtained from the NOAA website for the 1%, 2%, 4% and 10% annual exceedance probability (AEP) events (i.e., the "100-, 50-, 25- and 10- year rainfall events"). The NOAA rainfall depths were considered more representative than the depths originally listed in the Porter County Design Manual and reflective of changing weather conditions. The Huff 50% rainfall distributions were used with NOAA rainfall depths. See **Appendix II A and B** for copies of the Huff Rainfall distributions and NOAA rainfall depths.

Currently, the Porter County Stormwater Design Manual requires new storm sewers be designed for the 10% AEP event with the hydraulic gradeline (HGL) below the crown of the pipe. In addition, the 4% AEP event ("the 25-year event") should be below the top of the rim of any structure. Less frequent storm events (e.g., the 1% and 2% AEP events) require an overflow route to protect habitable structures. No records of the design parameters used for the storm sewers in these subdivisions was available.

After discussions with the Porter County Stormwater Management Department, the 10% AEP storm event was chosen to evaluate the storm sewers with the exception the 10% AEP HGL could be below the rim elevation instead of the 4% AEP HGL as required currently. This exception was based on the unknown design requirements at the time of construction of these subdivisions and availability of roadway overflow routes documented by the topographic data.

2.3.8 Hydraulic Capacity Evaluation

Using the 10% AEP event and maintaining the HGL below the casting, the hydraulic capacity of the existing storm sewers was examined. Two scenarios were modeled (assuming pipes clean and in good conditions):

- Existing pipe materials and diameters
- Pipes assumed to be lined

For the lined conditions, the existing pipe diameters were reduced, inverts adjusted, and the Manning's n value reduced to account for lining as described by INDOT standards. A Manning's n value of 0.012 was used for lined pipes and the dimensional changes are summarized in the table below.

Table 2-1 : Pipe Lining Adjustments		
Pipe Diameter	**Parameter**	**Change (in)**
24 in and less	Diameter	-1
	Invert	+0.5
27 thru 48 in	Diameter	-2
	Invert	+1

2.4 Water Quality

As a permitted Municipal Separate Storm Sewer System (MS4) entity, Porter County would be required to consider implementing Best Management Practices (BMPs) on storm water management projects including repair and restoration projects. BMPs include structural and non-structural practices. Non-structural practices primarily address outreach and education while structural practices include physical infrastructure.

Since this study was focused on the condition and rehabilitation of the physical infrastructure, structural practices were considered. Economically, retrofitting existing stormwater management systems can be less costly than constructing new infrastructure or retrofitting manufactured water quality systems. Manufactured water quality systems can be a preferred option when the available footprint is limited.

Using these considerations, the study's approach was to retrofit existing ponds for water quality first then to consider manufactured units where existing storm sewers included direct outfall without detention. Retrofitting existing ponds will require constructing forebays or other pretreatment and constructing new staged outlets to detain the water quality event for 12 to 24 hours.

Estimating the runoff rates for manufactured water quality units requires separating pervious and impervious surfaces in each watershed and running 0.3 inch storm event. The study included XPSWMM computer models of the existing and proposed storm sewers where the watershed surfaces are described by composite curve numbers, i.e. pervious and impervious surfaces are averaged together. Because this study is primarily for scoping and budgetary purposes, the XPSWMM models were rerun using a one (1) inch storm to estimate the water quality flow rate to compensate for the average curve numbers. During the design phase, the water quality flow rate can be refined separating the pervious and impervious surfaces and using the 0.3 inch storm.

The proposed water quality practices for each subdivision are discussed in the respective section and budgetary costs included.

2.5 Public Meetings

Four (4) public meetings were held after the field activities were completed to gather public input. One meeting was held for each subdivision with the fourth scheduled covering all three subdivisions as an alternative date to allow for those who could not attend previous meetings. The meetings where held a 6 pm on September 21, 23, 28 and 30 2021. The meetings were held at the New Hope Church, John Simativich Elementary School, and the American Legion.

In general, the information provided by those attending confirmed previous reports and complaints of sinkholes, off-site runoff flooding inlets, areas of standing water, etc.

3.0 Coventry Subdivision

As noted above, the Coventry subdivision storm sewers almost entirely have HDPE as the material of construction. These pipes are sensitive to the installation conditions and methodology. As noted previously, no records of inspection during installation have been found.

The Coventry subdivision can be divided into two primary areas, north and south, with Franciscan Drive being the approximate dividing line (see **Appendix III A**). The north section has been almost completely developed while the south section has been platted but not developed. A significant area of the north section's runoff is conveyed through sewers that pass through the undeveloped southern section before reaching the outfall.

During field reconnaissance the following observations were noted:

- Deep sinkholes were observed in the southern section over pipes near the outfalls;
- Structures with standing water inside;
- Structure with tops made of concrete poured over plywood, and;
- Significant erosion at pipe outfalls.

3.1 Video Inspection Results

Deformed pipes, separated joints, longitudinal cracks along the top and bottoms and non-uniform slopes creating pools on standing water were observed throughout the entire storm sewer system.

Appendix III B provides photographs of some of the typical pipe conditions. These are representative of the conditions found throughout the entire subdivision.

3.2 Video Pipe Evaluation

Approximately 5,099 ft of storm sewer was video inspected in Coventry. This length includes both upstream and downstream directions and the subsequent overlap from inspecting from both the upstream and downstream directions. The total pipe length in the GIS database is 9,065 ft, including roadway culverts that were not inspected.

The pipe material types are summarized in **Table 3-1** below.

Table 3-1 : Pipe Materials Summary		
Pipe Material	**Length**	**%**
CMP	417	8.2
HDPE	4,522	88.7
RCP	150	2.9
PVC	10	0.2
Totals	**5,099**	**100**

The majority of the pipe is HDPE followed by much smaller lengths of CMP and RCP.

The material observed in the bottom of the pipes and described as debris included:

- Rocks (Rk)
- Silt / Sediment (S)
- Leaves (L)
- Roots (Rt)

The following table summarizes the lengths of pipe with rock, sediment, and leaves.

Table 3-2 : Pipe Debris Characteristics	
Debris	**Length**
Leaves (L)	562
Sediment (S)	3,520
Rocks (Rk)	829

The debris descriptions above in **Table 3-2** are not exclusive. The pipe segments could also include debris from the other description (e.g., the length of pipe in the leaves (L) category may include leaves and sediment). The lengths are a total length of all pipes that included the specific debris description.

The depth of sediment in the pipes is an indicator of the effort that may be required to clean the pipes in preparation for possible lining. The depths of sediment in the pipes were visually estimated from the video records. **Table 3-3** below provides a summary of the depths and lengths of pipe.

Table 3-3 : Pipe Sediment Depth Summary	
Sediment Depth (ft)	**Pipe Length**
≤0.2	2,820
>0.2 & ≤0.3	1,566
>0.3 & ≤0.4	228
>0.4 & ≤0.5	186
>0.5	0
ND	299
Total =	**5,099**

The table indicates a modest length of the pipes would require immediate heavy cleaning.

After review of the video observations, a preliminary determination of potential for lining was performed. **Table 3-4** below provides a summary of the pipe lengths for lining and replacement.

Table 3-4 : Pipe Lining and Replacement Summary	
Category	**Pipe Length**
Lining	0
Replacement	5,099
Total =	**5,099**

Almost all the pipe is HDPE and are not in acceptable condition. Deformed geometry, irregular slopes and stress cracks throughout the system indicate the pipe may have not been installed with correct backfill material, placement, and compaction.

The table indicates lining is not considered a viable option.

3.3 Topographic Review - Watershed / Subwatershed Delineation

The County 2010 GIS contour data was used to delineate the watersheds served by the existing storm sewer (**Appendix III A**). No significant off-site watershed appears to be served by the Coventry storm sewer system.

The topographic review found an area north of Friar's Green that may have served as detention previously. The contours around this area may have once been closed but are now open. Field reconnaissance confirmed the opening between two berms on both sides of the creek. The area upstream of the berms was wooded and included brush indicating no maintenance had been performed.

The topographic review also found an area in the south undeveloped section between Coventry Lane and Dearborn Road that could be a detention pond (see **Appendix III A**). The contours are closed and there is a defined channel downstream across the utility pipeline easement. No field reconnaissance was performed in this area.

The County GIS data was also reviewed for future detention pond locations (see **Appendix III D**). The plat information indicated an area in the undeveloped south section across the pipeline easement had been designated for detention. However, the potential for crossing the utility pipeline with new storm sewer may be limited by the utility pipeline depth limiting the area that could be served. New detention along the creek may be possible but limited. The creek is shown on the wetland inventory and any detention constructed would likely be restricted to outside the wetland boundary. A wetland delineation would need to be performed to determine the wetland boundaries and subsequently the new detention limits.

3.4 Hydraulic Model Results

A hydrologic model was created in XPSWMM as noted in Section 2. The results indicate the current pipe sizes and average slopes would comply with the 10% AEP evaluation criteria if they were in good condition.

Copies of the XPSWMM HGL graphs for the proposed pipe system are provided in **Appendix III C**. The plots show some of the pipes have a negative slope but the 10% AEP event shows the HGL in the pipe is above the top of the pipe (surcharged) and the resulting velocity is sufficient to prevent accumulation of solids.

3.5 Inlet Casting Evaluation

Using the 10% AEP (10-year) storm event from the XPSWMM model, the runoff rates for the street inlets vary from as high as 4.83 cfs (watershed area = 3.8 ac) to as low as 0.1 cfs (watershed area =0.03 ac). Some of the highest flow rates for inlets are in the undeveloped southern section of the subdivision.

Assuming a street with 2% cross-slope, a 1.5 % longitudinal slope, Manning's n of 0.016, back-to-back curb width of 26 ft and limit the gutter spread to maintain one 10 ft lane, the approximate depth of flow that produces approximately 8 ft of spread would be 2 inches (0.16ft) and the flow rate would be 1.5 cfs. Additional inlets upstream in the gutter would have to be considered to meet the current gutter spread requirement.

Alternatively, and as a "temporary" solution, the existing inlets could be upgraded to allow higher flows. Taking a Neenah R-3227 grate and box casting as an example, a flow depth of 0.16 ft would produce a casting flow rate of approximately 1.8 cfs. There are approximately 14 inlets with flows that exceed 1.5 cfs. The Neenah R-3227 would need to be sumped approximately 0.3 to 0.4 ft (4 to 5 inches) to accommodate the higher flow of some of the watersheds to the streets.

Rear yard inlet flows reach up to 3.2 cfs peak rates. A Neenah R-4346 (30 in x 30 in beehive) would require a sump of approximately 6 inches to accommodate the flow rate.

In general, future modifications of the stormwater system should include inlets recessed from the back of the curb to allow for sumping and sumped rear-yard inlets should be constructed.

3.6 Water Quality Review

The existing Coventry stormwater management system include two (2) pond areas that receive runoff from four (4) storm sewer outfall pipes and three (3) storm sewer outfalls that discharge without detention. The existing pond areas can be retrofitted as water quality features. Two of the three direct discharge outfalls will require new practices such as manufactured units. The third direct discharge outfall is not included in any of the proposed projects and water quality is not considered at this time.

3.7 Public Questionnaire Review

The Flooding Survey from the 2010 County-wide Comprehensive Drainage Plan and the PCDDSWM complaint reports from 2018, 2019, 2020 and 2021 were reviewed for reports in the Coventry area. A GIS map showing the location of the complaints is provided in **Appendix III A**.

The original questionnaire from the 2010 County-wide Comprehensive Drainage Plan indicated an area of concern was flooding of the roadway and yard at the north end of Coventry Lane.

The PCDDSWM reports indicated the following:

- Flooding at the inlet in the southeast corner of Dearborn Road and Plymouth Road from a possible blocked inlet:
- Flooding at the rear yards at the end of the Cross Meadows Road cul-de-sac:
- Flooding in the rear yard on the north end of Friar’s Green:
- Flooding in low area between Dearborn Road and Cross Meadows Road cul-de-sac. The low area also appeared in the topographic review as a potential wetland and is owned by Portage Township per the Porter County GIS database:
- Back yards, residential structures, and park along Shrine Court.

The report of flooding on the north end of Coventry Lane is consistent with the results of the video inspection at that location. The camera was obstructed by blockage of the pipe exiting the inlet to the east. In addition, runoff from the rear yards of the parcels on the west side of the Coventry Lane may have contributed to flooding.

Since the reports of flooding in the rear yards at the end of Cross Meadows Road, the PCDDSWM has undertaken regrading and established a positive flow path in the rear yards to the roadside ditch along CR W 700N.

According to the PCDDSWM, the flooding in the rear yard of Friar’s Green was caused by a blocked private tile.

Since runoff from this area was included in the initial hydraulic modeling of the existing storm sewer, a storm sewer extension to this area was examined. A new 12" storm sewer could be installed from the existing inlet on Cross Meadows west then north along the rear lots lines of the parcels along Cross Meadows (see **Appendix III D** for a potential location). The pipe could then be extended north to the lowest elevation. Easements would need to be obtained from all the property owners.

3.8 Proposed Projects

Due to the overall poor condition of the existing HDPE pipe, all storm sewers should be replaced. With almost 5,100 feet of existing storm sewers, replacing all of Coventry's storm sewers as a single construction project would be a significant financial investment and likely beyond current budget appropriations. However, many of the storm sewers segments are currently functional and the replacement could be coordinated with future projects such as roadway maintenance. As an alternative, the replacement activities were divided into seven (7) smaller projects as described below.

3.8.1 COV 1 - South Area Storm Sewer Replacements

The area south of Franciscan Drive is part of the platted subdivision but is undeveloped at this time. The storm sewer was installed through this section and provides two of the four existing outfalls for the subdivision. With no pavement or structures, the cost of sewer replacement through this section will be reduced compared to other sections and would re-establish capacity for a large area of Coventry (see **Appendix III D for Project Areas Map**). It is recommended structural backfill be placed above the pipes to the surface in order to be prepared for the future roadways over the top of the sewers.

COV 1 would include outlet stabilization measures, pond retrofitting for water quality and one (1) manufactured water quality unit. The Opinion of Probable Cost is summarized in **Table 3-5** and a detailed Opinion of Probable Cost provided in **Appendix III E**.

3.8.2 COV 2 – Coventry Ponds Clean and Inlet / Outlet Stabilizations

There are two existing ponds serving the Coventry subdivision, one in the northeast and the other in the south (see **Appendix III D**). COV 2 involves repairing and rehabilitating the detention pond in the north and south areas. The parcel where the majority of the detention pond on the north is located appears to be currently owned by Portage Township with some possible overlap of the detention pond onto a private owned parcel to the west. The plat indicates the primary parcel was to be part of the POA parks land and there appears to be an easement line shown around the pond.

The parcels where the south detention pond is located are all shown as being privately owned per Porter County GIS. The plat shows a drainage easement around the south pond.

The Opinion of Probable Cost is summarized in **Table 3-5** and a detailed Opinion of Probable Cost provided in **Appendix III E**.

3.8.3 COV 3 – Cross Meadows Storm Sewer Replacement and Extension

Alternative 3 provides for the extension of the existing storm sewer to the low area west of Cross Meadows Drive in the rear yards as discussed in Section 3.5. This alternative will require easements from private property owners and Portage Township.

The Opinion of Probable Cost is summarized in **Table 3-5** and a detailed Opinion of Probable Cost provided in **Appendix III E**.

3.8.4 COV 4 – Coventry Court / Friars Green Storm Sewer Replacements

This alternative will replace the storm sewer systems serving Coventry Court and Friars Green and the eastern section of Franciscan Drive. Both of these systems outfall to the existing north pond and the project will include outfall stabilization.

The Opinion of Probable Cost is summarized in **Table 3-5** and a detailed Opinion of Probable Cost provided in **Appendix III E**.

3.8.5 COV 5 – Coventry Lane / Shrine Court Storm Sewer Replacements

The storm sewers in the COV 5 area include sections located in the rear yards between Shrine Court and Franciscan Drive and the rear yards of the homes on the east side of Coventry Lane north of Shrine Court. Access to both these areas is limited by easements and landscaping.

COV 5 includes relocating these storm sewers to inside the roadway R/W (see **Appendix III D**). The sewer from the roadway to the two rear yard drains between Shrine Court and Franciscan Drive will be replaced and the extension through the rear yard abandoned.

The Opinion of Probable Cost is summarized in **Table 3-5** and a detailed Opinion of Probable Cost provided in **Appendix III E**.

3.8.6 COV 6 – Dearborn Road Storm Sewer Replacements

COV 6 primarily addressed replacement of the storm sewer in the R/W of Dearborn Road. There are also connections to the R/W storm sewer from the rear yards on the west side of Dearborn Street. These appear to be 8-10 inches in diameter and installed to drain low areas. There appears to be no easement for these pipes indicating they may be private. COV 6 includes one (1) manufactured water quality unit at the outfall.

The Opinion of Probable Cost provides a cost for the R/W pipes and an option for replacement of the private side / rear yard pipes. The Opinion of Probable Cost is summarized in **Table 3-5** and a detailed Opinion of Probable Cost provided in **Appendix III E**.

3.8.7 COV 7 – Potential New Detention Ponds

COV 7 addresses new detention along the creek. As noted above, a wetland determination and delineation will be required to determine where additional detention could be constructed and how much volume would be provided.

Appendix III D shows potential detention outside of the wetlands as shown in the National Wetlands Inventory Map and these volumes are reflected in the Opinion of Probable Cost. The

Opinion of Probable Cost is summarized in **Table 3-5** and a detailed Opinion of Probable Cost provided in **Appendix III E.**

3.9 Opinions of Probable Construction Cost Summary

The opinions of probable cost for each alternative are summarized in the table below. Detailed Opinions of probable cost are provided in **Appendix III E**.

Table 3-5 : Summary of Opinions of Probable Construction Cost

Project ID	Probable Cost
COV 1 - South Area Storm Sewer Replacement	$740,000
COV 2 - Coventry Pond Rehabilitations	$380,000
COV 3 – Cross Meadows Replacement and Extension	$340,000
COV 4 – Coventry Court / Friars Green Storm Sewer Replacement	$440,000
COV 5 – Coventry Lane / Shrine Court Storm Sewer Replacement	$860,000
COV 6 – Dearborn Road Storm Sewer Replacement	$860,000
COV 7 – Additional Detention	$1,080,000
Total =	$4,700,000

3.10 Recommended Alternative Implementation

Replacement of all the storm sewers is recommended. However, the cost of replacement as one single project could be considered prohibitive and implementing them as a series of smaller projects would be more manageable.

Table 3-6 below proposes the stepwise implementation to maximize impact while financing over time.

Table 3-6 : Implementation Priority Summary

Project ID	Priority
COV 1 - South Area Storm Sewer Replacement	1
COV 5 – Coventry Lane / Shrine Court Storm Sewer Replacement	2
COV 3 – Cross Meadows Replacement and Extension	3
COV 2 - Coventry Pond Rehabilitations	4
COV 6 – Dearborn Road Storm Sewer Replacement	5
COV 4 – Coventry Court / Friars Green Storm Sewer Replacement*	6
COV 7 – Additional Detention	7

*(Replace north storm sewers with road maintenance)

4.0 New South Haven Subdivision

Storm sewers in New South Haven (NSH) are primarily corrugated metal pipe (CMP) and were constructed beginning in the 1960's. There are two main outlets, one on the unnamed tributary to Salt Creek approximately 450 feet west of CR 400W / McCool Road between Portland and Nantucket Roads (the north collector). The second is south of CR W 600N at the intersection of Olympic Road (the south collector). The south outlet is a pipe that continues south in the farm field approximately 230 feet before turning east and following the overland swale approximately 2,300 ft before outletting into a private lake (see **Appendix IV A**).

The north collector includes the pipe system that lies along the south bank of the overflow swale that runs from the west of Wolf Road through the subdivision to McCool Road. The swale conveys overland flows form the agricultural fields to the west under Wolf Road by three large culverts through the subdivision adjacent to the utility pipeline corridor (see **Appendix IV A**).

4.1 Video Inspection Results

As noted above, the primary material of construction of the pipes in New South Haven was CMP. The main outlet pipe in the field to the south is reinforced concrete pipe (RCP). Debris and sediment were found throughout the system with some large rocks and even landscape block too large to move by water jet. The CMP pipes were in various condition ranging from good to bottoms missing from corrosion (see **Appendix IV B** for video pictures).

Pipes with hardened sediment were found and required multiple passes. Despite the multiple attempts, cleaning in some pipes was abandoned because hardened sediment remained.

The video inspection did confirm a field tile entering the storm sewer system from the west side of CR 450W / Wolf Road. Field observations confirmed a riser approximately 275 ft west of the road in a large depressional area.

The video inspection also found:

- Negative slopes in the existing pipes on the west end of the north collector storm sewer. The negative pipe slopes are primarily at the west end near Wolf Road and the intersection of Olney Road and Nantucket Road.
- Negative slopes at intersections including the Nantucket Road / Olney Road intersection. Negative sloped pipes were noted at the pipes connecting inlets at the intersections.
- Backwater from Lake outfall of the south collector storm sewer, suggesting the pond outfall may need inspection and / or maintenance. However, the pond appears to be privately owned.
- Long runs without manholes and bends without manholes. The storm sewer along CR 400W includes a bend approximately 75 feet upstream of the outfall and no manhole was found between Nantucket Road and Niagara Road over the distance videoed (approximately 375 ft). No structure was visible from the surface.

4.2 Video Pipe Evaluation

Approximately 13,338 ft of storm sewer was video inspected in NSH. This length includes both upstream and downstream directions and subsequent overlap. The total pipe length per the GIS data is 21,167 ft including tiles and culverts that were not video inspected. The pipe material types are summarized in **Table 4-1** below.

Table 4-1 : Pipe Materials Summary		
Pipe Material	**Length**	**%**
CMP	9,511	71.3
HDPE	1,324	9.9
RCP	2,503	18.8
Totals	**13,338**	**100**

The majority of the pipe is CMP followed by RCP.

The material observed in the bottom of the pipes and described as debris included:

- Rocks (Rk)
- Silt / Sediment (S)
- Leaves (L)
- Roots (Rt)

Some of the sediment was hardened or crusty impairing the jetting. In some instances, jetting was abandoned after an initial effort indicated significant time would be required.

The rock diameter ranged from less than 1 in. to riprap size. In one pipe, the rock obstruction appeared to be a landscape block. Some of the larger rock required additional jetting to move or did not move and may require more labor intense methods to remove.

The following table summarizes the lengths of pipe with rock, sediment, and leaves.

Table 4-2 : Pipe Debris Characteristics	
Debris	**Length**
Leaves (L)	1,519
Sediment (S)	14,293
Rocks (Rk)	4,867

The debris descriptions above in **Table 4-2** are not exclusive. The pipe segments could also include debris from the other description (e.g., the length of pipe in the leaves (L) category may include leaves and sediment). The lengths are a total length of all pipes that included the respective debris description.

The depth of sediment in the pipes is an indicator of the effort that may be required to clean the pipes in preparation for possible lining. The depths of sediment in the pipes are visually estimated from the video records. **Table 4-3** below provides a summary of the depths and lengths of pipe.

Table 4-3 : Pipe Sediment Depth Summary

Sediment Depth (ft)	Pipe Length
≤0.2	8,384
>0.2 & ≤0.3	3,006
>0.3 & ≤0.4	1,132
>0.4 & ≤0.5	748
>0.5	4
ND	64
Total =	**13,338**

The table indicates a substantial amount of the pipes will need cleaning prior to any lining efforts. After cleaning, the area under the sediment may have corroded and compromised structural integrity. The deeper sediment the more pipe surface covered and the higher probability of pipe corrosion. The table indicates over 7,900 ft of pipe has a sediment depth greater than 0.2 ft, approximately half of the total pipe length.

After review of the video observations, a preliminary determination of potential for lining was performed. **Table 4-4** below provides a summary of the pipe lengths for lining and replacement.

Table 4-4 : Pipe Lining and Replacement Summary

Category	Pipe Length
Lining	9,465
Replacement	7
Not Determined *	540
None	3,633
Total =	**13,645**

* Includes “Not Determined” and Blank entries

The potential "Lining" pipes are subject to revision after full cleaning and the condition of the bottom of the pipe is determine. The pipes in the "None" category are generally the newer concrete pipes, in good condition and require no action.

The pipes listed as "Not Determined" were generally submerged or obstructed and the video camera could not provide useful observations.

The table indicates lining may be considered a viable option.

4.3 Topographic Review - Watershed / Subwatershed Delineation

The topographic review found the depressional area west of Wolf Road provided approximately 37 acre-feet of storage and serves and west of the field tile entering the north collector storm sewer. Field reconnaissance confirmed a field tile riser in the depressional. The watershed draining to this depressional area is large (see **Appendix IV C** for the off-site watersheds). The watershed immediately adjacent to the depressional area is 60 acres with an additional 82 acres that may discharge to the depressional area. The 82 acres also includes a depressional area that when it overflows would discharge overland. However, there is no known direct subsurface connection such as a field tile from the 82 acres watershed.

In addition to the north off-site watershed, there are five (5) other off-site watersheds entering the southern storm sewer system from the west (**Appendix IV C**), a total of approximately 27 acres. Three (3) of these watersheds enter the NSH system at structures 1012, 1110 and 1120, totaling approximately 13.5 acres. Per the contour data, two (2) other watersheds between the three enter the subdivision through rear yards before draining to the roadway and subsequently to structures 1101 and 1011. These two watersheds total approximately 13.7 acres.

The topographic review found the primary watershed divide between the north and the south lies about midway between Nantucket Road and Newcastle Road, approximately along Norfolk and Niagara Roads. The runoff along the divide generally runs south or north from the divide in the roadway gutter to either Nantucket Road or Newcastle Road before reaching an inlet. There is also a small watershed flowing east and entering through the roadside ditch along the eastside of CR 400W.

4.4 Modeling Methodology and Approach

The modeling continued to use the NRCS /SCS methodology as described in **Section 2.3.6**. The depressional area west of Wolf Road was added as a depressional area with a 15" outlet (the field tile) and an overflow weir at the top ground elevation. The GIS contour data indicated approximately 37 ac-ft of storage was available if the tile is assumed to be fully operational.

4.5 Hydraulic Model Results

Overall, many of the pipes would meet the capacity threshold set for this study if they were in good shape and set at uniform slopes. However, surcharging was observed in the model for the southern storm sewer system. After testing additional pipes and larger pipe diameters, the restriction was found to be in large part to the existing pipe diameter of the RCP south of CR 600N to the final outfall. A review of the topography indicated the surcharged runoff would overflow through the streets to the field with no impact to structures.

After adjustment for lining all pipes appeared to continue to meet the capacity criteria. See **Appendix IV E** for XPSWMM printout of the proposed pipeline HGLs. The plots show some of the pipes have a negative slope but the 10% AEP event shows the HGL in the pipe is above the top of the pipe (surcharged) and the resulting velocity is sufficient to prevent accumulation of solids.

The depressional area to the west appears to detain the 1% (100-yr) event. If this area is developed in the future, this natural storage capacity should be maintained as required by the current Porter County drainage design manual.

4.6 Inlet Casting Evaluation

Using the 10% AEP (10-year) storm event from the XPSWMM model, the runoff rates for the street inlets vary from as high as 11.24 cfs (watershed area = 8.88 ac) to as low as 0.06 cfs (watershed area =0.035 ac). If we assume a street with 2% cross-slope, a 1.5 % longitudinal slope, Manning's n of 0.016, back-to-back curb width of 26 ft and limit the gutter spread to maintain one 10 ft lane, the approximate depth of flow that produces approximately 8 ft of spread would be 2 inches (0.16ft) and the flow rate would be 1.5 cfs. Additional inlets upstream of the existing inlets would be required to meet the current gutterspread requirements.

Alternatively, and as a "temporary" solution, the existing inlets could be upgraded to allow higher flows. Taking a Neenah R-3227 grate and box casting as an example, a flow depth of 0.98 ft would produce a casting flow rate of approximately 11 cfs. Sumping a curb box 1 ft may not be practical. Alternatively, using a beehive casting (e.g., Neenah R-4346) behind the curb would allow a uniform depression of 8 inches to a foot to accommodate the high flows.

There are approximately 37 inlets with flows that exceed 1.5 cfs but are less than 5.5 cfs. The Neenah R-3227 would need to be sumped approximately 0.3 to 0.4 ft (4 to 5 inches) to accommodate these flows of some of the watersheds to the streets.

The highest rear yard inlet flow is approximately 8.3 cfs. A Neenah R-4346 (30 in x 30 in beehive) would require a sump of approximately 0.72 ft (8.4 inches) to accommodate the flow rate.

In general, future modifications of the stormwater system should include inlets recessed from the back of the curb to allow for sumping and sumped rear-yard inlets should be constructed.

4.7 Water Quality Review

The existing NSH stormwater management system has no detention ponds and includes six (6) storm sewer outfalls. Within the existing NSH boundaries, available open areas are minimal and too small for most constructed water quality practices. The only potential area is west of CR 400W and south of Portland Road (the existing stream). However, this area is potentially regulated wetlands and may be difficult to permit for a pond. Subsequently, manufactured water quality units were assumed for each proposed project.

4.8 Public Questionnaire Review

The Stormwater Flooding Questionaires from the 2010 County-wide Comprehensive Drainage Plan for reports of roadway flooding and the PCDDSWM complaint reports from 2018, 2019, 2020 and 2021 were reviewed for complaints in the NSH area. A GIS map showing the address of the responders is provided in **Appendix IV A**.

The questionnaire from the 2010 County-wide Comprehensive Drainage Plan indicated 18 reports of roadway flooding concern. The following table summarizes the depths of roadway water reported.

Table 4-5 : Roadway Flooding Depth Summary

Depth	# Of Occurrences)
0, ≤6 in	14
>6, ≤12 in	2
1-3 ft	2
>3 ft	0

Causes indicated by the questionnaire included:

- Leaves / debris blocking inlets:
- Depressed area of the road pooling water:
- Incorrect lot grading:
- Leaking foundation:
- Landscaping impeding flow:
- Creek / ditch filled in:
- Culverts blocked:

The PCDDSWM reports indicated complaints in the north section (North of Portland Drive). The complaints included:

- Neighbors putting leaves down the storm drain:
- Driveway sinking and washing away:
- Flooding at the mailbox with standing water (654 Newport Rd).

4.9 Proposed Projects

With almost 13,400 feet of existing storm sewers, replacing or lining all of NSH's storm sewers as a single construction project would be a significant financial investment and likely beyond current budget appropriations. However, as noted with the Coventry storm sewers, many of the storm sewers segments are functional and the replacement could be coordinated with future projects such as roadway maintenance. As an alternative, the replacement activities were divided into eight (8) smaller projects as described below (**Appendix IV D** for the Project Area Map).

4.9.1 NSH 1 – South Outfall Monitor, Replace and Spot Repair Project

NSH 1 covers the south outfall pipe from CR 600N south through the farm fields to the outfall, approximately 2,500 feet. The pipe is reinforced concrete pipe (RCP) and the video monitoring indicated areas between structures 1004 to 1002 where there is exposed rebar and another area where the pipe is displaying cracking. Neither of these areas appeared to be structurally failing and the pipe appears to perform acceptably during the 10-year (10% AEP) event, making this a low priority concern. As the outfall, a manufactured water quality should be installed. Due to the large watershed served, this unit will be large. It could be located downstream of the bypass sewer discussed in the next section to serve this area as well.

As a result, the recommendation for this section is to monitor and perform spot repairs for the exposed rebar and the cracked section at a convenient time in the future. A manufactured water unit should be constructed at that time and is included in the Opinion of Probable Cost.

The Opinion of Probable Cost is summarized in **Table 4-6** and a detailed Opinion of Probable Cost provided in **Appendix IV F**.

4.9.2 NSH 2 - Newcastle Road Storm Bypass Storm Sewer Project

As discussed in the topographic review, there is approximately 27 acres entering the southern storm sewer system on the west side of NSH. The NSH 2 project provides a new storm sewer system along the southwestern edge of NSH to intercept this runoff and divert it to the southern outfall at the intersection of Olympia Road and CR 600N. The length of the proposed sewer would be approximately 1,530 feet. Removing this runoff from the existing storm sewer system reduces flooding of the rear yards in this area as well as flooding of the existing storm sewer after lining.

The Opinion of Probable Cost is summarized in **Table 4-6** and a detailed Opinion of Probable Cost provided in **Appendix IV F**. Water quality for this area is addressed with NSH 1.

4.9.3 NSH 3 – Newcastle Road Storm Sewer Lining Project

Based on the XPSWMM model, the existing storm sewers in Newcastle Road appeared to have capacity to convey the 10-year (10% AEP) event after cleaning and lining (assuming the bypass sewer is functioning – Project NSH 2). NSH 3 is primarily a lining project with a couple of inlet upgrades (casting and connector pipes).

The Opinion of Probable Cost is summarized in **Table 4-6** and a detailed Opinion of Probable Cost provided in **Appendix IV F**. Water quality for this area is addressed with NSH 1.

4.9.4 NSH 4 – 400 West Storm Sewer Lining Project

NSH 4 addresses lining the existing storm sewer along CR 400W. The video inspection documented large debris that may require mechanical methods to remove increasing the costs.

In addition, the video inspection did not confirm the presence of a manhole south of Nantucket Road as indicated by the Porter County records (structure 3035). The total distance from Nantucket Road to Niagara is over 1,200 feet and the installation of two (2) new manholes may be required for lining and future maintenance.

The Opinion of Probable Cost is summarized in **Table 4-6** and a detailed Opinion of Probable Cost provided in **Appendix IV F**. One water quality unit for the outfall is included with the cost.

4.9.5 NSH 5 – Olney Area Storm Sewer Replacements

NSH 5 includes all structures upstream of structure 2260, including all pipes and inlets at the intersection of Olney Road and Nantucket Road. The video inspection documented negative slopes, submerged areas and transitions to other pipe diameters and materials east of CR 450W. All pipes should be replaced and reset to reduce deposition and allow ease of future maintenance. Water quality for this area is addressed with NSH 6.

The Opinion of Probable Cost is summarized in **Table 4-6** and a detailed Opinion of Probable Cost provided in **Appendix IV F**.

4.9.6 NSH 6 - Nantucket Lining and Inlet Connection Replacements

NSH 6 is the largest project and is primarily lining. Inlets and connection piping in the Portland Road and Pembroke Road intersection, the Nome Road and Portland Road intersection and the Oswego Road and Nantucket Road intersection will require upgrades to the inlets as discussed in Section 4.6 and replacement of associated connecting pipes. In addition, erosion protection for North interceptor and CR 400W outfall would be included in this project. Also included is one (1) manufactured water quality unit.

The Opinion of Probable Cost is summarized in **Table 4-6** and a detailed Opinion of Probable Cost provided in **Appendix IV F**.

4.9.7 NSH 7 – 450 West Storm Sewer Replacements

The CR 450W sewer replacement runs from north of Prescott Road intersection to an outlet north of the Nantucket Road intersection, a length of over 1,550 ft. The video inspection was limited by hard sediment that the jet could not move and blocked the camera. In addition, there appears to be a run of pipe between structure 6020 and 6010 of over 1,000 feet potentially requiring installation of a manhole for access if the pipe were lined. Based on these two observations, replacement is recommended.

The Opinion of Probable Cost is summarized in **Table 4-6** and a detailed Opinion of Probable Cost provided in **Appendix IV F** and includes one (1) manufactured water quality unit at the outfall.

4.9.8 NSH 8 – Plymouth Road Storm Sewer Replacements

NSH 8 is located in the northeast of the NSH subdivision and consists of storm sewer segments. The first segment is an 8 inch line starting approximately 400 ft west of the intersection of

Plymouth Road and CR 400W and proceeding to the intersection. The pipe turns to the north and outfalls into the ditch. The video inspection found this pipe was single walled HDPE with irregular slope, non-linear direction, and irregular cross-sectional geometry. The current minimum pipe diameter for Porter County is 12 inches.

The second area is the storm sewer from Prescott Street to Plymouth Road along Nome Road. The XPSWMM model indicated the current storm sewer did not have capacity for the 10-year (10% AEP) event and inlets should be upgraded.

Based on the observations above, replacement of the pipes is recommended. The Opinion of Probable Cost is summarized in **Table 4-6** and a detailed Opinion of Probable Cost provided in **Appendix IV F** and includes one (1) manufactured water quality unit at the outfall.

4.9.9 NSH 9 –600 North Storm Sewer Replacements

The section of the existing storm sewer on the north side of CR 600N west of New South Haven (NSH) is technically outside of the subdivision. However, this storm sewer connects to the NSH system and drains the intersection of CR 600N and CR 450W. The video inspection revealed plastic pipe with inconsistent slope and cross-sectional geometry.

The proposed replacement would start at the upstream manhole approximately 100 ft east of the intersection and proceed to the western side of NSH where it would intercept the proposed off-site bypass discussed in the NSH 2 project. The culverts and structures within the intersection would be considered a future County project.

The Opinion of Probable Cost is summarized in **Table 4-6** and a detailed Opinion of Probable Cost provided in **Appendix IV F**. Water quality is addressed by the NSH 1 project.

4.10 Opinions of Probable Construction Cost Summary

The detailed Opinions of Probable Cost are provided in **Appendix IV F. Table 4-6** below provides a summary of the opinions of probable construction costs for each alternative.

Table 4-6 : Summary of Opinions of Probable Construction Cost	
Project ID	**Probable Cost**
NSH 1 – South Outfall Monitor, Replace and Spot Repair Project	$340,000
NSH 2 - Newcastle Road Storm Bypass Sewer Project	$310,000
NSH 3 – Newcastle Road Lining Project	$1,250,000
NSH 4 – 400 West Lining Project	$900,000
NSH 5 – Olney Area Sewer Replacements	$180,000
NSH 6 - Nantucket Lining and Inlet Connection Replacements	$2,430,000
NSH 7 – 450 West Sewer Replacements	$930,000
NSH 8 – Plymouth Road Sewer Replacement	$430,000
NSH 9 –600 North Sewer Replacements	$160,000
Total =	$6,930,000

4.11 Recommended Alternative and Implementation

Table 4-7 below proposes the stepwise implementation to maximize impact while financing over time.

Table 4-7 : Implementation Priority Summary	
Project ID	**Priority**
NSH 2 - Newcastle Road Storm Bypass Sewer Project	1
NSH 5 – Olney Area Sewer Replacements	2
NSH 7 – 450 West Sewer Replacements	3
NSH 4 – 400 West Lining Project	4
NSH 3 – Newcastle Road Lining Project	5
NSH 6 - Nantucket Lining and Inlet Connection Replacements	6
NSH 8 – Plymouth Road Sewer Replacement	7
NSH 9 – 600 North Sewer Replacements	8
NSH 1 – South Outfall Monitor, Replace and Spot Repair Project	9

5.0 Salt Creek Valley Commons

Salt Creek Valley Commons (SCVC) is the southern-most subdivision studied. The subdivision's storm sewer system includes two (2) detention ponds, one on the south side adjacent to CR 500N and east of McCool Road and the other on the south side of CR 550N also east of McCool Road. These ponds serve most of the subdivision with some areas of the eastern subdivision direct discharging to Salt Creek without detention. The northern pond's discharge flows to the same private lake that receives runoff from New South Haven (see **Appendix V A**).

The western section of the subdivision (west of McCool Road) is more densely developed while the eastern section includes open space or common areas behind the residential lots.

Since the formation of the Twin Creeks Conservancy District, the storm sewer in McCool Road from its outfall upstream to Saginaw Drive was upgraded in size with concrete pipes and new inlets.

Some storm sewers are located between structures or in the rear yards. A review of the subdivision plats found easements were only 10 ft in some areas and adjacent to structures.

5.1 Video Inspection

The video inspection confirmed pipes were past or nearing the end of their service life. The observations included:

- Hardened Sediment – As noted in the New South Haven inspection, the sediment has become hardened and required multiple passe with the jet. In one location, the jet got stuck below a hardened section of sediment. After sending the camera to view the situation, an excavator had to be called to open the pipe, dislodge the camera, and repair the cut. Cleaning for potential lining activities in the future may not be possible or economically feasible.
- Missing pipe – One section of pipe (line 1043-1042) was missing an entire section from what appeared to be corrosion and the pipe opened up into a large "cavern" under a tree.
- Obstructed pipe – The section of pipe from 7070-7071 could not be videoed due to the condition of the pipe. The pipe was reported to be partially collapsed.
- Missing bottoms – as noted in previous video inspection in New South Haven, bottom missing due to corrosion were observed.
- Changes in pipe materials – Connecting pipes along McCool Road where the main collector pipe had been replaced appeared to have been sleeved using PVC pipe changing the diameter of the pipe.
- Submerged sections – Sections of the storm sewer included standing water indicating a low area in the pipe. Some sections were too deep for the camera to provide video images of the pipe condition.
- Irregular pipe geometry and slope – Some pipes in the eastern sections of SCVC were found to be HDPE and possibly installed incorrectly. The pipes were misshaped and included inconsistent slopes.

See **Appendix V B** for typical pictures.

5.2 Video Pipe Evaluation

Approximately 24,400 ft of storm sewer was video inspected in SCVC. This length includes both upstream and downstream directions and subsequent overlap from video inspection from both directions. The total pipe length per the GIS data is 20,205 ft. The pipe material types are summarized in **Table 5-1** below.

Table 5-1 : Pipe Materials Summary		
Pipe Material	**Length**	**%**
CMP	18,692	76.6
HDPE	1,477	6.1
PVC	134	0.5
RCP	3,646	14.9
HDPE+CMP	424	1.7
Blank	34	0.1
Totals	**24,407**	**99.9**

The majority of the pipe is CMP followed by RCP. The RCP is primarily located in McCool Road and is a result of the storm sewer replacement / upgrade performed by TCCD. The 424 ft of HDPE+CMP may have been a repair. The video showed a transition from HDPE to CMP and the CMP was corroded to the point soil was exposed. The blank entry was a 28” x 40” oval that the video camera was not able to enter.

The material observed in the bottom of the pipes and described as debris included:

- Rocks (Rk)
- Silt / Sediment (S)
- Leaves (L)
- Roots (Rt)

Some of the sediment was hardened or crusty impairing the jetting. In some instances, jetting was abandoned after an initial effort indicated significant time would be required. In one pipe, the camera was lodged in the pipe by hardened debris and required an excavator to retrieve the camera.

The rock diameter ranged from less than 1 inch to riprap size. The larger rock required additional jetting to move or did not move by additional jetting and may require more labor intense methods to remove.

The following table summarizes the lengths of pipe with rock, sediment, and leaves.

Table 5-2 : Pipe Debris Characteristics	
Debris	**Length**
Leaves (L)	2,674
Sediment (S)	15,294
Rocks (Rk)	8,855

The debris descriptions above in **Table 5-2** are not exclusive. The pipe segments could also include debris from the other description (e.g., the length of pipe in the leaves (L) category may include leaves and sediment). The lengths are a total length of all pipes that included the respective debris description.

The depth of sediment in the pipes is an indicator of the effort that may be required to clean the pipes in preparation for possible lining. The depth of sediment in the pipes was visually estimated from the video records. **Table 5-3** below provides a summary of the depths and lengths of pipe.

Table 5-3 : Pipe Sediment Depth Summary	
Sediment Depth (ft)	**Pipe Length**
≤0.2	10,923
>0.2 & ≤0.3	5,522
>0.3 & ≤0.4	3,534
>0.4 & ≤0.5	2,247
>0.5	1,901
Total =	**24,127**

The table indicates a substantial amount of the pipes will need cleaning prior to any lining efforts. After cleaning, the area under the sediment may be corroded provide compromised structural integrity. The deeper sediment depth the greater the corrosion potential and the table indicates over 13,000 ft of pipe has a sediment depth greater than 0.2 ft.

After review of the video observations, a preliminary determination of potential for lining was performed. **Table 5-4** below provides a summary of the pipe lengths for lining and replacement.

Table 5-4 : Pipe Lining and Replacement Summary	
Category	**Pipe Length**
Lining	6,717
Replacement	11,323
Not Determined	811
None	2,618
Total =	**18,851**

The potential "Lining" pipes are subject to revision after full cleaning and the condition of the bottom of the pipe is determine. The pipes in the "None" category are generally the newer concrete pipes and in good condition.

The pipes listed as "Not Determined" were generally submerged or obstructed and the video camera could not provide useful observations.

The table shows replacement length exceeds lining by a factor of nearly two.

5.3 Topographic Review

Overall, surface runoff in the western section of SCVC flows to the east and McCool Road. The north section of McCool Road (from just north of Sandalwood Drive) runs toward the north detention pond. The runoff from the intersection of Sandalwood Drive itself flows to the south detention pond.

Runoff in the eastern section of SCVC is split three ways with the divide running along approximately Sandalwood Drive then Raven Road. On the north side, the ground is loped toward the north pond. The majority of the east section of SCVC discharges east to Salt Creek with a small section in the southwest discharging to the south pond.

The GIS contour data indicated two large watersheds enter the SCVC storm sewer system from the west. The south off-site watershed is 28.8 acres of agricultural land and enters the subdivision behind the residential structures at the intersection of Robyn Road and Salt Creek Parkway. The north off-site watershed is 51.5 acres of agricultural land entering the subdivision behind the residential lots on Robyn Road between Sturgeon Drive and Sassafras Drive.

5.4 Modeling Methodology and Approach

With two (2) large watersheds off-site to the west directly impacting the storm sewer system, the first step with modeling was to investigate rerouting or attenuating this runoff before it enters the system. All subsequent modeling scenarios would be based on the rerouting or attenuating results.

5.4.1 Off-Site Runoff.

Two scenarios were modeled to manage the off-site runoff; rerouting the runoff around the subdivision in a ditch and creating detention ponds immediately west of the subdivision where the runoff enters the storm sewer system (see **Appendix V D**). GIS topographic data was used for both analyses.

A proposed diversion ditch serving the north end of the subdivision was created assuming a constant slope with an upstream elevation 4-5 feet below existing ground elevation and the downstream elevation set at the existing elevation of the receiving waterway north of CR 550N.

Likewise, the proposed south diversion ditch was modeled with a constant slope starting at 4-5 feet below existing ground elevation and ending in the low area south of CR 500N west of the school.

Construction of these ditches will require the purchase of land. Assuming an average ditch depth of 5 ft, a 5 ft bottom width and 3:1 side slopes, the top of bank width would be 35 ft. Adding a 20 ft maintenance road on one side of the bank requires a total width of 55 ft. With total length of approximately 2500 ft for the north ditch, the total property acquisition would be approximately 3.16 acres.

The south diversion ditch would need to be approximately 1150 ft in length requiring approximately 1.4 acres. Additional property could be obtained to create a linear park along the west side of the SCVC subdivision.

The detention option for the north and south off-site areas was created starting with the existing storm sewer invert and creating contours with a minimum 3:1 slope up the existing surface. The contours were expanded while keeping the release rate at the County required 0.2 cfs/ac.

Constructing the detention ponds would require the acquisition of approximately 8 acres. This would include a one-lane maintenance road / recreational trail along the entire western side of the subdivision.

After noting the construction of the diversion ditches would increase the runoff rate downstream in the XPSWMM model, construction the detention ponds was considered the recommended alternative since they attenuate the runoff rates and continue the drainage route through the subdivision. No negative impact downstream would be created.

All subsequent modeling and hydraulic evaluations incorporated detention for the west off-site watersheds.

5.4.2 Pond Maintenance

The field reconnaissance and survey of the north and south pond outfall structures found the inverts of the control structures were 4 to 5 feet below the adjacent ground. This observation suggested accumulation of sediment and other materials may have occurred over the years of service.

In order to evaluate the impact of the potential additional volume from removing this sediment, new contours were generated from the GIS data assuming a slope of 3:1 from the current ground elevation to the invert of the outfalls. The XPSWMM model was modified by adding this volume and the model rerun for all scenarios evaluating the conveyance / detention systems.

5.4.3 South Pond Outfall Evaluation

The Porter County Department of Development and Stormwater Management reported instances of flooding along CR W500N from the existing detention pond. The existing pond discharges across CR W500N and, per the survey data, the outlet pipe has a diameter of 8".

Assuming the pond sediment was removed as described in the previous section and the conveyance system was in good condition, the XPSWMM model was rerun increasing the control structure outlet pipe diameter until the pond no longer overtopped the roadway. The resulting pipe diameter was 24 inches in the model.

5.5 Model Results

The XPSWMM model found the existing pipe diameters and slopes would have capacity if they were in good condition and detention for the west off-site watersheds is provided. As noted previously, the evaluation criteria assumed the 10-year (10% AEP) storm event at or below the casting elevation was acceptable.

With the ponds cleaned of accumulated sediment and the outfall pipe of the south pond increased in diameter, both ponds would be filled without overtopping the adjacent roadways.

See **Appendix V D** for the resulting XPSWMM HGL profiles for the proposed mainline sewers.

5.6 Inlet Casting Evaluation

Using the 10% AEP (10-year) storm event from the XPSWMM model, the runoff rates for the street inlets vary from as high as 6.75 cfs (watershed area = 5.33 ac) to as low as 0.04 cfs (watershed area =0.03 ac). If we assume a street with 2% cross-slope, a 1.5 % longitudinal slope, Manning's n of 0.016, back-to-back curb width of 26 ft and limit the gutter spread to maintain one 10 ft lane, the approximate depth of flow that produces approximately 8 ft of spread would be 2 inches (0.16ft) and the flow rate would be 1.5 cfs. Additional inlets upstream of the existing inlets would be required to meet the current gutterspread requirements.

Alternatively, and as a "temporary" solution, the existing inlets could be upgraded to allow higher flows. Taking a Neenah R-3227 grate and box casting as an example, a flow depth of 0.16 ft would produce a casting flow rate of approximately 1.8 cfs. There are approximately 48 inlets with flows that exceed 1.5 cfs but are less than 5.5 cfs. The Neenah R-3227 would need to be sumped approximately 0.3 to 0.4 ft (4 to 5 inches) to accommodate the higher flow of some of the watersheds to the streets. No inlets appear to have runoff flow rates exceeding 5.5 cfs.

Rear yard inlet flows reach up to 6-7 cfs peak rates. A Neenah R-4346 (30 in x 30 in beehive) would require a sump of approximately 6 inches to accommodate the flow rate.

In general, future modifications of the stormwater system should include inlets recessed from the back of the curb to allow for sumping and sumped rear-yard inlets should be constructed.

5.7 Water Quality Review

SCVC includes two (2) detention ponds and seven (7) storm sewer outfalls without detention. As discussed in the Coventry Subdivision section, these ponds could be retrofitted with forebays and staged control structures to address water quality for the storm sewers served.

The seven (7) storm sewer outfalls are located in forested areas or adjacent to a pipeline / utility easement. Both areas may prohibit or make permitting difficult for construction of water quality features. Installation of manufactured water quality units is presumed in the Opinions of Probable Cost of each project below with a direct storm sewer outfall.

5.8 Public Questionnaire Review

The Stormwater Flooding Questionaires from the 2010 County-wide Comprehensive Drainage Plan were reviewed for reports of roadway flooding and the PCDDSWM complaint reports from 2018, 2019 and 2020 were also reviewed for complaints in the SCVC area. A GIS map showing the address of the responders is provided in **Appendix V A**.

The questionnaire from the 2010 County-wide Comprehensive Drainage Plan indicated 11 reports of roadway flooding concern. The following table summarizes the depths of roadway water reported.

Table 5-5 : Roadway Flooding Depth Summary

Depth	# Of Occurrences
0, ≤6 in	8
>6, ≤12 in	2
1-3 ft	1
>3 ft	0

Causes indicated by the questionnaire included:

- Leaves / debris blocking inlets:
- Depressed area of the road pooling water:
- Incorrect lot grading:
- Leaking foundation:

- Landscaping impeding flow:
- Creek / ditch filled in:
- Culverts blocked:
- High water table:
- Down spouts:
- Sump pump failure:
- Need storm structures.

The PCDDSWM reports indicated complaints throughout the subdivision with a cluster on Rainier Road. The complaints included:

- Sinkholes:
- Sump pumps discharging into the street:
- Rear yard flooding:
- Runoff coming down Sherman Drive.

The majority of the complaints are on Rainier Road. The video investigation found standing water and sediment in the outfall pipe from Rainier Road to the south detention pond. The PCDDSWM has subsequently opened up the outfall and the pond and cleaned the pipe. Fewer complaints have been reported since the cleaning.

Most complaints in the PCDDSWM spreadsheet have been addressed or referred to the Porter County Highway Department.

5.9 Recommended Capital Improvement Projects

SCVC includes over 20,200 feet of existing storm sewers, replacing or lining all of SCVC's storm sewers as a single construction project would be a significant financial investment and likely exceed current budget appropriations. However, as noted with the previous subdivisions, many of the storm sewers segments are functional and the replacement could be coordinated with future projects such as roadway maintenance. As an alternative, the replacement activities were divided into ten (10) smaller projects as described below (**Appendix V E** for the Project Area Map).

5.9.1 SCVC 1 – South Detention Pond Cleaning and Outfall Replacement

As discussed in section 5.4.2, the existing pond would be cleaned to remove accumulated sediment and debris to the bottom of the control structure. The bottom slope would be uniform to the control structure and side slopes would be kept at minimum of 4:1 and possibly increased to 3:1. Cleaning could increase the storage volume by approximately 0.8 ac-ft to 1 ac-ft for the pond. Forebays would also be constructed at the storm sewer inlets to the pond.

In addition, the upgrade of the south pond outfall would be included in this alternative. Using the criteria to maximize the storage in the pond while preventing overtopping of the roadway, a 24 inch pipe was found to be the required pipe diameter using the XPSWMM model. The outfall would be re-routed from crossing CR 500N directly east along the north R/W and discharge directly into Salt Creek. The outfall would also be modified to address water quality.

The Opinion of Probable Cost of this project is summarized in **Table 5-6** and a detailed cost is provided in **Appendix V F**.

5.9.2 SCVC 2 – North Detention Pond Cleaning

Similar to the south detention pond, the north pond would be cleaned to remove accumulated sediment and debris to the bottom of the control structure. The bottom slope would be uniform to the control structure and side slopes would be kept at minimum of 4:1 and possibly increased to 3:1. Cleaning could increase the storage volume by approximately 0.8 ac-ft to 1 ac-ft for the pond. Forebays would also be constructed at the storm sewer inlets to the pond, the pond control structure checked and modified for water quality performance.

No outfall upgrades are proposed. The XPSWMM model indicate the revised storage volume would be filled without overtopping the roadway.

The Opinion of Probable Cost of this project is summarized in **Table 5-6** and a detailed cost is provided in **Appendix V F.**

5.9.3 SCVC 3 – New Detention Ponds

This project will construct two (2) new detention ponds as discussed above. The two ponds would require excavation and removal of approximately 17 ac-ft of soil, connection to the existing storm sewer and acquisition of land. Grass filter strips will need to be provided around the pond to provide pretreatment for the sheet flow entering and the outfall staged to enhance water quality.

The acquisition of land would include a 30 foot wide access along the west side of the subdivision initially for maintenance and potentially construction of a future pedestrian path. The Opinion of Probable Cost includes a cost for construction of the pedestrian path with stone as a line item.

The Opinion of Probable Cost of this project is summarized in **Table 5-6** and a detailed cost is provided in **Appendix V F**.

5.9.4 SCVC 4a / 4b – Sandalwood to 500 North Storm Sewer Replacements

The SCVC 4 capital improvement project is broken into two (2) sections. The project basically includes installation of new storm sewers in Sequoia Drive, Salt Creek Parkway, Sable Drive, Saginaw Drive and Sandalwood Drive to the existing inlets and abandoning the existing storm sewers between the houses. The existing space between houses and obstructions from fencing and landscaping limits future maintenance access from the surface.

During the video inspection, the existing pipe from Salt Creek Parkway (Structure 1042) to the north was found to be corroded away and a “dome” above the pipe from washed out soil was forming. The existing tree roots appear to be supporting the remaining soil at this time. The formation of a sinkhole is considered imminent, and abandonment of this line can be considered one the highest priorities. Construction of the new storm sewer in Sable Drive will have to be completed to abandon the line. This portion of project is SCVC 4a (Sandalwood to 500 North -Sable Drive Component) so it might be expedited. The remaining storm sewer construction was considered lower priority and addressed as SCVC 4b (Sandalwood to 500 North).

Both SCVC4a / 4b will provide an opportunity to add additional inlets and reduce gutter spread. Water quality will be addressed with the SCVC 1 project.

The Opinion of Probable Cost of this project is summarized in **Table 5-6** for SCVC 4a and for SCVC 4b. A detailed cost is provided in **Appendix V F.**

5.9.5 SCVC 5 – 550 North to Sherman Drive Storm Sewer Replacements

SCVC 5 includes new storm sewers in Sherman Drive, Sassafras Drive, Sturgeon Drive and CR 550N. As noted in SCVC 4a / 4b, the new sewers allow for abandonment of the sewers between residential structures where access is limited. The Sassafras Drive storm sewer will intercept the outflow from the proposed NW detention basin. There is an existing storm sewer in the rear yards between Sturgeon Drive and CR 550N that is in good condition and will remain in service, allowing the rear yards drain to continue operation.

Similar to SCVC 4a / 4b, the new storm sewers in the roadway will allow for installation of additional inlets and the reduction of the gutter spread. Water quality will be addressed with the SCVC 2 project.

The Opinion of Probable Cost of this project is summarized in **Table 5-6** and a detailed cost is provided in **Appendix V F.**

5.9.6 SCVC 6 – Raven to Riviera Storm Sewer Replacements and Enhancements

The video inspection found one storm sewer segment at the end of Raven Court was too deteriorated to video. This storm sewer flows southeast east from the cul-de-sac and lies between two residential structures. This segment should be considered a high priority and replaced as soon as possible. However, the distance between the residential structures is minimal and the residential structures are very near the edge of the easement. In addition, the storm sewer segment at the end of Raven Circle has minimal clearance between residential structures.

Similar to SCVC 4, SCVC 6 was divided into two (2) projects, SCVC 6a (Raven to 550 North Storm Sewer Replacements) and SCVC 6b (Riviera to Sunshine Court Storm Sewer Replacements), allowing for a faster response to the deteriorated pipe section (see **Appendix V E**). SCVC-6a provides a new storm sewer from Raven Circle to Raven Road, then north along Raven Road where it is joined by a new storm sewer from Raven Circle and continues north along Raven Road to CR 550N where it turns to the northwest along the existing storm sewer route. This new storm sewer will allow for the abandoning of the storm sewers between the Raven Circle and Raven Court residential structures. Water quality will be addressed using a manufactured water quality unit.

SCVC 6b addresses improvements on the remaining storm sewer system flowing south including lining and replacements. There are two (2) areas in the community spaces rain gardens could be installed to store rainfall events greater than the 10- year event (see **Appendix V E**). Water quality will also be enhanced by installing a manufacterd water quality unit in the final outfall section of the storm sewer.

The Opinion of Probable Cost both of these projects are summarized in **Table 5-6** and a detailed cost is provided in **Appendix V F.**

5.9.7 SCVC 7 – Ranier to Rushmore Storm Sewer Replacements

SCVC 7 provides primarily in-kind replacement of the existing storm sewers and outfall erosion protection. HDPE pipe was used for many of the storm sewer segments but appears to have been installed improperly resulting in irregular slopes, distorted geometries, and physical damage to the pipe's integrity.

Water quality will include three (3) water manufactured water quality units for the storm sewers outfalls that do not empty into the pond.

The Opinion of Probable Cost of this project is summarized in **Table 5-6** and a detailed cost is provided in **Appendix V F.**

5.9.8 SCVC 8 – Salt Creek Parkway Storm Sewer Replacements

SCVC 8 reroutes structures 13090, 13070, 13080 and 13081 south on Sandalwood Drive to Salt Creek Parkway then west to Structure 4050. This reroute allows for the abandoning of the line from 13070 to 13060. This line includes a section with limited access near two residential structures.

SCVC 8 includes two rain gardens at structures 4031 and 4051 in the community spaces behind the residential structures to buffer events than the 10-year storm. Final water quality will be addressed with the SCVC 1 pond cleaning and enhancements.

The Opinion of Probable Cost of this project is summarized in **Table 5-6** and a detailed cost is provided in **Appendix V F.**

5.9.9 SCVC 9 – Sassafras Storm Sewer Replacements

Similiar to SCVC 8, SCVC 9 provides a reroute of structures 14030, 14020 and 14010 to 13010 from Sassafras Court allowing abandonment of the pipe from 14010 to 14000. This segment, as noted in other areas, has limited access between residential structures. Water quality will addressed by the SCVC 2 pond project.

The Opinion of Probable Cost of this project is summarized in **Table 5-6** and a detailed cost is provided in **Appendix V F.**

5.9.10 SCVC 10 – Salt Creek Parkway and 550N Storm Sewer Lining

SCVC 10 provides lining and replacement for segments of the existing storm sewer. SCVC 10 also includes raingardens at structures 8040 and 8031 to buffer rainfalls. than the 10-year (10% AEP) events. Final water will be addressed by installing two (2) manufactured water quality units.

The Opinion of Probable Cost of this project is summarized in **Table 5-6** and a detailed cost is provided in **Appendix V F.**

5.10 Opinions of Probable Construction Cost Summary

The detailed Opinions of Probable Cost are provided in **Appendix V F. Table 5-6** below provides a summary of the opinions for each alternative.

Table 5-6 : Summary of Opinions of Probable Construction Cost

Project ID	Probable Cost
SCVC 1 – South Detention Pond Cleaning and Outfall Replacement	$410,000
SCVC 2 – North Detention Pond Cleaning	$180,000
SCVC 3 – New Detention Ponds	$2,700,000
SCVC 4a – Sable Drive Storm Sewer	$750,000
SCVC 4b – Sandalwood to 500 North Storm Sewer Replacements	$1,450,000
SCVC 5 – 550 North to Sherman Drive Storm Sewer Replacement	$2,390,000
SCVC 6a – Raven to 550 North Storm Sewer Replacements	$760,000
SCVC 6b – Riviera to Sunshine Court Lining	$750,000
SCVC 7 – Ranier to Rushmore Storm Sewer Replacements	$580,000
SCVC 8 – Salt Creek Parkway Storm Sewer Replacements	$710,000
SCVC 9 – Sassafras Storm Sewer Replacements	$230,000
SCVC 10 – Salt Creek Parkway and 550 North Storm Sewer Replacement	$450,000
Total =	**$11,360,000**

5.11 Recommended Alternative Implementation

Table 5-7 below proposes the stepwise implementation to maximize impact while financing over time.

Table 5-7 : Implementation Priority Summary	
Project ID	**Priority**
SCVC 4a Sandalwood to 500 North Storm Sewer Replacements	1
SCVC 6a – Raven to 550 North Storm Sewer Replacements	2
SCVC 1 – South Detention Pond Cleaning and Outfall Upgrade	3
SCVC 2 – North Detention Pond Cleaning	4
SCVC 3 – New West Detention Ponds	5
SCVC 9 – Sassafras Storm Sewer Replacement	6
SCVC 10 – Salt Creek Parkway and 550 North Storm Sewer Replacements	7
SCVC 7 – Ranier to Rushmore Storm Sewer Replacements	8
SCVC 8 – Salt Creek Parkway Storm Sewer Replacements	9
SCVC 4b Sandalwood to 500 North Storm Sewer Replacements	10
SCVC 5 – 550 North to Sherman Drive Storm Sewer Replacements	11
SCVC 6b – Riviera to Sunshine Court Lining	12