5-2016

Urban flooding and sewer inundation on the University of Louisville Belknap campus.

Justin T. Hall

URBAN FLOODING AND SEWER INUNDATION ON THE UNIVERSITY OF LOUISVILLE BELKNAP CAMPUS

By

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B.S., University of Louisville, 2013

A Thesis
Submitted to the Faculty of the
College of Arts and Sciences of the University of Louisville
in Partial Fulfillment of the Requirements
for the Degree of

Master of Science in Applied Geography

Department of Geography and Geosciences
University of Louisville
Louisville, KY

May 2016

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A Thesis Approved on

April 13, 2016

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ACKNOWLEDGMENTS

This thesis would not be possible if it wasn't for the patience, guidance and assistance from many people. First I want to thank my wife Jennifer Hall for supporting me through graduate school. I would like to thank Dr. C. Andrew Day my graduate advisor for his guidance and support through the past couple of years and his extreme patience. A very special thanks to Dr. Michael Croasdaile and the Speed School of Engineering Stream Institute for their valuable assistance and guidance in solving on campus time-lapse/time series data solutions. Arron Boggs University Physical Plant Assistant Director of Maintenance for his help in getting the time-lapse camera installed. Kevin Bailey of the Kentucky Transportation Cabinet for his assistance in getting the site permit for the time-lapse camera. Dr. David Howarth for his assistance and academic advice. Dr. Margath Walker for my introduction to the Geography and Geoscience Department. And last I would like to thank Bob Forbes and DJ Biddle for all their GIS education and technical support, and the Department of Geography and Geoscience for creating an exceptional graduate program.

ABSTRACT

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Over the past few decades on the University of Louisville Belknap campus urban flooding has become more frequent as a result of surface water runoff and sewer inundation. This urban flooding is a result of ongoing watershed urbanization and rapid expansion of the local sewer system to accommodate the expanding city of Louisville. However little research has been conducted on this issue, despite continued flooding on and adjacent to campus. Using the EPA Storm Water Management Model (SWMM) we applied a dual drainage modeling approach that combines both surface and subsurface drainage data to produce a flood hydrograph at the main outlet drainage point for a series of storm events. The output from this modeling was then compared to a real-time series dataset through the use of time-lapse photography for model verification. From our results we were able to identify and isolate key choke points in the campus drainage system that promotes sewer system inundation and surface flooding across campus.

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1.0 INTRODUCTION

1.1 Global Urbanization

Flooding in urban areas is on the rise as a result of increasing watershed urbanization. As people migrate to urban centers this creates a compounded effect where the need for housing and the expansion of the physical work place to accommodate new jobs forces cities to develop beyond sustainable development levels (Biemer and Schardein Jr. 1998; Cohen 2006). According to the World Urbanization Prospects: The 2014 Revision report published by the United Nations, as of 2014 54% of the global population lives in urban areas, with a projected increase to 66% by 2050 as compared to 1950 where only 30% of the world population lived in urban centers (United Nations, Department of Economic and Social Affairs, Population Division 2015). According to another United Nations report it is estimated that there will have been approximately 187,000/day people added to urban environments every day, between 2012 and 2015 (United Nations System Task Team 2012).

The process of urbanization itself increases the total amount of impervious surfaces such as asphalt, concrete, structures and buildings. Most urban surfaces produce an increased amount of surface water runoff by reducing rate of infiltration as compared to other surface materials such as soil, grass, and other naturally occurring surfaces and materials (Espey. Morgan, and Masch 1966; Arnold, Boison, and Patton. 1992; Brabec, Schulte and Richards 2002).

Accelerated urbanization poses challenges for cities to keep pace with expanding utilities and urban services (Cohen 2006) such as water and sewer as fast as developers can build them. In western developed countries urbanization is currently not as critical as compared to developing countries, however as we describe in the following paragraphs the problems in some modern cities from urbanization over the past century have created issues that are currently being dealt with today (Biemer and Schardein Jr. 1998).

Radical changes to the surface of the earth as a result of global population changes and urbanization places immense pressure on the local ecology in and around urban centers that depend on the quality of surface water runoff. Water quality in and around urban centers is a common problem that needs to be addressed and monitored, especially when the source of urban water runoff feeds reservoirs that are used to supply fresh water to an ever growing urban population (Prigent et al. 2012).

Urbanization also increases surface water runoff which leads to geomorphic issues in which erosion shapes the terrain in ways that can compound urban flooding issues by creating undesirable gullies and catchments (Junior et al. 2010). These urban flooding issues and others expressed in the literature review of this thesis supports the need for further urban flooding research.

1.2 Urbanization in Louisville, Kentucky

The local sewer system in and around the University of Louisville Belknap Campus is part of the greater Louisville and Jefferson County Metropolitan Sewer District (MSD). The MSD sewer system has been constructed over the past 100 years in response to periodic population growth and rapid urbanization. Post World War I Louisville saw an increase in population as soldiers came home from the war and the

Army training center at Camp Taylor expanded. Historical sewer inundation locally is a result of early Sewerage Commissioners, who choose the combined sewer and storm water drainage systems (Biemer and Schardein Jr. 1998). The combined sewer and drainage system is a sewer system which is designed to have storm water drain into the underground sewage system. This combined system in conjunction with urbanization has led to frequent urban flooding in Louisville, and more specifically on Belknap Campus.

The research question we were seeking to answer in this research focuses on whether the inconsistencies in the local sewer system design are responsible for frequent flooding on the University of Louisville Belknap Campus? While flooding on campus has been addressed in part by the addition of rain barrels, biowales and pervious pavement projects (Mog 2015), there is inadequate academic literature on the root causes or contributing factors of flooding on Belknap Campus. Our hypothesis was that the multi-line junction sewer drainage system currently in use in the local sewer district around the University of Louisville Belknap Campus is exacerbating the frequent campus flooding.

The objective of this thesis is to determine the sensitivity of the local sewer system around the University of Louisville Belknap Campus to surface water runoff and sewer inundation through the use of GIS, storm water modeling and campus flood time series data produced by time-lapse photography of flood events at a specific storm drain location on campus that corresponds with identified sewer network choke points.

2.0 LITERATURE REVIEW

2.1 Urban Flooding Research

Urban flooding research has been an expanding field as a result of an increase in urban flooding events in cities due to the costs incurred from flood clean up (Hsu, Chen, and Chang 2000; Mark et al. 2004; Schmitt, Thomas, and Ettrich 2004). As cities become more urbanized and the sewer networks expanded urban flooding research is essential to urban planners to help mitigate the costs of urban flooding and help develop sustainable urban planning methods that could help reduce the negative impacts on local ecosystem (Paul and Meyer 2001).

The effects of urbanization on surface water runoff can be visualized and measured as a decrease in lag time, which is the amount of time between the mean rainfall excess and the peak of the hydrograph (Espey, Morgan, and Masch 1966). An example of this effect can be seen in research in the Mercer and Newaukum creeks in western Washington, which showed urban streams have a much higher peak discharge and shorter lag times than a nearby rural stream (Konrad 2014).

2.2 Dual Drainage Model

The common approach to examining urban flooding and surface water runoff in academic literature is the dual drainage method (Huber and Dickinson 1988; Djokic and Maidment 1991; Djordjevic, Prodanovic and Maksimiovic 1999; Hsu, Chen, and Chang 2000; AMK Associates 2004; Smith 2006; Nania, Leon, and Garcia 2014). The dual drainage method examines two parts of the storm drainage system: 1. The surface system or terrain, which consists of buildings, sidewalks, streets, gutters and storm drains and 2. The subsurface system which is comprised of the sewer network/pipes, network junctions and storm drainage pipes that connect to storm drains to the sewer pipes (figure 1). Since both systems may be represented in Geographical Information Science (GIS) data formats, the dual drainage method can be applied using a variety of GIS modeling software (Huber and Dickinson).

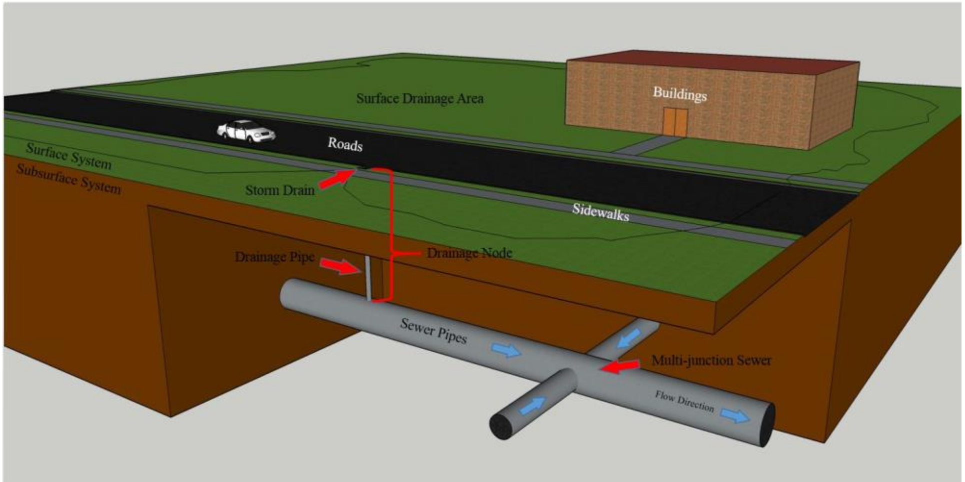

Figure 1.
Components of the dual drainage model

As urban flooding becomes more common, there has been an increasing number of urban flooding studies that used the dual drainage method. Studies using the dual drainage method have been published over the past five decades and are becoming more complex and detailed as technology and computer modeling advances. Some of these studies examined flooding in Austin, Texas (Espey, Morgan, and Masch 1966), Asheville, North Carolina (Djokic and Maidment 1991), the city of Taipei, Taiwan (Hsu, Chen, and Chang 2000), the city of Kaiserslautern, Germany (Schmitt, Thomas and Ettrich 2004), the historic center of Genoa, Italy (Aronica and Lanza 2005), the University of Memphis campus in Memphis Tennessee (Chen, Hill and Urbano 2009), and the Dolton suburb of Chicago, Illinois (Nania, Leon, and Garcia 2014). These studies examine urban flooding that is a result of inadequate storm drainage networks that consists of surface drainage and/or the subsurface sewer network.

From the previously examined studies the dual drainage model provides a more inclusive examination of all the components of urban flooding (Djordjevic, Prodanovic and Maksimiovic 1999; Mark et al. 2004; Schmitt, Thomas and Ettrich 2004). This method of examining urban flooding provided the framework for organizing and examining geographically the factors that contribute to urban flooding and sewer system inundation on the University of Louisville, Belknap Campus.

3.0 STUDY AREA

The University of Louisville Belknap Campus, located in the City of Louisville, Kentucky has seen several urban floods in recent years. On August 4 2009, one such flood caused approximately $21 million in flood damages, 92 of 150 buildings had been affected (figure 2) and 50 people had to be evacuated by boat, after 7.2 inches of rainfall fell in just under 80 minutes (U of L Today 2010; Mog 2015). Flooding on Belknap Campus has also been observed, photographed and/or video recorded, such dates are: May 29 2012, May 28 2014, July 27 2014, August 11 and 23 2014, September 11 2014, April 2 2015 and February 2 2016 which was caught on time-lapse camera. The above list of flooding events on campus demonstrate the frequency of urban flooding on Belknap Campus.

The study area for this thesis was selected by examining the August 4th 2009 Belknap campus flood, as well as other documented flooding events and the local sewer network that runs through campus. The area was closely examined the main section of the sewer network that runs through the eastern part of campus along South Brook Street to determine the southern multi-line sewer junction that we believe contributed to campus flooding and sewer inundation.

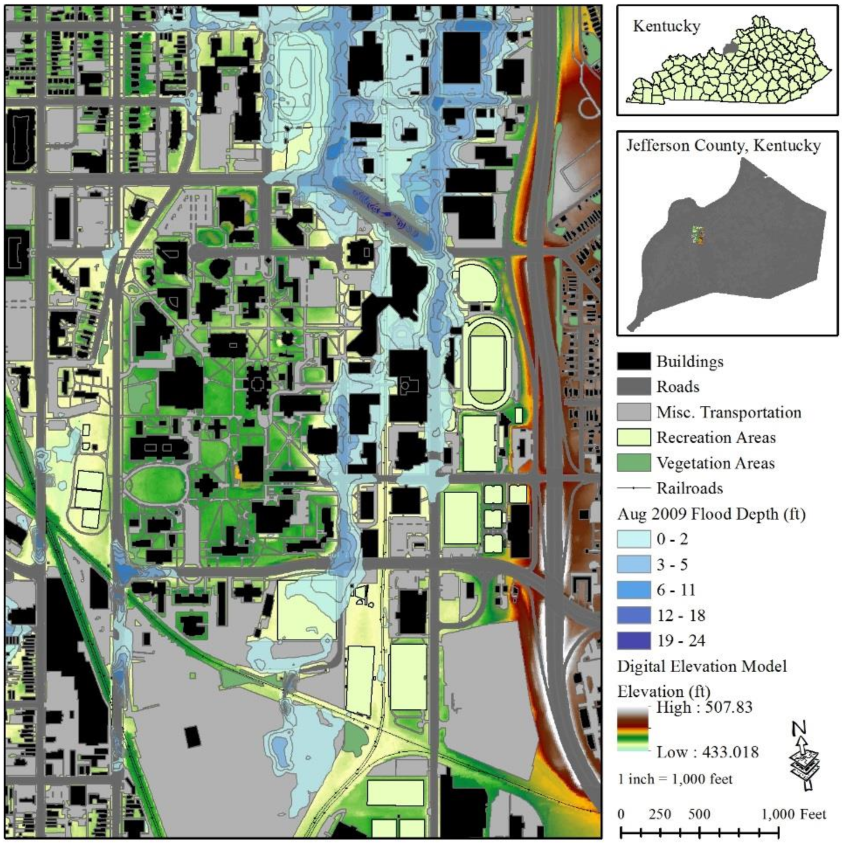

Figure 2.
University of Louisville Belknap Campus August 2009 Flood.

Since the flow direction of the sewer system runs from the southern part of campus northwards we decided to examine the area that surface water runoff would most likely feed the specific multi-line junction located in the southern part of campus and to the south (figure 3).

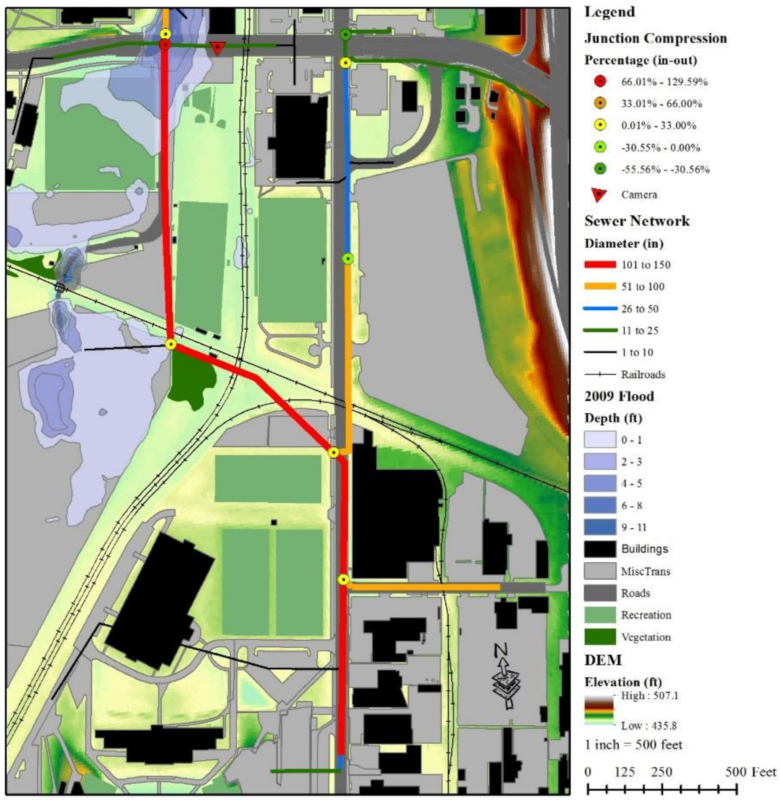

Figure 3.
Sewer Network within the study area and the 2009 Campus Flood.

The multi-line junction that is the focus of this study is located at the intersection of South Brook Street and Old Eastern Parkway, located under the Eastern Parkway overpass. There are two main pipes that feed into this junction, the main pipe that flows from the south is 120 inches in diameter and the second pipe which comes from the east and is fed by storm drains from one of two low points on campus that is 20 inches in diameter. These drain into a single 90 inch in diameter pipe.

The study area is approximately 0.478 square kilometers or 118.08 acres created in ArcGIS that includes portions of the University of Louisville Belknap campus and surrounding urban communities (figure 4).

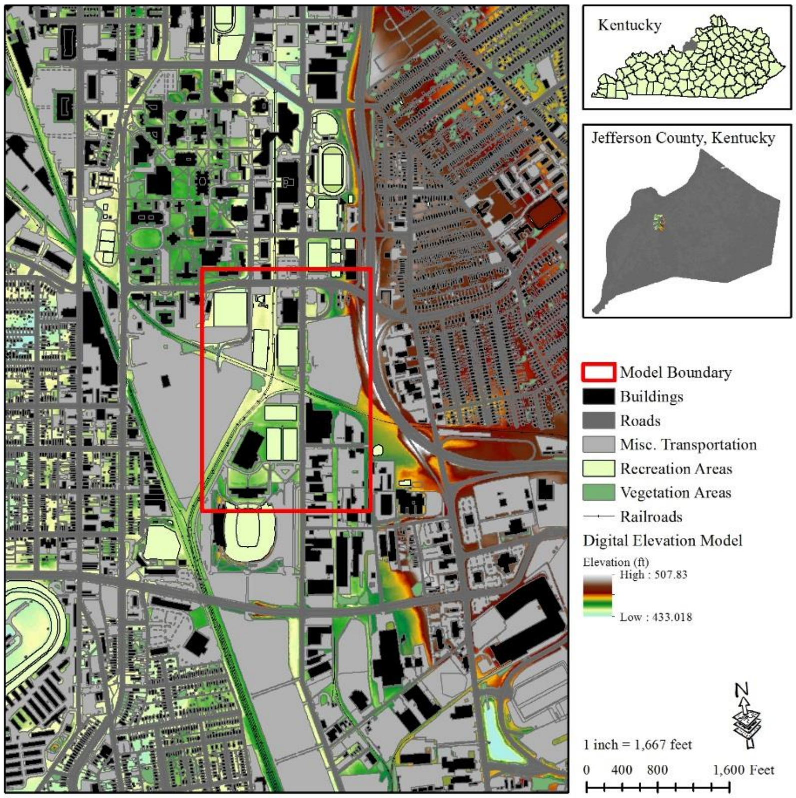

Figure 4.
University of Louisville Belknap Campus and Thesis Study Area.

The railroad tracks that run through campus play a small role in the topography of the study area in that some of the railroad tracks are slightly elevated which creates shallow sink catchments on the far side away from the study area outlet.

The main focus point of this study is one of two low points on Belknap campus as a result of being beneath a railroad underpass. The southern point underneath Eastern Parkway can be seen in figure 5.

Due to its location and terrain, this location acts as a sink catchment with a storm drain located at the bottom. This area is difficult to see on most maps and GIS datasets, which could easily be ignored without a site survey. This location demonstrates why site surveys play a critical role in urban flood research and management (Diaz-Nieto, Lerner, Saul, and Blanksby 2011), and why the use of time-lapse photography is beneficial due to this location not being visible from satellite imagery or aerial photography.

Figure 5.
University of Louisville Belknap Railroad underpass.

Since the study area was located on campus, weekly and daily access to the study area provides a wealth of observations and the ability to collect visual documentation through additional photographs and video.

The close proximity of the time-lapse camera allowed data to be pulled weekly and bi-weekly depending on the weather events, from the time-lapse camera to be compared with the MSD weather data that could be used to further calibrate the dual drainage model, as described in the Time Series Data sub-section.

4.0 METHODS

4.1 Dual Drainage Model

The methods used in this study are adapted from several urban and rural flooding studies discussed in the literature review that use ArcGIS, Watershed Modeling System (WMS) and Storm Water Management Model (SWMM).

The software used in this study was ArcMap 10.2 created by ESRI, the Watershed Modeling System 8.4 created by Aquaveo and the EPA Storm Water Management Model 5.1 (EPA-SWMM) created in conjunction between the Environmental Protection Agency/National Risk Management Research Laboratory and the University of Florida (Huber at el. 1981; Huber and Dickinson 1988; Rossman 2009). The dual drainage method of modeling using WMS and EPA-SWMM requires a more robust level of accuracy of the data used in modeling as compared to other methods that used alternative approaches in examining urban flooding (Djokic and Maidment 1991; Djordjevic, Prodanovic and Maksimiovic 1999; Werner 2001). An example of the dual drainage method requires sewer and drainage node data to be current, and if possible, with storm drains visually verified as part of the process of increasing the integrity of drainage node data and the model itself.

4.2 Data

The data used in this study consisted of terrain data, land cover data, storm drainage data, sewer network data, rainfall rate data and time-lapse photography images (table 1).

Table 1.

Data Types

Data			
File	Type	Resolution	Source
Terrain	DEM	3 meter	National Map Viewer
Land Cover	Shapefile	polygon	LOJIC
Storm Drainage	Shapefile	point	LOJIC
Sewer Network	Shapefile	line	LOJIC
Rainfall Rate	Weather	Inches/Hour	MSD
Time-lapse Video	AVI	1920x1080	TLC200 Pro

The terrain or topographic data was obtained from a digital elevation model (DEM), raster data format at the resolution of 3 meters by 3 meters and acquired from the United States Geological Survey's website (USGS 2015). Previous research shows the use of 1m to 3m resolution for small study areas to allow the model to simulate surface water flow of streets and sidewalks which could not be represented in lower resolutions through the use of triangulated irregular network data (Djordjevic, Prodanovic and Maksimiovic 1999).

The land cover data used was obtained in a polygon shapefiles format through the Louisville/Jefferson County Information Consortium (LOJIC). There are multiple land cover layers used which consist of buildings, roads, recreation, vegetation and miscellaneous transportation data (Aquaveo 2012).

The sewer network data used in this study was in the line shapefile format and was acquired through LOJIC (LOJIC 2012) which originates from MSD. The sewer network data includes pipe diameter, length, upstream and downstream pipe elevation, flow direction, pipe shape and pipe construction materials (Aquaveo 2012).

The rainfall intensity data in this study consist of both historic rainfall data of 1, 2, 5, 10, 25, 50 and 100 year precipitation frequency data (Aquaveo 2012) as documented by the Precipitation Frequency estimates published online by the National Weather Service/National Oceanic and Atmospheric Administration (NWS/NOAA 2016) and MSD (MSD 2016) rain gauge data. Rainfall intensity data also included rainfall for specific storm events that coincided with the time-lapse photography time series data and historical storm events. The MSD rainfall gages used included TR21 Wheeler Basin and TR12 Nightingale PS.

Time-lapse photography has been used to study a variety of natural systems that pose challenges for traditional remote sensing satellite data (Kramer and Wohl 2014; Natural England 2014) Due to the location of storm drains and the short time period of storm events, time-lapse photography provided a more realistic flood time series data set. The time series data set included flood start time, peak time, and end time of specific study locations (storm drainage nodes) and flood events.

4.3 GIS Processing

The data used in this study were compiled from their various sources and uploaded into ArcGIS. It was then labeled and organized by surface and subsurface systems. This allowed for easier processing and migration to the WSM and EPA-SWMM software. The end result from GIS processing produced a surface layer, drainage layer, sewer network layer and a runoff layer.

Land cover data polygon shapefiles were uploaded to ArcGIS for the study area. This data layer represented the different types of impervious surfaces, which would provide specific runoff coefficients for each surface type (table 2) polygon that was used in the model (Werner 2001; Djokic and Maidment 1991; Djordjevic, Prodanovic and Maksimiovic 1999).

Two surface layers were created, one with the maximum runoff coefficients for all the surfaces and one with the minimum runoff coefficients. Both layers used the runoff coefficients for each surface type as defined in Table 2 (Chow 1964; Chow, Maidment, and Mays 1988; Chow, Maidment, and Mays 2013; Kentucky Transportation Cabinet. 2010).

Table 2.

Runoff Coefficients

Landuse	Runoff Coefficient (C)	
Type of Drainage Area	Low	High
Business:		
Downtown areas	0.70	0.95
Neighborhood areas	0.50	0.70
Residential:		
Single-family areas	0.30	0.50
Multi-units, detached	0.40	0.60
Multi-units, attached	0.60	0.75
Suburban	0.25	0.40
Apartment dwelling areas	0.50	0.70
Industrial:		
Light areas	0.50	0.80
Heavy areas	0.60	0.90
Parks, cemeteries	0.10	0.25
Playgrounds	0.20	0.40
Railroad yard areas	0.20	0.40
Unimproved areas	0.10	0.30
Lawns:		
Sandy soil, flat, 2%	0.05	0.10
Sandy soil, average, 2-7%	0.10	0.15
Sandy soil, steep, 7%	0.15	0.20
Heavy soil, flat, 2%	0.13	0.17
Heavy soil, average, 2-7%	0.18	0.22
Heavy soil, steep, 7%	0.25	0.35
Streets:		
Asphaltic	0.70	0.95
Concrete	0.80	0.95
Brick	0.70	0.85

The surface topography terrain data used was in the Digital Elevation Model (DEM) format. The DEM was initially used to help define the study area boundaries within the larger watershed that the University of Louisville resides in (Jenson and

Dominigue 1988; Maidment 2002). The DEM was uploaded to ArcMap and vertical elevation data was converted from meters to feet. The local basin was then identified through the use of the watershed tool within the Hydrology toolbox in ArcMap (Maidment 2002). A polygon shapefile of the study area was created and used to extract the study area within the original DEM. Once the study area DEM was extracted it was then converted to the .hdr format for use in WMS and EPA-SWMM. The .hdr format was used to allow WMS to convert it to the .tin file format. The .tin format was preferred since the point elevation allows for a more accurate rendering of surface flow (Djokic and Maidment 1991).

The study area polygon shapefile that was created was also used to extract the study area from other shapefile data (Djokic and Maidment 1991; Maidment 2002).

The sewer network data was processed in ArcGIS to find multi-line junction or choke points where the volume of the input pipes would flow into the junction which would possibly exceed the volume of the output pipes (figure 3). These choke points were also used to help determine the locations at which the time-lapse camera would be installed.

In order to connect surface structures to the subsurface sewer network data, storm drains to drainage pipes or “drainage nodes” were included. In order to increase the accuracy of the model’s hydrograph output, storm drains within the surface drainage area were visually verified through a site survey in order to remove nodes that were either no longer in use or connected to the campus sewage service. This drainage node data was created in ArcGIS by combining node data from LOJIC and visual verification of storm drains within the study area (Djokic and Maidment 1991; Djordjevic, Prodanovic and

Maksimiovic 1999). The drainage node data, storm drain data and sewer network data was used to create the sewer layer in ArcGIS for WMS.

The drainage layer was created using data generated from the hydrology tool box to identify surface flow paths using the flow accumulation tool (Maidment 2002) and the surface layer. The surface drainage paths would include surface structures such as streets and railroads that would directly affect surface drainage paths. The surface drainage paths did not follow the flow accumulation path as a result of buildings, roads, railroad tracks, fences, and other man made obstructions (Syme 2008) This issue was taken into account when defining the final surface flow paths used in WMS and EPA-SWMM.

Once the files were created, they were then compiled, checked for projection, and labeled. Each of the layers used was then given the appropriate attributes for use in WMS (table 3). The GIS layers were then transferred to WMS for model construction.

Table 3.

Dual Drainage Model Layer Attributes

Layer	Attributes		
Surface	FID	POLYGON	RUNOFFC
Sewers	FID	POLYLINE	ELEVATION
Drainage	FID	POLYLINE	DRAINAGETYPE
	LENGTH	SLOPE	DMANNINGS
	BASINID		
Runoff	FID	POLYGON	DRAINTYPE
	BASINID	BASINAREA	BASINSLOP
	MFDIST	MFDSLOPE	CENTDIST
	CENTOUT	PSOUT	PNORT
	MSTDIST	MSTSLOPE	BAINLEN
	SHAPFACT	SINUOSIT	PERIMETER
	MEANELEV	CENTROIDX	BASINNAME
	LAGTIME	TC	CN
	PRECIP	HYDROVOL	HYDROTP
	HYDROPEAK		

4.4 Time Series Data

Time series data were used to calibrate the dual drainage model, when specific storm drain nodes experienced a surcharge as a result of the sewer network reaching maximum capacity. This required observations of when the surcharge begins, when it peaks and when it ends to provide a time series dataset that could be used to calibrate the model.

A single time-lapse camera, TLC 200 Pro manufactured by Brinno, was installed on a support column on the Eastern Parkway by-pass and just above the storm drain at the bottom of Old Eastern parkway at a location that was identified as a multi-line junction and choke point (figure 3). The camera stayed in place throughout the entire study to capture as many surcharge events as possible. The time-lapse camera was set to take photos once every minute (Kramer and Wohl 2014). The time-lapse data was routinely retrieved on a weekly basis to be examined for any surcharge events that would match storm events. The images from the time-lapse data was then converted to time series data, start time, peak time and end time of any surcharge events. The time-lapse data was then compared to the hydrograph produced by EPA-SWMM model that used specific storm data.

4.5 Watershed Modeling System Processing

In this study the dual drainage model was used following the procedures outlined in the WSM 8.4 tutorial (Aquaveo 2012) to produce a hydrograph using EPA-SWMM. A hydrograph shows the increase and decrease of water for a specific amount of time (Hendriks 2010). The WMS 8.4 tutorial includes importing GIS layers, entering in time of concentration, precipitation frequency, and sewer network elevation data. Runoff coefficients while added to the surface layer in GIS processing can be altered in the surface layer while building the dual drainage model in WMS.

4.5.1 Time of Concentration

The WMS model required a time of concentration (T_c) to produce the models hydrograph. The time of concentration is the amount of time that surface water takes to flow from the most distant point of the watershed to the point of the watershed outlet (Chow 1964). To find the T_c we used Kirpich's formula (Chow 1964; McCuen, Rawls and Wong 1984; Maidment 1993; Maidment and Djokic 2000) using the distance, change in elevation and slope (Chow 1964). The T_c was calculated in ArcMap by finding the distance from the drainage outlet and the furthest point within the study area that drains to the drainage outlet (equation 1).

$$T_c = 0.007 * L^{0.77} * S^{-0.385} \qquad \text{[eq 1]}$$

Where T_c is the time of concentration (minutes), L is the distance (ft) from the outlet to the furthest point that drains to the outlet, S is the slope of the path from the furthest point to the outlet. This formula is recommended for use on smaller watersheds where the time of concentration is close to the lag time (Chow 1964). The T_c was calculated to be approximately 15 minutes.

4.5.2 Precipitation Frequency

The WMS model required precipitation frequency (PF) data, which is the rates in which precipitation falls at different durations 5, 10, 15, 30 and 60 minutes for storm recurrence of 1, 2, 5, 10, 25, 50 and 100 year storm events. This data was collected from NWS/NOAA and MSD rain gauges for storm events caught on time-lapse. The PF and rain gauge data was compiled and used in WMS dual drainage model precipitation frequency (PF). The NWS/NOAA PF estimates are calculated with a 90% confidence interval (table 4).

Table 4.

Precipitation Frequency (PF)

	Precipitation Frequency (PF) estimates in inches						
Duration	1 year	2 year	5 year	10 year	25 year	50 year	100 year
5 min	0.369	0.438	0.516	0.578	0.659	0.722	0.783
10 min	0.575	0.684	0.803	0.896	1.010	1.100	1.180
15 min	0.705	0.838	0.988	1.100	1.250	1.360	1.470
30 min	0.937	1.120	1.360	1.540	1.770	1.950	2.130
60 min	1.150	1.380	1.700	1.960	2.300	2.580	2.860

4.5.3 Sewer Network Layer

The sewer network was uploaded to WMS while MSD upstream and downstream elevation data were entered to build the dual drainage model's sewer network as directed in the WMS 8.4 tutorial (Aquaveo 2012). Each node in the sewer network represented a storm drain and each link represented a sewer pipe. The upstream and downstream elevations reflected pipe invert elevations and storm drain node elevation reflect invert node elevations (figure 6). The sewer network consisted of 49 links and 50 nodes. The sewer network's total length of pipe (links) was 9,266.8 feet, which gives the total capacity of the sewer network in this model a total of 253,175.6 cubic feet. This volume would require approximately 0.59 inches of rainfall with a 1.00 runoff coefficient and zero amount of water leaving the study area in order to completely fill the sewer network within the study area.

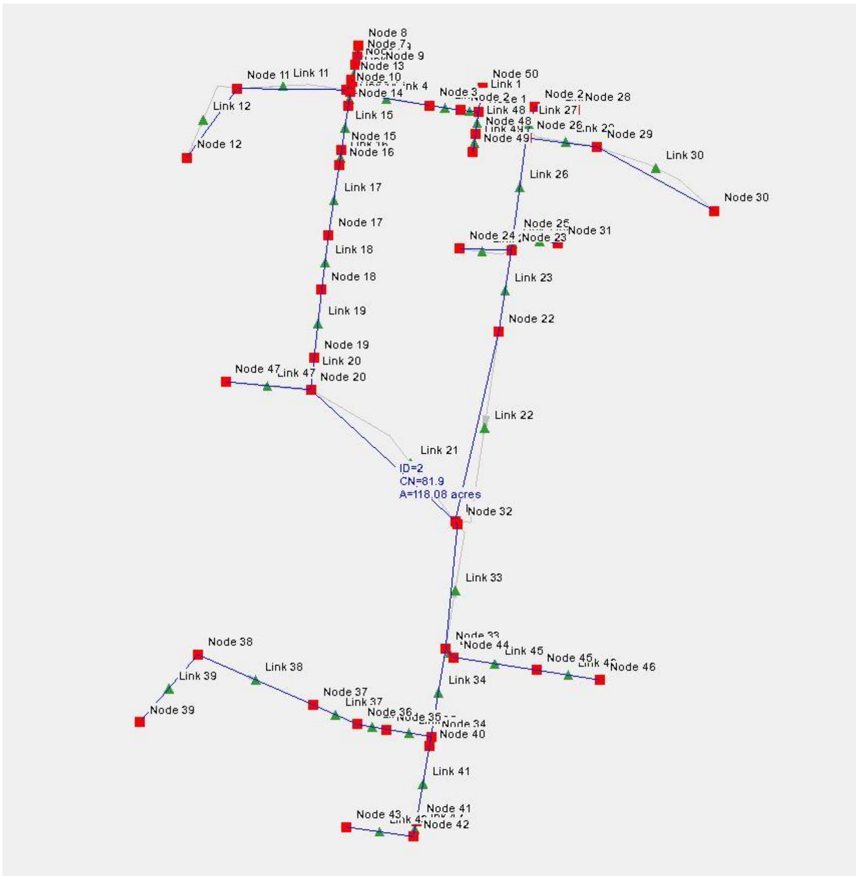

Figure 6.
Links and Nodes in the Sewer Network in WMS.

Once the dual drainage model was completed in WMS, the model was saved and executed in EPA-SWMM from WMS as directed by the WMS 8.4 tutorial (Aquaveo 2012). The dual drainage model sewer network was saved in EPA-SWMM for repeated use by importing it into multiple runoff coefficient and precipitation variations for comparison (table 5).

The side profile of the multi-line sewer junction can be seen in the EPA-SWMM profile plot that shows how the pipes change from 10 feet (120 inches) in diameter to feeding the outflow pipe which is 7.5 feet (90 inches) in diameter (figure 7). The time-lapse pipe that feeds into the multi-line junction node is 1.67 feet (20 inches) in diameter (figure 8). This demonstrates the inconsistency in the sewer network, where a larger diameter pipe drains into a smaller diameter pipe effectively creating a 'choke point' at the multi-line junction node.

Once the multi-line pipe and the outflow pipe in this study reaches and exceeds 1.66ft in depth at the multi-line junction node for an extended period of time the time-lapse pipe would then become inundated. The flow from upstream of the time-lapse node would start to back up and eventually become inundated to the point a surcharge would then exist on the surface. The WMS and EPA-SWMM dual drainage model does not include the surcharge reentering the system (Nania, Leon, and Garcia 2014).

Table 5.

Dual Drainage Model Variations

Runoff Coefficient	T_c	Precipitation Frequency
Low	15min	February 2nd 2016
High	15min	February 2nd 2016
High	15min	1 Year
High	15min	2 Year
High	15min	5 Year
High	15min	10 Year
High	15min	25 Year
High	15min	50 Year
High	15min	100 Year

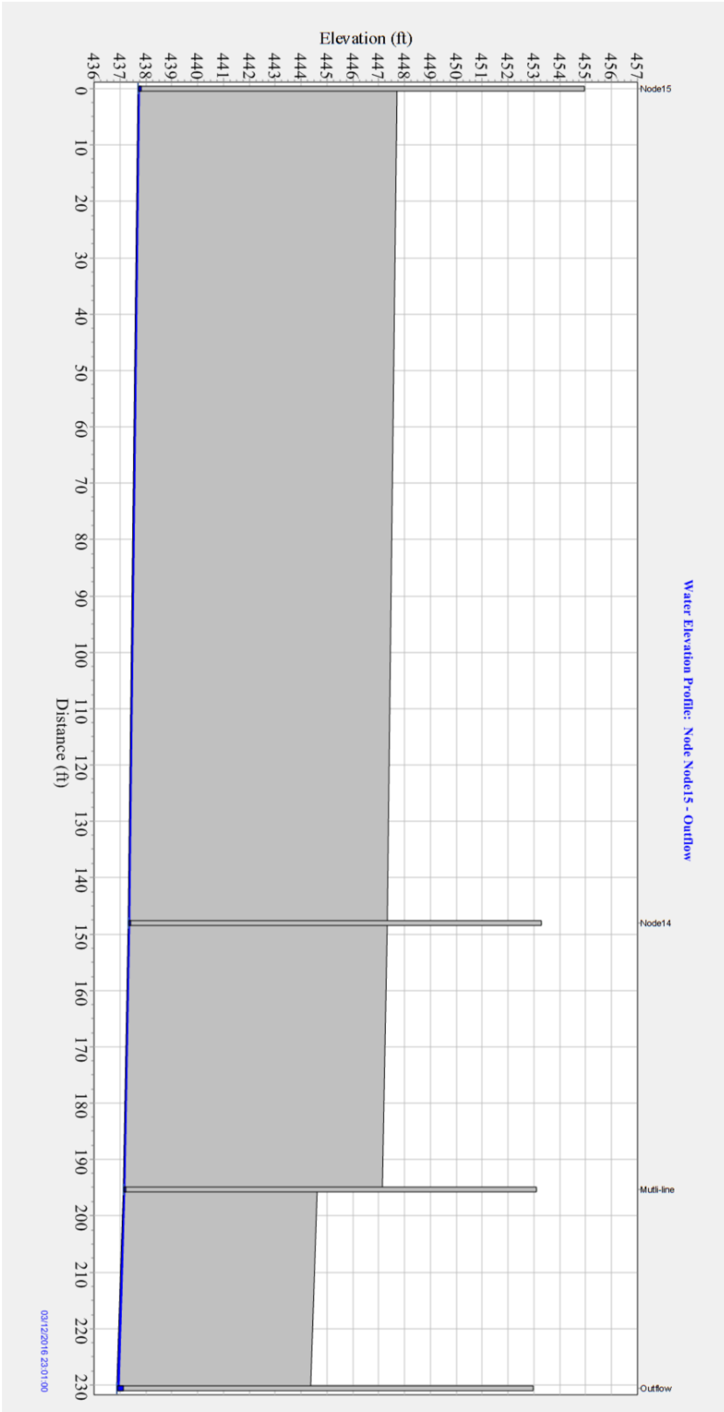

Figure 7.
Side Profile of the sewer network links that lead to the Multi-line Junction and outflow node and link.

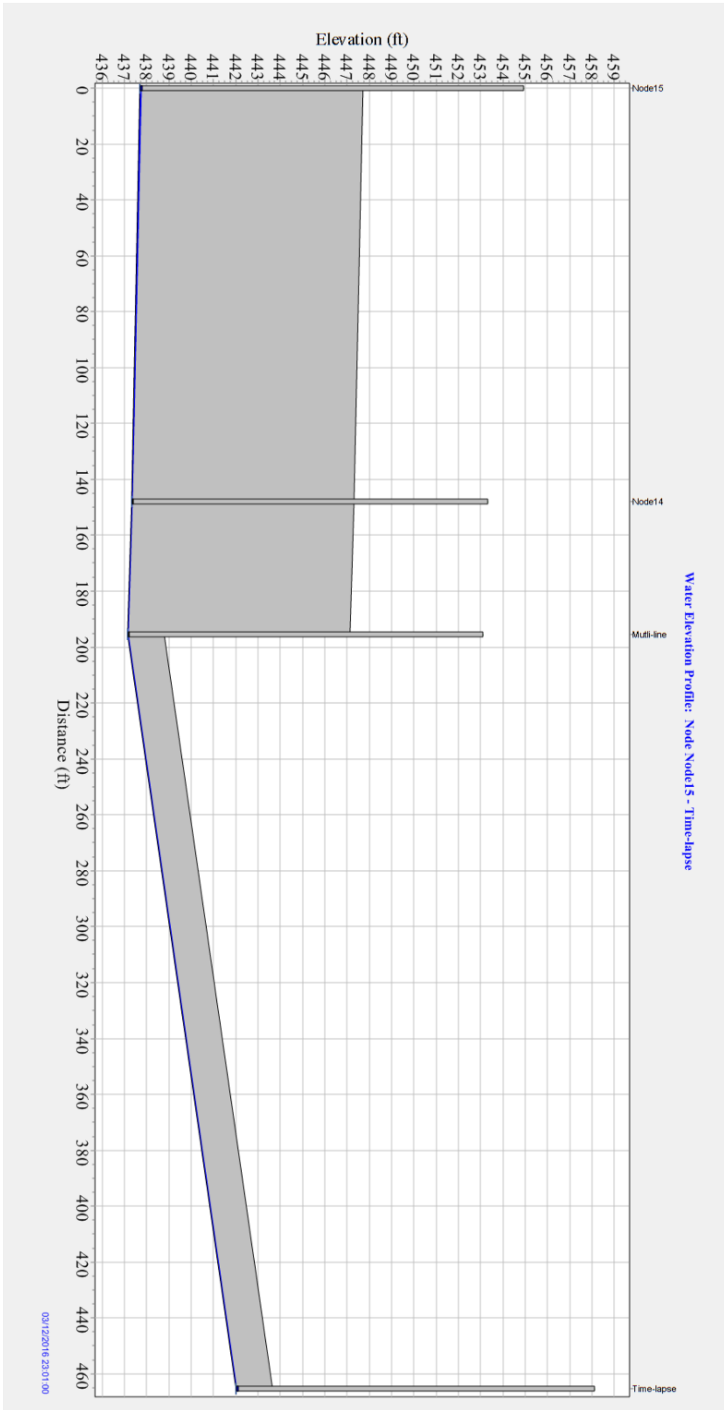

Figure 8.
Side Profile of the sewer network links that lead to the Multi-line Junction and the time-lapse node and link that also leads to the multi-line junction.

4.6 Campus Urban Flooding Event

The first documented campus flood caught on time-lapse occurred on February 2nd 2016 at 11:50pm (figure 9), peaked at 11:55pm (figure 10) and ended at 12:00am (figure 11) on February 3rd lasting approximately 10 minutes. While the depth generated was only a few inches (figure 10) this flooding event provided photographic evidence that the sewer network was at maximum capacity at the location of time-lapse storm drain node which produced a surcharge as a result of sewer system inundation. The precipitation data for this event was collect from MSD online rain gauges and used in the dual drainage model which produced two hydrographs for the February 2nd storm event, one with the low runoff coefficient, and one with the high runoff coefficient. The time-lapse images were used to verify the dual drainage model by comparing the time series data of the campus flood to the time of maximum depth of the time-lapse node as seen in the model's hydrographs (figures 13 and 14).

Figure 9.
Time-lapse image of urban flood start time

Figure 10.
Time-lapse image of urban flood peak time.

Figure 11**.**
Time-lapse image of urban flood end time.

5.0 RESULTS

The results of the EPA-SWMM dual drainage model's hydrographs showed the sewer system at the time-lapse storm drain node reaching maximum depth as a result of the multi-line junction node or choke point being inundated which was verified with time-lapse images of campus flooding on February 2nd 2016. The hydrograph and time series data showed that particular flooding event for an approximately similar amount of time.

The only variable used in the dual drainage model that required calibration were the surface runoff coefficients. Since the time series data of the February 2nd flooding event produced specific rainfall data, the time of concentration is a physically calculated value that reflects the terrain mathematically. With fixed Tc and rainfall data this only leaves the runoff coefficient to be calibrated in this model. Using the Feb 2nd event data the dual drainage model was executed with the low and high runoff coefficients (Chow 1964; Maidment, and Mays 1988; Chow, Maidment, and Mays 2013; Kentucky Transportation Cabinet. 2010) producing two hydrographs (figure 13 and 14). The hydrograph in figure 13 does not show the time-lapse node (solid blue line in figure 13) reaching maximum depth at any time. The hydrograph in figure 14 however matches the time period seen in the time-lapse images running approximately 10 minutes.

In order to examine the multi-line junction as a factor for the time-lapse sewer node reaching maximum capacity and subsequent surcharge, we compared the multi-line junction to the time-lapse node and the outflow node for all model precipitation variations. In all of the hydrographs produced we can see the multi-line junction node and the outflow node depths are similar throughout the hydrograph. The flow of the multi-line junction link falls below the outflow link as a result of the time-lapse link draining into the multi-line junction node and then to the outflow link giving the outflow link an increased flow rate. It should be noted the increase from the time-lapse link to the outflow link matches the flow rate of the time-lapse link itself.

The model variations for different precipitation frequency years that produced a surcharge does not occur for the 1 year precipitation frequency, but does occur at all frequencies above this (table 6). The maximum depth of the time-lapse node is 1.67 feet (20 inches), but the 1 year PF hydrograph (figure 15) does not reach this maximum depth. For the 2 year PF, the hydrograph shows the depth reaching this maximum for a few minutes. The observed comparison of the Time-lapse Node Depth (solid blue line) between the 1 year PF (figure 15) and the 2 year PF (figure 16) demonstrates the sensitivity of the time-lapse storm drain location to relatively small rainfall events.

Table 6.

Dual Drainage Model Variation Results

Runoff Coefficient	T_c	Precipitation Frequency	Produces Inundation	Figure
Low	15min	February 2nd 2016	No	13
High	15min	February 2nd 2016	Yes	14
High	15min	1 Year	No	15
High	15min	2 Year	Yes	16
High	15min	5 Year	Yes	17
High	15min	10 Year	Yes	18
High	15min	25 Year	Yes	19
High	15min	50 Year	Yes	20
High	15min	100 Year	Yes	21

When examining the maximum depth, flow rate, and inundation time for the 5, 10, 25, 50 and 100 year model PF variations (table 7) the time-lapse node depth reaches maximum depth in the 2 year PF (table 7 and figure 16) Above the 2 year PFs the depth does not increase, however the inundation time does increase up to approximately 10 minutes for the 100 year PF (table 7 and figure 21). The diameter of the time-lapse link is 1.67 feet (20 inches). As the time-lapse node depth extends in time, the multi-line junction and outflow node depth continues to increase but does not reach maximum depth in any of the hydrographs because the diameter of the multi-line link is 10 feet (120 inches) and the outflow link is 7.5 feet (90 inches). This comparison between the maximums of the hydrographs of the 2, 5, 10, 25, 50 and 100 year PF (table 7) clearly demonstrates the effects of the multi-line junction as a choked point on the time-lapse node.

The peak flow of the time-lapse link in the February 2nd storm event also supports the peak time of the time-lapse node depth (figure 14) and time-lapse camera flood (figure 10) to be approximately 5 minutes from the beginning of time-lapse node maximum depth and beginning of the surcharge on time-lapse camera (figure 9) and approximately 5 minutes before the end of the surcharge seen in figure 11.

The external inflow and outflow for all model variations shows the volumes in cubic feet (table 8) and the continuity error for each model variation. The total inflow includes rainfall and initial watershed storage. The total outflow includes evaporation, infiltration, runoff and final storage. The continuity error expresses the amount of water lost or gained in surface and subsurface routing and provides a measure of the numerical accuracy of the models performance. This value can be either negative or positive. Continuity error is calculated from the total external outflow divided by the total external inflow (Huber at el. 1981; Huber and Dickinson 1988; Rossman 2009). Continuity errors below 1 percent are considered “excellent”, 2 percent are “great”, 5 percent are “good” with continuity errors above 10% require further examination of the model and corrections to model components (Huber at el. 1981; Huber and Dickinson 1988; Rossman 2009).

The continuity error values of all the model variations (table 8) are below 2 percent and all models that produced a surcharge for any amount of time are below 1 percent, which shows a higher level of integrity within the model.

Inundation times for all precipitation frequency variations range between four to ten minutes in length. While the time-lapse camera shows a surcharge for approximately ten minutes, the surcharge threshold for the time-lapse storm drain should exist within the 1 year PF and the 2 year PF.

Table 7.

Maximum Depth, Flow Rate, and Inundation Time for Dual Drainage Model Variations

	Multi-Line		Time-Lapse		Outflow		Inundation
	Depth (ft)	Flow (cfs)	Depth (ft)	Flow (cfs)	Depth (ft)	Flow (cfs)	Time (mins)
Feb 2nd Low	2.01	89.44	1.47	9.72	2.01	99.32	0.0
Feb 2nd High	2.36	121.79	1.67	11.20	2.36	135.61	10.0
1 Year PF	1.92	82.05	1.38	9.01	1.92	91.18	0.0
2 Year PF	2.07	95.31	1.67	9.73	2.07	105.47	4.0
5 Year PF	2.14	101.11	1.67	10.07	2.14	112.70	5.0
10 Year PF	2.20	106.49	1.67	10.37	2.20	118.86	7.0
25 Year PF	2.29	114.61	1.67	10.77	2.29	127.93	8.0
50 Year PF	2.33	118.56	1.67	11.01	2.32	131.82	9.0
100 Year PF	2.36	121.53	1.67	11.21	2.36	135.85	10.0

Table 8.

External Inflow and Outflow Volumes for the Dual Drainage Model Variations

	External Inflow (cu.ft)	External Outflow (cu.ft)	Flooding Loss (cu.ft)	Continuity Error (%)
Feb 2nd Low	206,997.1	144,793.4	59,590.1	1.26
Feb 2nd High	286,624.8	190,879.9	94,438.1	0.47
1 Year PF	189,006.8	133,990.6	52,620.5	1.27
2 Year PF	219,847.3	153,113.4	64,773.7	0.90
5 Year PF	236,225.9	162,565.9	71,830.4	0.79
10 Year PF	249,598.8	170,232.5	77,623.9	0.70
25 Year PF	267,284.2	179,989.9	85,508.3	0.67
50 Year PF	278,871.1	186,741.7	90,822.6	0.46
100 Year PF	287,539.6	191,359.1	94,830.1	0.47

To find the minimum range of external inflow volume needed for a surcharge in the time-lapse node we examined all variations by external inflow and external outflow with $R^2 = 0.9995$ (figure 12). Since inundation occurs at the 2 year precipitation frequency and above (figure 12), this shows that inundation begins between 207,000 cu.ft and 219,000 cu.ft or within about a 12,000 cu.ft range of the February 2nd Low variation.

Figure 12 also shows inundation can exist with an external outflow between 145,000 cu.ft and 153,000 cu.ft or higher.

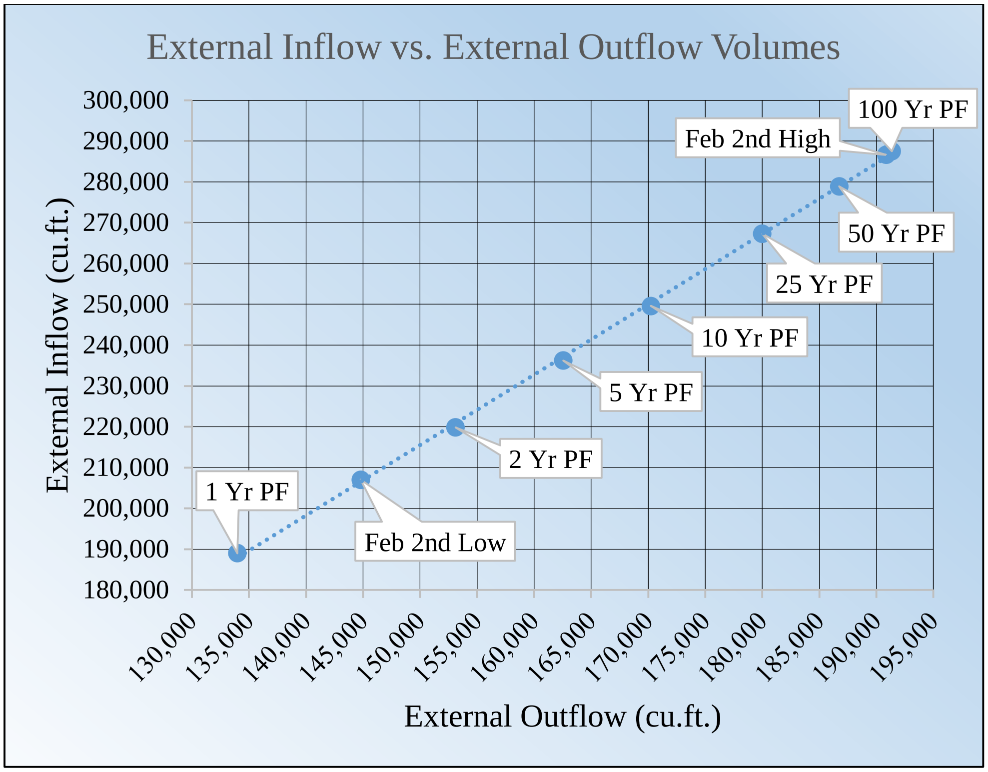

Figure 12.
External Inflow and External Outflow Volumes.

The external inflow volumes seen in table 8 shows that the Feb 2nd high precipitation, 25, 50 and 100 year PFs appear to exceed the maximum capacity of the sewer network for the study area at 253,175.6 cubic feet. While it may appear that the external inflow exceeds the maximum capacity of the network the volume of water of the external outflow (table 8) is also leaving the model system for the entire time length of the model simulation so the system does not necessarily reach maximum capacity in any of the variations. The inundations produced for the relevant scenarios, shown in table 7, are instead a function of the choke point in the system which restricts the flow of the water leaving rather than system capacity exceedance.

Further evidence of the multi-line junction being a choke point can be seen in any of the hydrograph variations where a surcharge occurs. In all of the graphs with a surcharge, the multi-line depth increases and exceeds the maximum depth of the time-lapse node, this is when the surcharge begins in the time-lapse storm drain.

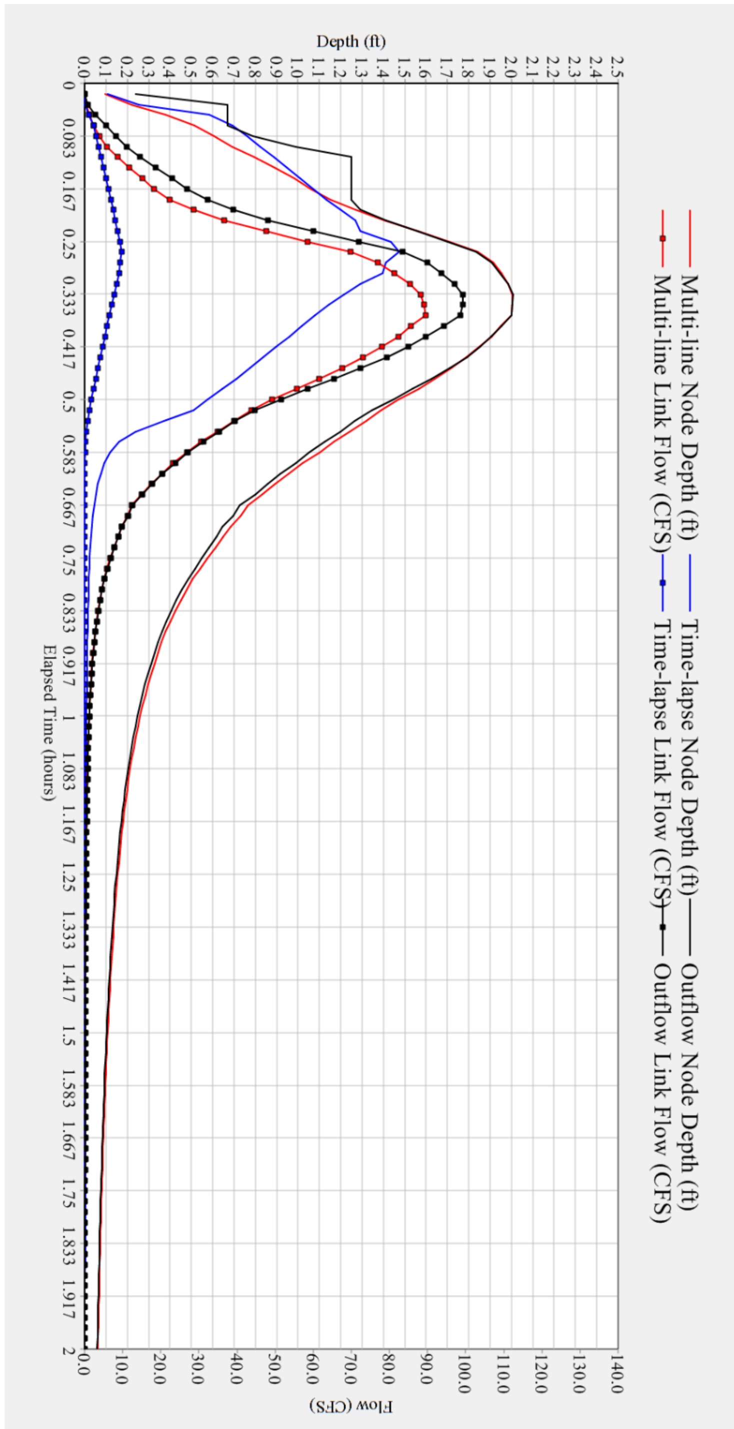

Figure 13.

February 2nd 2016 Storm Hydrograph with Low Runoff Coefficient

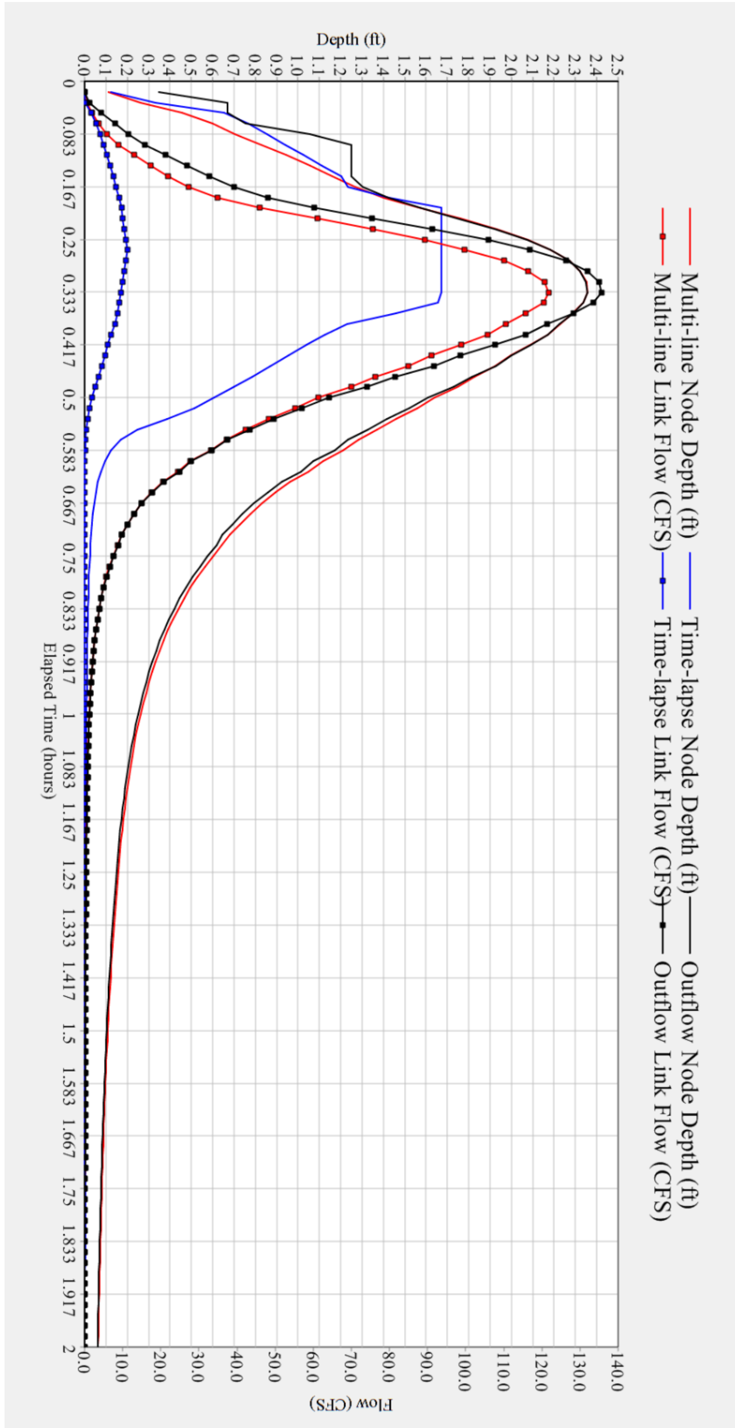

Figure 14.
February 2nd 2016 Storm Hydrograph with High Runoff Coefficient.

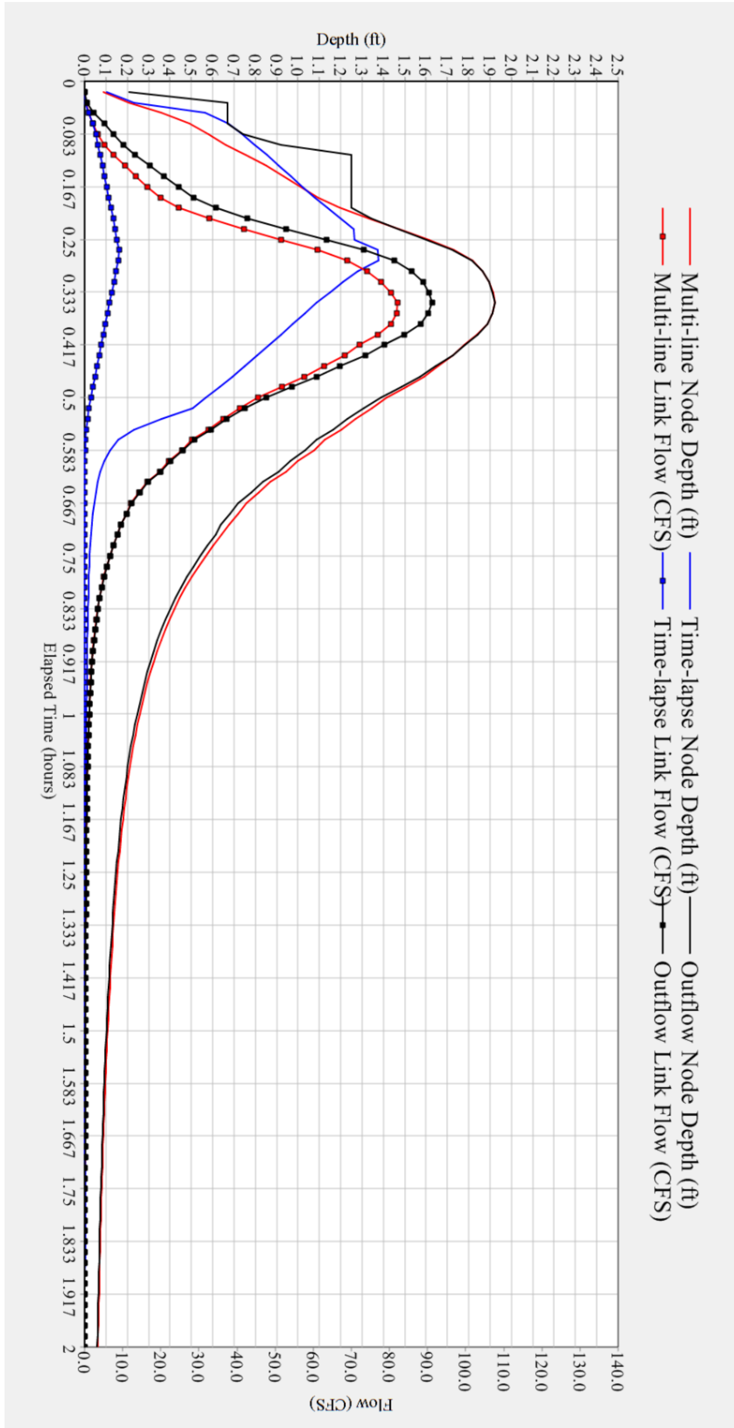

Figure 15.
1 Year PF Hydrograph with High Runoff Coefficient.

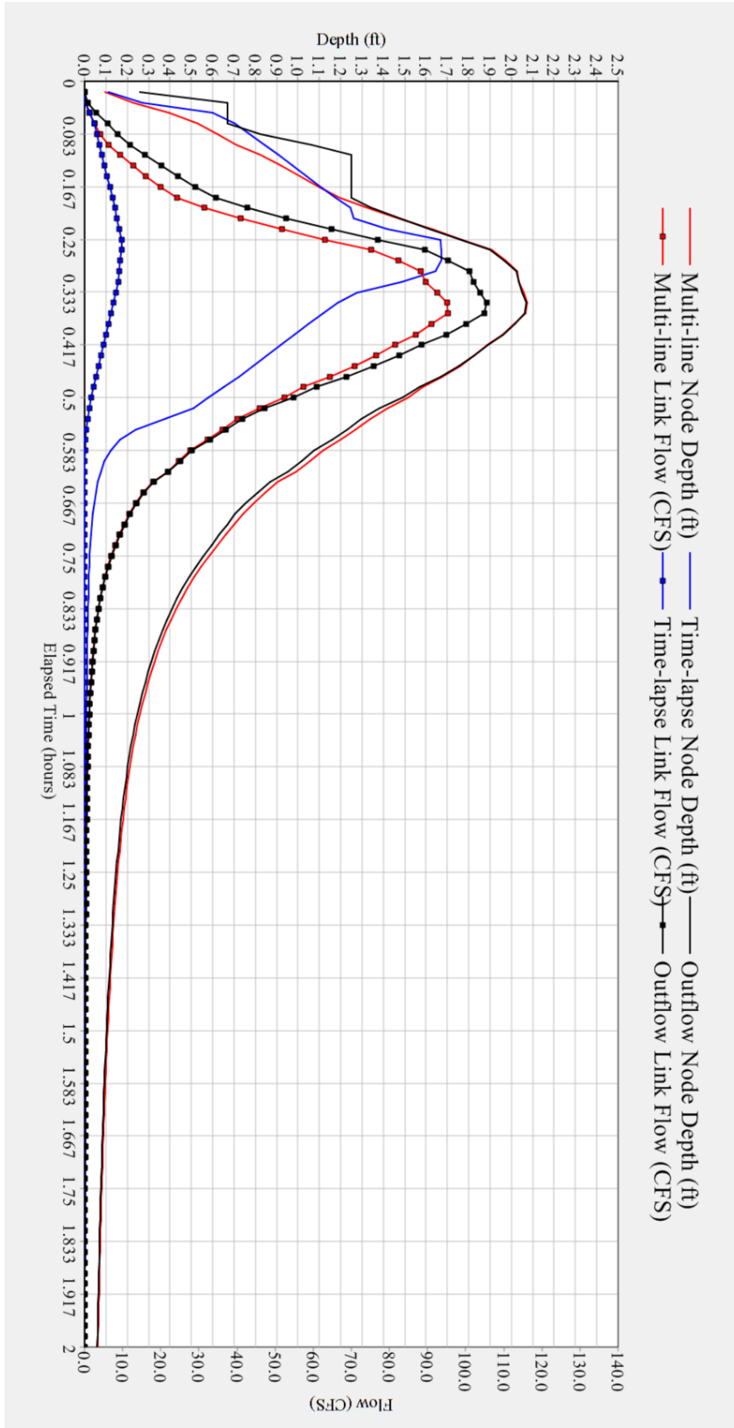

Figure 16.
2 Year PF Hydrograph with High Runoff Coefficient.

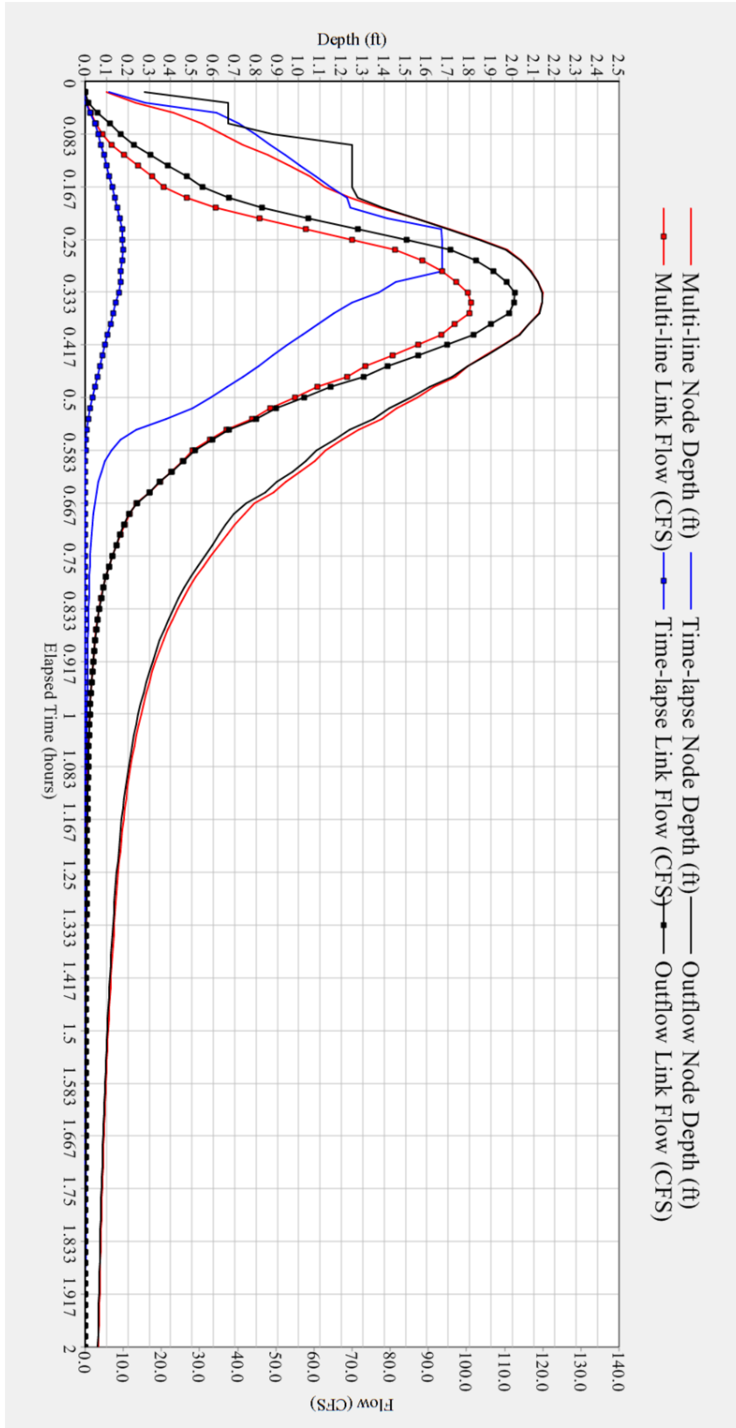

Figure 17.
5 Year PF Hydrograph with High Runoff Coefficient.

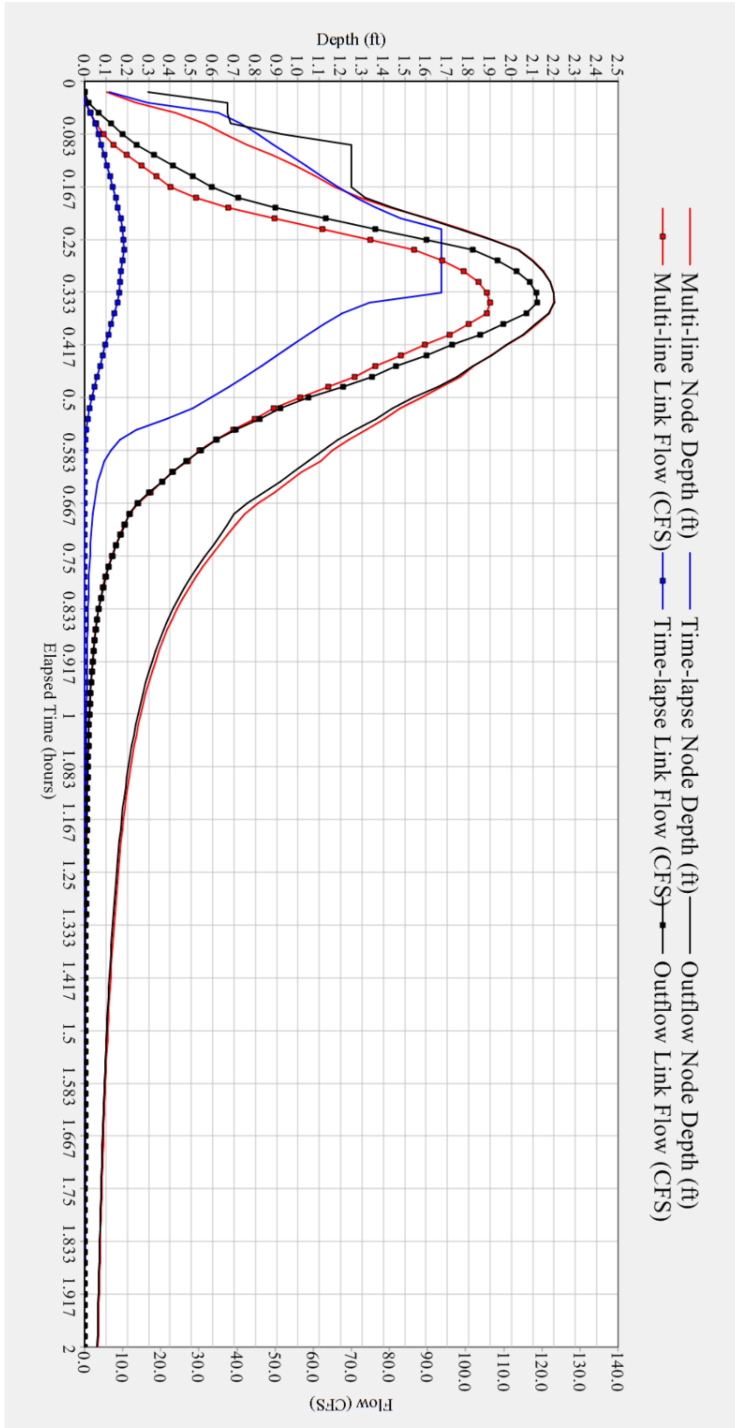

Figure 18.
10 Year Storm PF Hydrograph with High Runoff Coefficient.

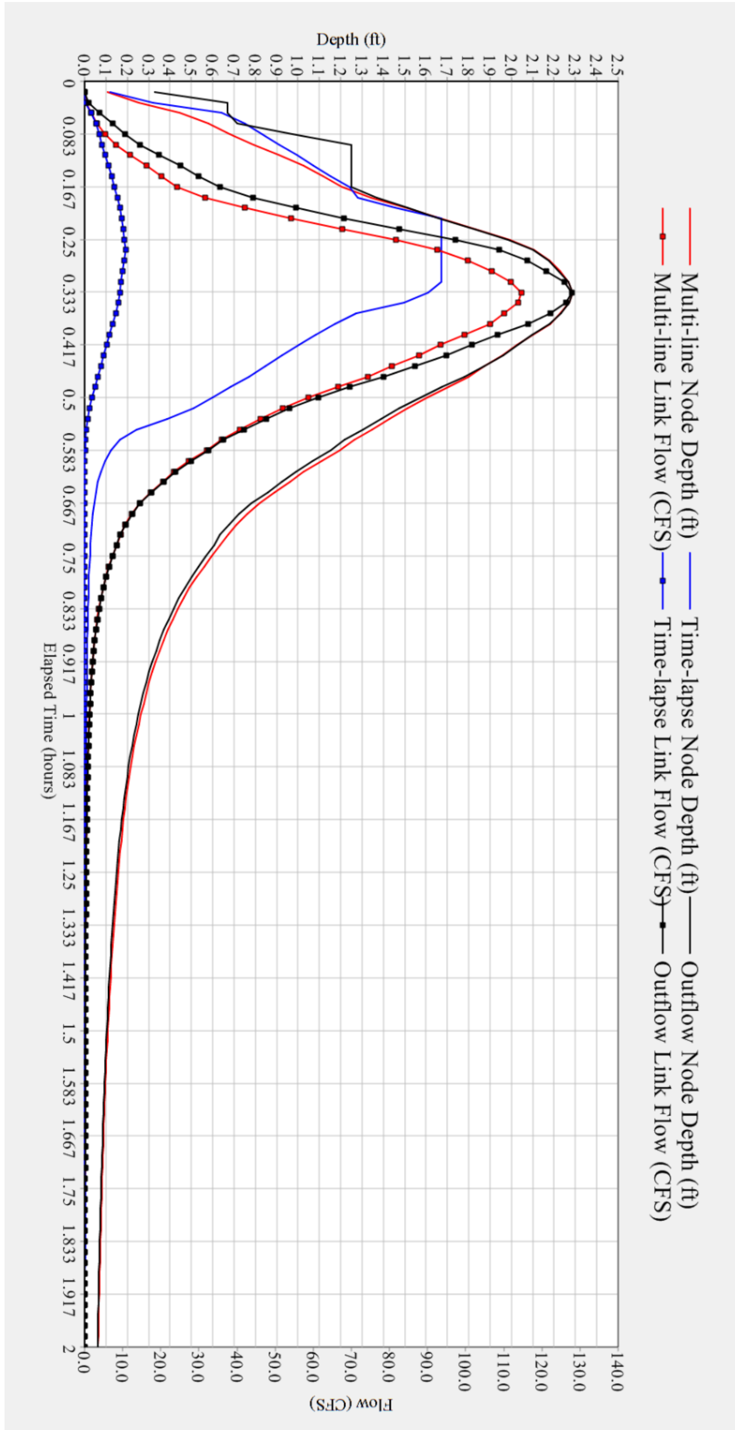

Figure 19.
25 Year Storm Frequency Hydrograph with High Runoff Coefficient.

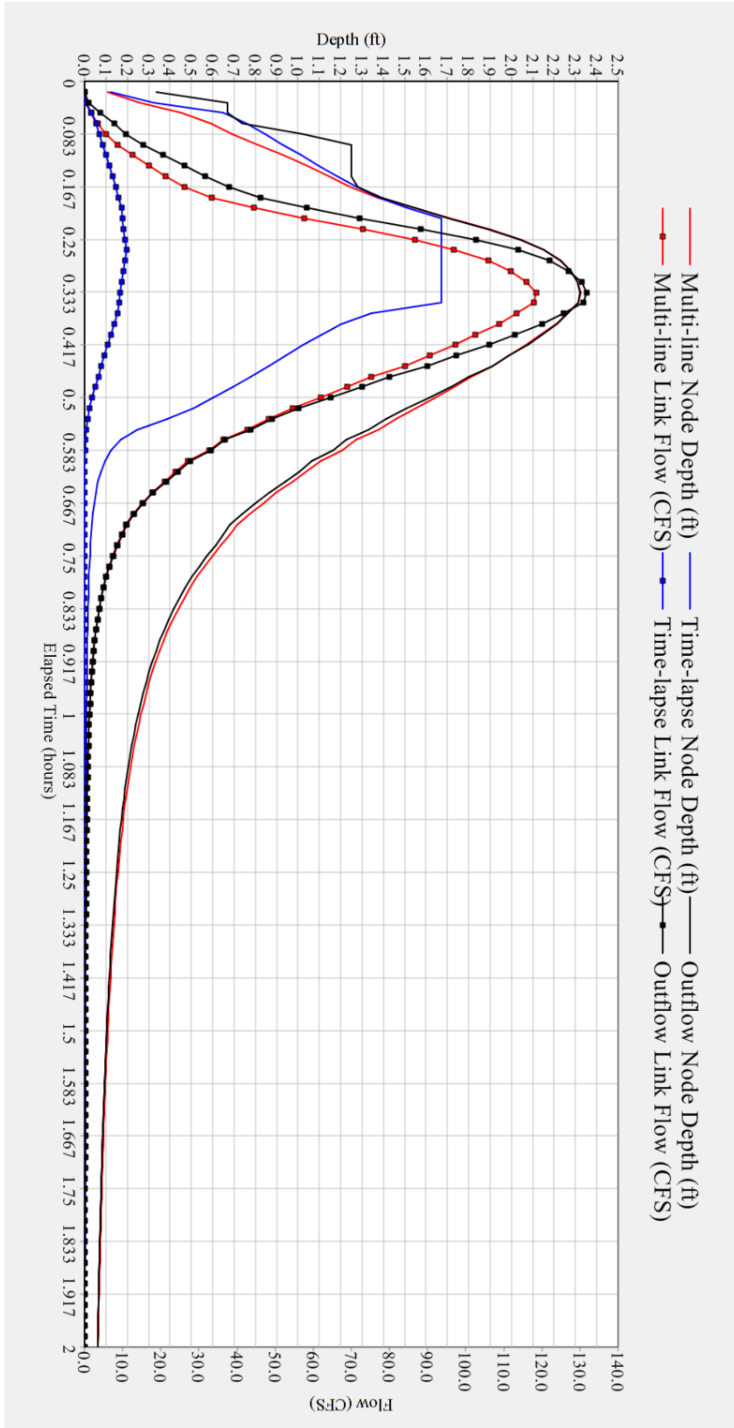

Figure 20.
50 Year Storm Frequency Hydrograph with High Runoff Coefficient.

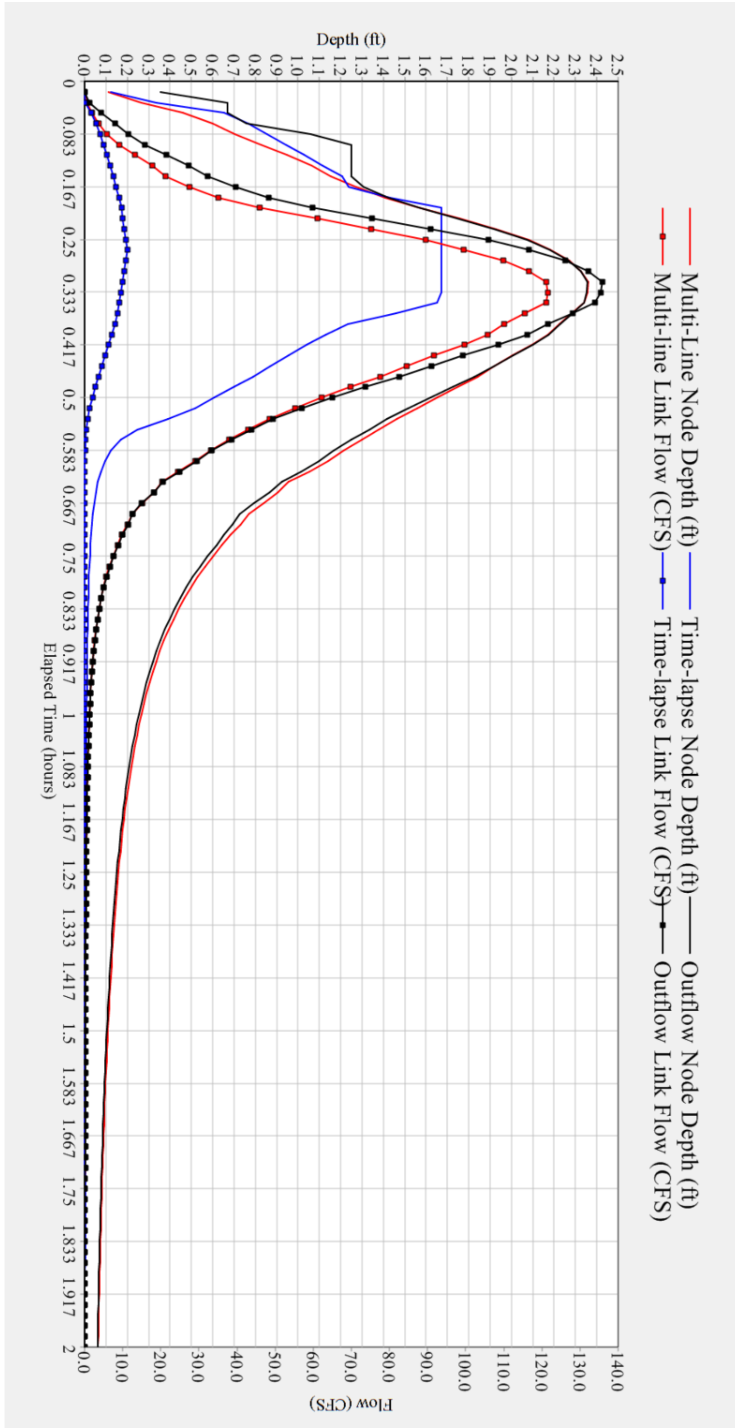

Figure 21.
100 Year Storm Frequency Hydrograph with High Runoff Coefficient.

6.0 CONCLUSION

This research provides several significant outcomes, the first being the use of time-lapse photography in studying urban flooding. Time-lapse images used as time series data can be used to calibrate the WMS/EPA-SWMM's dual drainage model and validate the model's hydrograph. Second a better understanding of the various factors that contribute to urban flooding on the University of Louisville Belknap Campus from both the dual drainage model and photographic evidence. Thirdly is a methodology in developing models that can help assist urban flooding researchers and urban planners in developing safety measures that can possibly reduce hazards for students, faculty and staff, as well as possibly reduce future flood damage through providing a tool in predicting flood events using real-time rain fall data. It is important to the success and the safety of the University of Louisville, both students and faculty to study campus flooding as both a natural and anthropogenic hazard.

According to the results of dual drainage models in this study precipitation intensity and inconsistent multi-line sewer junctions appear to play a central role in urban flooding. Since it proves to be costly to fix many of the sewer network junctions to reduce the number of choke points it places the focus of further research on factors that influence precipitation intensity, for example climate change. According to Walsh et al. (2014), the number of heavy rainfall events has increased significantly in intensity across the US over the past few decades. The increase in amount of rainfall results from a warmer atmosphere which can hold more water vapor as compared to cooler air. It is suggested that with increasing temperature projections that there will also be an increase in the amount of water vapor which leads to heavier precipitation events. Walsh et al. (2014) further notes that the south eastern US, including Kentucky has seen a 27% increase in heavy precipitation events from 1958 to 2012.

Along with climate change the urban heat island effect also affects local air temperatures. Warmer urban environments can increase the amount of water vapor in the air on top of the already increasing air temperatures caused by climate change. Warmer air in urban environments also acts as a natural green-house gas which assists in trapping heat, which leads to more water vapor in the air.

Further research could also examine the full upstream accumulation of the sewer network on the university, which would include the entire sewer network that feeds into the study's multi-line junction node. This would include over 250+ sewer pipes and an additional surface area that would also include residential and additional industrial zones.

Further research could also include Manning's roughness coefficient, which describes the roughness of a particular surface that produces surface friction (Chow 1964; Hendricks 2010). The Manning's roughness coefficient could be included for each surface type to the land cover data to increase the accuracy of the results for the time of concentration. It could also be useful to include Manning's roughness coefficients in defining a more detailed surface drainage flow path for the entire study area that would include every street and drainage ditch.

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