Industrial Application for Storm Water Modeling—GE Aviation Case Study

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GE Aviation (GE) selected Camp Dresser & McKee (CDM)) to perform an in-depth evaluation of the storm water system at its Evendale, Ohio, plant site. The occurrences of rainfall derived flooding across the site, in both July 2001 and June 2009, and basement flooding in one of the buildings because of high groundwater levels, each displaced personnel and interrupted normal work and production activities. As GE embarks on the revitalization of the 1940s-era site, this phased storm water program will provide findings and recommendations that GE can integrate into its overall improvement plans for the site.

15.1 Data Collection and Problem Identification

15.1.1 Site Description

Construction of the site currently occupied by the GE Aviation plant began in October 1940, and the production of engine parts started in April 1941. The site now provides headquarters, administrative, engineering, laboratory, manufacturing, engine assembly and testing functions to the GE Aviation business. The facility occupies approximately 420 acres (170 ha), of which more than 138 acres (55.8 ha) is building rooftop. The site is bordered on the west by I-75, Mill Creek on the east, Glendale-Milford Road to the north and Shepherd Lane to the south. The site is shown in Figure 15.1.

Figure 15.1 Overall watershed boundaries.

The site has separate storm and sanitary sewer systems, a municipally supplied potable water supply, a private industrial water supply, a water

treatment plant, process water, cooling water systems, and requires an environmental permit to discharge to Mill Creek. Storm water runoff from offsite sources is conveyed across the site through concrete lined open channels, natural ditches, and storm sewer pipe. These storm waters originate in the Ohio Department of Transportation (ODOT) right-of-way (I-75), as well as in the municipalities of Lincoln Heights, Woodlawn, Glendale and Evendale. Modifications to the onsite drainage system were made in the mid-1990s to segregate offsite runoff from runoff originating onsite for certain storm events. Additional discussions and the implications of the commingling storm water flows are in Section 15.3.

15.1.2 Interviews

In November, 2009 CDM held four separate interview sessions to obtain first-hand accounts of past flooding occurrences from persons who have different responsibilities at the site. The interviewees had worked at the plant from 11 y to 32 y.

15.1.3 Historical Rainfall

Figure 15.2 Locations of the four MSDGC rain gauges used for the event specific rainfall analysis.

CDM evaluated historical rainfall data to perform event specific and long term rainfall analysis. The event specific evaluations used data collected by the Metropolitan Sewer District of Greater Cincinnati (MSDGC) for four recent rainfall events, three of which caused flooding at the site. The long term evaluations used 60 y data from the National Weather Service (NWS) for statistical analysis of rainfall intensity and duration. A reference document, *Rainfall Frequency Atlas of the Midwest*, was used to identify the recurrence interval of the evaluated storms based on rainfall duration. The event specific rainfall analysis used data collected by MSDGC at the following four rain gauge sites: Glendale-Milford Road, Sharonville, Wastewater Collections-Galbraith Road and Glenview Golf Course. Figure 15.2 locates the 4 MSDGC rain gauges relative to the site and Table 15.1 summarizes the information obtained from the gauges. During the July 2001 event, the Glendale-Milford Road gauge was not operational, thus the next three nearest gauges were used for event specific analysis.

Table 15.1 Summary of rainfall values and durations used for the event specific rainfall analysis.

Rainfall event date	Duration (h)	Total Rainfall Event			Peak hour Recurrence interval ⁵
		Depth (in.) [cm]	Recurrence interval ⁵	Depth (in.) [cm]	
July 17–18, 2001 ¹	4.75	6.39 (16.20)	>100 y	4.19 (10.6)	>100 y
July 17–18, 2001 ²	7.5	3.27 (8.30)	15 y to 20 y	2.01 (5.10)	15y to 20 yr
July 17–18, 2001 ³	5.3	3.78 (9.60)	35 y to 40 y	2.44 (6.20)	40 y to 45 y
June 2–3, 2009 ⁴	2	2.96 (7.52)	25 y to 50 y	2.80 (7.11)	100 y
June 26, 2009 ⁴	3	1.12 (2.84)	5 month	0.76 (1.93)	4 month
August 4, 2009 ⁴	9.25	1.56 (3.96)	6 month	0.64 (1.63)	2 month

¹ Based upon rainfall measured at WW Collections–Galbraith Road gauge.

² Based upon rainfall measured at Sharonville gauge.

³ Based upon rainfall measured at Glenview Golf Course gauge.

⁴ Measured at Glendale–Milford Road gauge.

⁵ Recurrence interval as documented or interpolated from *Rainfall Frequency Atlas of the Midwest*.

Rainfall data collected at a rain gauge within the facility was reviewed to evaluate the July 2001 rain event at the site. At 15:00 on July 17 the gauge indicated no rainfall accumulation. At 00:00 on July 18 the gauge indicated 2.11 in. (5.36 cm) rainfall. When checked at 07:00 on July 18, the gauge indicated 0.21 in. (0.53 cm) with the comment “Debris” made. For the three rain gauges operational at the time, the peak hourly rainfall measurements occurred between 23:00 on July 17 and 01:00 on July 18. Given the proximity of the site (and its rain gauge), it is reasonable to assume that 2.11 in. (5.36 cm) fell around this same time period and can be considered representative of the peak hourly rainfall at the site for this event.

The long term rainfall analysis used data collected by the NWS at Wilmington, Ohio and approximately 60 y records were analyzed using a

program called NetSTORM. This evaluation was performed to identify trends in rainfall intensity and the duration of storm events based upon monthly statistics using 1 h precipitation depths measured in inches. Results of the long term rainfall evaluation are shown in Figures 15.3 and 15.4.

Figure 15.3 Long term (60 y) rainfall evaluation identifying the monthly mean and standard deviation of hourly precipitation depth.

Figure 15.4 Long term (60 y) rainfall evaluation identifying the monthly average rainfall event duration and maximum hourly precipitation.

Based on the event specific and long term rainfall analyses, the following conclusions can be made:

- the site realized extensive building flooding during rainfall events with a 100 y recurrence interval (June 3 event) and approaching a 50 y event (July 17 event) based upon peak 1 h intensities.
- the events causing basement flooding at GE the site had rainfall durations <6 h;
- statistically, high intensity rainfall is more likely to occur between May/June and October.
- short duration storms (<6 h) have correspondingly high intensities between June and September.
- the three events that recently caused building flooding at the site correlate well with the long term rainfall data analysis.

15.1.4 Stakeholder Collaboration

A number of stakeholders were contacted during the course of this study to identify synergies that could be mutually beneficial.

Hamilton County Storm Water District (HCSWD)

Shortly after the initiation of this storm water study, HCSWD approved a pilot study to quantify the efforts necessary to map its storm water infrastructure in accordance with National Pollutant Discharge Elimination System Phase 2 requirements throughout Hamilton County. The pilot study took place in Lincoln Heights, Woodlawn and Evendale and included much of the watershed upstream of the site. GE and HCSWD met a number of times during the five-month storm water study in order to maintain open communication, work together to meet similar objectives cost effectively, and share the surveyed storm water infrastructure collected by each.

Ohio Department of Transportation (ODOT)

As CDM began to gather storm water data on site and to communicate with the HCSWD, it became clear that additional coordination with ODOT was required as part of the storm water study. As well, CDM exchanged information with ODOT's design firms for the I-75 reconstruction project, which was in the early planning phases for the reach of I-75 adjacent to the GE facility.

15.1.5 Infrastructure Investigation

The storm water infrastructure for which data was collected includes onsite and offsite storm sewers, concrete lined open channels and natural ditches, four pump houses and associated electrical services, and two permitted site

discharge locations designated as outfalls 001 and 002. The information collected on this infrastructure was obtained by:

- field survey;
- research and review of historical electronic drawing files;
- research and review of historical paper drawing files and reports; and
- site visits, photo documentation and staff interviews.

The onsite survey also included concrete lined open channels and natural ditches. Cross-sections of the channel were surveyed and tied to continuous stationing along the channel centerline.

The offsite survey was performed under HCSWD's direction. The main tributaries of offsite storm sewers were identified, and all relevant data necessary to create a calibrated computer model was assembled. This information was transferred electronically in a GIS format. To supplement the site survey data, historical electronic and paper files were researched and reviewed to assist with understanding the overall storm water system and the site and buildings.

The infrastructure investigations also included site visits for a visual assessment of the physical systems being surveyed and researched through paper documents. Through visual observations and photo documentation, the connectivity and condition of the system became better understood. Based on water levels in the open channels and through culverts, it also provided data used later during model calibration.

15.2 Hydraulic Capacity Analysis

15.2.1 Model Selection

The Stormwater Management Model version 5.0 (SWMM 5) was used to develop the watershed model for the site. SWMM 5 uses various data to generate the peak rates and volumes from individual subcatchments (smaller drainage areas within each watershed), which include the area of the subcatchment, the percentage of the area that is impervious, the soil infiltration information, the slope of the watershed and rainfall information.

Percentage Impervious

The percentage of impervious coverage for each subcatchment within the site, as well as the offsite areas tributary to the facility, was obtained from HCSWD's utility billing file, which contains the percentage of impervious coverage for each parcel, coupled with estimates from aerial photogrammetry.

Soil Infiltration

The soil infiltration data (such as infiltration rates and capacities), which is necessary to establish the amount of runoff from the pervious surfaces, were obtained from typical engineering values for the various hydrologic soil groups present in Hamilton County (i.e. A, B, C and D). Soil coverages within each watershed were obtained from digital hydrologic soil group files available for Hamilton County, which included the urbanized areas.

Watershed Slope

Watershed slope, which is important in establishing the proper timing of hydrograph peak flows, was calculated based on the topographic contour lines available from the Cincinnati Area Geographic Information Systems (CAGIS), maintained by the City of Cincinnati Metropolitan Sewer District and others.

Rainfall

Two different types of rainfall data were used during the course of this study: design storm rainfall events, and observed rainfall for specific events that either created flooding conditions within the site or were experienced firsthand by CDM or GE personnel.

The design rainfall information used peak rainfall values from the *Rainfall Frequency Atlas of the Midwest* (Bulletin 71) and rainfall distribution from the Natural Resource Conservation Service, formerly the Soil Conservation Service, and *Technical Paper 40* (TP 40). Design rainfall values for the 1 y, 2 y, 5 y, 10 y, 25 y, 50 y and 100 y rainfall events were created to determine the ability of the primary drainage system to convey flow under a broad range of rainfall and runoff conditions. The observed rainfall information is detailed in subsection 15.3.2 below.

Open Channel Cross Sections, Bridges and Culverts

Data and dimensions depicting the open channel cross sections, bridges and culverts are necessary to calculate the flow and velocity in the open channels. The information was obtained from the following sources:

- Open channel cross sections: The shapes and geometries of the open-channel cross sections were obtained from a combination of field survey information, where available, and CAGIS 2 ft contour interval topography. Roughness coefficients for the main channel and overbanks were obtained from a combination of field observations and aerial photogrammetry; and

- Bridges and culverts: The sizes and shapes of these structures were obtained from a combination of field measurements and record plan data (if available). During the field measurements, the condition of the channel upstream and downstream of each structure was noted to develop representative entrance and exit losses and roughness coefficients.

Boundary Conditions

All of the drainage from the site, as well as the offsite areas that are routed around the site, are tributary to Mill Creek. Flow from outfall 001 and the offsite areas discharges into open channels that convey the runoff to Mill Creek. Flow from outfall 002 is conveyed through a largely closed-conduit system to Mill Creek. See Figure 15.5 for the location of the outfalls.

Two approaches were taken to establish boundary conditions for the various design storms modeled. For design storms other than the 100 y event, all outfalls were considered to be free of any downstream influence from Mill Creek (i.e. free discharge). This approach was chosen due to the expected large differences in time between the occurrence of peak flood elevation for any flood event along Mill Creek and the occurrence of peak flood elevations at the site. The Mill Creek watershed is very large when compared to the watershed area that is tributary to the site and the peak flow within Mill Creek would likely occur sometime after the peak flow has already passed through the site. This would allow for the site drainage to work largely as a free outfall to Mill Creek.

For the rainfall events that were analyzed, a similar approach was taken, with the additional reason for a free outfall being a lack of evidence indicating the level in Mill Creek served as a downstream hydraulic constraint. No events occurred during the study that would allow for observations to validate this approach. For the 100 y event, the flood insurance study (FIS) for Hamilton County was used (*Flood Insurance Study for Hamilton County and Incorporated Areas*, May 17, 2004) to establish the 100 y flood elevation along Mill Creek at the locations of the outfalls to the creek. Rather than use a constant elevation for the boundary condition a simple triangular hydrograph was created using the FIS elevation as the peak at the midpoint of the rainfall hydrograph to establish a condition where Mill Creek was at peak elevation at the approximate time of the peak from the site, which creates the most conservative conditions for analysis.

Storm Sewer System Connectivity

To develop the physical characteristics of the storm water conveyance system, multiple data sources had to be reviewed and combined into one complete database of information. The data sources that were used included offsite

information obtained from ODOT and HCSWD, as well as a field survey of the storm sewer within the site, and limited data within the buildings onsite. Within the limits of the site, the primary source of storm sewer information was from a field survey that was performed on a portion of the storm sewer system that was deemed to be critical to the operation of the system.

Figure 15.5 Site detail.

The field survey was combined with historical information on the storm sewers within the site to create a more complete understanding of the drainage within the site as no consistent source of record drawings existed. ODOT was able to provide limited information on the storm sewer infrastructure within the I-75 right-of-way, which was used to extend the known drainage within the site to the limits of the Village of Lincoln Heights. To more fully understand the pathways that storm water runoff follows to enter the site, it was necessary to gather information on the storm sewer system within Lincoln Heights.

HCSWD was in the process of beginning to map the storm sewer system within their member jurisdictions at the same time that GE was initiating this project. Through a collaborative effort, it was determined that HCSWD could begin its mapping project within Lincoln Heights and provide CDM with the information necessary to complete the SWMM model for this project. HCSWD provided digital information on the location, size and inverts of the storm sewer system within Lincoln Heights, and then the same information within Evendale and Woodlawn. See Figure 15.5 for the locations of the various objects for which information was obtained.

Each of the data items described above was then combined within one database of information to develop the necessary physical parameters for the SWMM model, as well as to provide a comprehensive database for gaining a better understand how drainage flows from the west of I-75, through the I-75 corridor and then through, or around, the site prior to discharging into Mill Creek.

15.2.2 Model Calibration and Validation

Model calibration and validation is the process by which results generated by the SWMM model are compared with conditions observed in the field during specific rainfall events. This process demonstrates how closely the model can recreate observed field conditions, and, if within allowable tolerances, provides an additional level of confidence in the model's ability to predict design storm conditions. To calibrate the model, it is necessary to obtain monitored rainfall data for at least two rainfall events and the subsequent high water mark data that is observed within the storm water collection system. One rainfall event is used to adjust or calibrate model parameters such that the model results are closely matching field observed conditions and a second independent rainfall event is used to validate that the model, as adjusted and calibrated, can recreate field observed conditions without any further adjustments being required.

To determine which rainfall events should be used for comparison with model results, GE personnel provided the dates of three events that caused flooding conditions within the facility. One additional event was selected because of eyewitness accounts of a rainfall event that occurred during a field visit by CDM and GE personnel. The four events were selected based on the availability of data and are summarized in Table 15.2.

Table 15.2 Summary of rainfall events for calibration.

Date	Total rainfall (in) [cm]	GE rainfall (in) [cm]	Reason for use of event
July 17–19, 2001 ¹	3.425 (8.7)	2.56 (6.5)	Flooding event (validation)
June 2–3, 2009 ²	2.604 (6.6)	3.07 (7.8)	Flooding event (validation)
June 25–26, 2009 ²	0.929 (2.36)	1.08 (2.7)	Flooding event (calibration)
August 3–5, 2009 ²	1.589 (4.04)	1.66 (4.22)	Observed event (calibration)

¹ Sharonville rain gauge

² Calibrated radar rainfall data

A rain gauge exists within the site that provides rainfall totals recorded by GE personnel. This gauge was used to verify rainfall totals obtained from other sources as summarized in Table 15.3 as multiple gauges are within close proximity to the site with different rainfall totals for each gauge. Calibrated radar rainfall data was used for certain events as noted in Table 15.2 to supplement data from gauges in the vicinity. The radar measurements were collected using the National Weather Service's (NWS) NEXRAD (NEXt generation RADar). The radar images provide reliable aerial templates of rainfall at the desired resolution; they may not, however, provide data that represent actual ground capture rates. The individual precipitation gauges, if installed and maintained properly, provide reliable point estimates of the rainfall reaching the ground surface. Combining these two types of rainfall measurement techniques provides reliable aerial distribution of the rainfall, accomplished by calibrating (or ground truthing) radar data using rain gauge measurements.

In addition to the use of observed rainfall data, the calibration process requires that data be obtained regarding the nature of the flow, i.e., depth, velocity and total volume. The site has a monitor that measures only the volume of flow that passes over a control weir at Outfall 001 on the east side of the site. This monitor also provided the daily total volume for each of the observed rainfall events. This information is summarized in Table 15.3.

The observed rainfall data is then entered into the SWMM model, and the results are compared to observed flow information and any high water marks that may have been recorded at the time of the flood event. The SWMM model was run for all four of the observed events and the results of that

modeling as compared with the observed data is in Table 15.3. It should be noted that the only means of comparison between the model and monitored data is a total daily volume of flow as the meter does not provide an actual flow hydrograph of the observed values.

Table 15.3 Summary of calibration and validation events.

Date	Observed flow (mgd) [m ³ /d]	Modeled flow (mgd) [m ³ /d]	Difference from observed (%)	Type of event
July 17, 2001	3.180 (12,037)	10.506 (39,770)	230	Validation
July 18, 2001	11.545 (43,703)	14.156 (53,586)	23	Validation
July 19, 2001	4.312 (16,323)	0.516 (1,953)	-88	Validation
June 2, 2009	4.694 (17,769)	15.061 (59,056)	221	Validation
June 3, 2009	4.870 (18,435)	3.556 (13,461)	-27	Validation
June 26, 2009	6.046 (22,887)	5.070 (19,192)	-16	Calibration
August 4, 2009	7.392 (27,982)	8.737 (33,073)	18	Calibration

With the type of modeling being performed for this study and the availability of data to confirm the model results, model results that are within 20% of the observed conditions can be expected. The June 26, 2009 and August 4, 2009 rainfall events are within 20% of the observed flow while the July 17–19, 2001 event and the June 2–3, 2009 events are not.

The data for the July 2001 event was examined to determine if there might be a reporting or modeling error for that event. Several things stand out as contributing to the differences between the modeled and observed events. The rainfall began at approximately 23:00 on July 17 and extended through approximately 01:00 on July 18 at the Sharonville gauge. The three-day total for the model (25.178 MG, 95 309 m³) vs the observed event (19.037 MG, 72 063 m³) indicates that the timing of the runoff accounts for some of the difference between the daily results. The rainfall total at the Sharonville gauge (3.425 in., 8.70 cm) is approximately 25% higher than the observed rainfall at the site gauge (2.56 in., 6.50 cm), which leads directly to a higher flow volume from the model results. The log of rainfall from the site indicates that debris was present at outfall 001, which likely prevented the meter from recording an accurate flow measurement. In addition to the debris at the meter itself, it was reported that debris was present in the North ditch, which contributed to a breach in the earthen embankment, and then flow was directed out of the North ditch into the northern end of the facility and would have lowered the observed flow volume at the meter. The model results are based on a clean channel condition and do not represent the blockage that was present on the day of the flooding. Based on this analysis, it is believed that the observed flow data for the July 2001 event does not accurately represent the full flow that discharged from the site.

Rainfall records for the June 2–3 event indicate that approximately 2.6 in. (6.60 cm) rain fell on the site in 5 h. This information was used to develop an approximation of the amount of runoff from the 253 acre (102.4 ha) tributary area, which would be necessary to generate 4.694 MG (17 768.7 m³) flow for the day (4.694 MG = 627 540 ft³ = 14.4 acre-ft runoff = 0.68 in. runoff).

By comparison, the June 26 event of 0.94 in. (2.39cm) rainfall resulted in 0.88 in. (2.24 cm) runoff (93.6% runoff), and the August 4 event of 1.584 in. (4.02cm) resulted in 1.08 in. (2.74cm) runoff (68% runoff). If an average of the percentage runoff from the August 4 and June 26 events was used (81% runoff) on the June 2 event rainfall, runoff from that storm would be 2.11 in. (5.36 cm). Using this value on the 253 acre (102.4 ha) watershed results in 14.502 MG (54 896 m³) runoff for the day. This is about 10% higher than the modeled flows for the June 2 event, and suggests that the observed flow data for the June 2 event does not accurately represent the full flow that discharged from the site.

The model represents the actual system response within standard modeling tolerances and can be used to determine the existing storm sewer system capacity within the site with a reasonable level of confidence.

15.2.3 Existing System Capacity Analysis

The storm water conveyance system was analyzed for seven design storms. The results for each of the design storms were compared to existing top of casting elevations within the site to determine whether or not the hydraulic grade line exceeded the top of casting elevation, which would indicate that flooding was occurring. A detailed description of each of the primary drainage systems within the site follows.

North Ditch

The drainage area that is tributary to the North ditch is almost entirely outside the site (see Figure 15.1 above). The watershed starts north of Glendale-Milford road, crossing I-75 three times before discharging into the site just south of Glendale-Milford Road with a total drainage area of 572.8 acres (231.8 ha). The only portion of the site that is tributary to the North ditch consists of the large undeveloped area in the northwest corner of the site and the area immediately east of the undeveloped area (a total of 50.4 acres, 20.4 ha). The construction of the North ditch was recommended as part of a 1961 flood study to alleviate flooding of the site at that time. The North ditch passes under two abandoned railroad spurs before crossing under an access road (to a currently undeveloped portion of the site) just before it enters into

the Split ditch, which is described in more detail below. The North ditch is largely an earthen channel with woody overgrowth along its banks for most of the length between I-75 and Split ditch. Portions of the North ditch have been concrete lined to provide for stability at bends in its course or in areas where erosion has been problematic.

Model results indicate that the North ditch has the capacity to convey the 2 y event without exceeding the top of bank elevation of the earthen embankment. It is possible for flow to leave the ditch during the 5 y, 10 y and 25 y events, north of the two abandoned railroad spurs at a low spot in the earthen embankment. This area is where flood waters appear to have entered the site during the 2001 flooding event. In addition to the low spot in the earthen embankment, the access road to East Acres is low enough that it could allow flood waters to enter the site during the 5 y event.

West Ditch

The drainage area tributary to the West ditch is much smaller today than it was in the past. Prior to the construction of the North ditch, the area that now discharges into the North ditch was previously conveyed through the West ditch. The 63.4 acre (25.7 ha) drainage area discharges into a primarily concrete lined trapezoidal ditch with a portion of it being a grass lined trapezoidal section. The downstream end of the West ditch is routed through a single 42 in. (106.7 cm) concrete pipe (there are two other 42 in., 106.7 cm, pipes at this location that have been filled in) before entering the channel that becomes the Split ditch.

Although the drainage area to the West ditch has decreased, it has the capacity to convey the 2 y event without causing flooding in adjacent areas. The flooding that may be caused by events larger than the 2 y event results from a lack of capacity in the existing 42 in. (106.7 cm) concrete pipe at the downstream end of the West ditch as well as other factors described below.

There is apparently a control weir at the downstream end of the West ditch which is actually two weirs back to back. The downstream weir has a 6 in. (15.2 cm) valve at the invert of the channel. The upstream weir is higher than the downstream weir, but it is located above the invert of the channel to allow low flow conditions to pass under the upstream weir and then initially through the 6 in. (15.2 cm) valve and then over the top of the downstream weir. The weir arrangement is not in the model and will likely contribute to backups experienced in the West ditch during large storm events.

ODOT Drainage through 60 in. (152.4 cm) Pipe

As part of the construction of I-75, a 60 in. (152.4 cm) storm sewer was constructed through the site, passing under one of the security entry gates, and

then on to the East ditch as shown on Figure 15.5. This storm sewer was constructed parallel to an existing 48 in. (121.9 cm) storm sewer that also carried flow from the highway through the site and into the East ditch. Portions of those systems were altered in 1995 to maintain a separation between the onsite and offsite runoff. The three primary changes that occurred as part of that project are the construction of the Split ditch (described in more detail below), the construction of an inverted siphon under the Split ditch for the 60 in. (152.4 cm) pipe, and the separation of the onsite and offsite flow into parallel pipes.

To maintain separation of the onsite and offsite flows, a large junction chamber was built that connected the 48 in. (121.9 cm) and 60 in. (152.4 cm) storm sewers just west of the entry gate. Both storm sewers discharge into the junction chamber, but a solid steel plate is currently installed over the outlet of the 48 in. (121.9 cm) pipe from the chamber. This plate forces all of the runoff from the drainage area into the 60 in. (152.4 cm) storm sewer that conveys flow to the east. The 48 in. (121.9 cm) pipe is then used to convey only onsite discharges to the Split ditch and is described in more detail below.

The final element from the 1995 project that maintains the separation of flows was the construction of an inverted siphon under the Split ditch for conveyance of offsite flows. Originally, the 48 in. (121.9 cm) and 60 in. (152.4 cm) pipes both terminated just west of the West ditch and mixed their flows in the open channel that existed there. As part of the 1995 project, the 60 in. (152.4 cm) pipe was extended to the east. However, it carries offsite flow and therefore needed to pass under the 48 in. (121.9 cm) pipe, which carries onsite flow for it to rejoin the offsite area flows from the North ditch. The inverted siphon achieves this goal and ensures that the 93.97 acre (38.0 ha) offsite watershed remains separate from onsite flows.

The model results show the existing 48 in. (121.9 cm) and 60 in. (152.4 cm) storm sewers have the ability to convey the 2 y storm event without flooding. The 5 y and 10 y events are shown to have the potential for flooding at only one location, which is located approximately 200 ft (61 m) east of the entry gate. Events larger than the 10 y event have the potential for flooding at the manhole directly in front of the guard house and within the ODOT right-of-way.

Onsite Drainage Through 48 in. (121.9 cm) Pipe

The 48 in. (121.9 cm) storm sewer used to convey a mixture of offsite (I-75) flows and onsite flows now conveys only onsite flow to the Split ditch. The 17.07 acre (6.91 ha) drainage area consists primarily of two buildings as shown in Figure 15.5 above. This storm sewer discharges into the onsite por-

tion of the Split ditch just west of the West ditch and has the capacity to convey between the 10 y and the 25 y storm with no flooding. Events larger than the 25 y event will potentially flood at the two structures located approximately 260 ft (79 m) and 380 ft (116 m) east of the entry gate.

Split Ditch

The creation of the Split ditch was part of the 1995 project to separate onsite and offsite flows around and through the site. The Split ditch begins at the point where the North ditch passes under the access road to East Acres and extends for approximately 800 ft (243.8 m) to the east and south before the offsite flows are routed through five 60 in. (152.4 cm) circular culverts under a railroad embankment and into their own open channel across the East Acres, and eventually into Mill Creek. The ditch is designed as a trapezoidal concrete channel with a 4 ft (1.22 m) wall in the channel to separate onsite and offsite flows. The onsite portion of the channel has a 2 ft (0.61 m) bottom width, while the offsite portion of the channel has a 6 ft (1.83 m) bottom width. The plan analysis of this wall indicated that a storm event between a 2 y and a 3 y event will cause the offsite flows to overflow the wall and mix with the onsite flows.

The SWMM analysis of the Split ditch, however, indicates that the 1 y event will generate flows from the offsite side of the channel that are above the height of the wall, which indicates that any event larger than that will cause offsite flows to mix with onsite flows. This result is not unexpected as a different rainfall value was used for the SWMM analysis (Bulletin 71) than for the construction project (TP 40), as well as the drainage areas being slightly different. The August 3, 2009 rainfall event was smaller than a 1 y event and was observed to have an offsite water surface elevation that was less than 1 ft from overflowing the central wall. The overall ditch section can convey events up to and including the 100 y event, with onsite and offsite flows mixing over the wall.

In addition to all of the areas described above, there is an additional 203.67-acre (82.4 ha) offsite area that discharges into the Split ditch as shown on Figure 15.5 above. This area consists of locations north of Glendale-Milford road that are conveyed under the roadway and into open-ditch sections alongside the railroad right-of-way and eventually into the Split ditch.

Pump House 506

The 41.88-acre (16.9 ha) drainage area that is tributary to pump house 506 (PH 506) consists primarily of the two buildings and their associated parking areas, as shown on Figure 15.5 above. Flow from the storm sewer system

that is tributary to PH 506 has no gravity outlet, and must travel through the pumps before being discharged into the East ditch. As long as the pumps are functioning as designed, the system has the capacity to convey the 5 y storm event without causing surface flooding. Results from the 10 y event analysis show that flow has the possibility of leaving the system, so it is possible that portions of the storm sewer system that are not included in the modeling could also experience flooding, which raises the possibility of flooding other portions of the buildings.

Pump House 423

The 62.6-acre (25.3 ha) drainage area that is tributary to pump house 423 (PH 423) consists primarily of the largest building within the facility as shown on Figure 15.5 above. Flow from the storm sewer system that is tributary to this pump house discharges into the East ditch via gravity unless the flap gates that exist on the east side of the wet well are closed. If the flap gates are closed, the pumps are used to lift the flow over the flap gates and then discharge via gravity into the East ditch. The storm sewer system tributary to PH 423 also contains two 48 in. (121.9 cm) inverted siphons that exist to convey rooftop runoff from the building under the basement. The two inverted siphons are parallel to one another, one along the north side of the building and one along the south side of the building. The inverted siphons are approximately 120 ft (36.6 m) long and are near the midpoint of the building. The gravity system has the capacity to convey between a 1 y and 2 y storm event without causing surface flooding. It should be noted that all levels of modeling greater than the 2 y show the possibility of a surcharged storm sewer system west of the two inverted siphons, but still within the footprint of the building. This condition could lead to flow escaping the storm sewer system through floor drains, storm sewer access points, or even through the connections of the roof drains to the storm sewer system, which could create flooding conditions within the building.

Pump House 422

The 7.79 acre (3.15 ha) drainage area that is tributary to pump house 422 (PH 422) consists of portions of the rooftop drainage for one building, the roadway adjacent to the building and the parking and storage area just east of the building, as shown on Figure 15.5 above. The storm sewer system that is tributary to PH 422 has no gravity outlet and the flows must pass through the pumps before being discharged into the East ditch. Based on the model results, and considering that only one of the pumps was functional due to required pump maintenance, the storm sewer system tributary to the pump house has the capacity to convey the 1 y storm event without causing surface

flooding. Assuming all pumps are functional, the storm sewer system can convey between a 10 y and 25 y event without causing surface flooding.

While the results show that no surface flooding is caused for storms less than the 10 y event, it is possible and even probable, that flooding will begin to occur in the basement of the building (because of the shallow cover of the storm sewer within the building itself) when the hydraulic grade line rises above the top of the storm sewer pipe. The storm sewer outside of the building (the portion in the model) has much more cover on it, which allows for a greater level of surcharging before surface flooding occurs.

Outfall 001

Outfall 001 is the point of the system where the flows from all the onsite areas described above combine before they exit the site under the railroad embankment, continue to a constructed channel and then go to Mill Creek. In addition to all of the onsite areas described above, there is also an additional 37.61-acre (15.2 ha) watershed that consists primarily of another building and the areas immediately around it, as shown on Figure 15.5 above. As flow is conveyed through the storm sewer that collects rooftop runoff from the building, it passes through a long 54 in. (137.2 cm) diameter inverted siphon to the east of the building. It appears that this inverted siphon was constructed to avoid conflict with the many other utility lines in the area. The storm sewer has capacity to convey between the 1 y and 2 y storm event without causing surface flooding.

Outfall 002

Outfall 002 is not within the site. The outfall consists of a junction chamber that has multiple storm sewers discharging into a concrete lined open topped chamber that directs flow under the railroad embankment and thence to Mill Creek. Three of the pipes that discharge into the chamber carry flow from the site and the fourth carries flow from Lincoln Heights and Shepherd Lane. The flow passes under the railroad embankment and enters a 72 in. (182.9 cm) culvert and is briefly daylighted in a short open section before re-entering into four storm sewers. These are a 36 in. (91.44 cm) sewer, two 42 in. (106.7 cm) sewers and a 48 in. (121.9 cm) sewer that convey flow through Reading and into Mill Creek. Each of these storm sewers has a flap gate installed on them at Mill Creek to prevent Mill Creek flows from entering the storm sewer.

Outfall 002 West Area

The West Area refers to the 32.95 acre (13.3 ha) watershed that is made up of nearly all of the parking lot and internal roadway space in front of two

buildings and is shown on Figure 15.5 above. Flow from this storm sewer is conveyed south along the western face of the two buildings, through a 36 in. (91.4 cm) siphon near the northwest corner of one of the buildings to allow the storm sewer to go over two 24 in. (61 cm) water lines, and then south parallel to Shepherd Lane before discharging into Outfall 002. During the surveying of this storm sewer, several structures were noted to have sediment deposits that may be adding to flooding experienced in this area.

This storm sewer also enters into an open-topped junction chamber that is part of the floodwall around the south end of the site. The chamber has flap gates on the West Area storm sewer as well as the storm sewer conveying flow from Lincoln Heights across the South Parking (described below). Flow exits this junction chamber through two storm sewers that are the same size as those entering, a 42 in. (106.7 cm) sewer and a 60 in. (152.4 cm) sewer, and discharges into outfall 002. The flap gates and floodwall are intended to protect the site from a potential flood along Mill Creek. There is an additional flap gate at the discharge end of this storm sewer at outfall 002.

The storm sewer in this area has the ability to convey less than the 1 y storm event without causing surface flooding. The first location that will experience flooding is adjacent to an entry gate into the facility. The next location that is likely to experience flooding is at the location of the siphon over the 24 in. (61 cm) water lines. All of the other locations likely to experience flooding along this storm sewer are located in front of the two buildings.

The watershed area tributary to this storm sewer is largely parking lot that is generally sloped directly towards the front entrances of the two buildings (both of which have experienced flooding in the past). This indicates that much of the runoff will be shallow sheet flow, and particular attention needs to be given to the way in which this runoff enters the storm sewer system. The model that is developed assumes that the runoff enters the system, with no direct calculations as to how that occurs. In other words, flooding may occur on the surface for events smaller than the capacity of the storm sewer simply because the runoff cannot enter into the storm sewer due to a lack of inlet capacity.

Outfall 002 East Area

This 17.8 acre (7.2 ha) watershed consists of one building and a portion of the area that was occupied by another building before its demolition, and is shown on Figure 15.5 above. The storm sewer that conveys runoff from this area to outfall 002 also has to pass through the south floodwall, and has a flap gate within a junction chamber at the floodwall. In addition to the flap gate at the floodwall, there is a second flap gate at the discharge end this

storm sewer at outfall 002. The storm sewer has the capacity to convey the 2 y storm event without causing surface flooding. The areas that have the potential for flooding exist at the upstream extent of the modeled storm sewer.

Outfall 002 Lincoln Heights and South Parking

This watershed comprises 98.1 acres (39.7 ha) of Lincoln Heights and I-75, and 17.7 acres (7.16 ha) of onsite drainage that is the south parking lots and the fuel farm at the south end of the site, as shown on Figure 15.5 above. This system conveys flow through the same junction chamber within the south floodwall as outfall 002 west area before discharging to outfall 002. The system has the capacity to convey a 5 y storm event without causing surface flooding. In larger storm events, the likely location for flooding is a sewer structure within the facility, from where overland flow will be routed east, and into the west area storm sewer. If this is not possible, flow will continue overland to the southeast and eventually discharge at outfall 002.

Runoff from Lincoln Heights reaches the site through a large storm sewer that passes under I-75. The storm sewer appears to have been in place during the construction of I-75, as it is shown as an existing 4 ft x 6 ft (1.2 m x 1.8 m) storm sewer on the plans for I-75 from 1963. Access to the portion of the storm sewer under I-75, and even within portions of the site, does not appear to be possible based on information gathered from both surveying efforts.

Outfall 002 Shepherd Lane

Although the area that is tributary to the storm sewer located within the Shepherd Lane right-of-way is not part of the site, and does not enter the site, it has can indirectly impact the overall capacity of the site drainage system. This area, which conveys flow from Shepherd Lane, portions of I-75 and portions of Lincoln Heights, for a total drainage area of 118.08 acres (35.99 ha) discharges into outfall 002 through a 60 in. (152.4 cm) diameter concrete pipe. Runoff from this area may either back up the site storm sewers, or force the flap gates that exist at this location to remain closed, which can thus affect the flow conditions within the site. The system has the capacity to convey between the 2 y and 5 y storm event without causing surface flooding. When runoff exceeds the capacity of this system, it will mostly flow overland along Shepherd Lane and eventually discharge into Outlet 002. None of the overland flood routing will have a direct impact on the site.

15.3 Conclusions

Based on the results of all of the modeling performed to determine the overall level of service for the site, it is apparent that the level of service

provided by the storm sewer varies throughout the site. Table 15.4 is a summary of the level of service for each of the individual areas identified previously.

Table 15.4 Summary of the overall system level of service.

Area	Level of Service
North ditch	<5 y
West ditch	<5 y
ODOT drainage through 60 in. (1.52 m) pipe	<5 y
Onsite drainage through 48 in. (1.22 m) pipe	>25 y and <50 y
Split ditch	100 y
PH 506	10 y to 25 y (all pumps functional)
PH 423	2 y (with no pumps running)
PH 422	10 y to 25 y (all pumps functional)
Outfall 001	1 y to 2 y
Outfall 002 West area	<1 y
Outfall 002 East area	2 y to 5 y
Outfall 002 Lincoln Heights and South Parking	5 y to 10 y
Outfall 002 Shepherd Lane	2 y to 5 y

It should be noted that the level of service currently identified is based on the design storm at which the hydraulic grade line within the storm sewer will exceed the top of casting elevation. It is possible to set a different threshold for the definition of level of service within certain areas of the facility. For example, in the area tributary to either Pump House 422 or 506, the basement floor elevation may be a more appropriate level of service criteria to prevent flooding of the building basements. If this criterion is chosen, the currently provided level of service will decrease. Multiple possibilities exist to increase the level of service provided by the storm sewer system within the facility. Development of alternatives for improvement have not yet been completed, but will likely include a combination of reduction in impervious surfaces, increases in storm sewer pipe size and increased pump capacity.

15.4 Recommendations and Next Steps

15.4.1 Recommendations

The storm water system within the site currently provides varying levels of service. Improvements to the storm water system will vary depending on the location within the site.

To more fully understand the nature of any alternatives for improvement that may be implemented to mitigate flooding concerns within the site, more information is necessary within the various locations identified previously.

15.4.2 Next Steps

Phase I of the storm water program focused on identifying the flooding problems at the site and creating a modeling tool that will allow for detailed alternative evaluations and what-if scenarios. The following steps identified here are prioritized to identify how Phase II of the storm water evaluation may be appropriate to address some of the flooding problems.

1. Complete a scope for additional storm water work including:
 - a. define a desired level of service for what frequency storm event the storm water system should be designed to convey.
 - b. develop a risk-based process that focuses prioritized locations at the site to be protected.
 - c. develop alternative analysis options for infrastructure improvements that reduce the volume of storm water runoff or increase storm water conveyance capacity to meet storm water system desired level of service.
 - d. develop order of magnitude costs for improvement recommendations.
 - e. align improvement recommendations with other proposed improvements to integrate overall site improvements cost effectively.
2. Perform prioritized pump house flow tests to support improvements.

References

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