PART III, ATTACHMENT 2

APPENDIX III-2A

DETAILED DRAINAGE CALCULATIONS

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For Sheets 1-8; Tables III-2A-1.1 to III-2A-1.7, III-2A-2.1 to III-2A-2.7, III-2A-3, III-2A-4, III-2A-5, and III-2A-6.

1.0 OBJECTIVE

Develop a surface water management plan for the proposed development at the Hawthorn Park RDF located in Harris County, Texas.

Compare pre- and post-development peak flows to analyze impact of proposed development and determine whether existing drainage patterns will be adversely altered.

Design surface water control structures to contain and convey stormwater in accordance with applicable regulations.

2.0 METHOD

2.1 Pre-Development and Post-Development Calculations

The proposed Hawthorn Park Recycling and Disposal Facility (RDF) project site is greater than 200 acres. Therefore, Golder utilized the USACE Hydrologic Engineering Center software, HEC-HMS, to model pre-development drainage conditions and post-development drainage conditions. Overall pre-development drainage subbasins were delineated using the permitted cover grades, permitted drainage design, and existing topography. Overall post-development drainage conditions subbasins were delineated using proposed final cover grades and proposed drainage design. See Figures III-2A-1 and III-2A-2, respectively, for the pre-development and post-development drainage conditions.

The methodology outlined in the National Resources Conservation Service (NRCS) Technical Release 55 (TR-55) was used to approximate travel times for flow path segments and calculate the total time of concentration (Tc) for each subbasin in both pre- and post-development. TR-55 provides equations and calculations for sheet flow, shallow concentrated flow, travel time as a function of velocity, and time of concentration. Reach lag time was also calculated using NRCS TR-55 method for travel time of each reach lag segment flow path. The National Engineering Handbook (NRCS)'s wave equation was used to estimate wave velocity as the travel velocity through water such as the detention pond.

Runoff coefficient composite C values were calculated using a weighted average. Runoff coefficient C values were taken from the City of Houston Infrastructure Design Manual (IDM) Stormwater Design Requirements section and the Harris County Flood Control District (HCFCD)'s Policy, Criteria, and Procedure Manual (PCPM).

Rainfall intensity, I (in/hr), was interpolated from Atlas-14 data local to the project site. The Atlas 14 precipitation data was pulled from the National Oceanic and Atmospheric Administration (NOAA)'s Precipitation Frequency Data Servers (PFDS). The rainfall intensity is dependent on the time of concentration for the subbasin which was used as the duration of the storm.

Composite SCS curve numbers (CN) were estimated for each subbasin using tables in the TR-55 (NRCS) document. Impervious cover percentages were estimated using a combination of aerial imagery of the existing site, knowledge of permitted land usage, and the proposed permit amendment design.

The Rational Method for calculating peak runoff flows was used for delineated subbasins. These flows were matched in the HEC-HMS model in accordance with the HCFCD PCPM Section 3.7 for Clark's Unit Hydrograph method for the 25-year and 100-year storm. An HEC-HMS model was created for the 25-year Pre-Development conditions, 100-year Pre-Development conditions, 25-year Post-Development conditions, and 100-year Post-Development conditions. The HEC-HMS input and output for each model are provided in Appendix III-2A-1.

2.2 Drainage Control Structures

The Rational Method was used to calculate the peak flow contributing to the add-on berms and downchutes, as is permitted for on-site drainage structures according to the TCEQ regulatory guidance for surface water drainage for individual drainage areas less than 200 acres. See Figure III-2A-3 for the delineated drainage areas and analysis points for add-on berms and downchutes. Manning's equation was used to determine the capacity of the add-on berm and downchutes using the modeling software HydraFlow Express. The HydraFlow normal depth calculations are provided in Appendix III-2A-2.

HEC-RAS was used to model the perimeter ditch system. Input for the peak flow was determined by the Rational Method, permitted by TCEQ Regulatory Guidance as individual contributing drainage areas are less than 200 acres in size. See Figure III-2A-4 for the perimeter ditch plan. The perimeter ditches were modeled in AutoCAD Civil3D as three-dimensional surface models, and the surface models were exported into a HEC-RAS input file. HEC-RAS takes the geometric information from the exported file as geometric input for the channel design. Additional inputs, such as the channel Manning's "n" roughness coefficient, culvert design, and steady state flow values were used to perform steady state analysis on the perimeter ditches. The HEC-RAS schematic, input, and output are provided in Appendix III-2A-3.

TCEQ requires evaluation of the pre- and post-development drainage conditions for a 25-year storm. For the Hawthorn Park RDF, the 25-year and 100-year storm events were modeled in HEC-HMS and HEC-RAS. The perimeter ditches were designed to provide 0.50-ft of freeboard for a 25-year design storm and convey the 100-year frequency storm. The other stormwater drainage structures were designed to provide 0.50-ft of freeboard for a 25-year design storm.

3.0 ASSUMPTIONS

In September 2018, Atlas 14 Intensity-Duration-Frequency (IDF) data were released by the National Oceanic and Atmospheric Administration (NOAA). The Precipitation Frequency Data Server (PFDS) data selected were for the project site coordinates. For rainfall intensities with durations in-between tabulated values, the intensities were calculated using linear interpolation, as recommended by the Texas Department of Transportation (TxDOT)'s Hydraulic Design Manual. These interpolated intensities were used to calculate peak flow rate in Rational Method calculations.

Atlas 14, 24-hour rainfall depths and intensities:

- 2-year = 5.00 in (used in time of concentration calculations as P_2)
- 25-year = 11.30 in (used in meteorological input in HEC-HMS model)
- 100-year = 16.70 in (used in meteorological input in HEC-HMS model)

Duration	25-Year I (in/hr)	100-Year I (in/hr)
10-min	9.47	11.80
15-min	7.90	9.80
30-min	5.56	6.85
60-min	3.77	4.70

Minimum 10-minute duration storm was used, in accordance with recommendations from the Texas Department of Transportation (TxDOT) Hydraulic Manual Design for Rational Method calculations.

Runoff Coefficient, C (consistent with previous permit and local regulations/practice) used in Rational Method calculations, Q = C I A:

- Unimproved areas, C = 0.20
- Top slope of final cover, C = 0.30
- Side slope of final cover, C = 0.70
- Industrial, C = 0.60
- Ditch, C = 0.60
- Pave/ROW, C = 0.90
- Pond, C = 0.95

Curve Number (consistent with previous permit and local regulations/practice), used in HEC-HMS input:

- Landfill final cover and other open areas, CN = 85

- Areas where minimum infiltration are expected (ponds), CN = 98
- Developed areas (paved), CN = 98
- Developed areas (unpaved), CN = 92

Manning's roughness coefficient used in HEC-RAS input and Manning's Equation calculations:

- Grass-lined surface, n = 0.035 (for add-on berms)
- Per the HCFCD PCPM design manual, n = 0.040 for grass-lined surfaces in HEC-RAS modeling for conservative reach capacity (for perimeter ditches)
- Concrete-lined surface, n = 0.015
- Geomembrane-lined (plastic) surface, n = 0.012
- Reinforced concrete box (RCB) culvert, n = 0.013

Landfill downchutes are to be constructed with geomembrane lining. Typical downchutes are trapezoidal channels with 2:1 side slopes, 5-ft bottom width, 2-ft depth, and 25% longitudinal slope. Downchutes were designed sized to convey runoff from the 25-year storm event with a minimum 0.50-ft of freeboard.

Typical add-on berms are triangular channels with 4:1 and 2:1 (H:V) side slopes, 2% longitudinal slope on top of the final cover slope, and 2-ft depth. Add-on berms were designed to convey runoff from the 25-year storm event with a minimum 0.50-ft of freeboard.

Perimeter ditch channels are trapezoidal channels with varying H:V side slopes (3:1 or 2:1), varying longitudinal slopes, varying bottom channel widths, and varying channel depths. The perimeter ditches were designed to convey run-off from the 25-year storm event with a minimum 0.50-ft of freeboard and contain the 100-year storm with no freeboard. Grass-lined channels are to be armored with riprap where flow velocities exceed 5 fps, at downchute crossings, after transitions from concrete-lined channels, before/after culverts, and before the outfall.

4.0 CALCULATIONS

4.1 Pre-Development and Post-Development Conditions

Tables III-2A-1.1 and III-2A-2.1 contain the time of concentration calculations for the pre- and post-development conditions, respectively. The time of concentration calculations consider the longest flow path to get to the analysis point (Figure III-2A-1 and Figure III-2A-2) for each drainage area. The applicable Tc was used to interpolate the rainfall intensity value I_{25} (in/hr) and I_{100} (in/hr) in Tables III-2A-1.5 and III-2A-2.5 for the pre- and post-development Rational Method calculations.

Tables III-2A-1.2 and III-2A-2.2 contain the Reach Time calculations for the pre- and post-development conditions, respectively. Tables III-2A-1.3, III-2A-1.4, III-2A-2.3, and III-2A-2.4 present Runoff Coefficient C calculations and Curve Number calculations for pre- and post-development conditions.

The Storage Coefficient, R, was determined for each HEC-HMS model and basin using an iterative approach by running the HEC-HMS model until the Rational Method peak flowrate and HEC-HMS peak flowrate were within 1% accuracy. Table III-2A-1.6 is the summary of the HEC-HMS input values for pre-development conditions; Table III-2A-1.7 is the HEC-HMS output summary for pre-development drainage conditions. Table III-2A-2.6 is the summary of the HEC-HMS input values for post-development conditions; Table III-2A-2.7 is the HEC-HMS output summary for post-development drainage conditions.

Meteorological input for HEC-HMS used precipitation data from the National Oceanic and Atmospheric Administration (NOAA)'s precipitation frequency data servers for Atlas-14 local to the project coordinates. The input selected was a hypothetical SCS Type 3 storm for each frequency storm for a 24-hour duration.

Velocities at the control points were obtained through various computation methods. For the outfall CP-1, the velocity was estimated dividing the peak discharge flowrate by the area of the outfall structure (10' x 3' reinforced concrete box). Velocities for the post-development ditches, CP-10 through CP-12, were taken from the HEC-RAS results at the most downstream reach station. Velocities for the pre-development perimeter ditch discharge points, CP-10 and CP-11, were calculated using HydraFlow Express for Manning's Equation and the existing drainage ditch geometry and slope.

4.2 Drainage Control Structures

The delineated drainage areas contributing to individual downchutes and add-on berms are shown on Figure III-2A-3. The HydraFlow Express modeling software was used to perform Manning's Equation calculations for the downchutes and add-on berms to solve for velocity, normal depth, etc.

The downchutes are to be constructed as trapezoidal channels. The Rational Method was used to calculate the peak discharge for each downchute, and the results are included in Table III-2A-4. The intensity value I (in/hr) used in the Rational Method calculation is dependent on the time of concentration to the bottom of the downchute, which takes travel times for the sheet flow, shallow concentrated flow, add-on berm flow, and downchute flow paths from Table III-2A-2.1. The runoff coefficient, C, was determined for each downchute drainage area using a weighted average with C = 0.30 for top slope areas and C = 0.70 for side slope areas. The resulting Q, peak discharge (cfs) was input into HydraFlow Express along with the downchute geometric design for outputs of velocity and flow depth. The downchute HydraFlow Express outputs are included in Appendix III-2A-2.

The add-on berms on the slope of the proposed final cover are to be constructed as triangular channels. The Rational Method was used to calculate the peak discharge for each drainage area, and the results are included in Table III-2A-5. The add-on berm drainage areas, in acres, are from Figure III-2A-3. The runoff coefficient C used in calculations was 0.70 for add-on berms with contributing areas consisting of only side slope areas (designated as areas S1-S44). The runoff coefficient was otherwise weighted for using C = 0.30 for top slope areas and C = 0.70 for side slope areas. Since all the add-on berms on the final cover side slopes will be identical in geometry, only the add-on berm with the largest peak discharge was analyzed using Manning's equation in HydraFlow Express. As shown in Table III-2A-5, the delineated add-on berm drainage area with the largest peak discharge was determined to be T14. The HydraFlow simulation for add-on berm area T14 is included in Appendix III-2A-2.

The perimeter ditches, and in-line culverts, were modeled in HEC-RAS. The geometric input for the perimeter ditches were exported from the Civil3D surface data terrain model and manual tabular inputs for the Manning's roughness coefficient, n. The perimeter ditch geometry was designed, within the restrictions of the permit boundary and existing infrastructure, so that there would be a minimum 0.50-ft of freeboard from the resulting 25-year water surface elevation to the top of the perimeter ditch and that the 100-year water surface elevation would be contained within the perimeter ditch at all cross-sections. The culvert design, including material selection, number of barrels, and slope of the culvert, was done so that water surface elevations of the perimeter ditches upstream and downstream of the culvert provide adequate freeboard or containment. The steady state flow input and peak flow change locations are shown in Figure III-2A-4. The perimeter ditch calculation points were taken as a percentage of the full basin calculation points determined in the post-development HEC-HMS model and Figure III-2A-2 for Post-Development Drainage Conditions. The HEC-RAS geometric schematic, inputs, and output – which include tabulated results, perimeter ditch flow profiles, and culvert cross-sections – can be found in Appendix III-2A-3.

Riprap sizing of the ditch lining varied depending on the velocity and flow rate of storm water conveyance. Calculations for riprap sizing is included in Table III-2A-6.

5.0 CONCLUSIONS

The results of the analyses described herein support the design of the proposed surface water management features to be adequate for the 25-year and 100-year storm events. The table below summarizes the peak flow rates from HEC-HMS for pre-development and post-development drainage conditions at the control points determined in the overall drainage conditions figures.

Peak Discharge Summary Table

Control Point	Pre- Development (cfs)		Post-Development (cfs)	
	25-year	100-year	25-year	100-year
CP-1 Outfall	151.2	174.9	141.8	169.8
CP-2 Offsite	30.7	38.2	0.0	0.0
CP-3 Onsite	1.8	2.3	1.8	2.3
CP-4 Onsite	1.1	1.4	1.1	1.4
CP-5 Onsite	1.8	2.3	1.8	2.3
CP-6 Onsite	6.3	7.8	6.3	7.8
CP-7 Onsite	8.6	10.7	8.6	10.7
CP-8 Onsite	6.8	8.5	6.8	8.5
CP-9 Onsite	4.5	5.7	4.5	5.7
CP-10 North Ditch	402.6	514.1	262.5	333.0
CP-11 East Ditch	60.1	74.3	126.2	156.1
CP-12 South Ditch	0.0	0.0	282.1	357.3

Run-off Velocity Summary Table

Control Point	Pre- Development (fps)		Post-Development (fps)	
	25-year	100-year	25-year	100-year
CP-1 Outfall	5.0	5.8	4.7	5.7
CP-2 Offsite	*Sheet/Shallow Concentrated Flow*		-	-
CP-10 North Ditch	2.0	2.1	4.4	4.7
CP-11 East Ditch	2.7	2.9	4.7	4.9
CP-12 South Ditch	-	-	3.5	3.8

There is an overall increase to the peak discharge at CP-10, CP-11, and CP-12 from pre-development to post-development drainage conditions. These control points are associated with the discharge locations for the North, East, and South perimeter ditches, respectively, into a detention facility. The detention facility is privately owned and maintained by the permittee and is separately permitted with the Harris County Flood Control District (HCFCD). The detention pond releases at an outfall designated CP-1, and there is no increase in peak discharge from the pre-development to post-development drainage conditions at CP-1. Therefore, downstream of the CP-1 outfall structure, adjacent properties and structures will not be affected by the development of the Hawthorn Park RDF expansion. The summary tables show no adverse impact due to the proposed development of the Hawthorn Park RDF expansion.

6.0 REFERENCES

1) Surface Water Drainage and Erosional Stability Guidelines for a Municipal Solid Waste Landfill, TCEQ Regulatory Guidance, TCEQ. May 2018

2) Policy, Criteria, & Procedure Manual (PCPM), Harris County Flood Control District (HCFCD). October 2018

3) Urban Hydrology for Small Watersheds TR-55, Natural Resources Conservation Service (NRCS), United States Department of Agriculture. June 1986

4) Precipitation Frequency Data Server (PFDS), National Oceanic and Atmospheric Administration (NOAA). September 2018

5) Hydraulic Design Manual, Texas Department of Transportation (TxDOT). September 2019

6) Infrastructure Design Manual (IDM), City of Houston Public Works. July 2019

7) National Engineering Handbook, Part 630 Hydrology, National Resources Conservation Service (NRCS), United States Department of Agriculture. May 2019

7.0 ATTACHMENTS

Atlas 14 Point Precipitation Frequency Estimates (Reference 4)

Hydraulic Design Manual (Reference 5)

Infrastructure Design Manual (Reference 6)

National Engineering Handbook (Reference 7)

Policy, Criteria, and Procedure Manual (Reference 2)

Urban Hydrology for Small Watersheds TR-55 (Reference 3)