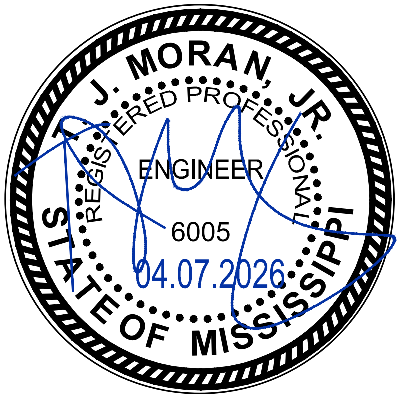

DRAINAGE ANALYSIS

Project: Webster Street Development (Bay St. Louis, MS)
Prepared for: Owner/Developer
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Date: 04.07.2026

1. Purpose and Design Intent

This drainage analysis documents the proposed stormwater approach for the Webster Street development and demonstrates compliance with the City of Bay St. Louis requirement that **post-development discharge be controlled such that discharge does not exceed pre-development discharge** for the design storms evaluated.

Per City direction, runoff from the developed catchments is routed to **two detention ponds**, each with its **own outlet structure** discharging to the **existing ditch**. Each pond is sized to **store post-construction runoff** and **release at the pre-construction peak discharge rate** for its respective contributing drainage area.

2. Drainage Areas and Routing

2.1 Drainage Area Routing

- **DA1 → Pond 1 → Outlet 1 → Existing Ditch**
- **DA2 → Pond 2 → Outlet 2 → Existing Ditch**

2.2 Drainage Area Sizes

- **DA1 area:** 6,709.3952 SF = **0.15403 ac**
- **DA2 area:** 29,915.8142 SF = **0.68677 ac**
- **Total DA1+DA2:** 36,625.2094 SF = **0.84080 ac**

(An additional undeveloped area beyond limits of construction exists downstream of these drainage areas and is not included in DA1/DA2 detention sizing.)

3. Hydrologic Method and Inputs

3.1 Rational Method

Peak runoff rates are calculated using:

$$Q = C\,i\,A$$

Where:

- Q= peak runoff (cfs)
- C= runoff coefficient
- i= rainfall intensity (in/hr)
- A= area (ac)

3.2 Time of Concentration

- $T_c = 10$minutes = 0.167 hours

3.3 Rainfall Intensities (10-minute duration)

- 10-yr: $i_{10} = 7.99$in/hr
- 25-yr: $i_{25} = 9.45$in/hr
- 100-yr: $i_{100} = 11.70$in/hr

4. Runoff Coefficients

4.1 Pre-Development Coefficient

- $C_{pre} = 0.30$

4.2 Post-Development Weighted Coefficient (DA1+DA2 basis)

Impervious coverage provided (within the developed catchment basis):

- Asphalt: 0.524 ac
- Buildings: 0.201 ac
- Sidewalk: 0.051 ac
- Total impervious = 0.776 ac

Since DA1+DA2 total area is 0.84080 ac, pervious area **within DA1+DA2** is:

$$A_{perv} = 0.84080 - 0.77600 = 0.06480 \text{ ac}$$

Assumed coefficients:

- Asphalt $C = 0.90$
- Buildings $C = 0.95$
- Sidewalk $C = 0.90$
- Pervious/grass $C = 0.30$

Weighted post coefficient:

$$C_{post} = \frac{0.524(0.90) + 0.201(0.95) + 0.051(0.90) + 0.0648(0.30)}{0.8408} = 0.866$$

Use: $C_{post} = 0.866$

5. Peak Runoff Calculations (Pre vs Post)

5.1 DA1 Peak Discharges ($A = \mathbf{0.15403}$ ac)

10-yr

$$Q_{pre} = 0.30(7.99)(0.15403) = 0.369 \text{ cfs}$$
$$Q_{post} = 0.866(7.99)(0.15403) = 1.066 \text{ cfs}$$

25-yr

$$Q_{pre} = 0.30(9.45)(0.15403) = 0.437 \text{ cfs}$$
$$Q_{post} = 0.866(9.45)(0.15403) = 1.261 \text{ cfs}$$

100-yr

$$Q_{pre} = 0.30(11.70)(0.15403) = 0.541 \text{ cfs}$$
$$Q_{post} = 0.866(11.70)(0.15403) = 1.561 \text{ cfs}$$

5.2 DA2 Peak Discharges ($A = \mathbf{0.68677}$ ac)

10-yr

$$Q_{pre} = 0.30(7.99)(0.68677) = 1.646 \text{ cfs}$$
$$Q_{post} = 0.866(7.99)(0.68677) = 4.752 \text{ cfs}$$

25-yr

$$Q_{pre} = 0.30(9.45)(0.68677) = 1.947 \text{ cfs}$$

$$Q_{post} = 0.866(9.45)(0.68677) = 5.620 \text{ cfs}$$

100-yr

$$Q_{pre} = 0.30(11.70)(0.68677) = 2.411 \text{ cfs}$$
$$Q_{post} = 0.866(11.70)(0.68677) = 6.959 \text{ cfs}$$

6. Detention Storage Method (Vst)

$$V_{st}(\text{ac-ft}) = 0.08264\ (Q_{in} - Q_{out})\ T_c(\text{hr})$$

With $T_c = 0.167\text{hr}$:

$$V_{st}(\text{ac-ft}) = 0.08264\ (Q_{in} - Q_{out})\ (0.167) = 0.01380\ (Q_{in} - Q_{out})$$

For each pond:

- $Q_{in} = Q_{post}$
- $Q_{out} = Q_{pre}$

6.1 Pond 1 (DA1) Storage

10-yr

$$\Delta Q = 1.066 - 0.369 = 0.697$$
$$V_{st} = 0.01380(0.697) = 0.00959 \text{ ac-ft}$$

25-yr

$$\Delta Q = 1.261 - 0.437 = 0.824$$
$$V_{st} = 0.01380(0.824) = 0.01135 \text{ ac-ft}$$

100-yr

$$\Delta Q = 1.561 - 0.541 = 1.020$$
$$V_{st} = 0.01380(1.020) = 0.01405 \text{ ac-ft}$$

6.2 Pond 2 (DA2) Storage

10-yr

$$\Delta Q = 4.752 - 1.646 = 3.106$$
$$V_{st} = 0.01380(3.106) = 0.04278 \text{ ac-ft}$$

25-yr

$$\Delta Q = 5.620 - 1.947 = 3.673$$
$$V_{st} = 0.01380(3.673) = 0.05059 \text{ ac-ft}$$

100-yr

$$\Delta Q = 6.959 - 2.411 = 4.548$$
$$V_{st} = 0.01380(4.548) = 0.06264 \text{ ac-ft}$$

6.3 Storage Summary

Pond	**10-yr (ac-ft)**	**25-yr (ac-ft)**	**100-yr (ac-ft)**
Pond 1 (DA1)	0.00959	0.01135	**0.01405**

Pond	10-yr (ac-ft)	25-yr (ac-ft)	100-yr (ac-ft)
Pond 2 (DA2)	0.04278	0.05059	**0.06264**

7. Pond Geometry and Stage-Storage Checks

7.1 Pond 1 Geometry

- Bottom Elev = **12.0**
- Top Elev = **14.5**
- Bottom Area = **54.67 SF**
- Top Area = **509 SF**

Using a linear area-growth (frustum) storage approximation, storage is sufficient through the 100-year requirement.

Computed WSEs (from required storage volumes):

- **WSE10 ≈ 13.864**
- **WSE25 ≈ 14.051**
- **WSE100 ≈ 14.312**

7.2 Pond 2 Geometry

- Bottom Elev = **13.0**
- Top Elev = **16.0**
- Bottom Area = **316 SF**
- Top Area = **1,516 SF**

Storage is sufficient through the 100-year requirement.

Computed WSEs:

- **WSE10 ≈ 15.363**
- **WSE25 ≈ 15.622**
- **WSE100 ≈ 15.987**

8. Outlet Structures, Overflow, and Outfall to Existing Ditch

8.1 Allowable Releases (targets)

Each pond is controlled such that:

- Pond 1 $Q_{out} \leq Q_{pre}$for DA1 (0.369 / 0.437 / 0.541 cfs)
- Pond 2 $Q_{out} \leq Q_{pre}$for DA2 (1.646 / 1.947 / 2.411 cfs)

8.2 Primary Outlet (controlled discharge)

HDPE outlet barrels are standard sizes; discharge control is achieved at the outlet structure using a restricted opening (orifice plate / reducer as needed).

- Pond 1 primary outlet: 4-inch HDPE outlet barrel with a reduced control opening sized to meet the allowable release targets (typ. via removable orifice plate).
- Pond 2 primary outlet: 8-inch HDPE outlet barrel with a reduced control opening sized to meet the allowable release targets (typ. via removable orifice plate).

8.3 Emergency Overflow (grate inlet + overflow pipe)

Overflow is provided at each pond by a grate inlet set 2 inches below the top of pond, with an overflow pipe exiting the overflow structure.

- Pond 1 overflow grate elevation: $14.50 - 0.17 = 14.33$
- Pond 2 overflow grate elevation: $16.00 - 0.17 = 15.83$

Overflow pipe sizes:

- Pond 1 overflow pipe: 12-inch HDPE
- Pond 2 overflow pipe: 15-inch HDPE

8.4 Outfall / Overflow pipe slope and drop check

Adopt 0.30% slope ($S = 0.003$) for overflow/outfall conveyance runs.
Vertical drop:

$$\Delta z = S\,L$$

- Pond 1 (L = 20 ft): $\Delta z = 0.003(20) = 0.060$ ft ≈ 0.72 in
- Pond 2 (L = 28 ft): $\Delta z = 0.003(28) = 0.084$ ft ≈ 1.01 in

9. Conclusion

The proposed drainage system routes DA1 and DA2 to two detention ponds with separate outlet structures discharging to the existing ditch. Each pond is sized using the Vst method to store post-development runoff and restrict discharge to the pre-development peak rate for the 10-, 25-, and 100-year events evaluated. Pond geometry and stage checks confirm both ponds provide sufficient storage through the 100-year event. Emergency overflow is provided at each pond via a grate inlet set 2 inches below top of pond with an overflow pipe exiting the overflow structure.