

Numerical simulation of conventional and staggered dry ponds for stormwater management

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Abstract. Conventional dry ponds are typically constructed as single ponds featuring a fixed-size orifice outlet at the end. This design inherently maintains a high water stage which severely limits the pond's dynamic storage capacity and resilience to increasing runoff from climate change. This paper presents a case study investigating the hydraulic performance of novel staggered dry pond designs. The modification involved strategically staggering an 80 m long dry pond into two smaller, interconnected ponds using a second internal restriction, outlet 1. Three design scenarios were simulated by placing outlet 1 at varying distances from the inlet (20 m, 40 m, and 60 m). The performance of outlet 1 was further assessed across a wide range of orifice diameters (0.1 m to 0.8 m), while the original discharge orifice, outlet 2, was maintained. A Storm Water Management Model was utilized to simulate and compare the conventional and staggered pond characteristics. The results showed that the upstream ponds consistently maintained high flows and stages, similar to the conventional pond. However, the downstream basins exhibited dramatically different, desirable hydraulic patterns. Peak flows in the downstream segments were significantly attenuated, reducing from a maximum of 2.0 m³/s (upstream) to below 1.0 m³/s. The stage reduction was highly sensitive to the size of outlet 1. Optimal diameters (0.1 m to 0.4 m) reduced the downstream stage by more than 50% (to 0.7 m or less) compared to the maximum 1.4 m stage. This staggered design offers a viable and practical engineering solution for enhancing the flood mitigation capacity of existing dry ponds.

1. Introduction

This paper proposes a modification to the conventional stormwater detention pond or dry pond, an improvement not yet incorporated into existing engineering standards or codes of practice. The conventional dry pond utilizes a fixed-size orifice outlet to regulate discharge flow. This restricted flow creates a high water stage upstream of the orifice [1],[2], as depicted in **Figure 1**, which, in turn, limits the effective storage capacity of the pond. The primary drawback of this constant-stage design is its inability to accept and store additional runoff volume, especially



during more severe or adverse weather events [3]. Therefore, there is a critical need to enhance the pond's dynamic storage volume to ensure adequate resilience against future climate change scenarios.

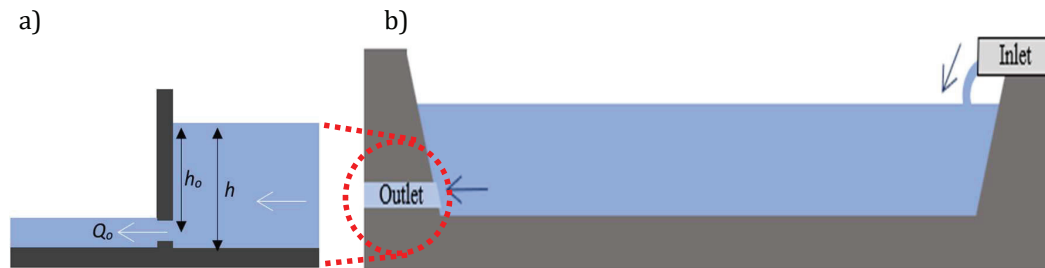


Figure 1. Cross-sectional view, a) orifice outlet and b) conventional dry pond.

Conservation of mass states that the flow rate into the pond equals to the flow rate out of the pond [4], as in **Equation 1**:

$$Q_i = Q_o \quad (1)$$

where

Q_i = inflow of pond (m^3/s);

Q_o = outflow of pond (m^3/s).

From the principle of Bernoulli, flow through the orifice outlet is defined, as in **Equation 2** [5]:

$$Q_o = C_d A_o \sqrt{2gh_o} \quad (2)$$

where

C_d = orifice coefficient of discharge (unitless);

A_o = area of orifice outlet (m^2);

g = gravity acceleration (m/s^2);

h_o = water head (m).

In an effort to optimize the stormwater detention pond's storage volume, a modification involving the installation of a second orifice is proposed, as illustrated in **Figure 2**. This design utilizes two orifices: an upstream orifice (Outlet 1) and a downstream orifice (Outlet 2). This dual-orifice system fundamentally breaks the previously constant stage pattern observed in the single-orifice design.

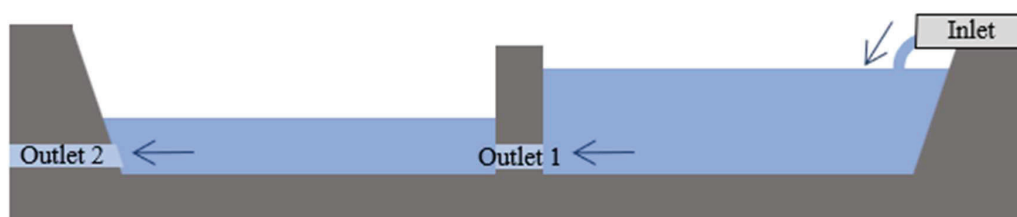


Figure 2. Cross-sectional view of staggered dry pond.

Instead, the regulation provided by the two restrictions induces fluctuating water stages throughout the pond, even when the inflow rate is maintained consistently for both the conventional and staggered configurations. Specifically, the upstream section of the pond maintains a high stage, similar to the original single pond setup. However, the downstream section exhibits a significantly lower stage, and this reduction in water level effectively creates unfilled, available storage space within the downstream area. The subsequent investigation delves into analysing these fluctuating stages by examining the hydraulic performance when Outlet 1 is strategically placed at various distances from the inlet (specifically 20 m, 40 m, and 60 m).

The proposed installation of the dual-orifice system effectively staggers the single dry pond into two smaller, interconnected basins. While a staggered design is uncommon for dry ponds, the practice of creating sequential basins is well-established and highly effective in other stormwater management contexts. Precedents of such a practice could be observed in the designs of constructed wetland, wet pond and sediment pond.

Staggering is standard practice in constructed wetlands to achieve multiple objectives. For instance, in systems combining horizontal and vertical flow treatment (**Figure 3a**), wetlands are often divided into two ponds, each designed for a distinct treatment process [6]. Similarly, separation is used to isolate areas dedicated to different types of vegetation (**Figure 3b**) [7]. In wet pond applications (**Figure 3c**), a series of basins are employed to significantly increase the hydraulic retention time. This extended retention time is critical for allowing natural, time-dependent treatment processes to fully execute [8]. The installation of skimmers and baffles achieves physical separation (**Figure 3d**), which slows the flow and facilitates the settlement of suspended solids as the water navigates the obstacles [9].

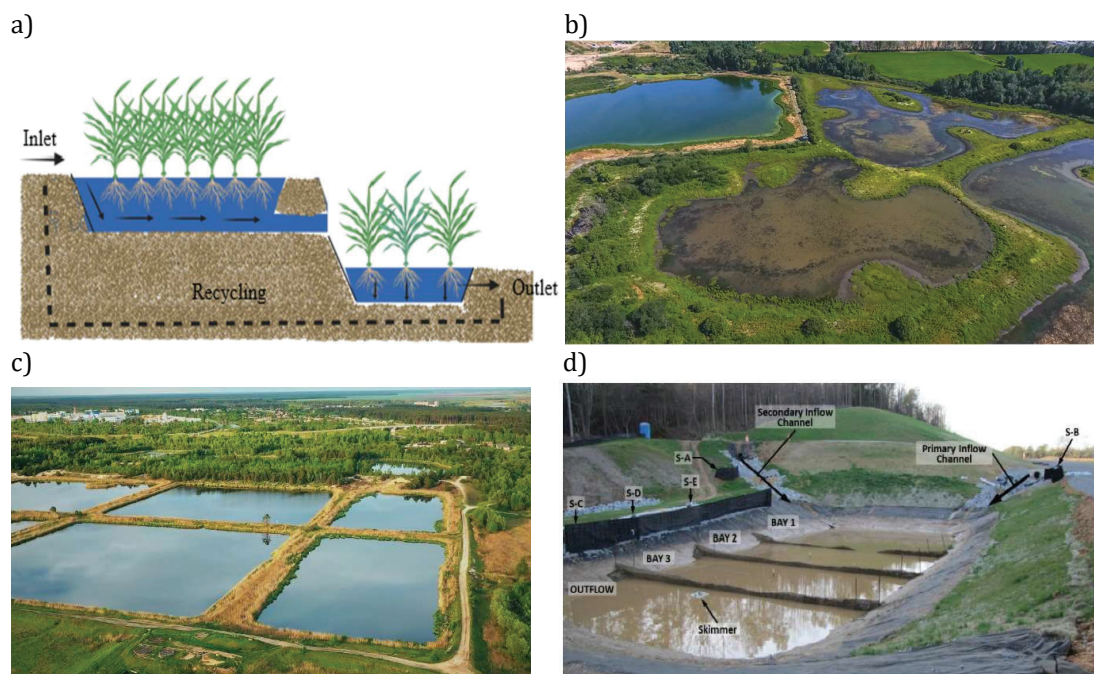


Figure 3. Staggered ponds, a) combined horizontal and vertical flow constructed wetlands [6], b) natural treatment wetlands [7], c) wet ponds [8] and sediment pond [9].

These successful applications demonstrate that dividing a stormwater basin is a valid engineering technique. By adopting this established staggering strategy, we aim to leverage the resulting flow dynamics to maximize the dynamic storage volume of the dry pond, directly addressing the limitations of the single dry pond design.

2. Materials and Methods

This study employs a two-pronged methodology, as outlined in **Figure 4**, combining field investigation with hydraulic modelling. A specific, representative site was identified to ensure the study is grounded in the real-world context of a conventional dry pond [10]. This step provides empirical data and geometric parameters essential for developing an accurate hydraulic model. Drainage modelling was conducted using the Storm Water Management Model Version 5 (SWMM5), a widely utilized software package developed by the United States Environmental Protection Agency (USEPA). SWMM5 was selected due to its recognized capability and robust performance in simulating the complex processes of urban stormwater systems and their associated control structures [11], [12].

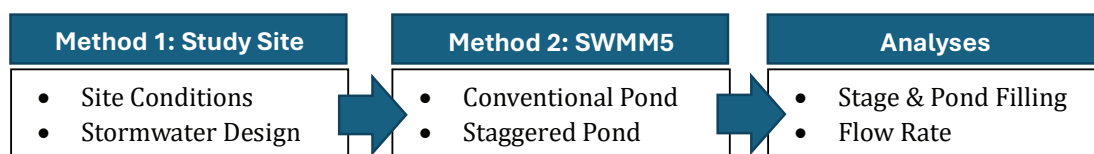


Figure 4. Flowchart of methods.

2.1 Study Site

The study site selected for this investigation is a dry detention pond located at the Institute for Rural Advancement (INFRA), situated approximately 14 km from Kuching International Airport in Sarawak. The pond's coordinates are $1^{\circ} 27' 20''$ N and $110^{\circ} 24' 59''$ E. An aerial view of the site is provided in **Figure 5**.



Figure 5. Location of study site.

This dry pond, as shown in **Figure 6**, was designed in accordance with local engineering manuals [13], [14]. Its primary function is to manage runoff from a 28-hectare (approximately 27,800 m²) campus area. The design specifications are as follows: design storm up to an intensity of 100-year Average Recurrent Interval (ARI) which equivalent to a design rainfall depth of 96.3 mm with a time of concentration, t_c of 30 minutes. It was built with a dimension of 80 m in length, 10 m in top width and 1.8 m in depth. It had a storage capacity of 1050 m³. These parameters provide a robust baseline for calibrating the hydraulic model prior to simulating the proposed modifications.

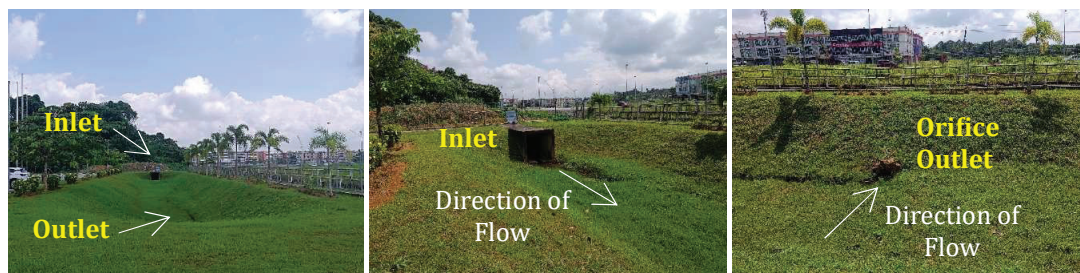


Figure 6. Photos of the selected dry pond.

The peak flow rate which could be generated from connecting catchments to be directed to the dry pond is calculated using Rational Method, as in **Equation 3**:

$$Q_c = \frac{CIA}{360} \quad (3)$$

where

- Q_c = flow rate from catchment (m³/s);
- C = runoff coefficient (unitless);
- I = rainfall intensity (mm/hr);
- A = catchment area (hectares).

For post-development condition, C value was assumed to be 0.9, resulting in a calculated peak flow of 0.72 m³/s. For pre-development condition, C value was assumed to be 0.4 which corresponded to a peak flow rate of 0.64 m³/s. With that, a permissible site discharge (PSD) which served as the target outflow, Q_o from the pond, was set at 0.58 m³/s. By substituting the value into Equation 2, the necessary orifice area, A_o was calculated, leading to a determined orifice diameter of 0.45 m.

Besides, the peak stage within the dry pond was calculated indirectly using the Manning Formula, as in **Equation 4**:

$$Q_m = \frac{1}{n} \cdot A_f \cdot R^{2/3} \cdot S^{1/2} \quad (4)$$

where

- Q_m = flow rate through conduit (m³/s);
- n = Manning's roughness coefficient (unitless);
- A_f = cross-sectional flow area (m²);
- R = hydraulic radius (m), defined as flow area divided by wetted perimeter;
- S = conduit slope (m/m).

In this context, the values of A_f and R were dependent on the water stage. Therefore, by solving the Manning Formula for the peak flow rate, the peak stage confined within the dry pond could be determined.

2.2 SWMM5 for Conventional Pond

This study utilized SWMM5 (Storm Water Management Model Version 5) to simulate the overall drainage system. The resulting model layout is depicted in **Figure 7**. The developed model integrates two primary processes, namely hydrological simulation and hydraulic routing. It first simulated the rainfall-to-catchment process, calculating the transformation of rainfall input into surface runoff. The generated runoff was then hydraulically routed through the stormwater network. This routing specifically included the simulation of the flow dynamics within the key control structures, such as the dry pond and its associated orifice outlets [15], [16].



Figure 7. Developed SWMM5 model for conventional pond.

The mentioned first process involved SWMM components 1 and 2 stated in the figure, in which the hydrological process was computed using nonlinear differential equation of sheet flow, as in **Equation 5**:

$$Q_a = W \frac{1.49}{n} (d - d_p)^{5/3} S_c^{1/2} \quad (5)$$

where

- Q_a = flow rate from catchment generated by SWMM;
- W = catchment width (m);
- d = depth of water over the catchment (m);
- d_p = maximum depression storage (m);
- S_c = catchment slope (m/m).

The above-mentioned second process involved SWMM components 3 to 7. Flow routing was the generation of hydrographs along the flow path. The flow was routed through dynamic wave approximation, as in **Equation 6**:

$$Q_b = \frac{\partial A_c}{\partial t} + \alpha m A_c^{(m-1)} \frac{\partial A_c}{\partial x} \quad (6)$$

where

- Q_b = routed flow (m³/s);
- A_c = cross-sectional area of the conduit (m²);
- x = distance along the flow path (m);
- ∂t = time step (s);
- α = flow geometry due to the conduit (unitless);
- m = surface roughness of conduit (unitless).

2.3 SWMM5 for Staggered Pond

While the diameter and location of an orifice outlet significantly influence the overall hydraulic performance and water distribution within a dry pond, there is a recognized lack of comprehensive information regarding the resulting in-pond flow patterns. To address this knowledge gap, the current study systematically investigated two primary experimental variables, namely orifice location and diameter.

The position of the outlet 1 was varied to assess its impact on flow dynamics and stage fluctuation. Three distinct locations along the 80 m pond length were simulated, namely 20 m, 40 m and 60 m from the inlet. These configurations are visually represented in **Figure 8**. Simultaneously, the hydraulic effect of the orifice diameter was assessed by simulating a varying diameters from 0.1 m to 0.8 m.

In the SWMM5 model representing the conventional pond, the 80 m long dry pond was discretized into four channel segments, each 20 m in length. The original single orifice outlet with 0.45 m diameter was located at the downstream end of the final channel segment. The model was subjected to 100-year ARI design storm and the resultant runoff hydrographs were generated from the upstream catchments and then hydraulically routed through the segmented dry pond and its single outlet structure.

The staggered pond designs, incorporating the dual-orifice system, were implemented in the SWMM5 model by introducing a second internal restriction. The original single orifice was designated as outlet 2 and kept at the downstream end of the 80 m pond. The new, variable restriction, termed outlet 1, was strategically placed within the pond to create three distinct staggering ratios (upstream segment length : downstream segment length), namely 20:60, 40:40 and 60:20 designs.

For 20:60 design, the second orifice was placed at the end of the 0–20 m segment, thus creating a separation of 20 m long upstream pond and 60 m long downstream pond. Following the same method, for 40:40 design, the second orifice was placed at the end of 20–40 m segment; while for 60:20 design, at the end of 40–60 m segment. The model was subjected to the same 100-year ARI design storm used in the baseline simulation. In contrast to the fixed-size outlet 2, the diameter of outlet 1 was systematically varied across a functional range, from 0.1 m to 0.8 m, to comprehensively assess the sensitivity of the hydraulic performance to internal restriction size.

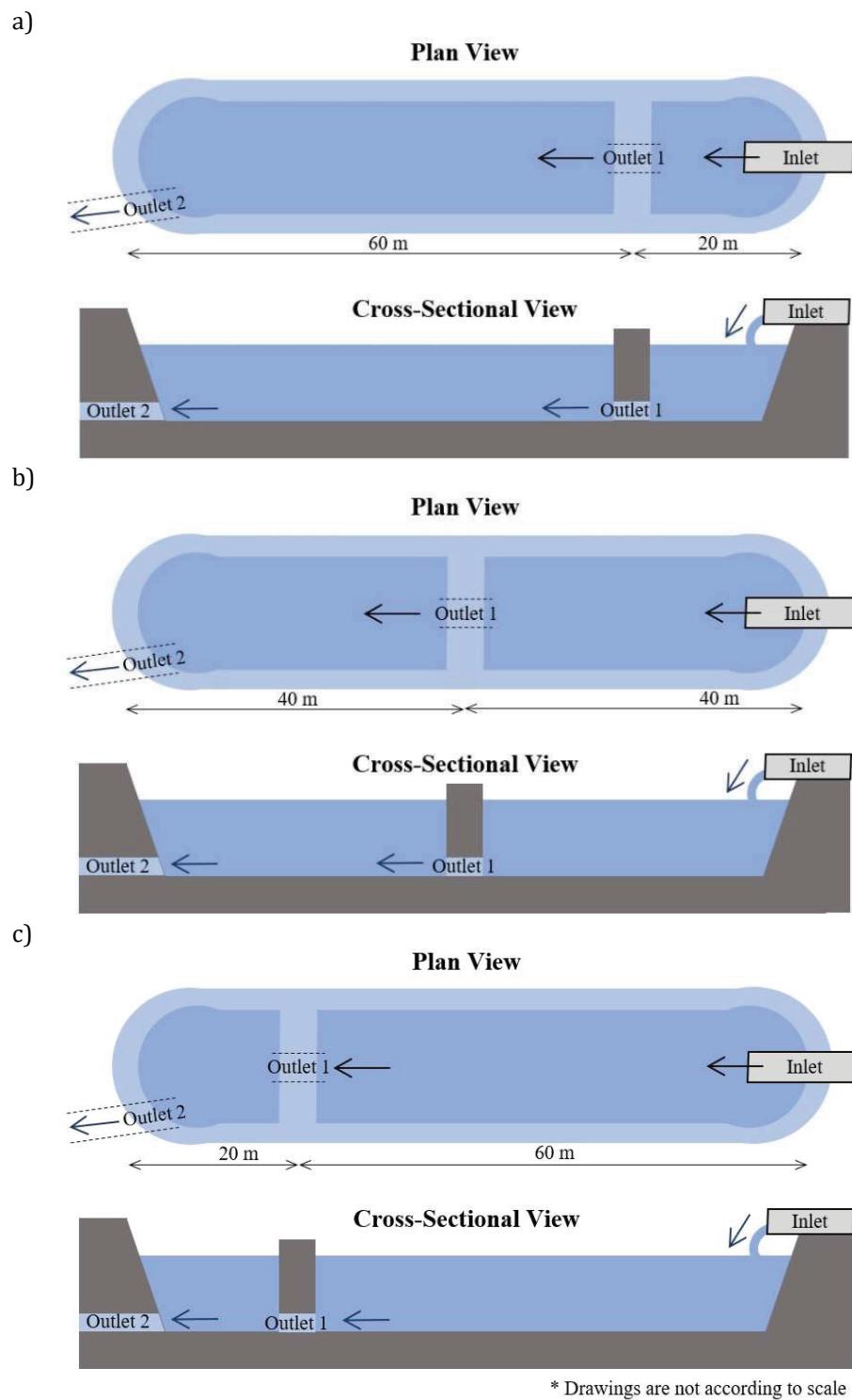


Figure 8. Staggered dry ponds, a) 20:60, b) 40:40 and c) 60:20 designs.

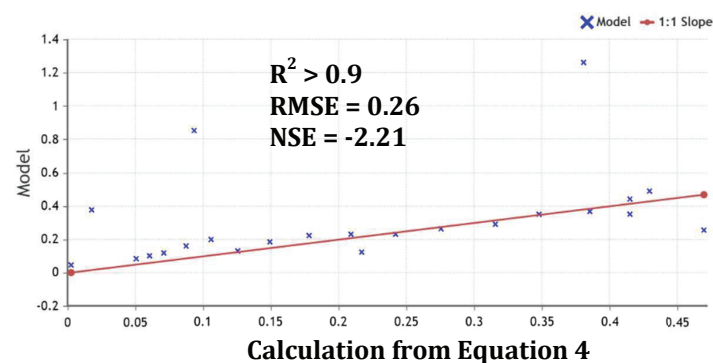
2.4 Model Evaluation

The developed SWMM5 model was evaluated to ensure its reliable performance in simulating both the conventional and staggered pond designs. The computed hydraulic data, including the flow rates routed through the pond and discharged via the orifice outlets (computed using **Equation 6**), were extracted for analysis. Simulated flow within the pond was compared against the principles of open channel flow, defined by the Manning Formula (**Equation 4**) with $n = 0.05$ for medium grassed. Simulated discharge through the orifices was compared against the hydraulic principles of flow through an opening, derived from the Bernoulli's principle (**Equation 2**) with $C_d = 0.60$ [3].

The degree of agreement between the simulated data and the theoretical benchmarks was quantified using standard statistical goodness-of-fit indicators. Selected statistical tools included coefficient of determination (R^2), root mean squared error (RMSE), and Nash-Sutcliffe efficiency (NSE). The plots are presented in **Figure 9**.

R^2 in statistical measurement, used to assess the proportion of the variance in the theoretical data explained by the model output and its values range between 0 and 1, in which 1 indicates complete match of a variable which 0 indicates otherwise [17]. RMSE used to measure the average magnitude of the errors between the simulated and theoretical values which are between 0 to infinity, in which 0 indicates complete match [18]. NSE used to determine the relative fit, where its values range from negative infinity to 1, in which 1 indicates complete match, and negative values indicate that the mean of calculated data provides better predictions than the model [19].

a)



b)

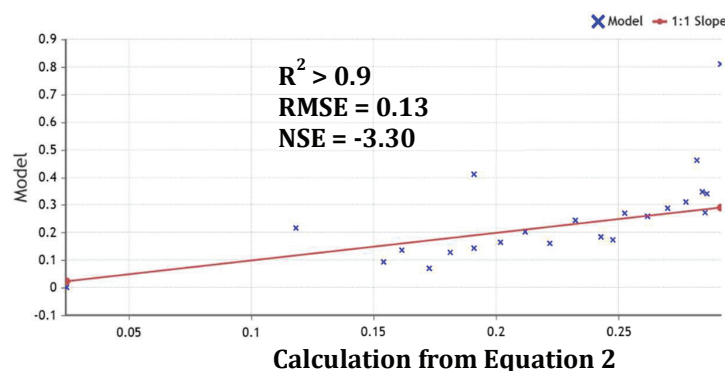


Figure 9. Model evaluation for flows through a) pond and b) orifice.

The simulated flow data within the dry pond channels demonstrated an overall correlation with the theoretical predictions derived from the Manning Formula. As depicted in **Figure 9a**, a few outliers were observed. These instances correspond to high-velocity, transient discharges occurring immediately downstream of the internal orifice outlets. While flow patterns in an unobstructed channel would not exhibit such sharp velocity peaks, these localized high flow rates were short-lived and rapidly attenuated by the stored water volume within the downstream segment. Excluding these minor, localized deviations, the bulk of the data showed reasonable agreement, as it is evident in the statistical indices (more than 0.9 in R^2 , 0.26 in RMSE and -2.21 in NSE).

Other than that, the model's computed flow through the orifice outlets similarly showed a robust match with the principles of Bernoulli's principle. **Figure 9b** shows minor outliers in the orifice flow data. These few data points were directly associated with the peak inflow events at the pond inlet, where highly dynamic head conditions temporarily violated the steady-state assumptions inherent in the simplified Bernoulli application. The overall statistical agreement verified the orifice flow simulation (more than 0.9 in R^2 , 0.13 in RMSE and -3.30 in NSE).

The high R^2 values for both flow types indicate a strong linear relationship between the simulated and theoretical flow. Although the NSE values are negative, which suggests a poor overall predictive capability relative to the mean across the entire dataset, the highly localized nature of the outliers explains this deviation, while the overall trend (as confirmed by the low RMSE values) is maintained. These confirming the SWMM5 model is suitable for simulating the different design scenarios.

3. Results and Discussion

3.1 Stage and Pond Filling

The implementation of the internal restriction effectively staggers the original dry pond, creating a cascading series system comprising two interconnected basins: an upstream pond and a downstream pond, following the direction of runoff flow. To assess the impact of this modification on hydraulic performance, both conventional and staggered ponds were subjected to the identical design storm inflow. The resultant modelled peak stages for these scenarios are presented in **Figure 10**, illustrating the distinctive stage trends achieved by the staggered design.

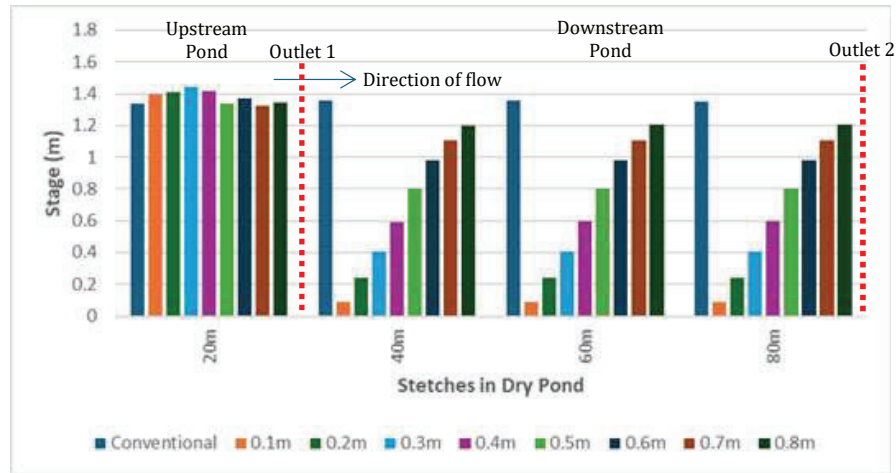
Orifice outlet is a form of flow control, and in this case, to regulate the flow being discharged from the pond. The permissible site discharge dictates the sizing of the final discharge control, outlet 2, which is typically designed to match or nearly match the pre-development flow rate [20]. In contrast, the optimal sizing and performance of the internal restriction, outlet 1, is less understood, making the analysis of its hydraulic effect a primary focus of this investigation.

In the baseline single pond design, the peak water stage was maintained consistently at approximately 1.4 m across the entire length of the dry pond, illustrating the typical high-stage, limited-storage behaviour of a conventional setup.

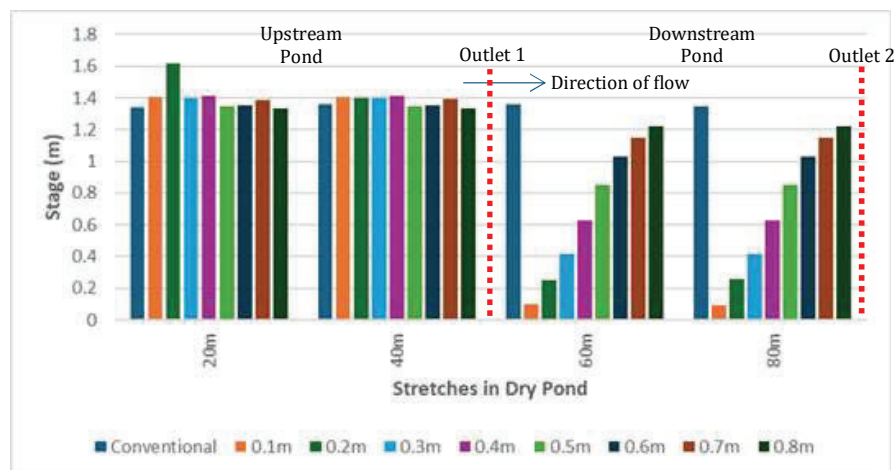
The staggered designs exhibited distinctly different peak stage patterns (refer to **Figure 10** for modelled trends). The upstream ponds consistently maintained a constant, high peak stage of approximately 1.4 m, irrespective of the diameter or location of outlet 1. This behaviour was consistent with the known hydraulic effect of a single restriction, where a significant backwater head is always generated immediately upstream of the orifice structure.

The downstream pond, which is hydraulically sandwiched between outlet 1 and outlet 2, demonstrated a highly sensitive response to the sizing of Outlet 1. Small diameters (0.1 m to 0.4 m) with smaller restriction effectively choked the flow into the downstream segment. This resulted in a dramatic stage reduction, with peak stages lowered by 50% or more (ranging from

a)



b)



c)

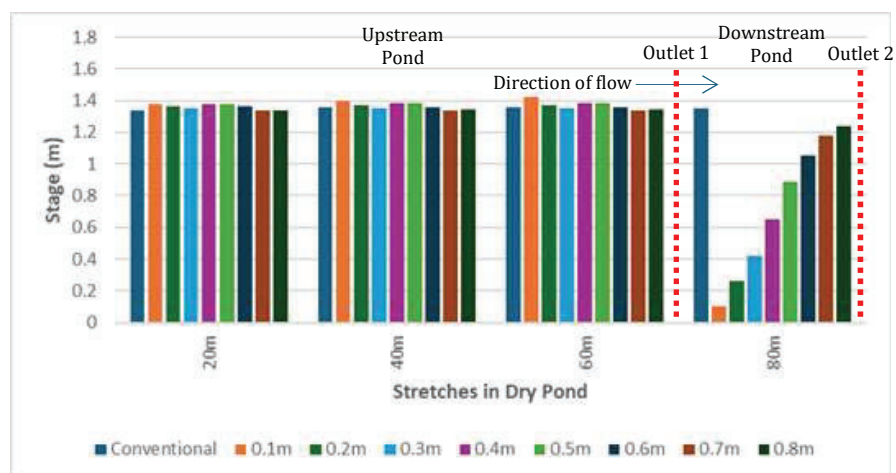


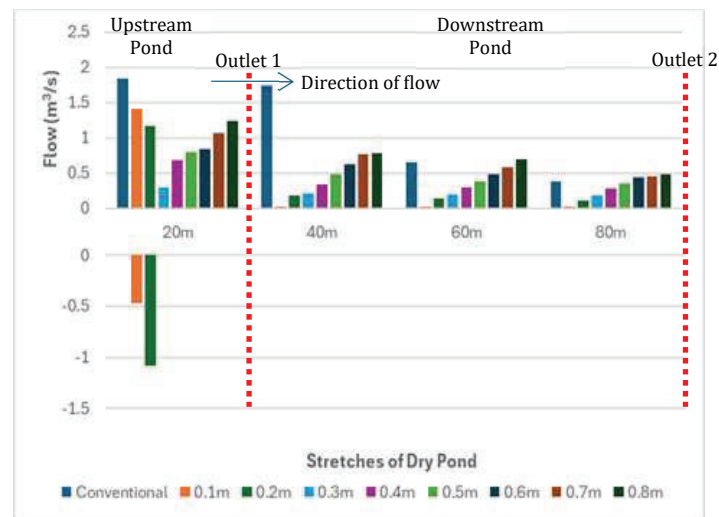
Figure 10. Peak stages of conventional pond and staggered ponds due to varying diameters of orifice outlet 1 according to a) 20:60, b) 40:40 and c) 60:20 designs.

0.1 m to 0.7 m), compared to the upstream peak stage. As the diameter of outlet 1 increased, from 0.5 m to 0.8 m, the flow restriction was reduced. Consequently, the peak stages in the downstream pond also increased, rising from 0.8 m to 1.2 m, signifying a reduced efficiency in creating dynamic storage. This demonstrates that the diameter of outlet 1 is the primary control mechanism for creating available storage in the downstream pond segment.

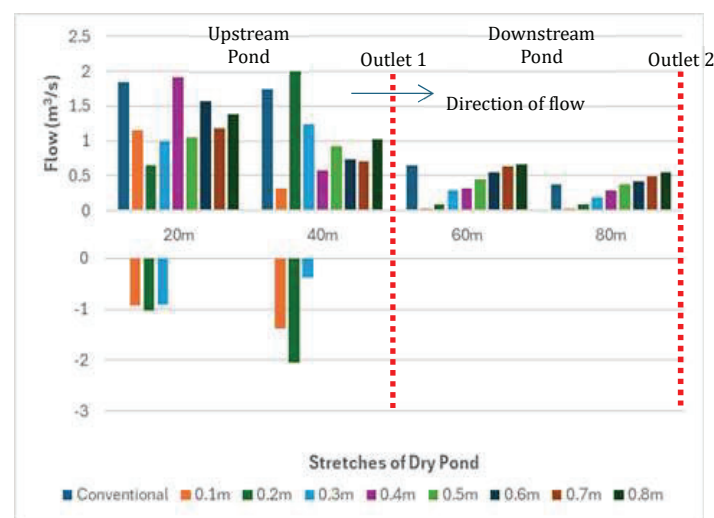
3.2 Flow Rate

As presented in **Figure 11**, the conventional single dry pond exhibited the highest peak flow rates immediately downstream of the inlet. The flow rates then gradually diminished toward the 80 m outlet. While this flow attenuation is a desired characteristic in stormwater management, the associated consistently high water stage (as discussed previously) remains the primary design drawback.

a)



b)



c)

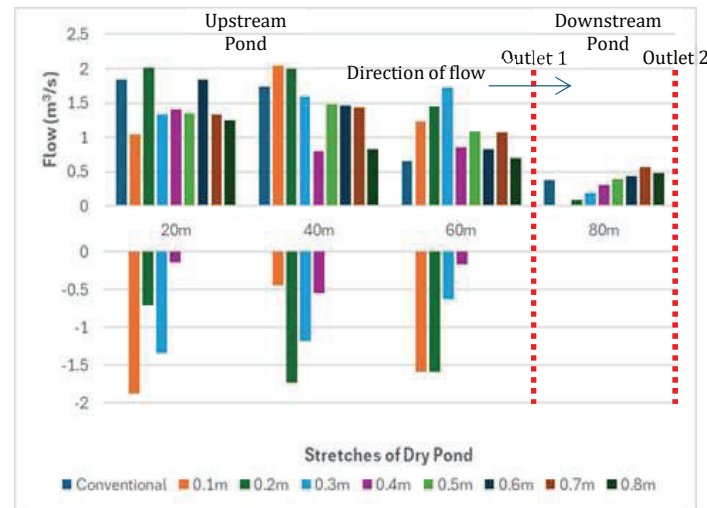


Figure 11. Peak flow rates of conventional pond and staggered ponds due to varying diameters of orifice outlet 1 according to a) 20:60, b) 40:40 and c) 60:20 designs.

The introduction of outlet 1 fundamentally altered the flow dynamics, creating distinct hydraulic regimes in the upstream and downstream segments. Across all three staggered designs (20:60, 40:40, 60:20), the flow was significantly attenuated. The peak flow rates and peak stages were consistently and significantly reduced in the downstream pond segments, regardless of the position of outlet 1. Specifically, the peak flow rates dropped from a maximum of approximately 2 m³/s in the upstream segment to below 1 m³/s throughout the downstream segment in all staggered cases.

In the upstream pond segments, the flow rates were highly dynamic and exhibited significant fluctuation. The most notable observation occurred with smaller outlet 1 diameters (0.1 m to 0.4 m). These highly restrictive settings caused the computed SWMM5 flow to register negative flow rates. This phenomenon indicates the occurrence of backwater, where outlet 1 had caused a temporary, mixed-direction flow (both forward and backward movement) within the upstream segment. The Froude Number (Fr) calculated by the SWMM5 model remained consistently below unity ($Fr < 1$) throughout the upstream segments, confirming that the flow remained in the subcritical (laminar) regime. The magnitude of the backwater effect was directly correlated with the length of the upstream segment. The 60:20 design (longest upstream segment) produced the most substantial backwater effect, demonstrating that a greater upstream length amplifies the hydraulic resistance imposed by outlet 1.

3.3 Flow and Stage Relationships

The relationship between flow rate and water stage is critical for assessing hydraulic performance. **Figure 12** presents the modeled flow-stage curves for all three staggered pond designs, juxtaposed with the performance of the conventional single pond for comparison. The plots are separated into upstream and downstream segments. Reference lines are included at 1.4 m (the maximum conventional peak stage) and 0.7 m (representing the 50% stage reduction benchmark) to facilitate the following analysis.

The flow-stage relationships for all three downstream ponds (displayed on the right of **Figure 12**) exhibit generally repeated flow-stage patterns. When outlet 1 diameters were larger

(0.5 m to 0.8 m), the resulting peak flow rates into the downstream segment clustered around 0.5 m³/s to 1.0 m³/s. These higher flows resulted in higher downstream stages. Conversely, smaller diameters (0.1 m to 0.4 m) severely restricted flow, limiting peak flow rates to below 0.5 m³/s, thereby achieving the low stages necessary for storage creation. Crucially, while the flow-stage patterns were similar, the effective dynamic storage volume differed significantly based on the downstream pond length. The 20:60 design possessed the longest downstream segment (60 m), this design achieved the largest available storage volume at any given reduced stage. This was followed by 40:40 design which exhibited a moderate storage volume due to its intermediate length (40 m) and 60:20 design yielded the smallest storage volume due to its shortest downstream segment (20 m).

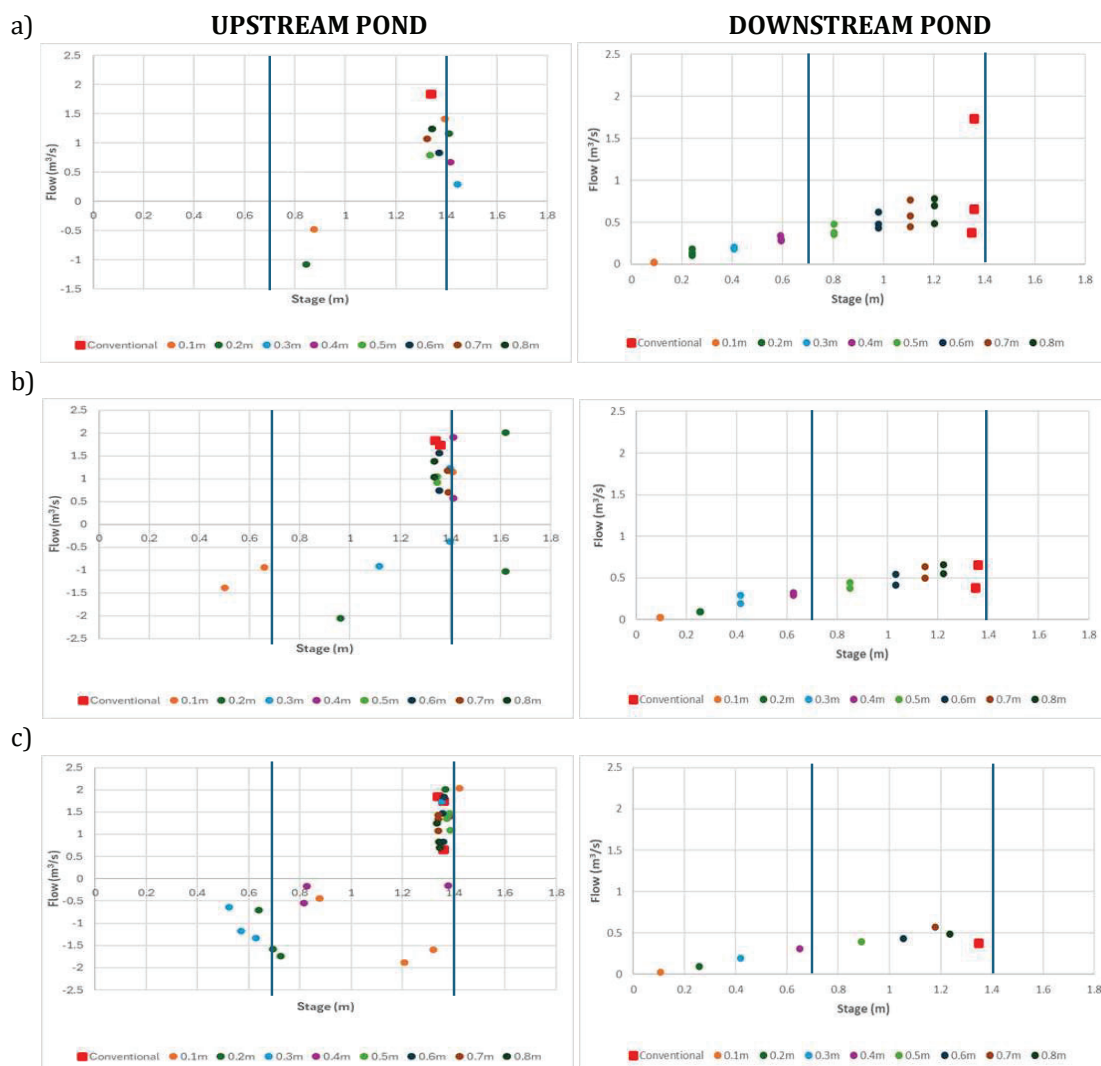


Figure 12. Peak flow and stage relationships for upstream and downstream ponds according to a) 60:20, b) 40:40 and c) 60:20 designs.

The upstream pond segments (left of **Figure 12**) demonstrated distinct hydraulic characteristics, categorized by flow direction. The group of positive flow regime data points representing forward flow exhibited high flow rates, fluctuating between 0.5 m³/s and 2.0 m³/s. These flows were strongly centred around the maximum stage of 1.4 m in all three designs, confirming that the backwater effect consistently maintains the maximum design head. Besides, the group of negative flow regime data points indicating backwater flow were more scattered but appeared to be centred around the 0.7 m stage. This suggests that flow reversal is most pronounced when the instantaneous water level is near the mid-range, possibly indicating the point where the inertial head is overcome by the back pressure from outlet 1.

4. Conclusion

This study successfully utilized a combined hydrological-hydraulic SWMM5 model to investigate the performance differences between conventional single dry pond and novel staggered dry pond designs.

The single pond baseline design maintained a consistently high water stage of 1.4 m across its entire length. Flow rates initiated high (up to 2.0 m³/s at the inlet) and gradually attenuated to the required discharge limit (0.6 m³/s at the outlet). This high-stage characteristic served as the critical benchmark for assessing storage improvement.

The upstream segment of the staggered pond, regardless of the internal orifice (outlet 1) placement, mirrored the conventional pond's hydraulic behaviour, consistently maintaining the high stage of 1.4 m. However, the key to storage optimization lies in the downstream segment's performance. Small outlet 1 diameters (0.1 m to 0.4 m) created a strong flow restriction, drastically lowering the downstream stage to 0.7 m (a 50% stage reduction) or less. This reduction was achieved while maintaining low, attenuated flow rates (below 0.5 m³/s). Conversely, larger diameters (0.5 m to 0.8 m) increased the downstream stage significantly (up to 1.2 m) as flow rates approached 1.0 m³/s, confirming a diminished capacity to create storage.

This research establishes a clear and quantifiable relationship between the physical parameters of an uncommon staggered dry pond design and its hydraulic performance. By strategically creating a downstream segment with a consistently low operational stage, the dual-orifice system effectively generates dynamic, unused storage capacity—a feature largely absent in conventional designs. This approach provides a practical, adaptable modification to existing stormwater management infrastructure, offering a viable strategy to enhance the resilience of dry ponds against the increased runoff volumes associated with future climate change scenarios.

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