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https://doi.org/10.5109/7395714

出版情報：Proceedings of International Exchange and Innovation Conference on Engineering & Sciences (IEICES). 11, pp.1554-1561, 2025-10-30. International Exchange and Innovation Conference on Engineering & Sciences
バージョン：

Free water surface constructed wetlands area estimation through river flow interpolation

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Abstract: *Urbanization and climate change can lead to the rapid and excessive accumulation of runoff in each area due to the change in its natural hydrology. Free water surface constructed wetlands (FWS CW) are a type of low impact development (LID) structure that aims to address this problem by encouraging flow attenuation to help mitigate the impacts associated with excessive runoff accumulation. This study aims to develop and assess a sizing method for FWS CWs based on streamflow using river flow interpolation. The drainage-area ratio (AR) method and the geographically weighted flow distribution curve (GWFDC) method were used to interpolate streamflow from 17 nearby donor rivers. Using the hydraulic retention time (HRT) equation, the FWS CW sizes were estimated using the interpolated flows and probability of exceedances, ranging from 10 to 90%. Results show that it is possible to size an FWS CW based on interpolated streamflow but it tends to produce large values for the area, especially for lower exceedance probabilities. Since this sizing originally captures the whole river flow, its design was resized to only accommodate the runoff from the river and the nearby bridge. It was assessed that the optimal size maximizing its hydraulic performance was approximately 229.5 – 269.0 m2 and a 1 m water depth. Further studies are needed to optimize the FWS CW for effluent treatment.*

Keywords: free water surface constructed wetland, low impact development, river flow interpolation.

1. INTRODUCTION

The growing trend of urbanization has promoted the expansion of impervious surfaces, which disrupts the natural hydrological regime in urbanized areas. This is primarily caused by the impervious surfaces hastening runoff processes [1] and is characterized by post-development hydrographs having higher peaks, shorter durations, and lower baseflows [2]. This marked increase in runoff rates and volumes present in urbanized areas leads to more frequent and severe flood events characterized by a reduction in lag time and an increase in flood peak flow of up to 30% [3]. With climate change, stronger storm events can also lead to floods with greater impacts on surrounding communities. Extreme flows, high-intensity rainfall, and storm surge also severely affect river systems, where an increase in water flows and speed could lead to serious compound flooding [4,5].

Green infrastructures such as low impact development (LID) solutions have then been recommended to many areas to promote flow attenuation [4]. LID systems aim to preserve the natural and hydrological behavior of the watersheds, which lessens the effects caused by the increased runoff. It is known to be a "near nature" solution because it mimics natural processes to manage stormwater runoff, thereby combatting the impact of climate change on urban flooding [6, 7, 8, 9]. Constructed wetlands are LID systems that provide secondary treatment to wastewater, and they provide a variety of ecological benefits, which include the enrichment of aquifers and atmosphere, protection from flooding, entrapment of solids, sediments, and other substances, absorption of carbon dioxide, storage and release of heat, mitigation of greenhouse gas emissions, and improvement of climate, water quality, and biodiversity [10, 11].

Wetlands are generally classified into two types: the free water surface constructed wetlands (FWS CWs) and sub-surface flow constructed wetlands (SSFCWs) [11 ,12]. Free water surface constructed wetlands (FWS CW) are a type of wetland that is primarily used for the treatment of wastewater [13]. The use of FWS CWs emerged in the 1990s and 2000s and have mainly been used in the treatment of a variety of polluted water, from industrial wastewater, winery wastewater, sewage, and landfill leachate [14]. While FWS CWs is generally known for their effective removal of pollutants such as total suspended solids (TSS), heavy metals, and biochemical oxygen demand in wastewater [13], several studies emphasized their hydraulic performance. Choi et al. [15] assessed the performance efficiency of two hybrid CW for runoff treatment in Dangjin City (Type 1) and Cheonan City (Type 2) in South Korea, where the two CWs were able to attain an average volume reduction of 88% and 30%, respectively, as well as pollutant removal efficiencies of ranging from 49 – 93% in treating stormwater runoff.

Rizzo et al. [16] investigated CWs for flood reduction and the treatment of combined sewer outflows, where it was able to maintain a peak flow reduction ranging from 52.7 - 95.4%. Some CWs have been connected to rivers to accommodate additional flow for reduction and treatment, such as what is explored in the study of Cryder et al. [17] where CWs treated common urban-use insecticides and Rizzo et al. [16] where an FWS was used in the treatment of a river discharge with combined sewer overflows. In recent years, the performance of CWs in tropical [18] and developing countries [19] has also been researched.

The complex relationship between the FWS CW sizing criteria and respective hydraulic performance, however, makes wetland design challenging [20]. The effective volume ratio, characterized by the length-to-width ratio, and water depth has been named as some of the important factors in maximizing the pond's hydraulic performance together with the location of outlets, plant spacing, and the addition of a subsurface berm [21, 22, 23]. The FWS CW design of Guo and Cui [20] also dictates that the aspect ratio and the water depth were the most influential design parameters in controlling the hydraulic performance of the FWS CW. The hydraulic residence time (HRT) is another key parameter that is correlated with the wetland's age [24].

This design process of FWS CWs becomes even more challenging in areas with very little to no hydrological data. About the potential difficulty in maximizing the FWS CW hydraulic performance with lacking hydrological data, this study aims to develop and assess a sizing method for FWS CWs based on streamflow using river flow interpolation. This research promotes LID and other nature-based solutions, especially in areas where there is little hydrological data and where such solutions are not extensively used. Furthermore, insights from this paper can inform decisions when it comes to sizing, design, and construction of future CWs.

2. METHODOLOGY

2.1 Study Site

The study area is located at the Binahaan River in Leyte, Philippines (11°7'0" N, 125°1'59" E), in the eastern part of the country. Figure 1 shows the hypothetical site of the proposed FWS CW, near a bridge denoted by the red dot. The province of Leyte is under a tropical climate, having monthly temperatures ranging from 23.9°C – 31.7°C (75°F - 89°F) and on average, experiences around 2220 mm (87.4 in.) of rainfall each year. The National Economic and Development Authority [25] has considered the Eastern Visayas region to be abundant with surface water bodies, with 29 of its 52 identified rivers located in the province of Leyte. Since flow data was unavailable on the site, nearby rivers were used as the basis for the estimation of streamflow amounts. Figure 2 shows the locations of all nearby rivers used in the assessment. This consisted of seventeen (17) total rivers that are within the vicinity, having drainage areas ranging from 15 – 430 km^2 compared to Binahaan River's 123 km^2.

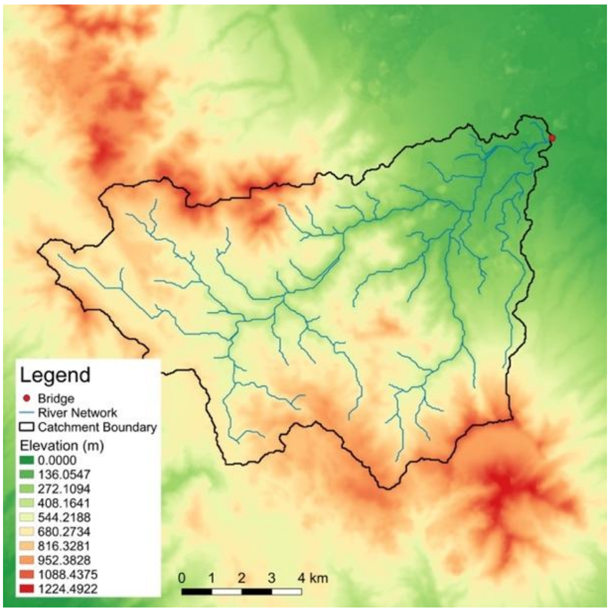

Fig. 1. Study area catchment area

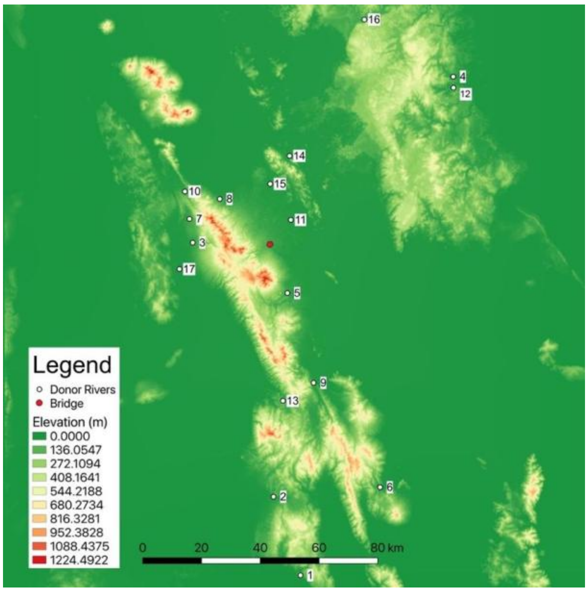

Fig. 2. Map of the nearby rivers

2.2 Rainfall scenarios

Daily rainfall data was taken from the nearest rain gauge in the study area for the years 1991-2020. A cumulative distribution function (CDF) was then created to analyze the probability of exceedance rainfall for each year of data. This study utilized the Weibull plotting position which is deemed to be one of the most theoretically sound methods for estimation [26]. The Weibull method of determining the probability of exceedance is given by Equation 1. Using the Weibull method, the precipitation of the 90th percentile per year was obtained as this corresponds to the rainfall depth with a 90% non-exceedance probability. The minimum, maximum, mean, and median was then assessed based on the yearly data.

$$P = r/(n+1) \qquad \text{(Eq. 1)}$$

Where P is the probability of occurrence, r is the rth probability of occurrence, and n is the number of observations.

2.3 River flow interpolation

Different methodologies and models have been researched to deduce flow values in rivers, streams, and watersheds alike. In areas with limited data, the concept of transferring data from gauged sites, called donors, to ungauged sites has been used in various studies to estimate streamflow data. Stagnitta et al. [27] predicted six different streamflow statistics of ungauged sites in the Apalachicola–Chattahoochee–Flint watershed in the United States using four estimation techniques, namely: regional regression, regional regression plus estimated hydrogeologic indices, baseflow correlation, and scaling. Yilmaz and Onoz [28] applied three estimation methods, namely the single-donor drainage area ratio method, the multiple-donor drainage area ratio method, and the inverse similarity weighted methods in estimating the daily streamflow of the ungauged site in the Euphrates basin in Turkey.

For this study, two methods were utilized to interpolate the streamflow of the site area: the drainage area ratio (AR) method and the geographically weighted flow distribution curve (GWFDC) method. These were then presented in flow distribution curves (FDC) to present their values across the 90%, 80%, 70%, 60%, 50%, 40%, 30%, 20%, and 10% exceedance percentiles.

2.3.1 River Area-ratio (AR) method

The AR method, as shown in Equation 2, is commonly used to statistically transfer streamflow from a gauged location to an ungauged location based on the drainage areas of both sites [29]. This method has previously been used in research that aims to estimate discharge values at ungauged sites or catchments [28, 30]. The selection of the stream to transfer to the ungauged site in this study was mainly based on its distance from the study area and its drainage area.

$$Q_1 = Q_2 \left(\frac{A_1}{A_2}\right)^{\phi} \quad \text{(Eq. 2)}$$

Where Q_1 is the estimated streamflow for the ungauged site, Q_2 is the streamflow at the gauged site, A_1 is the drainage area of the ungauged site, A_2 is the drainage area of the gauged site, and $\Phi = 1$.

2.3.2 Geographically weighted flow distribution curve (GWFDC) method

The GWFDC method, shown in Equations 3 and 4, is a way to spatially interpolate streamflow at an ungauged site given multiple source sites. This method builds on the drainage-area ratio method by maximizing regional information and weighting streamflow at gauged stations by their distance to the ungauged station [31]. All seventeen nearby rivers' streamflow data were used for this method.

$$Q_d = \frac{\sum_{j=1}^{n} w_j Q_{dj}}{\sum_{j=1}^{n} w_j} \quad \text{(Eq. 3)}$$

$$w_j = \frac{1/d_j}{\sum_{j=1}^{n} 1/d_j} \quad \text{(Eq.4)}$$

Where Q_d is the combined estimate of streamflow at the ungauged site, n is the number of gauged sites, Q_{dj} is the estimation from gauged site j, w_j is the weight corresponding to that gauged site, and d_j is the distance between the ungauged site and the gauged site.

2.4 Constructed wetland sizing

The interpolated streamflow values were used in conjunction with the hydraulic retention time (HRT) to assess appropriate sizes for the FWS CW. Equation 5 shows the formula for the HRT. The relationship of four important FWS CW sizing parameters; namely, the streamflow, the area, the HRT, and the depth were graphed to give insight into the relationship of these variables and to assess the FWS CW across different scenarios.

$$t = \frac{Ad\varepsilon}{Q} \quad \text{(Eq. 5)}$$

Where t is the hydraulic retention time in days, A is the area of the constructed wetland in square meters, d is the water depth in m, $\mathcal{E}$ is the wetland porosity, and Q is the flow rate of the river in cubic meters per day.

Several assumptions have been made to limit the design criteria of the proposed wetlands. For this study, the depth of water investigated ranged from 0.3 to 2 meters, based on the North American Database, which states that existing FWS CW have depths ranging from 0.1 m - 2 m, and the study of Bendoriccio et. al. [32], who suggests an open water depth of 1.3 m to 2.5 m for FWS CWs. The hydraulic retention time (HRT) was limited to 2 – 10 days as there were variations for HRT in literature to maximize pollutant removal. Bendoriccio et al. [32] also suggests an HRT of 5 to 10 days for proper TSS, BOD, and nutrient removal efficiency. Another study by Crites [33] mentions that a value of 8 to 14 days is appropriate for dissolved nitrogen removal. On the other hand, Merino-Solís et al [34] recommend an HRT of 2 to 3 days is most effective to remove organic matter and nutrients from the water. The aspect ratio was set to be greater than 2:1 so that plug flow conditions are met [35] and less than or equal to 4:1 to avoid any form of hydraulic, overflow, and detention time problems [36, 37] while the porosity was presumed to be 0.75 as it was an acceptable value for vegetated wetlands observed in previous studies [38, 39].

3. RESULTS AND DISCUSSION

3.1 Rainfall Analysis

Figure 3 shows the amount of precipitation for each month over the 31 years of data gathered from 1990 to 2020. The area receives an average rainfall of 2,754 mm and it is distributed evenly throughout the year with no dry seasons, considering that it is under a Type IV Philippine climate [40]. The months of December to January appeared to have the highest occurrence of precipitation, which can peak up to 1000 mm, while the months of April to May have the lowest occurrence of precipitation, only reaching around 5000 mm.

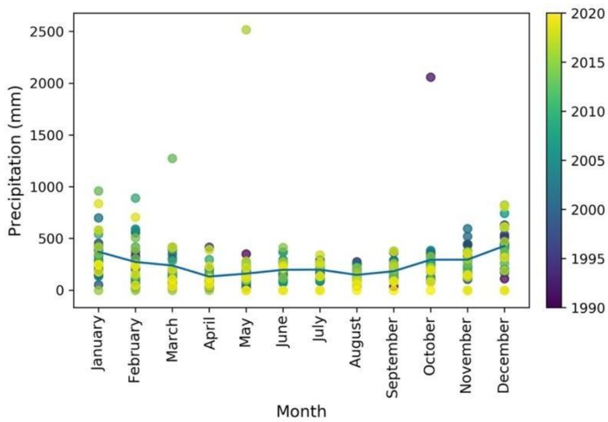

Fig. 3. Rainfall frequency experienced in the study site

A cumulative distribution function per year was also produced as this would provide a more thorough and comprehensive statistical analysis of the rainfall data. Figure 4 shows the cumulative distribution generated from the rainfall data. This study evaluated the 90th percentile of rainfall as it is commonly used in LID design, and it was also used in previous studies that focused on it such as Frias & Maniquiz-Redillas [41] and Garbanzos & Maniquiz-Redillas [42]. Performing basic statistical analysis on the 90th percentile of historical rainfall, it was found that the minimum value was 13.8 mm, observed in the year 2019, and the maximum value was 35.1 mm, observed in the year 2017. The mean 90th percentile value was estimated to be 23.07 mm in the rain gauge.

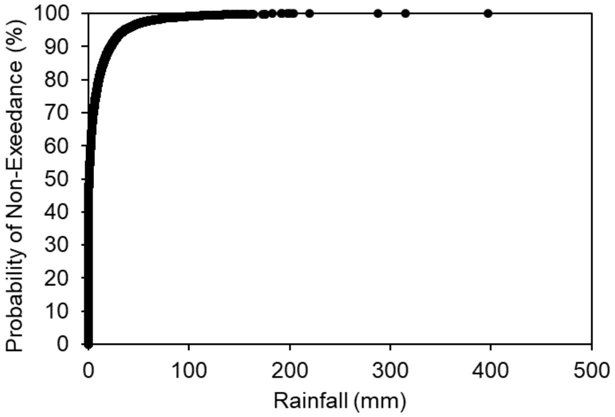

Fig. 4. The cumulative distribution function for the 30-year rainfall data

3.2 Interpolated flow distribution curves using the AR and GWFDC methods

Figure 5 shows the interpolated FDCs based on each of the seventeen donor rivers using the AR method. Because the AR method makes use of a ratio to transfer flows, it is only the streamflow in relation to the catchment size that influences the final interpolated flow. Despite this, there is still a significant amount of variability between the interpolated FDCs, depending on which donor river is used. Among the 17 donor rivers, river 5 was selected as the representative river for the AR method because it was the closest river to the study area, and it also had a similar drainage area (131.33 km^2) to the Binahaan River (122.92 km^2).

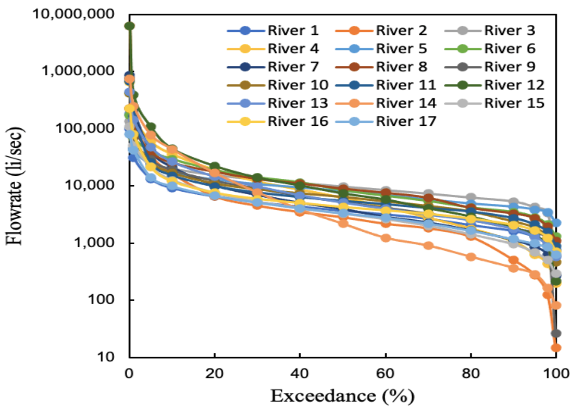

Fig. 5. Interpolated flow distribution curves using the AR method based on the nearby rivers

In Figure 6, the AR interpolated FDC was plotted to give insight into the behavior of the flow compared to the mean, minimum, and maximum. As can be seen from the graph, the interpolated FDC intersects with the average at roughly 20% exceedance. This indicates that the chosen curve may provide an overestimation of flows with exceedances greater than 20% and an underestimation of flows with exceedances less than 20% when using the average interpolated FDC as a reference. The GWFDC method was then performed, using values from all 17 donor rivers. The interpolated FDC from the GWFDC method was plotted against the interpolated FDC from the AR method in Figure 7. The two FDCs intersect at around the 20% exceedance mark, and there was a deviation in the extreme ends of the exceedance axis. This indicates that the GWFDC method predicts higher flows for low exceedances and lower flows for high exceedances. Both the AR and the GWFDC methods for interpolating flows were deemed acceptable, but certain important factors such as land coverage were not taken into consideration in the AR method due to a lack of data.

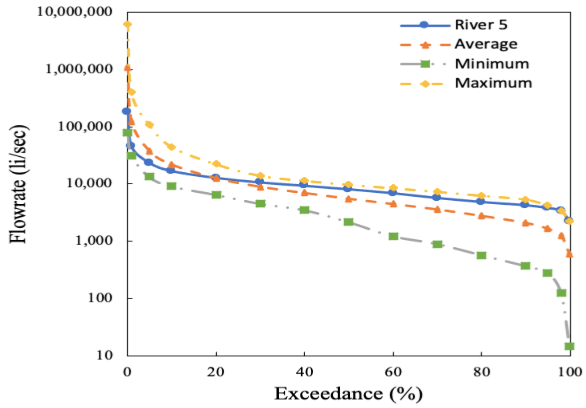

Fig. 6. Interpolated flow distribution curves using the AR method based on the nearby rivers

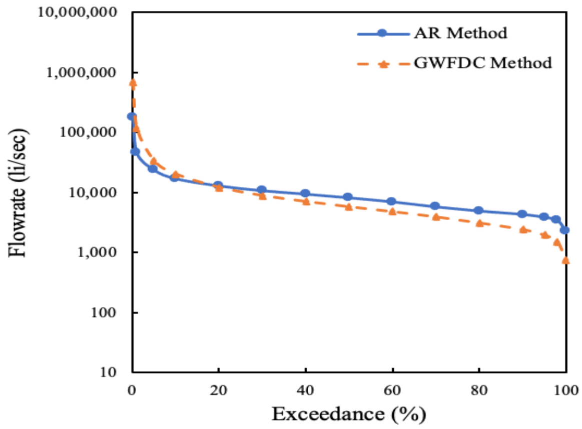

Fig. 7. Comparison of FDCs from the GWFDC and AR methods

3.3 CW sizing based on interpolated river flows

Multiple designs of the FWS CW were created based on the flow rate from varying exceedances and flow interpolation methods used. Figures 8a-i shows this relationship for flows based on the AR method, while Figures 9a-i shows the same relationship based on the GWFDC method. Results have depicted the direct relationship between HRT and area and the indirect relationship between depth and area, which are features of the HRT equation. Surface areas range from 2 to 60 km2 in the AR method and 1 to 70 km^2 in the GWFDC method, which shows the slight difference in values between the two interpolation techniques across all compared exceedances. Larger FWS CWs were created from this assessment as the flows used to size the FWS CW have exceedances $\leq$ 90%, whereas typically, wetlands are designed to accept much less water. More research is still needed to determine the appropriate exceedance to select.

Choi et al. [15] also mentions that increasing the surface area over the catchment area (SA/CA) ratio of the structure yields better volume reduction and pollutant removal efficiencies, although large CWs were often not necessary to attain their best performance. The researchers generally recommend the use of deeper CWs in the site to provide considerable space for construction in the site area, but an increase in water depth could also affect the pollutant efficiency of the CW [43] as well as its hydraulic performance [44].

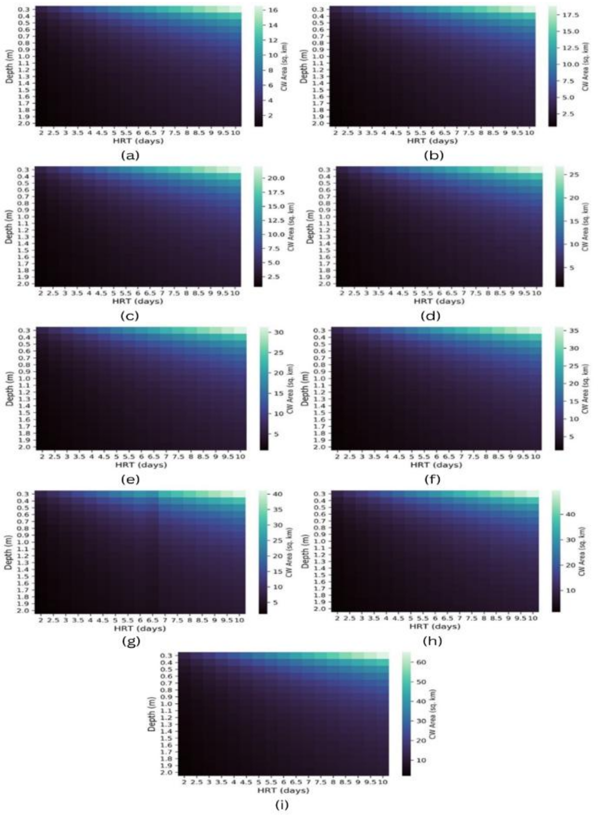

Fig. 8. Possible FWS CW areas based on the interpolated flows using the AR) method for the (a) 90%, (b) 80%, (c) 70%, (d) 60%, (e) 50%, (f) 40%, (g) 30%, (h) 20%, and (i) 10% exceedance

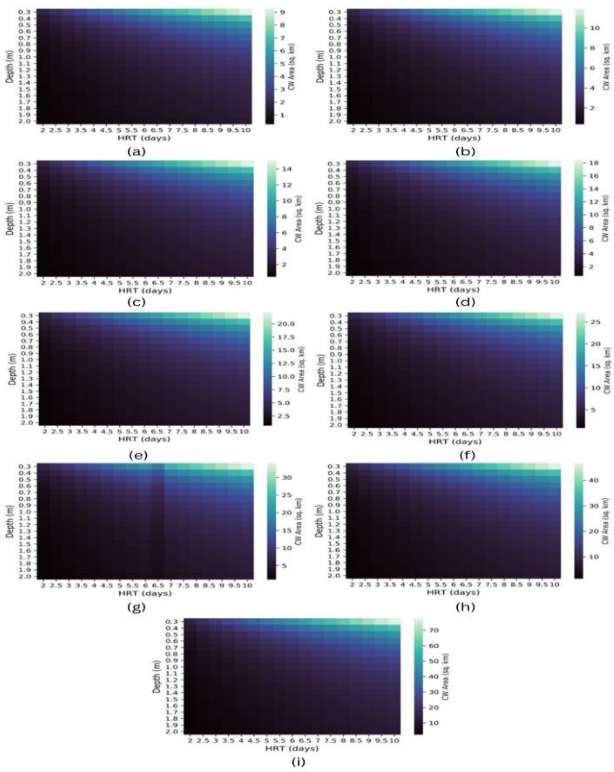

Fig. 9. Possible FWS CW areas based on the interpolated flows using the GWFDC method for the (a) 90%, (b) 80%, (c) 70%, (d) 60%, (e) 50%, (f) 40%, (g) 30%, (h) 20%, and (i) 10% exceedance.

3.4 Resizing CW size based on bridge and river runoff

CW sizes in the previous assessment were not ideal given that it was sized to accept the entire river flow. To reduce the size of the wetland, the CW was then proposed to accept only a portion of the runoff from the river and the runoff generated from a nearby bridge. This method includes obtaining the runoff from the streamflow data and adding it to the bridge surface runoff attained from the simulation. The sizing criteria were limited to a water depth of 1 meter and an aspect ratio (L: W) of 3:1. The HRT value set for this assessment was 2 days because the antecedent dry days (ADD) of the study area was between 2 to 3 days. Results were then presented in terms of minimum, maximum, and average size from the Weibull probability of exceedance calculations.

The runoff computation was taken in two parts: the runoff from the streamflow and the runoff generated on the nearby bridge. For the river runoff, this study used the 100% exceedance of rainfall in the site, which is approximately 182.6 mm of rainfall at a return period of 3.6 years. This yields a runoff amount of 110 mm, which was computed by multiplying the discharge by the area of the catchment. For the bridge runoff, the minimum, maximum, and average values obtained in the Weibull analysis were used, which were 13.8, 35.1, and 23.07 mm, respectively. These values were also deduced as the runoff generated on the bridge given that the bridge was fully impervious without any controls to infiltrate nor store incoming stormwater. Adding these two runoff types and using the set limitations for CW design, the size of the new CW was then created.

Table 1 shows the specific dimensions of the resized constructed wetland. Results have shown that the resized CW has an area that ranges from 229.5 – 269.0 m2. This size is comparably larger than most laboratory or pilot scale FWS [45- 47], whose CW sizes were less than 100 m^3, although they cannot accommodate larger amounts of flows and treatment processes that larger wetlands could provide. This has been observed in the study of Zhang et al. [48], whose 15,000 m^2 wetland was used to treat 2500 m3/day of municipal effluent from a wastewater treatment plant as tertiary treatment and Wallace and Davis [49], whose 110,000 m^2 subsurface wetland was utilized to treat 7,949 m^3/day of design flow to treat benzene, toluene, ethylbenzene, and xylene (BTEX) contaminated groundwater.

There are similar FWS sizes seen in other research compared to the results produced in this study, such as the Toenepi FWS in the study of Tanner and Sukias [50], which had a surface area of 293 m2 and the constructed wetland system from the study of Gottschall et al. [51] with two FWS cells having volumes of 202.95 m^3 and 163.35 m^3, both to assess the nutrient removal of incoming runoff and sewage. Like the two mentioned studies, most papers still focus on the FWS CW's efficiency in treating polluted stormwater and sewage [52], rather than focusing purely on its hydraulic performance.

FWS design is also highly variable to its purpose as well as other factors such as hydraulic budget, slope, flow patterns, and site selection [53]. Characterizing and understanding the land-cover, water quality, and flows of the site will be essential and the planning for the construction of the CWs, and optimization studies such as what is performed by Garbanzos, et al [55] could be utilized in the determination of the location and sizing of the FWS CWs.

Table 1. Recomputed sizing of the constructed wetland considering bridge runoff

Case	Bridge Runoff (mm)	Wetland area assuming 1m depth (m^2)
Minimum	13.80	229.5
Maximum	35.10	269.0
Average	23.07	246.7

4. CONCLUSION

This paper investigated an alternative method of sizing FWS CW using interpolated streamflow based on nearby rivers. Two streamflow interpolation techniques were used: namely, the area-ratio (AR) method, and the geographically weighted flow distribution curve (GWFDC) method. The GWFDC method appeared to predict higher flows for low exceedances and lower flows for high exceedances, whereas the opposite was observed in the AR method. The seventeen FDCs from the AR method exhibited high variation, especially towards the extreme ends of the exceedance axis.

The interpolated flows from both the AR and GWFDC methods were then used in the hydraulic retention time (HRT) equation to devise different sizes for an FWS CW. The area, depth, and HRT were then graphed for the 90%, 80%, 70%, 60%, 50%, 40%, 30%, 20%, and 10% flows for both the AR and GWFDC interpolated FDCs. Both methods showed a linear relationship between the HRT and surface area and an inverse relationship for the depth and surface area. The resulting areas were relatively large because the flows used to size the FWS CW have exceedances $\leqslant$ 90%, whereas typically, wetlands are designed to accept much less water.

The CW was then proposed to accept only a portion of the runoff from the river and the runoff generated from a nearby bridge by utilizing rainfall data and estimating river runoff. This study proposes an FWS size of about 229.5 – 269.0 m2 and a 1 m water depth to accommodate the runoff of both the river and the nearby bridge. Future research is recommended to optimize the sizing criteria of CWs for pollutant reduction of incoming effluent.

5. ACKNOWLEDGEMENT

This research was supported by the Department of Science and Technology (DOST) under the 'DOST's Grants to Outstanding Achievement in Science and Technology' Program and the DOST project entitled 'Rainfall and Stormwater Runoff Management Technology for Tropical Climate (RainS-TECH)'.

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