MINIMIZING POND SIZE USING AN OFF-SITE POND IN A CLOSED BASIN: A STORM FLOW MITIGATION DESIGN AND EVALUATION

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ABSTRACT

Construction of large on-site detention pond to manage the stormwater runoff in the project area is not only expensive, but also a waste of developable land. To minimize the pond size, a systematic design of storm flow routing followed by model verification is necessary. This study presents a challenging stormwater management design for a site located in a complex urban setting at Tallahassee, Florida, USA. The site, a 4.6-hectare (11.4 acre) wooded area, was developed into a swimming pool complex resulting in increased post-development runoff. This increased runoff was managed by designing an on-site pond, minimized by placing it in series with an existing downstream off-site pond of a closed basin. The available storage of the downstream pond was efficiently used to reduce the upstream pond size. To minimize the on-site pond, the design considered re-arrangement and re-sizing of pre-development basins that allowed releasing some portion of post-development runoff below its pre-development level in the directions where it was allowed to drain. The excess runoff generated from the area was routed through the on-site pond and discharged into the existing off-site pond, where all runoff was retained to meet the guidelines of a closed basin. The short duration simulation results (8-hr and 24-hr design storms) confirmed significant off-site runoff reduction for the post-development condition. Besides short duration simulations, the extended simulation results (for the entire 1-yr period) also revealed that the on-site and off-site ponds can jointly manage all extreme runoff including the runoff of a historical extreme wet year.

Keywords: Closed basin, design evaluation, extended simulation, land development, runoff, SCS curve number, stormwater.

1 INTRODUCTION

The increasing cost of land, especially in urban area requires efficient and effective use of property during the process of its development. One of the major costs of site development is construction of a new retention or detention facility for stormwater management. This cost increases with the size of the pond, because it is directly related to the amount of earthwork and loss of developable land. The size of the pond depends on the impervious area draining to it, the infiltration rate of the area, and allowable discharge rate. Besides these, the size of the pond becomes excessively large when it is located in a closed basin where retention of all runoff is mandatory. This leads to enormous development cost and it forces engineers and developers to find alternatives to reduce the pond size.

Traditional designs of detention ponds are typically based on routing the runoff through the pond and releasing it downstream at controlled rates [1]. This routing is typically done by allowing controlled release through an orifice of discharge structure of the pond [2]. The main objective of designing the storage facility is to reduce the peak flow [1, 3] and volume of runoff generated as a result of land use modification and to provide water quality treatment. The basic policy that guides the stormwater management design is the peak flow, after development, is required not to exceed the pre-development level for one or more design storms with a given return period and storm duration [4, 5]. The guideline of many regulatory agencies for a pond located in a closed basin is even more stringent and requires retaining all runoff within the basin [6]. To meet the design standard, hydrologic

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models must be developed to simulate the runoff for design evaluation. The model predictions of pre- and post-development runoff determine the size and configuration of the detention pond [7].

The design and the evaluation of the performance of stormwater detention or retention ponds using hydrologic models have been discussed by many noted experts. The use of design storm to simulate the runoff routing through various best management practices such as ponds were studied and presented (e.g. [8]). The efficient sizing of wet detention ponds using design curves (that relates capture volume and treatment efficiency) generated by extended simulation results using the SWMM4 model were examined [2]. Design and flow routing of ponds or depressions in karst geologic formations were analyzed [4, 9]. These studies examined the flow routing of ponds and the attenuated flow is finally discharged either in downstream watercourse, streams or in sinkholes that are assumed to have unlimited available storage. The focus of this paper is to evaluate a design of a dry-detention pond that efficiently utilizes the available storage of an existing downstream pond, which must retain all runoff without any discharge (closed basin criteria). The increased runoff due to the development of 4.6 hectare site was managed by a systematic design: re-organizing and re-sizing the pre-development basins and use of a dry-detention pond on the site. To minimize the pond size, the post-development runoff rate was designed to be released below its pre-development level in the direction where they were draining. The remaining runoff was retained by two ponds (on-site and off-site ponds) in series. The performance of the design was evaluated by developing a short duration runoff model and an extended simulation model. The short duration model was used to determine off-site runoff and optimizing the size of the on-site pond. This was achieved by simulating runoff of 8-hr and 24-hr design storms of 10-yr, 25-yr, 50-yr and 100-yr return periods. Besides the short duration simulation, an extended simulation of 1-yr period was used to simulate runoff to on-site and off-site ponds to confirm that they both can retain all runoff of any extreme wet event. The rainfall information for the wettest year in the area (1994) was used for this purpose.

2 MATERIAL AND METHODS

As site development usually results in increased runoff, a systematic hydrologic analysis is performed for the stormwater management design. The hydrologic evaluation process starts with characterization of the site (e.g. land cover, infiltration rate) to be developed, delineation of pre- and post-development basins of the site. The runoff is then simulated from these basin areas using simulation models, which are usually developed using computer programs. A common practice used by engineers and developers to simulate the runoff is the use of design rainfall events of that locality. These design rainfall events are derived from intensity–duration–frequency (IDF) curves (developed by applying statistical method to long time series of rainfall data) [10]. The simulated runoff is then routed through the designed facilities (e.g. ponds) to determine the sustainability and performance of the mitigation plan.

2.1 Study area

The project was located in the city of Tallahassee, Florida, USA. Figure 1 shows the topography of the area. As seen in Fig. 1, the project area was 4.6 hectare and it was within a large parcel (93 hectares) that encompasses a golf course. The property is surrounded by roads on all sides. The project site, located at the east side of the property was a wooded area; mainly consisted of pine, oak and cedar trees. An existing dry-detention pond (is addressed 'off-site pond' hereafter) located outside the project area (in the golf course), shown in Fig. 1, was considered to be in an isolated small closed basin. According to local rules, a pond in a closed basin must retain all the runoff that it receives.

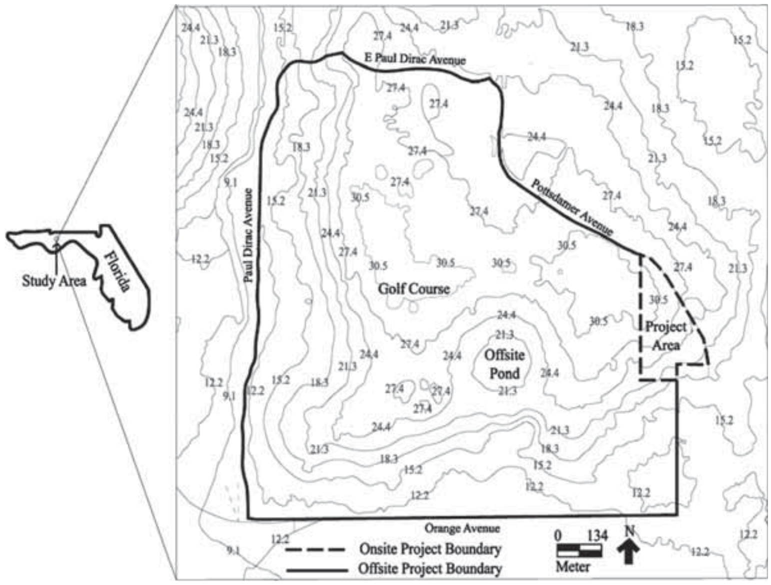

Figure 1: Location of the study area (contours are in meter above MSL).

This site was developed to facilitate a swimming pool complex using current regulations and construction practices of the local area. A diving pool, swimming facility, buildings, parking lot and driveway access were constructed on the site. The new construction resulted in 1 hectare impervious area on the site. To mitigate the stormwater runoff from the site, a dry-detention pond (is addressed 'new pond' hereafter) was constructed within the project area.

According to Soil Survey of Leon County Florida, USDA [11], the soil in this area is classified as Orangeburg Fine Sandy Loam (2–5% grade) with hydrologic group 'B'. The soil investigation showed a low infiltration rate at the project site and the golf course. Infiltration rate was found 3.8 cm per day and 5.1 cm per day respectively at the project site and the off-site pond. The investigation also suggested that the groundwater was between 7 to 8.2 m below the surface and it might change 3.1 m or more during wet season.

2.2 Basin delineation

The hydrologic evaluation process of the drainage design was initiated with the delineation of basins for both pre-development and post-development situations. These basins were developed based on the contours of the terrain that determined the runoff direction from the area. Figure 2 presents the basins developed for project area (4.6 hectare) and Fig. 3 displays the delineated off-site basin that drains to the off-site pond located in the closed basin at the golf course.

2.2.1 Pre-development basins

Before development, the project area consisted of two basins (Fig. 2a) and both of them were wooded areas. The 1.7 hectare Pre-East Basin drained towards the east. The runoff from this area was discharged through a 61-cm corrugated metal pipe towards the east side of the property. The other Pre-South Basin (2.9 hectare) drained (overland flow) towards the south side of the project area. The off-site basin (located in the golf course) with an area of 17 hectare drained to the off-site pond

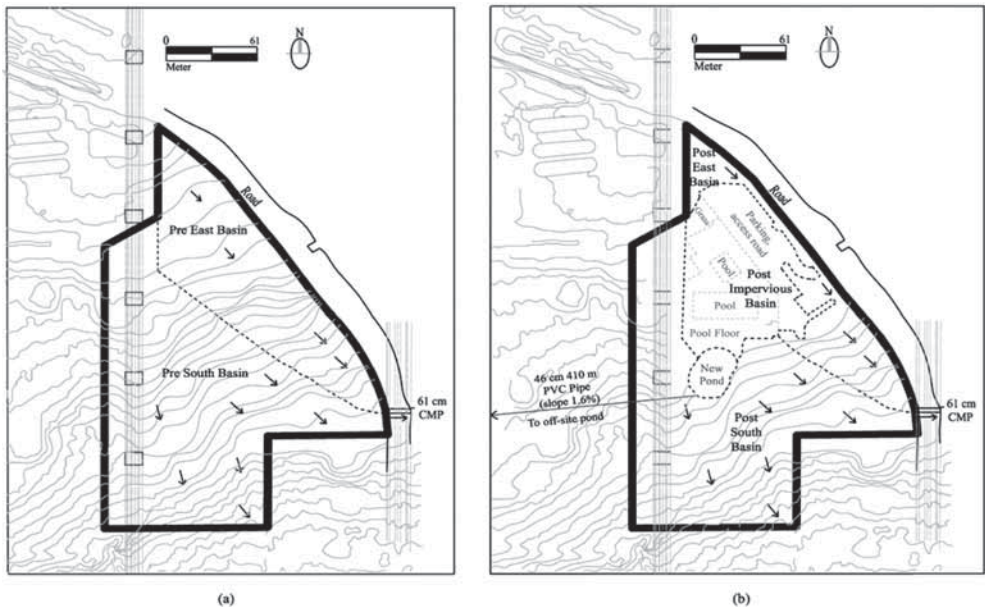

Figure 2: Basin maps in project area – (a) Pre-development basin map, (b) Post-development basin map.

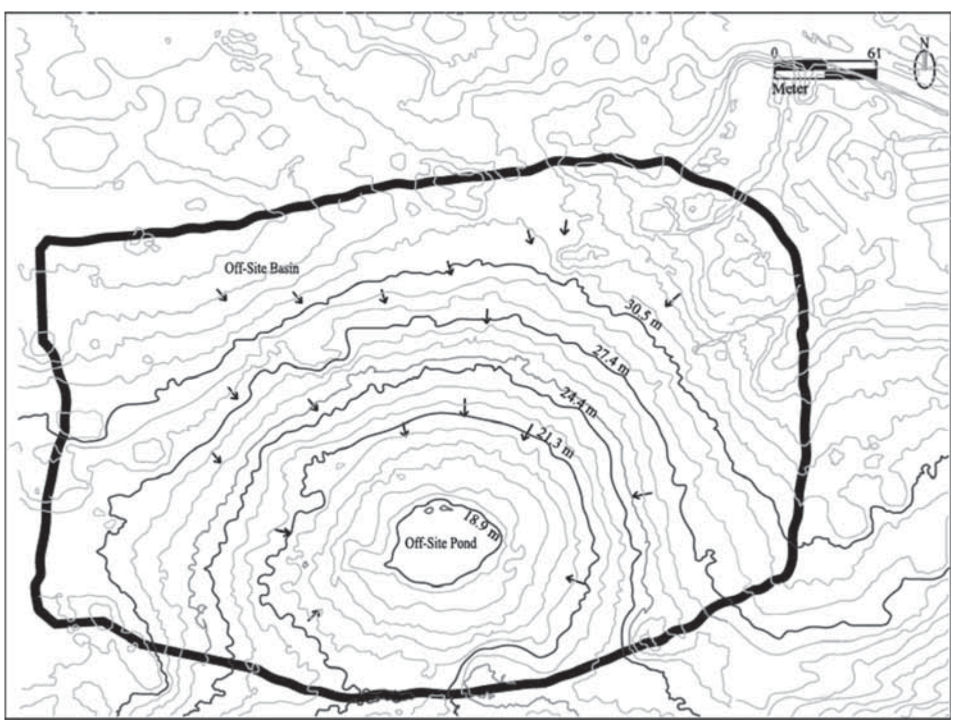

Figure 3: Off-site basin map remains same in pre- and post-development.

(Fig. 3). This basin had mainly grass all over it. Table 1 depicts the information of all pre-development basins. Based on the surface cover of all these basins, the Soil Conservation Service (SCS) curve numbers were assigned and listed in Table 1. At the same time, the time of concentration for each basin was calculated for using in the Interconnected Channel and Pond Routing (ICPR) model which is discussed later.

2.2.2 Post-development basins

With the objective to reduce off-site runoff towards the east and south, the project area was divided into three drainage basins (Fig. 2b) for post-development condition. The land-cover of these drainage basins was significantly different from its pre-development condition. Two basins, namely; Post-East Basin (0.8 hectare) and Post-South Basin (2.6 hectare) were developed to drain towards the east and south side of the property respectively. The surface cover of these basins changed from wooded-area to landscape and grass cover. Another 1.3-hectare post-impervious basin created at the centre of the project had mostly impervious area (1 hectare) due to the addition of new buildings, car-parking, access roads, and pools. This post-impervious basin also had some (0.3 hectare) landscape and parking medians. The off-site basin (Fig. 3) remained the same as it was during pre-development condition because it did not experience any change due to the development in the project area. The SCS curve numbers (combined) and time of concentrations for each basin was determined for post-development scenario and listed in Table 1.

2.3 Storm flow mitigation design

The stormwater management system for the post-developed site was designed to reduce off-site runoff to less than the pre-development levels for both east and south direction. Therefore, the size of post-developed basin areas (Post-East Basin and Post-South Basin) that drained towards the east and south were made smaller than the pre-developed basin areas (Pre-East Basin and Pre-South

Table 1: Pre- and post-development basin characteristics.

	Project Area					
	Pre-development basins		Post-development basins			
Parameters	Pre-East Basin	Pre-South Basin	Post-East Basin	Post-South Basin	Post-impervious Basin	Off-site Basin
Impervious Area, m^2 (hectare)	–	–	–	–	9,713 (1.0)	–
Pervious Area, m^2 (hectare)	17,401 (1.7)	28,732 (2.9)	8,094 (0.8)	25,900 (2.6)	2,833 (0.3)	169,563 (17.0)
Total, m^2 (hectare)	17,401 (1.7)	28,732 (2.9)	8,094 (0.8)	25,900 (2.6)	12,546 (1.3)	169,563 (17.0)
Curve Number (combined)	55.0	55.0	63.1	56.8	90.0	61.0
Time of concentration, t_c (min)	37.7	27.6	32.4	26.3	16.0	22.7

Table 2: The new-pond water-level elevations vs. storage area.

Elevation (m)	Area (m^2)	Cumulative volume (m^3)
25.9	445	0.0
26.2	526	150
26.5	648	333
26.8	769	552
27.1	931	810
27.4	1,052	1,019
27.7	1,214	1,453

Basin). The basin at the centre of the project area (post-impervious basin) was designed to have an underground runoff collection system (through underground pipe networks). The runoff from this basin was designed to be collected by inlets, carried through underground pipes and discharged into the newly designed dry-detention facility (new pond). This flow (from the post-impervious basin) was allowed to route through the new pond on the 4.6 hectare site. The new pond (Table 2) was designed to retain as much water as possible and it was also able to release excess volume safely to downstream to the off-site pond at the golf course. To achieve this, the outflow structure of the new pond was optimized for maximum storage (with 0.91 m freeboard). The top opening (0.61 m by 0.91 m) of the outflow structure was 0.67 m above the bottom of the pond (at an elevation of 26.58 m), which allowed the pond to have 444 m^3 treatment capacity, a requirement set by local regulatory agency. To convey the excess water safely to the off-site pond, a 46-cm diameter 410 m long underground pipe (slope 1.6%) was used to connect the outflow structure (of new pond) to the off-site pond (at golf course). This network allowed the two ponds to operate in series. A side bank filter (infiltration rate 1.43 m per hr) with top area 45.24 m^2 was also provided in the new pond.

2.4 Model development and runoff simulation

A dynamic rainfall-runoff simulation model was developed to design and evaluate the storm flow management facility discussed in the previous section. The ICPR (v3.10) program [12] that performs short-duration analysis was applied for this purpose. ICPR is a widely used program, popularly used in design and sizing of the detention ponds. Specific design storm (e.g. 25 yr 24 hr) events are commonly used to design the drainage facilities. Initially, the ICPR model calculates pre- and post-development runoff hydrograph using the pre- and post-development runoff parameters (such as basin areas, curve numbers, time of concentrations). The hydrographs are generated by SCS Unit Hydrograph method. The program then estimates the detention pond 'footprint' based on volume difference between pre- and post-development conditions. The size of the discharge structure is determined based on the pre- and post-development peak discharge rate. The program then tests the design through flow-simulation generated by chosen design-storms and refines the design. The iteration continues until a final design is achieved.

The runoff model developed by ICPR program was used to simulate the off-site runoff from the project area (towards the east and south) and to design the new pond. Besides these, the same model was also used to check the capacity of the off-site pond. The drainage areas, SCS curve numbers and time of concentrations (e.g. Table 1) for each basin allowed the model to simulate the runoff for different storm events. The Florida Department of Transportation rainfall distribution information

Table 3: Rainfall depth of design storms – input parameters for the ICPR model.

	10-yr Storm		25-yr Storm		50-yr Storm		100-yr Storm	
	8 hr	24 hr	8 hr	24 hr	8 hr	24 hr	8 hr	24 hr
Rainfall depth (cm)	15.8	19.1	18.8	22.1	20.3	24.4	22.6	29.2

Table 4: Land-use parameters.

Condition	Impervious Manning's roughness coefficient (weighted)	Pervious Manning's roughness coefficient (weighted)	Impervious depression storage (weighted) (cm)	Pervious depression storage (weighted) (cm)
Pre	0.025	0.40	0.51	0.89
Post	0.022	0.365	0.45	0.77

Table 5: Off-site pond water level elevations vs. storage.

Elevation (m)	Area (m^2)	Cumulative volume (m^3)
18.3	2,165	0.0
18.9	3,307	1,668
19.5	4,608	4,080
20.1	8,213	7,988
20.7	13,285	14,541
21.3	18,488	24,225
22.0	25,345	37,585
22.6	33,262	55,449

(IDF curves) [13] was used to simulate the runoff. Simulations were run for design storms of four return periods. The 25-yr and 100-yr design storms were run to evaluate the design to meet the local design criteria and extreme flow condition respectively. Besides these, a relatively small storm of 5-yr return period and an intermediate storm of 50-yr return period were used to simulate the runoff to observe the trend and performance of the design. The rainfall depths for different deign storms used in the model are shown in Table 3.

Besides the design and capacity-verification of the off-site pond using design storms (applying the ICPR model), an extended simulation was conducted with an existing historical extreme rainfall information. The purpose of this simulation was to get an idea if the off-site pond can withstand and retain runoff (including the additional runoff due to site-development) of an extreme wet year similar to 1994, which occurred in Tallahassee, Florida. The extended simulation was conducted using a locally developed excel program that has the in-built rainfall data for an entire wet year of 1994 of Tallahassee, Florida. This program, CBasin (v1.0) [14], uses hydrologic input parameters (basin areas, percent impervious, terrain slope, depression storage, Manning's roughness coefficient), Green-Ampt equation parameters (hydraulic conductivity, initial moister deficit, soil suction coefficient), retention pond information (elevation, area, volume) and simulate the runoff to determine water level in the retention pond.

Runoff from two basins (off-site basin and post-impervious basin) that drains into the off-site pond at golf course (closed basin) was simulated for the entire wet year of 1994. The hydrologic input parameters (Tables 1 and 4) allowed the model to calculate the runoff volume throughout the year. Incorporating off-site pond information (Table 5) allowed the model to show water level elevations during the extended simulation period. The model also used terrain slope of 5.2% for off-site basin and 1.7% for post-impervious basin. The model used an initial moister deficit 0.3, capillary suction 15.2 cm and hydraulic conductivity 1.5 cm/hr. It may be mentioned that during 1994, three major storm events dropped over 15.24 cm of rain in 24 – 48-hr periods. In that same year, there was a fourth storm of lesser magnitude (7.62 inches in 5 hr) [15].

3 RESULTS AND DISCUSSIONS

3.1 Short duration simulation

3.1.1 Off-site runoff reduction towards east and south

As the first step to evaluate the performance of the storm flow mitigation design, the runoff towards the east and south side from the project area was determined. The runoff volume and peak discharge from the pre- and post-development basins were calculated using the model developed by ICPR program. The numeric values depicted in Tables 1 and 3 were used for this purpose.

Figure 4 compares the pre- and post-development runoff results. As seen from the figure, the simulated runoff (rate and volume) towards east and south from the post-developed basins was less than the pre-developed condition for 5-yr, 25-yr, 50-yr and 100-yr storms (8 hr and 24 hr duration).

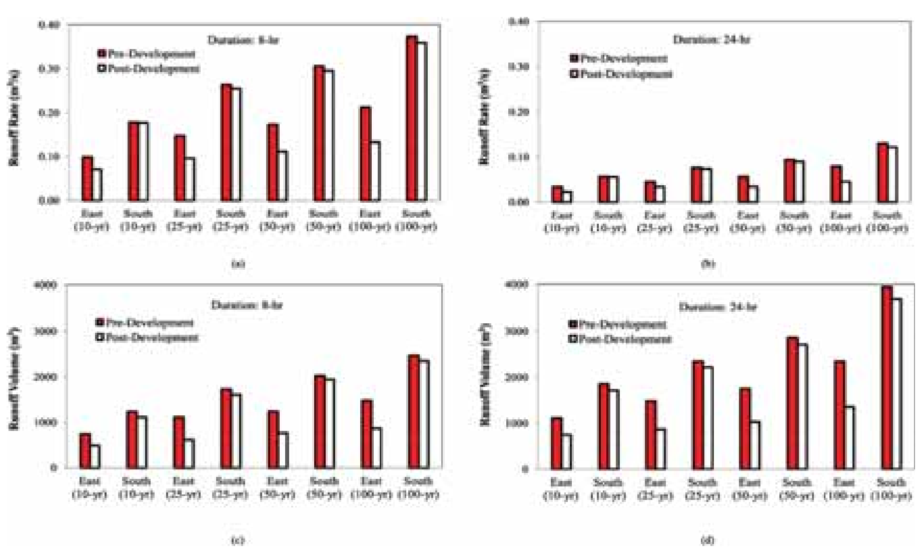

Figure 4: Pre- and post-development runoff from project area. (a) Peak rate, 8-hr storm, (b) Peak rate, 24-hr storm, (c) Runoff volume, 8-hr storm, (d) Runoff volume, 24-hr storm.

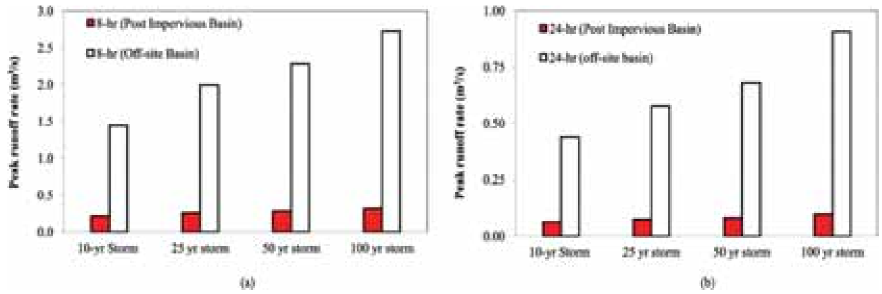

Figure 5: Peak runoff rate from post-impervious basin and off-site basin, (a) 8-hr storm, (b) 24-hr storm.

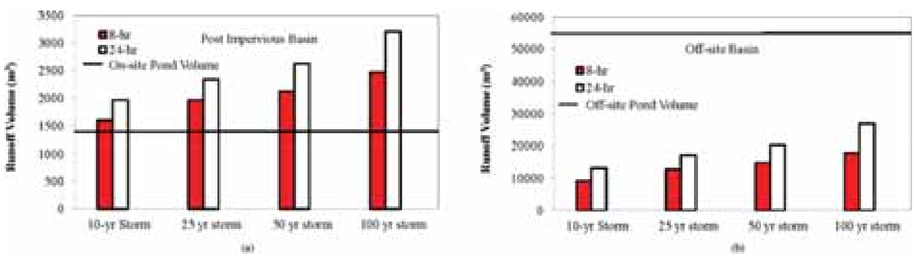

Figure 6: Runoff volume, (a) runoff volume from post-impervious basin, (b) runoff volume from off-site basin.

Though the surface cover and topography were changed due to the site development, the newly designed basins reduced the runoff from its pre-development level. For instance, for the local design criteria of 25-yr design storm, peak flow towards east decreased from 0.15 m^3/s to 0.10 m^3/s (25 yr 8 hr). The volume of runoff towards east also decreased from 1,480 m^3 to 863 m^3 (25 yr 24 hr). On the other hand, in pre-development the peak flow towards the south was 0.26 m^3/s (from Pre-South Basin), while in post-development condition, peak flow towards the south was reduced to 0.25 m^3/s (from Post-South Basin) for 25-yr 8-hr storm. The volume of runoff towards the south was also decreased from 2,344 m^3 to 2,220 m^3 (25 yr 24 hr).

3.1.2 Excess runoff retention within close-basin

As the second step to evaluate the storm flow management design, runoff to new pond and off-site pond (generated from post-impervious basin and off-site basin) was determined. These runoff volumes were determined using the model developed by ICPR program.

The simulated peak discharge and runoff volume from the post-impervious basin and off-site basin are presented in Figs 5 and 6, respectively. As mentioned earlier, the runoff generated in post-impervious basin was routed through the new pond and finally discharged to the off-site pond. Besides this flow, the off-site pond also receives runoff from the off-site basin. From Fig. 6a, it is evident that the new pond was designed much smaller than the runoff volume generated from the post-impervious basin. For instance, the runoff volume generated due to 25-yr 24-hr storm (local design criteria) was found to be 2,344 m^3, which is higher than the total volume of 1453 m^3 (Table 2) of the new pond (without considering required 0.91 m freeboard) and the treatment volume of

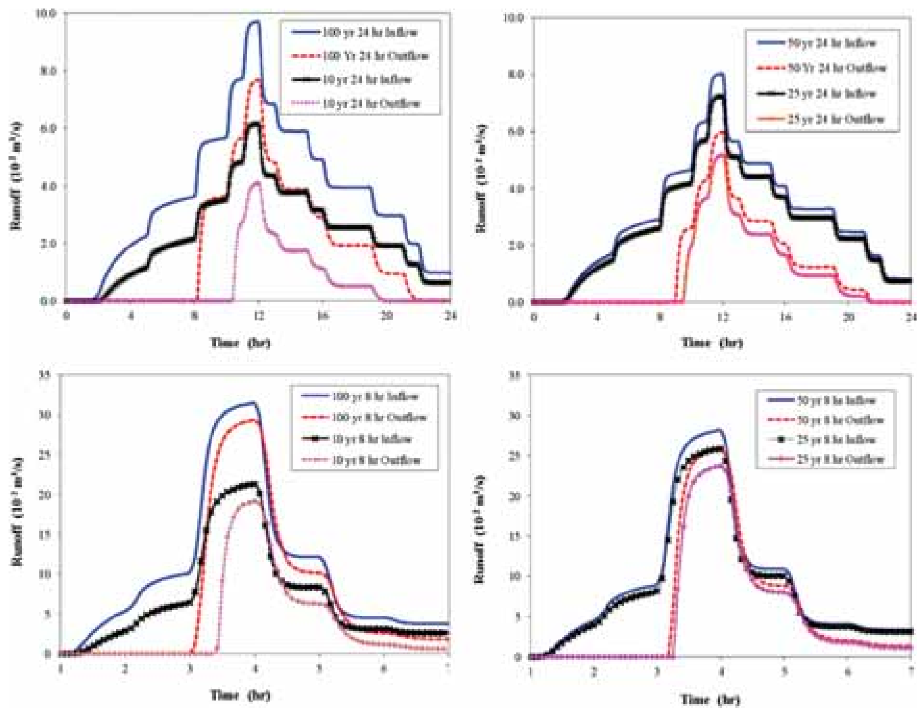

Figure 7: Inflow and outflow hydrographs – new pond.

493 m^3 (up to the top opening of the discharge structure). This excess runoff volume (e.g. 891 m^3 of 25-yr 24-hr storm) needed to be managed and retained within the property of the owner. It can be seen that if this runoff (25-yr 24-hr storm) is retained within the project area, the new pond needs to be 1.6 times bigger (even with no freeboard) than what was designed. This would not only increase cost but also required more land space.

To save the area of the project site for future use (for future tennis complex), the new-pond size was minimized and the available volume of the off-site was utilized. Figure 6b presents the runoff volume from the off-site basin to the off-site pond. This figure also shows the available volume in the off-site pond. For example, for the design storm 25-yr 24-hr storm (local design criteria) the runoff volume to the off-site pond was 17,145 m^3 (total volume of the off-site pond was 55,449 m^3). So, a careful analysis is performed with the simulated runoff from off-site basin and the attenuated discharge from the new pond (Fig. 7) to effectively utilize the available storage volume in the off-site pond, which must retain all runoff.

3.1.2.1 The new-pond water level

The water levels of the new pond due to the simulated runoff from post-impervious basin are presented in Fig. 8 for different storm events. The numeric values of Tables 1–3 were used to run the model and determine these water surface elevations. It is clear that the new pond can route the flow (Fig. 7) and safely pass (Fig. 8) runoff to the off-site pond (through the 46-cm diameter 410 m long pipe with 1.6% slope). The pipe that connected the discharge structure of new pond and the off-site pond was used to convey excess runoff from the new pond to efficiently utilize the available storage in the off-site pond. The maximum water level in this new pond reached at 26.71 m for 25- yr 8-hr

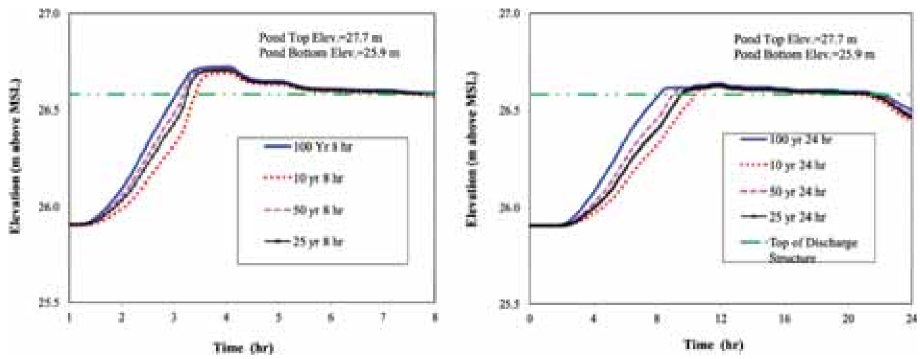

Figure 8: Water levels in the new pond.

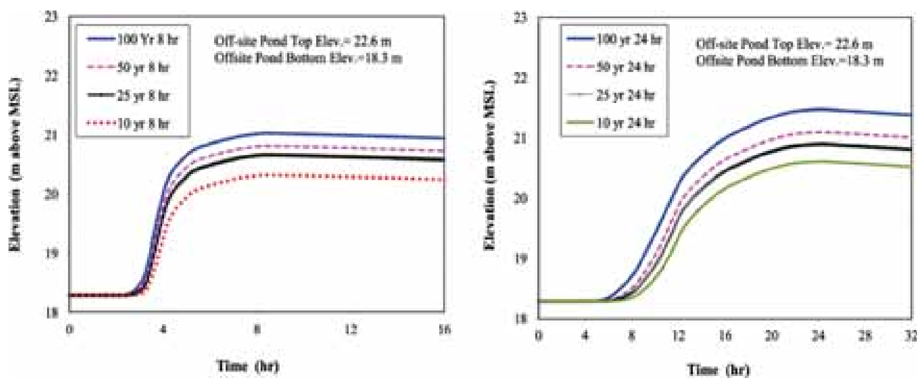

Figure 9: Water levels in the off-site pond (short duration simulation).

storm (local design criteria) and the maximum level for extreme storm event (100 yr 8 hr) was determined to be 26.73 m. The top elevation of the new pond is 27.7 m and for all storms and the new pond maintains a minimum safe freeboard of 0.9 m (Fig. 8), a requirement for new pond construction in the area.

3.1.2.2 Excess runoff accommodation in the off-site pond

Since the off-site pond is located in a closed basin, it was of prime importance to verify that the pond had enough capacity to retain all runoff. For this reason, the performance of the off-site pond was evaluated for short duration simulation as well as extended simulation. Figure 9 presents the results of water surface elevations (for post-development condition) during the short duration simulation using ICPR program. The numeric values of Tables 1–3 and 5 were used to simulate the runoff and determine the water levels. The pond percolation rate, obtained from an on-site test, was considered 1 inch per day. Maximum water levels computed for 25-yr and 100-yr storm were 20.9 m (25 yr 24 hr) and 21.5 m (100 yr 24 hr) respectively, meeting freeboard requirement of local regulation for an existing pond. The short duration simulation showed that the off-site pond (top elevation 22.6 m) can also retain runoff of all storm events including the extreme storm event of 100-yr 24-hr storm (Fig. 9).

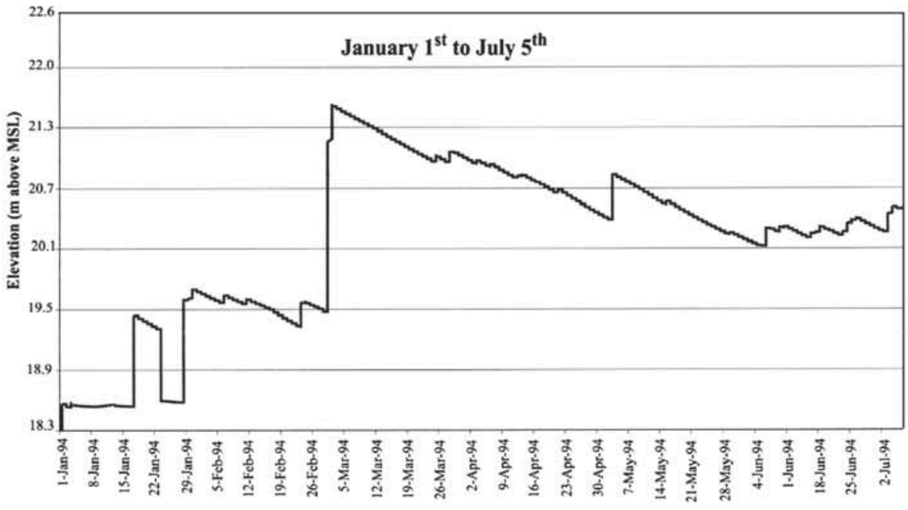

Figure 10: Water levels in the off-site pond (from January 1st to July 5th, 1994).

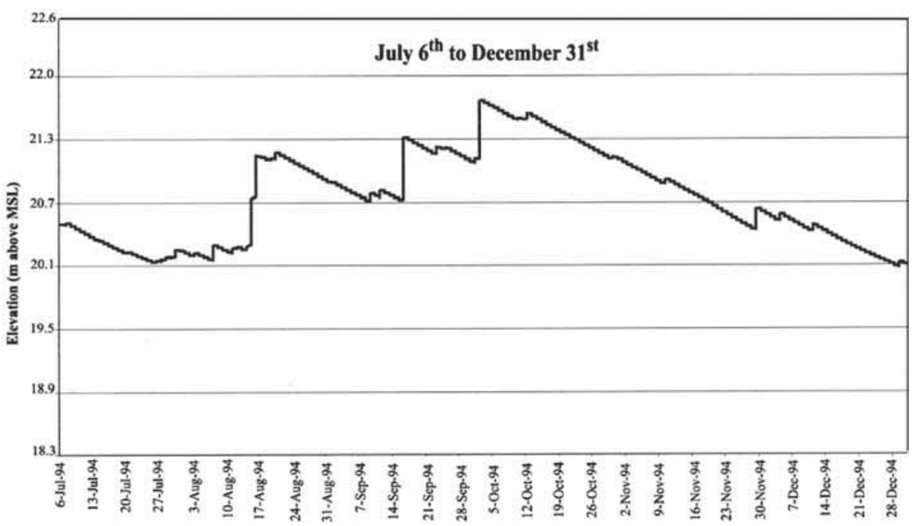

Figure 11: Water levels in the off-site pond (from July 6st to December 31st, 1994).

3.2 Extended simulation

An extended simulation of runoff was performed to re-confirm that the off-site pond had the capacity to accommodate runoff similar to an historical entire wet year occurred in Tallahassee, Florida in 1994. A stormwater management model was developed using the CBasin (v1.0) program [14] for this purpose. The model simulated the runoff to the off-site pond from January 1st to December 31st of 1994. Figure 10 shows the water level in the off-site pond from January 1st to July 5th and Fig. 11 shows the water level from July 6st to December 31st. The maximum stage of 21.7 m was observed on October 4th, 1994 (Fig. 11). The water level during this extreme wet condition was below the top of the off-site pond confirming the pond would safely retain runoff of the extreme wet year. Besides the maximum stage of October, 1994, the off-site pond showed high water levels at the beginning of March (Fig. 10) and August 1994 (Fig. 11), indicating the effect of heavy rainfall that occurred in the area as mentioned by Singhofen [15].

4 CONCLUSION

As construction cost of large pond in a land development project is a major cost, an effective stormwater management design was developed to minimize the on-site pond size. This paper presents the details of the mitigation design where the size of the on-site detention pond was minimized on an 4.6 hectare site, which had undergone development. The land surface of the site changed from a wooded area to a swimming pool complex. To mitigate the runoff and reduce the on-site pond size, the stormwater management strategy employed: (1) releasing runoff below its pre-development level in the direction where it was allowed to drain (east and south side of the property), (2) capturing the excess runoff by two ponds in series; an on-site new pond and an off-site pond. The size of the on-site pond was minimized by maximum utilization of the available storage in the downstream off-site pond.

The performance of the mitigation design was evaluated using a short duration simulation model and an extended simulation model. The analysis focused on both peak flows discharged out of the property (towards east and south) as well as volumetric flows to be captured within the basin with the two ponds. The short duration simulations used 8-hr and 24-hr design storms of 10-yr, 25-yr, 50-yr and 100-yr return period. The results showed that the post-development runoff towards the east and south side of project area was significantly reduced from its pre-development level due to re-sizing and re-arranging the basins. The simulated runoff results from post-impervious basin confirmed the minimization of the size of the on-site new pond. It was also evident that the new pond would route and safely pass the excess runoff to effectively utilize the extra volume available in the off-site pond. The maximum utilization of the off-site pond was achieved, which in turn resulted in reduction of the new-pond size. Besides the short duration simulation, the extended simulation (for entire 1 yr) was run for an extreme wet year (1994) in the region. The results of extended simulation confirmed that both the on-site new pond and off-site pond can safely retain all runoff of the extreme wet year with adequate freeboard. Both the short term and long term calculations revealed that the stormwater management strategy employed in this complex urban setting is sufficient to manage the extreme runoff in a reliable manner.

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