

Faculty of Engineering

Modelling of Urban Stormwater Drain with Orifice Flow in Series using SWMM

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Modelling of Urban Stormwater Drain with Orifice Flow in Series using SWMM

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DECLARATION

I declare that the work in this thesis was carried out in accordance with the regulations of Universiti Malaysia Sarawak. Except where due acknowledgements have been made, the work is that of the author alone. The thesis has not been accepted for any degree and is not concurrently submitted in candidature of any other degree.



……

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ABSTRACT

Stormwater infrastructure requires modification to solve the urban flooding problem due to increasing stormwater runoff rate and volume in the urban areas. This study investigated an urban drain (0.5 m W x 0.5 m D x 170 m L) divided into compartments with orifice flow restrictors to detain stormwater, subjecting the drain to three drain slopes of 0%, 0.2%, and 0.4%. The orifices had diameters ranging from 0.1 m to 0.5 m. A design rainfall of 5 minutes with a 10-year ARI intensity and three historical rainfalls were selected for stormwater flow analyses. Two rows of 12-unit terrace houses with 4,227 m² land area were selected as the study area. Three interventions were designed, namely: Drain with Single-Orifice (Scenario 1), Double-Orifice (Scenario 2), and Triple-Orifice (Scenario 3). The research utilized SWMM version 5.0 to effectively simulate the unique characteristics of the three interventions, creating sub-models that allowed for a detailed evaluation of the flow conditions in the study area and the impacts of different slopes, orifice sizes, numbers of orifice at varied locations. Overall, the 0.2% slope for design rainfall had the highest peak flow at the downstream end, while the 0% and 0.4% slopes peaked at the upstream end; for historical rainfall, the highest peak flow occurred at the downstream end for the 0% and 0.4% slopes, with the 0.4% slope also showing the highest peak flow upstream. However, the peak water level for the 0.1 m diameter result was the highest and decreased with increasing orifice diameter from 0.2 m to 0.5 m. The results indicated that Scenario 3, featuring diameters from 0.3 m to 0.5 m at a 0.2% slope, demonstrated the best performance with consistent reductions in peak flow and water level hydrographs, and no overflow was predicted for this scenario.

Keywords: Orifice, stormwater, sustainable development, SWMM, urban runoff

Pemodelan Saluran Air Hujan Bandar yang mengalir Melalui Orifis dalam Siri dengan Pelbagai Aliran Lateral

ABSTRAK

Infrastruktur saliran air ribut memerlukan pengubahsuaian untuk menyelesaikan masalah banjir bandar yang berpunca daripada peningkatan kadar dan jumlah aliran air ribut di kawasan bandar. Kajian ini meneliti sebuah saliran bandar (0.5 m L x 0.5 m T x 170 m P) yang dibahagikan kepada petak-petak dengan penyekat aliran orifis untuk menahan air ribut, dan mengenakan saliran tersebut kepada tiga cerun iaitu 0%, 0.2%, dan 0.4%. Orifis yang digunakan mempunyai diameter antara 0.1 m hingga 0.5 m. Hujan reka bentuk selama 5 minit dengan intensiti ARI 10 tahun dan tiga kejadian hujan sejarah dipilih untuk analisis aliran air ribut. Dua barisan rumah teres 12-unit dengan keluasan tanah 4,227 m² dipilih sebagai kawasan kajian. Tiga intervensi direka, iaitu: Saliran dengan Orifis Tunggal (Senario 1), Orifis Berganda (Senario 2), dan Orifis Tiga (Senario 3). Penyelidikan ini menggunakan SWMM versi 5.0 untuk mensimulasikan ciri-ciri unik ketiga-tiga intervensi, menghasilkan sub-model yang membolehkan penilaian terperinci terhadap keadaan aliran di kawasan kajian serta kesan cerun, saiz orifis, dan bilangan orifis di lokasi-lokasi yang berbeza. Secara keseluruhannya, cerun 0.2% untuk hujan reka bentuk menunjukkan aliran puncak tertinggi di bahagian hilir, manakala cerun 0% dan 0.4% mencatatkan puncak aliran di bahagian hulu; bagi hujan sejarah, aliran puncak tertinggi berlaku di bahagian hilir untuk cerun 0% dan 0.4%, dengan cerun 0.4% juga mencatatkan aliran puncak tertinggi di bahagian hulu. Walau bagaimanapun, paras air puncak bagi diameter 0.1 m adalah yang tertinggi dan menurun dengan peningkatan diameter orifis daripada 0.2 m kepada 0.5 m. Hasil kajian menunjukkan bahawa Senario 3, yang menampilkan diameter antara 0.3 m hingga 0.5 m pada cerun 0.2%, menunjukkan prestasi terbaik dengan

pengurangan konsisten dalam aliran puncak dan graf paras air, serta tiada limpahan diramalkan untuk senario ini.

Kata kunci: *Orifis, air ribut, pembangunan lestari, SWMM, aliran air bandar*

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LIST OF ABBREVIATIONS

1D	One Dimensional
2D	Two Dimensional
ARI	Average Recurrent Interval
BMP	Best Management Practice
DID	Department of Irrigation and Drainage
IDF	Intensity-Duration-Frequency
IL	Invert Level
LID	Low Impact Development
MSMA	*Manual Saliran Mesra Alam*
SuDS	Sustainable Drainage Systems
SWMM	Storm Water Management Model
USBR	United States Burau of Reclamation

CHAPTER 1

INTRODUCTION

1.1 Background

Stormwater management challenges have recently gained a lot of attention from both the public and professionals in the industry (Faram et al., 2010). Urban flooding appears to have become more frequent, and as a result, significant amounts of attention are being paid to this in the media, exposing the public to the issues experienced more directly. Unplanned urban growth and development interfere with the natural flow of water and hence usually increases the risk of urban flooding.

Overflowing stormwater drains are one of the causes of flooding in most urban areas, in which it has been found that the stormwater drainage problem occurs due to a combination of urban sprawl, structural, hydrological and climatic factors (Pervin et al., 2019). In the existing urban stormwater drainage design, for example, consider a row of terrace houses as shown in Figure 1.1, an urban drain is meant to receive water from each of the houses. As such, there are multiple sources of water which are termed lateral inflows. The drain is designed with a slope and water storage capacity to carry the accumulated volume of stormwater from the connected houses. Stormwater runoff propagates along the length of the urban drainage system, with flow congestion frequently occurring at the downstream end during storm events. Bisht et al. (2016) utilized hydrodynamic modeling with SWMM and MIKE URBAN to demonstrate that urban drainage networks are susceptible to downstream bottlenecks due to increased surface runoff and limited hydraulic capacity, exacerbating flood risks in urban areas. Based on the figure below, more water is accumulated at Point 2 (downstream end) than Point 1 (upstream end). As

such, urban flooding is more likely at the downstream end of a drainage system. However, such a drain is rarely equipped with flow control mechanisms. Computer modelling of a drain with intended flow control mechanisms could provide insights into possibilities for improving the flow impediment in the drain mentioned above.

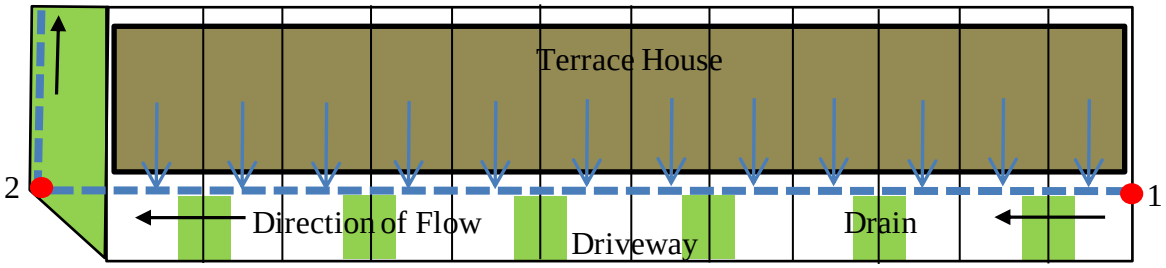


Figure 1.1: Urban Drain for Terrace Houses

In practice, flow control devices such as simple orifice plates, float controlled valves, and vortex valves are employed (Faram et al., 2010). Orifices are the most used form of differential flowmeters for measurements of single-phase and multiphase flows. From Tang et al. (2016), an orifice is an opening that allows a fluid to flow out; however, the size of the opening is limiting the flow discharge. Generally, the orifice is durable, cost-effective, easy to install and repair, simple to design, long-lasting, and free from wear-out failure (Araoye et al., 2017, Gao and Wu, 2019, Düz, 2021).

1.2 Significance of Study

Inserting orifices in series shall be able to distribute the stormwater flow over the length of a long drain. Depicted in Figure 1.2, the flow restrictors appear to be obstacles placed at strategic locations (which are subjected to investigation in this study), thus, are a good way for semi-on-site-detention. Stormwater shall be detained at designated chambers at the upstream and middle stretches, avoiding water congestion at the downstream end.

By implementing flow control with the orifices, peak flow rates can be reduced (Brandes and Barlow, 2012). As such, urban flooding could be lessened in the urban drains.

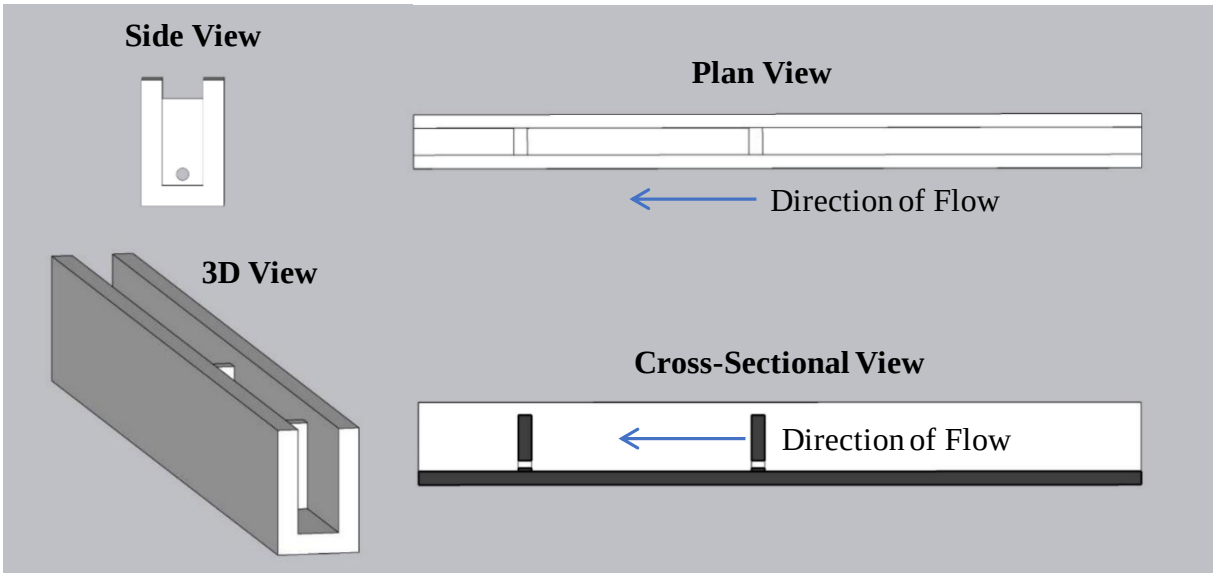


Figure 1.2: Orifices in Series along Urban Drain

1.3 Problem Statement

Current urban drainage systems are predominantly based on gravity-driven open channel flow and are typically designed using empirical equations such as Manning's formula. However, these systems generally lack localized flow control mechanisms, making them less effective in managing runoff during intense or short-duration rainfall events. Without the ability to regulate flow at specific points within the network, existing systems are prone to overloading, backflow, and surface flooding, particularly in densely developed urban areas. Moreover, rapid urbanization has significantly increased the extent of impervious surfaces, accelerating both runoff volume and peak discharge rates. These challenges expose the limitations of conventional drainage designs in adapting to dynamic hydrological conditions. Therefore, there is a pressing need for innovative flow control

strategies such as the strategic implementation of orifice restrictors to enhance the resilience and performance of urban drainage networks under varying flow regimes.

Orifice flow restrictors in an urban drain are uncommon in practices. However, the theories of open channel and orifice are long standing text-book materials. Referring to a simple fundamental theory of flow in the drain using Manning's formula, flow discharge is influenced by a conduit geometry, material and its associated drain slope. On the other hand, flow discharge through the orifice is influenced by the orifice geometry and water head acting on the orifice. Putting the open channel and orifice together, the water head much needed by the orifice (namely the range of water heads limits the flow rates out of the orifice) and its relationships with the drain slope are sparse in the literatures. Logically, the steeper the slope, the higher water head before the orifice. These and its resultant flow patterns are unknown. As such, there is a need to characterize the flow patterns due to orifice restrictions and drains slopes.

Continuing from the fact of water head before the orifice, usually high-water head is accompanied by the occurrences of high-water level. Sadly, a higher water level coupled with a steeper drain slope, may accelerate flow velocity and induce overflow more rapidly. As such, the sizes of the orifices play important role to have in-time flushing to avoid overflowing. On top of that, the number of orifices and their locations also have roles in the drain flow. As such, three research questions below are formulated:

a) What are the interactions of drains slopes with orifices in conveying flow in the urban drain?

b) What are the sizes of orifice restrictors when facing fast and rising water head before the orifice to avoid overflowing?

c) What are the number and location of orifices being inserted in the urban drain best to mitigate urban flooding?

1.4 Aim and Objectives

This study aims to investigate the flow patterns in the urban stormwater drain with orifice flow restrictors. A computer modelling by means of using Storm Water Management Model (SWMM) version 5 is utilized as the main tool to assist in the investigation. A case study of stormwater drainage system is explored in terms of:

Objective 1: To evaluate hydrological responses in the urban drain under study with and without orifice flow restrictors in response to drain slopes;

Objective 2: To evaluate the performances of orifice flow restrictors in regulating drain water levels in terms of the orifice sizes;

Objective 3: To deduce the numbers of orifices and their locations for flood mitigation purposes.

1.5 Scopes of Work

The scopes of this study include the followings:

a. Climate conditions are based on Kuching and Samarahan, Sarawak;

b. Chosen land use type is densely populated residential area;

c. Selected historical extreme rainfall events occurred in Kuching and Samarahan;

d. Design rainfall for Kuching and Samarahan is based on MSMA 1st Edition;

e. Urban drainage design is based on MSMA 2nd Edition, in which minor drainage system for residential area is designed to 10-year Average Recurrent Interval;

f. Urban drainage simulation is conducted using SWMM version 5.0, which is based on system characteristics and not physically constructed.;

g. The model results are verified by hydrological and hydraulic principles.

1.6 Structure of Thesis

The thesis is divided into five chapters. The first chapter of this thesis is an introduction to the study. It consists of the general view of the topic that illustrates the background and the significance of the study, a problem statement, aim and objectives, and scopes of the study.

The second chapter is literature review which gives reviews from the literature on the flood, hydrological and hydraulic parameters, and stormwater infrastructure, with emphasis on flood characteristics and records, hydrological and hydraulic design considerations, and the applicability of the proposed design of the compartmentalized drain. This chapter also involves the SWMM modelling of stormwater drainage system.

The third chapter discusses the methodology and procedures used in order to achieve the objectives of this study. This chapter explains the method adopted for the research, which include SWMM modelling to assess the hydrological and hydraulic aspects of compartmentalized drain.

The fourth chapter presents the results obtained from the usage of methods adopted in Chapter 3. The results are elaborated and analysed to investigate the implementation of

different orifice sizes and numbers into open channels with varying slopes. Tables and graphs are presented and analyzed to investigate the objectives of the study.

The last chapter concludes the main findings of the study and presents recommendations of the future works.

CHAPTER 2

LITERATURE REVIEW

2.1 Introduction

Stormwater refers to rainwater that may carry pollutants, infiltrate the ground, accumulate on the surface, or flow as runoff. However, urbanization has altered the natural water balance and hydrological cycle, making it difficult for stormwater to infiltrate the ground in urban areas due to impervious surfaces, which resulting in flooding and water pollution. According to Jefferson et al. (2017) and Ronalds and Zhang (2019), the increase in impervious surfaces due to urbanization has resulted in a significant increase in stormwater runoff. The increasing population and physical developments in cities have caused impermeable surfaces to limit the access of water to soil and plants, and instead quickly convey runoff into pipelines and channels. According to the Australian Environment, Communications, Information Technology, and the Arts Legislation Committee (2002), up to 90% of rainfall can be absorbed and infiltrated in an undeveloped catchment, while only 10% of precipitation can be infiltrated in a developed catchment or built environment (see Figure 2.1). In short, the urbanization process has changed the hydrological cycle and natural water balance, leading to an increase in stormwater runoff due to the presence of impervious surfaces.

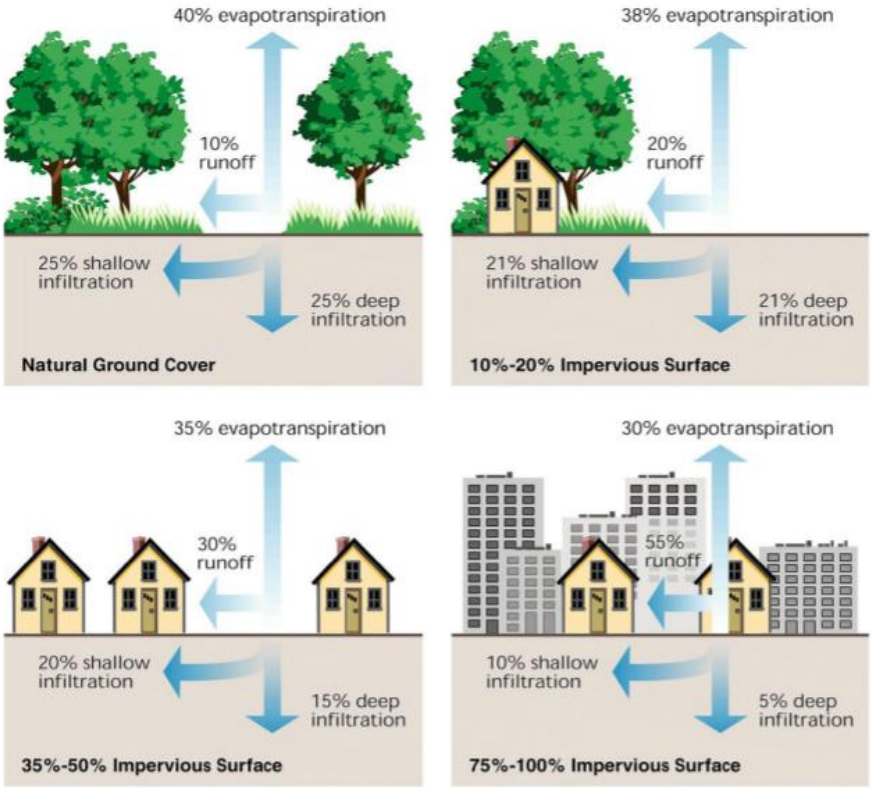


Figure 2.1: Impact of Urbanisation on Infiltration and Runoff (Ishimatsu et al., 2016)

Urban development profoundly alters natural hydrological patterns, often resulting in increased surface runoff and more frequent urban flooding. Wang et al. (2020), in a study of Zhengzhou, China, demonstrated that rapid urban expansion particularly the spread of impervious surfaces led to significant changes in runoff behaviour. From 1999 to 2012, the city experienced flood peaks that arrived 25% to 30% earlier and saw runoff coefficients increase by up to 35%. These shifts were attributed to decreased infiltration and a growing concentration of rainfall over built-up areas, underscoring the strong connection between urbanization and heightened flood risk. Urban environments become congested, especially in residential and commercial areas, due to human activities and changes in land use. Haris et al. (2016) described the congestion of stormwater drainage systems as one of the impacts of urbanization, which leads to flash flood problems and water quality degradation. Extreme rainfall, one of the impacts of the changes in the hydrological cycle, especially during

monsoon season triggers flash flooding in the urban area and converts the precipitation to stormwater runoff. Surface water flooding in urban areas has become a crucial concern due to changing precipitation patterns, expanding urban areas and aging drainage infrastructure (Webber et al., 2018). Peak runoff values continue to rise even as partially developed catchments experience intensifying urbanization (Muñoz et al., 2017). Flooding, erosion, channel widening, sediment deposition, increased pollutant delivery to streams, and other ecological problems have been linked to increased peak runoff (Berland et al., 2017).

2.2 Stormwater Management

Stormwater management is widely practiced in many countries, such as the USA and the UK, particularly in urban drainage systems (Lai et al., 2015). According to Manual Saliran Mesra Alam (MSMA) or Urban Stormwater Management Manual for Malaysia instituted by the Department of Irrigation and Drainage, stormwater management is a mechanism for controlling stormwater runoff, minimizing catchment discharges, runoff volumes, and flood frequencies. This involves implementing erosion and sediment control, quantity control, and treating BMPs during construction to reduce catchment flow rates, runoff volumes, and flooding frequency, while also improving surface water quality. The primary objective is to mitigate the environmental effects of land use changes through effective stormwater management practices. It involves regulating both the quality and quantity of stormwater through structural and operational measures. Integrated water management is a critical component of urban infrastructure (Teshome and Devi, 2020).

Urban drainage systems pose challenges for cities worldwide, and these challenges intensify as urban populations rise (van Leeuwen et al., 2019; Hernández-Hernández et al., 2020). Increased precipitation intensity due to climate change, which is occurring in many

places, exacerbates the difficulty of stormwater management. Therefore, evaluating stormwater management performance is essential to maintaining urban resilience against flood risks (Zhang et al., 2020). Stormwater management looks at how to best control these flowing waters from source to receiving water bodies utilizing various management strategies.

Drainage means the removal of excess water from a given place such as the ground surface, root zone, etc. Urban stormwater drainage is the ranging from small to large-scale drainage with the objective of draining surplus water from urban areas using open drains, culverts, and other stormwater infrastructure. When urban development takes place in a natural catchment, the ordered system of natural drainage lines is replaced by a system of roof and property drains, inlets, swales, pipes, and channels (DID, 2012). Stormwater drainage issues are widely recognized as a major cause of urban flooding, particularly in rapidly growing cities. Pervin et al. (2019) highlighted that flooding in South Asian cities such as Sylhet (Bangladesh) and Bharatpur (Nepal) is driven by a combination of urban sprawl, inadequate drainage infrastructure, poor solid waste management, changing rainfall patterns due to climate change, and socioeconomic pressures. Unplanned urban development leads to increased impervious surfaces and runoff, while solid waste dumped into canals significantly reduces their capacity. Climate projections suggest that short-duration, high-intensity rainfall events will intensify, exacerbating flood risks. The study found that even with structural improvements, failure to manage solid waste can negate long-term flood mitigation efforts, underscoring the need for integrated drainage and waste management strategies.

Rapid urbanization and global climate change could cause a substantial increase in the frequency and magnitude of urban flooding in many regions of the world. Therefore, flooding occurrences in urban settings are one of the most serious concerns with urban drainage systems. These occurrences can be caused by a variety of factors, including pipe size, structural failures in the system, objects producing blockages, or an increase in rainfall intensity due to climate change, which causes higher runoff flow. Climate change influences the strength and frequency of current weather phenomena, including a progressive increase in rainfall intensities in many places throughout the world, causing previously well-designed systems to fail (Ngamalieu-Nengoue et al., 2019). As the cost and time needed for restoration and installation of conventional drainage systems are tremendous, expanding these systems as a countermeasure to climate change and urbanization may not meet the criteria of sustainability (Hellstrom et al., 2000).

In response to increasing flood risks associated with urbanization and climate change, sustainable drainage systems (SuDS) particularly Low Impact Development (LID) strategies have gained significant attention for their cost-effective and environmentally sustainable approach to stormwater management. These systems replicate natural hydrological processes to promote infiltration and reduce surface runoff. Feng et al. (2020) evaluated the hydrological performance of LID interventions in two urban districts of Daejeon, South Korea, using the Storm Water Management Model (SWMM) to simulate pre-development, post-development, and LID-enhanced scenarios under various rainfall events. Their results indicated that urbanization substantially increased runoff volumes, with post-development runoff exceeding 80% of total rainfall in some instances. However, the application of LID measures, including pervious pavements and rain gardens, significantly

improved infiltration and reduced peak runoff. In some configurations, infiltration was restored to over 50%, closely approximating pre-development conditions.

Similarly, Zhang et al. (2021) assessed the effectiveness of Sponge City stormwater strategies in a high-density, aging residential area in Suzhou, China. By integrating LID techniques such as bio-retention cells, permeable pavements, grassed pitches, and stormwater gardens and simulating system performance using SWMM, they achieved a 91% annual runoff capture rate, surpassing the national target of 80%, and reduced suspended solids by 56%. Furthermore, the interventions effectively mitigated flooding by reducing peak runoff and the duration of surface water accumulation during storm events with up to 20-year recurrence intervals. Collectively, these studies highlight the critical role of LID-based SuDS in enhancing urban flood resilience, improving water quality, and restoring ecological function, reinforcing the need to embed sustainable stormwater management strategies into contemporary urban planning.

2.3 Urban Drain Design

An effective urban drainage design must ensure not only adequate capacity but also resilience to clogging, which significantly reduces system performance and increases flood risk. Wang et al. (2025) used a coupled 1D and 2D model in Wuhan's Qingshan District to show that clogging of street inlets and pipes leads to higher inundation depths and larger flooded areas. For example, 50% clogging increased flood depths in over 14% of the area, and pipe overflow volumes rose up to six times under severe clogging scenarios. Even partial blockages in non-critical areas caused noticeable impacts, emphasizing that proper design must be paired with regular maintenance. These findings highlight the importance of

designing drainage systems that are both efficient and maintainable to effectively reduce urban flood hazards.

Urban drainage systems are typically designed as dual systems, comprising a minor system and a major system, to effectively manage stormwater under varying rainfall intensities. This dual approach ensures that frequent, low-intensity rainfall events are handled efficiently while also providing capacity for extreme, high-intensity events that could otherwise lead to catastrophic flooding. The minor system includes components such as street gutters, stormwater pipes, culverts, and catch basins, and is designed to manage routine rainfall events with return periods typically ranging from 2 to 10 years. Its primary purpose is to prevent nuisance flooding and maintain normal urban functions during moderate rainfall. However, the minor system has limited capacity and can be overwhelmed during extreme rainfall events, leading to localized flooding. In contrast, the major system serves as a secondary drainage network designed to handle extreme rainfall events that exceed the capacity of the minor system. It comprises overland flow paths, natural waterways, floodplains, retention basins, and even streets that temporarily function as flow channels during major storms. The major system is designed for larger return periods, typically 50 to 100 years, and focuses on preventing catastrophic flooding and protecting critical infrastructure and public safety.

According to DID (2012), the dimensions of lined open drains are limited to ensure public safety and facilitate maintenance. As shown in Figure 2.2, the minimum and maximum widths for open drains are 0.5 m and 1.2 m, respectively. However, the maximum depth for an uncovered open channel is 0.6 m, while that of a covered open channel ranges from 0.6 m to 1.2 m.

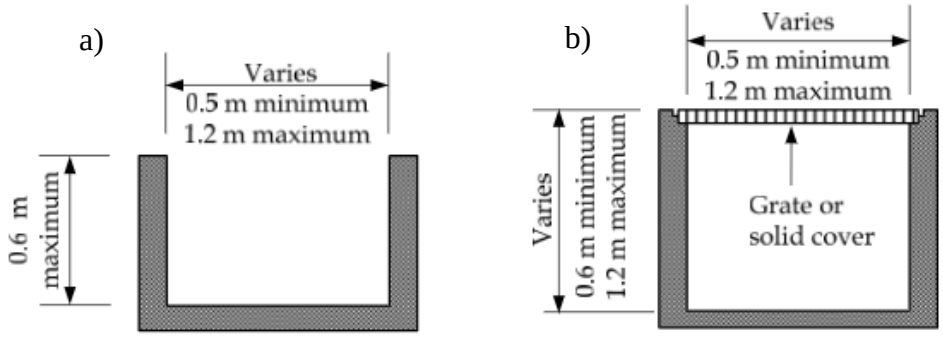


Figure 2.2: Dimension for a) Uncovered, b) Covered Open Channel (DID, 2012)

The hydraulic capacity of a drain is determined by its size and invert level. A lack of hydraulic capacity is one of the factors that may contribute to surface runoff accumulation and flash floods (Er et al., 2017). Manning's equation is a long-established formula for drain design. It assumes steady-state flow in an open channel and is used to describe the relationship between flow rate, depth, and slope in conduits. Manning's Equation is defined as:

$$Q_M = \frac{1}{n} A R^{2/3} S^{1/2} \qquad \text{Equation 2.1}$$

where,

Q_M = Flow rate (m^3/s);

n = Manning's roughness coefficient (unitless);

A = Flow area (m^2);

R = Hydraulic radius (m); and

S = Friction slope (m/m).

The amount of stormwater runoff entering the drain comes from the connected catchments. To estimate runoff from catchments, the MSMA applies the Rational Method to calculate peak discharge and is frequently used in Malaysia and other parts of the world. The Rational Method was employed to calculate surface runoff during the simulation. The rational technique is the most often used method for the analysis of runoff response from small catchments. According to DID (2012), the Rational Method is recommended for catchment areas smaller than 80 hectares and with the assumptions of uniform rainfall intensity over the entire catchment and a duration equal to the time of concentration, t_c. Peak discharge is calculated using the runoff coefficient, rainfall intensity, and catchment area which grouping all contributing factors into these three variables. Rainfall intensity, in hydrology and meteorology, refers to the amount of precipitation occurring over a specific time period, typically expressed in millimeters per hour (mm/hr). It plays a crucial role in flood modeling, drainage system design, and urban stormwater management. Rainfall duration is associated with the runoff concentration characteristics of the catchment. The runoff coefficient can be used to account for both hydrologic abstractions and runoff diffusion, as well as frequency. Thus, the Rational Method accounts for the key hydrologic processes influencing runoff response. The equation is given in MSMA as follows:

$$Q_R = \frac{C.I.A}{360} \qquad \text{Equation 2.2}$$

where,

Q_R = Peak discharge (m^3/s);

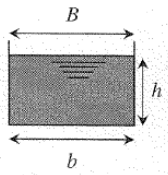
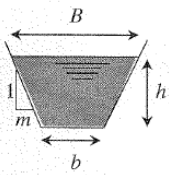
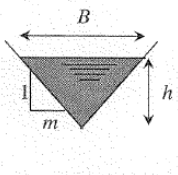
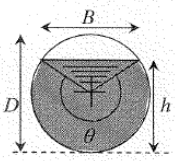
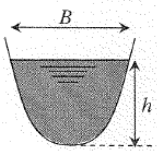
C = Runoff coefficient (unitless);

I = Rainfall intensity (mm/hr); and

A = Drainage area (ha).

The accumulated peak discharge of the total catchment area of a project area is used to design the drain size. As such, Equation 2.1 is equated to Equation 2.2 to ensure the design accommodates the peak discharge from the project area within the intended drain. In terms of drain sizing, the principle of the most economical section is applied, where the chosen drain type has the minimum wetted perimeter for a given discharge. Geometric elements from Table 2.1 are substituted into Manning's formula. Working steps of drain design are available in Appendix A.

Table 2.1: Most Economic Section (Akan and Seshadri, 2021)

	rectangular	*trapezoidal*	*triangular*	*circular*	*parabolic*
					
flow area A	bh	$(b+mh)h$	mh^2	$\frac{1}{8}(\theta-\sin\theta)D^2$	$\frac{2}{3}Bh$
wetted perimeter P	$b+2h$	$b+2h\sqrt{1+m^2}$	$2h\sqrt{1+m^2}$	$\frac{1}{2}\theta D$	$B+\frac{8}{3}\frac{h^2}{B}$ *
hydraulic radius R_h	$\frac{bh}{b+2h}$	$\frac{(b+mh)h}{b+2h\sqrt{1+m^2}}$	$\frac{mh}{2\sqrt{1+m^2}}$	$\frac{1}{4}\left[1-\frac{\sin\theta}{\theta}\right]D$	$\frac{2B^2h}{3B^2+8h^2}$ *
top width B	b	$b+2mh$	$2mh$	$\frac{(\sin\theta/2)D}{2\sqrt{h(D-h)}}$ or	$\frac{3}{2}Ah$
hydraulic depth D_h	h	$\frac{(b+mh)h}{b+2mh}$	$\frac{1}{2}h$	$\left[\frac{\theta-\sin\theta}{\sin\theta/2}\right]\frac{D}{8}$	$\frac{2}{3}h$

* *Valid for* $0<\xi\le 1$ *where* $\xi = 4h/B$

If $\xi > 1$ *then* $P=(B/2)\left[\sqrt{1+\xi^2}+(1/\xi)\ln\left(\xi+\sqrt{1+\xi^2}\right)\right]$

2.4 Flow Through Orifice

Conventional urban drainage systems (UDSs) are increasingly unable to manage the intensifying and unpredictable nature of modern rainfall events due to their rigid design capacities, lack of adaptive operation, and minimal integration of real-time flow regulation. Ma et al. (2024) demonstrated that such systems, originally developed under historical climate assumptions, are easily overwhelmed during high-intensity storms, particularly those induced by tropical cyclones. Their study proved that integrating intelligent scheduling models combining the Storm Water Management Model (SWMM) with multi-objective particle swarm optimization can significantly enhance drainage performance without the need for costly physical expansion. This clearly highlights the structural and operational limitations of conventional UDSs and emphasizes the need for adaptive, data-driven approaches to improve urban flood resilience under evolving climatic conditions. These limitations often lead to system overflow and localized flooding. As an alternative to costly infrastructure upgrades, this study explores the use of flow regulators such as orifice configurations in open channels as a passive, scalable solution to enhance flow regulation under varying storm intensities. Flow regulators are devices used to control and divert flows within a conveyance system. They are typically used to control releases from storage facilities, prevent unacceptable surcharging, and divert flow to treatment facilities and interceptors (Figure 2.3). An orifice is a type of flow-regulating device, consisting of a regularly shaped, submerged opening through which flow rate is proportional to the square root of the head across the opening, as shown in Equation 2.3.

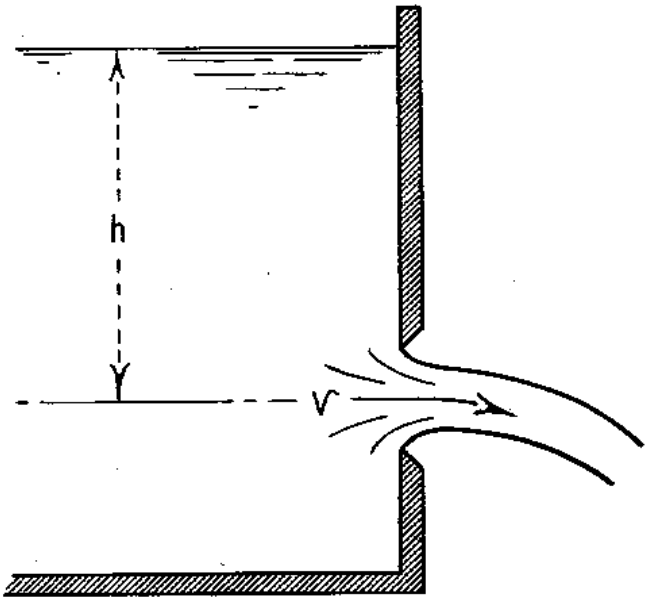


Figure 2.3: Orifice Flow (USBR, 2001)

By knowing the orifice size, the discharge rate by it can be determined by Equation 2.3. According to Rossman (2017), the most commonly used discharge coefficient for orifices (C) is 0.6. Moreover, the maximum head to the center of the orifice (h) can be determined from the orifice design which was illustrated in Figure 2.3. Acceleration due to gravity (g) is taken as 9.81 m/s.

$$Q_O = AC\sqrt{2hg} \qquad \text{Equation 2.3}$$

Where,

Q_O = Flow discharge from orifice outlet (m^3/s);

A = Orifice diameter (m^2);

C = Discharge coefficient of orifice (unitless);

h = Maximum head to centre of orifice (m);

g = Acceleration due to gravity (m/s^2).

In stormwater drainage management, orifices are commonly used to regulate the flow of stormwater in drainage systems. During heavy rainfall events, large volumes of water can accumulate quickly, causing flooding and damage to infrastructure and property. Orifices are used to slow down the flow of stormwater, reducing the risk of flooding. Due to its compact size and lightweight design, this application is widely used in industrial plants, which usually involve gas and liquid systems. Orifices can be installed in a variety of stormwater management structures, such as detention ponds, infiltration systems, and swales. By regulating the flow rate through these structures, orifices help prevent downstream flooding and reduce the volume of water entering the municipal stormwater system.

A single-hole orifice plate (Figure 2.4) is a small opening or hole on a plate which is used to control the flow rate and the direction of the flow. In general, the single-hole orifice plate is adapted to the pipe flanges as it provides a hole that permits the flow to pass through it. It typically exhibits a lower discharge coefficient. Due to the restriction of flow through a single hole, the pressure loss at the upstream and downstream regions of the plate is relatively high (Singh and Tharakan, 2015). This was observed and confirmed in an experimental study by Testud et al. (2007). A significant level of noise and high vibration of structures are noticeable in the use of single-hole orifice plates. In this case study, a single-hole orifice plate is adapted to the open channel as it provides a hole that permits the flow to pass through it (Figure 2.5).





(*Custom SS Orifice Plate 600 X 600mm*, n.d.) (*Piping Engineering - Knowledge Base* -, n.d.)

Figure 2.4: Examples of Single-hole Orifice Plate

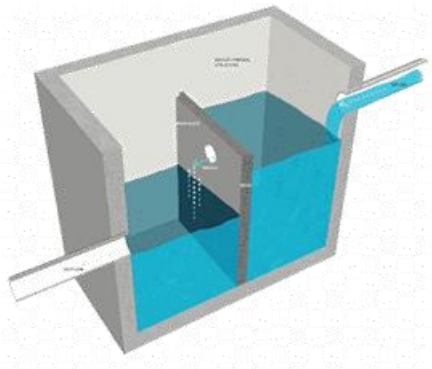


Figure 2.5: Stormwater Orifice Plate in Drainage System

(*Free Download | Weir Stormwater Orifice Plate Drainage Infiltration, Three Openings, Angle, Drainage, Bioretention Png | PNGWing*, n.d.)

Orifices in series refer to multiple orifices, with fluids flowing through them one after the other. In industrial applications, orifices in series are one of the essential instruments used for controlling and measuring the fluid flow rate accurately within a system as an integral part or as a whole. An orifice is inserted into a thin wall that creates the compartments which form a detention to restrict the flow. The main goal of the intervention of the orifice is to prevent the water from being congested at the downstream end or exit point of the drain. As a result, it may disperse stormwater volume in the drain's upper and intermediate stretches. However, the size and spacing of the orifice will influence the flow

rate of the fluid. To reduce the flow rate in an open channel and minimise downstream congestion, the working steps of flow through orifice design are available in Appendix C.

A concerns for orifice installation is clogging, especially in open channel where leaves, plastics, garden waste, and domestic waste can easily obstruct small orifice openings. Such blockages are likely to occur during the initial stages of a storm event or following windy or poorly maintained conditions. Additionally, sedimentation poses a concern, as fine debris and dust from driveways, pavements, or vehicle activity may accumulate near or within the orifice, gradually reducing its effective discharge area. The ground-level placement of open channel in residential area may further increases vulnerability to physical obstruction or tampering by residents, who may unknowingly block the orifice with flower pots, bins, or other objects. In some cases, aesthetic concerns or perceived safety issues may also lead to intentional covering of the orifice. These challenges highlight the importance of considering maintenance accessibility and raising public awareness.

Seven (7) examples on the applications of single-hole orifice and orifice in series are presented in (a) Araoye et al., 2017; (b) Zheng et al., 2020; (c) Düz, 2021; (d) Kuok and Chiu, 2022; (e) Sembiring et al.; f) Assran et al., 2024; and (g) Ma et al., 2024.

a. Flow Characteristics Through a Serial Arrangement of Orifice Plates (Araoye et al., 2017)

A study investigating the effect of two serial orifice plates within a pipe of 25.4 mm inner diameter examined pressure losses for consistent orifice diameter ratios. The impact of 1D and 2D spacing between the orifice plates was evaluated for diameter ratios of 0.5, 0.63, and 0.77, as illustrated in Figure 2.6. The streamline patterns at an inlet velocity of 2 m/s (Figure 2.7) revealed increased flow velocity near the upstream orifice and a velocity

reduction in the reattachment zone downstream. Recirculation zones, which exhibited the lowest velocities, became more pronounced as the diameter ratio decreased.

Figure 2.8 and Figure 2.9 present the static pressure differential variation along normalized axial distances for different diameter ratios under single and double orifice conditions. As expected, smaller diameter ratios resulted in greater pressure losses and lower flow rates. Interestingly, the minimum pressure points were similar across configurations single orifice, double orifice with 1D spacing, and the first trough in the 2D spacing configuration. Among the configurations, the double orifice with 1D spacing produced the lowest pressure drop, whereas the 2D spacing induced the highest. For the case of Dr = 0.63, the 1D-spaced double orifice reduced pressure loss by approximately 23% compared to a single orifice, while 2D spacing increased it by about 21%. Similar patterns were observed across other diameter ratios.

The findings from this study provide valuable insights into how orifice diameter and spacing affect pressure losses in closed conduit systems. These principles are relevant to open channel applications as well, particularly in systems that utilise orifices for flow regulation, such as those investigated in the present research. While the flow conditions differ between closed pipes and open channels, the observed trends in pressure drop and flow behavior due to orifice configurations offer a foundational understanding applicable to the development of stormwater management strategies involving multiple orifices under varying slopes and rainfall intensities.

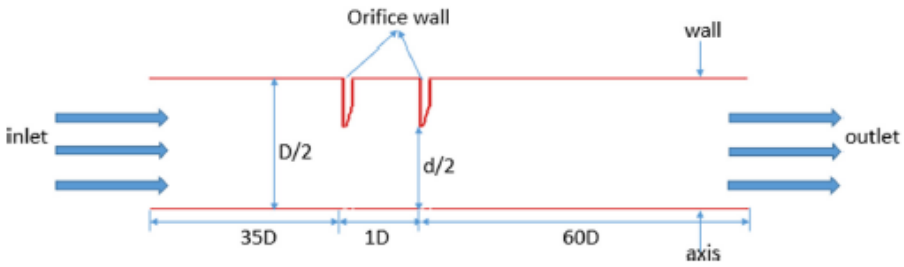


Figure 2.6: Schematics of Flow Domain Around the Orifice (Araoye et al., 2017)

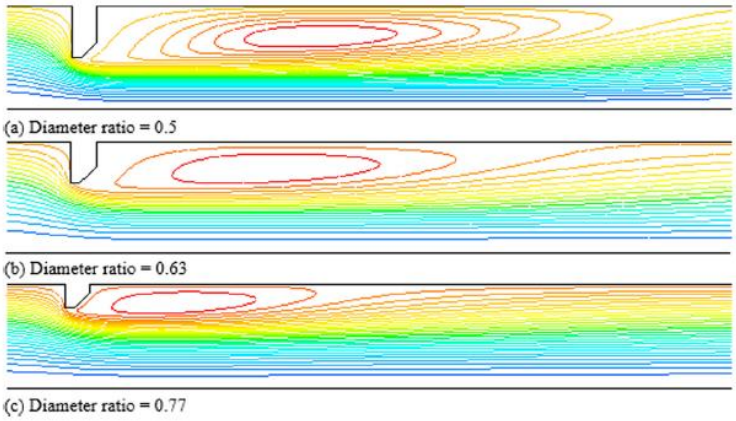


Figure 2.7: Comparison between Streamline Plots for a Single Orifice with Various Diameter Ratio a) Dr = 0.5; b) Dr = 0.63; and Dr = 0.77 (Araoye et al., 2017)

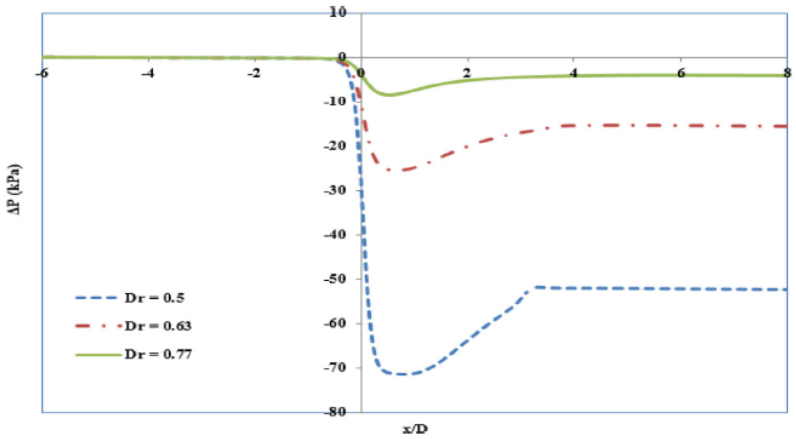


Figure 2.8: Variation of Static Pressure Difference for Single Orifice (Araoye et al., 2017)

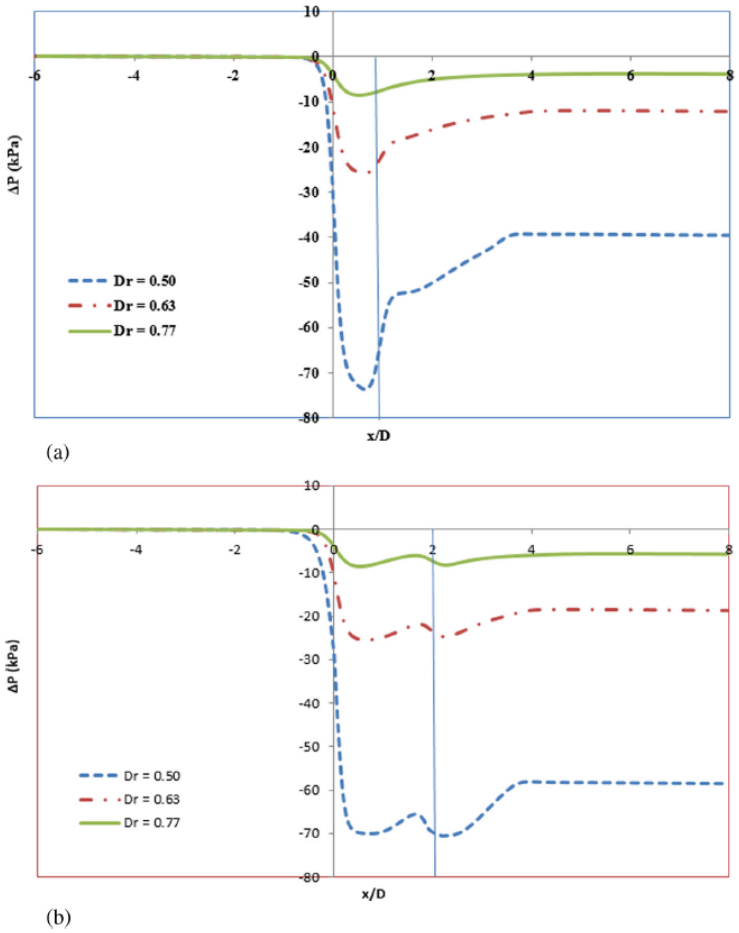


Figure 2.9: Variation of Static Pressure Difference for Double Orifice with a) 1D; and b) 2D spacing (Araoye et al., 2017)

b. Flow Through Multiple Sub-Orifices in Series (Zheng et al., 2020)

To enhance both the restriction effect and structural rigidity of orifice-type aerostatic bearings, a novel restrictor design incorporating multiple orifices arranged in series was proposed and evaluated numerically. The study examined the performance of these multi-orifice restrictors in enhancing flow restriction and bearing stiffness, using a circular aerostatic thrust bearing model with a central feedhole. As illustrated in Figure 2.10, the gas was subjected to successive stages of throttling as it passed through the serial orifices, thereby increasing the overall restriction effect.

The computational fluid dynamics (CFD) model was validated by comparing the mass flow rate of a simple orifice restrictor with theoretical nozzle flow predictions, as shown in Figure 2.11. Across various pressure drop conditions, all multi-orifice configurations exhibited lower mass flow rates than single-orifice designs, indicating a stronger restriction effect. Moreover, the restriction capability was further enhanced by increasing the number of sub-orifices, without the need to reduce individual orifice diameters. This approach addresses the practical limitations associated with manufacturing extremely small orifices.

The findings confirmed that a multi-orifice restrictor can effectively achieve greater restriction by increasing orifice count rather than decreasing aperture size. This principle is particularly relevant to applications requiring controlled flow regulation. While the study focused on closed-system aerostatic bearings, the underlying concept is applicable to open-channel flow systems as well. In the context of the present research, which investigates the influence of single, double, and triple orifices in stormwater channels under varied slopes and rainfall conditions, the demonstrated benefits of multi-orifice arrangements offer a valuable theoretical foundation for flow modulation strategies.

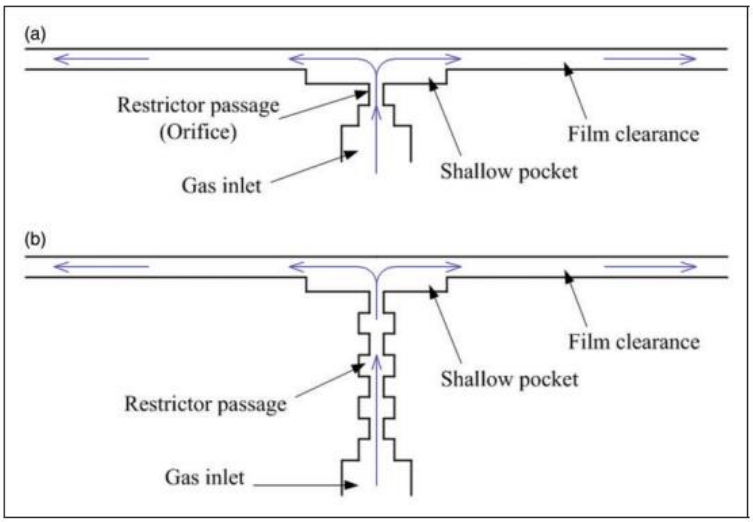


Figure 2.10: Gas Flow in the Bearings with a) Simple Orifice Restrictor; b) Multi-orifice Series Restrictor (Zheng et al., 2020)

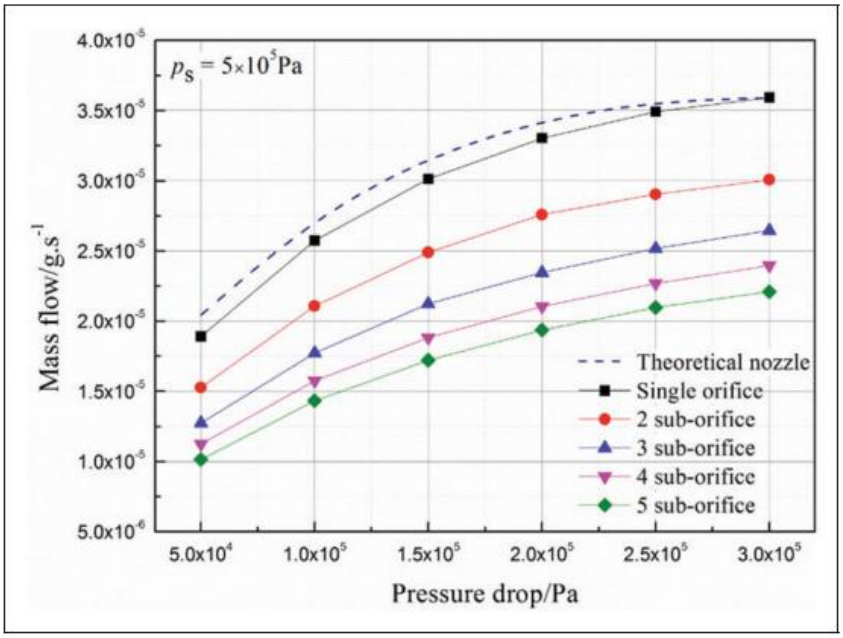


Figure 2.11: Mass Flows of Simple Orifice and Multi-orifice Series Restrictors with Different Numbers of Sub-orifices (Zheng et al., 2020)

c. Effect of Conical Angle of Orifice Plate Flows (Düz, 2021)

A recent study investigated the impact of orifice plate geometry on pressure losses by testing orifices with varying diameter ratios (0.4, 0.5, and 0.63) and beveled-edge designs at different conical angles (15°, 30°, and 45°). These configurations were evaluated numerically via flow discharge simulations in a pipe with an inner diameter of 50.8 mm, as illustrated in Figure 2.12. Among the tested angles, the orifice plate with a 30° conical bevel demonstrated the most effective performance in minimizing pressure losses, achieving reductions between 37% and 52% compared to conventional sharp-edged orifices. Additionally, when compared to the 45° beveled orifice, the 30° design further reduced pressure losses by approximately 13 to 21%.

The findings suggest that bevelled orifices particularly those with a 30° conical angle, should be considered in the design of flow regulation systems to enhance hydraulic efficiency. Moreover, in multi-hole plate flow conditioners, bevelling at 30° was shown to significantly lower pressure losses compared to standard 45° bevels. This improvement is attributed to the smoother flow transition and reduced turbulence induced by the optimised edge geometry.

These results highlight the importance of both the orifice diameter and edge angle in influencing flow characteristics and pressure behaviour. While the study was conducted within a closed pipe system, the underlying principles are applicable to open-channel flow control scenarios as well. In the context of the present research, which explores the use of various orifice configurations under different slopes and rainfall intensities in stormwater channels, the role of geometric optimization such as bevelled orifice edges offers valuable insight into improving flow regulation and minimizing energy losses.

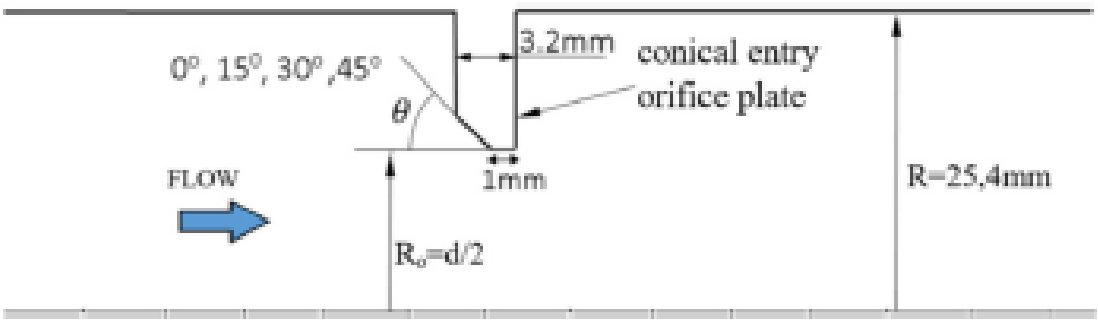


Figure 2.12: 2D Pipe Flow through an Orifice Plate (Düz, 2021)

d. Flow through Various Size of Orifice (Kuok and Chiu, 2022)

A study conducted in Kota Samarahan, Sarawak, proposed the integration of a detention pond system with various orifice outlet sizes to manage urban runoff (Figure 2.13). The Storm Water Management Model (SWMM) was employed to simulate different scenarios, incorporating rainfall events with 10- and 100-year average recurrence intervals (ARIs) and orifice diameters ranging from 1.5 m to 4.5 m. The resulting outflow hydrographs were compared with those from pre-development conditions (without detention) to assess the effectiveness of the proposed system.

The findings, presented in Figure 2.14, demonstrated that the inclusion of orifice outlets significantly reduced peak discharge rates when compared to the post-development scenario without flow regulation. Specifically, orifice configurations measuring 2.5 m × 2.5 m, 3.5 m × 3.5 m, and 4.0 m × 4.0 m were found to be effective in attenuating flow for the 10-year and 100-year ARIs, respectively. These results indicated that the orifice size plays a critical role in regulating outflow and mitigating flood risks by maintaining discharge rates within acceptable thresholds.

This study supports the concept that strategically sized orifices can serve as an effective tool for urban stormwater management. The findings are particularly relevant to the present research, which examines orifice-based flow regulation mechanisms in open channel systems. While the scale and context differ, the demonstrated ability of orifices to modulate discharge in response to varying rainfall intensities provides valuable insight into the role of orifice sizing in flood control design.

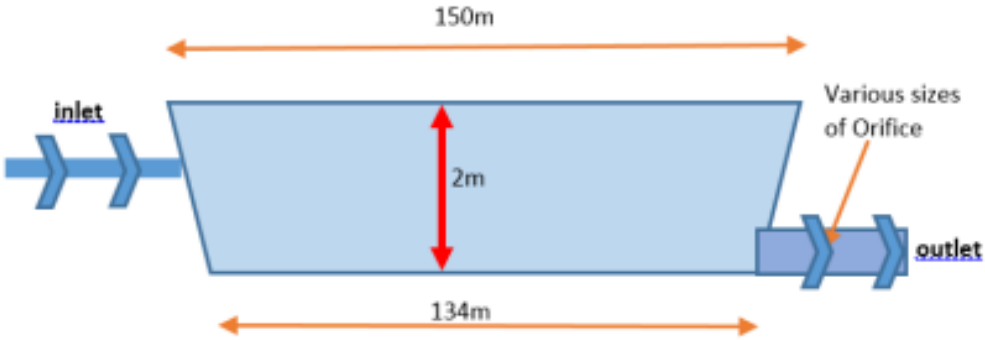


Figure 2.13: Geometry of Detention Pond (Kuok and Chiu, 2022)

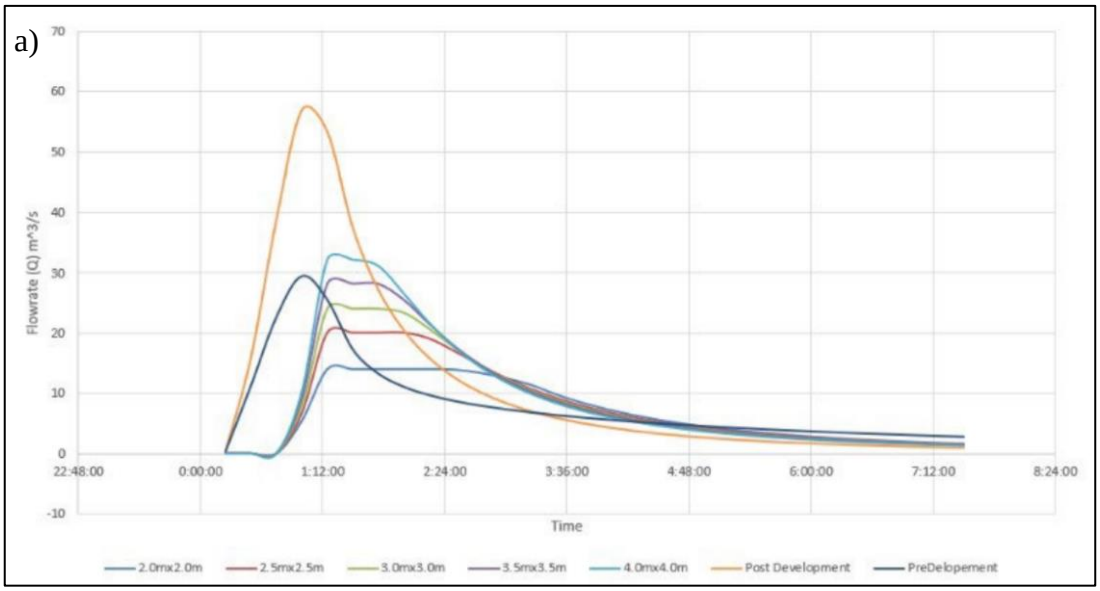


Figure 2.14: Outflows of orifices for a)10- and b) 100-year ARI Storm Events (Kuok and Chiu, 2022)

b)

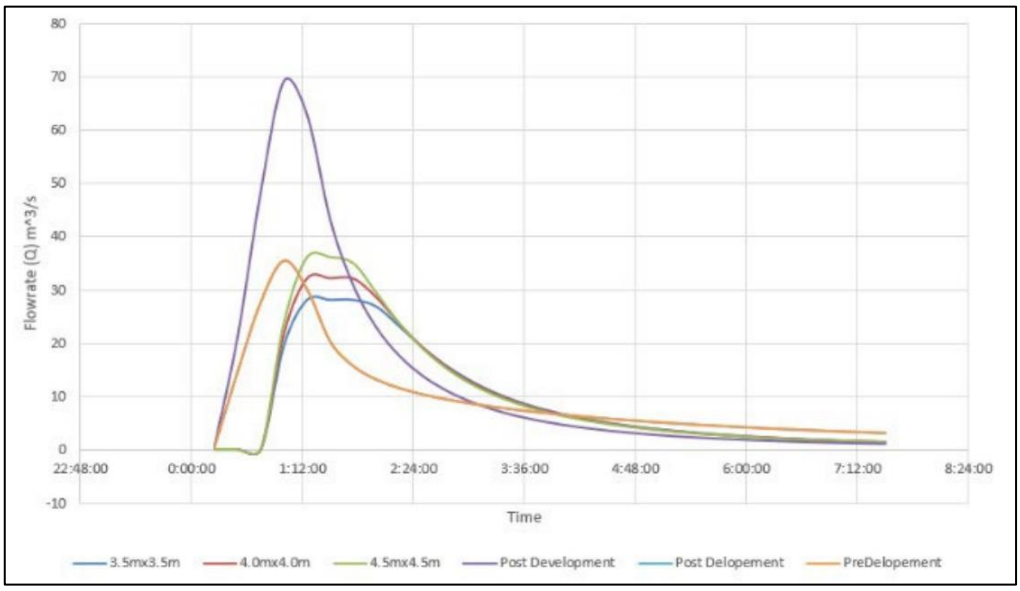


Figure 2.14: (Continue)

e. Testing the Performance of a Water Restrictor with the Orifice Principle (Sembiring et al., 2022)

Sembiring et al. (2022) conducted an experimental study to evaluate the performance of a water restrictor equipped with a 2.5 mm central orifice. Installed in a ½ inch diameter tap, the restrictor was tested under varying static pressures. The researchers measured discharge rates with and without the restrictor and analyzed pressure losses to determine the contraction and expansion coefficients. Their findings indicated that the restrictor significantly reduced water discharge by approximately 60–70% for input flows between 0.15 and 0.25 l/s, and by 30–40% for lower input flows (below 0.15 l/s). The output discharge was consistently reduced to approximately 0.06–0.07 l/s.

Although the study focused on closed-pipe household applications, the findings are relevant to stormwater management systems where orifices serve a similar role in modulating flow. The concept of flow restriction through orifices and the associated energy

losses directly support the present study, which expands this principle into open channel drainage under varying slopes and rainfall intensities. Moreover, while Sembiring et al. analyzed a single-orifice configuration, the current research further contributes by examining the influence of multiple orifices, thereby addressing a gap in the literature regarding complex orifice arrangements in stormwater attenuation.

f. Characteristics of Flow Through Orifice-Meter (Assran et al., 2024)

This study investigated the influence of orifice geometry on energy loss and discharge coefficient using both experimental and computational approaches. Two orifice shapes which are circular and triangular were examined, each tested with four different cross-sectional areas, as illustrated in

Figure **2.15**. The experimental setup involved a 10 cm transparent pipe with a 45° inclination and a 10 mm wall thickness, fitted with an orifice plate to evaluate flow behaviour under controlled conditions. It was observed that as fluid velocity increased through the orifice opening, the corresponding pressure decreased, highlighting the inverse relationship between flow velocity and pressure.

Computational simulations were carried out using ANSYS Fluent to complement the experimental findings and to analyse flow dynamics in greater detail. As shown in Figure 2.16, the triangular orifice produced a higher longitudinal centreline velocity. Both experimental and numerical results consistently indicated that triangular orifices outperformed circular ones, exhibiting larger discharge coefficients and reduced pressure head losses.

These findings underscore the significance of orifice geometry in governing flow characteristics and hydraulic performance. While the study focused on pressurized pipe

systems, the demonstrated advantages of non-circular orifice shapes particularly in reducing energy losses and enhancing flow discharge offer valuable insights for broader hydraulic applications. In relation to the present research, which evaluates the effect of single, double, and triple orifices on open-channel stormwater flow, the role of geometric design in optimizing flow behaviour is a relevant and supportive consideration.

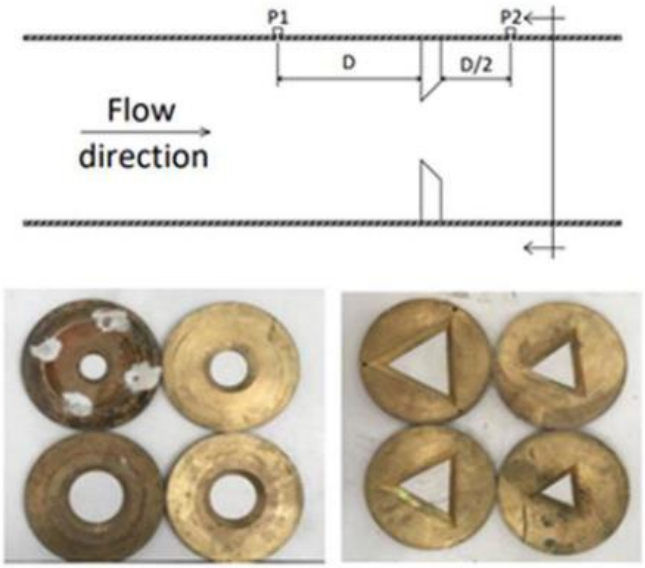


Figure 2.15: Orifice Plate Shape used in This Study (Assran et al., 2024)

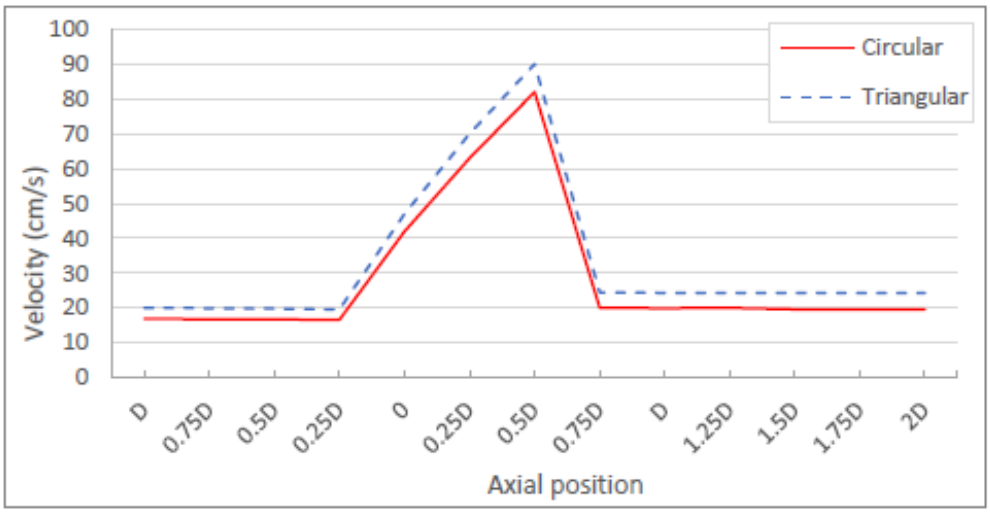


Figure 2.16: Velocity Variations for Two Orifices (Assran et al., 2024)

g. Optimizing Urban Drainage Systems Using Intelligent Scheduling for Flood Mitigation (Ma et al., 2024)

Urban flooding has become more frequent and severe due to climate change and rapid urbanization, putting pressure on traditional urban drainage systems (UDSs). Designed based on historical rainfall patterns, these systems are increasingly unable to manage extreme weather events. To address this challenge, Ma et al. (2024) developed a multi-objective intelligent scheduling model that integrates the Storm Water Management Model (SWMM) with a particle swarm optimization algorithm to dynamically manage UDSs during tropical cyclone (TC) and non-TC events. The model optimizes the operation of hydraulic structures, such as pumps, weirs, and gates, to regulate flow and reduce flood volume (TFV) and water level fluctuations (WLF) without requiring costly infrastructure expansions.

Ma et al. (2024) demonstrated that optimizing the scheduling and operation of these hydraulic structures rather than expanding infrastructure can significantly reduce TFV (3.11% to 57.77%) and WLF (26.32% to 65.48%). Their approach highlights the potential of intelligent scheduling to enhance UDS performance, particularly in urban areas where space and budgets are limited. Furthermore, Ma et al. (2024) showed that intelligent optimization can outperform traditional infrastructure upgrades or low-impact development (LID) measures, making it a practical solution for resource-constrained urban environments.

Similarly, this study explores the use of strategically placed orifices in open channels as a low-cost, space-efficient flood mitigation strategy. By adjusting orifice configurations, flood risk can be reduced without the need for significant infrastructure changes, aligning with the goal of improving UDS performance in space-limited urban settings.

2.5 Storm Water Management Model (SWMM)

Many environmental choices are made with the use of models. Models can be used to create future projections for situations that are predicted to exist at the time. SWMM is a distributed, dynamic rainfall-runoff simulation model that could be used to simulate runoff quantity and quality from single rainfall events and long-term (continuous) simulations in predominantly urban areas (Rossman, 2010). SWMM was developed and distributed by the United States Environmental Protection Agency (USEPA) in 1971. It is one of the most widely used and well-known numerical models for controlling urban runoff quantity and quality (Shahed Behrouz et al., 2020). The most current version 5.1 offers an integrated environment for designing and sizing drainage systems, including their appurtenances, particular components, and facilities. To put it another way, SWMM 5.1 provides an integrated environment for changing study area input data, running hydrologic, hydraulic, and water quality simulations, and viewing results in various formats. In drainage and stormwater studies, it is becoming more commonly employed.

Stormwater is conceptualized as moving through a series of water and material flows between several major environmental compartments (Figure 2.17). First, rain is represented in SWMM as a rainfall gauge, which can be utilized in the atmospheric compartment to represent the system's rainfall inputs. The land surfaces that intercept rainfall is then classified as sub-catchment. An event or precipitation occurs within the atmospheric compartment, which falls and is deposited on the ground surface. A sub-catchment represented by a land surface compartment is described by its area, width, slope, degree of imperviousness, depression storage, and soil attributes. Precipitation falls on the sub-catchment, and before reaching the outfall, it is conveyed into conduits and nodes represented by the transport compartment as surface runoff.

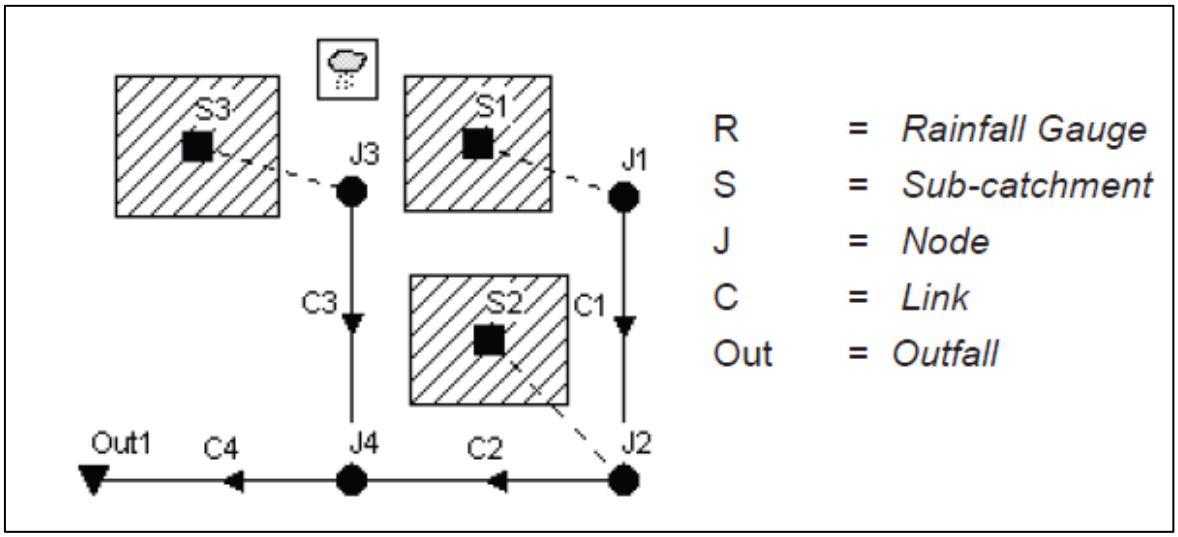


Figure 2.17: Concept of Drainage Modelling (Mah, 2016)

Stormwater flow is computed from Equation 2.4 based on the catchment characteristics, governs the flow discharge after landfall in SWMM. SWMM can be used to measure parameters such as width (W), slope (S), depression storage (d_p), and water depth over the catchment (d). Manning's n value of 0.4 is used for the pre-development condition, and for the post-development condition, an n value of 0.8-1.0 is used (PUB, 2010). An n value of 1.0 was used for roofing and road catchments based on a previous study by Mah et al. (2020b), assuming that water losses from the impermeable roof and road surfaces were minimal.

$$Q = W\frac{1.49}{n}\left(d - d_p\right)^{5/3}S^{1/2} \qquad \text{Equation 2.4}$$

where,

Q = Flow discharge from catchment (m^3/s);

W = Width of catchment (m);

S = Slope of catchment (m);

n = Manning roughness coefficient (unitless);

d_p = Maximum depression storage (m);

d = Depth of water over the catchment (m).

The "node-link" idea is used in SWMM to depict a drainage system conceptually. In SWMM, a node is a point where hydraulic elements meet. A node can also be a storage device such as a pond or lake, a point junction representing a point of change in channel or conduit design, a model boundary condition, a sewage treatment facility in the transport layer, or a watershed in the runoff layer. Flow is transmitted from one node to the next via the link. The hydraulic elements for flow and constituent conveyance through the system, such as pipe, channel, pump, weir, and orifice regulator, are represented by a link. Stormwater finally exits the system through an outfall.

Equation 2.5 governs the flow routing. SWMM routes the stormwater flow from node to node until it reaches the final discharge point, accounting for the distance between two nodes (x), cross-sectional area of the drain (A), flow geometry (α) and surface roughness of drain (m). These variables can be measured at the study site. The flow routing time step is a variable. A time step of 30 seconds was shown to be suitable for drains in an urban area in a previous study by Mah et al. (2020b).

$$q = \frac{\partial A}{\partial t} + \alpha m A^{(m-1)} \frac{\partial A}{\partial x} \qquad \text{Equation 2.5}$$

where,

q = Routed flow discharge (m^3/s);

A = Cross-sectional area of the drain (m^2);

x = Distance along the flow path (m);

t = Time step (s);

α = Flow geometry due to drain (unitless);

m = Surface roughness of drain (unitless).

Three (3) examples on the application of SWMM in urban drainage modelling are presented in (a) Mah et al., 2018; (b) Mah et al., 2020a; and (c) Mah et al., 2023.

a. Case Study at Pandungan, Kuching (Mah et al., 2018)

This study's technique was based on computer modelling which also applied in the case study. In SWMM, rain is represented as  which can be used to represent the system's rainfall inputs in the atmospheric compartment. Sub-catchments represented by  was defined as land surfaces that intercepted rainfall which indicated in the Padungan. Precipitation fell on the sub-catchment and was carried into conduits and nodes which represented by  and  respectively as the surface runoff before reaching the outfall. Outfall was represented as  in this modelling.

Two models were created in this study. One of them was indicated the current drainage system facility as shown in Figure 2.18. The other model was the implementation of permeable road along the back of Padungan Street (Figure 2.19) to accommodate runoff from the entire two rows of buildings. The permeable road was represented with storage unit or  with an orifice outlet represent by  in SWMM. As a result, implementing the permeable road was a possible approach based on modelling scenarios because it incorporated both current and future needs into the design consideration.

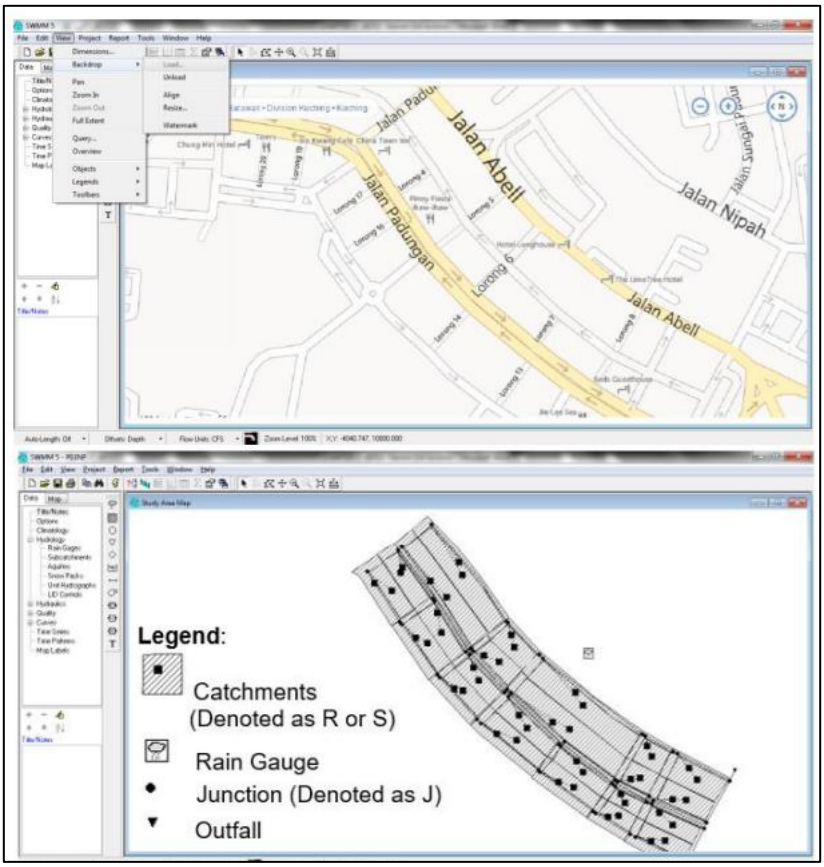


Figure 2.18: Conveyance Network for Urban Drainage System (Mah et al., 2018)

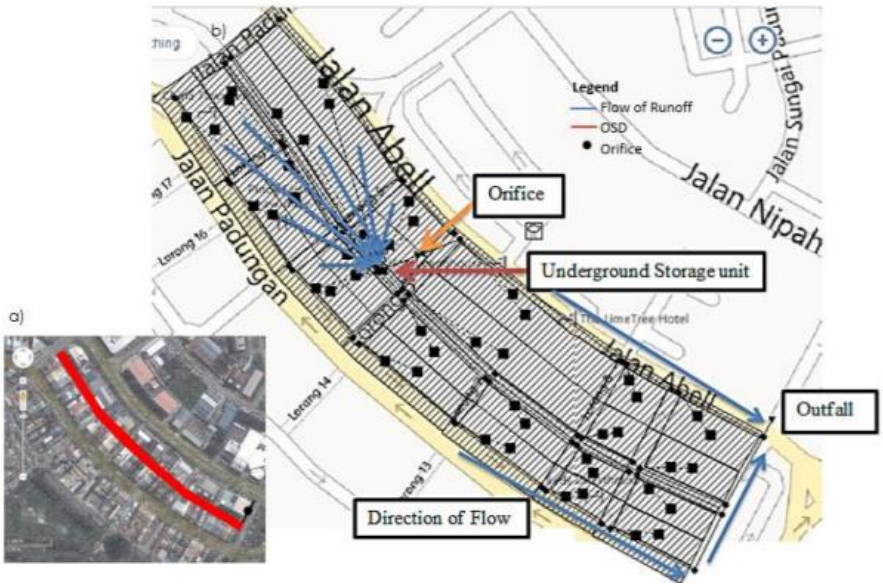


Figure 2.19: Conveyance Network for On-Site Detention Tank, a) Location of Tank, and b) Model Components (Mah et al., 2018)

b. Case Study at Housing Area (Mah et al., 2020a)

In the study of Mah et al. (2020a), on-site detention tank was installed as part of a house's car porch to detain stormwater from the house's roof (Figure 2.20a). SWMM version 5.1 was used in this study. The drainage model was divided into five components (Figure 2.20b) which were "rain gauge", "catchment", "storage unit", "orifice" and "outfall". The first component was the icon  for the rain gauge, which was where rainfall data was entered. The rainfall data was linked to the catchment which was represented by  and the roof catchment's characteristics were specified. This component was the hydrological simulation of rainfall-runoff relationship. The calculated runoff was moved from the catchment to the second component which was the on-site detention tank. The roof gutter and downpipe were included in the catchment rather than being expressly modelled. The stormwater detention tank was depicted by a symbol as  or storage unit. The term storage unit was used to denote the water storage volume based on the dimensions of the designed tank. The outlet was shown as a "orifice" which also denoted as  . It was connected to a "outfall" or the final discharge point, which was represented by a symbol  after the "orifice." No "link" was inserted in this study because no channel was required in the flow discharge mechanism.

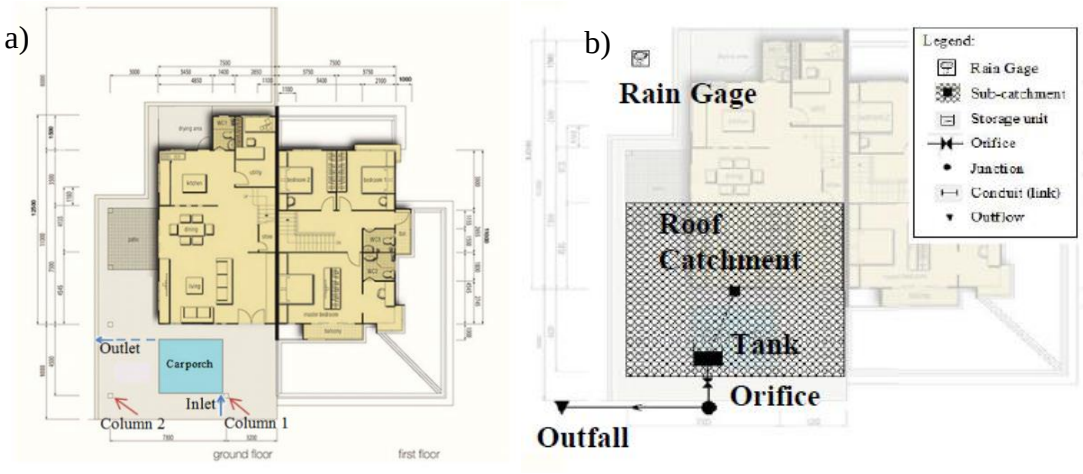


Figure 2.20: Conveyance Network in Residential House, a) Layout Plan and b) Model Components (Mah et al., 2020a)

c. Case Study at Commercial Area (Mah et al., 2023)

SWMM was used to evaluate a permeable road with an orifice outlet in a commercial area called Palm Square (Figure 2.21). The modelling process began with the model receiving historical extreme rainfall data via its "Rainfall" interface, which created runoff onto the attached "Catchment". In Commercial area, the "Catchments" are referred to roofs, highways, and parking lots. Running water was channelled from "Catchments" to permeable road. "Storage Units" and "Orifices" symbolise the permeable road, in which the tarred surfaces were divided in half along the road crown, enabling flowing water to flow down the slope of the road crown. "Nodes" and "Links" depicted the concrete drain network. The system's flowing water finally exited at the "Outfall".

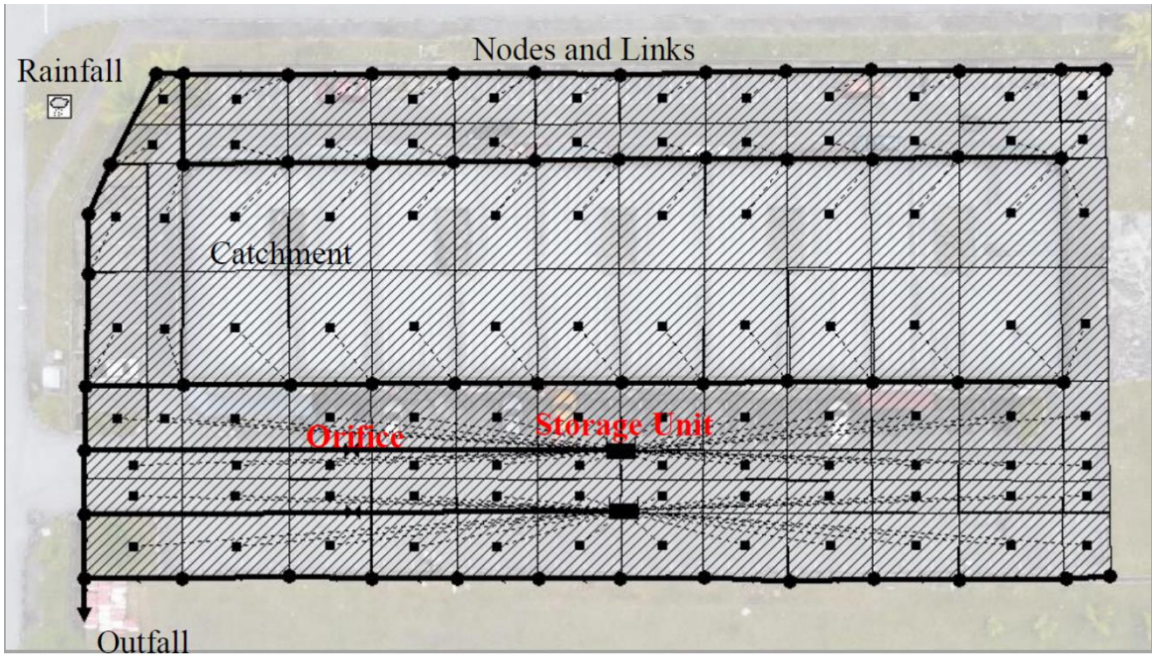


Figure 2.21: Conveyance Network for Commercial Area (Mah et al., 2023)

2.6 Performance Evaluation Metrics

To assess the performance of the developed model, two statistical metrics were employed: the Root Mean Squared Error (RMSE), and Nash-Sutcliffe Efficiency (NSE). These metrics provide quantitative measures of the model's accuracy and reliability by comparing the predicted values with the observed data. Each metric is briefly described below.

The Root Mean Squared Error (RMSE) measures the average magnitude of prediction errors. It is a widely used metric due to its interpretability, as it expresses error in the same unit as the variable being predicted. The RMSE penalizes larger errors more than smaller ones, making it sensitive to outliers. A lower RMSE value indicates a more accurate model (Sanders et al., 2022).

The RMSE is calculated using the following equation:

$$RMSE = \sqrt{\frac{1}{n}\sum_{i=1}^{n}(y - z)^2}$$

Where:

- n = Number of observations
- y = Observed value
- z = Predicted value

The Nash-Sutcliffe Efficiency (NSE) is commonly applied in hydrological modeling to assess the predictive accuracy of models (Fang et al., 2023). It evaluates how well the predicted values match the observed data relative to the mean of the observed data. The NSE value ranges from $-\infty$ to 1, with values closer to 1 indicating a perfect match between predicted and observed values. A negative NSE value implies that the mean of the observed data is a better predictor than the model itself.

The NSE is calculated using the following equation:

$$NSE = 1 - \frac{\sum(y - z)^2}{\sum(y - \bar{y})^2}$$

Where:

- y = Observed value
- z = Predicted value
- $\bar{y}$ = Mean of observed values

In this study, the NSE serves as a robust indicator of the model's predictive power. A value approaching 1 suggests that the model accurately simulates the observed data, whereas a value below 0 indicates poor performance.

2.7 Research Gap

Intervention to urban stormwater management are done by numerous researchers to reduce the peak flow. In this study, orifice was implemented in the open channel to reduce the peak flow at the downstream area to prevent the congestion along the open channel. Table 2.2 summarizes the relevant case studies on the applications of orifice. Based on Table 2.2, it is found that these studies were applied on pipeline (Studies No 1 to 3 and 5 to 6) and detention structures (Study No 4 and 7). No past study of applying orifice in drain was found at the time of writing.

Most of the studies were reporting on changing the diameter, spacing and shape of orifice plate, for within a pipeline in Studies No 1, 3,5 and 6; and for a detention structure in Study No 4, and 7. Another study was reporting on increasing the number of the orifice for a pipeline in Study No 2. Following these studies, the current explores on the reported properties (diameter of orifice, its spacing and shape as well as the number of orifices) but applying them with various design slopes within drain of a housing estate.

Table 2.2: Relevant Case Studies on Applications of Orifice

No.	Studies	Descriptions of Orifice
1.	Araoye et al., 2017	Testing two difference orifice plate spacings (1D and 2D) and three diameter ratios (0.5, 0.63, and 0.77) to test the flow characteristics through orifice plate in a pipe.
2.	Düz, 2021	Testing three difference diameter ratios (0.4, 0.5, and 0.63) and bevelled with various conical angles (15°, 30°, and 45°) to test the flow characteristics through orifice plate in a pipe.
3.	Zheng et al., 2020	Testing single orifice and multiple sub-orifice in series to test the restrictor impact by using a circular aerostatic thrust bearing with a central feedhole in a pipe.
4.	Kuok and Chiu, 2022	Testing varied orifice diameters ranging from 1.5m to 4.5m to test discharges of a detention pond subjected to design rainfall events under 10-year and 100-year ARIs.
5.	Sembiring et al., 2022	Testing the performance of a water restrictor with the 2.5mm central opening of orifice principle
6.	Assran et al., 2024	Testing circular and triangular orifice plate with various cross-sectional area fitted in a 10cm transparent pipe with an inclination angle of 45 degrees.
7.	Ma et al., 2024	Addressed urban flooding through intelligent scheduling of active hydraulic structures like pumps, weirs, and gates.

Table 2.2: (Continued)

No.	Studies	Descriptions of Orifice
8.	This study, 2025	Testing varied number of orifices in various design slopes (0%, 0.2%, and 0.4%) of drain with various diameters (0.1m, 0.2m, 0.3m, 0.4m, and 0.5m), various spacing and shape subjected to 5-min, 10-year ARI design rainfall and three historical extreme rainfall events.

CHAPTER 3

METHODOLOGY

3.1 Introduction

This study explores the flow restrictor system, specially tailored for housing estate's drain, which would solve the problem of flash flood and does not occupy spaces. A housing estate is known to have a stretch of drain in front of the row of houses. In this study, a flow restrictor in the form of orifice outlets are installed in the drain. The main idea of this study matches with the primary goal of stormwater management, which is to control the stormwater near its source and slowly release it into the downstream system.

A flow chart of methodology regarding this study is depicted in Figure 3.1. The concept of the urban drainage system is divided into three main stages which are rainfall, catchment and drain. The rain falls onto the catchment such as roof, road, and carpark before it discharges to drain. According to Gajjar and Dholakia (2014), to accomplish its objective, stormwater design should be supported by the essential information; as a result, the required data should be collected, such as rainfall duration and intensity at a certain period. Stormwater design starts by evaluating the state and functionality of the current drainage system (Torgersen et al., 2014). From there, by determining the physical storage volume of structures, the major goal of stormwater design is flood control. The design of a sustainable stormwater drainage system is subjected to social, economic, and environmental limitations. Following Figure 3.1, presentation in the following sub-sections is about rainfall, catchment, drain and lastly the SWMM modelling. The result of the SWMM modelling for catchment, and drain with or without the flow restrictor are then compared with the theoretical data for verification purposes.

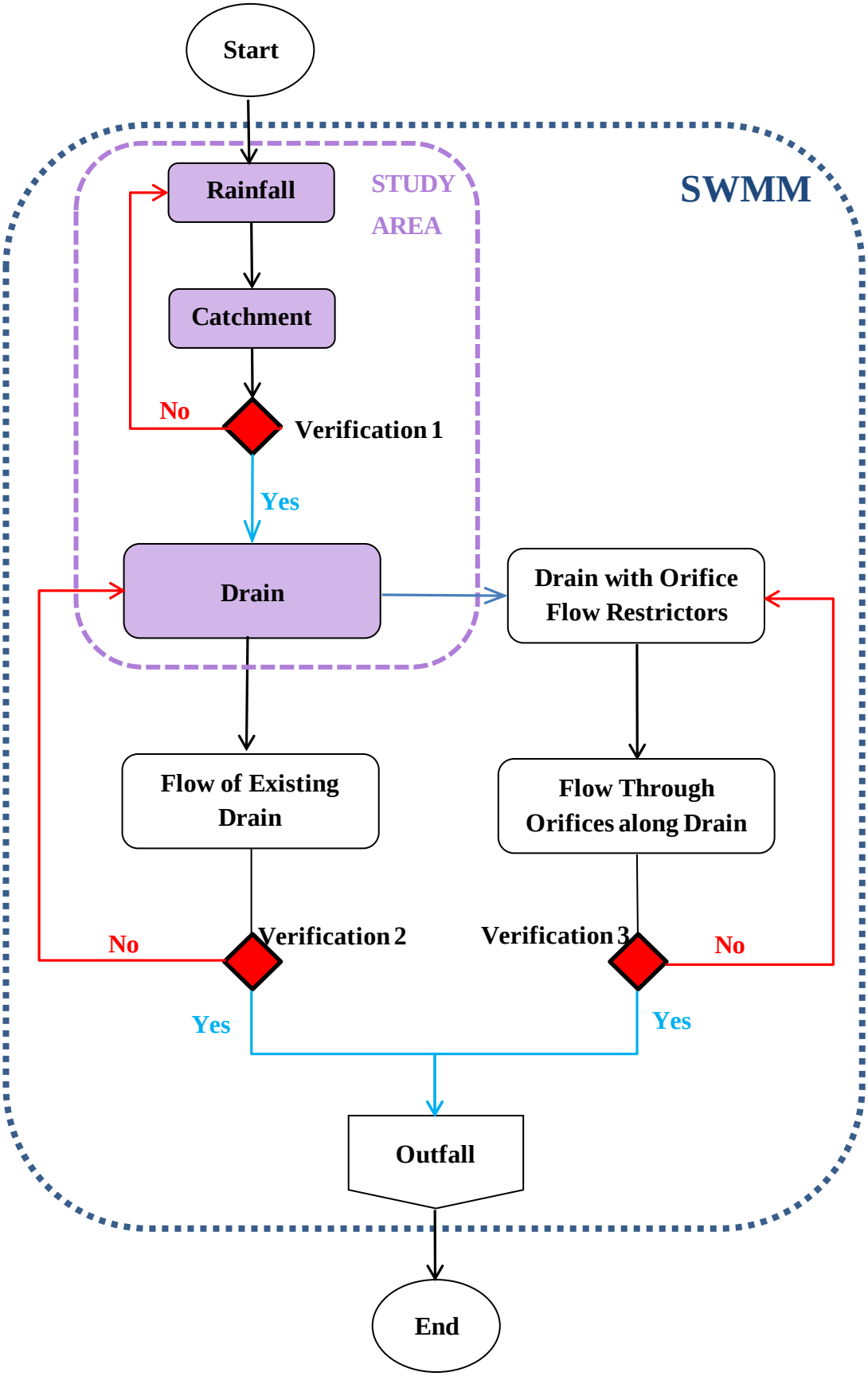


Figure 3.1: Flow Chart of Methodology

3.2 Rainfall

Under the recommendation of MSMA 2nd Edition (DID, 2012), a minor system for terrace housing estate relates to 10-year ARI design rainfall. PUB (2010) maintains that 5-minute rainfall duration is recommended for site area of 0.2 to 2 ha. Due to the area of the study area (0.42 ha) is between 0.2 to 2 ha, rainfall duration of 5 minutes was taken. Hence, 5-minute 10-year ARI design rainfall was used for runoff computation in this case. A design rainfall intensity of 278 mm/hr was derive from the Intensity-Duration-Frequency (IDF) curve for Kota Samarahan provided by the Department of Irrigation and Drainage (DID) Sarawak (refer to Appendix D). From the rainfall intensity, it could be converted to an equivalent rainfall depth of 23 mm.

Besides, three historical extreme rainfall events were collected from "*Public Info banjir*" official website for Kuching and Samarahan areas. As a single rainfall event was not uniformly distributed, extreme rainfall events caused flooding in different housing areas surrounding Kuching and Samarahan areas. These events were found with peak hourly rainfall depths ranging from 43.0 mm to 70.5 mm. Table 3.1 shows the summary of the three historical extreme rainfall events which applied in this study. The graphical representation of these storms is displayed in Figure 3.2. The historical extreme rainfall data were verified with evidences from news and Facebook posts. The evidences of the extreme rainfall are documented in Appendix E whereas the historical extreme rainfall data are tabulated in Appendix F.

Table 3.1: Selected Historical Extreme Rainfall Events in Kuching and Samarahan

Storm Event	Date of Occurrence	Storm Duration (Hours)	Peak Hourly Rainfall (mm)	Total Rainfall Depth (mm)
Event 1	19 Jan 2015	14	43.0	188.0
Event 2	17-18 Dec 2017	15	47.5	178.5
Event 3	11-12 Dec 2019	15	70.5	131.0

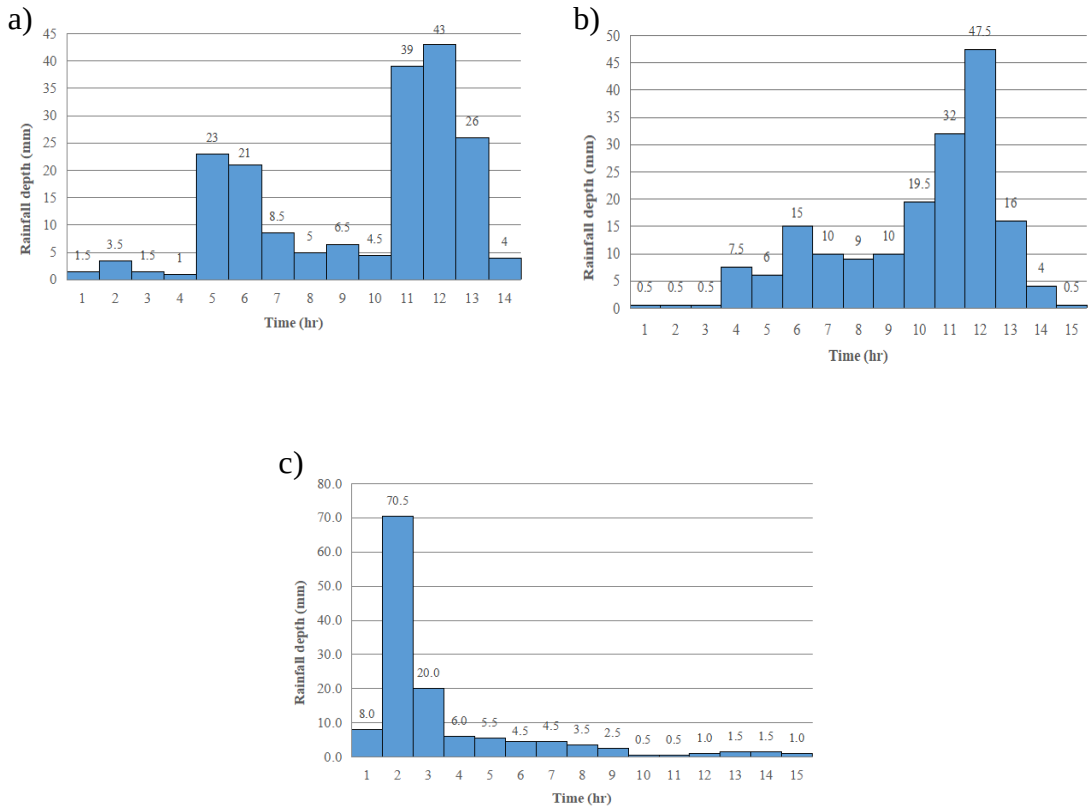


Figure 3.2: Rainfall Hyetographs of Selected Historical Extreme Rainfall Events, a) Event 1, b) Event 2, and c) Event 3

Referring to another source of rainfall classification, the Department of Irrigation and Drainage has classified rainfall depths into four categories as tabulated in Table 3.2. Based on the categorization, rainfall depths of the estimated design and collected observed rainfall events are summarized in Table 3.3. As a result, the rainfall events were found as "Heavy" to "Very Heavy" events.

Table 3.2: Categorization of Peak Rainfall Depths

(https://ihydro.sarawak.gov.my/iHydro/en/datatable/rainfall/hourly-rainfall.jsp)

Category	Light	Moderate	Heavy	Very Heavy
Rainfall	0.5 -10	11 - 30	31 - 60	> 60

Table 3.3: Categorization of Design and Observed Rainfall Events

Rainfall	Event /Peak Rainfall	Category
Design Rainfall	5-min 10-year ARI / 23 mm	Very Heavy
Historical Rainfall	Event 1 / 43 mm	Heavy
	Event 2 / 47.5 mm	Heavy
	Event 3 / 70.5 mm	Very Heavy

3.3 Catchment

Catchment data could be measured from a study area. The Mekar Sari housing estate, a typical Malaysian housing estate was chosen due to its housing layout with a long drain

(170 m) for each row of houses which the flow restrictor can be placed. As each row was identical to each other, only one row of 24 double-storey terrace houses was selected as study area for analysis of flow through orifices in the drain as shown in Figure 3.3 and Figure 3.4. Having an identified study area, the catchment area could be measured. To be specific, the roof and road catchments could be identified and separated allowing catchment areas for each property lot to be measured one by one. It is a realistic representation as runoff is generated from these catchments.

The total area of the catchment is 4227 m^2 (0.4227 ha) that consists of 24 units of double-storey houses. Approximately 75% of the catchment, which is 3170 m^2, is occupied by the terrace houses. There are two blocks of houses in each row and as such, there are four (4) corner and twenty (20) intermediate lots. An intermediate lot has a built-up space of 6.7m x 20.4m. A corner lot has the same built-up space as the intermediate lot, but with an additional 20 m^2 or 10 m^2 of side yard. The remaining 25% of the catchment, which is 1057 m^2, is occupied by carpark, concrete drain, green verge, footpath, hard shoulder, and tarred roads.

The selected catchment was further divided into sub-catchments. This was carried out following the flow of the stormwater. Running water from the front half of a property lot shall discharge to the front drain; and running water from the back half of the same property lot shall discharge to the back drain. As such, each of the property lot was divided to two sub-catchments. Similarly, following the crown of the road, half the road shall discharge to the drain adjacent to it. The details of the sub-catchments are presented in Table 3.4.

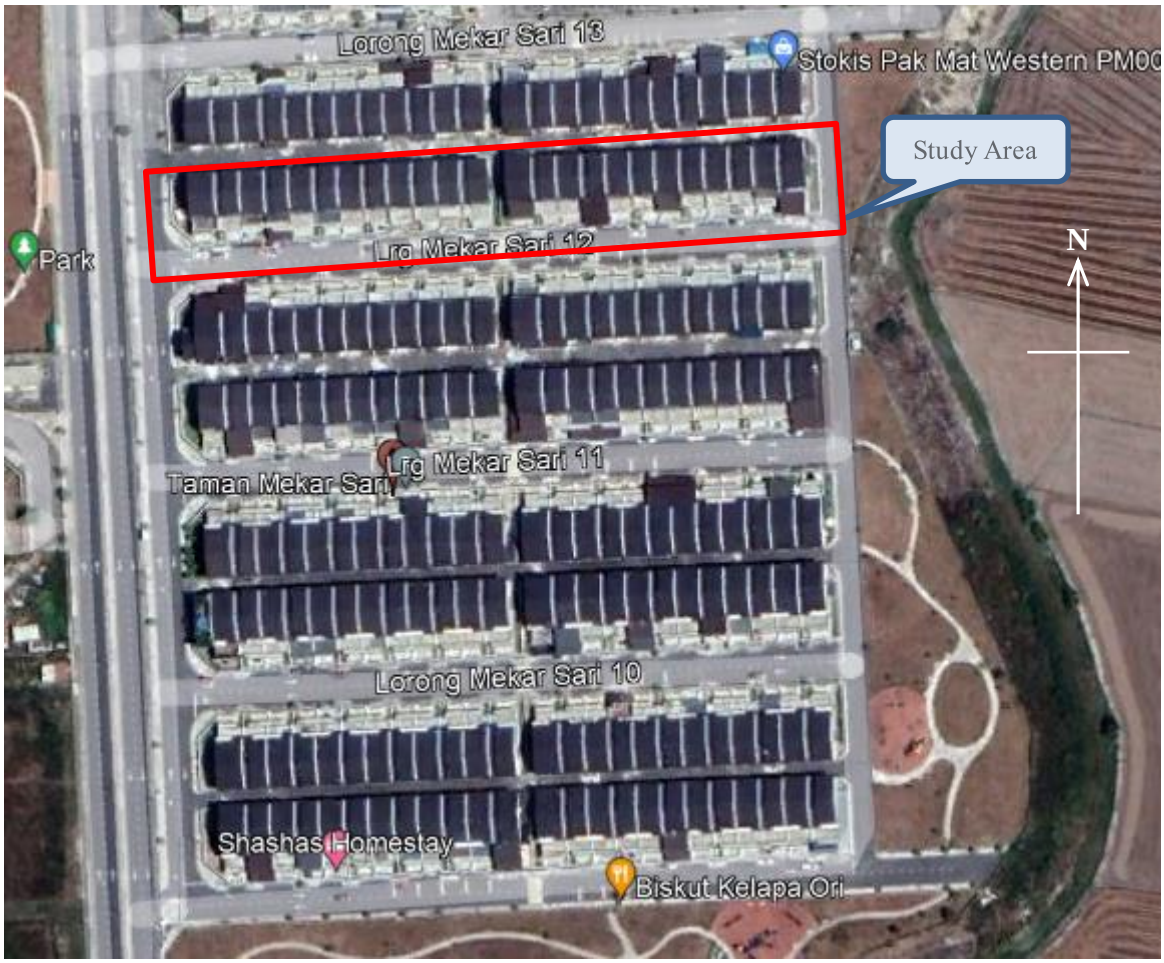


Figure 3.3: Selected Row of Terrace House (Captured from Google Earth)

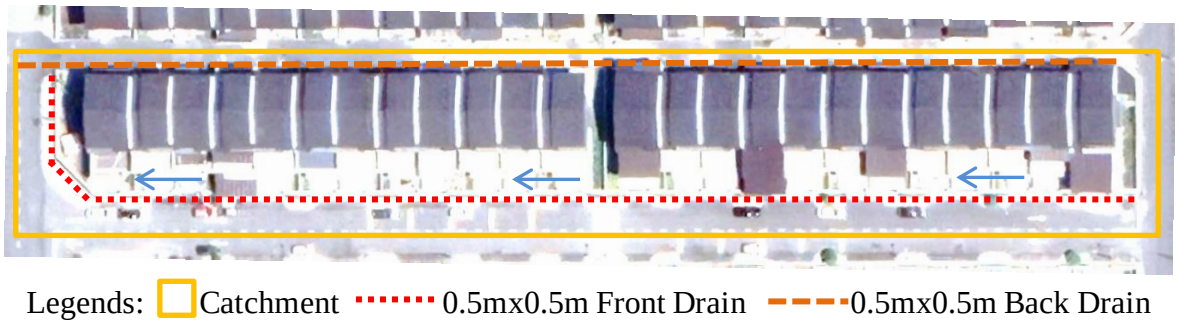


Figure 3.4: Front and Back Drains in the Selected Row of Terrace Houses

Table 3.4: Sub-Catchment Areas for the Selected Study Area

Type of Lot	Sub-Catchment	Area (m^2) per Polygon	Number of Polygon	Total Area (m^2)
Corner Lot with $20m^2$ Side Yard	Front Lot	88.70	1	88.70
	Back Lot	61.30	1	61.30
	Front Road	24.12	1	24.12
	Back Lane	20.00	1	20.00
Corner Lot with $10m^2$ Side Yard	Front Lot	83.70	3	251.10
	Back Lot	56.30	3	168.90
	Front Road	34.92	3	104.76
	Back Lane	15.00	3	45.00
Intermediate Lot	Front Lot	78.70	20	1574.00
	Back Lot	51.30	20	1026.00
	Front Road	24.12	20	482.40
	Back Lane	10.00	20	200.00
Others	East-side Road	90.00	1	90.00
	West-side Road	90.00	1	90.00
			TOTAL	4226.28 ≈ 4227

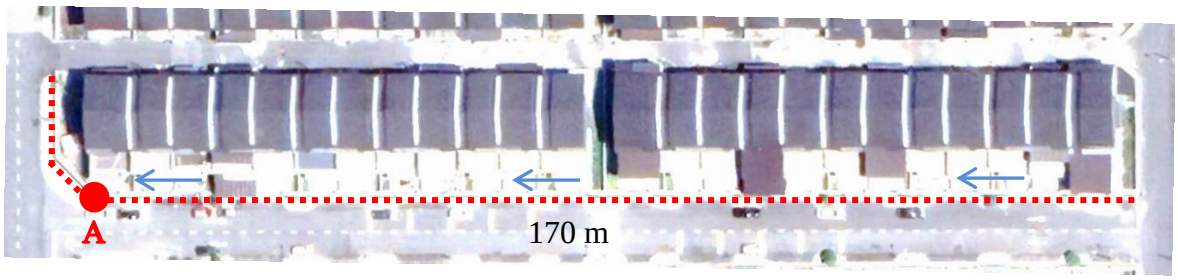
3.4 Drain

The front and back of the selected row of terrace houses are provided with a stretch of 0.5 m (H) x 0.5 m (W) drain in Figure 3.4, of which the design of the drain is presented in Appendix A. This type of the drain is taken as post-development system with uncontrolled runoff. Addition of stormwater control, in this study, in the form of orifice outlets, is taken as post-development system with controlled runoff.

This study is exploring single-, double-, and triple-orifice control in series along the various design slopes of front drain. 0%, 2%, and 4% of design slope for the front drain was exploring. The invert levels of the drain were presented at Appendix B. Figure 3.5, Figure 3.6 and Figure 3.7 depict the three scenarios of implementing orifice outlets, in which circular orifice of 100 mm, 200mm, 300 mm, 400mm and 500 mm diameters were tried. The main function of the orifice in this case study was to regulate flow through channels which separated the urban drain into one or more compartments to detain the stormwater which were:

a) Scenario 1 involved single-orifice at point A. Location A which was 170 m from the upstream end, represented the confluence point as it gathered the flow discharge from the front drain;
b) Scenario 2 consisted of double-orifice which located at the point A and B. Location B which was 86.5 m from the upstream end, was approximately at the mid-point of the terrace houses;
c) Scenario 3 decreased the distances between the orifices by increasing the number of orifices to three which located at point A, C and D. Location C and D which were 109.5 m and 50.0 m from the upstream end.

a)



b)

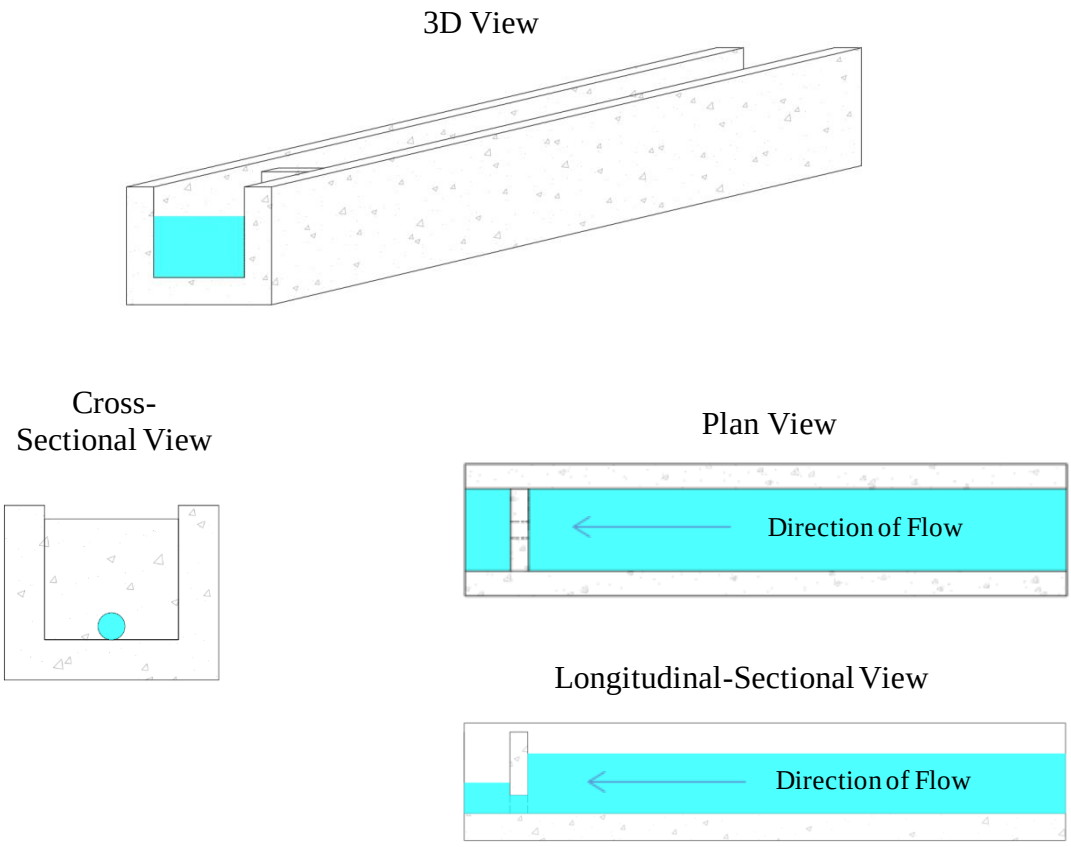
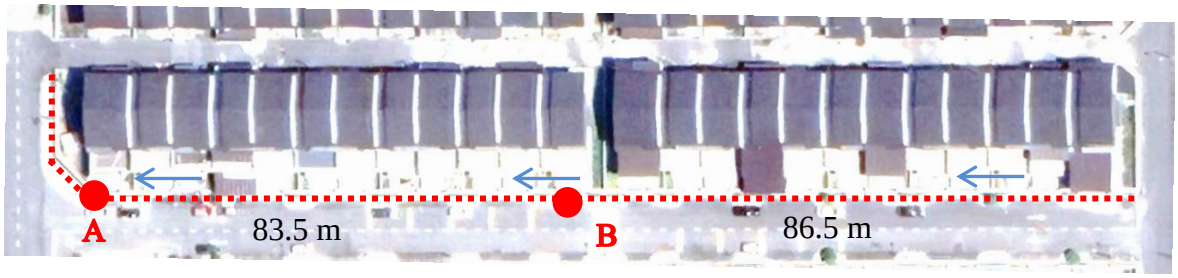


Figure 3.5: Single-Orifice Control in Series along the Front Drain, a) Location of Orifice Outlet, and b) Design of Drain with Orifice Outlet

a)



b)

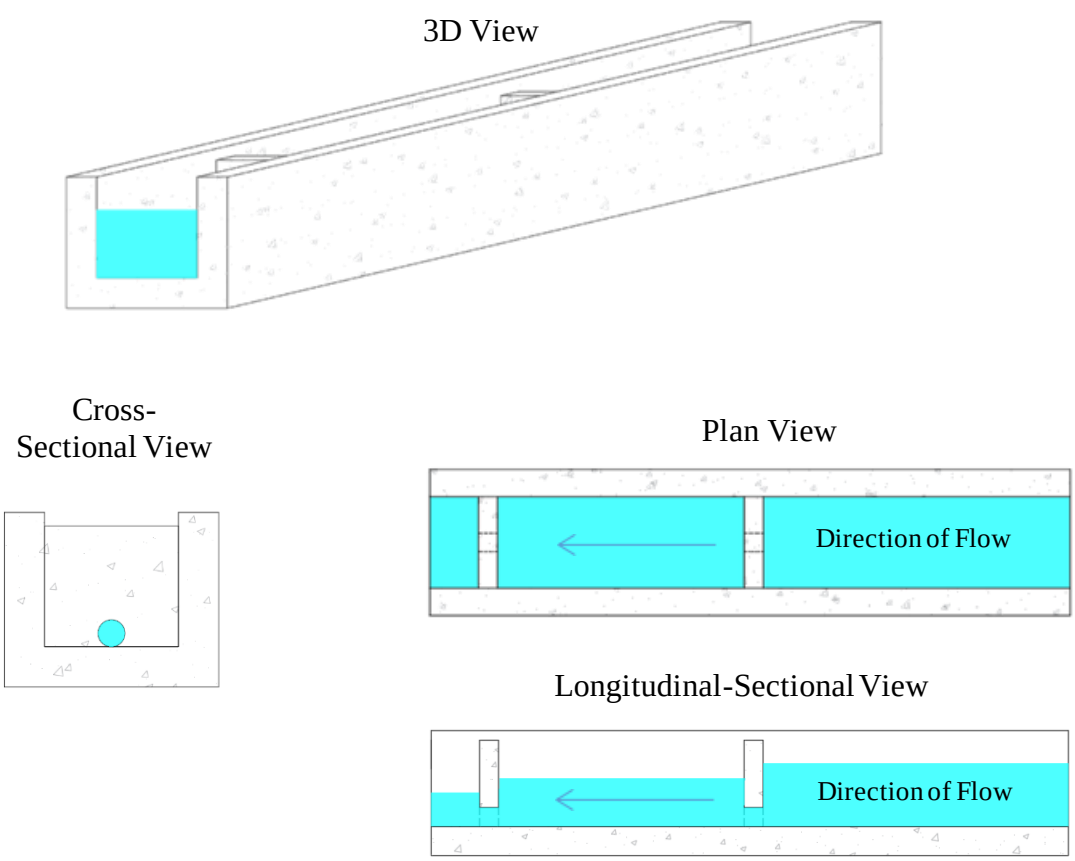
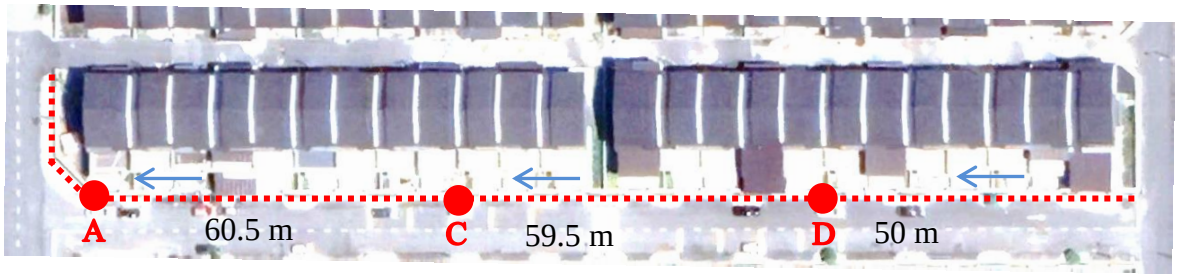


Figure 3.6: Double-Orifice Control in Series along the Front Drain, a) Location of Orifice Outlet, and b) Design of Drain with Orifice Outlets

a)



b)

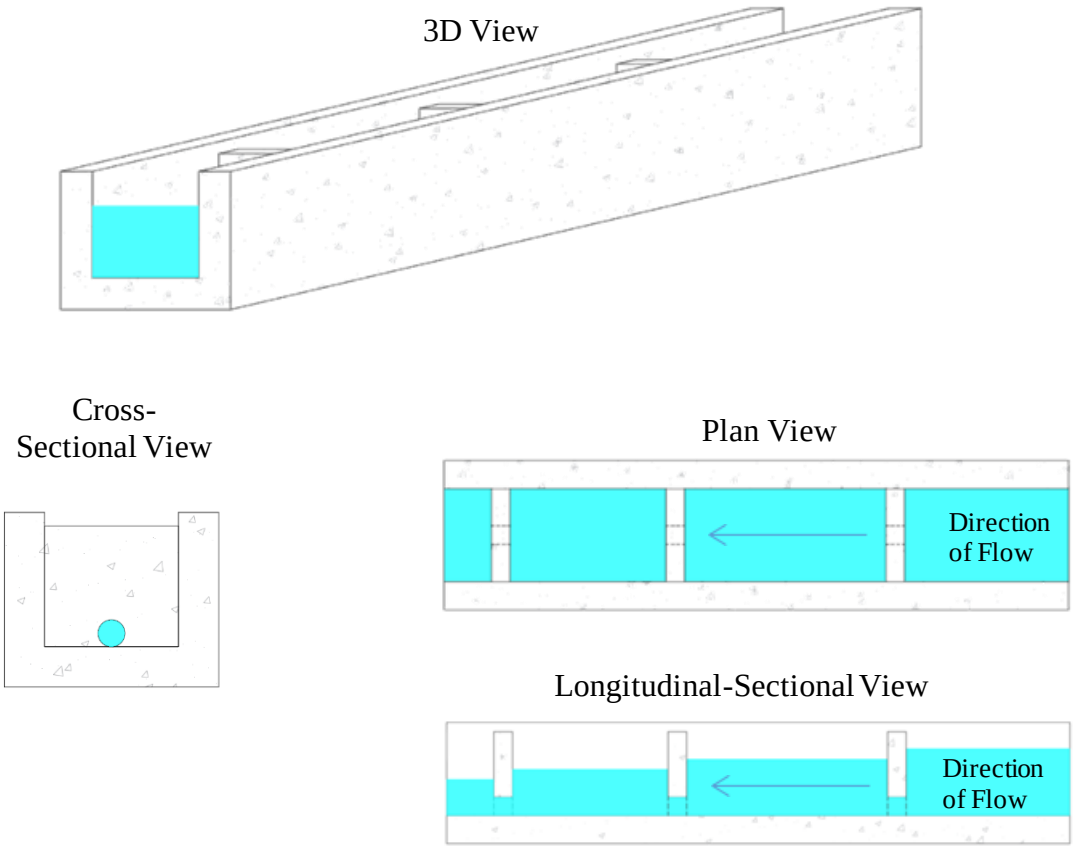


Figure 3.7: Triple-Orifice Control in Series along the Front Drain, a) Location of Orifice Outlet, and b) Design of Drain with Orifice Outlets

3.5 Model Development

This study employed SWMM 5.0 to model the anticipated orifice outlets along urban drain as flow restrictors. The software was well known and commonly used for urban water cycle modelling (Elliott and Trowsdale, 2007; Jang et al., 2007; Lee et al., 2013). SWMM 5.0 was used to assess stormwater flows with and without orifices. The model was developed following a chronological order: Rainfall → Sub-catchment → Runoff → Outfall.

In general, the runoff quantity from impervious road surface, roof and car park can be simulated as sub-catchment areas that received rainfall. A backdrop was imported as plan view of the Mekar Sari housing estate into SWMM environment to begin a new project as shown as Figure 3.8. This step was required to assist digitization of nodes and other components.



Figure 3.8: Loading of Backdrop of Study Area

After that, rainfall data were inputted. The rain gauge component was the interface to input collected design and historical extreme rainfall data into the model.

Properties of the interface was activated from the pop-out menu that displayed the time series editor, in which critical data like rainfall intensity or rainfall depth data were keyed in (Figure 3.9).

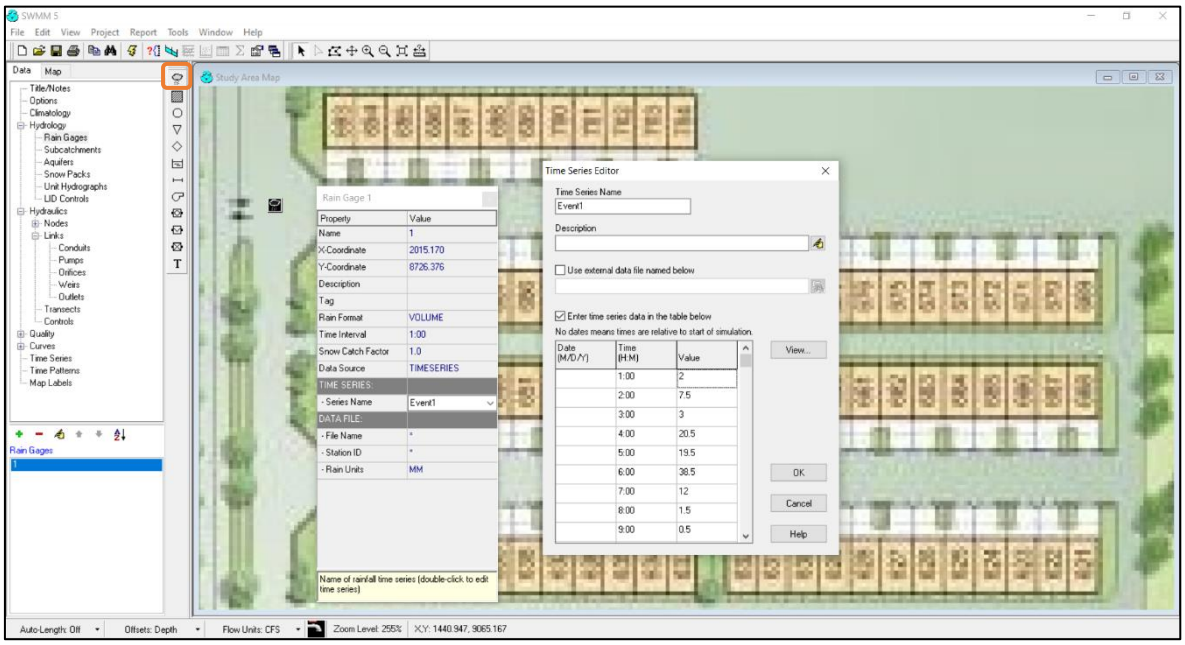


Figure 3.9: Input of Rainfall Data

The next step was to digitize sub-catchments,  to the loaded map. As illustrated in Figure 3.10, the selected catchment was divided into sub-catchments with smaller diagonal polygons. Hence, the study area was constructed as sub-catchments that received rainfall. Name, rain gauge, outlet, and area were the fields that must be completed in the sub-catchment property editor. Rain gauge was also connected to the sub-catchment.

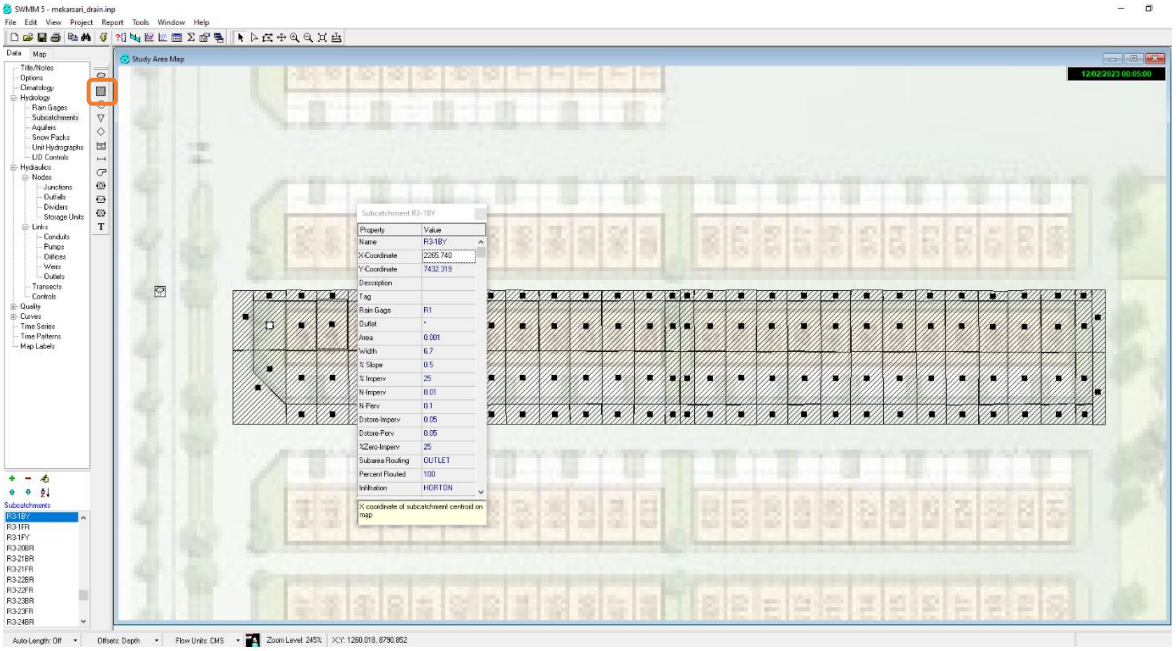


Figure 3.10: Digitization of Sub-Catchments

The component of junction was used to build a junction. The sub-catchments were connected to junctions and as such, the runoff generated from the sub-catchments are lateral flows being injected to the drain. The drain is represented via conduit , in which the conduits were connecting upstream junction to downstream junction following the direction of flow discharge (Figure 3.11). Parameters of a conduit such as name, inlet node, outlet node, shape, max depth, length, and roughness were inputted to the conduit (Figure 3.12). For the outfall, component was used, and located at the end of flow path.

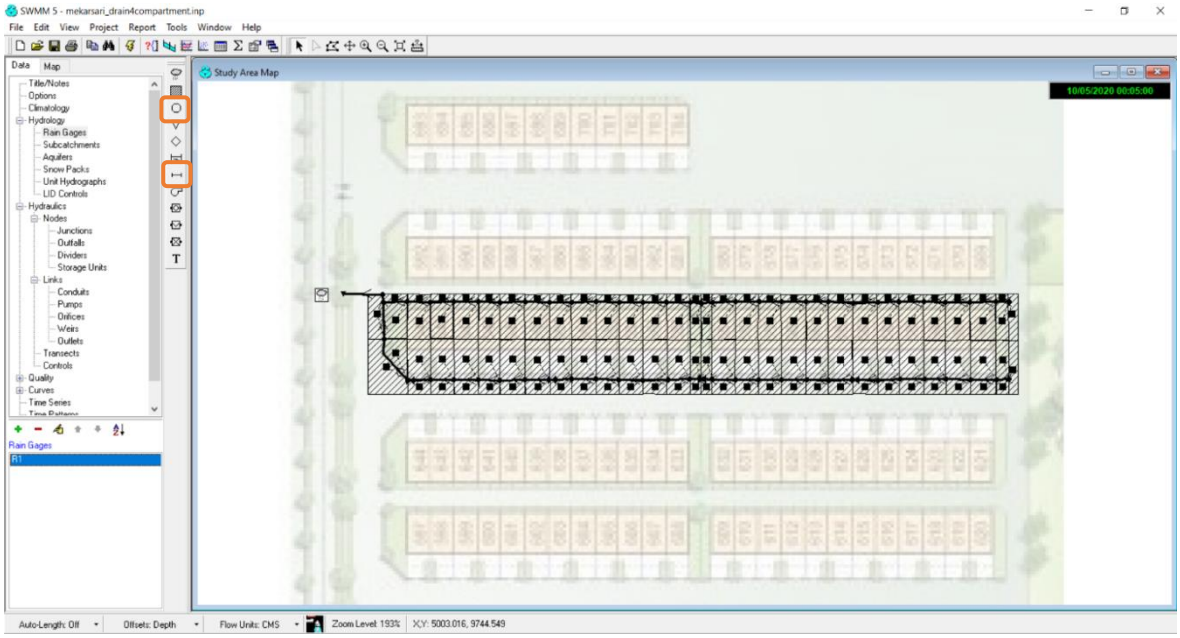


Figure 3.11: Digitization of Junction, Conduit and Outfall

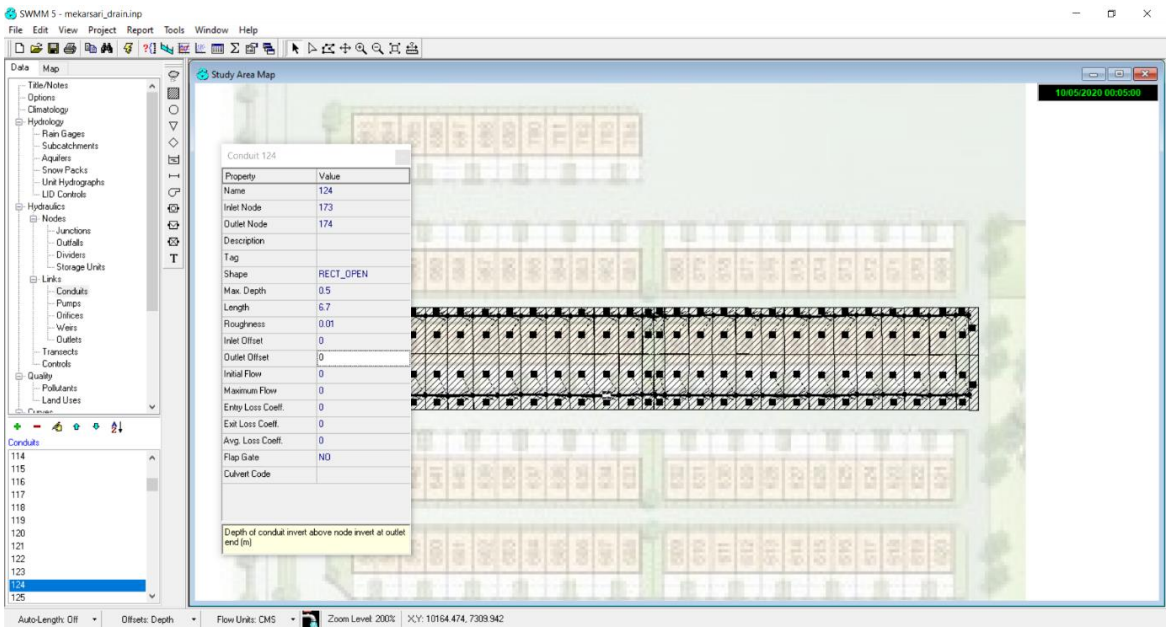


Figure 3.12: Input of Conduit Data

Orifice outlets were added to represent the flow restrictors in the post-development condition. By adding component  , orifice outlets were inserted at identified points along the drain. The model required *name, type, shape, and height* in the property editor (Figure

3.13). In these cases, additional junction was used to connect between orifice and the downstream conduit at each point of insertion. Table 3.7 shows the summary of input data for drain and orifice outlet in this investigation.

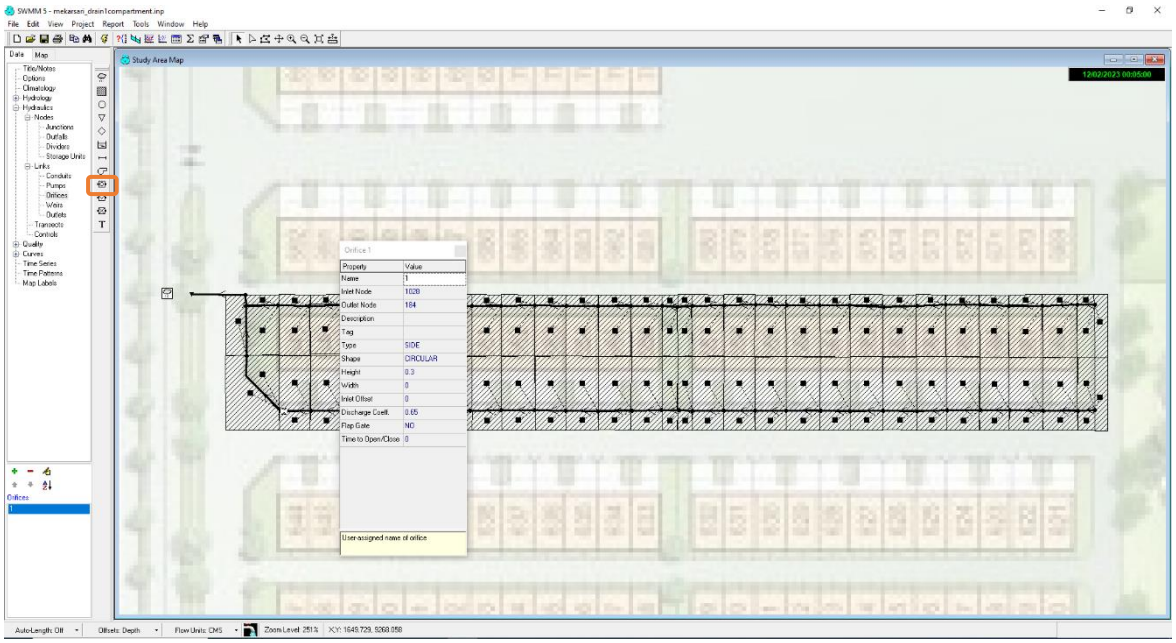


Figure 3.13: Input of Orifice Data

As mentioned in Section 3.4 before, three scenarios were created in this study. Scenarios of stormwater control measures were added into the existing drainage model as shown in Figure 3.14 to Figure 3.16. These three scenarios were required in this study to investigate the result after adding the orifice flow restrictor without physically constructing the project.

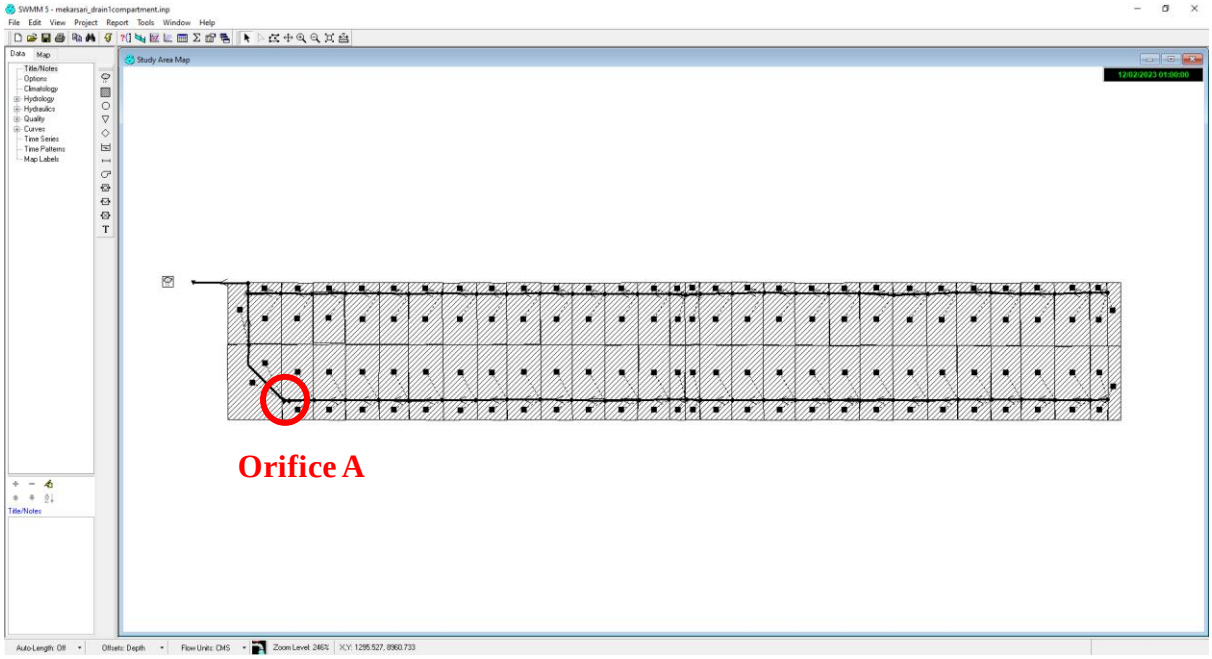


Figure 3.14: Single-Orifice Control in Series along the Front Drain (Scenario 1)

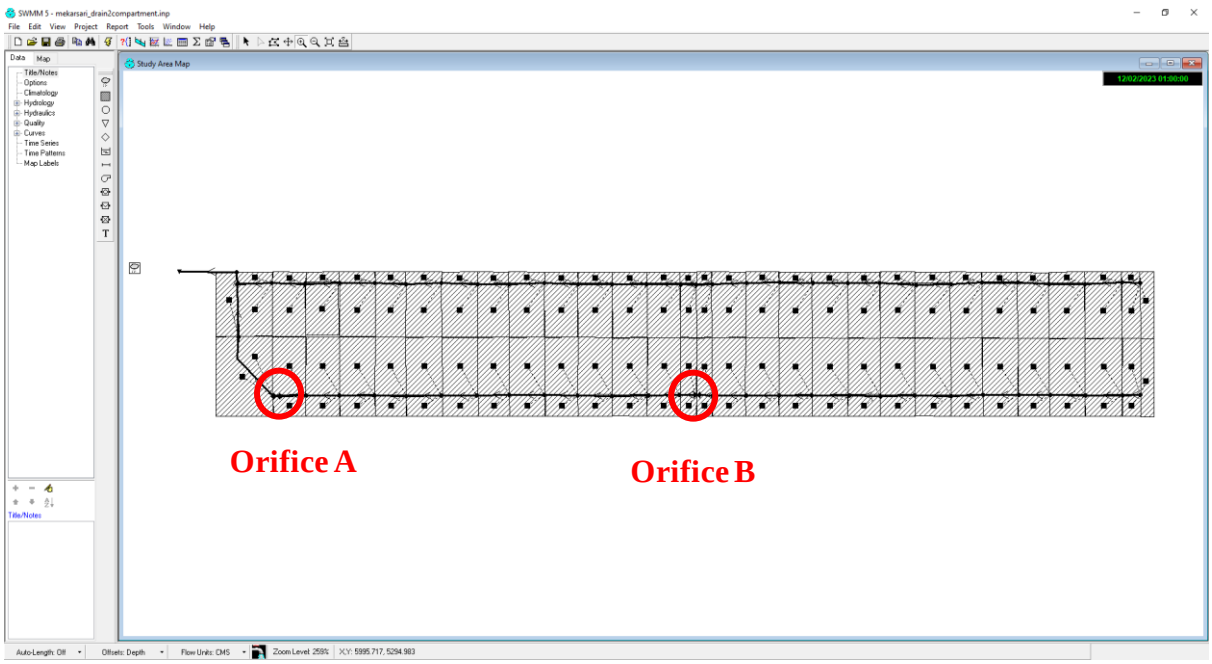


Figure 3.15: Double-Orifice Control in Series along the Front Drain (Scenario 2)

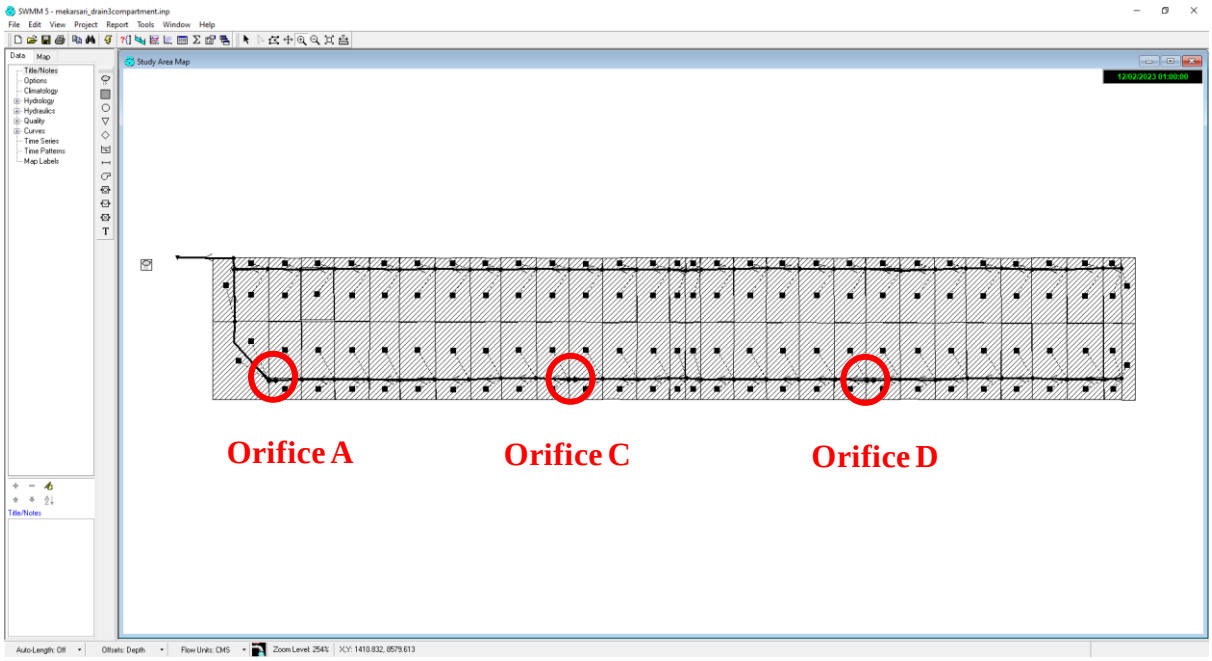


Figure 3.16: Triple-Orifice Control in Series along the Front Drain (Scenario 3)

The above-mentioned steps in the SWMM model development are summarized in Figure 3.17.

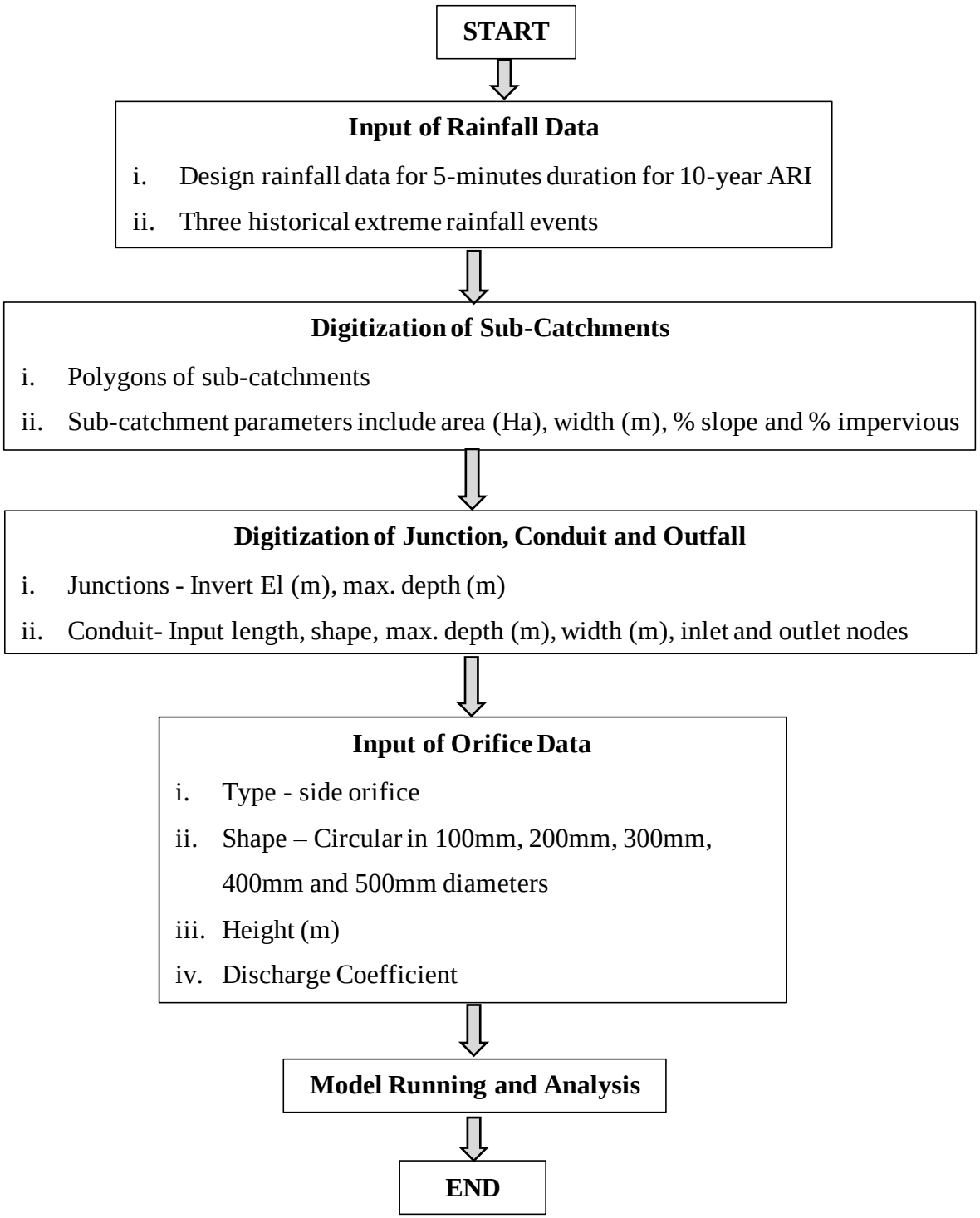


Figure 3.17: Flow Chart of Model Development

3.6 Modelling Approaches and Verification

SWMM was used to examine the hydrologic impact of urbanisation in post-development condition. Hence, post-development condition was modelled as it depicted the situation after urbanisation (Feng et al., 2020), as shown in Figure 3.18. A flow chart of the post-development condition without flow restrictor is shown in Figure 3.19. The flow mechanism or the routing of the road drainage system developed in the model were initiated from the rainfall and went to the catchment. There were 2 types of catchments, which were road and roof catchments, which received the rainfall and generated runoff. The runoff from the road catchments went to the drain with or without the flow restrictor. A model must be verified before applying it to a real scenario according to Tsai et al. (2017). Model verification was carried out to determine the accuracy of the implemented model so that the data required were inputted and the model was programmed correctly. It was a step to prevent any oversights or errors in the simulation.

In the post-development condition, Verification 1 was done between the runoff values computed by SWMM (Equation 2.4) and manual calculation using the Rational Method (Equation 2.2) for the runoff from road and roof catchments. As mentioned in Section 3.2, design rainfall of 5-min, 10-year ARI as minor system was chosen. Therefore, the runoff coefficient (C) for a terrace house is 0.80, based on MSMA 2nd Edition, as in Table 3.5. For the existing in-channel drain flow, Verification 2 was done between the runoff values computed by SWMM (Equation 2.5 - Kinematic wave equation) and manual calculation using the Manning formula (Equation 2.1). The Manning's n roughness coefficient for lined drain with rough finishes is 0.018, based on MSMA 2nd Edition, as in Table 3.6.

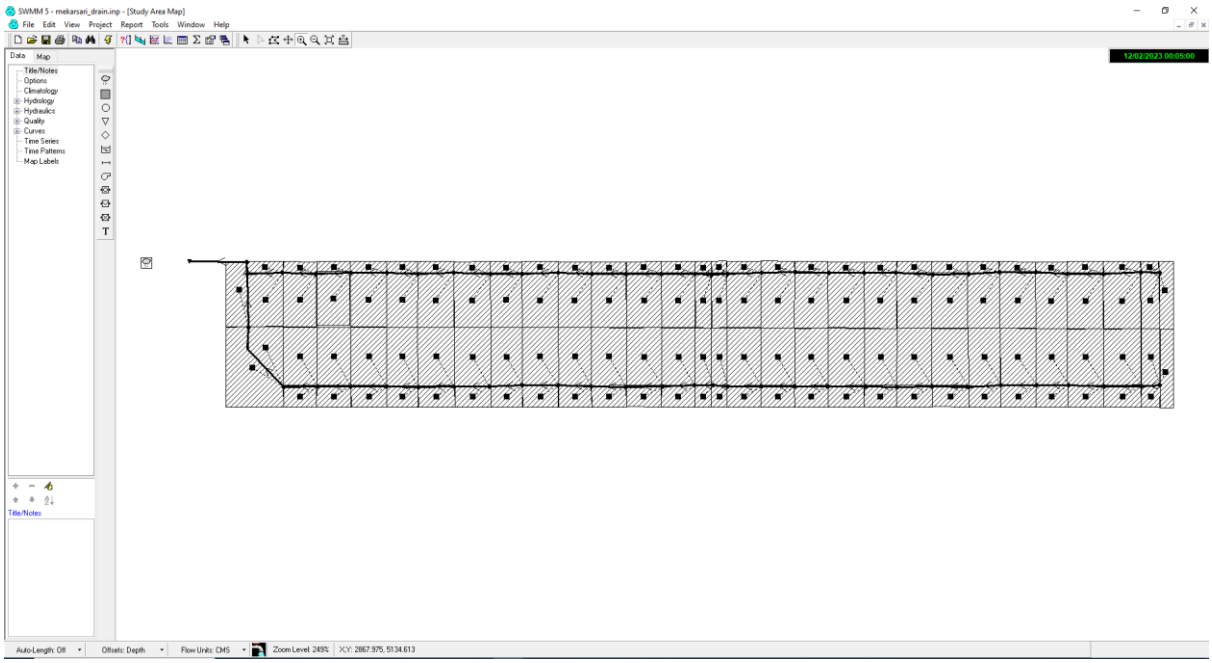


Figure 3.18: Model Developed for Post-Development Without Flow Restictor

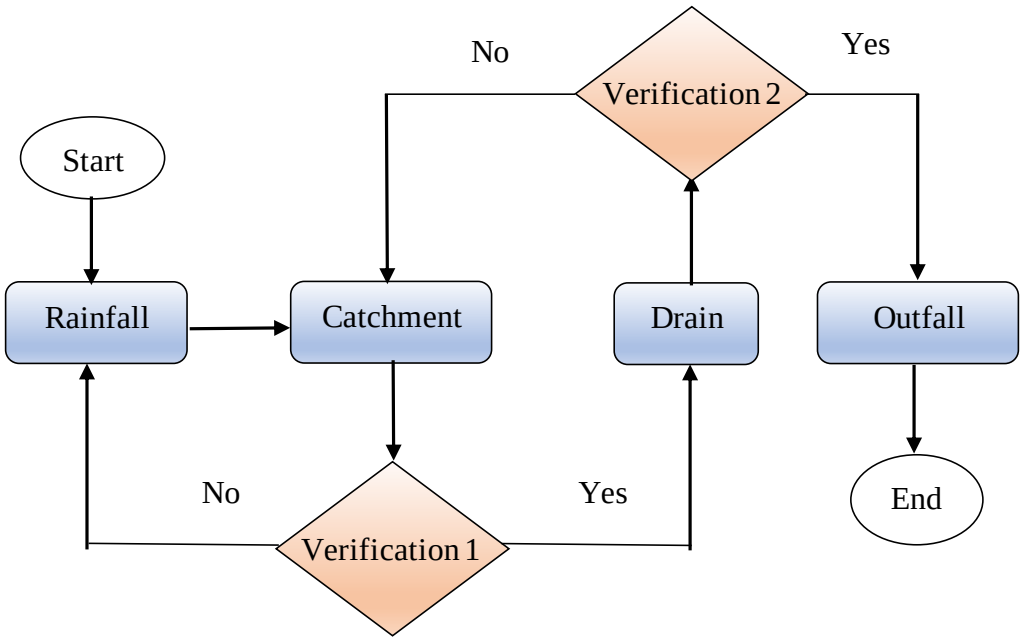


Figure 3.19: Flow Chart for Post-Development without Flow Restrictor

Table 3.5: Runoff Coefficient for Various Landuse (DID, 2012)

Landuse	Runoff Coefficient (C)	
	For Minor System (≤10 year ARI)	For Major System (> 10 year ARI)
Residential		
Bungalow	0.65	0.70
Semi-detached Bungalow	0.70	0.75
Link and Terrace House	0.80	0.90
Flat and Apartment	0.80	0.85
Condominium	0.75	0.80
Commercial and Business Centres	0.90	0.95
Industrial	0.90	0.95
Sport Fields, Park and Agriculture	0.30	0.40
Open Spaces		
Bare Soil (No Cover)	0.50	0.60
Grass Cover	0.40	0.50
Bush Cover	0.35	0.45
Forest Cover	0.30	0.40
Roads and Highways	0.95	0.95
Water Body (Pond)		
Detention Pond (with outlet)	0.95	0.95
Retention Pond (no outlet)	0.00	0.00

Note: The runoff coefficients in this table are given as a guide for designers. The near-field runoff coefficient for any single or mixed landuse should be determined based on the imperviousness of the area.

Table 3.6: Manning's Roughness Coefficient (n) for Open Drain (DID, 2012)

Drain/Pipe	Manning Roughness *n*
Grassed Drain	
Short Grass Cover (< 150 mm)	0.035
Tall Grass Cover (≥ 150 mm)	0.050
Lined Drain	
Concrete	
Smooth Finish	0.015
Rough Finish	0.018
Stone Pitching	
Dressed Stone in Mortar	0.017
Random Stones in Mortar or Rubble Masonry	0.035
Rock Riprap	0.030
Brickwork	0.020
Pipe Material	
Vitrified Clay	0.012
Spun Precast Concrete	0.013
Fibre Reinforced Cement	0.013
UPVC	0.011

Figure 3.20 shows ta flow chart for post-development condition with flow restrictor. Verification for drain flow with orifice were needed for this condition compared to the condition without flow restrictor. For each orifice along the open channel in each scenario, Verification 3 was done between the runoff values computed by SWMM and manual calculation by using orifice formula (Equation 2.3). The maximum head (h) was measured from the centroid of the opening, based on MSMA 2nd Edition, which is shown in Figure 3.21.

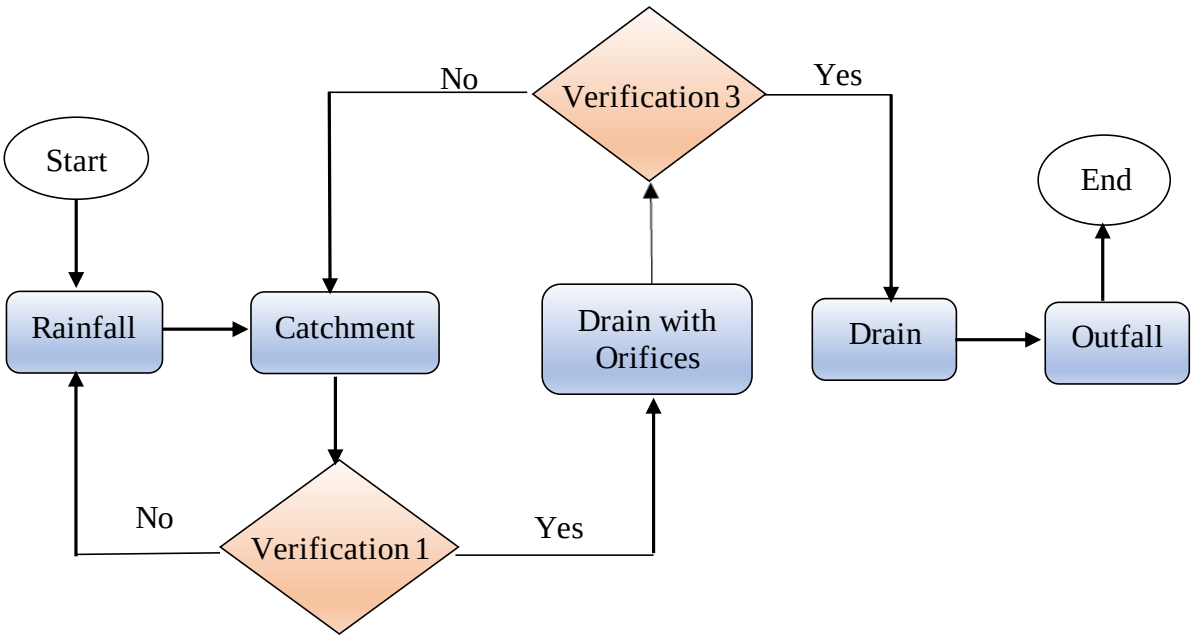


Figure 3.20: Flow Chart for Post-Development with Flow Restrictor

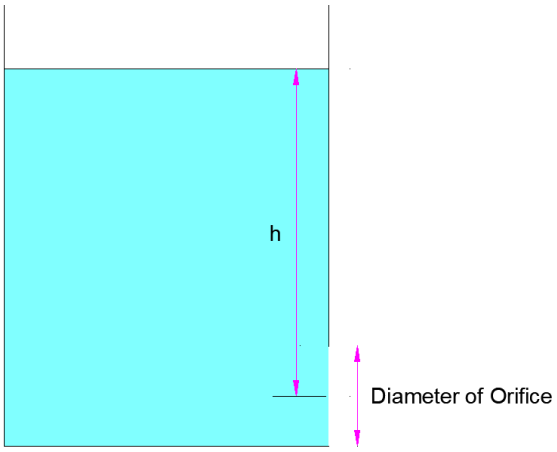
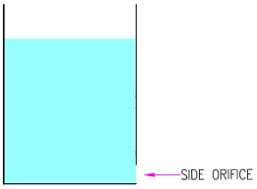


Figure 3.21: Maximum Head of Orifice Flow

Table 3.7 shows the summary of conduit and orifice data used in this study.

Table 3.7: Summary of Conduit and Orifice Input Data

Property	Value
A. Conduit	
1. Shape	Rectangular Open / Rectangular Close
Size	0.5m x 0.5m
	 Closed Rectangular Box Conduit
	 Open Rectangular Channel
2. Maximum Depth	0.5 m
3. Roughness	0.018 (Based on Table 3.6)
B. Orifice	
1. Type	Side Orifice
	
2. Shape	Circular
3. Height	100 mm, 200mm, 300 mm, 400mm and 500 mm **Diameter of the circular orifice*

This study performed model verification using three key statistical metrics which are Root Mean Squared Error (RMSE), and Nash-Sutcliffe Efficiency (NSE).

Various statistical performance metrics are used to assess model accuracy, with the Nash–Sutcliffe Efficiency (NSE) coefficient and Root Mean Square Error (RMSE) being among the most widely applied. RMSE evaluates the fit of model predictions against observations solely based on their numerical values. It is commonly used in the assessment of stage forecasting models (Sanders et al., 2022). RMSE measures the average magnitude of prediction errors, providing an absolute error estimate in the same unit as the observed variable. A lower RMSE indicates a model with fewer errors and higher predictive accuracy (Hodson, 2022).

In contrast, the NSE coefficient quantifies how well a model's simulated values match observed data by comparing the variance of residual errors to the variance of the observed data. An NSE value close to 1 indicates a highly accurate model, while values approaching or below 0 suggest poor predictive capability. According to Fang et al. (2023), model performance is classified as follows: an excellent fit for NSE > 0.65, a very good fit for $0.5 < NSE \leq 0.65$, a good fit for $0.2 < NSE \leq 0.5$, and a poor fit for NSE < 0.2.

Figure 3.22 to Figure 3.28 shows the error analysis for theoretical and modelling peak flow rate for catchment, drain, and orifice flow with 100mm to 500mm diameter respectively. The results presented in Table 3.9 summarize the model's performance for various flow types, including catchment, drain, and orifice flows with different sizes. For catchment flow, the model achieved an NSE of 0.94, indicating an excellent fit with a very low RMSE of 0.0003. Similarly, the drain flow model showed an NSE of 0.98 and an RMSE of 0.0083, also representing an excellent fit and high model accuracy. For orifice flows,

performance varied depending on the orifice size. The 100 mm orifice demonstrated an excellent fit, with an NSE of 0.96 and a very low RMSE of 0.0002. The 200 mm orifice had an NSE of 0.75 and an RMSE of 0.001, which is still classified as an excellent fit. The 300 mm orifice also showed excellent performance, with an NSE of 0.97 and an RMSE of 0.0042.

However, for the 400 mm orifice, the NSE dropped to 0.85, though it remained within the excellent category. The RMSE of 0.0107 for this orifice size was the highest among all cases. Lastly, the 500 mm orifice recorded an NSE of 0.99 and an RMSE of 0.004, indicating the most accurate and reliable performance across all orifice sizes.

In summary, the model consistently demonstrated an excellent fit for catchment, drain, and most orifice flows, confirming its reliability in predicting flow conditions.

Table 3.8: RMSE and NSE Coefficient

Type of Flow		**RMSE**	**NSE**
Catchment		0.0003	0.94
Drain		0.0083	0.98
Orifice	100 mm	0.0002	0.96
	200 mm	0.001	0.75
	300 mm	0.0042	0.97
	400 mm	0.0107	0.85
	500 mm	0.004	0.99

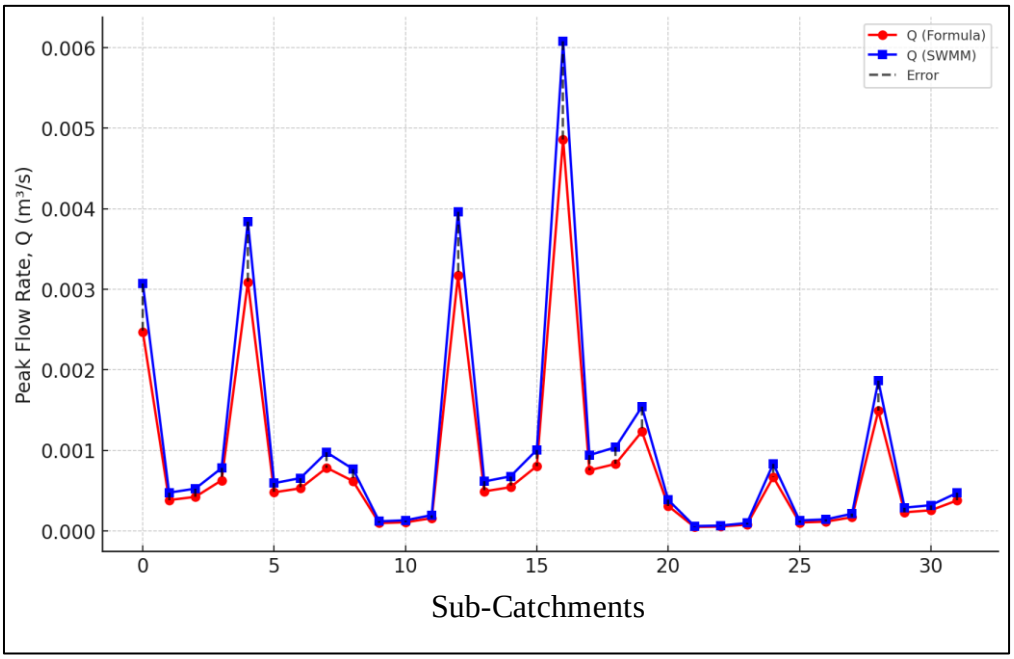


Figure 3.22: Error Analysis of Theoretical and Modelling Peak Flow Rates for Catchment Flow

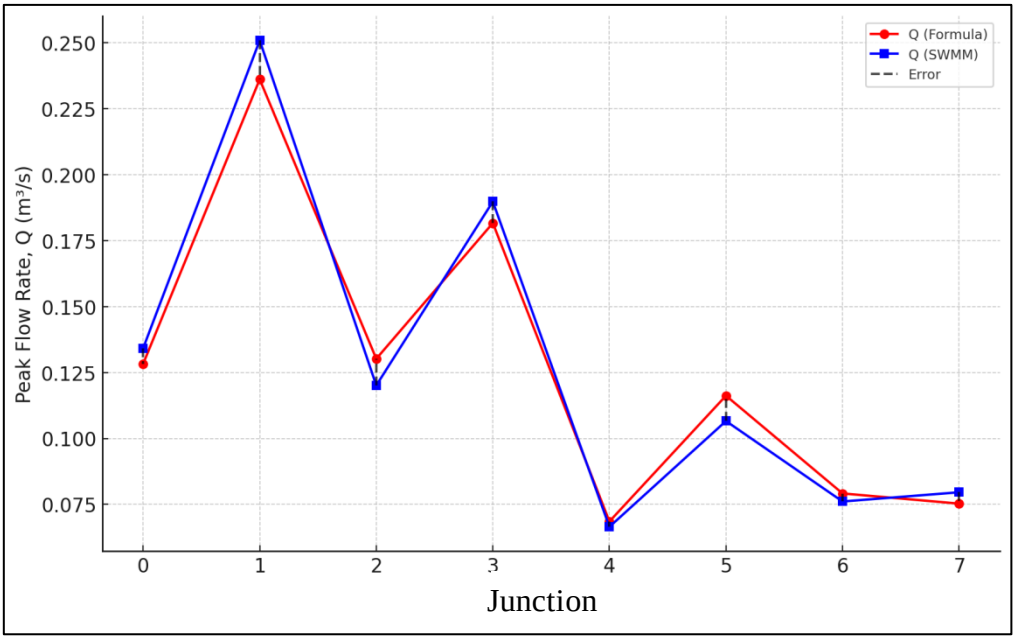


Figure 3.23: Error Analysis of Theoretical and Modelling Peak Flow Rates for Drain Flow

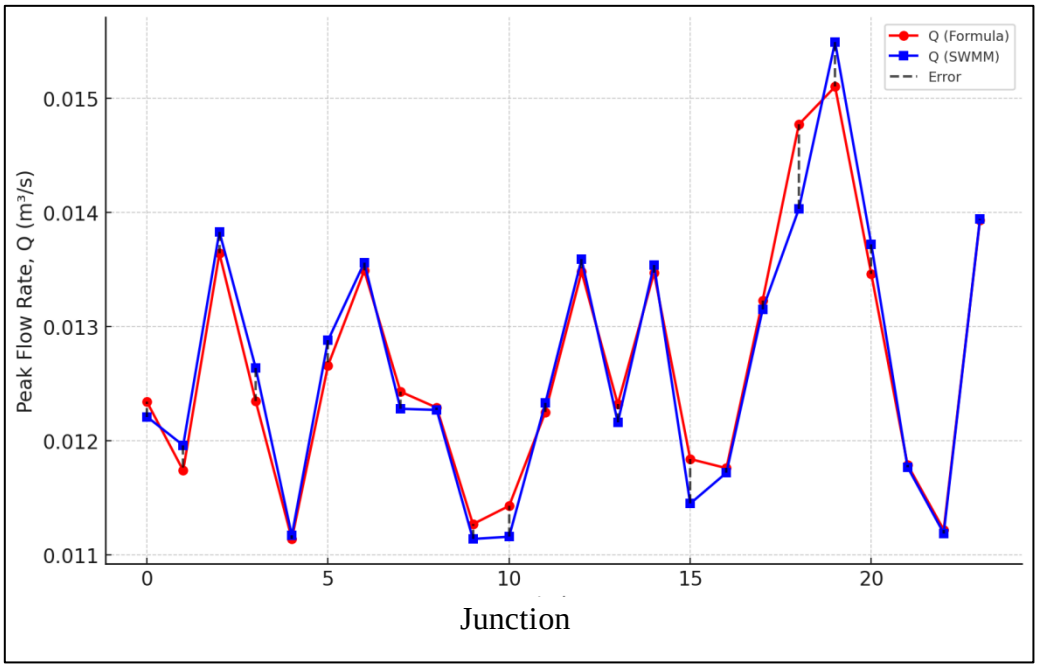


Figure 3.24: Error Analysis of Theoretical and Modelling Peak Flow Rates for Orifice Flow with 100mm Diameter

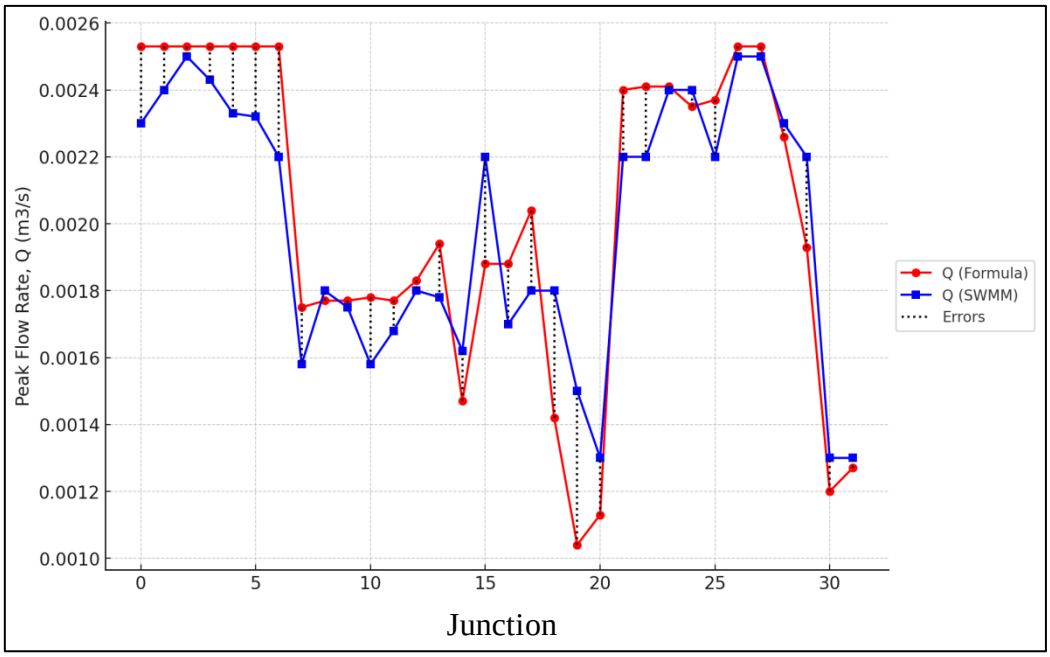


Figure 3.25: Error Analysis of Theoretical and Modelling Peak Flow Rates for Orifice Flow with 200mm Diameter

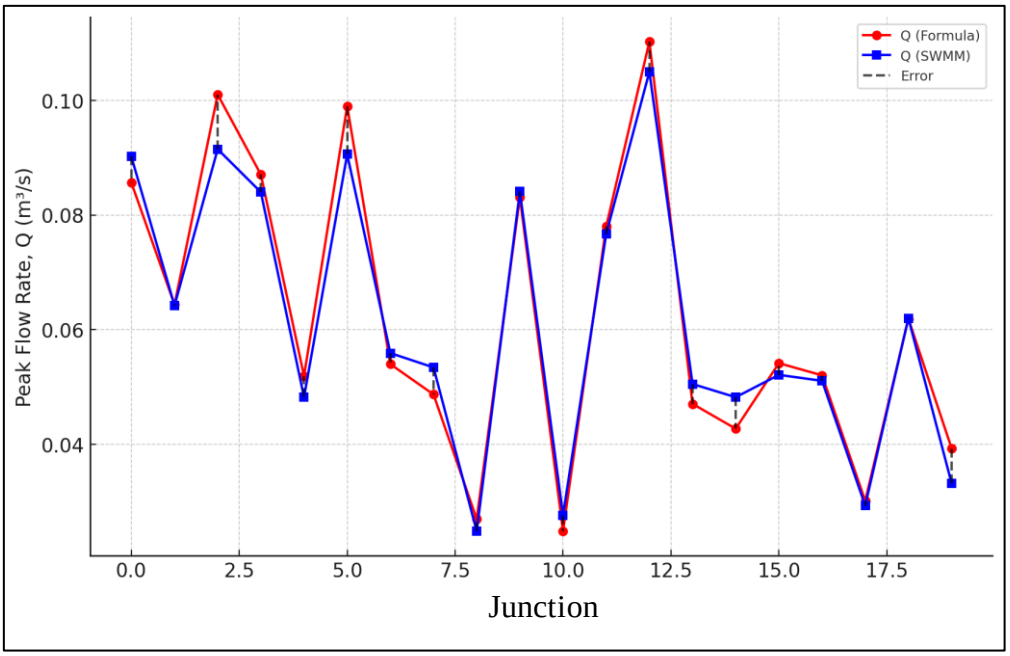


Figure 3.26: Error Analysis of Theoretical and Modelling Peak Flow Rates for Orifice Flow with 300mm Diameter

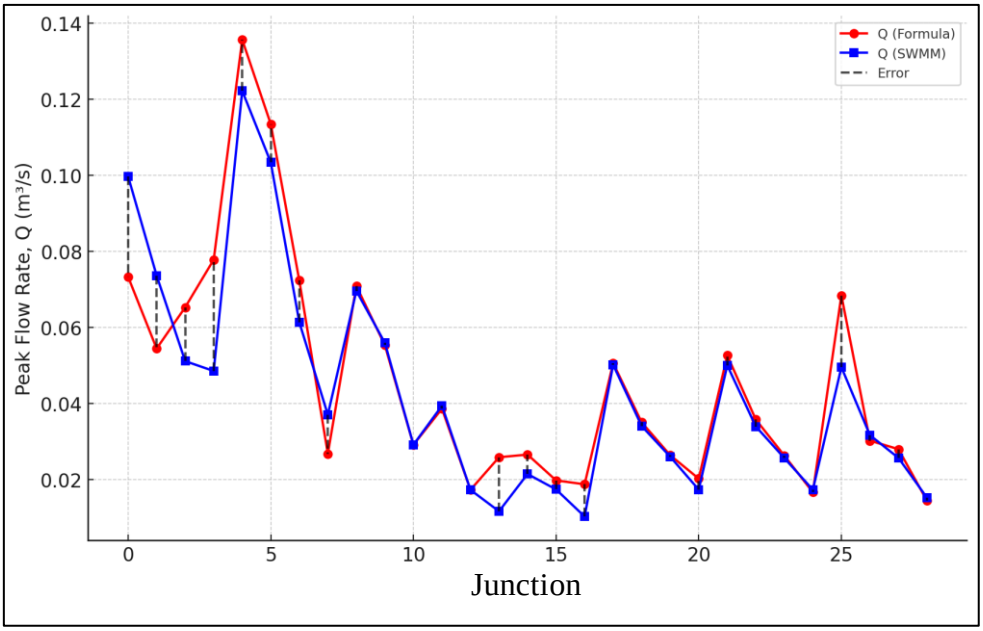


Figure 3.27: Error Analysis of Theoretical and Modelling Peak Flow Rates for Orifice Flow with 400mm Diameter

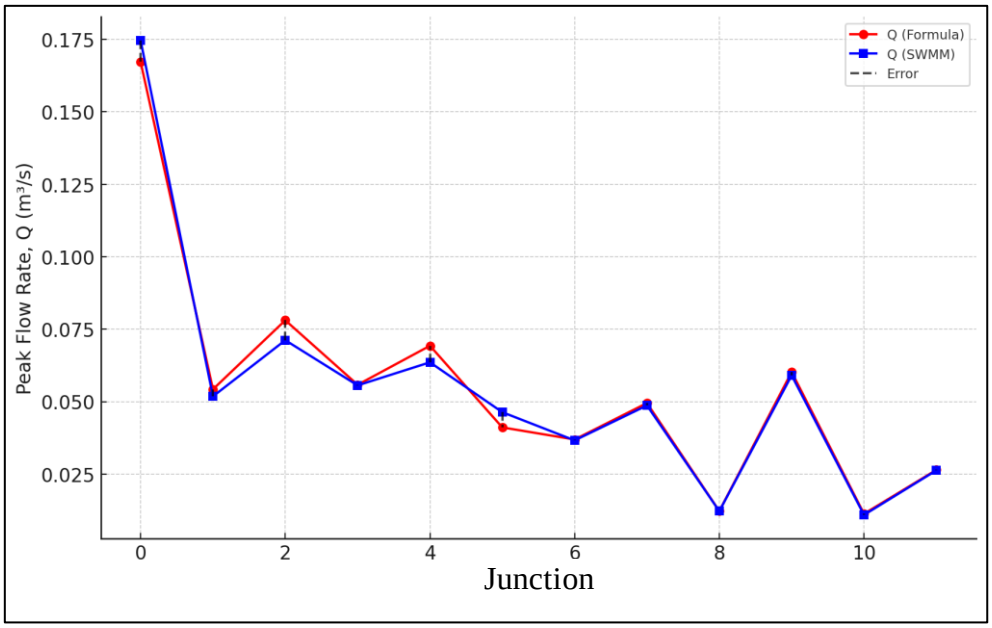


Figure 3.28: Error Analysis of Theoretical and Modelling Peak Flow Rates for Orifice Flow with 500mm Diameter

CHAPTER 4

RESULTS AND DISCUSSION

4.1 Introduction

This chapter reports the findings of the flow modelling through orifice in the urban drain with SWMM 5.0. The study area, which is a residential housing estate, was integrated with orifice to promote and practice a sustainable drainage system. This study area was subjected to 5-minute 10-year ARI design rainfall and three extreme historical rainfall events, covering three types of scenarios mentioned in Section 3.5. Therefore, drain flow under various slope and drain with orifice flow restrictors for design and historical rainfall were simulate and the results are shown in Section 4.2 to Section 4.4. The simulated results were analysed according to the devised objectives, namely the hydrological responses in drain, performances in orifice flow restrictors and the number of orifice and its location are presented in the subsequent writings.

4.2 Simulation Results of Flow Patterns

Four locations (Points A to D) in the developed SWMM model were chosen to analyse the flow patterns along the drain. In this case, hydrographs at the Locations D to A gave an overall picture about the flow rate and water level from upstream to downstream. Locations of the mentioned points in the study area is showed in Figure 4.1. The flow patterns were analysed at the four locations as those nodes were placed with an orifice plate in different scenarios. Location A was the point which flow accumulated from Locations B, C, and D. Descriptions of the flow rate and water level for Locations A, B, C and D are presented in Section 4.2.1 to Section 4.2.3. The analyses were presented in the forms of

hydrographs showing the resulted flows out of single (Scenario 1), double (Scenario 2), and triple (Scenario 3) orifices. Three different design slopes (0%, 0.2%, and 0.4%) and five different diameters of orifice (0.1m Ø, 0.2m Ø, 0.3m Ø, 0.4m Ø and 0.5m Ø) were analysed. The data of the flow and water level analyses for design and historical rainfall are documented in Appendix H.

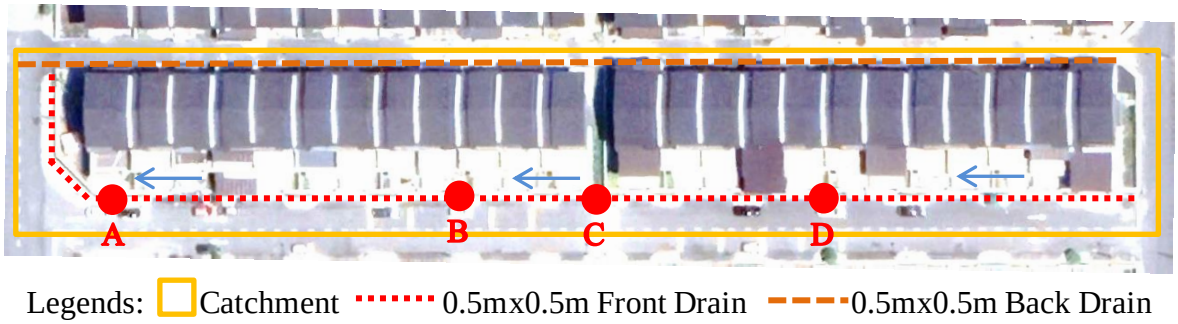


Figure 4.1: Locations Selected for Flow and Water Level Analyses

4.2.1 Drain with 0% Slope

4.2.1.1 Design Rainfall

The hydrograph representations resulting from the 5-minute, 10-year ARI design rainfall, both with and without a flow restrictor in rectangular open drain for 0% design slope, are depicted in Figure 4.2, Figure 4.4, Figure 4.6 and Figure 4.8 for flow rates, and Figure 4.3, Figure 4.5, Figure 4.7, and Figure 4.9 for water levels. Analyses of the hydrographs reveal that the peak flow rate and peak water level for all scenarios occurred at the 10-minute mark.

Notably, reduction of flow rates was found when comparing scenarios with and without the flow restrictor. Scenarios incorporating flow restrictor generally exhibited reduction in flow rates, with these effects particularly evident at the 10-minute mark.

However, it was evident that the flow rates, excluding the existing flow, increased from sub-figures (a) to (e) throughout.

Water level is an important parameter as too high the water level in the drain, overflowing shall follow. The drain's maximum height is 0.5 m. The model took that achieving 0.5 m water level as flooding. As such, it is favourable to have water levels below the drain's maximum height. While flow rates were found to reduce, the subsequent figures that are portraying the water levels with 0% drain slope, were found to increase. It is evident that in the modified water levels were found to be higher than the existing water levels. However, the increments differ from each subfigure. The increments in water levels were the highest in subfigure (a) and the lowest in subfigure (e).

a. Location A

Flow rates for all scenarios remained below the existing condition before the 35-minute mark which is shown in Figure 4.2a. Scenario 3, which included three orifice flow restrictors along the open channel, provided greater detention capacity compared to Scenarios 1 and 2. Due to the peak flow discharge at the 10-minute mark, the 0.1m diameter orifice in Scenario 2 resulted in a higher flow rate compared to Scenarios 1 and 3 at Location A. At the peak flow (10-minute mark), Scenarios 1, 2, and 3 demonstrated reductions of 90.6%, 89.1%, and 90.0%, respectively, compared to the existing condition, with Scenario 1 showing the most substantial reduction in flow rate at Location A. Post the 35-minute mark, the flow rate for all scenarios surpassed the existing condition, indicating an overall increase in flow rate. Scenarios 1, 2, and 3 experienced a 58.6% increase in water level compared to the existing condition illustrated in Figure 4.3a. This resulted in flooding, as the water level reached 0.5m, due to the combination of the 0.1m orifice flow restrictor and

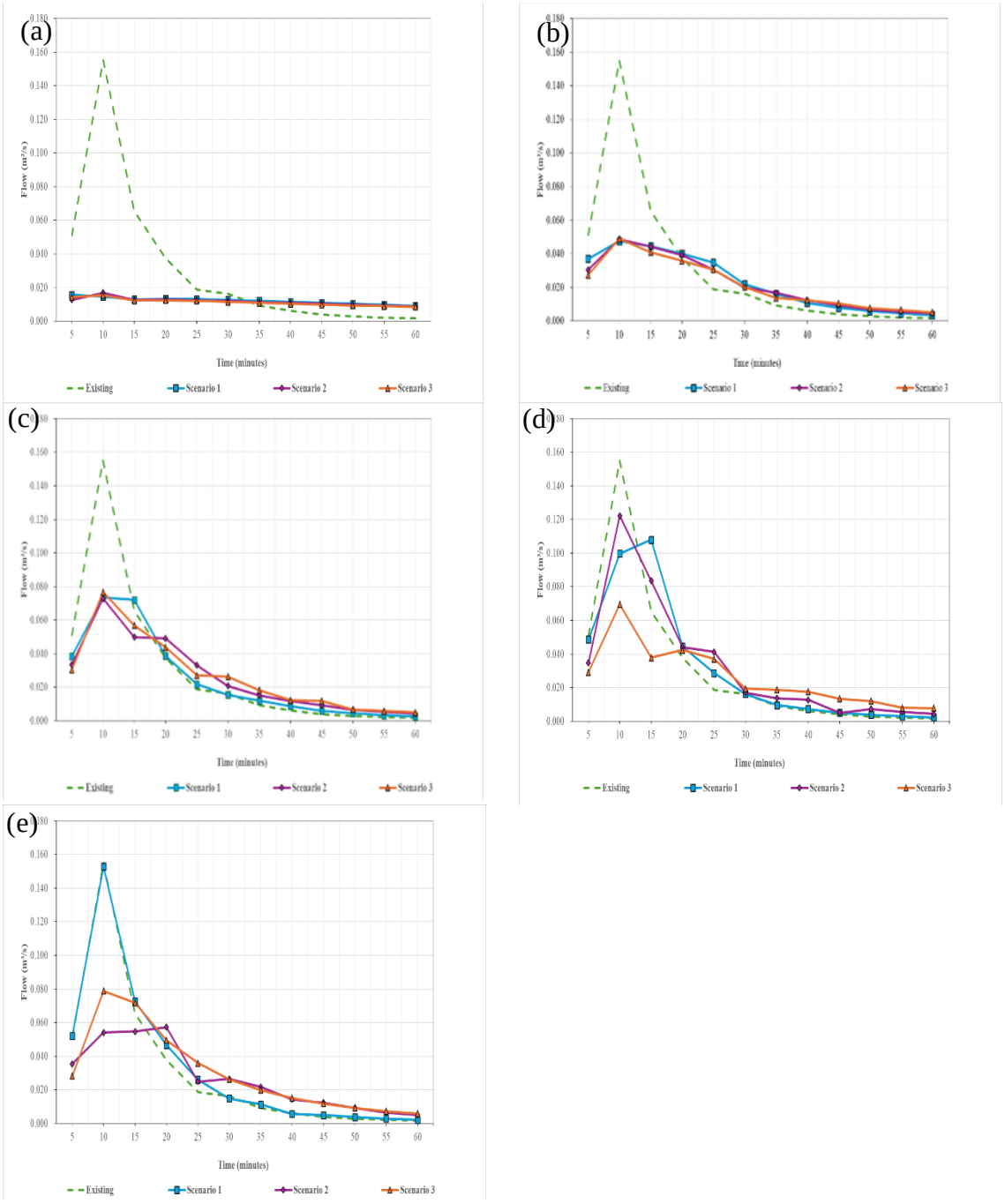


Figure 4.2: Flow Plots for Drain with 0% Slope at Location A Subjected to Design Rainfall, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

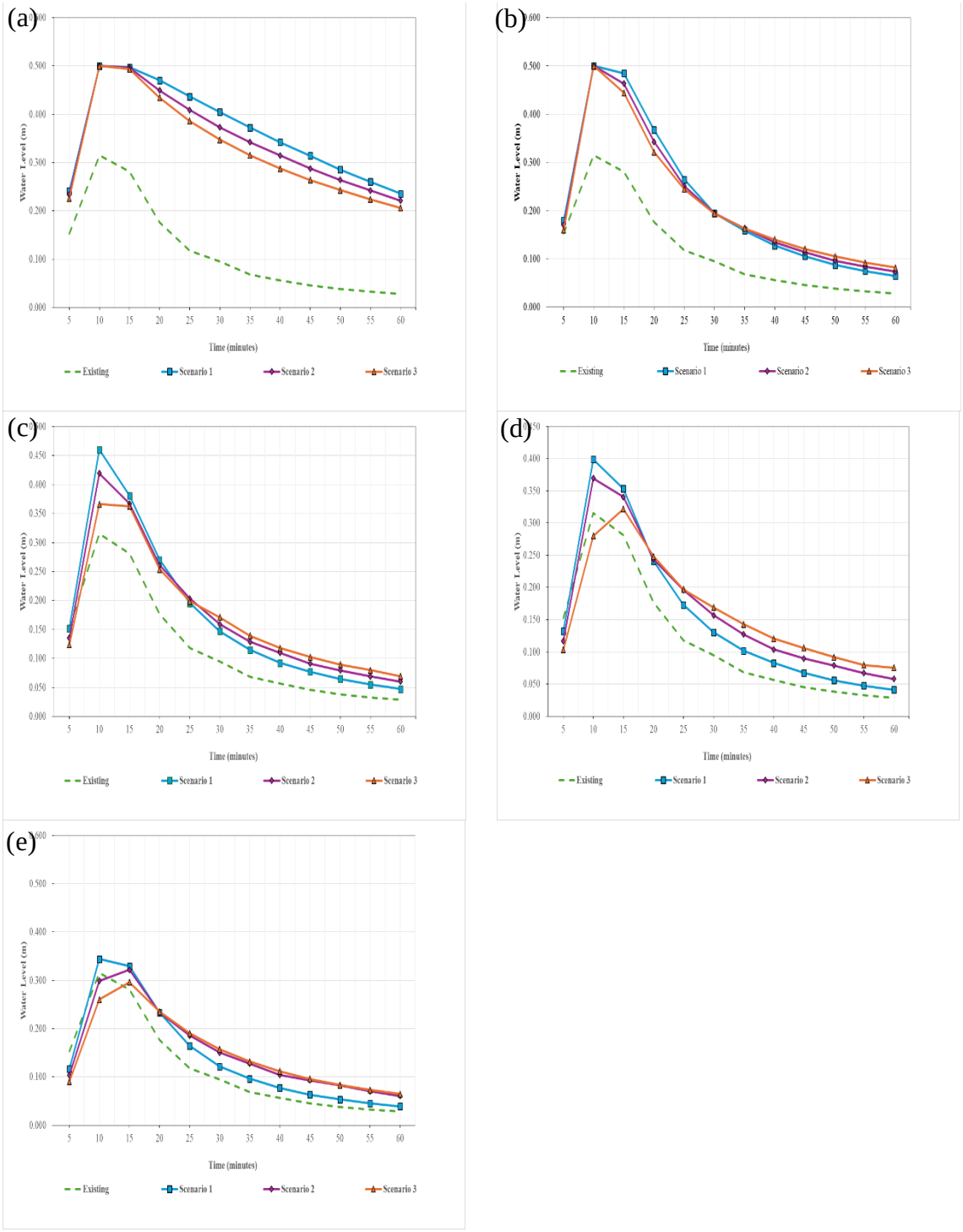


Figure 4.3: Water Level Plots for Drain with 0% Slope at Location A Subjected to Design Rainfall, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

the 0% drain design slope in the open channel.

Orifice size had increased from 0.1m to 0.2m in diameter in Figure 4.2b, the flow rates for all scenarios remained below the existing condition before the 20-minute mark and exceeded the existing condition thereafter. At the 10-minute peak flow rate, Scenarios 1, 2, and 3 demonstrated reductions of 69.4%, 68.6% and 68.5%, respectively. Scenario 1 exhibited a significant reduction in flow rate compared to Scenario 2 and 3 with the increased orifice size. In Figure 4.3b, all scenarios with a 0.2m diameter orifice flow restrictor experienced a 58.6% increase in water level compared to the existing condition, resulting in flooding as the water level reached 0.5m.

Existing conditions and the implementation of a 0.3m diameter orifice in three scenarios were illustrated in Figure 4.2c. The flow rate before the 15-minute mark was below the existing condition, except for Scenario 1. After the 15-minute mark, the flow rate exceeded the existing condition. Compared to the existing condition, Scenarios 1, 2, and 3 demonstrated reductions of 52.6, 53.0% and 50.5%, respectively. Scenario 3 exhibited the most significant reduction compared to the existing condition. In Figure 4.3c, Scenario 1, 2, and 3 with a 0.3m diameter orifice flow restrictor experienced a 45.9%, 33.1%, and 16.2% increase respectively in water level compared to the existing condition, however, no flooding occurred for all the scenario as the water level were below 0.5m.

The implementation of a 0.4m diameter orifice in three scenarios is shown in Figure 4.2d. The flow rate before the 15-minute mark remained below the existing condition, while it exceeded the existing condition after the 15-minute mark except for Scenario 1. Compared to the existing condition, Scenario 3 showed a significant reduction of 55.1%, followed by Scenario 1 with 35.7% and Scenario 2 with 21.1%. In Figure 4.3d, Scenario 3 exhibited the

smallest reduction by 11.2% in water level whereas Scenario 1 and 2 experienced an increment in water level compared to the existing condition with increments of 26.5%, and 17.2%, respectively. No flooding occurred in the long open channel with the application of the 0.4m diameter flow restrictor.

The effects of increasing the orifice size from 0.4m to 0.5m in diameter, which allowed a greater volume of water to flow were shown in Figure 4.2e. More than 90% of the flow rates exceeded the existing condition due to the larger orifice size. At the peak flow (10-minute mark), the flow rates for Scenarios 1, 2, and 3 were reduced by 14%, 65.1%, and 49.1%, respectively. Scenario 2 demonstrated the most substantial reduction in flow rate compared to the other scenarios. In Figure 4.3e, Scenario 1 showed the smallest increment by 9.1% in water level compared to the existing condition, whereas Scenario 2 and 3 had a minor reduction of 5.2%, and 17.4%, respectively. No flooding occurred in the long open channel with the application of the 0.5m diameter flow restrictor.

b. Location B

The flow rates shown in Figure 4.4a for all scenarios remained below the existing condition after the 32.5-minute mark. Due to the peak flow discharge at the 10-minute mark, the 0.1m diameter orifice in Scenario 3 resulted in a higher flow rate compared to Scenarios 2 and 3 at Location B, indicating that it did not effectively reduce downstream flow discharge. At the peak flow (10-minute mark), Scenarios 1, 2, and 3 demonstrated reductions of 82.2%, 77.7%, and 88.2%, respectively, compared to the existing condition. Scenarios 1, Scenario 2, and Scenario 3 with a 0.2m diameter orifice flow restrictor experienced a similar increment by 6.2% in water level compared to the existing condition as shown in Figure 4.5a. This resulted in flooding for all the scenarios as the water level reached 0.5m at the

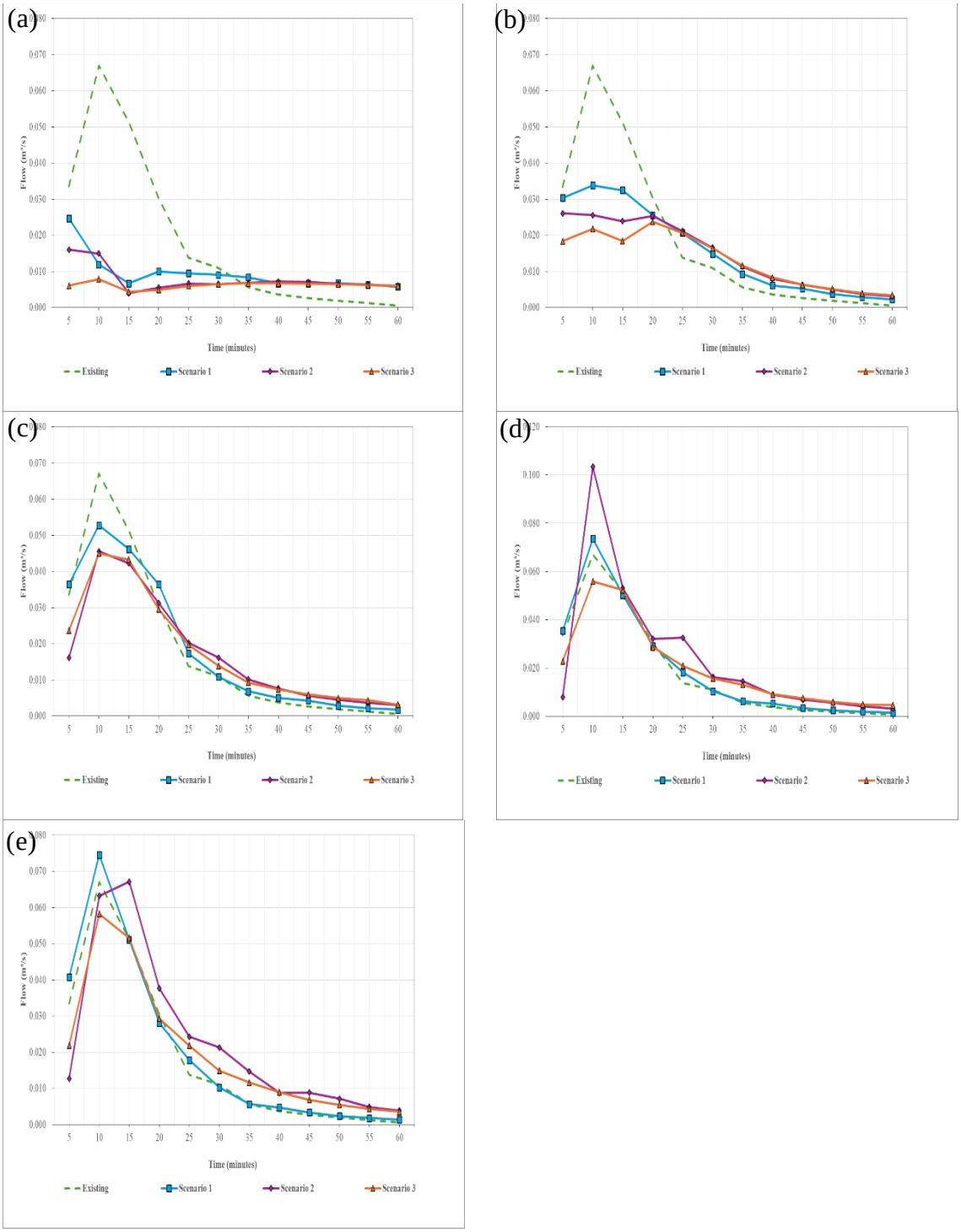


Figure 4.4: Flow Plots for Drain with 0% Slope at Location B Subjected to Design Rainfall, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

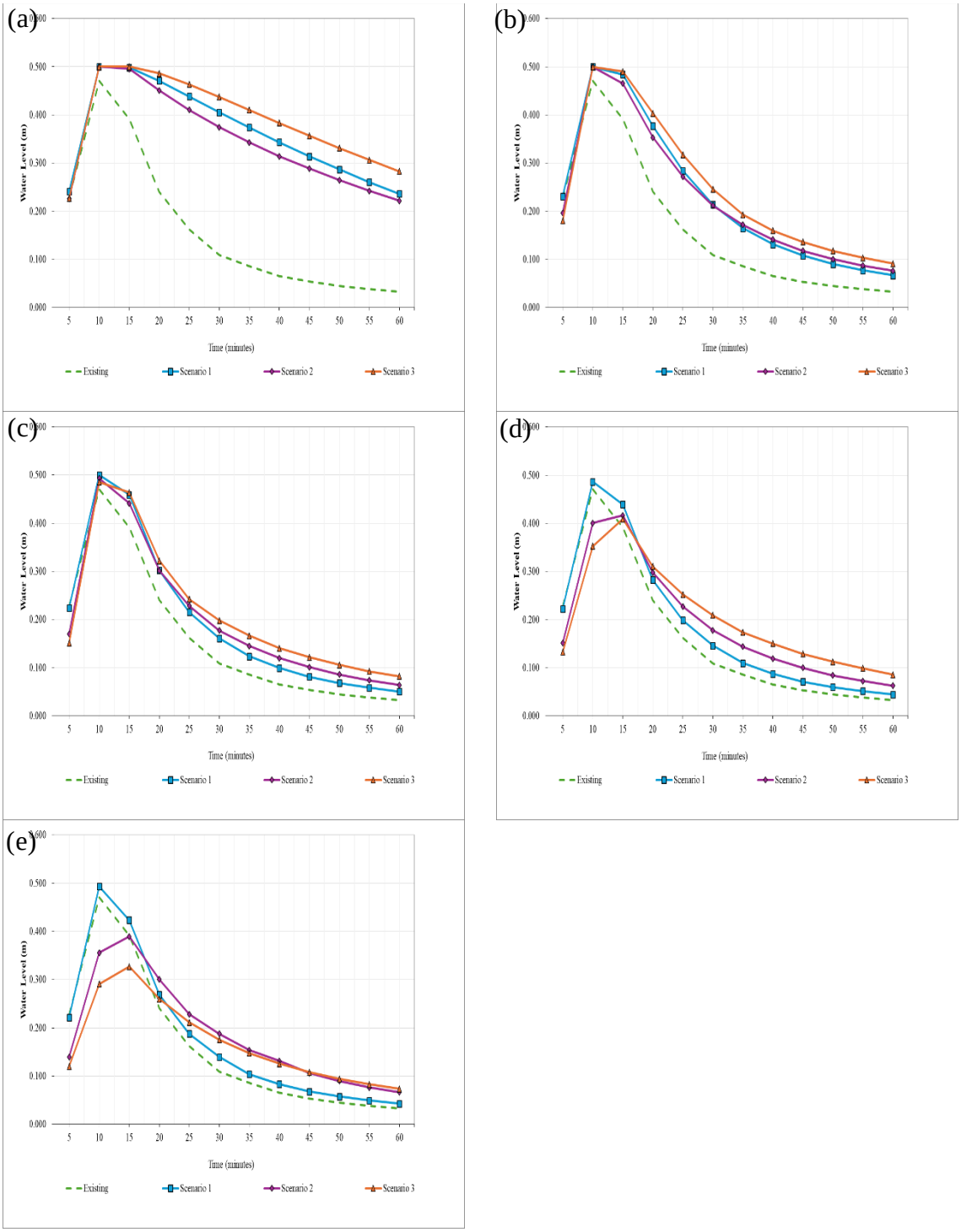


Figure 4.5: Water Level Plots for Drain with 0% Slope at Location B Subjected to Design Rainfall, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

interval of 10-minute to 15-minute mark.

The orifice size increased by 0.2m in diameter illustrated in Figure 4.4b, the flow rates for all scenarios remained below the existing condition before the 22.5-minute mark and exceeded the existing condition thereafter. At the 10-minute peak flow rate, Scenario 1 showed the least reduction followed by Scenario 2 and Scenario 3 by 49.5%, 61.8%, and 67.4% respectively at the 10-minute peak mark by comparing to the existing condition. Scenarios 1, Scenario 2, and Scenario 3 with a 0.2m diameter orifice flow restrictor experienced a similar increment by 6.2% in water level compared to the existing condition as shown in Figure 4.5b. This resulted in flooding for all the scenarios as the water level reached 0.5m at the 10-minute mark.

The existing condition and the implementation of a 0.3m diameter orifice in three scenarios as illustrated in Figure 4.4c. The flow rate before the 20-minute mark was below the existing condition. After the 15-minute mark, the flow rate exceeded the existing condition. Compared to the existing condition, Scenarios 1, 2, and 3 demonstrated reductions of 21.3%, 32.0% and 32.9%, respectively. Scenario 3 exhibited the most significant reduction compared to the existing condition. In Figure 4.5c, Scenario 1 had the highest increment followed by Scenario 2 and Scenario 3 which were 6.2%, 4.7%, and 3.1% respectively. No flooding was found for Scenario 1 at the 10-minute peak mark as the water level is above 0.5m.

Implementation of a 0.4m diameter orifice in three scenarios is shown in Figure 4.4d. The flow rate before the 20-minute mark remained below the existing condition, while it exceeded the existing condition after the 20-minute mark. Compared to the existing condition, Scenario 3 showed a reduction of 16.5%, whereas Scenario 1 and 2 experienced

an increment of 9.9% and 54.3% respectively. In Figure 4.5d, Scenario 1 showed the smallest increment in water level by 3.3% whereas Scenario 2 and 3 showed a minor reduction by 14.9%, and 25.1% by comparing to the existing condition. No flooding occurred in the long open channel with the application of the 0.5m diameter flow restrictor.

The effects of increasing the orifice size from 0.4m to 0.5m in diameter, allowed a greater volume of water to flow as shown in Figure 4.4e. More than 90% of the flow rates exceeded the existing condition due to the larger orifice size. At the peak flow (10-minute mark), the flow rates for Scenarios 2, and 3 were reduced by 5.7%, and 13.1%, respectively. Scenario 3 demonstrated the most substantial reduction in flow rate compared to the other scenarios. However, Scenario 1 shows a minor increment of 11.3% by compared to the existing condition. In Figure 4.5e, Scenario 1 showed the smallest increment in water level by 4.8% whereas Scenario 2 and 3 showed a minor reduction by 24.4%, and 38.2% by comparing to the existing condition. No flooding occurred in the long open channel with the application of the 0.5m diameter flow restrictor.

c. Location C

Flow rates for all scenarios as illustrated in Figure 4.6a remained below the existing condition prior to the 35-minute mark compared to the existing condition for 0.1m diameter of orifice. Due to the peak flow discharge at the 10-minute mark, the 0.1m diameter orifice in Scenario 1 resulted in a higher reduction in flow rate compared to Scenarios 2 and Scenario 3 at Location C, indicating that it effectively reduces flow discharge which is 95.1%, 86.0%, and 80.5% respectively. Figure 4.7a illustrates that the water level had a similar increment of 0.6% compared to the existing condition at the 10-minute mark. Floods happened for all the scenarios at the 10-minute mark as the water level reached 0.5m.

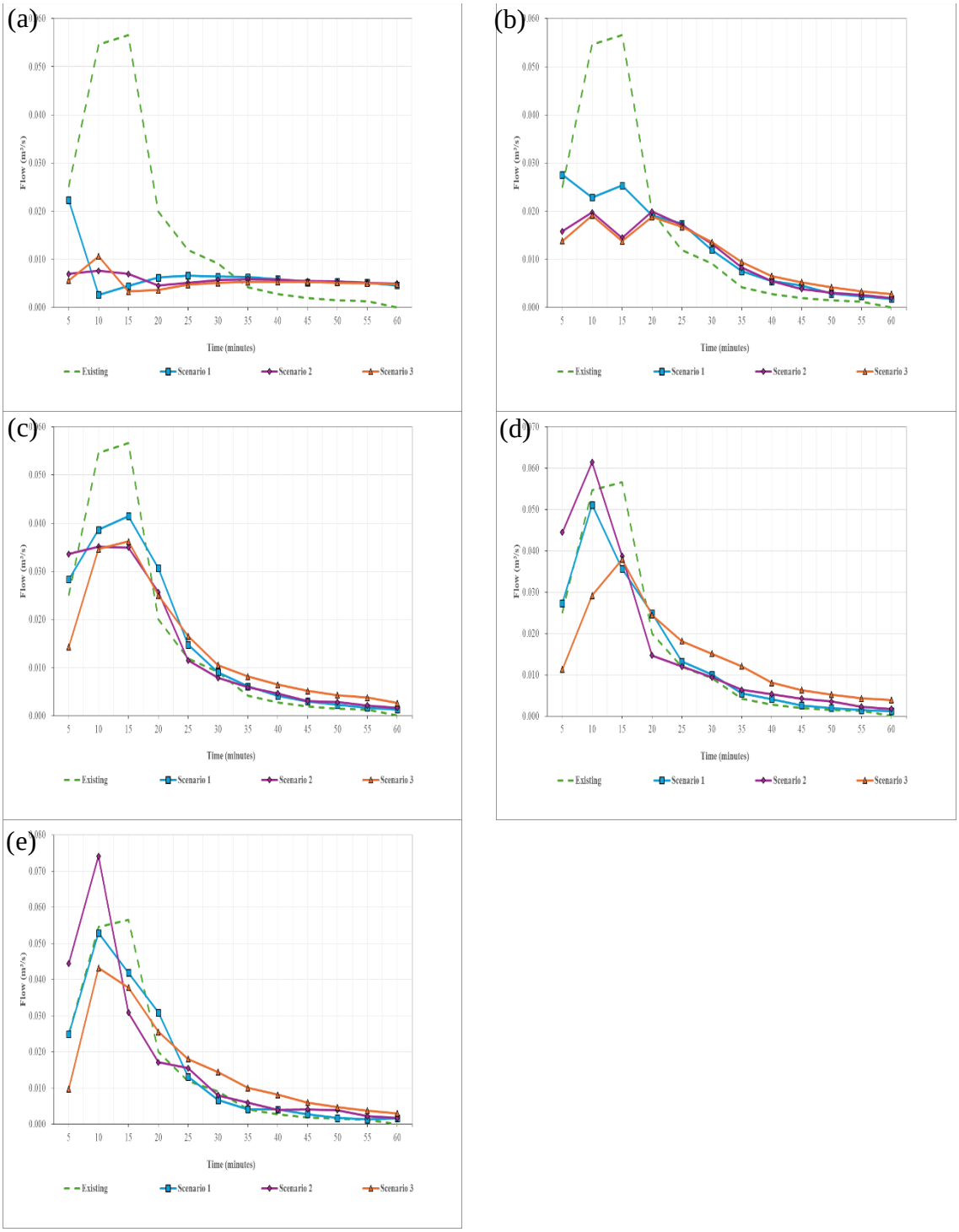


Figure 4.6: Flow Plots for Drain with 0% Slope at Location C Subjected to Design Rainfall, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

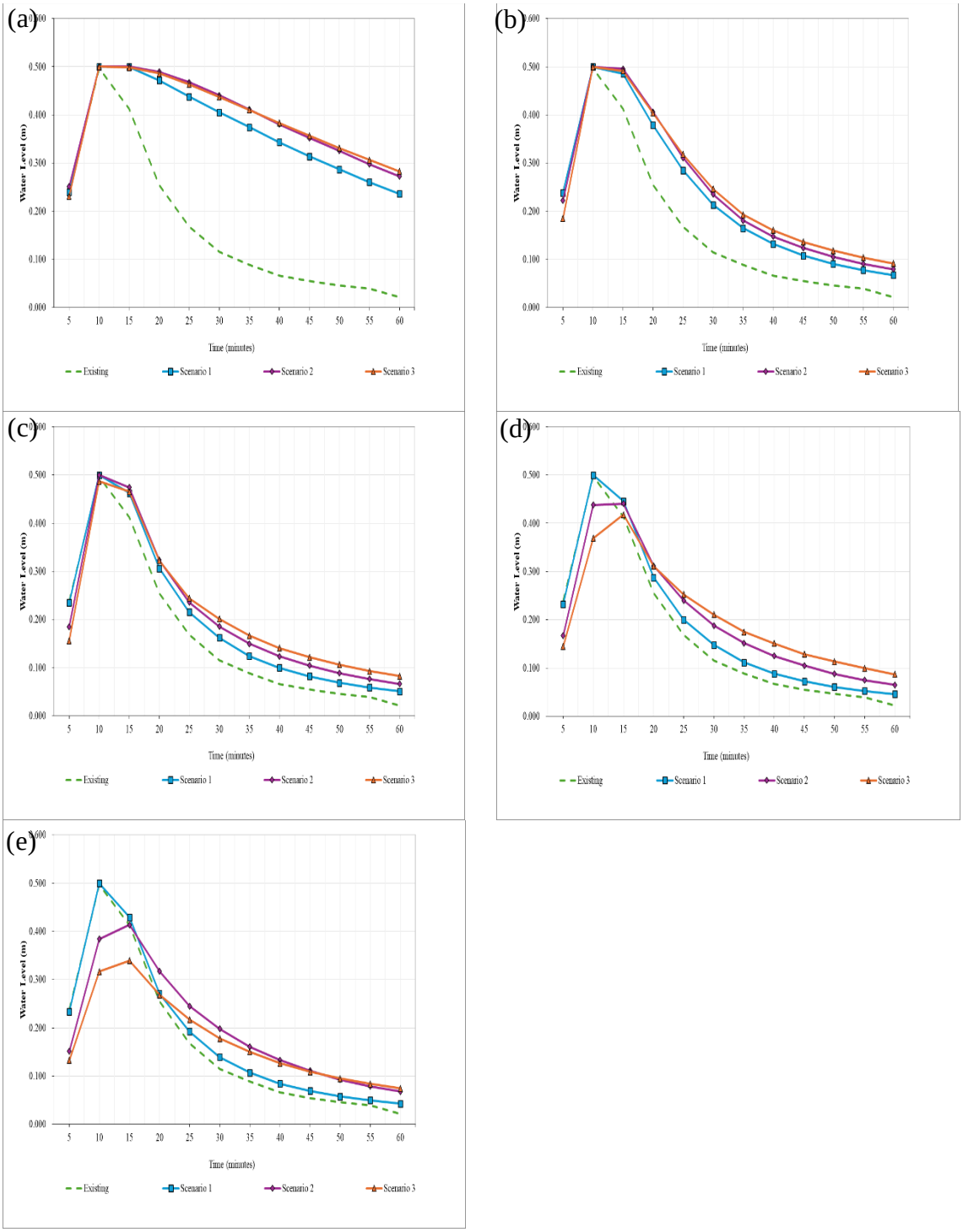


Figure 4.7: Water Level Plots for Drain with 0% Slope at Location C Subjected to Design Rainfall, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

Orifice size increased to 0.2m in diameter as illustrated in Figure 4.6b, the flow rates for Scenario 1, 2, and 3 show the reduction of the flow rate which are 58.1%, 63.8%, and 64.9% respectively by comparing to existing conditions. Figure 4.7b, illustrates that the water level had a similar increment by 0.6% compared to the existing condition at the 10-minute mark. Flood happened for Scenario 2 at the 3-hour mark of Location C.

The existing condition and the implementation of a 0.3m diameter orifice in three scenarios as shown in Figure 4.6c. The flow rate before the 20-minute mark was below the existing condition, except for Scenario 1. After the 20-minute mark, the flow rate exceeded the existing condition. Compared to the existing condition, Scenarios 1, 2 and 3 demonstrated reductions of 29.3%, 35.6%, and 36.6%, respectively. Scenario 3 exhibited the highest reduction compared to the existing condition among the scenarios. In Figure 4.7c, Scenario 3 showed a reduction in water level by 2.0% whereas Scenario 1 and 2 show a similar minor increment were 0.6% when compared to the existing condition. Flooding happened for Scenario 1 and 2 at the 10-minute peak flow whereas no floods were found in Scenario 3.

Implementation of a 0.4m diameter orifice in three scenarios is shown in Figure 4.6d. The flow rate before the 25-minute mark remained below the existing condition, while it exceeded the existing condition after the 25-minute mark. Compared to the existing condition, Scenario 3 showed a significant reduction of 46.5%, followed by Scenario 1 with 6.3% whereas Scenario 2 shows a minor increment of 12.4% at the 3-hour mark. In Figure 4.7d Scenarios 2 and 3 showed a reduction in water level by 11.8% and 25.8% respectively whereas Scenario 1 shows a minor increment were 0.6% when compared to the existing condition. Flooding happened in Scenario 1 at the 10-minute peak flow whereas no floods were found in Scenario 2 and 3.

The effects of increasing the orifice size from 0.4m to 0.5m in diameter, allowed a greater volume of water to flow as illustrated in Figure 4.6e. At the peak flow (10-minute mark), the flow rates for Scenarios 1, and Scenario 3 were reduced by 3.1%, and 20.8% respectively whereas Scenario 2 shows a minor increment of 35.8% by comparing to the existing condition. In Figure 4.7e, Scenario 2 and 3 showed a reduction in water level by 22.6% and 36.2% respectively whereas Scenario 1 shows a minor increment were 0.6% when compared to the existing condition. Flooding happened for Scenario 1 at the 10-minute peak flow whereas no floods were found in Scenario 2 and 3.

d. Location D

The flow rates for all scenarios remained below the existing condition prior to the 30-minute mark as illustrated in Figure 4.8a. Due to the peak flow discharge at the 10-minute mark, the 0.1m diameter orifice in Scenario 2 resulted in a higher flow rate compared to Scenarios 1 and 3 at Location D, indicating that it did not effectively reduce upstream flow discharge. At the peak flow (10-minute mark), Scenarios 1, 2, and 3 demonstrated reductions of 70.4%, 64.5%, and 87.7%, respectively, compared to the existing condition. Post the 30-minute mark, the flow rate for all scenarios surpassed the existing condition, indicating an overall increase in flow rate. Figure 4.9a illustrates that all the scenario shows no reduction in water level by comparing to existing condition. Hence, flooding was found in all the scenarios at the 10-minute peak flow as the water level of had reached 0.5m which is the maximum water level of the open channel.

The orifice size increased from 0.1m to 0.2m in diameter, and the flow rates for all scenarios remained below the existing condition before the 25-minute mark and exceeded the existing condition thereafter as shown in Figure 4.8b. At the 10-minute peak flow rate,

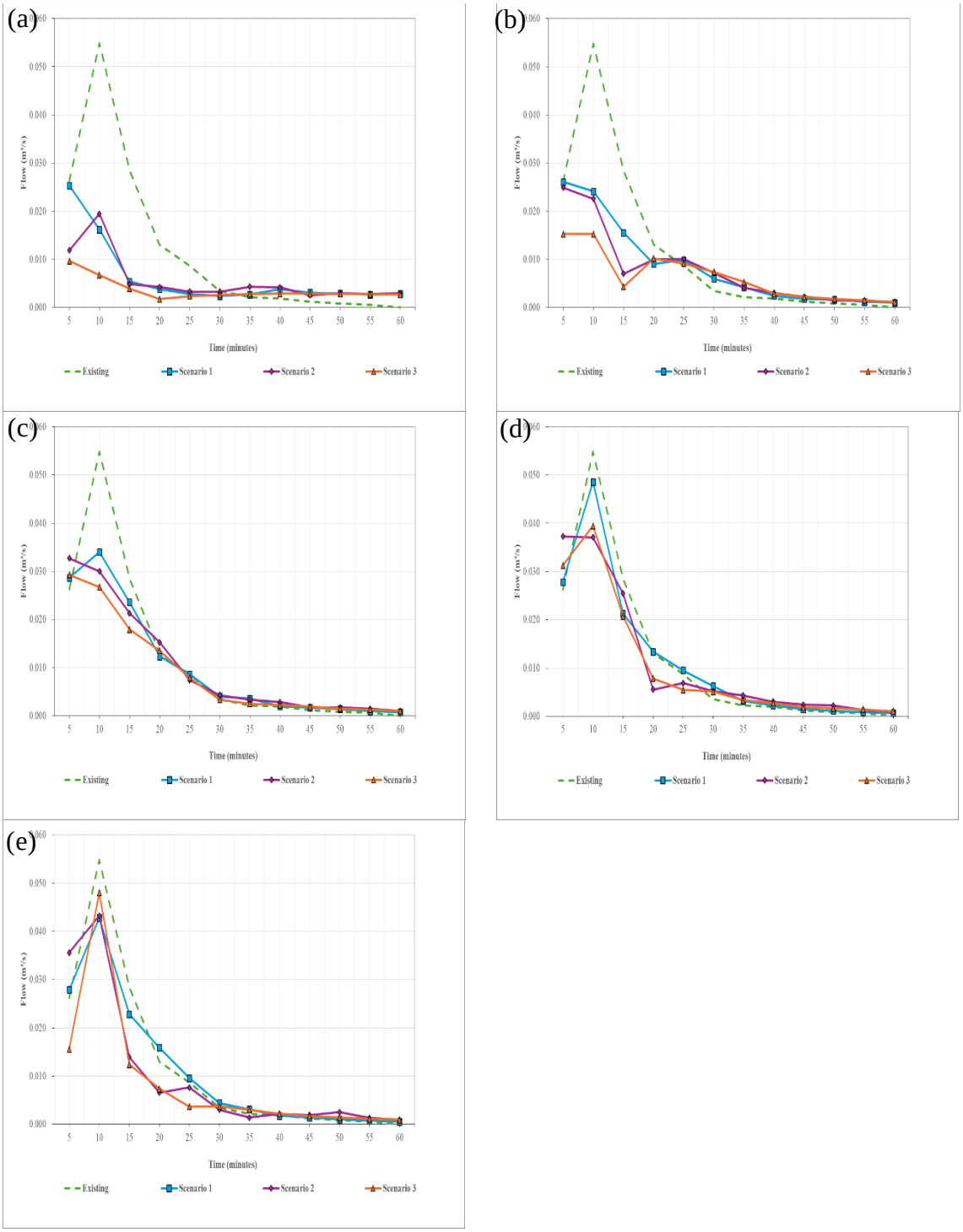


Figure 4.8: Flow Plots for Drain with 0% Slope at Location D Subjected to Design Rainfall, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

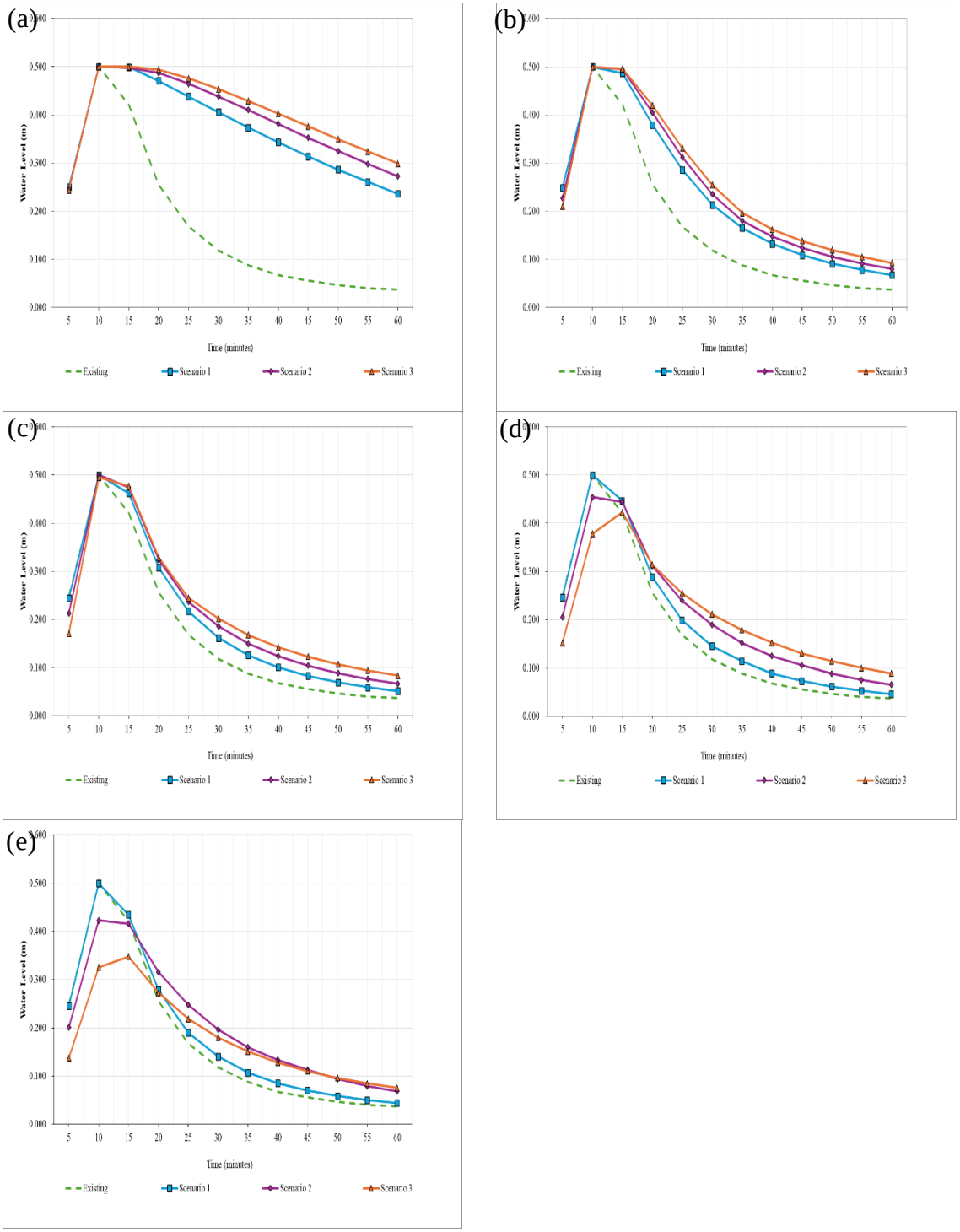


Figure 4.9: Water Level Plots for Drain with 0% Slope at Location D Subjected to Design Rainfall, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

no reduction in water level by comparing to existing condition. Hence, flooding was found in all the scenarios at the 10-minute peak flow as the water level of had reached 0.5m which is the maximum water level of the open channel. Scenarios 1, 2, and 3 demonstrated reductions of 56.0%, 58.8%, and 72.2% respectively. Scenario 3 exhibited a significant reduction in flow rate. In Figure 4.9b, all the scenario shows no reduction in water level by compared to an existing condition. Hence, flooding was found in all the scenarios at the 10-minute peak flow as the water level had reached 0.5m which is the maximum water level of the open channel.

The orifice size increased from 0.1m to 0.2m in diameter, and the flow rates for all scenarios remained below the existing condition before the 25-minute mark and exceeded the existing condition thereafter as shown in Figure 4.8b. At the 10-minute peak flow rate, Scenarios 1, 2, and 3 demonstrated reductions of 56.0%, 58.8%, and 72.2% respectively. Scenario 3 exhibited a significant reduction in flow rate. In Figure 4.9b, all the scenario shows no reduction in water level by compared to an existing condition. Hence, flooding was found in all the scenarios at the 10-minute peak flow as the water level had reached 0.5m which is the maximum water level of the open channel.

The existing condition and the implementation of a 0.3m diameter orifice in three scenarios as shown in Figure 4.8c. The flow rate before the 20-minute mark was below the existing condition. After the 20-minute mark, the flow rate exceeded the existing condition. Compared to the existing condition, Scenarios 1, 2, and 3 demonstrated reductions of 38.0%, 45.2% and 51.2%, respectively. Scenario 3 exhibited the most significant reduction compared to the existing condition. In Figure 4.9c, Scenario 1 and Scenario 2 show no reduction in water level by comparing to existing conditions whereas Scenario 3 had a minor reduction of the water level by 0.9%. Flooding was found in Scenario 1 and 2 at the 10-

minute peak flow whereas no flooding occurred in the long open channel with the application of the 0.5m diameter flow restrictor at Scenario 3.

The implementation of a 0.4m diameter orifice in three scenarios is shown in Figure 4.8d. The flow rate before the 25-minute mark remained below the existing condition, while it exceeded the existing condition after the 25-minute mark. Compared to the existing condition, Scenario 2 showed a significant reduction of 32.4%, followed by Scenario 3 with 28.1% and Scenario 1 with 11.4%. In Figure 4.9d, Scenario 1 shows no reduction in water level by comparing to existing conditions whereas Scenario 2 and 3 had a minor reduction of the water level by 9.2%, and 24.4% respectively. Flooding was found in Scenario 1 at the 10-minute peak flow whereas no flooding occurred in the long open channel with the application of the 0.5m diameter flow restrictor at Scenario 2 and 3.

The effects of increasing the orifice size from 0.4m to 0.5m in diameter as shown in Figure 4.8e, allowed a greater volume of water to flow. At the peak flow (10-minute mark), the flow rates for Scenarios 1, 2, and 3 were reduced by 21.9, 21.2%, and 12.4%, respectively. Scenario 1 demonstrated the most substantial reduction in flow rate compared to the other scenarios. In Figure 4.9e, Scenario 1 shows no reduction in water level by comparing to existing conditions whereas Scenario 2 and 3 had a minor reduction of the water level by 15.4%, and 34.9% respectively. Flooding was found in Scenario 1 at the 10-minute peak flow whereas no flooding occurred in the long open channel with the application of the 0.5m diameter flow restrictor at Scenario 2 and 3.

e. Discussion

Figures 4.10 and 4.11 summarize the peak flow rate and water level for a design slope of 0% under a 5-minute, 10-year ARI design rainfall.

At Location A, the peak flow rate decreased for all scenarios and orifice sizes. This indicates that the reduction in flow rate decreases the rainfall discharge to the outfall, thereby reducing congestion at the upstream end. The highest reduction in flow rate occurred in all the Scenarios with a 0.1m orifice diameter, while the lowest reduction was in Scenario 1 with a 0.5m orifice diameter. However, the peak water level increased for all scenarios and orifice sizes except Scenario 2 with 0.5m diameter of orifice and Scenario 3 with 0.4 and 0.5m diameter of orifice had the reduction in the water level compared to existing condition. Every scenario with orifice diameters of 0.1m, and 0.2m, reached the maximum water level of 0.5m, causing potential flooding. Thus, these scenarios failed to meet the objective.

At Location B, the peak flow rate decreased for all scenarios and orifice sizes, except for Scenario 1 with orifice diameters of 0.4m, and 0.5m, and Scenario 2 with a 0.4m diameter of the orifice which showed an increase. The increased flow rate compared to the existing flow rate would lead to more rainfall water flowing to Location A for discharge. Thus, reducing the flow rate is a key objective in this study. The graph shows that Scenario 2 with a 0.1m orifice diameter had the highest reduction, whereas Scenario 2 with a 0.5m orifice diameter showed the least reduction. For peak water levels at Location B, all scenarios showed an increase compared to the existing water level, except Scenario 2 and Scenario 3 with a 0.4m and 0.5m orifice diameter, which shows the reduction. Flooding occurred in Scenario 1 (0.1m, 0.2m, and 0.3m diameter of orifice), Scenario 2 (0.1m, 0.2m, and 0.3m diameter of orifice), and Scenario 3 (0.1m, and 0.2m diameter of orifice) where water levels increased to the maximum drain water level of 0.5m. Hence, these analyses failed as they

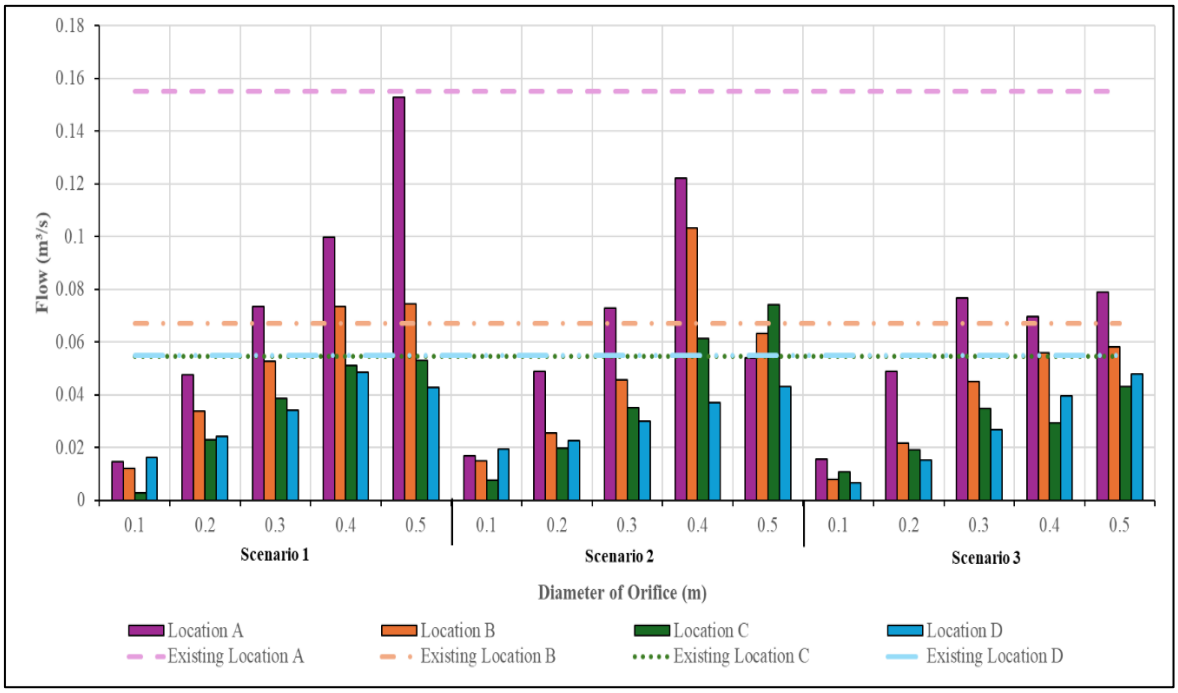


Figure 4.10: Summary of Peak Flow Rate for 5-minute 10-year ARI Design Rainfall at 0% Design Slope of Open Channel

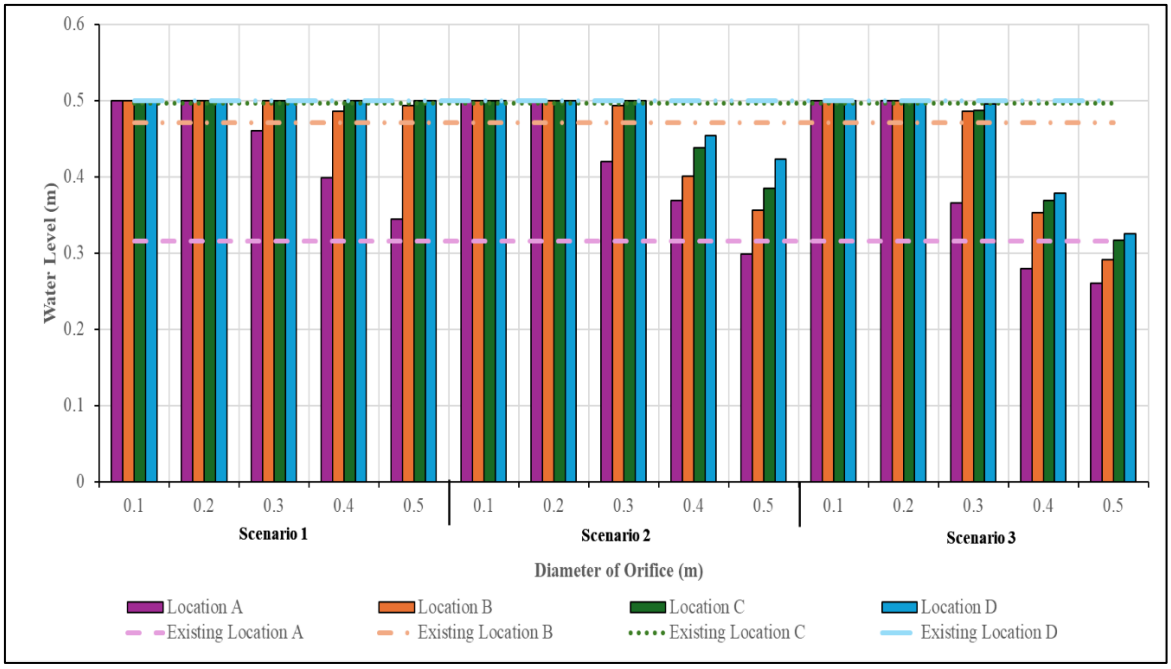


Figure 4.11: Summary of Peak Water Level for 5-minute 10-year ARI Design Rainfall at 0% Design Slope of Open Channel

caused flooding.

At Location C, the peak flow rate decreased for all scenarios and orifice sizes, except for Scenario 2 with a 0.4m, and 0.5m orifice diameter, which showed an increase. Scenario 1 with a 0.1m orifice diameter had the highest reduction in flow rate. Greater reductions in flow rate result in slower rainfall discharge to Locations A and B. Peak water levels for all scenarios had reached the maximum water level except for Scenario 2 (0.4m and 0.5m diameter of orifice) and Scenario 3 (0.3m, 0.4m, and 0.5m diameter of orifice) shows a reduction in water level by compared to existing conditions. However, the scenario and size of the orifice that reached the maximum water level of 0.5m led to failure in this analysis due to flooding.

At Location D (upstream end), The peak flow rate decreased for all scenarios and orifice sizes by comparing to existing condition. The highest reduction in flow rate occurred with Scenario 3's 0.1m orifice diameter, while the lowest reduction was with Scenario 1's 0.4m orifice diameter. Peak water levels increased across all scenarios compared to existing conditions. However, all the scenarios had reached or approximately to the maximum water level of 0.5m except for Scenario 2 and 3 (0.4m and 0.5m diameter of orifice), which caused flooding and thus failed to meet the objectives.

In summary, Scenario 2 and Scenario 3 with 0.4m and 0.5m diameter orifices were the most effective configurations, as they successfully reduced or only slightly increased the peak flow rates while preventing flooding along the open channel with a 0% design slope. These configurations demonstrate a more balanced hydraulic performance, effectively slowing down the stormwater discharge without causing excessive upstream water accumulation. In contrast, scenarios involving smaller orifice sizes, particularly 0.1m and

0.2m, although capable of significantly reducing flow rates, led to water levels reaching the maximum drain height of 0.5m, thereby resulting in flooding at multiple locations. This highlights the trade-off between flow attenuation and flood risk, where overly restrictive outlets may lead to localized inundation. The findings are consistent with existing literature which suggests that optimal orifice sizing is critical for maximizing detention efficiency while minimizing flood hazards (Araoye et al., 2017; Kuok and Chiu, 2022; Ma et al., 2024). Therefore, the selection of appropriate orifice size particularly 0.4m and 0.5m in this case plays a crucial role in achieving effective and flood-resilient drainage design under design rainfall events.

4.2.1.2 Historical Rainfall

The hydrographs presented in Figures 4.12, Figure 4.14, Figure 4.16 and Figure 4.18 illustrate the simulated flow rates, while Figures 4.13, Figure 4.15, Figure 4.17 and Figure 4.19 depict the corresponding water levels for Historical Rainfall Event 1. This event is selected for detailed presentation and discussion in this chapter. Meanwhile, the hydrographs for Event 2 and Event 3 are included in Appendix I for reference, as their flow rate and water level patterns were found to be largely consistent with those observed in Event 1. These were meant for 0% slope. Different drain slope had different characteristics that were described in Section 4.3.

Analyses of the hydrographs reveal that the peak flow rates for the Event 1 consistently pointed to only sub-figures (a) throughout managed to reduce the flow rates, while the remaining sub-figures (b) to (e) maintained similarity to hydrographs of the existing condition. In response to this observation, it was found that the peak water levels

for the three events, only sub-figures (a) had a significant higher water levels compared to the rest of sub-figures (b) to (e).

a. Location A

Flow rates shown in Figure 4.12a describe for all scenarios generally remained below the existing condition. Scenario 3, which included three orifice flow restrictors along the open channel, provided greater detention capacity compared to Scenarios 1 and 2. At the 13-hour peak flow discharge, the 0.1m diameter orifice in Scenario 1 and 2 resulted in a higher flow rate compared to Scenario 3 at Location A, indicating it did not effectively reduce downstream flow discharge. At the peak flow (13-hour mark), Scenarios 1, 2, and 3 showed reductions of 54.1%, 54.1%, and 55.1%, respectively, compared to the existing condition, with Scenario 3 showing the most substantial reduction. Figure 4.13a illustrates that Scenarios 1, 2, and 3 experienced a 278.2%, 278.2%, and 264.7% respectively increase in water level compared to the existing condition, Scenario 1 and Scenario 3 resulting in flooding as the water level reached 0.5m, due to the combination of the 0.1m orifice flow restrictor and the 0% drain design slope in the open channel.

The orifice size increased from 0.1m to 0.2m in diameter, and the flow rates for all scenarios generally remained below the existing condition throughout the rainfall event as shown in Figure 4.12b. At the 13-hour peak flow rate, Scenarios 1, and 2 showed reductions of 2.7%, and 11.7% respectively whereas Scenario 3 shows a minor increment of 7.4%. Figure 4.13b shows that Scenario 1, 2, and 3 with a 0.2m diameter orifice flow restrictor experienced an 85.7%, 88.6%, and 52.0% increase respectively in water level compared to the existing condition. However, no flooding occurred at Location A as the water level

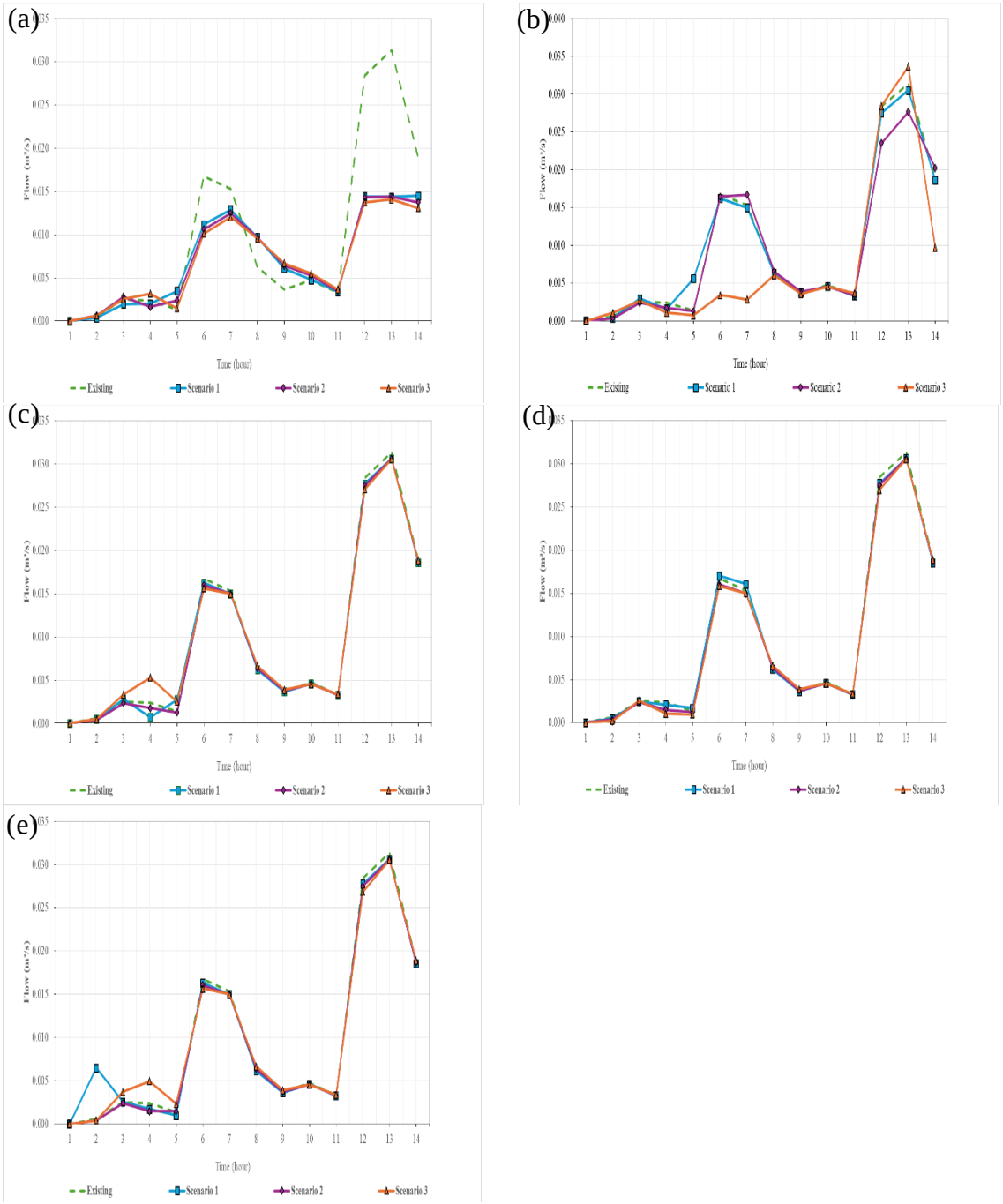


Figure 4.12: Flow Plots for Drain with 0% Slope at Location A Subjected to Event 1, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

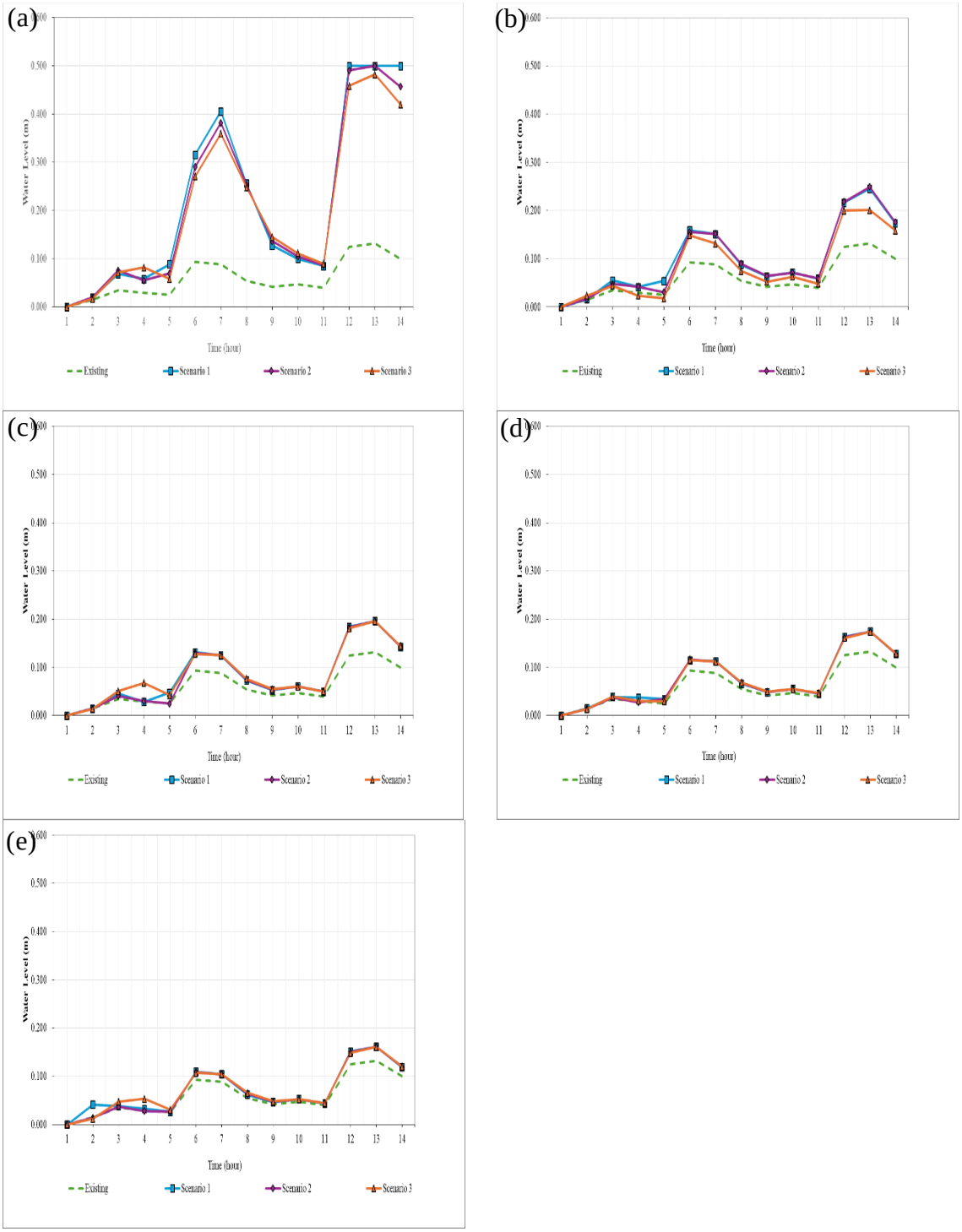


Figure 4.13: Water Level Plots for Drain with 0% Slope at Location A Subjected to Event 1, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

remained below 0.5m.

The existing condition and the implementation of a 0.3m diameter orifice in three scenarios are shown in Figure 4.12c. The flow rate for all scenarios was below the existing condition, showing a nearly uniform reduction of 2.3%. However, Figure 4.13c indicates that all scenarios with a 0.3m diameter orifice flow restrictor experienced a 48.5% increase in water level compared to the existing condition. No flooding occurred with the application of a 0.3m diameter orifice in this simulation.

Orifice size increased to 0.4m in diameter in Figure 4.12d, the flow rates for all scenarios generally remained below the existing condition throughout the rainfall event. At the 13-hour peak flow rate, Scenarios 1, 2, and 3 showed reductions of 2.3%, 2.3%, and 2.5% respectively. All scenarios with a 0.4m diameter orifice flow restrictor experienced a 31.7% increase in water level compared to the existing condition in Figure 4.13d. However, no flooding occurred at Location A as the water level remained below 0.5m.

The orifice size increased to 0.5m in diameter in Figure 4.12e, and the flow rates for all scenarios generally remained below the existing condition throughout the rainfall event. At the 13-hour peak flow rate, Scenarios 1, 2, and 3 demonstrated reductions of 2.3%, 2.3%, 2.5% respectively. Figure 4.13e shows that Scenario 1, 2, and 3 with a 0.5m diameter orifice flow restrictor experienced a 21.9%, 21.8%, and 21.6% respectively increase in water level compared to the existing condition. However, no flooding occurred at Location A as the water level remained below 0.5m.

b. Location B

Flow rates for all scenarios generally remained below the existing condition as shown in Figure 4.14a. At the 13-hour peak flow discharge, the 0.1m diameter orifice in Scenario

3 resulted in a higher reduction in flow rate compared to Scenarios 2 and 1 at Location B, indicating it effectively reduced flow discharge. At the peak flow (13-hour mark), Scenarios 1, 2, and 3 showed reductions of 77.6%, 78.1%, and 85.5%, respectively, compared to the existing condition. Figure 4.15a illustrates that Scenarios 1, 2, and 3 experienced a similar increase in water level compared to the existing condition by 225.4% at the 13-hour peak mark by comparing to the existing condition. This had resulting in flooding after the 12-hour mark as the water level reached 0.5m for Scenario 3 and the flood happened at the 13-hour peak mark for Scenario 1 and 2.

Orifice size increased to 0.2m in diameter, and the flow rates for all scenarios generally remained below the existing condition throughout the rainfall event as shown in Figure 4.14b. At the 13-hour peak flow rate, Scenarios 1, 2, and 3 had a minor reduction which was 0.4%, 0.4%, and 0.3% respectively. Figure 4.15b shows that Scenario 2 had the highest increment followed by Scenario 1 and Scenario 3 which were 72.9%, 62.5%, and 13.5% respectively. However, no flooding occurred at Location B for all the scenario as the water level remained below 0.5m throughout the rainfall event.

The existing condition and the implementation of a 0.3m diameter orifice in three scenarios are illustrated in Figure 4.14c. The flow rate at Location B for Scenario 1 has a similar flow rate as the existing condition. However, Scenario 3 had a minor reduction compared to existing conditions which was 6.1% whereas Scenario 2 had a minor increment of 5.8%. Figure 4.15c indicates that Scenarios 1, 2, and 3 had a significant increment in water level by 32.8%, 32.8%, and 41.7% respectively at the 13-hour peak mark by comparing to the existing condition. No flooding occurred with the application of a 0.3m diameter orifice in this simulation.



Figure 4.14: Flow Plots for Drain with 0% Slope at Location B Subjected to Event 1, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

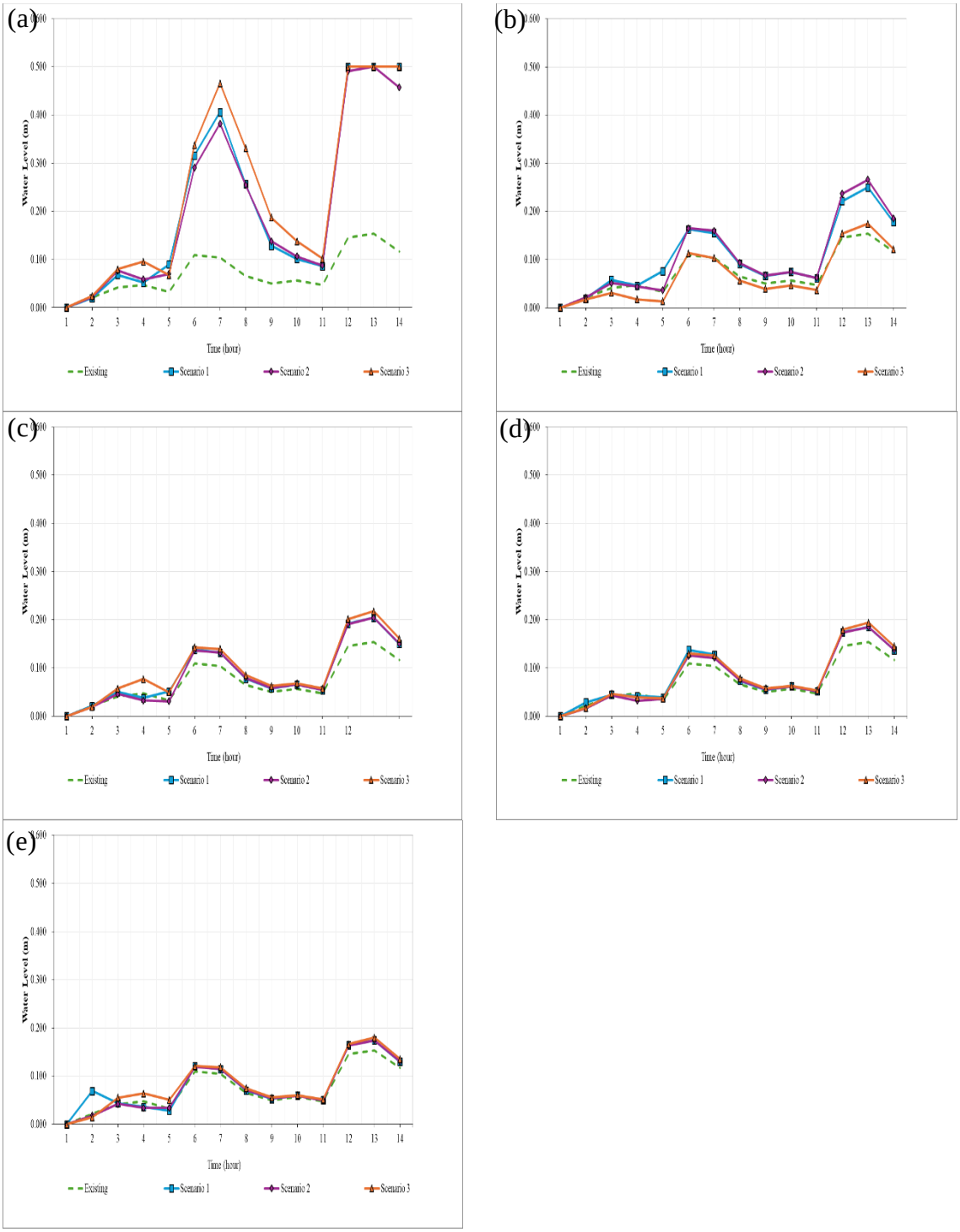


Figure 4.15: Water Level Plots for Drain with 0% Slope at Location B Subjected to Event 1, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

Orifice size increased to 0.4m in diameter in Figure 4.14d, and the flow rate of Scenario 1 had a similar reading to the existing condition whereas Scenario 2, and 3 had a slight reduction compared to the existing condition by 0.1%, and 6.1% respectively. Figure 4.15d demonstrates that Scenario 3 had the highest increment followed by Scenario 1 and Scenario 2 which are 26.0%, 20.3%, and 20.2% respectively. No flooding occurred at Location B as the water level remained below 0.5m.

Orifice size increased to 0.5m in diameter in Figure 4.14e, there were no increments or reductions for Scenario 1 for the flow rate at Location B. However, Scenario 2 and 3 show a minor reduction which were 0.1% and 6.2% respectively. Figure 4.15e shows that Scenarios 1, 2, and 3 had a minor increment by 13.4%, 13.3%, and 17.2% at the 13-hour peak mark by comparing them to the existing condition. However, no flooding occurred at Location B as the water level remained below 0.5m.

c. Location C

Flow rates for all scenarios generally remained below the existing condition as shown in Figure 4.16a. At the peak flow (13-hour mark), Scenarios 1, 2, and 3 had the reduction of flow rate comparing with existing condition which were 79.0%, 82.3% and 83.6% respectively. Scenario 3 had the highest reduction compared to other scenarios. Figure 4.17a illustrates that Scenarios 1, Scenario 2, and Scenario 3 had similar increased the water level of 219.2% respectively. Therefore, the flood were happened for all the scenarios after the 12-hour mark.

The orifice size increased to 0.2m in diameter, the flow rates for all scenarios generally approximately to the existing condition throughout the rainfall event at Location C as illustrated in Figure 4.16b. Scenario 1,2 and 3 shows the reduction by 0.4%, 2.9% and

32.7%. Scenario 3 shows the highest reduction compared to other scenarios. Figure 4.17b shows the water level for Scenario 2 had increases by 89.0%, followed by Scenario 1 which increase by 59.9% whereas Scenario 3 had the minor reduction by 56.9% at Location C by comparing to the existing condition at the 13-hour peak flow mark. However, no flooding occurred at Location C as the water level remained below 0.5m.

The existing condition and the implementation of a 0.3m diameter orifice in three scenarios as shown in Figure 4.16c. The flow rate for Scenario 1 remains unchanged to the existing condition. However, Scenarios 2 and 3 show a minor reduction which was 1.2% and 0.1% respectively at the 13-hour peak mark by comparing to an existing condition. Figure 4.17c indicates that Scenario 3 has the highest increment of water level compared to existing conditions followed by Scenario 2 and Scenario 1 which were 39.7%, 36.4%, and 31.2% respectively. No flooding occurred with the application of a 0.3m diameter orifice in this simulation.

The existing condition and the implementation of a 0.3m diameter orifice in three scenarios as shown in Figure 4.16c. The flow rate for Scenario 1 remains unchanged to the existing condition. However, Scenarios 2 and 3 show a minor reduction which was 1.2% and 0.1% respectively at the 13-hour peak mark by comparing to an existing condition. Figure 4.17c indicates that Scenario 3 has the highest increment of water level compared to existing conditions followed by Scenario 2 and Scenario 1 which were 39.7%, 36.4%, and 31.2% respectively. No flooding occurred with the application of a 0.3m diameter orifice in this simulation.

Orifice size increased to 0.4m in diameter, and the flow rates for all scenarios generally remained below the existing condition throughout the rainfall event as shown in



Figure 4.16: Flow Plots for Drain with 0% Slope at Location C Subjected to Event 1, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

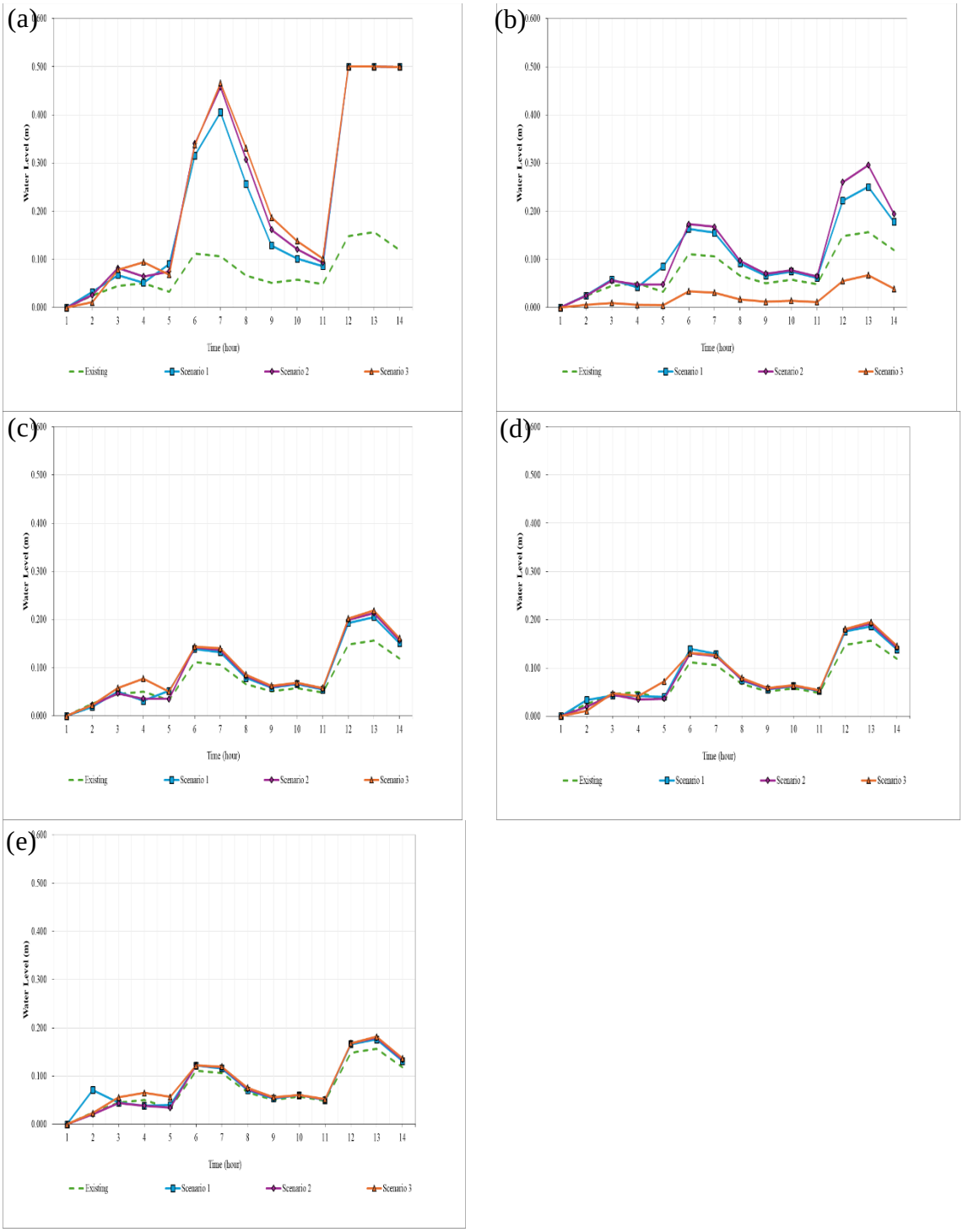


Figure 4.17: Water Level Plots for Drain with 0% Slope at Location C Subjected to Event 1, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

Figure 4.16d. The flow rate for Scenario 1 remains unchanged to the existing condition. However, Scenario 2 and 3 show a minor reduction which was 1.2% and 0.2% respectively at the 13-hour peak mark by comparing to existing conditions Figure 4.17d demonstrates that water level where Scenario 1 had the lowest increment in water level followed by Scenario 2 and Scenario 3 where increase by 19.1%, 22.5%, and 24.6% respectively. No flooding occurred at Location C as the water level remained below 0.5m.

Orifice size increased to 0.5m in diameter, and the flow rates for all scenarios generally remained below the existing condition throughout the rainfall event as illustrated in Figure 4.16e. The flow rate for Scenario 1 remains unchanged to the existing condition. However, Scenarios 2 and 3 show a minor reduction which was 1.2% and 0.3% respectively at the 13-hour peak mark by comparing to an existing condition. Figure 4.17e shows that the water level where Scenario 1 had the lowest increment in water level followed by Scenario 2 and Scenario 3 which increased by 12.6%, 14.8%, and 16.2% respectively. No flooding occurred at Location C as the water level remained below 0.5m.

d. Location D

Flow rates for all scenarios generally remained below the existing condition as illustrated in Figure 4.18a. At the 13-hour peak flow discharge, the 0.1m diameter orifice in Scenario 3 had the significant reduction followed by Scenario 2 and Scenario 1 which reduced by 79.9%, 66.7%, and 65.0% respectively. Figure 4.19a illustrates that all the scenarios experienced similar increment of water level by 214.3% at the 13-hour peak mark. Flood was found for Scenario 1 and 3 from 12-hour peak mark onward whereas Scenario 2 occur the flood from 11-hour mark onward.

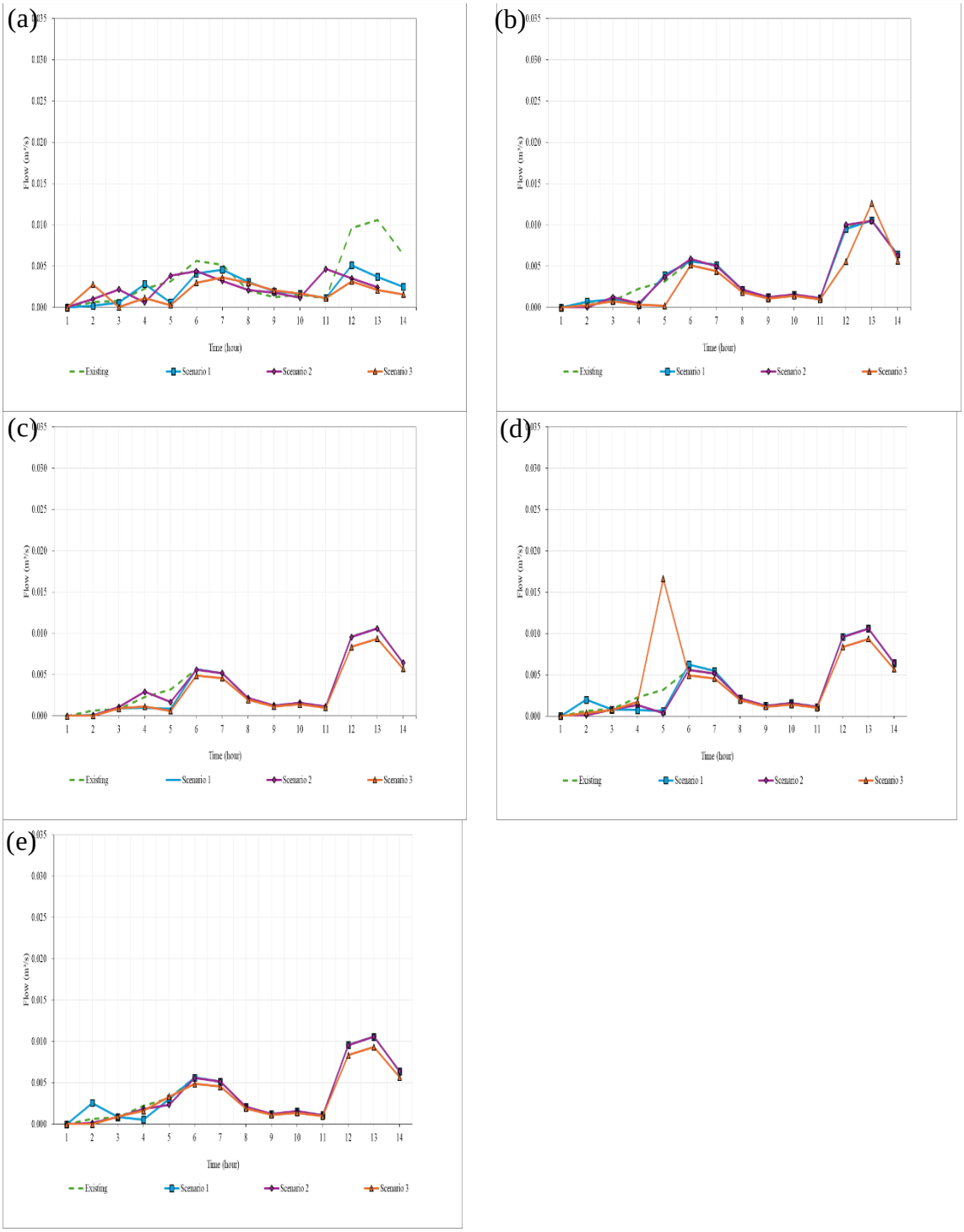


Figure 4.18: Flow Plots for Drain with 0% Slope at Location D Subjected to Event 1, (a) 0.1m, (b) 0. 2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

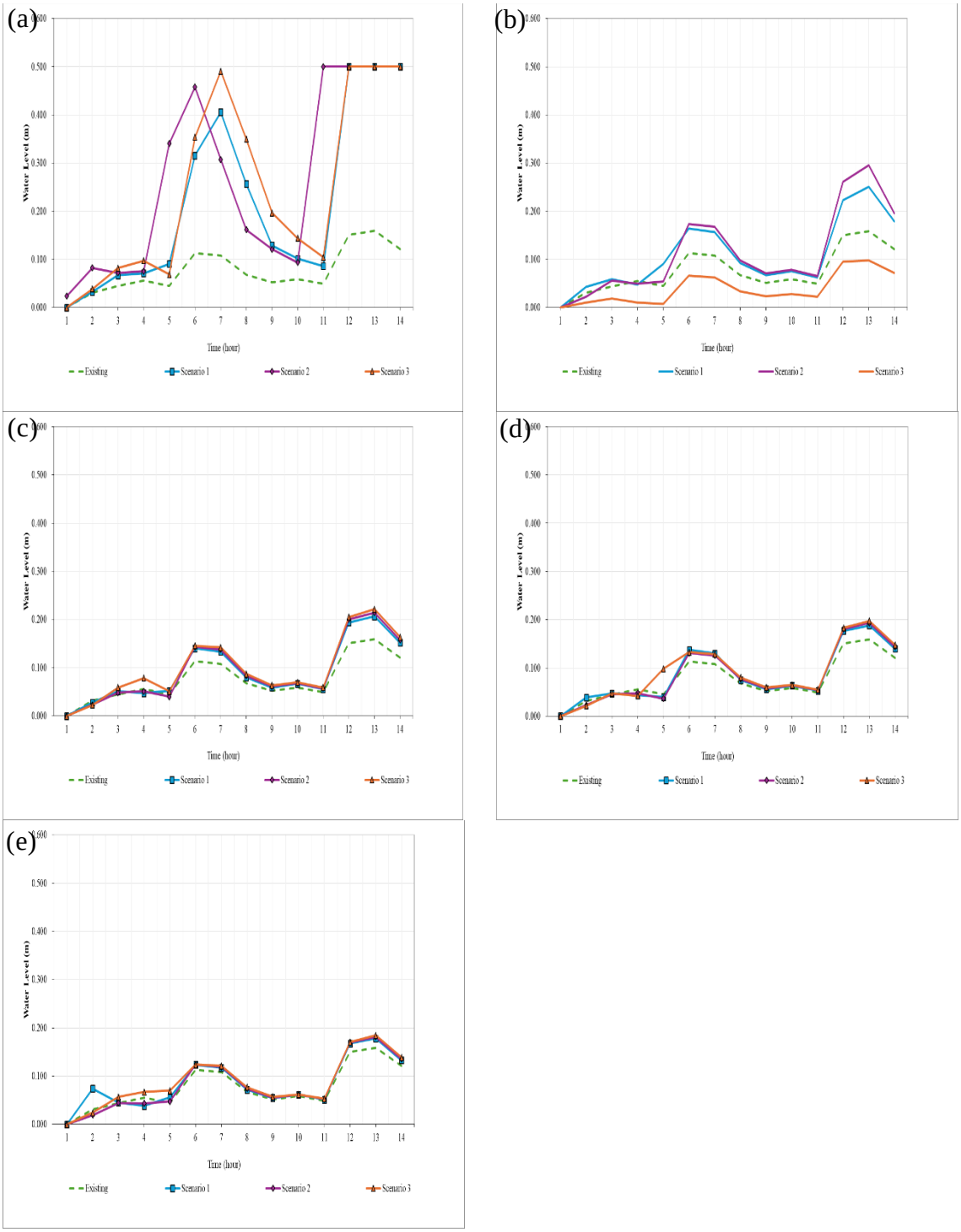


Figure 4.19: Water Level Plots for Drain with 0% Slope at Location D Subjected to Event 1, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

The orifice size increased from 0.1m to 0.2m in diameter, the flow rates for all scenarios generally remained below the existing condition throughout the rainfall event at shown in Figure 4.18b. At the 13-hour peak flow rate, Scenarios 1, and Scenario 2, shows a minor reduction by 0.3% and 1.1% respectively whereas Scenario 3 had a increment by 19.4% by comparing with existing condition. Figure 4.19b shows that the water level for Scenario 1 had the lowest increment follow by Scenario 2 where increase by 57.9%, and 86.1% respectively whereas Scenario 3 had an minor reduction by 38.3% by comparing to the existing condition. No flooding were found for all the scenarios as the water level were below the 0.5m which is the maximum water level of the open channel.

The existing condition and the implementation of a 0.3m diameter orifice in three scenarios as shown in Figure 4.18c. The flow rate for Scenario 1 and Scenario 2 remains unchanged whereas Scenario 3 has a slight reduction of 11.6% by comparing to existing conditions. Figure 4.19c indicates that the water level for Scenario 1, 2, and 3 had to increase in water level by 29.9%, 34.9%, and 39.4% respectively by comparing to existing conditions. No flooding occurred with the application of a 0.3m diameter orifice in this simulation.

The orifice size increased to 0.4m in diameter, and the flow rate for Scenario 1 and Scenario 2 remains unchanged whereas Scenario 3 has a slight reduction by 11.6% compared to the existing condition as illustrated in Figure 4.18d. Figure 4.19d demonstrates that the water level for Scenario 1, 2, and 3 had to increase in water level by 18.3%, 21.5%, and 24.3% respectively by comparing to an existing condition. However, no flooding occurred at Location D as the water level remained below 0.5m.

The orifice size increased to 0.5m in diameter, and the flow rate for Scenario 1 and Scenario 2 remains unchanged whereas Scenario 3 has a slight reduction by 11.7% compared to the existing condition as shown in Figure 4.18e. At the 13-hour peak flow rate, Scenarios 1, and Scenario 2 remain unchanged whereas Scenario 3 had a minor reduction by 11.5%. Figure 4.19e shows that the water level for Scenario 1, 2, and 3 had to increase in water level by 11.9%, 14.1%, and 16.0% respectively by comparing to existing conditions. However, no flooding occurred at Location D as the water level remained below 0.5m.

e. Discussion

Figures 4.20 and 4.21 summarize the peak flow rate and water level for a design slope of 0% during Event 1 historical rainfall. At Location A, the peak flow rate decreased for all scenarios and orifice sizes compared to the existing condition except for 0.2 diameters of orifice size in Scenario 3 shows a minor increment when compared to the existing condition. The reduction in the flow rate indicates a reduction in rainfall discharge to the outfall and decreased congestion at the upstream end. For this historical event, only the 0.1m diameter orifice for all the scenarios showed significant reductions in flow rate, with Scenario 2 having the highest reduction. Other conditions only showed minor reductions compared to the existing condition. However, the peak water level increased for all scenarios and orifice sizes, whereas Scenario 1 and 2 with 0.1m diameter orifice reaching the maximum water level of 0.5m, potentially causing flooding. No flooding occurred with the 0.2m, 0.3m, 0.4m, and 0.5m diameter orifices in any scenario. Thus, scenarios causing flooding failed to meet the objective.

At Location B, most peak flow rates remained unchanged compared to the existing condition, except for the 0.1m diameter orifice for all the scenarios and Scenario 3 (0.3m,

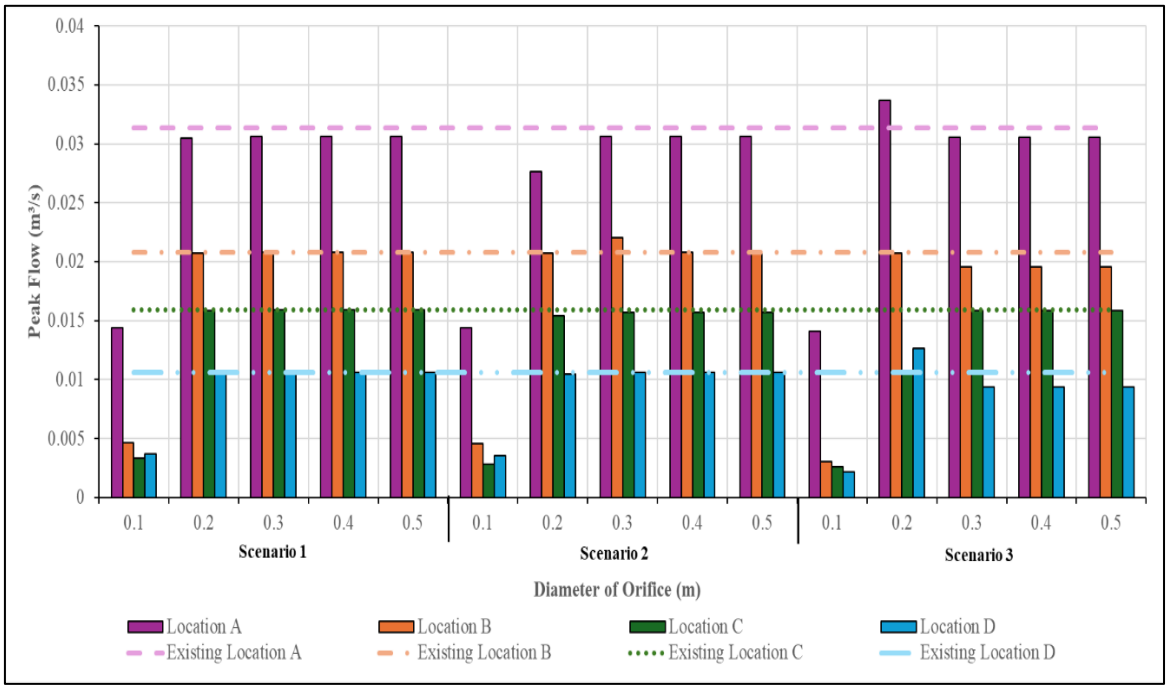


Figure 4.20: Summary of Peak Flow Rate for Event 1 Historical Rainfall at 0% Design Slope of Open Channel

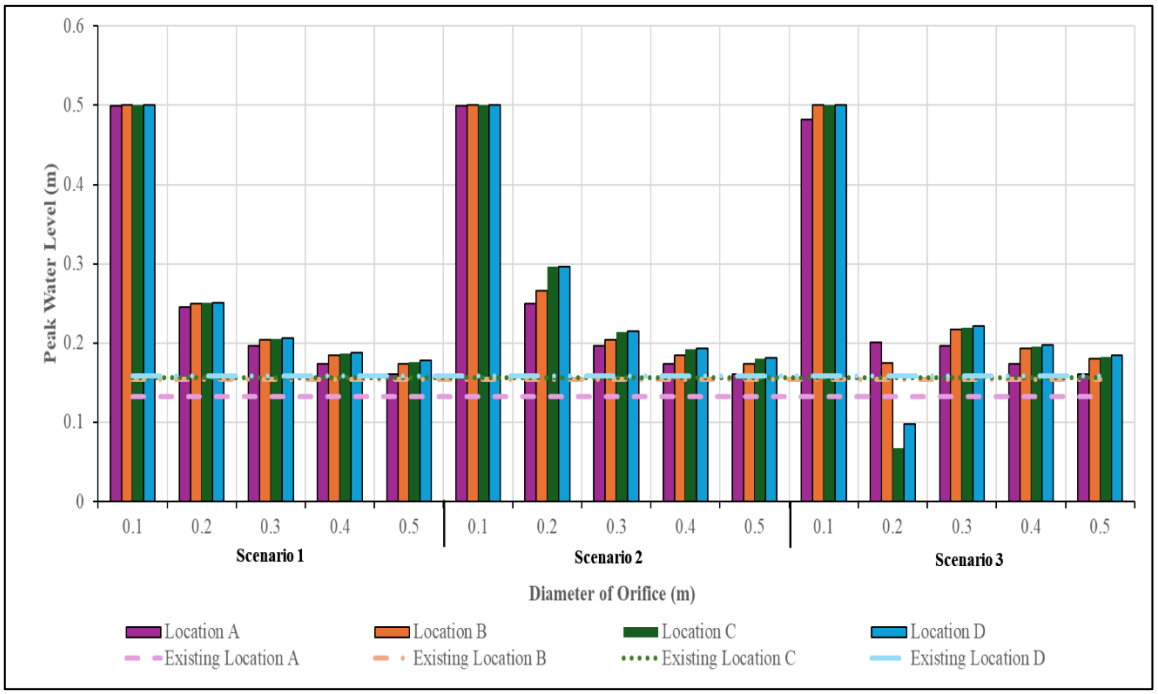


Figure 4.21: Summary of Peak Water Level for Event 1 Historical Rainfall at 0% Design Slope of Open Channel

0.4m, and 0.5m diameter of orifice), which showed a reduction in flow rate. However, the 0.3m diameter of the orifice in Scenario 2 shows a minor increase in flow rate compared to existing conditions which would lead to more rainfall water flowing to Location A for discharge. Therefore, reducing the flow rate is a key objective in this study. For peak water levels at Location B, all scenarios showed an increase compared to the existing water level. Flooding was observed with the 0.1m diameter orifice for all the scenarios, as the water level increased to 0.5m, the maximum drain water level. Hence, the analysis for all the scenarios with a 0.1m diameter orifice failed due to flooding.

At Location C, the peak flow rate for all scenarios and orifice sizes was almost similar to the existing condition, except for the 0.1m diameter orifice for all the scenarios, which showed the highest reduction in flow rate compared to other scenarios. Greater reductions in flow rate result in slower rainfall discharge to Locations A and B. Peak water levels for all the scenarios increased compared to the existing condition except for the 0.2m diameter of the orifice in Scenario 3 shows a significant reduction compared to the exiting condition. 0.1m diameter orifice flow restrictor for all the scenarios reaching the maximum water level of 0.5m, leading to failure in this analysis due to flooding.

At Location D, the upstream end, the peak flow rate for Scenario 1 and Scenario 2 was approximately similar to the existing condition except for the 0.1m diameter of the orifice shows a significant reduction in flow rate. However, Scenario 3 for 0.1m, 0.3m, 0.4m, and 0.5m shows the reduction of flow rate whereas the 0.2m diameter of the orifice shows a minor increase of the flow rate by comparing to the existing condition. Reducing the flow rate can help decrease the rainfall discharge to Location C. Peak water levels for all scenarios increased compared to the existing condition except the 0.2m diameter of the orifice in

Scenario 3 shows a significant reduction of the water level. However, the 0.1m diameter orifice for all three scenarios showed a significant increase in water level, which reaching the 0.5m water level and causing flooding.

The analysis revealed that implementing flow restrictors across different scenarios and orifice sizes at a 0% design slope could generally reduce peak flow rates compared to existing conditions, contributing to better stormwater management. Most scenarios achieved either a reduction or negligible change in flow rates, thereby alleviating discharge loads towards the downstream outfall. However, the use of a 0.1m diameter orifice consistently resulted in peak water levels reaching the maximum drain depth of 0.5m across all locations, leading to localized flooding and failure to meet the design objectives. This outcome emphasizes that while smaller orifice sizes effectively reduce flow rates, they can also excessively restrict outflow, causing upstream water accumulation and flooding. Larger orifice sizes (0.2m and above), by contrast, generally maintained flow rates at manageable levels without inducing flooding, particularly when used in multiple orifice scenarios. These findings align with previous studies which highlighted the importance of optimizing orifice geometry and sizing to achieve a balance between reducing flow discharge and preventing upstream congestion (Araoye et al., 2017; Düz, 2021; Kuok and Chiu, 2022; Ma et al., 2024). Therefore, careful selection of orifice size and configuration is crucial to enhancing the hydraulic performance of open channels under historical extreme rainfall events.

4.2.2 Drain with 0.2% Slope

4.2.2.1 Design Rainfall

The hydrograph representations resulting from the design rainfall, both with and without flow restrictor for open drain with 0.2% design slope, are depicted in Figure 4.22,

Figure 4.24, Figure 4.26, and Figure 4.28 for flow rates, and Figure 4.23, Figure 4.25, Figure 4.27, and Figure 4.29 for water levels.

Compared to previous sub-section, once a slope is introduced to the drain, in this case 0.2% slope, the significant change was observed on the surge of flow rate before the orifice. To point out an example, Scenario 1 (single orifice) had its peak flow rates were higher than existing condition before Location A (where orifice was inserted), particularly throughout sub-figures (a) to (c). However, the peak flow rates reduced in sub-figures (d) to (e). Similarly, the same observation was repeated in Scenario 2 (double orifice), peak flow rate was higher before Location C (where the first orifice was inserted). However, Scenario 3 (triple orifice) had no similar hike in flow rates as in the previous scenarios.

Water level, after being introduced with drain slope, its behaviours changed. With 0% slope, it showed opposite characteristics of flow rate and water level, namely high flow rate resulted in low water level and vice versa. However, with 0.2% slope, the water levels from the subsequent figures were longer following the trend. Instead, water levels were found to be higher than existing condition throughout. This was an unfavourable pattern.

a. Location A

Flow rates for all scenarios remained below the existing condition prior to the 20-minute mark, except for Scenario 1, which showed a 12.7% increase in flow rate at the 10-minute mark compared to the existing condition, which shown in Figure 4.22a. Scenario 3, which included three orifice flow restrictors along the open channel, provided greater detention capacity compared to Scenarios 1 and 2. Due to the peak flow discharge at the 10-minute mark, the 0.1m diameter orifice in Scenario 1 resulted in a higher flow rate compared to Scenarios 2 and 3 at Location A, indicating that it did not effectively reduce downstream

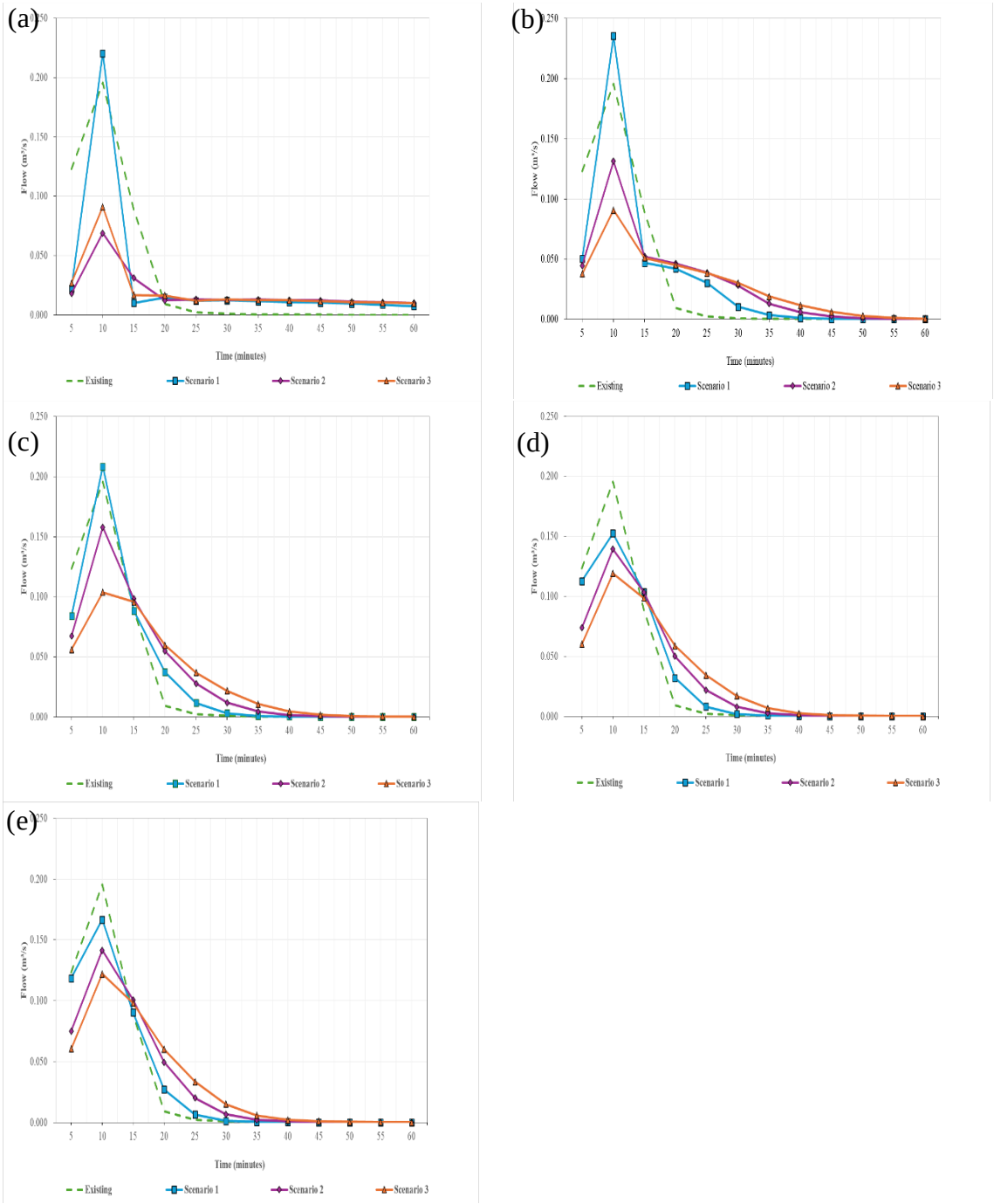


Figure 4.22: Flow Plots for Drain with 0.2% Slope at Location A Subjected to Design Rainfall, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

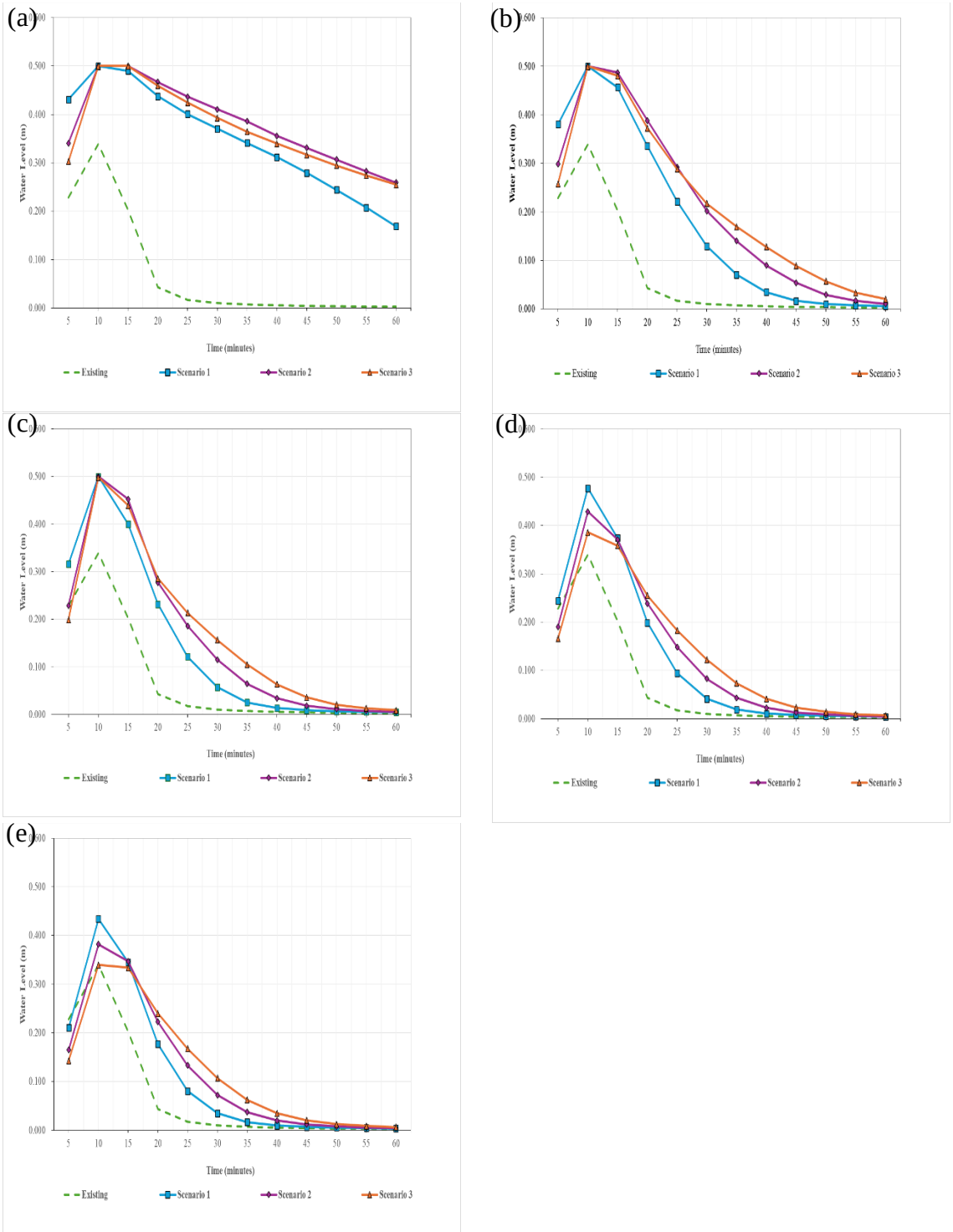


Figure 4.23: Water Level Plots for Drain with 0.2% Slope at Location A Subjected to Design Rainfall, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

flow discharge. At the peak flow (10-minute mark), Scenarios 2 and 3 demonstrated reductions of 64.7% and 53.4%, respectively, compared to the existing condition, with Scenario 2 showing the most substantial reduction in flow rate at Location A. Post the 20-minute mark, the flow rate for all scenarios surpassed the existing condition, indicating an overall increase in flow rate. Scenarios 1, 2, and 3 experienced a 47.3% increase in water level compared to the existing condition illustrates in Figure 4.23a. This resulted in flooding, as the water level reached 0.5m, due to the combination of the 0.1m orifice flow restrictor and the 0.2% drain design slope in the open channel.

Orifice size had increased from 0.1m to 0.2m in diameter in Figure 4.22b, the flow rates for all scenarios remained below the existing condition before the 20-minute mark and exceeded the existing condition thereafter. At the 10-minute peak flow rate, Scenario 1 showed a 20.4% increase in flow rate, while Scenarios 2 and 3 demonstrated reductions of 32.8% and 53.5%, respectively. Scenario 3 exhibited a significant reduction in flow rate compared to Scenario 2 with the increased orifice size. In Figure 4.23b, all scenarios with a 0.2m diameter orifice flow restrictor experienced a 47.3% increase in water level compared to the existing condition, resulting in flooding as the water level reached 0.5m.

Existing condition and the implementation of a 0.3m diameter orifice in three scenarios were illustrated in Figure 4.22c. The flow rate before the 15-minute mark was below the existing condition, except for Scenario 1. After the 15-minute mark, the flow rate exceeded the existing condition. Compared to the existing condition, Scenario 1 showed a 6.5% increase, while Scenarios 2 and 3 demonstrated reductions of 19.3% and 47%, respectively. Scenario 3 exhibited the most significant reduction compared to the existing condition. In Figure 4.23c, all scenarios with a 0.3m diameter orifice flow restrictor

experienced a 47.3% increase in water level compared to the existing condition, resulting in flooding as the water level reached 0.5m.

Implementation of a 0.4m diameter orifice in three scenarios is shown in Figure 4.22d. The flow rate before the 15-minute mark remained below the existing condition, while it exceeded the existing condition after the 15-minute mark. Compared to the existing condition, Scenario 3 showed a significant reduction of 39.1%, followed by Scenario 2 with 28.7% and Scenario 1 with 22.0%. In Figure 4.23d, Scenario 3 exhibited the smallest increment in water level compared to the existing condition, followed by Scenario 2 and Scenario 1, with increments of 13.7%, 26.4%, and 40.7%, respectively. No flooding occurred in the long open channel with the application of the 0.4m diameter flow restrictor.

The effects of increasing the orifice size from 0.4m to 0.5m in diameter, which allowed a greater volume of water to flow were shown in Figure 4.22e. More than 90% of the flow rates exceeded the existing condition due to the larger orifice size. At the peak flow (10-minute mark), the flow rates for Scenarios 1, 2, and 3 were reduced by 14.7%, 27.7%, and 37.6%, respectively. Scenario 3 demonstrated the most substantial reduction in flow rate compared to the other scenarios. In Figure 4.23e, Scenario 3 showed the smallest increment in water level compared to the existing condition, followed by Scenario 2 and Scenario 1, with increments of 0.03%, 12.5%, and 27.9%, respectively. No flooding occurred in the long open channel with the application of the 0.5m diameter flow restrictor.

b. Location B

The flow rates shown in Figure 4.24a for all scenarios remained below the existing condition after the 20-minute mark, except for Scenario 1, which showed a 29.8% increase in flow rate at the 10-minute mark compared to the existing condition. Due to the peak flow

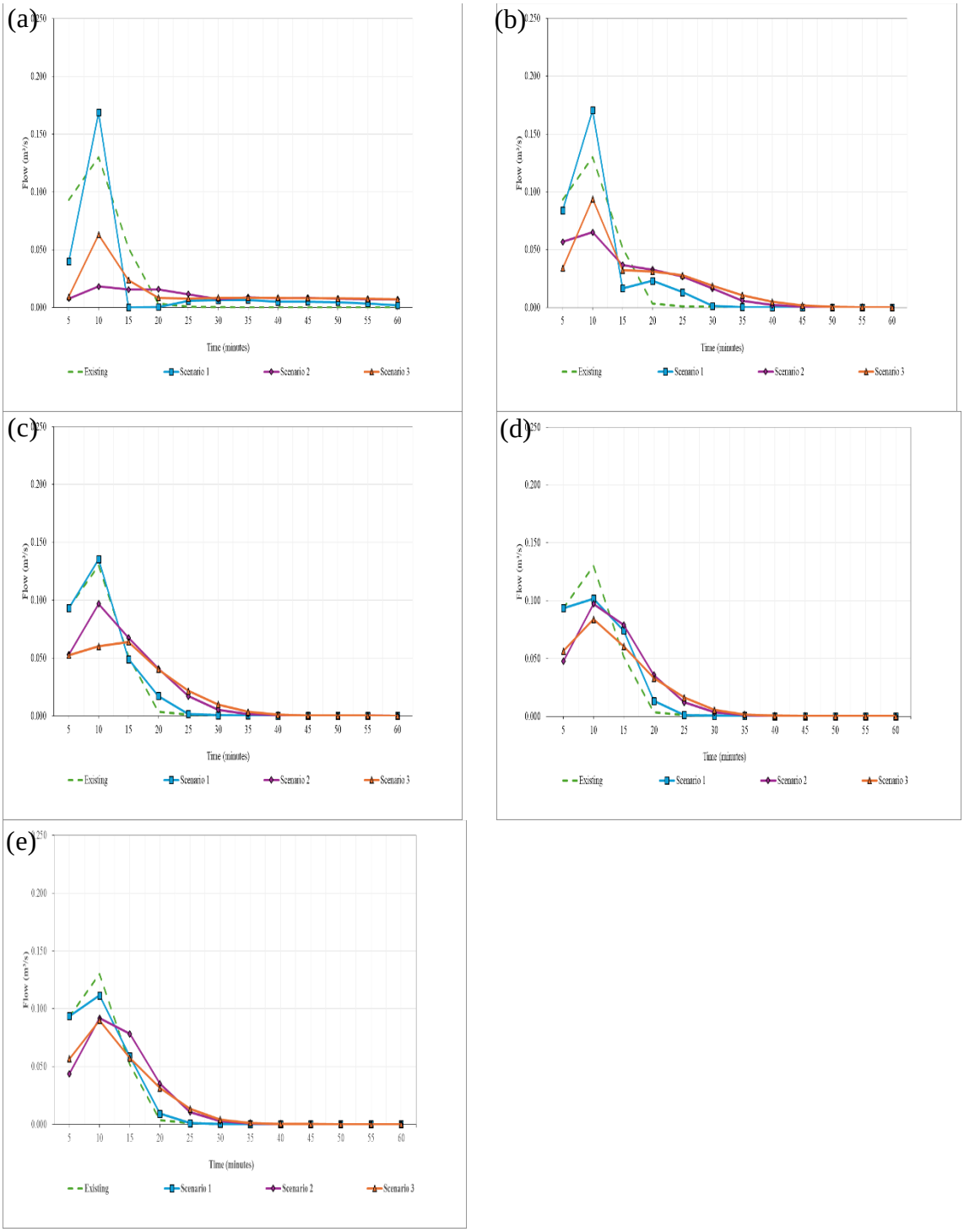


Figure 4.24: Flow Plots for Drain with 0.2% Slope at Location B Subjected to Design Rainfall, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

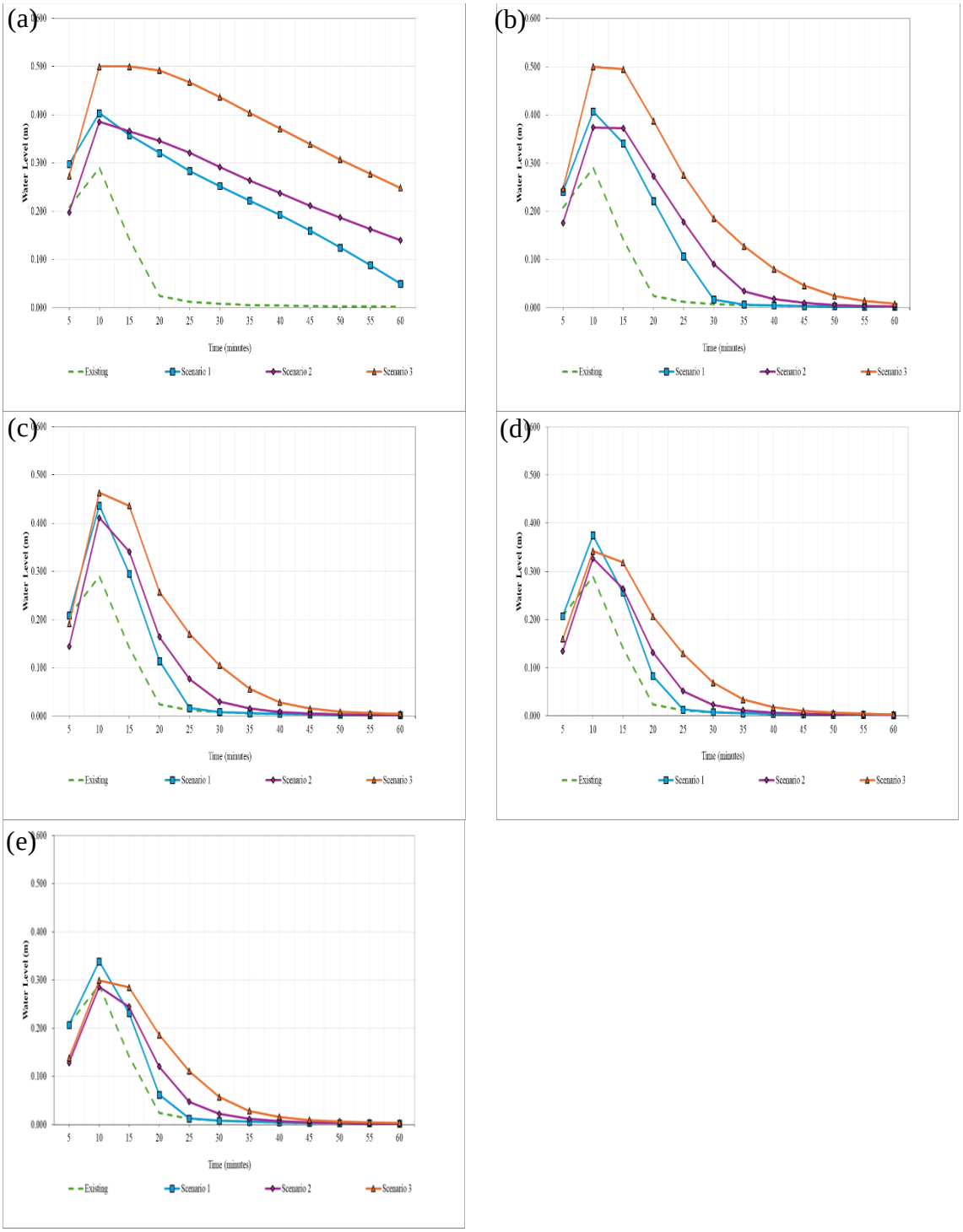


Figure 4.25: Water Level Plots for Drain with 0.2% Slope at Location B Subjected to Design Rainfall, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

discharge at the 10-minute mark, the 0.1m diameter orifice in Scenario 1 resulted in a higher flow rate compared to Scenarios 2 and 3 at Location B, indicating that it did not effectively reduce downstream flow discharge. At the peak flow (10-minute mark), Scenarios 2 and 3 demonstrated reductions of 85.9% and 51.4%, respectively, compared to the existing condition, with Scenario 2 showing the most substantial reduction in flow rate at Location B. Figure 4.25a illustrates that Scenarios 1, 2, and 3 experienced a 39.1%, 33%, and 72.4% respectively increase in water level compared to the existing condition. This resulted in flooding at Location B, as the water level reached 0.5m at 10-minute marks.

The orifice size increased 0.2m in diameter illustrated in Figure 4.24b, the flow rates for all scenarios remained below the existing condition before the 17.5-minute mark and exceeded the existing condition thereafter. At the 10-minute peak flow rate, Scenario 1 showed a 31.2% increase in flow rate, while Scenarios 2 and 3 demonstrated reductions of 50% and 27.7%, respectively. Scenario 2 exhibited a significant reduction in flow rate compared to Scenario 3 with the increased orifice size. Scenarios 1, Scenario 2 and Scenario 3 with a 0.2m diameter orifice flow restrictor experienced a 40.4%, 28.9%, 72.4% respectively increase in water level compared to the existing condition as shown in Figure 4.25b. This had resulting in flooding as the water level reached 0.5m in the interval of 10-minute to 15-minute mark.

The existing condition and the implementation of a 0.3m diameter orifice in three scenarios as illustrated in Figure 4.24c. The flow rate before the 15-minute mark was below the existing condition, except for Scenario 1. After the 15-minute mark, the flow rate exceeded the existing condition. Compared to the existing condition, Scenario 1 showed a 4.1% increase, while Scenarios 2 and 3 demonstrated reductions of 25.6% and 53.7%,

respectively. Scenario 3 exhibited the most significant reduction compared to the existing condition. In Figure 4.25c, Scenario 3 had the highest increment follow by Scenario 1 and Scenario 2 which were 59.8%, 50.5%, and 41.8% respectively. No flooding were found as the water level is below 0.5m.

Implementation of a 0.4m diameter orifice in three scenarios is shown in Figure 4.24d. The flow rate before the 15-minute mark remained below the existing condition, while it exceeded the existing condition after the 15-minute mark. Compared to the existing condition, Scenario 3 showed a significant reduction of 35.6%, followed by Scenario 2 with 25.2% and Scenario 1 with 21.9%. In Figure 4.25d, Scenario 2 exhibited the lowest increment in water level compared to the existing condition, followed by Scenario 3 and Scenario 1, with increments of 12.9%, 18.0%, and 29.4%, respectively. No flooding occurred in the long open channel with the application of the 0.4m diameter flow restrictor.

The effects of increasing the orifice size from 0.4m to 0.5m in diameter, which allowed a greater volume of water to flow as shown in Figure 4.24e. More than 90% of the flow rates exceeded the existing condition due to the larger orifice size. At the peak flow (10-minute mark), the flow rates for Scenarios 1, 2, and 3 were reduced by 14.3%, 29.4%, and 30.8%, respectively. Scenario 3 demonstrated the most substantial reduction in flow rate compared to the other scenarios. In Figure 4.25e, Scenario 2 showed the smallest increment in water level compared to the existing condition, followed by Scenario 3 and Scenario 1, with increments of 1.4%, 3.2%, and 16.8%, respectively. No flooding occurred in the long open channel with the application of the 0.5m diameter flow restrictor.

c. Location C

Flow rates for all scenarios as illustrated in Figure 4.26a remained below the existing condition prior to the 20-minute mark, except for Scenario 1, which showed a 33.8% increase in flow rate at the 10-minute mark compared to the existing condition. Due to the peak flow discharge at the 10-minute mark, the 0.1m diameter orifice in Scenario 3 resulted in a higher reduction in flow rate compared to Scenarios 2 at Location C, indicating that it effectively reduces flow discharge which are 55.7% and 26.2% respectively. Figure 4.27a illustrates that the water level for Scenario 2 had the highest increment follow by Scenario 3 and Scenario 1 which were 91.6%, 80.2% and 40.3% respectively. Flood happened for Scenario 2 at 3-hour mark of Location C.

Orifice size increased to 0.2m in diameter as illustrated in Figure 4.26b, the flow rates for Scenario 1 and Scenario 2 show the increment of the flow rate which are 27.7%, and 47.4% respective whereas Scenario 3 show the reduction of the flow rate which is 26.7% by comparing to existing condition. Figure 4.27b, Scenario 2 have the higher increment of water level compared to Scenario 3 and Scenario 1 which are 91.6%, 72.7%, and 44.7% respectively. Flood happened for Scenario 2 at 3-hour mark of Location C.

The existing condition and the implementation of a 0.3m diameter orifice in three scenarios as shown in Figure 4.26c. The flow rate before the 15-minute mark was below the existing condition, except for Scenario 1. After the 15-minute mark, the flow rate exceeded the existing condition. Compared to the existing condition, Scenario 1 showed a 8.2% increase, while Scenarios 2 and 3 demonstrated reductions of 39.5% and 52.5%, respectively. Scenario 3 exhibited the most significant reduction compared to the existing condition. In Figure 4.27c, water level in Scenario 2 shows the highest water level, follow

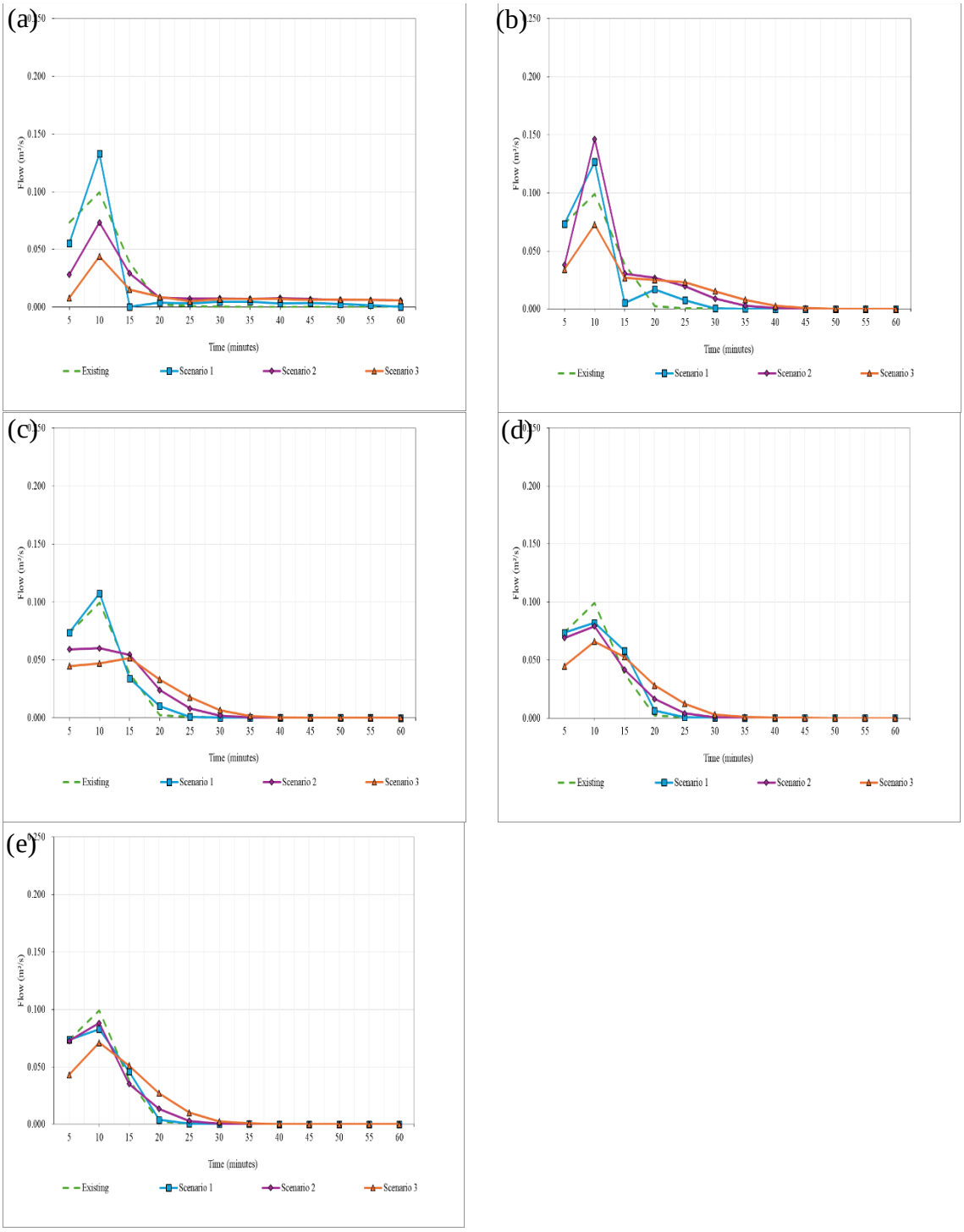


Figure 4.26: Flow Plots for Drain with 0.2% Slope at Location C Subjected to Design Rainfall, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

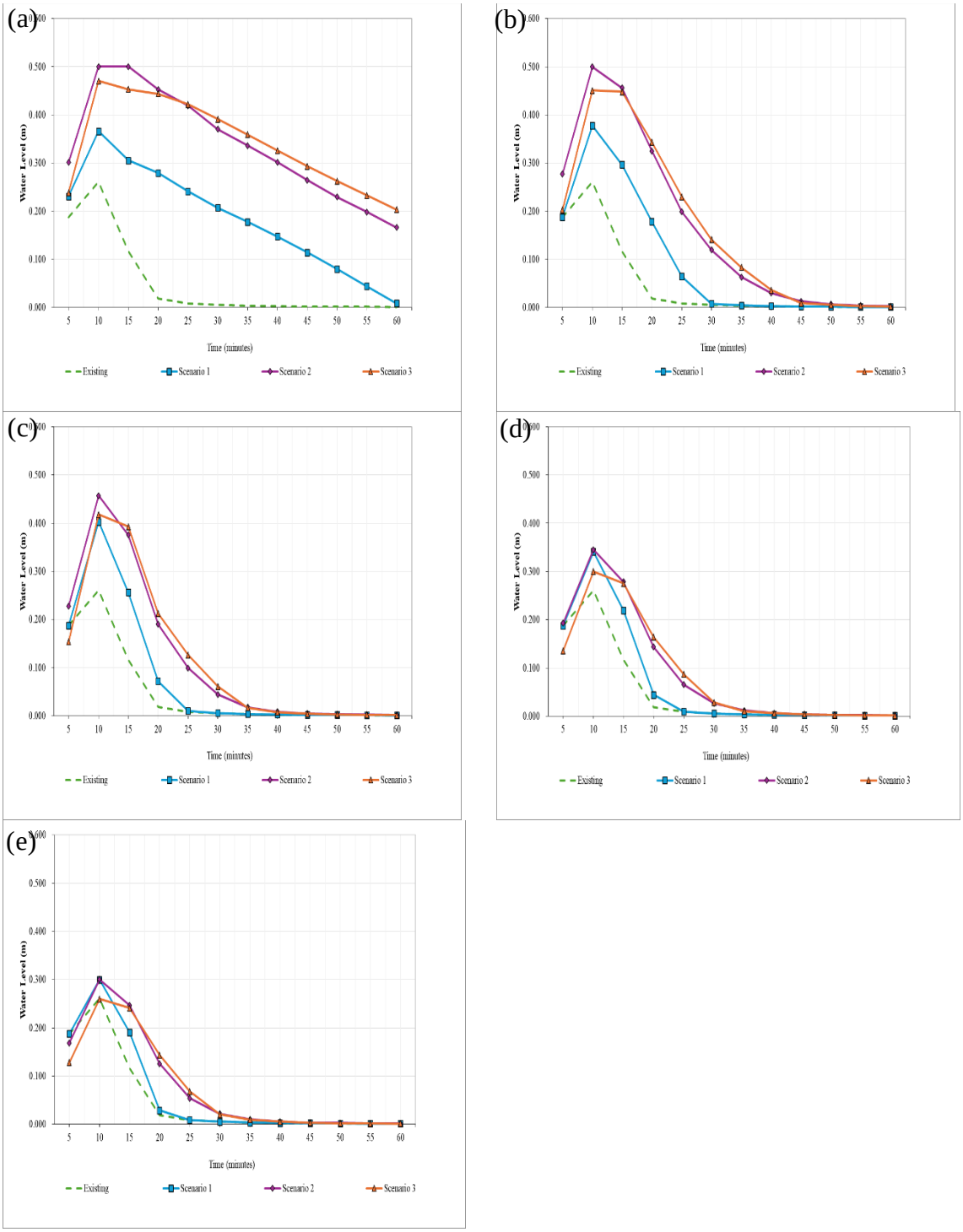


Figure 4.27: Water Level Plots for Drain with 0.2% Slope at Location C Subjected to Design Rainfall, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

by Scenario 3 and Scenario 1 where the water level is increase by 75.1%, 72.7%, and 54.5% respectively by comparing to existing condition.

Implementation of a 0.4m diameter orifice in three scenarios is shown in Figure 4.26d. The flow rate before the 15-minute mark remained below the existing condition, while it exceeded the existing condition after the 15-minute mark. Compared to the existing condition, Scenario 3 showed a significant reduction of 33.5%, followed by Scenario 2 with 20.3% and Scenario 1 with 17.2%. In Figure 4.27d, Scenario 2 exhibited the smallest increment in water level compared to the existing condition, followed by Scenario 1 and Scenario 3, with increments of 14.9%, 30.6% and 32.3% respectively. No flooding occurred in the long open channel with the application of the 0.4m diameter flow restrictor.

The effects of increasing the orifice size from 0.4m to 0.5m in diameter, which allowed a greater volume of water to flow as illustrated in Figure 4.26e. At the peak flow (10-minute mark), the flow rates for Scenarios 1, Scenario 2, and Scenario 3 were reduced by 16.4%, 11.2%, and 28.4%, respectively. Scenario 3 demonstrated the most substantial reduction in flow rate compared to the other scenarios. In Figure 4.27e, Scenario 3 showed the minor reduction in water level by 0.3% whereas Scenario 1 and Scenario 2 were 14.9% and 15.1 respectively when compared to the existing condition. No flooding occurred in the long open channel with the application of the 0.5m diameter flow restrictor.

d. Location D

The flow rates for all scenarios remained below the existing condition prior to the 20-minute mark as illustrated in Figure 4.28a, except for Scenario 1, which showed a 29.8% increase in flow rate at the 10-minute mark compared to the existing condition. Due to the peak flow discharge at the 10-minute mark, the 0.1m diameter orifice in Scenario 1 resulted

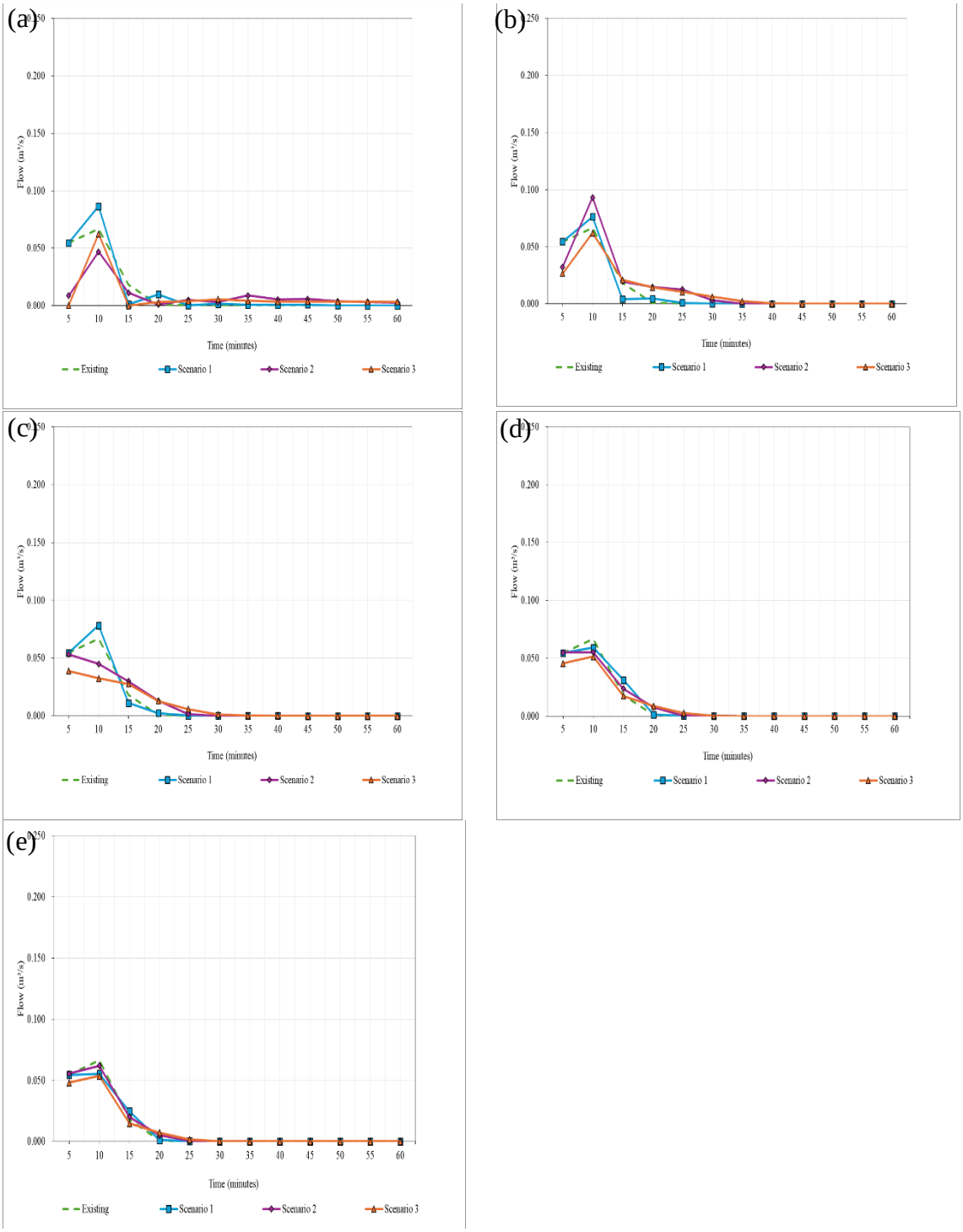


Figure 4.28: Flow Plots for Drain with 0.2% Slope at Location D Subjected to Design Rainfall, (a) 0.1m, (b) 0. 2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

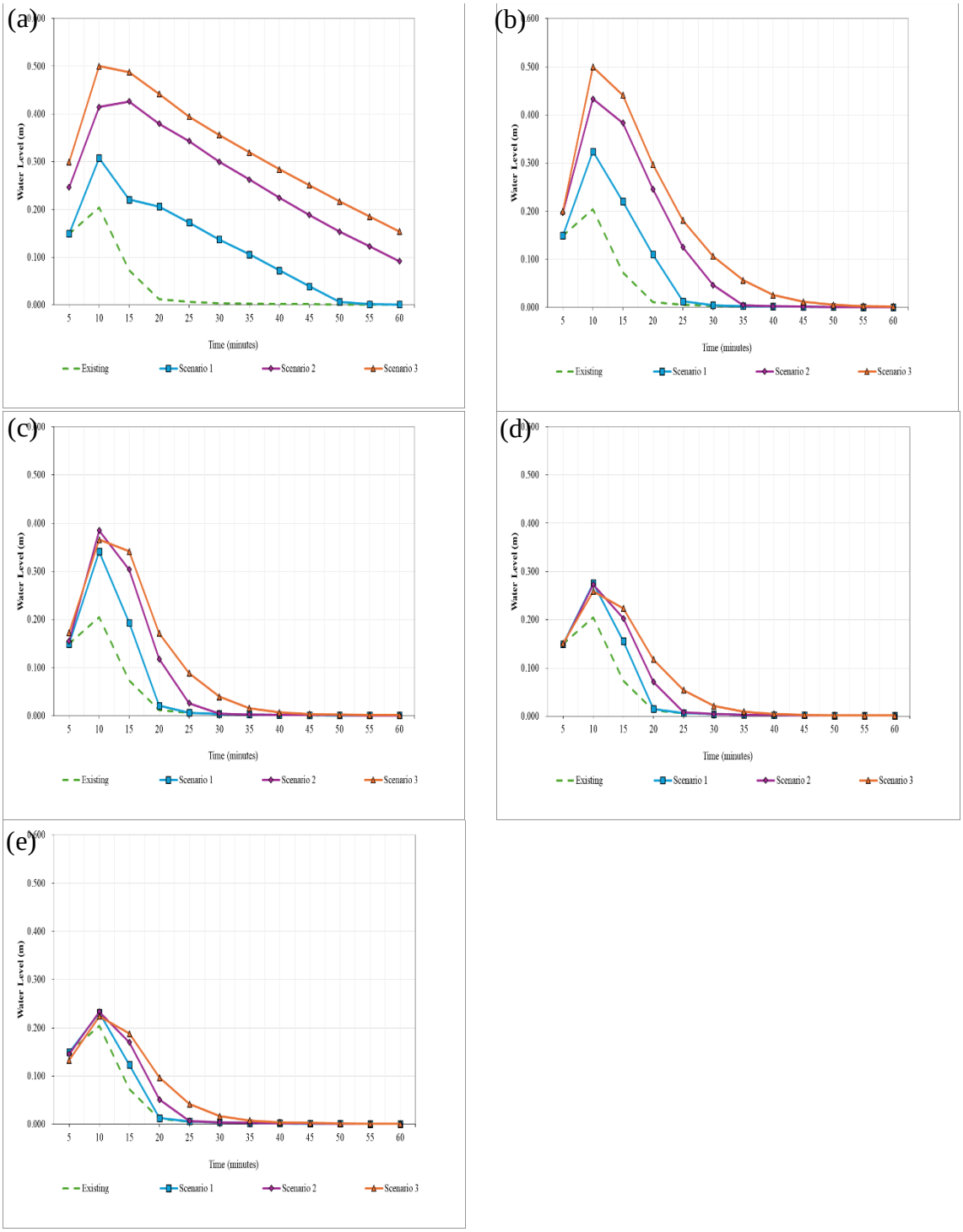


Figure 4.29: Water Level Plots for Drain with 0.2% Slope at Location D Subjected to Design Rainfall, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

in a higher flow rate compared to Scenarios 2 and 3 at Location D, indicating that it did not effectively reduce upstream flow discharge. At the peak flow (10-minute mark), Scenarios 2 and 3 demonstrated reductions of 29.5% and 6.2%, respectively, compared to the existing condition, with Scenario 2 showing the most substantial reduction in flow rate at Location A. Post the 20-minute mark, the flow rate for all scenarios surpassed the existing condition, indicating an overall increase in flow rate. Figure 4.29a illustrates that Scenarios 1, 2, and 3 experienced a 50.4%, 102.5%, and 144.4% respectively increase in water level compared to the existing condition. This resulted in flooding, as the water level reached 0.5m for Scenario 3 where Scenario 1 and Scenario 2 do not experience flood throughout the rainfall.

The orifice size increased from 0.1m to 0.2m in diameter, the flow rates for all scenarios remained below the existing condition before the 20-minute mark and exceeded the existing condition thereafter as shown in Figure 4.28b. At the 10-minute peak flow rate, Scenario 1 and Scenario 2 showed a 14.3% and 39.8% increase in flow rate, while Scenario 3 demonstrated reductions of 6.8%, respectively. Scenario 3 exhibited a significant reduction in flow rate. In Figure 4.29b, all scenarios with a 0.2m diameter orifice flow restrictor experienced increasing where Scenario 3 had the highest increment follow by Scenario 2 and Scenario 1 where were 144.4%, 11.8%, and 58.4% respectively. Flood was found at Scenario 3 at 10-minute mark whereas no flood was found at Scenario 1 and Scenario 2.

The existing condition and the implementation of a 0.3m diameter orifice in three scenarios as shown in Figure 4.28c. The flow rate before the 15-minute mark was below the existing condition, except for Scenario 1. After the 15-minute mark, the flow rate exceeded the existing condition. Compared to the existing condition, Scenario 1 showed a 17.4% increase, while Scenarios 2 and 3 demonstrated reductions of 32.6% and 51.3%,

respectively. Scenario 3 exhibited the most significant reduction compared to the existing condition. In Figure 4.29c, Scenario 2 had the highest increase in water level follow by Scenario 3 and Scenario 1 which were 88.1%, 78.8% and 66.9% respectively. No flood was found throughout the rainfall event.

The implementation of a 0.4m diameter orifice in three scenarios is shown in Figure 4.28d. The flow rate before the 15-minute mark remained below the existing condition, while it exceeded the existing condition after the 15-minute mark. Compared to the existing condition, Scenario 3 showed a significant reduction of 22.8%, followed by Scenario 2 with 17.7% and Scenario 1 with 11.05%. In Figure 4.29d, Scenario 3 exhibited the smallest increment in water level compared to the existing condition, followed by Scenario 2 and Scenario 1, with increments of 26.7%, 33.5%, and 34.9%, respectively. No flooding occurred in the long open channel with the application of the 0.4m diameter flow restrictor.

The effects of increasing the orifice size from 0.4m to 0.5m in diameter as shown in Figure 4.28e, which allowed a greater volume of water to flow. At the peak flow (10-minute mark), the flow rates for Scenarios 1, 2, and 3 were reduced by 17.1%, 7.1%, and 19.6%, respectively. Scenario 3 demonstrated the most substantial reduction in flow rate compared to the other scenarios. In Figure 4.29e, Scenario 3 showed the smallest increment in water level compared to the existing condition, followed by Scenario 1 and Scenario 2, with increments of 9.6%, 13.3%, and 14.1%, respectively. No flooding occurred in the long open channel with the application of the 0.5m diameter flow restrictor.

e. Discussion

Figures 4.30 and 4.31 summarize the peak flow rate and water level for a design slope of 0.2% under a 5-minute, 10-year ARI design rainfall.

At Location A, the peak flow rate decreased for all scenarios and orifice sizes, except for Scenario 1 with orifice diameters of 0.1m, 0.2m, and 0.3m, which showed an increase. This indicates that the reduction in flow rate decreases the rainfall discharge to the outfall, thereby reducing congestion at the upstream end. The highest reduction in flow rate occurred in Scenario 2 with a 0.1m orifice diameter, while the lowest reduction was in Scenario 1 with a 0.5m orifice diameter. However, the peak water level increased for all scenarios and orifice sizes. Every scenario with orifice diameters of 0.1m, 0.2m, and 0.3m reached the maximum water level of 0.5m, causing potential flooding. Thus, these scenarios failed to meet the objective.

At Location B, the peak flow rate decreased for all scenarios and orifice sizes, except for Scenario 1 with orifice diameters of 0.1m, 0.2m, and 0.3m, which showed an increase. The increased flow rate compared to the existing flow rate would lead to more rainfall water flowing to Location A for discharge. Thus, reducing the flow rate is a key objective in this study. The graph shows that Scenario 2 with a 0.1m orifice diameter had the highest reduction, whereas Scenario 1 with a 0.5m orifice diameter showed the least reduction. For peak water levels at Location B, all scenarios showed an increase compared to the existing water level, except Scenario 2 with a 0.5m orifice diameter, which had a minor reduction. Flooding occurred with orifice diameters of 0.1m and 0.2m in Scenario 3, where water levels increased to the maximum drain water level of 0.5m. Hence, these analyses for Scenario 3 failed as they caused flooding.

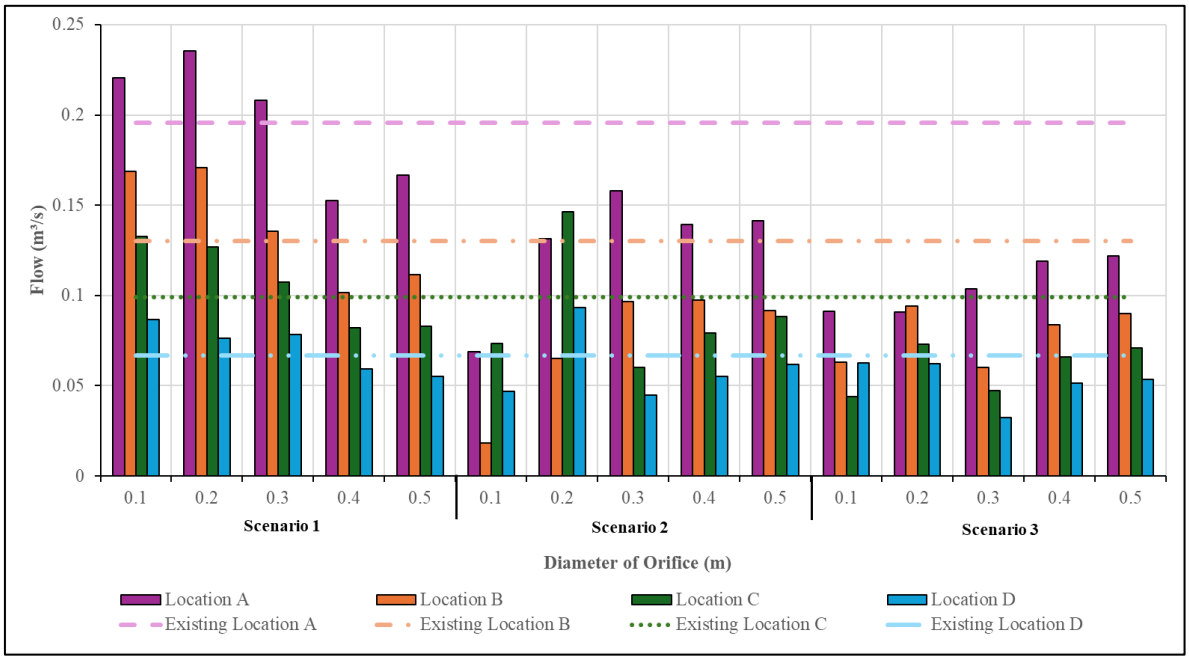


Figure 4.30: Summary of Peak Flow Rate for 5-minute 10-year ARI Design Rainfall at 0.2% Design Slope of Open Channel

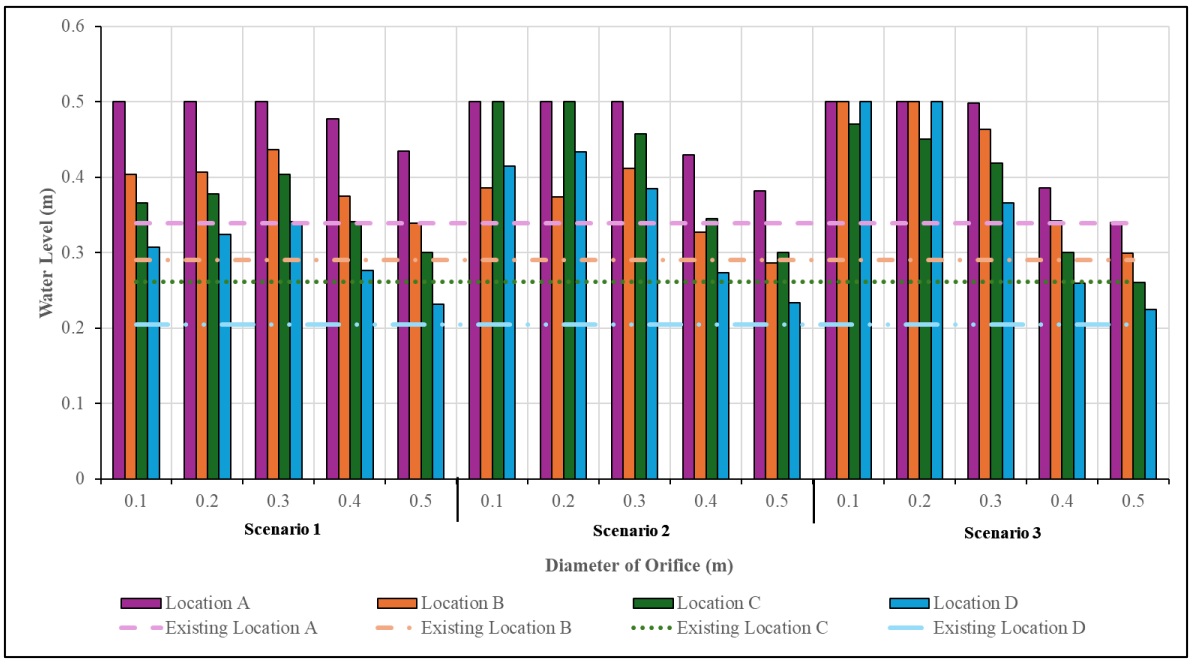


Figure 4.31: Summary of Peak Water Level for 5-minute 10-year ARI Design Rainfall at 0.2% Design Slope of Open Channel

At Location C, the peak flow rate decreased for all scenarios and orifice sizes, except for Scenario 1 with orifice diameters of 0.1m, 0.2m, and 0.3m, and Scenario 2 with a 0.2m orifice diameter, which showed an increase. Scenario 3 with a 0.1m orifice diameter had the highest reduction in flow rate. Greater reductions in flow rate result in slower rainfall discharge to Locations A and B. Peak water levels increased compared to existing conditions, with Scenario 3's 0.5m orifice diameter showing similar readings to existing conditions. However, Scenario 2 with orifice diameters of 0.1m and 0.2m reached the maximum water level of 0.5m, leading to failure in this analysis due to flooding.

At Location D (upstream end), The peak flow rate decreased for all scenarios and orifice sizes, except for Scenario 1 with orifice diameters of 0.1m, 0.2m, and 0.3m, and Scenario 2 with a 0.2m orifice diameter, which showed an increase. The highest reduction in flow rate occurred with Scenario 3's 0.3m orifice diameter, while the lowest reduction was with Scenario 3's 0.1m orifice diameter. Peak water levels increased across all scenarios compared to existing conditions. However, Scenario 3 with orifice diameters of 0.1m and 0.2m reached a water level of 0.5m, causing flooding and thus failing to meet the objectives.

In summary, the analysis showed that scenarios incorporating orifice diameters of 0.4m and 0.5m consistently performed the best, effectively reducing the peak flow rate while preventing water levels from reaching the drain's maximum capacity of 0.5m. These configurations demonstrated a more balanced hydraulic performance, especially under intense short-duration rainfall, by moderating flow discharge without triggering upstream flooding. In contrast, smaller orifice sizes (0.1m to 0.3m) in Scenario 1 and parts of Scenario 2 and 3 were prone to causing flooding, particularly at Locations A, B, and D, where water levels peaked at 0.5m. Although these smaller orifices often provided the highest reductions

in flow rate, their inability to maintain safe water levels made them unsuitable for this design scenario. The results are consistent with prior studies that highlight the necessity of balancing flow control and flood prevention when applying flow restriction mechanisms in open channel systems (Araoye et al., 2017; Kuok and Chiu, 2022).

4.2.2.2 Historical Rainfall

The hydrographs presented in Figures 4.32, Figure 4.34, Figure 4.36, and Figure 4.38 illustrate the simulated flow rates, while Figures 4.33, Figure 4.35, Figure 4.37, and Figure 4.39 depict the corresponding water levels for Historical Rainfall Event 1. This event is selected for detailed presentation and discussion in this chapter. Meanwhile, the hydrographs for Event 2 and Event 3 are included in Appendix I for reference, as their flow rate and water level patterns were found to be largely consistent with those observed in Event 1. These were meant for 0.2% slope. Different drain slope had different characteristics that were described in Section 4.3.

The patterns observed with 0% slope were repeated here. Firstly, the flow rates fluctuated the most in sub-figures (a) throughout. The other sub-figures (b) to (e) were of the similar patterns to existing condition. Secondly, the water levels had the most significant changes in sub-figures (a) throughout. Water levels in sub-figures (a)were the highest. Water levels in sub-figures (b) to (e) were lesser but in all cases, slightly higher than the existing condition.

a. Location A

Flow rates shown in Figure 4.32a describe for all scenarios generally remained below the existing condition. Scenario 3, which included three orifice flow restrictors along the

open channel, provided greater detention capacity compared to Scenarios 1 and 2. At the 13-hour peak flow discharge, the 0.1m diameter orifice in Scenario 1 resulted in a higher flow rate compared to Scenarios 2 and 3 at Location A, indicating it did not effectively reduce downstream flow discharge. At the peak flow (13-hour mark), Scenarios 1, 2, and 3 showed reductions of 2.0%, 23.1%, and 39.7%, respectively, compared to the existing condition, with Scenario 3 showing the most substantial reduction. Figure 4.33a illustrates that Scenarios 1, 2, and 3 experienced a 454% increase in water level compared to the existing condition, resulting in flooding as the water level reached 0.5m, due to the combination of the 0.1m orifice flow restrictor and the 0.2% drain design slope in the open channel.

The orifice size increased from 0.1m to 0.2m in diameter, the flow rates for all scenarios generally remained below the existing condition throughout the rainfall event as shown in Figure 4.32b. At the 13-hour peak flow rate, Scenarios 1, 2, and 3 showed reductions of 2.3%, respectively. Figure 4.33b shows that all scenarios with a 0.2m diameter orifice flow restrictor experienced a 138% increase in water level compared to the existing condition. However, no flooding occurred at Location A as the water level remained below 0.5m.

The existing condition and the implementation of a 0.3m diameter orifice in three scenarios were shown in Figure 4.32c. The flow rate for all scenarios was below the existing condition, showing a nearly uniform reduction of 2.3%. However, in Figure 4.33c indicates that all scenarios with a 0.3m diameter orifice flow restrictor experienced a 98.5% increase in water level compared to the existing condition. No flooding occurred with the application of a 0.3m diameter orifice in this simulation.

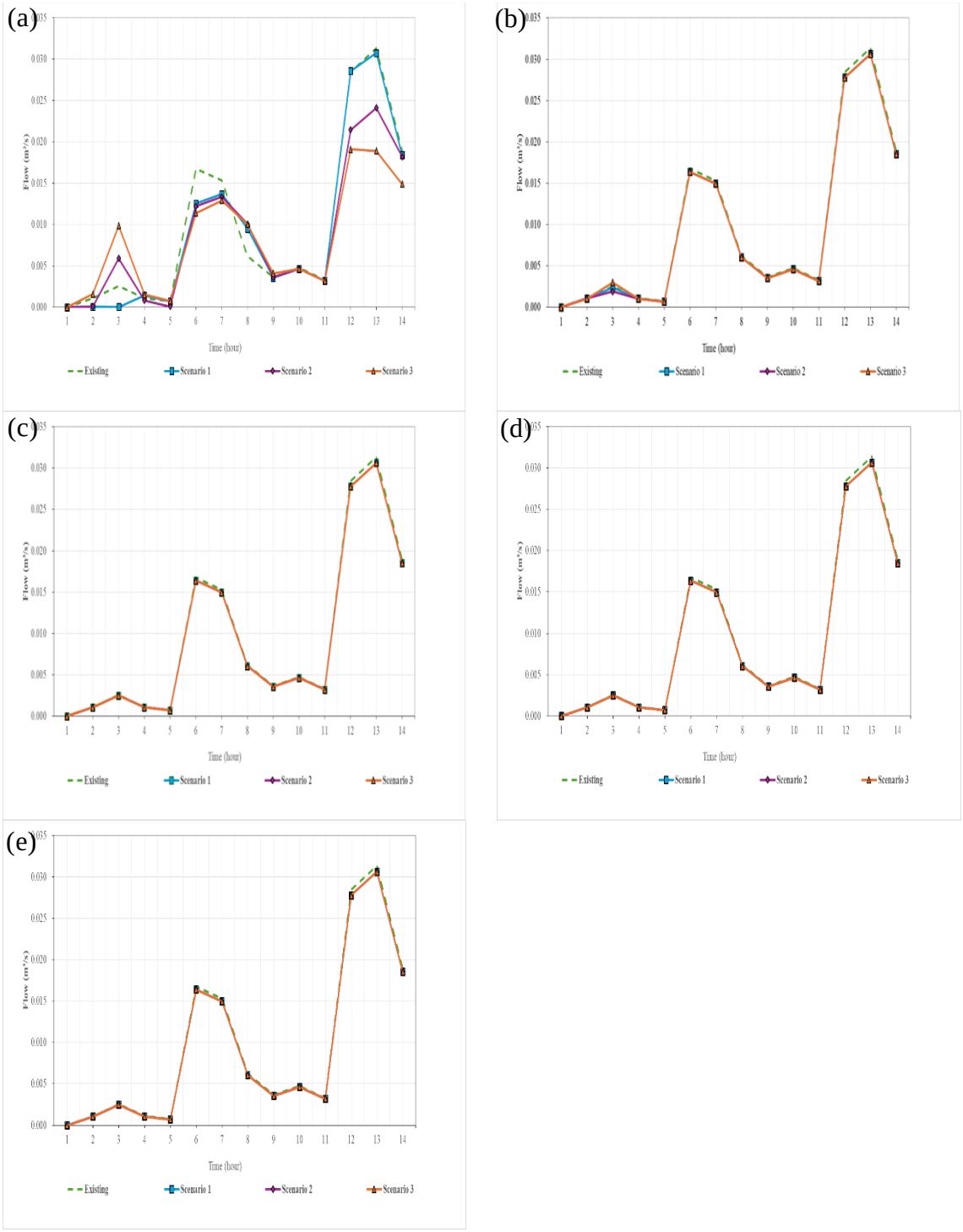


Figure 4.32: Flow Plots for Drain with 0.2% Slope at Location A Subjected to Event 1, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

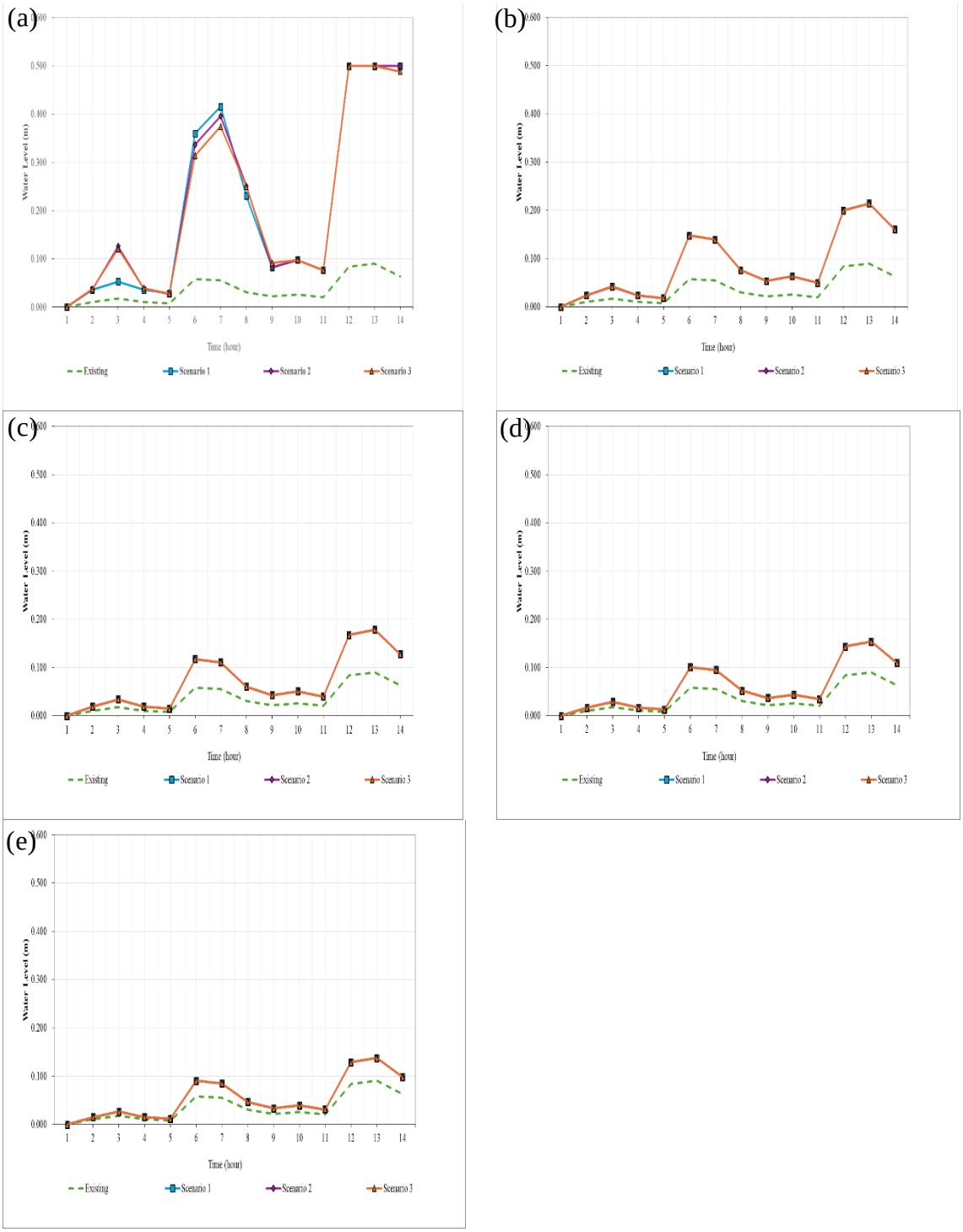


Figure 4.33: Water Level Plots for Drain with 0.2% Slope at Location A Subjected to Event 1, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

Orifice size increased to 0.4m in diameter in Figure 4.32d, the flow rates for all scenarios generally remained below the existing condition throughout the rainfall event. At the 13-hour peak flow rate, Scenarios 1, 2, and 3 showed reductions of 2.3%, respectively. All scenarios with a 0.4m diameter orifice flow restrictor experienced a 70.4% increase in water level compared to the existing condition in Figure 4.33d. However, no flooding occurred at Location A as the water level remained below 0.5m.

The orifice size increased to 0.5m in diameter in Figure 4.32e, the flow rates for all scenarios generally remained below the existing condition throughout the rainfall event. At the 13-hour peak flow rate, Scenarios 1, 2, and 3 demonstrated reductions of 2.3%, respectively. Figure 4.33e shows that all scenarios with a 0.5m diameter orifice flow restrictor experienced a 52.9% increase in water level compared to the existing condition. However, no flooding occurred at Location A as the water level remained below 0.5m.

b. Location B

Flow rates for all scenarios generally remained below the existing condition as shown in Figure 4.34a. At the 13-hour peak flow discharge, the 0.1m diameter orifice in Scenario 1 resulted in a higher flow rate compared to Scenarios 2 and 3 at Location B, indicating it did not effectively reduce flow discharge. At the peak flow (13-hour mark), Scenarios 1, 2, and 3 showed reductions of 0.38%, 31.4%, and 13.4%, respectively, compared to the existing condition, with Scenario 2 showing the most substantial reduction. Figure 4.35a illustrates that Scenarios 1, 2, and 3 experienced increasing in water level compared to the existing. condition where 445.4%, 445%, and 616.5%. This had resulting in flooding after 12-hour mark as the water level reached 0.5m.

Orifice size increased to 0.2m in diameter, the flow rates for all scenarios generally

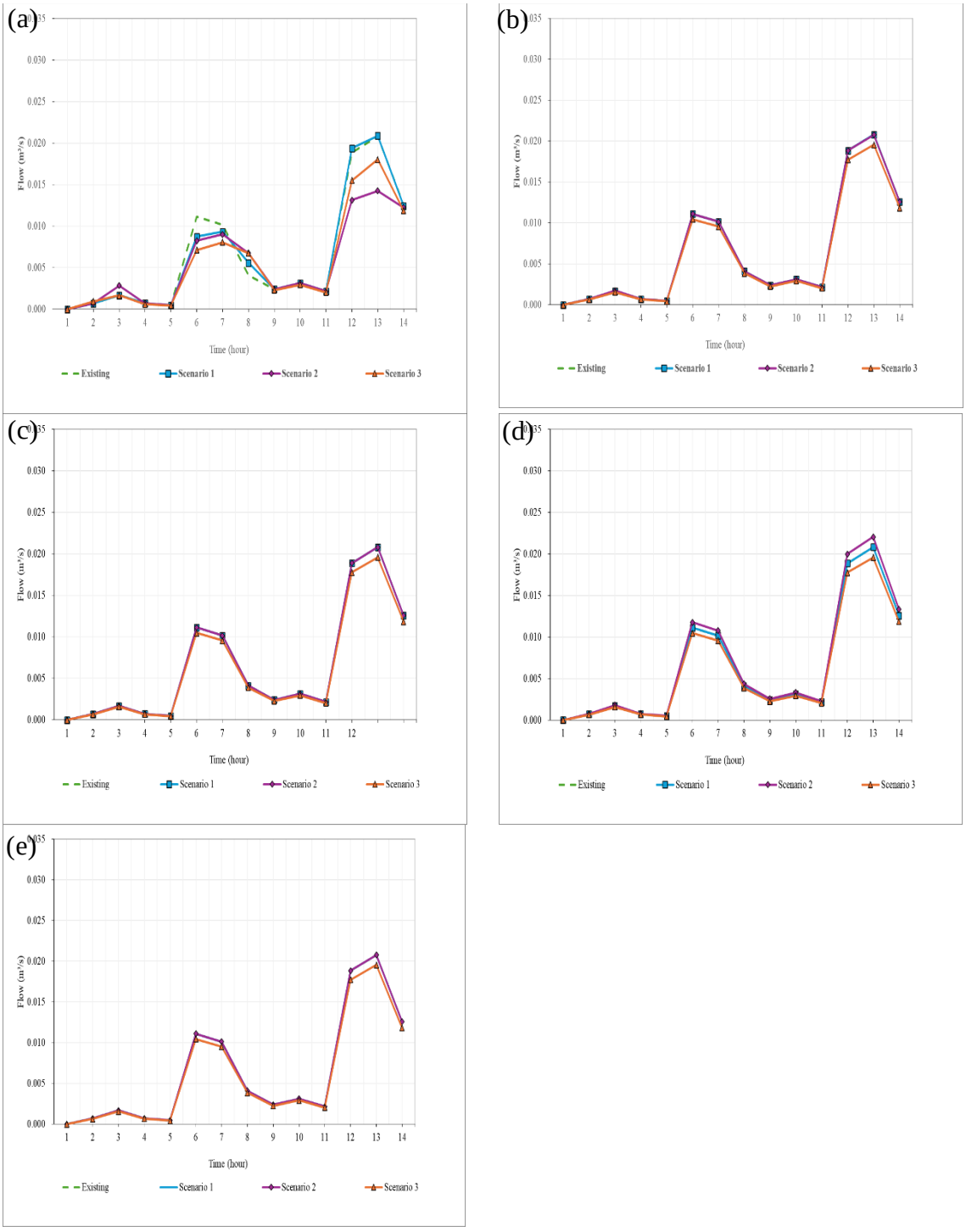


Figure 4.34: Flow Plots for Drain with 0.2% Slope at Location B Subjected to Event 1, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

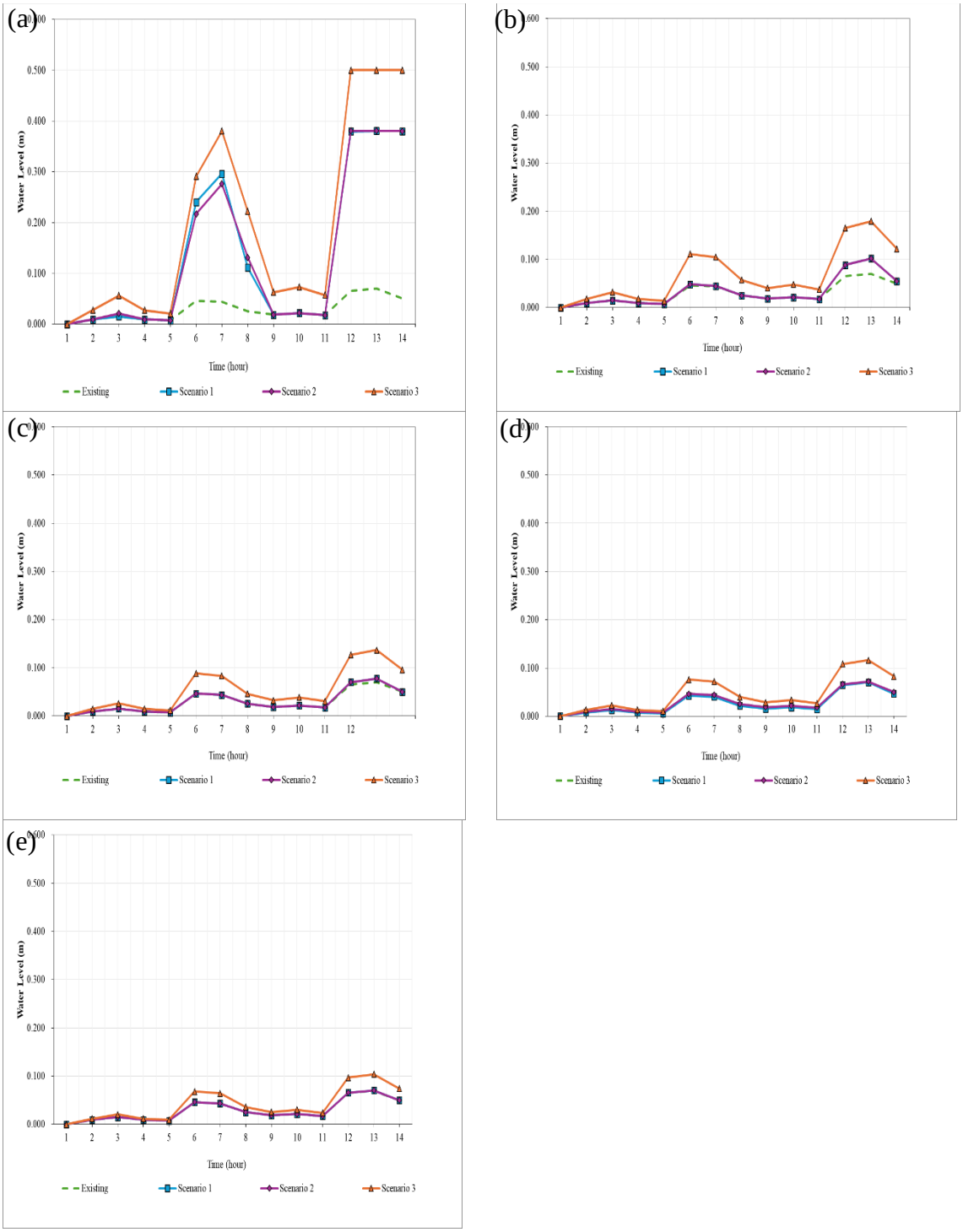


Figure 4.35: Water Level Plots for Drain with 0.2% Slope at Location B Subjected to Event 1, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

remained below the existing condition throughout the rainfall event as shown in Figure 4.34b. At the 13-hour peak flow rate, Scenario 1 at Location B remained the same flow rate as the existing condition. However, Scenario 2 and Scenario 3 had the minor reduction which were 0.05% and 6% respectively. Figure 4.35b shows that Scenario 3 had the highest increment follow by Scenario 1 and Scenario 2 which were 157%, 46.6%, and 46.5%. respectively. However, no flooding occurred at Location B for all the scenario as the water level remained below 0.5m throughout the rainfall event.

Existing condition and the implementation of a 0.3m diameter orifice in three scenarios is illustrated in Figure 4.34c. The flow rate at Location B for Scenario 1 and Scenario 2 have the similar flow rate as existing condition. However, Scenario 3 had a minor reduction compared to existing condition which were 5.9%. Figure 4.35c indicates that Scenario 1 and Scenario 2 at Location B only had minor increment in water level which are 11.5%. However, Scenario 3 had the highest increment in water level compared to Scenario 1 and 2, which was increase by 96.1%. No flooding occurred with the application of a 0.3m diameter orifice in this simulation.

Orifice size increased to 0.4m in diameter in Figure 4.34d, the flow rate of Scenario 1 had the similar reading with existing condition where Scenario 2 had a slightly increase at flow rate and Scenario 3 had the slight reduction compared to existing condition . Scenario 2 had increase by 5.9% whereas Scenario 3 had reduction by 5.9% by comparing to existing condition. Figure 4.35d demonstrates that Scenario 3 had the highest increment follow by Scenario 2 and Scenario 1 which are 66.3%, 2.7%, and 1% respectively. No flooding occurred at Location B as the water level remained below 0.5m.

Orifice size increased to 0.5m in diameter in Figure 4.34e, there were no increment or reduction for Scenario 1 and Scenario 2 for the flow rate at Location B. However, Scenario 3 shows the minor reduction which were 5.31%. Figure 4.35e shows that Scenarios 3 had the largest reduction which was 49.0%. However, Scenario 1 and Scenario 2 had the similar reduction were 1.03%. with a 0.5m diameter orifice flow restrictor However, no flooding occurred at Location A as the water level remained below 0.5m.

c. Location C

Flow rates for all scenarios generally remained below the existing condition as shown in Figure 4.36a. At the peak flow (13-hour mark), Scenarios 1 had the increasing in flow rate which are 0.6% while Scenario 2 and Scenario 3 had the reduction of flow rate comparing with existing condition which were 4.2% and 9.7% respectively. Figure 4.37a illustrates that Scenarios 1, Scenario 2, and Scenario 3 had increased the water level of 451.2%, 720.8%, and 647.0% respectively. Scenario 1 had the lowest increment compared to Scenario 2 and Scenario 3. However, the flood for Scenario 2 happened after the 12-hour mark. No flood was found for other scenario.

The orifice size increased to 0.2m in diameter, the flow rates for all scenarios generally approximately to the existing condition throughout the rainfall event at Location C as illustrated in Figure 4.36b. At the 13-hour peak flow rate, Scenario 2 had the similar flow rate with the existing condition whereas Scenario 2 and Scenario 3 shows the reduction by 1.2% and 0.1%. Figure 4.37b shows the water level for Scenario 2 had increases by 142.4%, followed by Scenario 3 which increase by 122.6% and Scenario 1 increase by 12.0% at Location C. However, no flooding occurred at Location C as the water level remained below 0.5m.

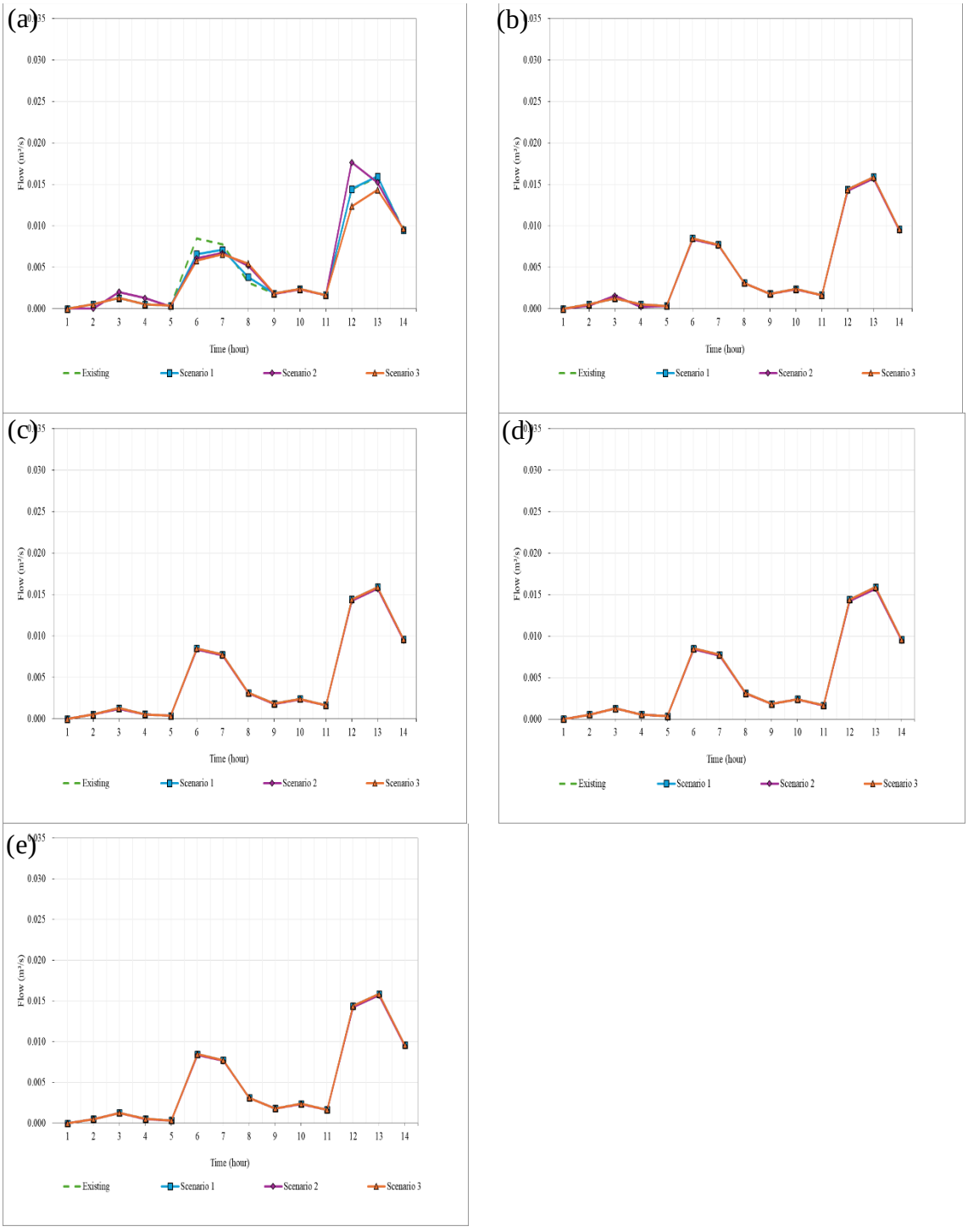


Figure 4.36: Flow Plots for Drain with 0.2% Slope at Location C Subjected to Event 1, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

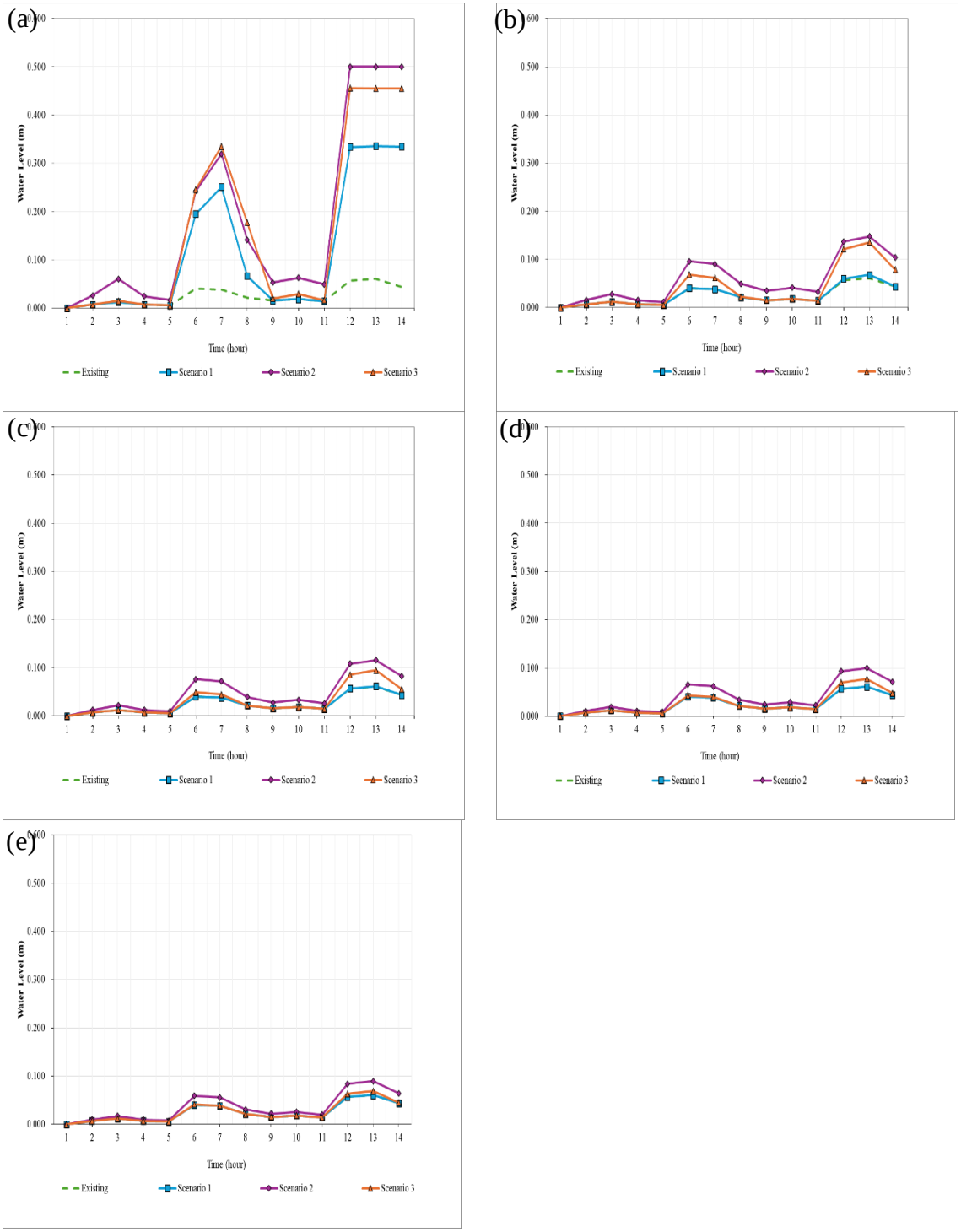


Figure 4.37: Water Level Plots for Drain with 0.2% Slope at Location C Subjected to Event 1, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

The existing condition and the implementation of a 0.3m diameter orifice in three scenarios as shown in Figure 4.36c. The flow rate for Scenario 1 and Scenario 3 remains unchanged to the existing condition. However, Scenario 2 show the minor reduction which was 1.2% by comparing to existing condition. Figure 4.37c indicates the Scenario 2 have the highest increment of water level compared to existing condition follow by Scenario 3 and Scenario 1 which were 90.5%, 56.1% and 1.2% respectively. No flooding occurred with the application of a 0.3m diameter orifice in this simulation.

Orifice size increased to 0.4m in diameter, the flow rates for all scenarios generally remained below the existing condition throughout the rainfall event as shown in Figure 4.36d. At the 13-hour peak flow rate, Scenario 1 and Scenario 3 remain unchanged compare to existing condition. However, Scenario 2 shows a slightly reduction by 1.1% by comparing to existing condition. Figure 4.37d demonstrates that water level where Scenario 1 had the lowest increment in water level follow by Scenario 3 and Scenario 2 where increase by 0.2%, 26.8%, and 63.9% respectively. No flooding occurred at Location C as the water level remained below 0.5m.

Orifice size increased to 0.5m in diameter, the flow rates for all scenarios generally remained below the existing condition throughout the rainfall event as illustrated in Figure 4.36e. At the 13-hour peak flow rate, Scenario 1 and Scenario 3 remain unchanged compared to existing condition. However, Scenario 2 shows a slightly reduction by 1.1% by comparing to existing condition. Figure 4.37e shows that water level where Scenario 1 had the lowest increment in water level follow by Scenario 3 and Scenario 2 where increase by 0.1%, 13.4%, and 47.5% respectively. No flooding occurred at Location C as the water level remained below 0.5m.

d. Location D

Flow rates for all scenarios generally remained below the existing condition as illustrated in Figure 4.38a. At the 13-hour peak flow discharge, the 0.1m diameter orifice in Scenario 1 resulted in a higher flow rate compared to Scenarios 2 and 3 at Location D, indicating it did not effectively reduce downstream flow discharge. At the peak flow (13-hour mark), Scenarios 1, had minor increase in flow rate by 0.8% whereas Scenario 2 and Scenario 3 had minor reduction in flow rate by 2.4%, and 15.4% respectively. Figure 4.39a illustrates that Scenarios 1, 2, and 3 experienced a increase in water level by 471.4%, 830.3%, and 994.6% respectively compared to the existing condition. Flood was found for Scenario 3 at 13-hour mark whereas no flood were found at Scenario 1 and Scenario 2.

The orifice size increased from 0.1m to 0.2m in diameter, the flow rates for all scenarios generally remained below the existing condition throughout the rainfall event at shown in Figure 4.38b. At the 13-hour peak flow rate, Scenarios 1, and Scenario 2, remained unchanged whereas Scenario 3 had a minor reduction by 11.6%. Figure 4.39b shows that the water level for Scenario 1 had the lowest increment follow by Scenario 2 and Scenario 3 where increase by 0.1%, 65.7%, and 142.6% respectively.

The existing condition and the implementation of a 0.3m diameter orifice in three scenarios as shown in Figure 4.38c. The flow rate for Scenario 1 and Scenario 2 remains unchanged whereas Scenario 3 have a slight reduction by 11.5% by comparing to existing condition. Figure 4.39c indicates that Scenario 1 had the lowest increment in water level follow by Scenario 2 and Scenario 3 which were 0.1%, 12.6% and 82.4% respectively. No flooding occurred with the application of a 0.3m diameter orifice in this simulation.

The orifice size increased to 0.4m in diameter, the flow rate for Scenario 1 and

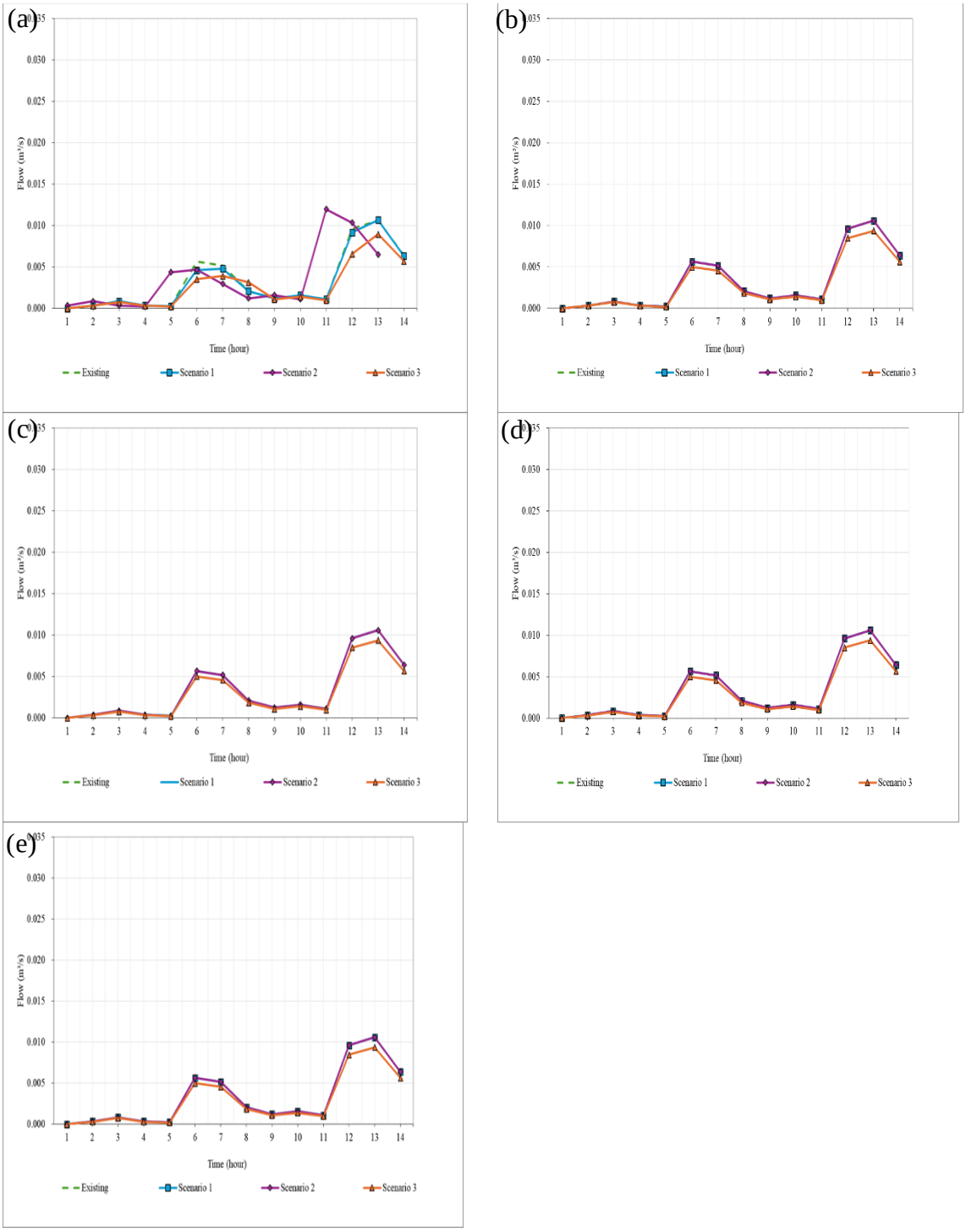


Figure 4.38: Flow Plots for Drain with 0.2% Slope at Location D Subjected to Event 1, (a) 0.1m, (b) 0. 2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

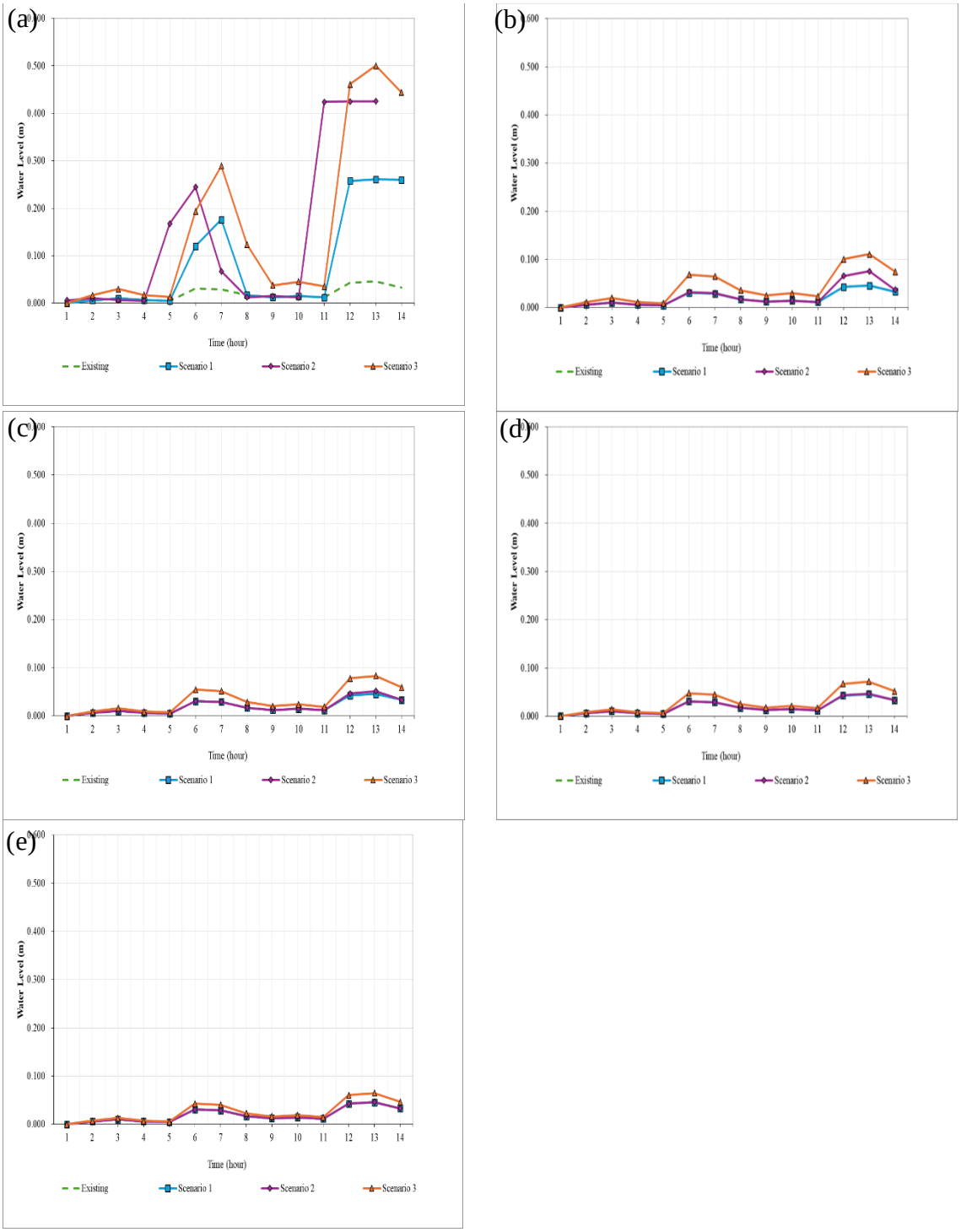


Figure 4.39: Water Level Plots for Drain with 0.2% Slope at Location D Subjected to Event 1, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

Scenario 2 remains unchanged whereas Scenario 3 have a slight reduction by 11.5% by comparing to existing condition as illustrated in Figure 4.38d. Figure 4.39d demonstrates that Scenario 1 had remain unchanged in water level whereas the Scenario 2 and Scenario 3 had slight increase in water level where were 2.7% and 57.3% respectively. However, no flooding occurred at Location D as the water level remained below 0.5m.

The orifice size increased to 0.5m in diameter, the flow rates for all scenarios generally remained below the existing condition throughout the rainfall event as shown in Figure 4.38e. At the 13-hour peak flow rate, Scenarios 1, and Scenario 2 remain unchanged whereas Scenario 3 had a minor reduction by 11.5%. Figure 4.39e shows that the water level for Scenario 1 remain unchanged whereas the Scenario 2 and Scenario 3 had increasing in water level by 1% and 42.1% respectively. However, no flooding occurred at Location D as the water level remained below 0.5m.

e. Discussion

Figures 4.40 and 4.41 summarize the peak flow rate and water level for a design slope of 0.2% during Event 1 historical rainfall. At Location A, the peak flow rate decreased for all scenarios and orifice sizes compared to the existing condition, indicating a reduction in rainfall discharge to the outfall and decreased congestion at the upstream end. For this historical event, only the 0.1m diameter orifice for Scenario 2 and Scenario 3 showed significant reductions in flow rate, with Scenario 3 having the highest reduction. Other conditions only showed minor reductions compared to the existing condition. However, the peak water level increased for all scenarios and orifice sizes, with every scenario using a 0.1m diameter orifice reaching the maximum water level of 0.5m, potentially causing flooding. No flooding occurred with the 0.2m, 0.3m, and 0.4m diameter orifices in any

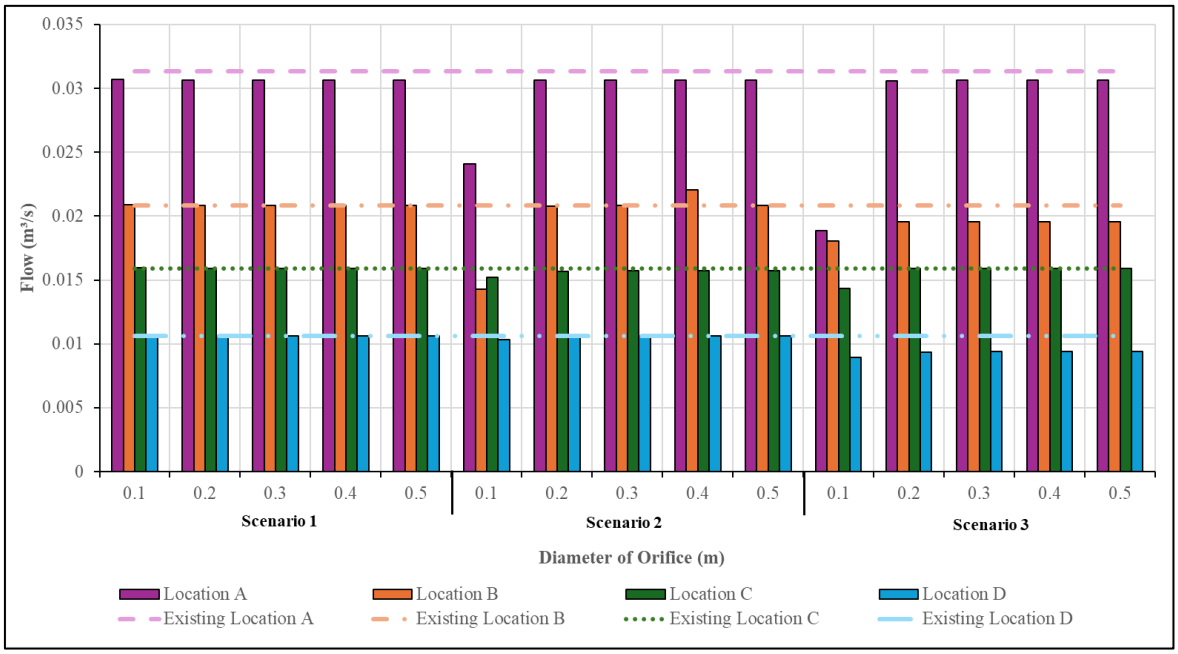


Figure 4.40: Summary of Peak Flow Rate for Event 1 Historical Rainfall at 0.2% Design Slope of Open Channel

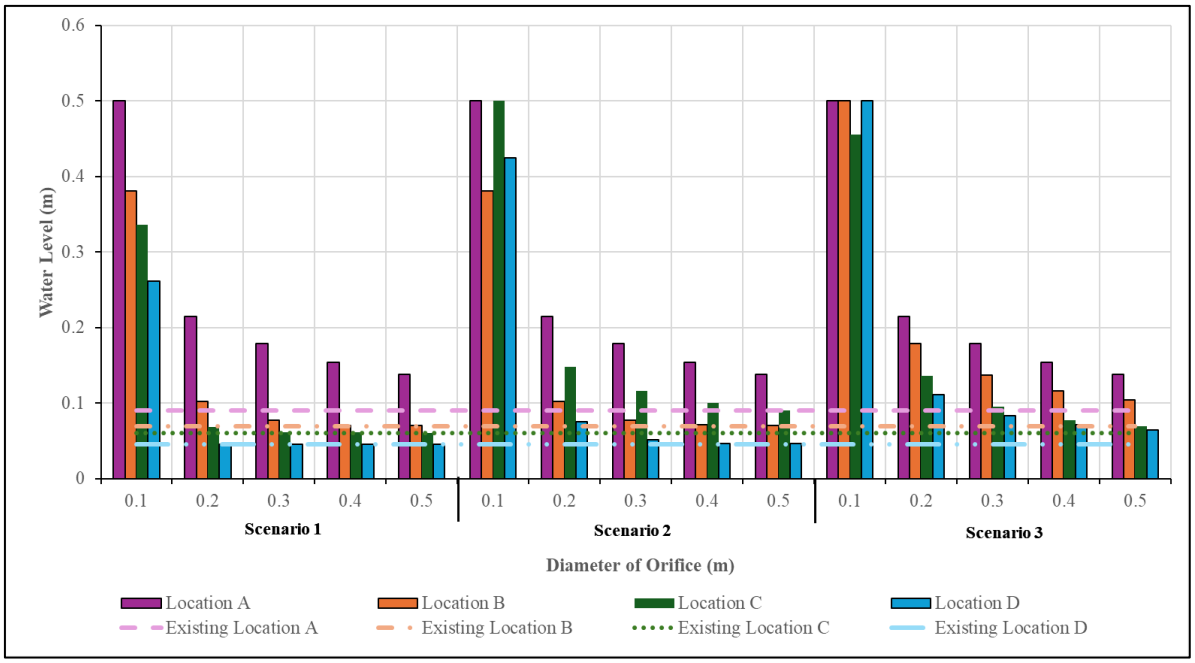


Figure 4.41: Summary of Peak Water Level for Event 1 Historical Rainfall at 0.2% Design Slope of Open Channel

scenario. Thus, scenarios causing flooding failed to meet the objective.

At Location B, most peak flow rates remained unchanged compared to the existing condition, except for the 0.1m diameter orifice for Scenario 2, which showed a significant reduction. The 0.4m diameter orifice in Scenario 2 had a slight increase in flow rate, which would lead to more rainfall water flowing to Location A for discharge. Therefore, reducing the flow rate is a key objective in this study. For peak water levels at Location B, all scenarios showed an increase compared to the existing water level. Flooding was observed with the 0.1m diameter orifice for Scenario 3, as the water level increased to 0.5m, the maximum drain water level. Hence, the analysis for Scenario 3 with a 0.1m diameter orifice failed due to flooding.

At Location C, the peak flow rate for all scenarios and orifice sizes was almost similar to the existing condition, except for the 0.1m diameter orifice for Scenario 3, which showed the highest reduction in flow rate compared to other scenarios. Greater reductions in flow rate result in slower rainfall discharge to Locations A and B. Peak water levels increased compared to the existing condition, with the 0.1m diameter orifice flow restrictor for Scenario 2 reaching the maximum water level of 0.5m, leading to failure in this analysis due to flooding. Additionally, the 0.1m diameter orifice for Scenario 3 had the second-highest increase in water level.

At Location D, the upstream end, the peak flow rate for Scenario 1 and Scenario 2 was approximately similar to the existing condition, whereas Scenario 3 showed a significant reduction in flow rate. Reducing the flow rate can help decrease the rainfall discharge to Location C. Peak water levels for all scenarios increased or remained unchanged compared to the existing condition. However, the 0.1m diameter orifice for all three scenarios showed

a significant increase in water level, with Scenario 3 reaching a 0.5m water level, causing flooding.

In summary, the smaller the size of orifices, especially the 0.1m diameter, were effective in reducing peak flow rates particularly in Scenario 3， they often led to flooding due to increased water levels reaching the system's maximum capacity. In contrast, larger orifices (0.2m to 0.4m) offered a more balanced performance, lowering flow rates moderately while keeping water levels within safe limits. These findings align with previous studies such as Zheng et al. (2020) and Sembiring et al. (2022), which emphasize the trade-off between flow restriction and flood risk. The results also support Kuok and Chiu (2022), who stress the importance of properly sizing orifices to match rainfall intensity, and Araoye et al. (2017), who highlight the influence of orifice diameter on flow behavior. Overall, this section reinforces that orifice-based control is effective, but its success depends on size selection and system context.

4.2.3 Drain with 0.4% Slope

4.2.3.1 Design Rainfall

The hydrograph representations resulting from the design rainfall, both with and without flow restrictor for open drain with 0.4% design slope, are depicted in Figure 4.42, Figure 4.44, Figure 4.46, and Figure 4.48 for flow rates, and Figure 4.43, Figure 4.45, Figure 4.47, and Figure 4.49 for water levels. Comparing the three slopes, this 0.4% slope is the steepest, and as such, the flow rates, both existing and modified conditions, were the highest. The modified flow rates were nearing the value of existing condition. In terms of water level, the estimated values were higher than existing condition in most cases. It indicates that the

steep slope introduced faster flows; and at the orifice point, the water levels before the orifice were found to rise faster and higher.

a. Location A

The flow rates for all scenarios remained below the existing condition before the 20-minute mark, except for Scenario 1, which showed a 2.0% increase in flow rate at the 10-minute mark compared to the existing condition as illustrated in Figure 4.42a. Due to the peak flow discharge at the 10-minute mark, the 0.1m diameter orifice in Scenario 1 resulted in a higher flow rate than Scenarios 2 and 3 at Location A, indicating that it did not effectively reduce downstream flow discharge. At the peak flow (10-minute mark), Scenarios 2 and 3 demonstrated reductions of 55.6% and 51.1%, respectively, compared to the existing condition, with Scenario 2 showing the most substantial decrease in flow rate at Location A. Post the 20-minute mark, the flow rate for all scenarios surpassed the existing condition, indicating an overall increase in flow rate. Figure 4.43a illustrates that Scenarios 1, 2, and 3 experienced a 68.7% increase in water level compared to the existing condition. This resulted in flooding from 5 to 15-minute mark, as the water level reached 0.5m, due to the combination of the 0.1m orifice flow restrictor and the 0.4% drain design slope in the open channel.

The orifice size increased from 0.1m to 0.2m in diameter as shown in Figure 4.42b, the flow rates for all scenarios remained below the existing condition before the 15-minute mark and exceeded the existing condition after that. At the 10-minute peak flow rate, Scenario 1 showed a 2.2% increase in flow rate, while Scenarios 2 and 3 demonstrated reductions of 28.5% and 40.4%, respectively. Scenario 3 exhibited a significant reduction in flow rate compared to Scenario 2 with the increased orifice size. In Figure 4.43b, all

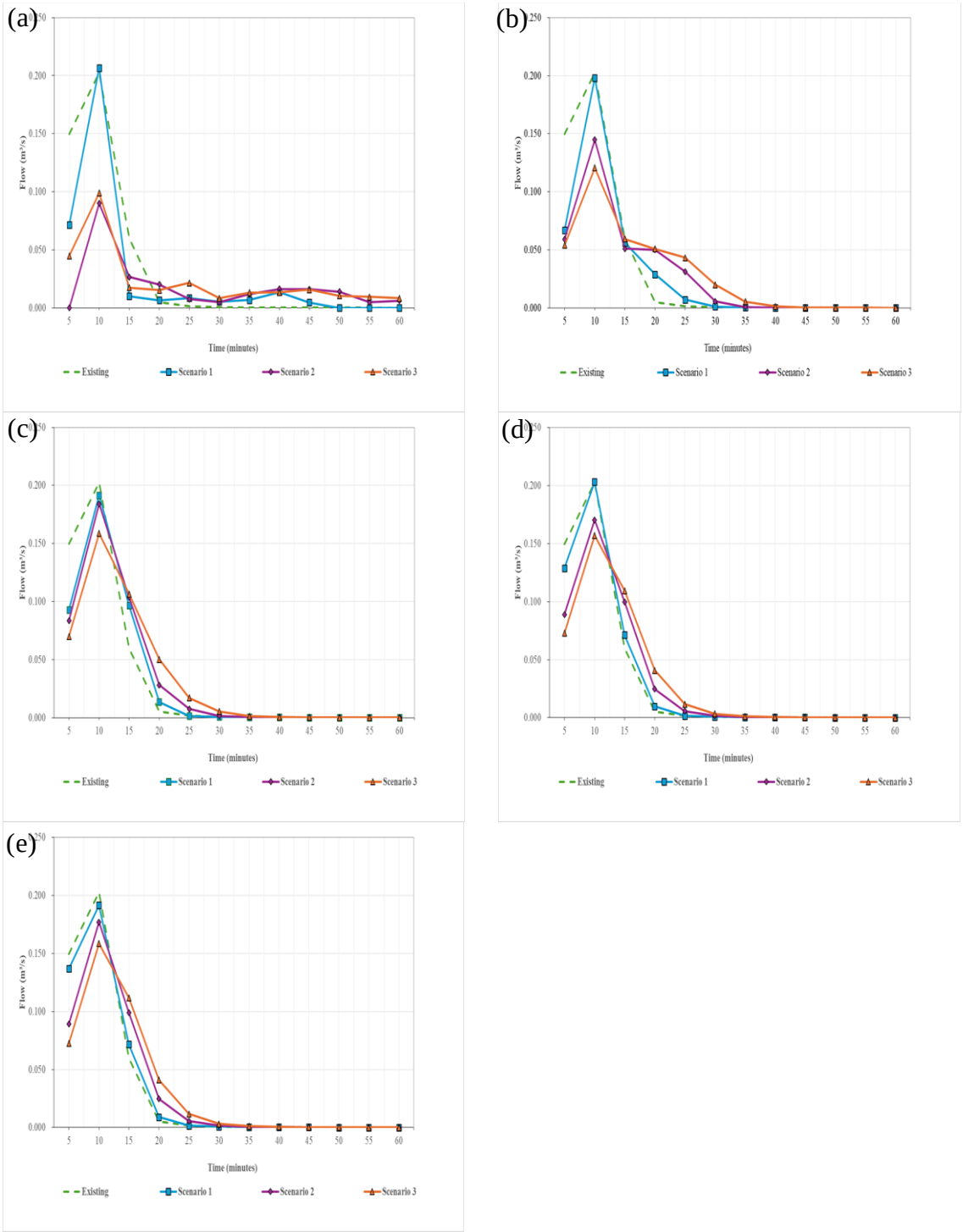


Figure 4.42: Flow Plots for Drain with 0.4% Slope at Location A Subjected to Design Rainfall, (a) 0.1m, (b) 0. 2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

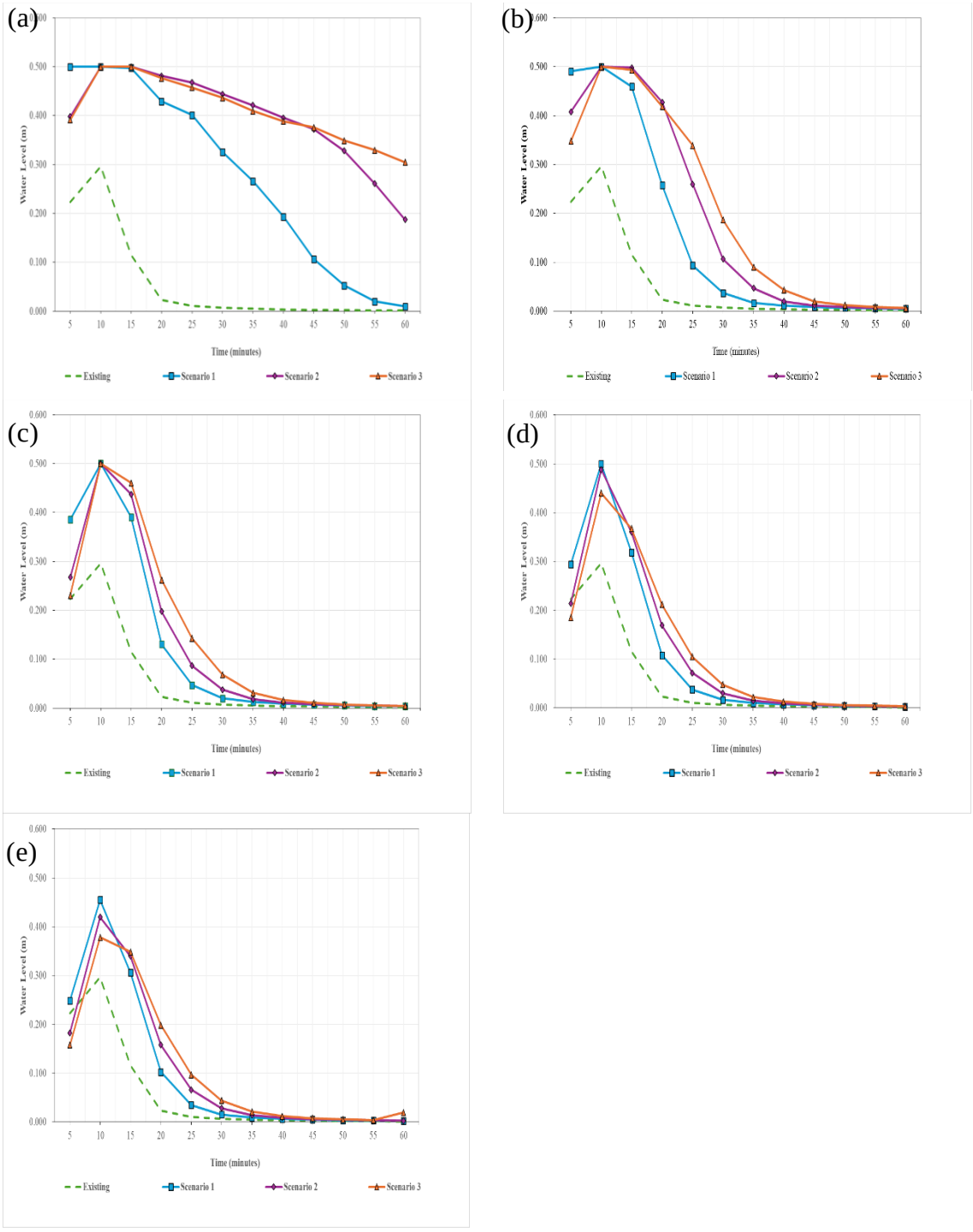


Figure 4.43: Water Level Plots for Drain with 0.4% Slope at Location A Subjected to Design Rainfall, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

scenarios with a 0.2m diameter orifice flow restrictor experienced a 68.7% increase in water level compared to the existing condition, resulting in flooding as the water level reached 0.5m at the 10-minute to 15-minute mark.

The existing condition and the implementation of a 0.3m diameter orifice in three scenarios as shown in Figure 4.42c. The flow rate before the 15-minute mark was below the existing condition, except for Scenario 1. After the 15-minute mark, the flow rate exceeded the existing condition. Compared to the existing condition, Scenario 1 showed a 5.5% increase, while Scenarios 2 and 3 demonstrated reductions of 9.1% and 21.7%, respectively. Scenario 3 exhibited the most significant reduction compared to the existing condition. In Figure 4.43c, all scenarios with a 0.3m diameter orifice flow restrictor experienced a 68.7% increase in water level compared to the existing condition, resulting in flooding as the water level reached 0.5m at the 10-minute mark.

Implementation of a 0.4m diameter orifice in three scenarios is shown in Figure 4.42d. The flow rate before the 15-minute mark remained below the existing condition, while it exceeded the existing condition after the 15-minute mark. Compared to the existing condition, Scenario 3 showed a significant reduction of 22.5%, and followed by Scenario 2 with 15.9%. Besides, Scenario 1 experienced a minor increment compared with the existing condition by 0.4%. In Figure 4.43d, Scenario 3 exhibited the smallest increment in water level compared to the existing condition, followed by Scenario 2 and Scenario 1, with increments of 48.5%, 64.8%, and 68.7%, respectively. Flooding occurred in the long open channel with the application of the 0.4m diameter flow restrictor for Scenario 1 and 2 at the 10-minute mark.

The effects of increasing the orifice size from 0.4m to 0.5m in diameter in Figure 4.42e, which allowed a greater volume of water to flow. More than 90% of the flow rates exceeded the existing condition due to the larger orifice size. At the peak flow (10-minute mark), the flow rates for Scenarios 1, 2, and 3 were reduced by 5.4%, 12.6%, and 21.7%, respectively. Scenario 3 demonstrated the most substantial reduction in flow rate compared to the other scenarios. In Figure 4.43e, Scenario 3 showed the smallest increment in water level compared to the existing condition, followed by Scenario 2 and Scenario 1, with increments of 27.6%, 41.6%, and 53.6%, respectively. No flooding occurred in the long open channel with the application of the 0.5m diameter flow restrictor.

b. Location B

The flow rates for all scenarios remained below the existing condition before the 20-minute mark which is present in Figure 4.44a. Due to the peak flow discharge at the 10-minute mark, the 0.1m diameter orifice in Scenario 1 resulted in a higher flow rate than Scenarios 2 and 3 at Location B, indicating that it did not effectively reduce downstream flow discharge. At the peak flow (10-minute mark), Scenarios 1, 2 and 3 demonstrated reductions of 2.8%, 63.7% and 32.5%, respectively, compared to the existing condition, with Scenario 2 showing the most substantial decrease in flow rate at Location B. Figure 4.45a illustrates that Scenarios 1, 2, and 3 experienced a 30.7%, 50.7%, and 136.2% respectively increase in water level compared to the existing condition. This resulted in flooding at Location B Scenario 3, as the water level reached 0.5m at the 10-minute mark. No flooding was found for other scenarios.

The orifice size increased by 0.2m in diameter as shown in Figure 4.44b, the flow rates for all scenarios remained below the existing condition before the 17.5-minute mark

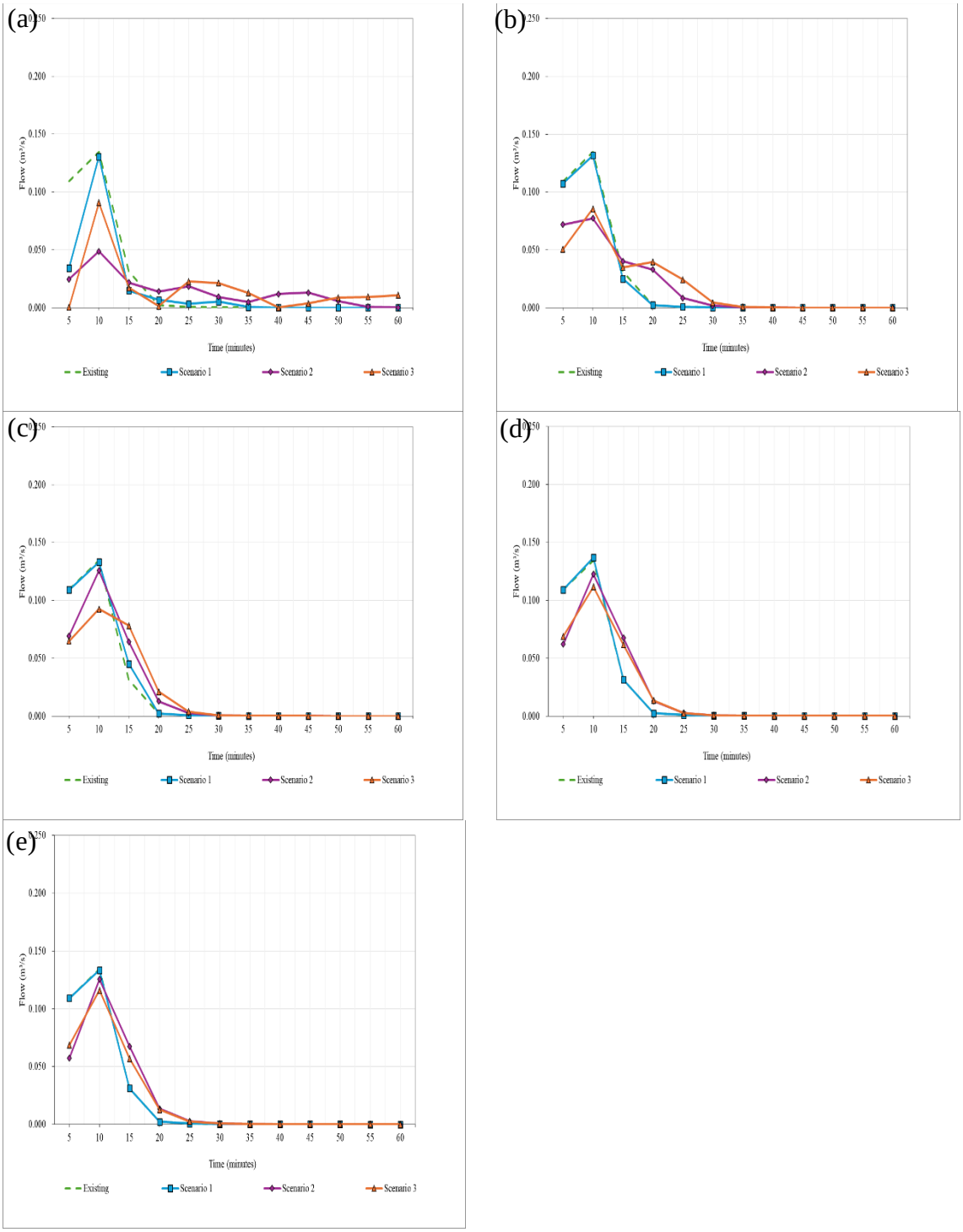


Figure 4.44: Flow Plots for Drain with 0.4% Slope at Location B Subjected to Design Rainfall, (a) 0.1m, (b) 0. 2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

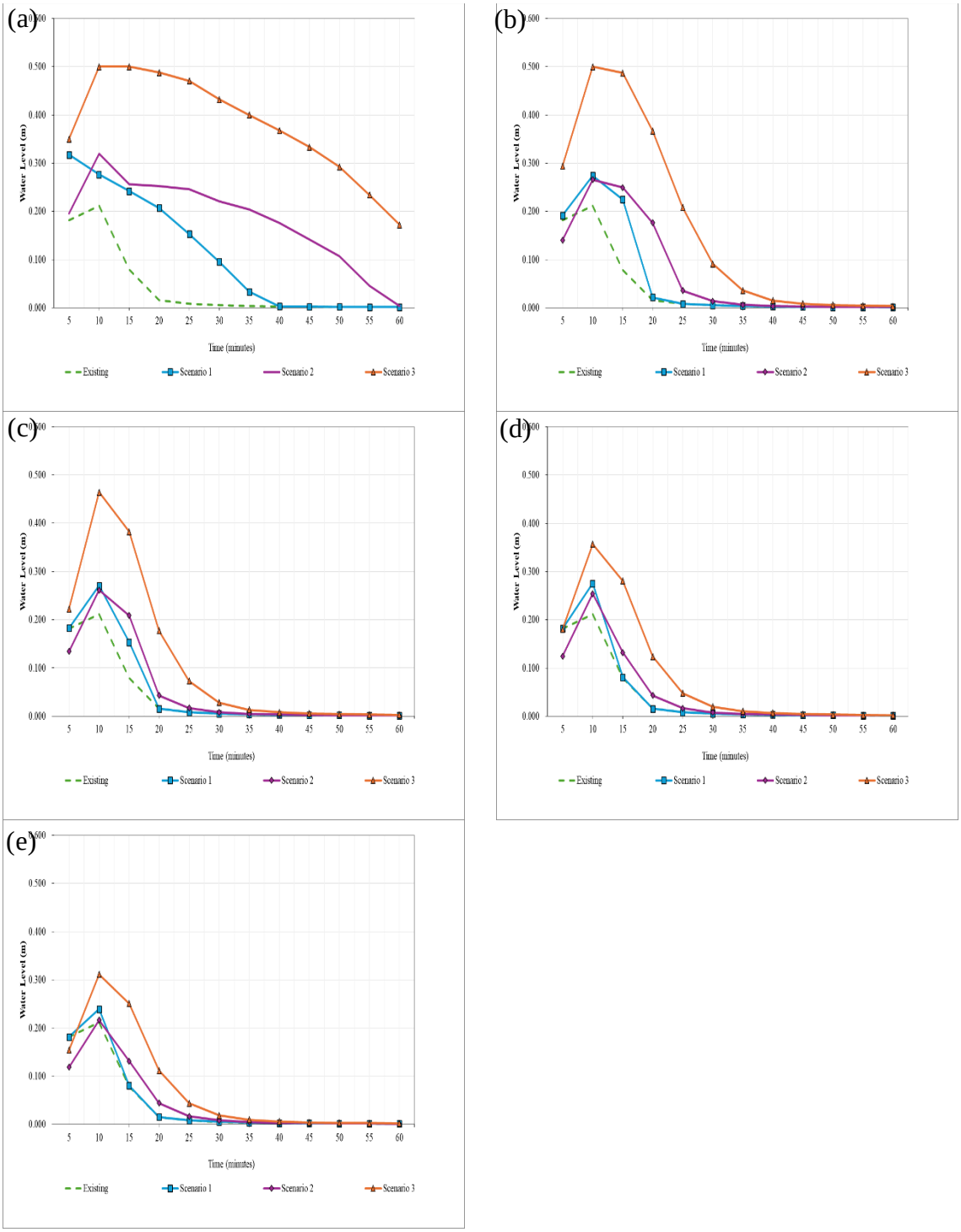


Figure 4.45: Water Level Plots for Drain with 0.4% Slope at Location B Subjected to Design Rainfall, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

and exceeded the existing condition after that. At the 10-minute peak flow rate, Scenarios 1, 2, and 3 demonstrated reductions of 2.1%, 42.5% and 36.3%, respectively. Scenario 2 exhibited a significant reduction in flow rate compared to Scenario 1 and Scenario 3 with the increased orifice size. In Figure 4.45b, Scenarios 1, Scenario 2, and Scenario 3 with a 0.2m diameter orifice flow restrictor experienced a 29.6%, 25.6%, and 136.2% respectively increase in water level compared to the existing condition. This resulted in flooding for Scenario 3 as the water level reached 0.5m in the interval of 10-minute to 15-minute mark. No flooding was found for other scenarios.

The existing condition and the implementation of a 0.3m diameter orifice in three scenarios as illustrated in Figure 4.44c. The flow rate before the 12.5-minute mark was below the existing condition, except for Scenario 1. After the 12.5-minute mark, the flow rate exceeded the existing condition. Compared to the existing condition, Scenario 3 showed a significant reduction of 31.2%, followed by Scenario 2 with 6.5% and Scenario 1 with 1.1%. In Figure 4.45c, Scenario 3 had the highest increment followed by Scenario 1 and Scenario 2 which were 119.5%, 27.9%, and 23.5% respectively. No flooding was found as the water level is below 0.5m.

The implementation of a 0.4m diameter orifice in three scenarios is shown in Figure 4.44d. The flow rate before the 12.5-minute mark remained below the existing condition, while it exceeded the existing condition after the 12.5-minute mark. Compared to the existing condition, Scenario 1 showed a 1.8% increase, while Scenarios 2 and 3 demonstrated reductions of 8.9% and 16.9%, respectively. In Figure 4.45d, Scenario 2 exhibited the lowest increment in water level compared to the existing condition, followed by Scenario 1 and Scenario 3, with increments of 19.9%, 30.3%, and 68.7%, respectively.

No flooding occurred in the long open channel with the application of the 0.4m diameter flow restrictor.

The effects of increasing the orifice size from 0.4m to 0.5m in diameter as shown in Figure 4.44e, which allowed a greater volume of water to flow. More than 90% of the flow rates exceeded the existing condition due to the larger orifice size. At the peak flow (10-minute mark), the flow rates for Scenarios 1, 2, and 3 were reduced by 0.8%, 6.7%, and 13.8%, respectively. Scenario 3 demonstrated the most substantial reduction in flow rate compared to the other scenarios. In Figure 4.45e, Scenario 2 showed the smallest increment in water level compared to the existing condition, followed by Scenario 1 and Scenario 3, with increments of 2.2%, 13.1%, and 47.2%, respectively. No flooding occurred in the long open channel with the application of the 0.5m diameter flow restrictor.

c. Location C

The flow rates for all scenarios remained below the existing condition prior to the 20-minute mark as shown in Figure 4.46a. Due to the peak flow discharge at the 10-minute mark, the 0.1m diameter orifice in Scenario 3 resulted in a higher reduction in flow rate followed by Scenarios 2 and Scenario 1 at Location C, indicating that it effectively reduces flow discharge, which is 73.0%, 40.9% and 1.0% respectively. Figure 4.47a illustrates that the water level for Scenario 2 had the highest increment followed by Scenario 3 and Scenario 1 which were 174.3%, 126.6%, and 12.5% respectively. A flood happened for Scenario 2 at the 3-hour mark of Location C.

The orifice size increased to 0.2m in diameter as shown in Figure 4.46b, the flow rates for Scenario 2 show the increment of the flow rate which was 29.7% whereas Scenario 1 and Scenario 3 show the reduction of the flow rate which were 1.1% and 24.1% by

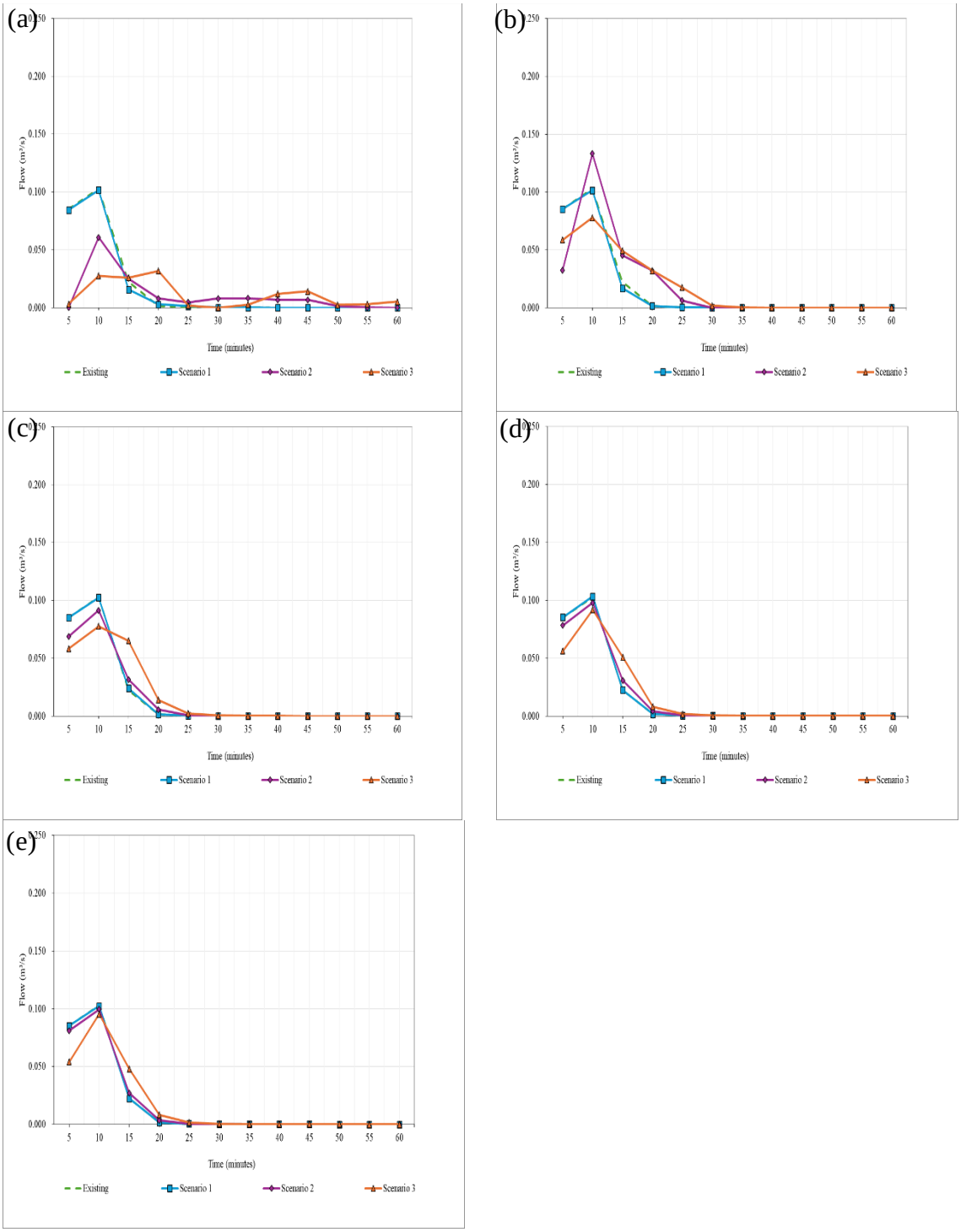


Figure 4.46: Flow Plots for Drain with 0.4% Slope at Location C Subjected to Design Rainfall, (a) 0.1m, (b) 0. 2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

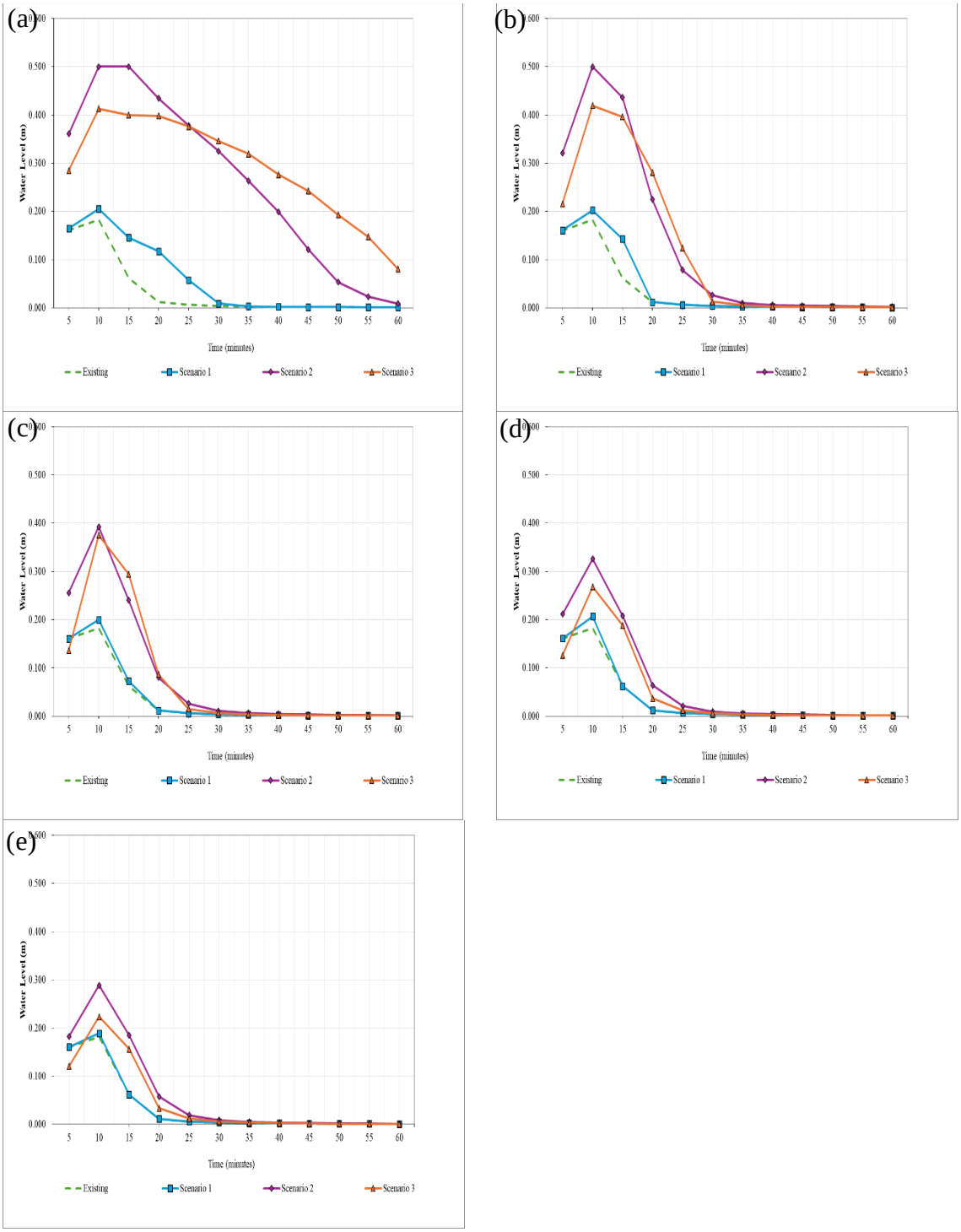


Figure 4.47: Water Level Plots for Drain with 0.4% Slope at Location C Subjected to Design Rainfall, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

comparing to an existing condition. Figure 4.47b, Scenario 2 has a higher increment of water level compared to Scenario 3 and Scenario 1 which are 174.3%, 130.2%, and 11.2% respectively. A flood happened for Scenario 2 at the 3-hour mark of Location C.

The existing condition and the implementation of a 0.3m diameter orifice in three scenarios is present in Figure 4.46c. The flow rate before the 15-minute mark was below the existing condition. After the 15-minute mark, the flow rate exceeded the existing condition. Compared to the existing condition, Scenario 1, Scenario 2, and Scenario 3 demonstrated reductions of 0.3%, 11.0%, and 24.3%, respectively. Scenario 3 exhibited the most significant reduction compared to the existing condition. In Figure 4.47c, the water level in Scenario 2 shows the highest water level, followed by Scenario 3 and Scenario 1 where the water level is increased by 115.4%, 105.9%, and 9.6% respectively by comparing to existing conditions.

The implementation of a 0.4m diameter orifice in three scenarios is shown in Figure 4.46d. The flow rate before the 15-minute mark remained below the existing condition except for Scenario 1 which increased by 0.6%, while it exceeded the existing condition after the 15-minute mark. Compared to the existing condition, Scenario 3 showed a significant reduction of 10.5%, followed by Scenario 2 with 4.7%. In Figure 4.47d, Scenario 1 exhibited the smallest increment in water level compared to the existing condition, followed by Scenario 3 and Scenario 2, with increments of 13.4%, 47.1%, and 78.7%, respectively. No flooding occurred in the long open channel with the application of the 0.4m diameter flow restrictor.

The effects of increasing the orifice size from 0.4m to 0.5m in diameter is shown Figure 4.46e, which allowed a greater volume of water to flow. At the peak flow (10-minute

mark), the flow rates for Scenarios 1, Scenario 2, and Scenario 3 were reduced by 0.3%, 3.2%, and 7.1%, respectively. Scenario 3 demonstrated the most substantial reduction in flow rate compared to the other scenarios. In Figure 4.47e, Scenario 1, Scenario 2, and Scenario 3 were increased by 3.7%, 58.4%, and 22.7% respectively when compared to the existing condition. No flooding occurred in the long open channel with the application of the 0.5m diameter flow restrictor.

d. Location D

The flow rates for all scenarios remained below the existing condition before the 20-minute mark compared to the existing condition as shown in Figure 4.48a. Due to the peak flow discharge at the 10-minute mark, the 0.1m diameter orifice in Scenario 2 resulted in a higher flow rate than Scenarios 1 and 3 at Location D, indicating that it did not effectively reduce upstream flow discharge. At the peak flow (10-minute mark), Scenarios 1, 2, and 3 demonstrated reductions of 33.3%, 85.1%, and 4.4%, respectively, compared to the existing condition. Post the 20-minute mark, the flow rate for all scenarios surpassed the existing condition, indicating an overall increase in flow rate. Figure 4.49a illustrates that Scenarios 1, 2, and 3 experienced a 0.7%, 178.3%, and 286.9% respectively increase in water level compared to the existing condition. Scenario 3 resulted in flooding, as the water level reached 0.5m at the 10-minute mark whereas Scenario 1 and Scenario 2 do not experience flooding throughout the rainfall.

The orifice size increased from 0.1m to 0.2m in diameter as shown in Figure 4.48b, the flow rates for all scenarios remained below the existing condition before the 20-minute mark and exceeded the existing condition after that. At the 10-minute peak flow rate, Scenarios 1, 2, and 3 showed a reduction of 33.3%, 21.7 and 58.0% in flow rate. Scenario 3

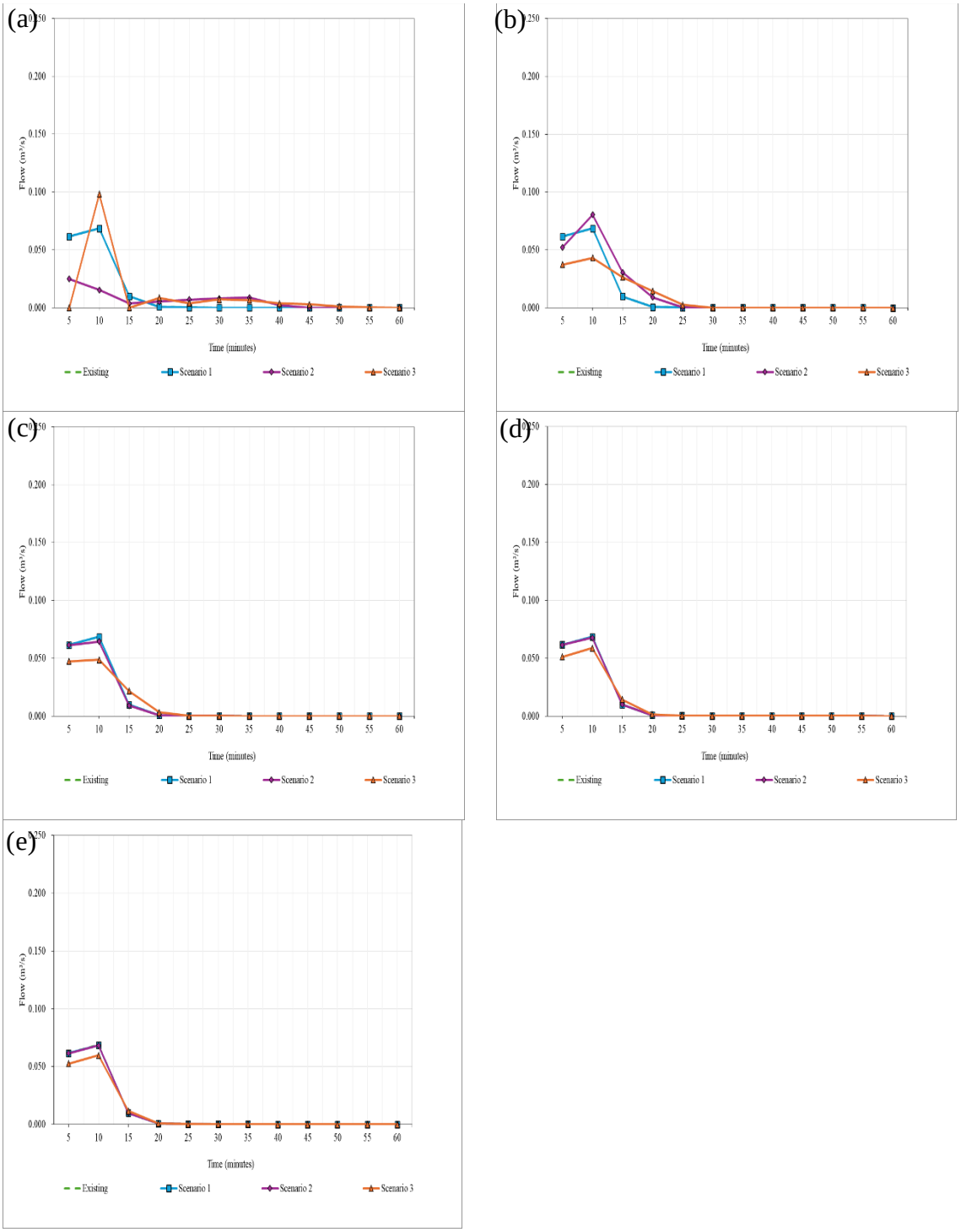


Figure 4.48: Flow Plots for Drain with 0.4% Slope at Location D Subjected to Design Rainfall, (a) 0.1m, (b) 0. 2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

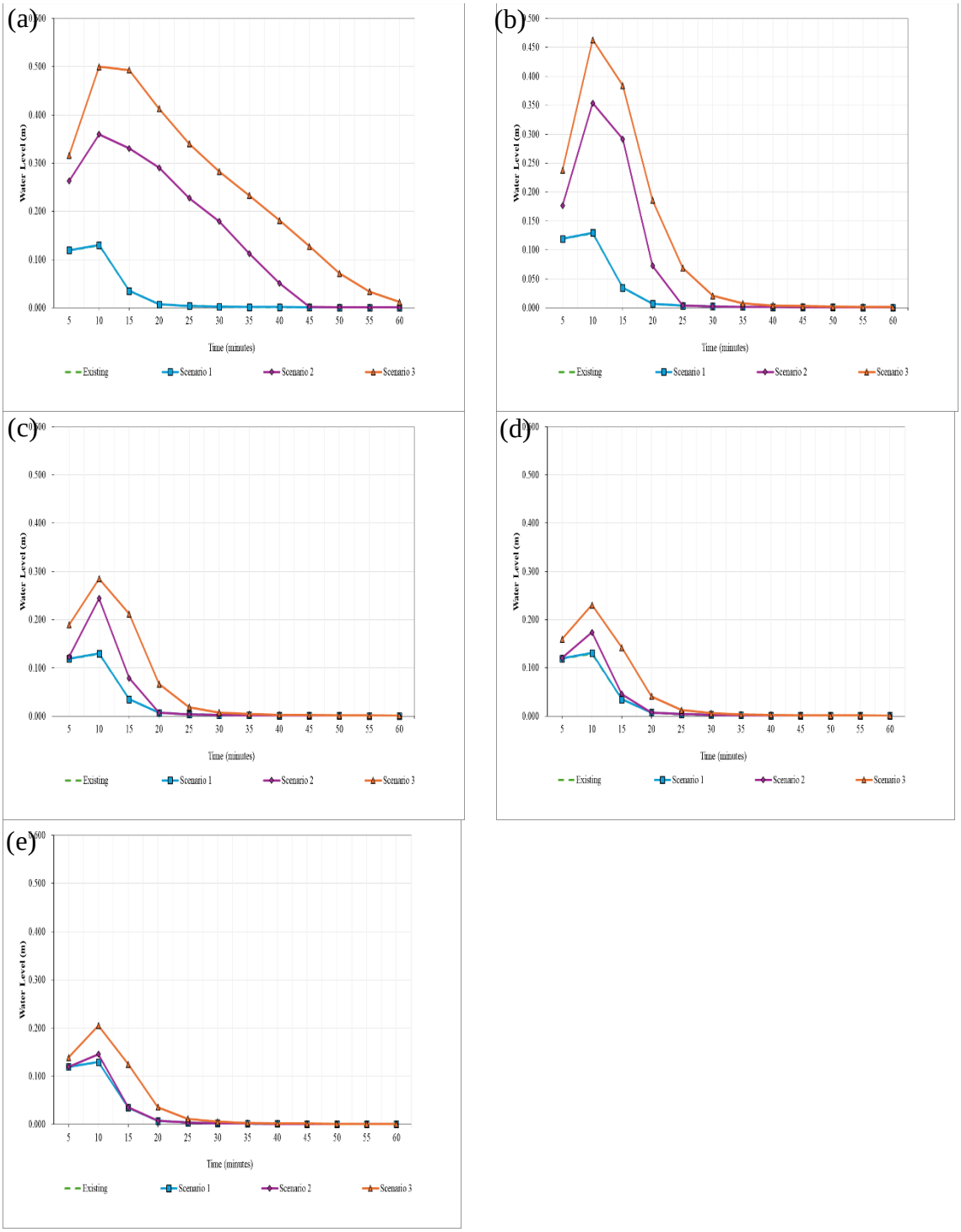


Figure 4.49: Water Level Plots for Drain with 0.4% Slope at Location D Subjected to Design Rainfall, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

exhibited a significant reduction in flow rate. In Figure 4.49b, all scenarios with a 0.2m diameter orifice flow restrictor experienced increasing where Scenario 3 had the highest increment followed by Scenario 2 and Scenario 1 which were 258.6%, 173.9%, and 0.6% respectively. No flood was found throughout the rainfall event.

The existing condition and the implementation of a 0.3m diameter orifice in three scenarios as shown in Figure 4.48c. The flow rate before the 15-minute mark was below the existing condition, except for Scenario 1. After the 15-minute mark, the flow rate exceeded the existing condition. Compared to the existing condition, Scenario 1, Scenarios 2 and 3 demonstrated reductions of 33.3%, 37.3% and 52.7%, respectively. Scenario 3 exhibited the most significant reduction compared to the existing condition. In Figure 4.49c, Scenario 3 had the highest increase in water level followed by Scenario 2 and Scenario 1 which were 120.7%, 88.8%, and 0.7% respectively. No flood was found throughout the rainfall event.

The implementation of a 0.4m diameter orifice in three scenarios is shown in Figure 4.48d. The flow rate before the 15-minute mark remained below the existing condition, while it exceeded the existing condition after the 15-minute mark. Compared to the existing condition, Scenario 3 showed a significant reduction of 42.8%, followed by Scenario 2 with 34.1% and Scenario 1 with 33.3%. In Figure 4.49d, Scenario 3 exhibited the highest increment in water level compared to the existing condition, followed by Scenario 2 and Scenario 1, with increments of 78.5%, 34.2%, and 1.2%, respectively. No flooding occurred in the long open channel with the application of the 0.4m diameter flow restrictor.

The effects of increasing the orifice size from 0.4m to 0.5m in diameter is shown in Figure 4.48e, which allowed a greater volume of water to flow. At the peak flow (10-minute mark), the flow rates for Scenarios 1, 2, and 3 were reduced by 33.3%, 33.5%, and 41.9%,

respectively. Scenario 3 demonstrated the most substantial reduction in flow rate compared to the other scenarios. In Figure 4.49e, Scenario 3 showed the highest increment in water level compared to the existing condition, followed by Scenario 2 and Scenario 1, with increments of 58.9%, 12.9%, and 0.2%, respectively. No flooding occurred in the long open channel with the application of the 0.5m diameter flow restrictor.

e. Discussion

Figures 4.50 and 4.51 summarize the peak flow rate and water level for a design slope of 0.4% under a 5-minute, 10-year ARI design rainfall.

At Location A, the peak flow rate decreased for all scenarios and orifice sizes, except for Scenario 1 with orifice diameters of 0.1m and 0.4m, which showed an increase. This indicates that the reduction in flow rate decreases the rainfall discharge to the outfall, thereby reducing congestion at the downstream end. The highest reduction in flow rate occurred in Scenario 2 with a 0.1m orifice diameter, while the lowest reduction was in Scenario 1 with a 0.5m orifice diameter. However, the peak water level increased for all scenarios and orifice sizes. Scenario 1 (0.1m, 0.2m, 0.3m, and 0.4m diameter of orifice), Scenario 2 (0.1, 0.2, and 0.3m diameter of orifice), and Scenario 3 (0.1m, 0.2m, and 0.3m diameter of orifice) had reached the maximum water level of 0.5m, causing potential flooding. Thus, these scenarios failed to meet the objective.

At Location B, the peak flow rate decreased for all scenarios and orifice sizes, except for Scenario 1 with orifice diameters of 0.4m which showed an increase. The increased flow rate compared to the existing flow rate would lead to more rainfall water flowing to Location A for discharge. Thus, reducing the flow rate is a key objective in this study. The graph shows that Scenario 2 with a 0.1m orifice diameter had the highest reduction, whereas

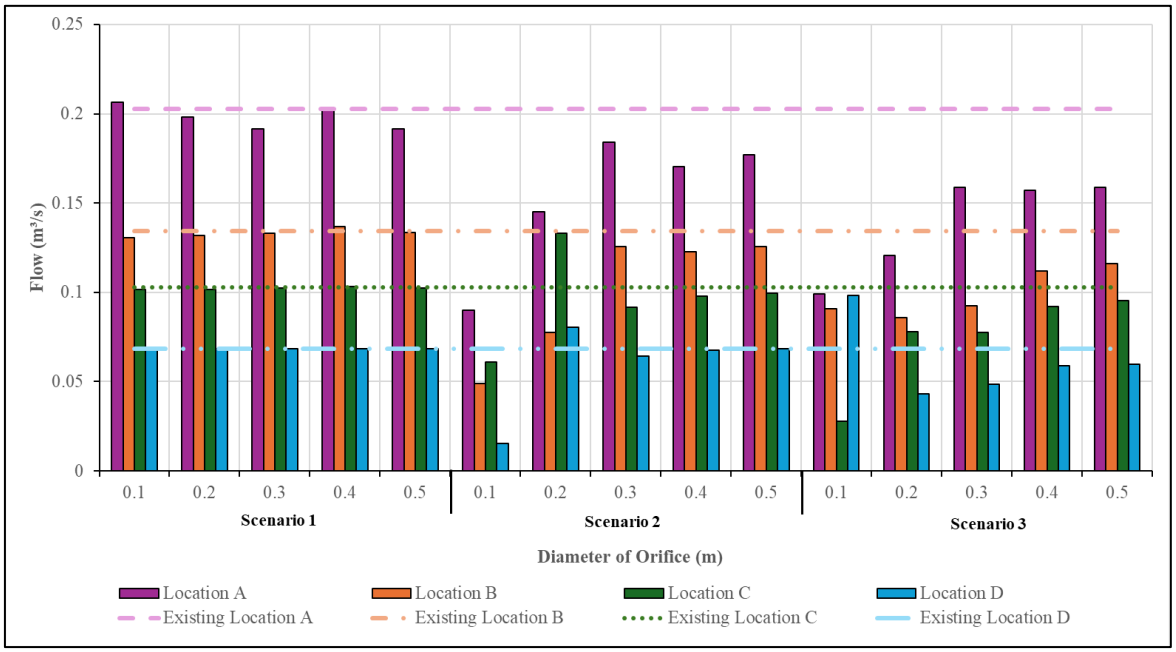


Figure 4.50: Summary of Peak Flow Rate for 5-minute 10-year ARI Design Rainfall at 0.4% Design Slope of Open Channel

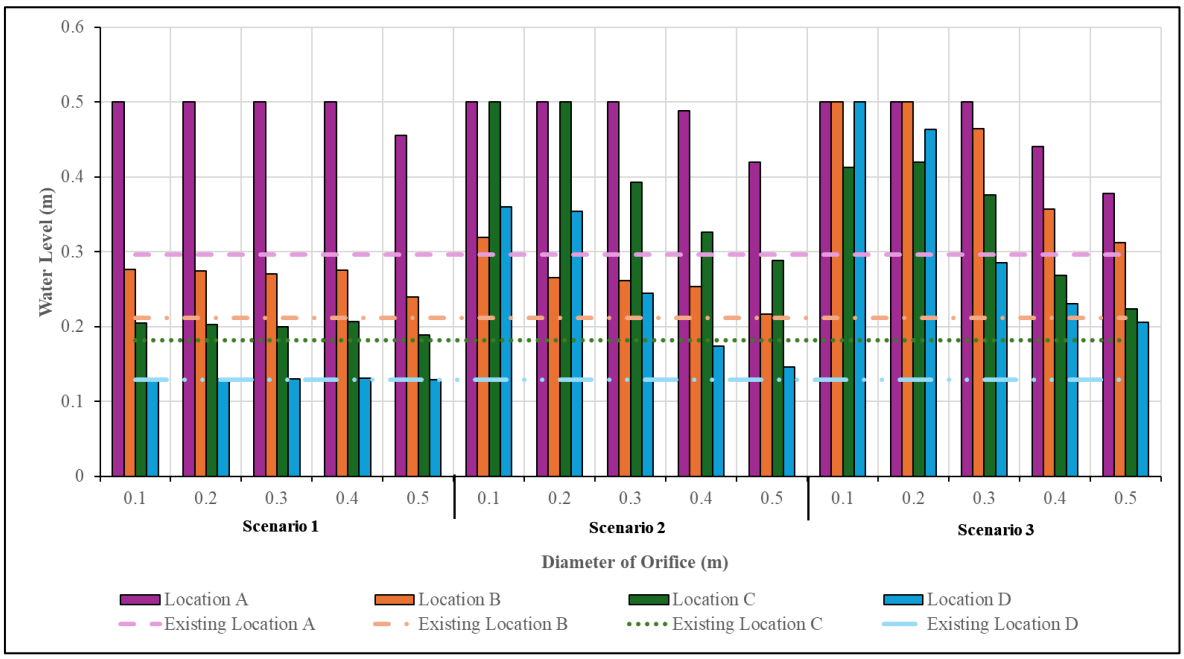


Figure 4.51: Summary of Peak Water Level for 5-minute 10-year ARI Design Rainfall at 0.4% Design Slope of Open Channel

Scenario 1 with a 0.5m orifice diameter showed the least reduction. For peak water levels at Location B, all scenarios showed an increase compared to the existing water level. Flooding occurred with orifice diameters of 0.1m and 0.2m in Scenario 3, where water levels increased to the maximum drain water level of 0.5m. Hence, these analyses for Scenario 3 failed as they caused flooding.

At Location C, the peak flow rate decreased for all scenarios and orifice sizes, except for Scenario 2 with a 0.2m orifice diameter, which showed an increase. The highest reduction in flow rate occurred with Scenario 3's 0.1m orifice diameter, while the lowest reduction was with all the orifice in Scenario 1. Greater reductions in flow rate result in slower rainfall discharge to Locations A and B. Peak water levels increased across all scenarios compared to existing conditions. However, Scenario 2 with orifice diameters of 0.1m and 0.2m reached a water level of 0.5m, causing flooding and thus failing to meet the objectives.

At Location D (upstream end), The peak flow rate decreased for all scenarios and orifice sizes, except for Scenario 2 with a 0.2m orifice diameter and Scenario 3 with a 0.1m orifice diameter, which showed an increase. However, the flow rate for Scenario 1 shows remains unchanged although changing the orifice size. The highest reduction in flow rate occurred with Scenario 2's 0.1m orifice diameter, while the lowest reduction was with Scenario 2 with 0.5m diameter of orifice. Peak water levels increased across all scenarios compared to existing conditions. However, Scenario 3 with orifice diameters of 0.1m reached a water level of 0.5m, causing flooding and thus failing to meet the objectives.

In summary, the evaluation under the 0.4% slope and 5-minute, 10-year ARI design rainfall revealed that peak flow rate reductions were generally achieved across most scenarios, particularly in Scenario 2 with a 0.1m orifice diameter, which consistently showed

the highest reduction at multiple locations. However, certain conditions, such as Scenario 1 with 0.1m and 0.4m diameter orifices at Location A and Scenario 2 with a 0.2m diameter orifice at Locations C and D, resulted in increased flow rates, indicating inefficiencies in controlling upstream discharge. This reinforces the need for careful orifice selection tailored to each specific location. While smaller orifices were more effective in reducing flow rates, they frequently led to water levels reaching the critical 0.5m mark, as observed in multiple scenarios, particularly in Scenarios 1 and 3 at Locations A, B, and D, thereby increasing the risk of flooding and failing to meet the objective. In contrast, larger orifice sizes such as 0.4m and 0.5m tended to maintain water levels below the flooding threshold, albeit with slightly less impact on flow reduction. The results demonstrate that a balance must be struck between effective flow rate control and maintaining safe water levels, and that Scenario 2 offers the best trade-off in several cases when paired with moderate orifice sizes. These findings emphasize the importance of designing context-specific flow restrictors that ensure both hydraulic efficiency and flood mitigation in stormwater systems with moderate slopes.

4.2.3.2 Historical Rainfall

The hydrographs presented in Figures 4.52, Figure 4.54, Figure 4.56, and Figure 4.58 illustrate the simulated flow rates, while Figures 4.53, Figure 4.55, Figure 4.57, and Figure 4.59 depict the corresponding water levels for Historical Rainfall Event 1. This event is selected for detailed presentation and discussion in this chapter. Meanwhile, the hydrographs for Event 2 and Event 3 are included in Appendix I for reference, as their flow rate and water level patterns were found to be largely consistent with those observed in Event 1. These were meant for 0.4% slope. Different drain slope had different characteristics that were described in Section 4.3.

In terms of flow rate, most changes were observed in sub-figures (a) and (b); and little changes in sub-figures (c) to (e). In terms of water level, most changes were observed only in sub-figure (a) throughout. Water levels were the highest in sub-figures (a).

a. Location A

Flow rates for all scenarios generally remained below the existing condition as shown in Figure 4.52a. At the 13-hour peak flow discharge, the 0.1m diameter orifice in Scenario 1 resulted in a higher flow rate than in Scenarios 2 and 3 at Location A, indicating it did not effectively reduce downstream flow discharge. At the peak flow (13-hour mark), Scenarios 2, and 3 showed reductions of 9.1%, and 30.2%, respectively, compared to the existing condition, with Scenario 3 showing the most substantial reduction. Scenario 1 showed an increment of 38.4% compared to the existing condition where the flow discharge to outfall had been increasing. Figure 4.53a illustrates that Scenarios 1, 2, and 3 experienced a 605% increase in water level compared to the existing condition, resulting in flooding as the water level reached 0.5m after the 12-hour mark, due to the combination of the 0.1m orifice flow restrictor and the 0.4% drain design slope in the open channel.

The orifice size increased from 0.1m to 0.2m in diameter as shown in Figure 4.52b, the flow rates for all scenarios generally remained below the existing condition throughout the rainfall event. At the 13-hour peak flow rate, Scenarios 1, and 2 showed reductions of 19.1 and 39.0%, respectively. However, Scenario 3 shows a minor increment of 6.8% compared to an existing condition. Figure 4.53b shows that Scenario 1, Scenario 2, and Scenario 3 with a 0.2m diameter orifice flow restrictor experienced a 233.5%, 222.6%, and 183.4% respectively increase in water level compared to the existing condition. However, no flooding occurred at Location A as the water level remained below 0.5m.

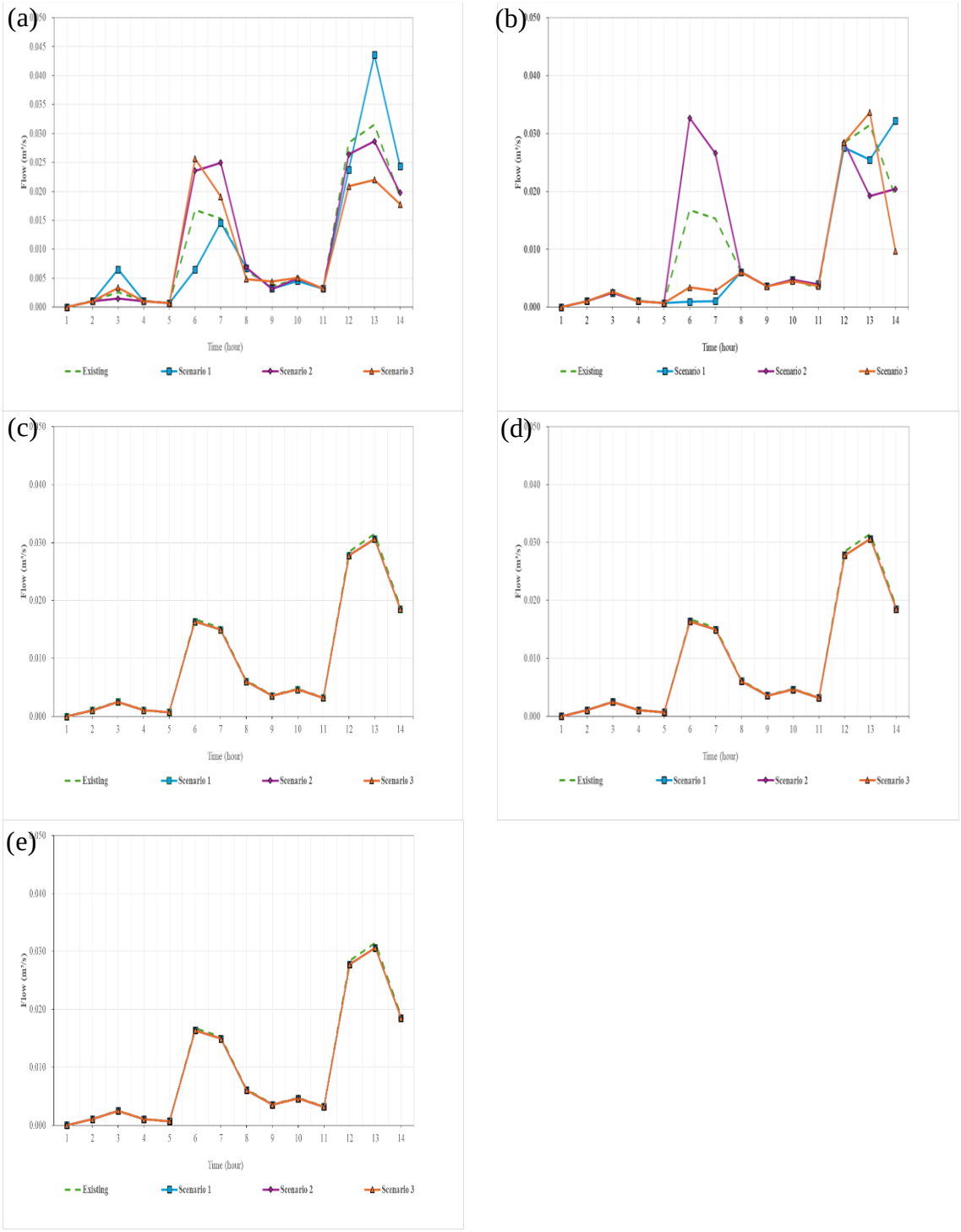


Figure 4.52: Flow Plots for Drain with 0.4% Slope at Location A Subjected to Event 1, (a) 0.1m, (b) 0. 2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

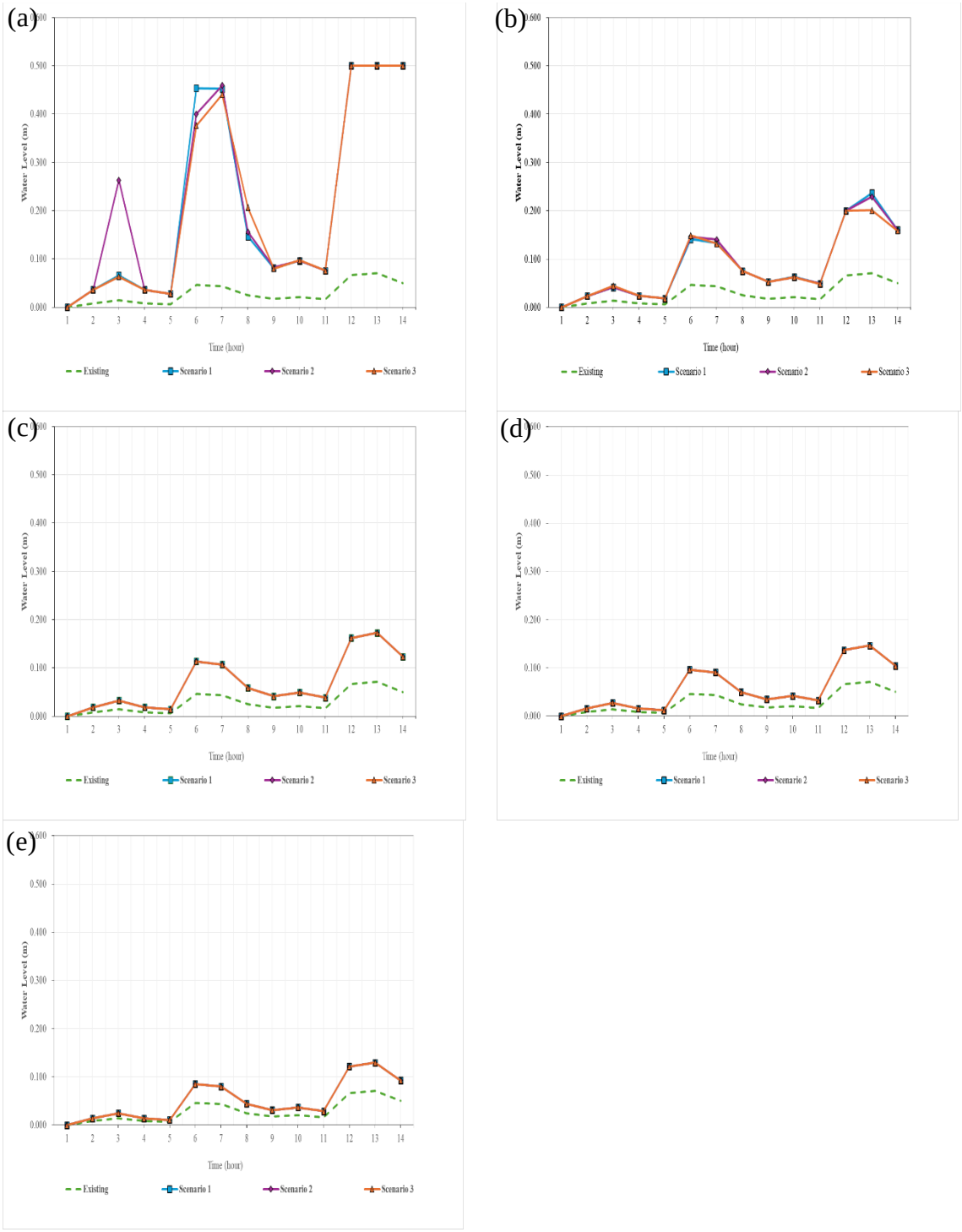


Figure 4.53: Water Level Plots for Drain with 0.4% Slope at Location A Subjected to Event 1, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

The existing condition and the implementation of a 0.3m diameter orifice in three scenarios as shown in Figure 4.52c. The flow rate for Scenario 1 and Scenario 3 was below the existing condition, showing a nearly uniform reduction of 2.7%, whereas Scenario 2 reduced by 2.8%. Figure 4.53c indicates that all scenarios with a 0.3m diameter orifice flow restrictor experienced a 143.6% increase in water level compared to the existing condition. No flooding occurred with the application of a 0.3m diameter orifice in this simulation.

The orifice size increased to 0.4m in diameter, the flow rates for all scenarios generally remained below the existing condition throughout the rainfall event as shown in Figure 4.52d. At the 13-hour peak flow rate, Scenarios 1, 2, and 3 showed reductions of 2.8%, respectively. Figure 4.53d demonstrates that all scenarios with a 0.4m diameter orifice flow restrictor experienced a 106.6% increase in water level compared to the existing condition. However, no flooding occurred at Location A as the water level remained below 0.5m.

The orifice size increased to 0.5m in diameter, the flow rates for all scenarios generally remained below the existing condition throughout the rainfall event as illustrated in Figure 4.52e. At the 13-hour peak flow rate, Scenarios 1, 2, and 3 demonstrated reductions of 2.8%, respectively. Figure 4.53e shows that all scenarios with a 0.5m diameter orifice flow restrictor experienced an 83.1% increase in water level compared to the existing condition. However, no flooding occurred at Location A as the water level remained below 0.5m.

b. Location B

The flow rates for all scenarios generally remained below the existing condition shown in Figure 4.54a. At the 13-hour peak flow discharge, the 0.1m diameter orifice in

Scenario 1 resulted in a higher reduction in flow rate compared to Scenarios 2 and 3 at Location B, indicating it did not effectively reduce flow discharge. At the peak flow (13-hour mark), Scenarios 1, 2, and 3 showed reductions of 69.9%, 11.7%, and 6.0%, respectively, compared to the existing condition. Figure 4.55a illustrates that Scenarios 1, 2, and 3 experienced increasing in water level compared to the existing condition where 349.8%, 360.3%, and 784.6%. This resulted in flooding after the 12-hour mark as the water level reached 0.5m at Scenario 3 whereas no flooding was found for other scenarios.

The orifice size increased to 0.2m in diameter as shown in Figure 4.54b, the flow rates for all scenarios generally remained below the existing condition throughout the rainfall event. At the 13-hour peak flow rate, Scenario 1 at Location B remained at the same flow rate as the existing condition. However, Scenario 2 had a minor increment of 0.7% whereas Scenario 3 had a minor reduction of 0.3%. Figure 4.55b shows that Scenario 3 had the highest increment followed by Scenario 2 and Scenario 1 which were 208.7%, 0.8%, and 0.4% respectively. However, no flooding occurred at Location B for all the scenarios as the water level remained below 0.5m throughout the rainfall event.

The existing condition and the implementation of a 0.3m diameter orifice in three scenarios as shown in Figure 4.54c. The flow rate at Location B for Scenario 1 has a similar flow rate as the existing condition. However, Scenario 2 had a minor reduction of 0.2% whereas Scenario 3 had a minor increment of 6.1%. Figure 4.55c indicates that Scenario 1 at Location B only had a minor increment in water level which was 0.1% and Scenario 2 has a similar value to the existing condition. However, Scenario 3 had the highest increment in water level which was increased by 128.5%. No flooding occurred with the application of a 0.3m diameter orifice in this simulation.

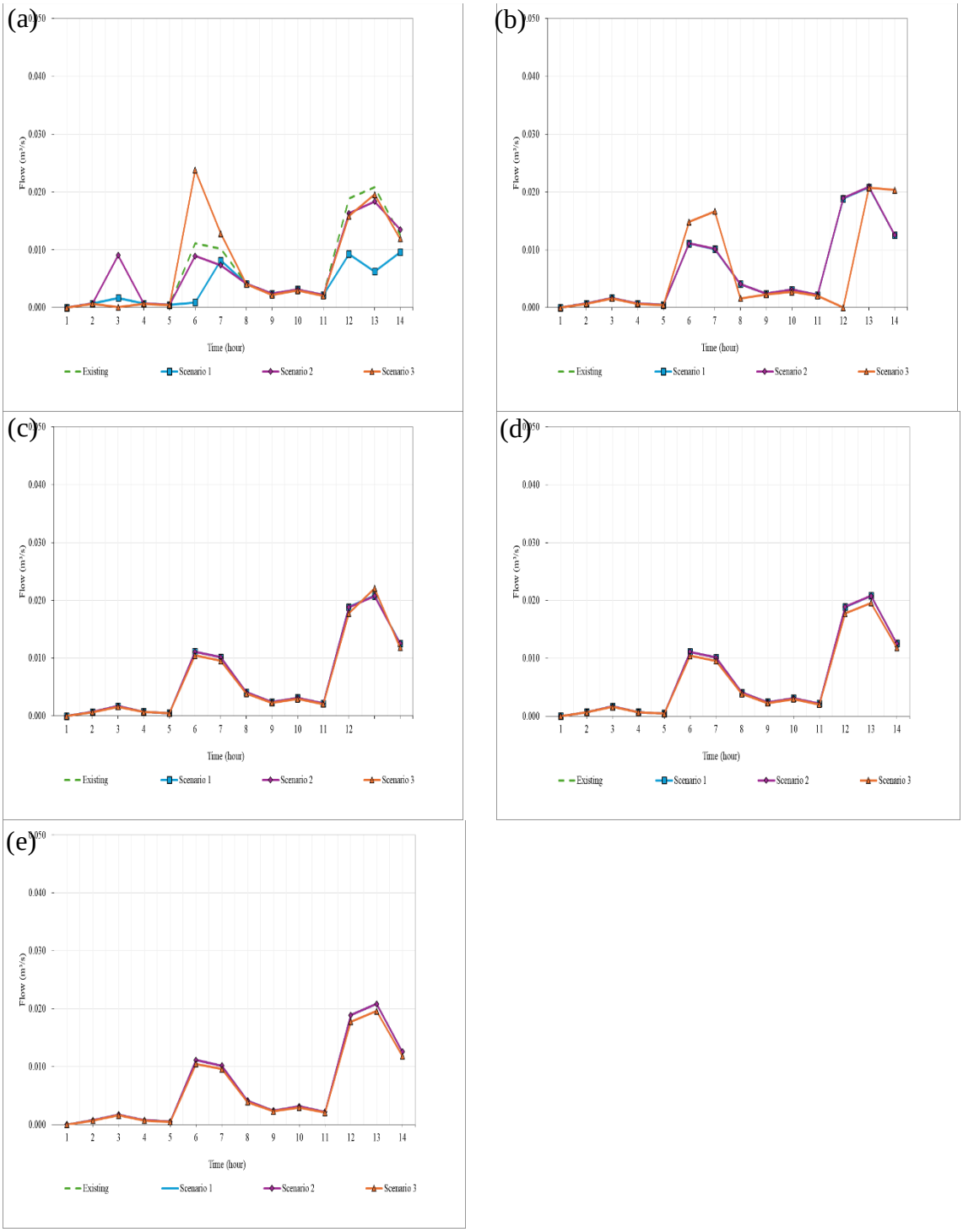


Figure 4.54: Flow Plots for Drain with 0.4% Slope at Location B Subjected to Event 1, (a) 0.1m, (b) 0. 2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

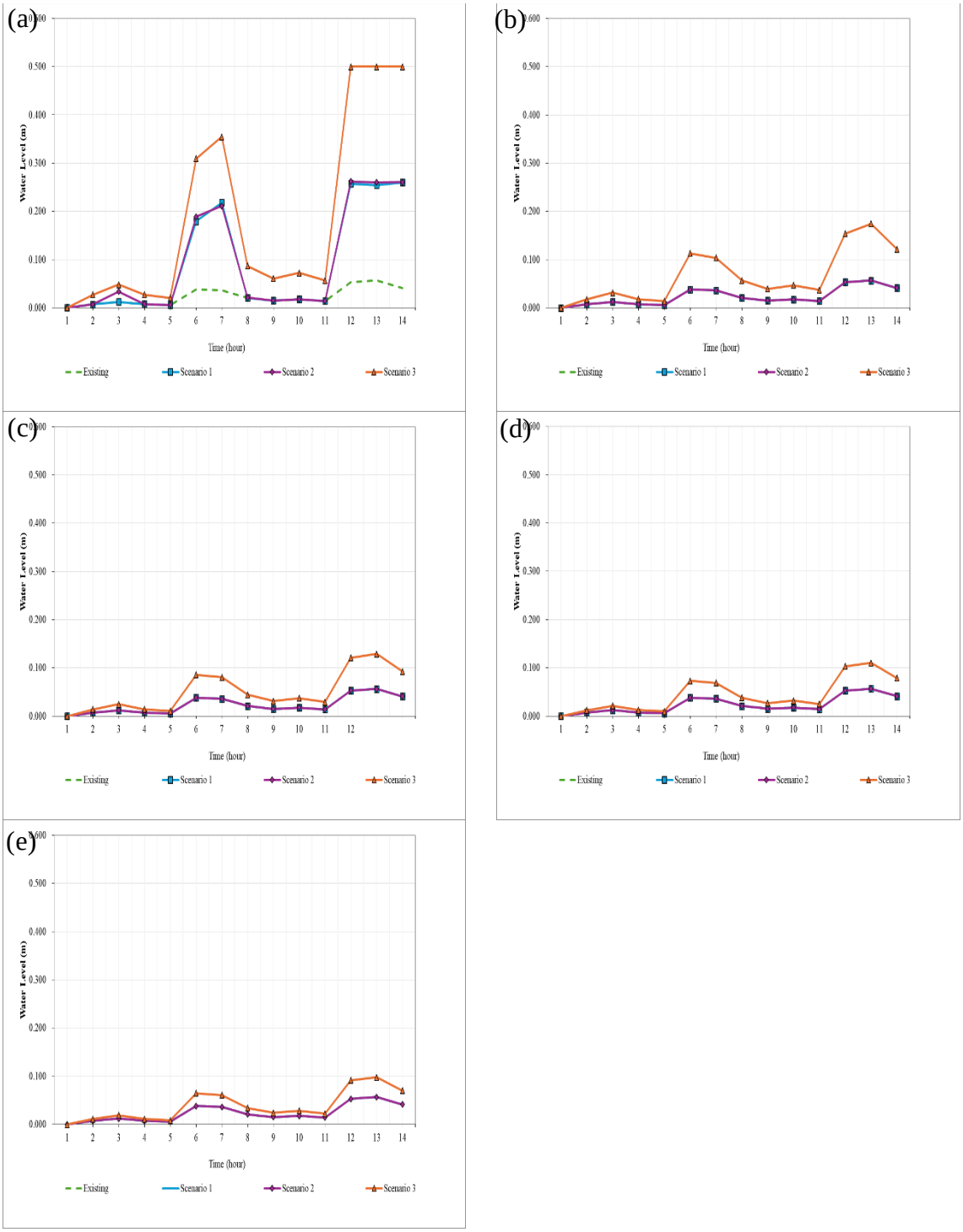


Figure 4.55: Water Level Plots for Drain with 0.4% Slope at Location B Subjected to Event 1, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

The orifice size increased to 0.4m in diameter as shown in Figure 4.54d, the flow rate of Scenario 1 had a similar reading with existing conditions where Scenario 2 and Scenario 3 had a slight reduction by 0.1% and 5.9% respectively by compared to an existing condition. Figure 4.55d demonstrates that Scenario 1 had a similar reading with existing conditions where Scenario 2 had a minor reduction of 0.1% and Scenario 3 had a significant increment of 94.7%. No flooding occurred at Location B as the water level remained below 0.5m.

The orifice size increased to 0.5m in diameter, there were no increments or reductions for Scenario 1 and Scenario 2 for the flow rate at Location B as illustrated in Figure 4.54e. However, Scenario 3 shows a minor reduction which was 5.9%. Figure 4.55e shows Scenario 1 and Scenario 2 do not show the change in water level whereas Scenario 3 had a significant increment of 73.1% compared to existing conditions. However, no flooding occurred at Location A as the water level remained below 0.5m.

c. Location C

Flow rates for all scenarios generally remained below the existing condition as shown in Figure 4.56a. At the peak flow (13-hour mark), Scenario 1 had an increase in flow rate which is 128.4% while Scenario 2 and Scenario 3 had a reduction of flow rate compared with an existing condition which was 3.7% and 0.3% respectively. Figure 4.57a illustrates that Scenarios 1, Scenario 2, and Scenario 3 had increased the water level of 312.5%, 923.3%, and 739.2% respectively. Scenario 1 had the lowest increment compared to Scenario 2 and Scenario 3. However, the flood for Scenario 2 happened after the 13-hour mark. No flood was found for another scenario.

The orifice size increased to 0.2m in diameter as shown in Figure 4.56b, the flow



Figure 4.56: Flow Plots for Drain with 0.4% Slope at Location C Subjected to Event 1, (a) 0.1m, (b) 0. 2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

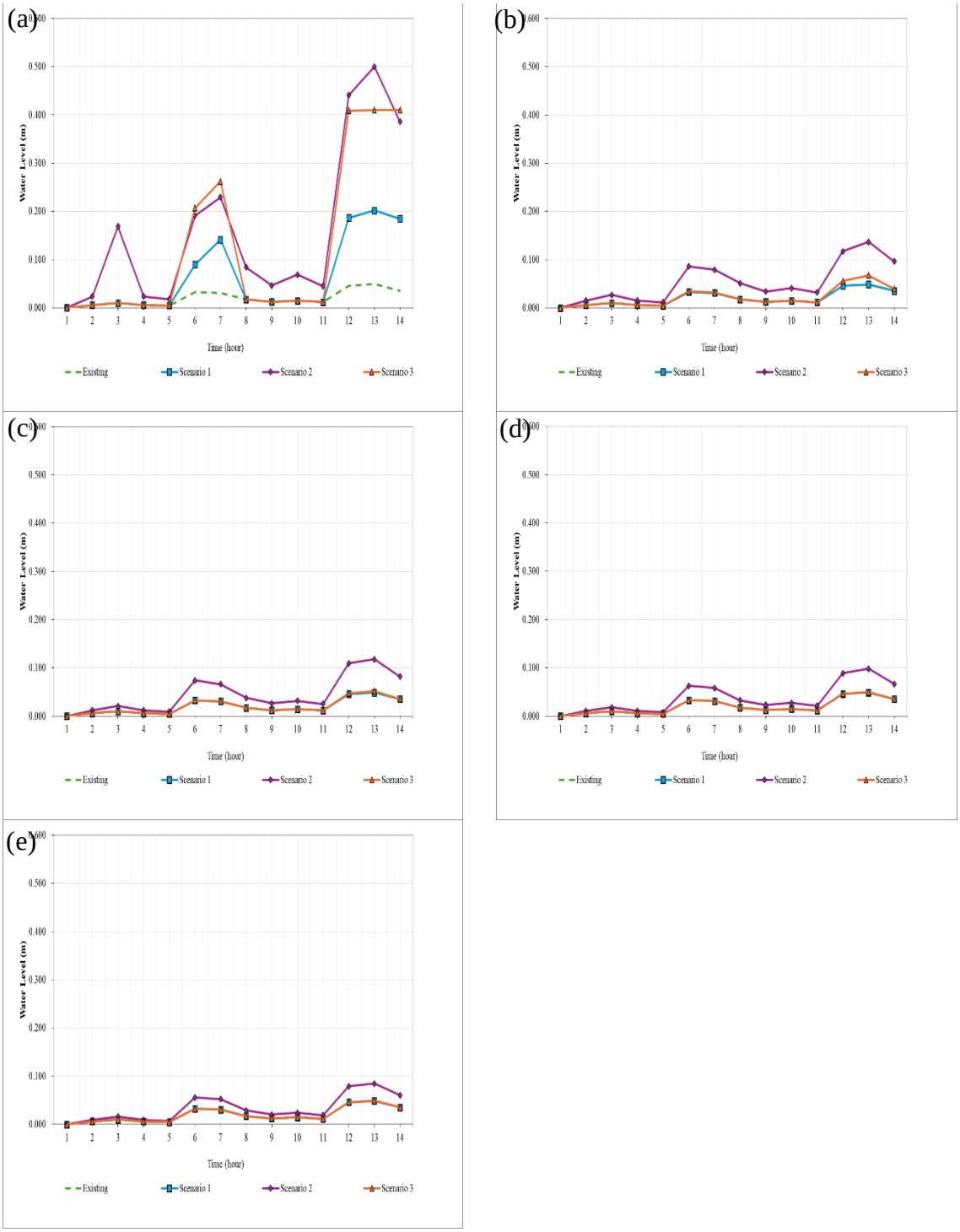


Figure 4.57: Water Level Plots for Drain with 0.4% Slope at Location C Subjected to Event 1, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

rates for all scenarios are generally approximately to the existing condition throughout the rainfall event at Location C. At the 13-hour peak flow rate, Scenario 1 had a similar flow rate with the existing condition whereas Scenario 2 shows an increment of 297.7% and Scenario 3 shows a reduction of 32.6%. Figure 4.57b shows the water level where Scenario 1 remained unchanged whereas Scenario 3 had the lowest increment followed by Scenario 2 which increased by 38.0%, and 179.6% respectively. No flooding occurred at Location C as the water level remained below 0.5m.

The existing condition and the implementation of a 0.3m diameter orifice in three scenarios as shown in Figure 4.56c. The flow rate for Scenario 1 remains unchanged to the existing condition. However, Scenarios 2 and 3 show a minor reduction which was 52.9% and 0.1% respectively by comparing to existing conditions. Figure 4.57c indicates that Scenario 1 remained unchanged in water level whereas Scenario 3 had the lowest increment followed by Scenario 2 which increased by 6.9%, and 141.2% respectively. No flooding occurred at Location C as the water level remained below 0.5m.

The orifice size increased to 0.4m in diameter as illustrated in Figure 4.56d, the flow rates for all scenarios generally remained below the existing condition throughout the rainfall event. At the 13-hour peak flow rate, Scenario 1 remains unchanged compared to existing conditions. However, Scenario 2 shows a slight reduction of 9.1%, whereas Scenario 3 had a minor increase of 0.1% compared to an existing condition. Figure 4.57d demonstrates that the water level where Scenario 1 remained unchanged whereas Scenario 3 had the lowest increment followed by Scenario 2 which increased by 1.8%, and 100.5% respectively. No flooding occurred at Location C as the water level remained below 0.5m.

The orifice size increased to 0.5m in diameter as shown in Figure 4.56e, At the 13-hour peak flow rate, Scenario 1 remains unchanged compared to an existing condition. However, Scenario 2 shows a slight reduction of 1.0% whereas Scenario 3 had a minor increase of 0.1% by compared to an existing condition. Figure 4.57e shows that the water level where Scenario 1 remained unchanged whereas Scenario 3 had the lowest increment followed by Scenario 2 which increased by 1.0%, and 72.9% respectively. No flooding occurred at Location C as the water level remained below 0.5m.

d. Location D

Flow rates for all scenarios generally remained below the existing condition as shown in Figure 4.58a. At the 13-hour peak flow discharge, the 0.1m diameter orifice in Scenario 1 resulted in a higher flow rate than in Scenarios 2 and 3 at Location D, indicating it did not effectively reduce downstream flow discharge. At the peak flow (13-hour mark), Scenario 1, had a minor increase in flow rate by 0.1% whereas Scenario 3 had a minor reduction in flow rate by 14.5% and Scenario 2 remained unchanged in the flow rate. Figure 4.59a illustrates that Scenarios 1, 2, and 3 experienced an increase in water level by 220.3%, 223.0%, and 3934.0% respectively compared to the existing condition. Flood was found for Scenario 3 at the 13-hour mark whereas no floods were found at Scenario 1 and Scenario 2.

The orifice size increased from 0.1m to 0.2m in diameter as shown in Figure 4.58b, the flow rates for all scenarios generally remained below the existing condition throughout the rainfall event. At the 13-hour peak flow rate, Scenarios 1, and Scenario 2, remained unchanged whereas Scenario 3 had a minor increment of 19.4%. Figure 4.59b shows that the water level for Scenario 1 had the lowest increment followed by Scenario 2 and Scenario 3 which increased by 223.0%, 223.1%, and 825.9% respectively. No flooding occurred with

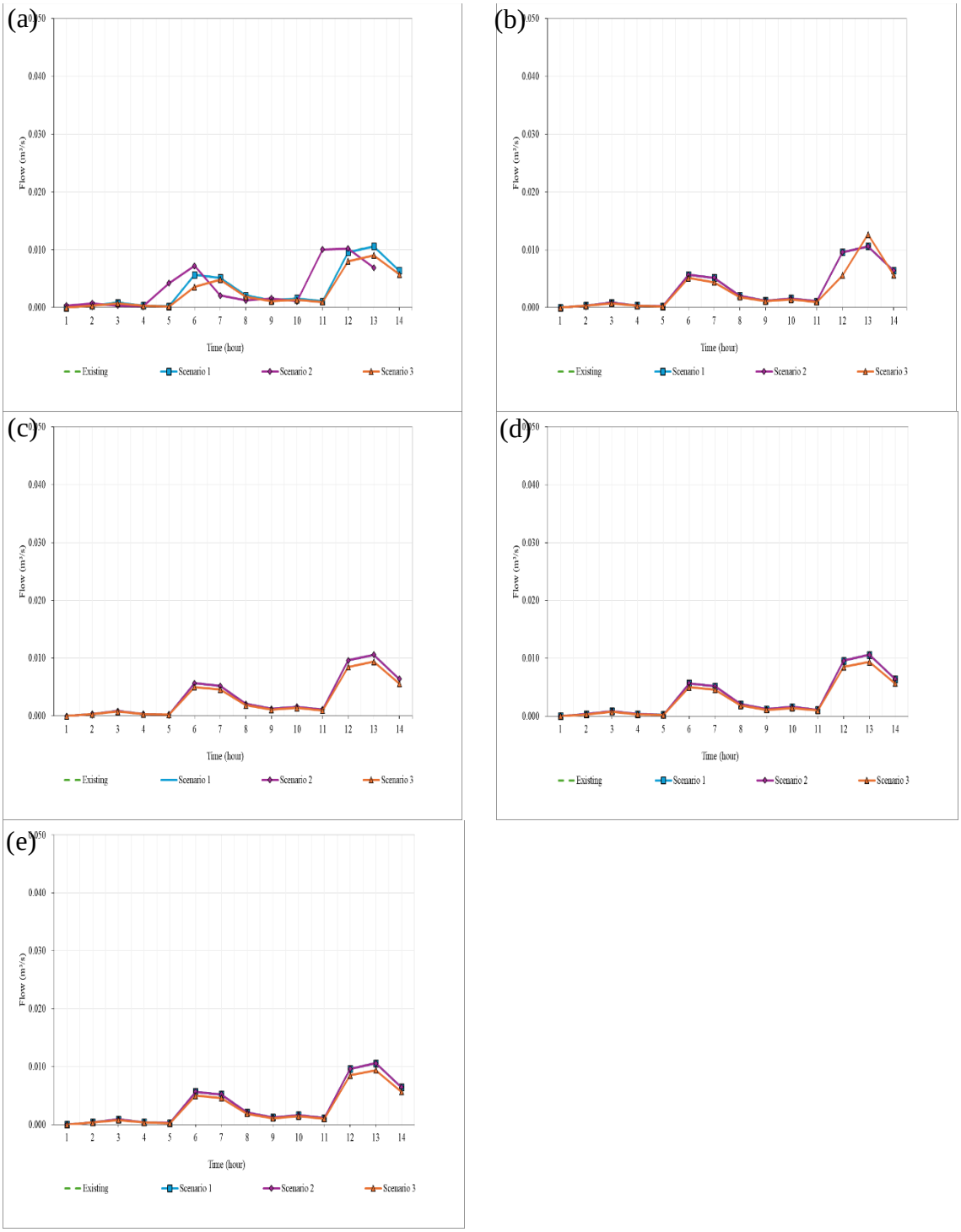


Figure 4.58: Flow Plots for Drain with 0.4% Slope at Location D Subjected to Event 1, (a) 0.1m, (b) 0. 2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

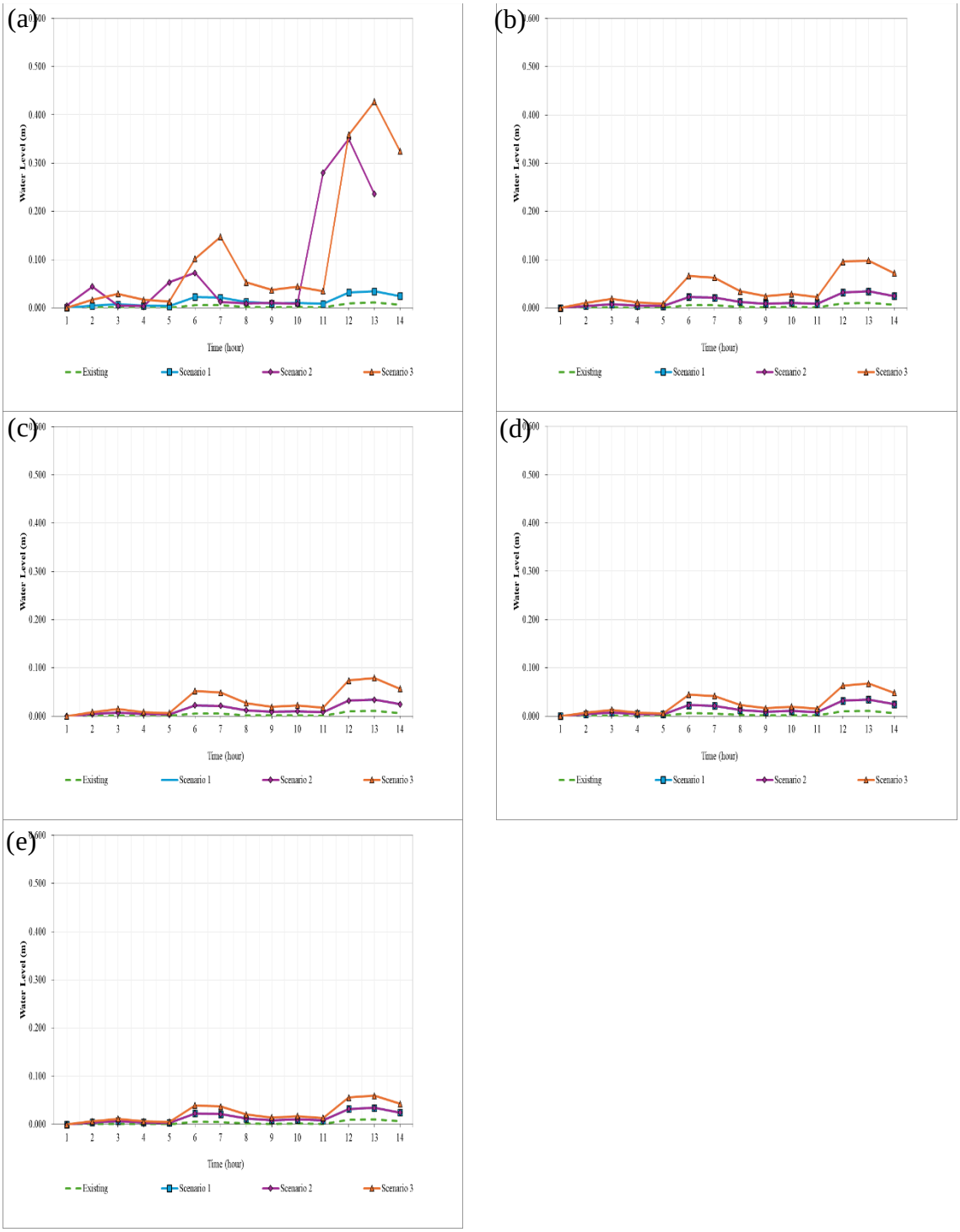


Figure 4.59: Water Level Plots for Drain with 0.4% Slope at Location D Subjected to Event 1, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

the application of a 0.2m diameter orifice in this simulation.

The existing condition and the implementation of a 0.3m diameter orifice in three scenarios as shown in Figure 4.58c. The flow rate for Scenario 1 and Scenario 2 remains unchanged whereas Scenario 3 has a slight reduction of 11.5% by comparing to existing conditions. Figure 4.59c indicates that Scenario 1 and Scenario 2 had the lowest and similar increment in water level followed by Scenario 3 which were 223.0% and 645.9% respectively. No flooding occurred with the application of a 0.3m diameter orifice in this simulation.

The orifice size increased to 0.4m in diameter as shown in Figure 4.58d, the flow rate for Scenario 1 and Scenario 2 remains unchanged whereas Scenario 3 has a slight reduction by 11.5% by comparing to an existing condition. Figure 4.59d demonstrates that Scenario 1, Scenario 2, and Scenario 3 had significant increases in water levels which were 223.0%, 223.0%, and 534.4% respectively. However, no flooding occurred at Location D as the water level remained below 0.5m.

The orifice size increased to 0.5m in diameter, the flow rates for all scenarios generally remained below the existing condition throughout the rainfall event as shown in Figure 4.58e. At the 13-hour peak flow rate, Scenarios 1, and Scenario 2 remain unchanged whereas Scenario 3 had a minor reduction of 11.5%. Figure 4.59e shows that the water level for Scenario 1, Scenario 2, and Scenario 3 had to increase in water level by 223.0%, 223.0, and 463.5% respectively. However, no flooding occurred at Location D as the water level remained below 0.5.

e. Discussion

Figures 4.60 and 4.61 summarize the peak flow rate and water level for a design slope of 0.4% during Event 1 historical rainfall. At Location A, the peak flow rate decreased for all scenarios and orifice sizes compared to the existing condition, indicating a reduction in rainfall discharge to the outfall and decreased congestion at the upstream end except for the flow rate for Scenario 1 (0.1m diameter of the orifice) and Scenario 3 (0.2m diameter of the orifice) had a minor increase in flow rate. For this historical event, only the 0.1m diameter orifice for Scenario 3 showed significant reductions in flow rate. Other conditions only showed minor reductions compared to the existing condition. However, the peak water level increased for all scenarios and orifice sizes, with every scenario using a 0.1m diameter orifice reaching the maximum water level of 0.5m, potentially causing flooding. No flooding occurred with the 0.2m, 0.3m, and 0.4m diameter orifices in any scenario. Thus, scenarios causing flooding failed to meet the objective.

At Location B, most peak flow rates remained unchanged compared to the existing condition, except for the 0.1m diameter orifice for all scenarios and the 0.4m and 0.5m diameter of the orifice for Scenario 3 showed a reduction. The 0.3m diameter orifice in Scenario 3 had a slight increase in flow rate, which would lead to more rainfall water flowing to Location A for discharge. Therefore, reducing the flow rate is a key objective in this study. For peak water levels at Location B, all scenarios showed an increase compared to the existing water level. Flooding was observed with the 0.1m diameter orifice for Scenario 3, as the water level increased to 0.5m, the maximum drain water level. Hence, the analysis for Scenario 3 with a 0.1m diameter orifice failed due to flooding.

At Location C, the peak flow rate for all scenarios and orifice sizes was reduced

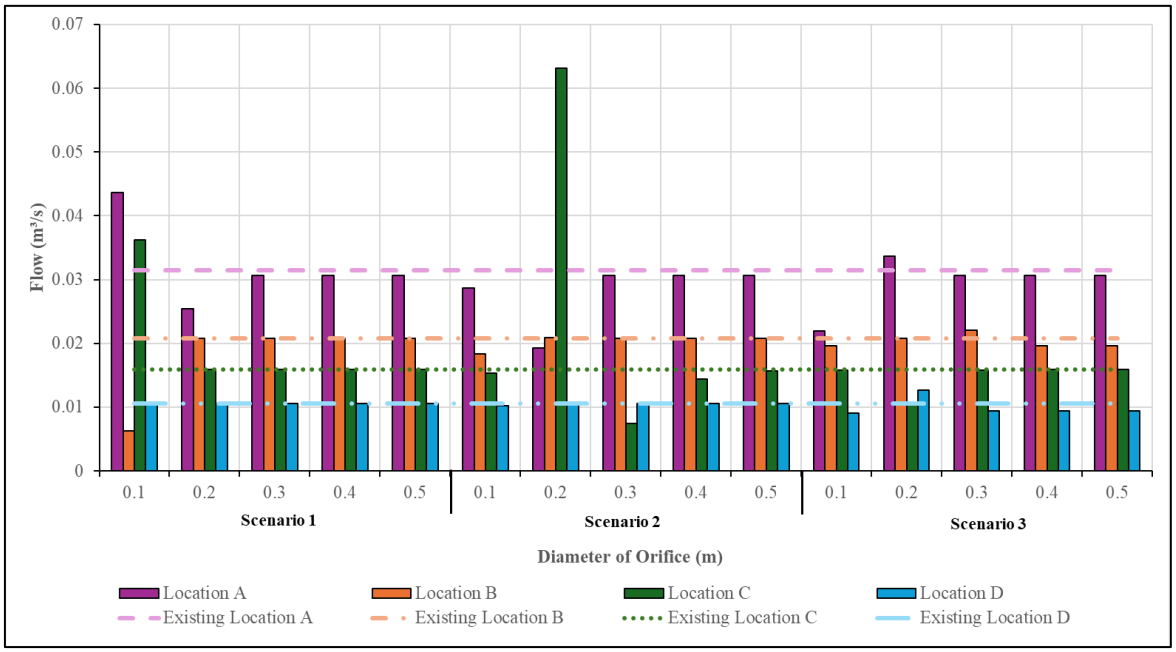


Figure 4.60: Summary of Peak Flow Rate for Event 1 Historical Rainfall at 0.4% Design Slope of Open Channel

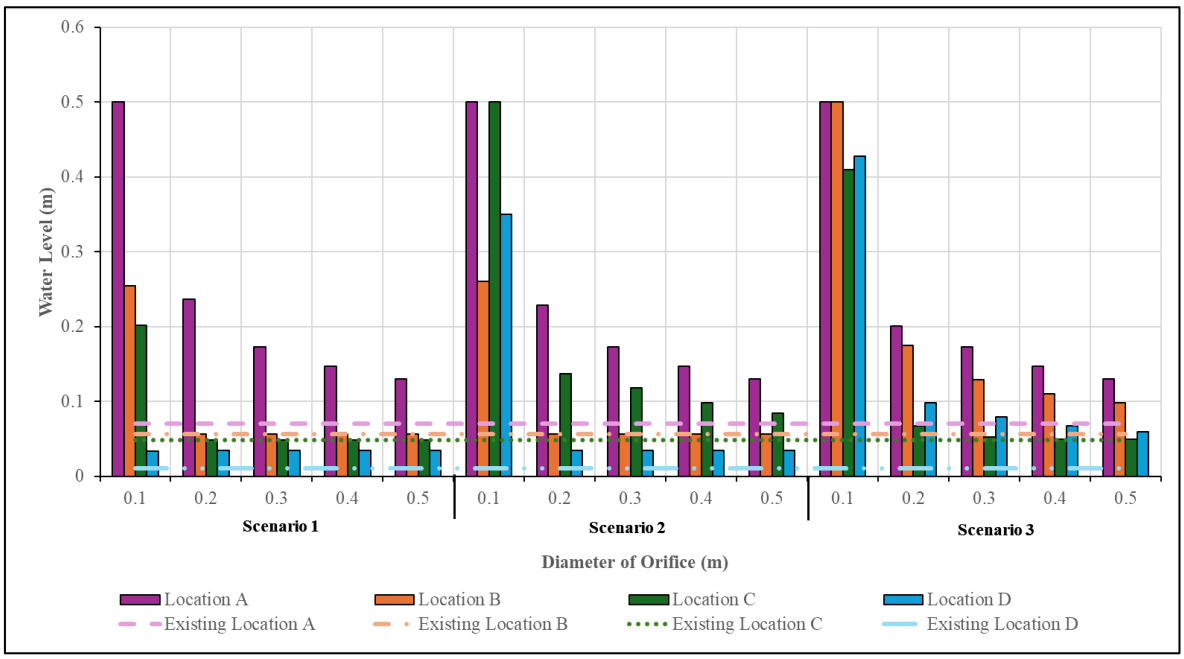


Figure 4.61: Summary of Peak Water Level for Event 1 Historical Rainfall at 0.4% Design Slope of Open Channel

compared to the existing condition, except for the 0.1m diameter orifice for Scenario 1, and 0.2m diameter orifice for Scenario 2 which showed an increase in flow rate compared to other scenarios. 0.3m diameter orifice in Scenario 2 shows a significant reduction. Greater reductions in flow rate result in slower rainfall discharge to Locations A and B. Peak water levels increased compared to the existing condition, with the 0.1m diameter orifice flow restrictor for Scenario 2 reaching the maximum water level of 0.5m, leading to failure in this analysis due to flooding. Additionally, the 0.1m diameter orifice for Scenario 3 had the second-highest increase in water level.

At Location D, the upstream end, the peak flow rate for Scenario 1 and Scenario 2 was approximately similar to the existing condition, whereas Scenario 3 showed a significant reduction in flow rate except for the 0.2m diameter of the orifice had a minor increment compared to an existing condition. Reducing the flow rate can help decrease the rainfall discharge to Location C. Peak water levels for all scenarios increased compared to the existing condition. However, the 0.1m diameter orifice for Scenario 2 and Scenario 3 showed a significant increase in water level. No flood will occur at Location D as the water level all below 0.5m.

In summary, the orifice-based flow control at a 0.4% slope can moderately reduce peak flow rates, but the benefits vary depending on orifice size and scenario. The 0.1m diameter orifice in Scenario 3 again showed strong flow reduction, especially at Locations A and B, but also led to flooding due to water levels reaching 0.5m similar to findings under the 0.2% slope. This reinforces earlier conclusions and aligns with Zheng et al. (2020) and Sembiring et al. (2022), who noted that highly restrictive orifices often trade flow control for increased upstream pressure. In contrast, larger orifices (0.2m to 0.4m) generally

prevented flooding while still reducing flow at several locations, supporting the balance emphasized by Kuok and Chiu (2022). Scenario 3 continued to show the most noticeable performance variation across locations, and the spatial differences in water level response again highlight the need for localized design consistent with Ma et al. (2024). Overall, this part of the study supports the idea that higher slopes may enhance flow reduction but also increase the risk of localized flooding when restrictive orifices are used without sufficient capacity for water level rise.

4.3 Implications of Drain Slope

Extracted from the peak flow points in the previous sub-section, the scatter plots in Figure 4.62 show the peak flows for 0%, 0.2%, and 0.4% design slopes subjected to the design rainfall, inclusive of various diameters of orifice and the scenarios of inserting orifices in the drain.

For design rainfall, the peak flow rates for the 0%, 0.2%, and 0.4% design slopes were estimated as 0.153 m^3/s, 0.235 m^3/s, and 0.207 m^3/s, respectively at Location A. Notably, the 0.2% and 0.4% design slopes showed increases in peak flow rates compared to the 0% slope, with increases of 54.0% and 35.1%, respectively. At Location B, the peak flow rates for the 0%, 0.2%, and 0.4% slopes were 0.103 m^3/s, 0.171 m^3/s, and 0.137 m^3/s, respectively. Here, the 0.2% and 0.4% slopes produced increment of 65.0% and 32.4% over the 0% slope. Similarly, at Location C, peak flow rates for the 0%, 0.2%, and 0.4% slopes were estimated at 0.074 m^3/s, 0.146 m^3/s, and 0.133 m^3/s, with the 0.2% and 0.4% slopes increasing the peak flow rates by 97.3% and 79.6%, respectively, compared to the 0% slope. Finally, at Location D, peak flow rates of 0.049 m^3/s, 0.093 m^3/s, and 0.098 m^3/s were estimated for the 0%, 0.2%, and 0.4% slopes, respectively, where the 0.2% and 0.4% slopes

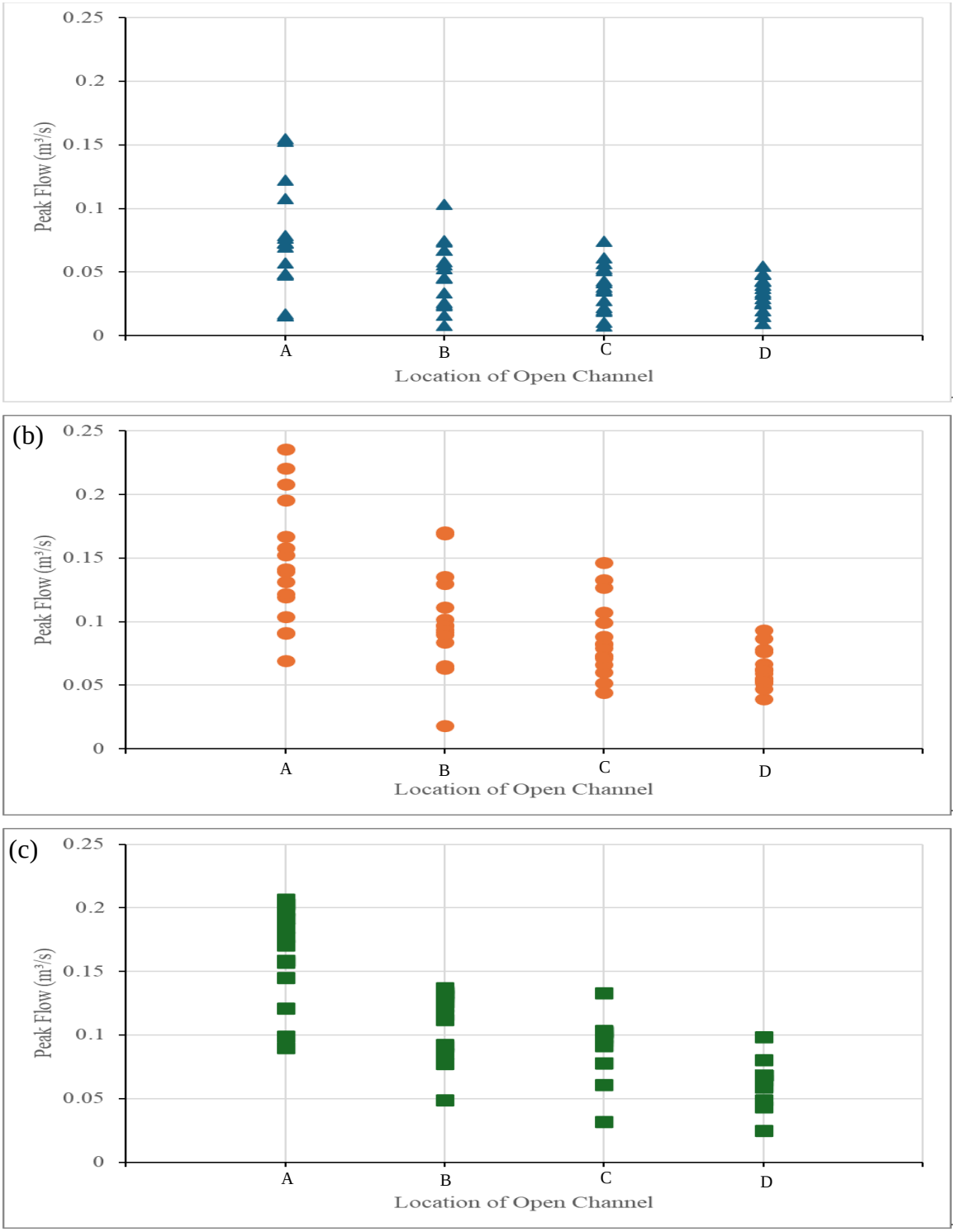


Figure 4.62: Peak Flow Plots at the Selected Locations Subjected to Design Rainfall for (a) 0%, (b) 0.2%, and (c) 0.4% Design Slopes

reflected significant increases of 92.0% and 102.2%, respectively, over the 0% slope.

Scatter plots of peak flows subjected to historical rainfall events are presented in Figure 4.63. For Event 1, at Location A, the estimated peak flow rates for the design slopes of 0%, 0.2%, and 0.4% were 0.034 m^3/s, 0.031 m^3/s, and 0.044 m^3/s, respectively. Notably, the 0.2% slope exhibited a slight reduction of 8.7%, while the 0.4% slope showed an increase of 29.6% compared to the 0% slope. At Location B, the peak flow rates for the same design slopes were 0.022 m^3/s, 0.022 m^3/s, and 0.024 m^3/s, indicating that the 0.2% slope remained unchanged, whereas the 0.4% slope yielded an increase of 8.0% over the 0% slope. Similarly, at Location C, the peak flow rates for the slopes were 0.021 m^3/s, 0.018 m^3/s, and 0.063 m^3/s, with the 0.2% slope showing a minor reduction of 17.4%, while the 0.4% slope demonstrated a significant increase of 195.0% compared to the 0% slope. Finally, at Location D, the peak flow rates were 0.017 m^3/s, 0.012 m^3/s, and 0.013 m^3/s for the 0%, 0.2%, and 0.4% slopes, respectively, reflecting reductions of 28.0% and 23.9% for the 0.2% and 0.4% slopes, respectively, in comparison to the 0% slope.

For Event 2, at Location A, the peak flow rates for the 0%, 0.2%, and 0.4% slopes were estimated at 0.043 m^3/s, 0.034 m^3/s, and 0.048 m^3/s, respectively. The 0.2% slope showed a reduction of 21.6%, while the 0.4% slope experienced an increase of 10.0% compared to the 0% slope. At Location B, the peak flow rates for the slopes were 0.023 m^3/s, 0.024 m^3/s, and 0.023 m^3/s, with the 0.2% slope reflecting a 5.9% increase and the 0.4% slope showing a 1.1% increase over the 0% slope. In Location C, the peak flow rates were 0.026 m^3/s, 0.018 m^3/s, and 0.042 m^3/s, where the 0.2% slope indicated a reduction of 31.0%, while the 0.4% slope showed an increase of 65.0% compared to the 0% slope. At Location D, the peak flow rates were 0.0117 m^3/s, 0.0118 m^3/s, and 0.0125 m^3/s for the 0%, 0.2%, and

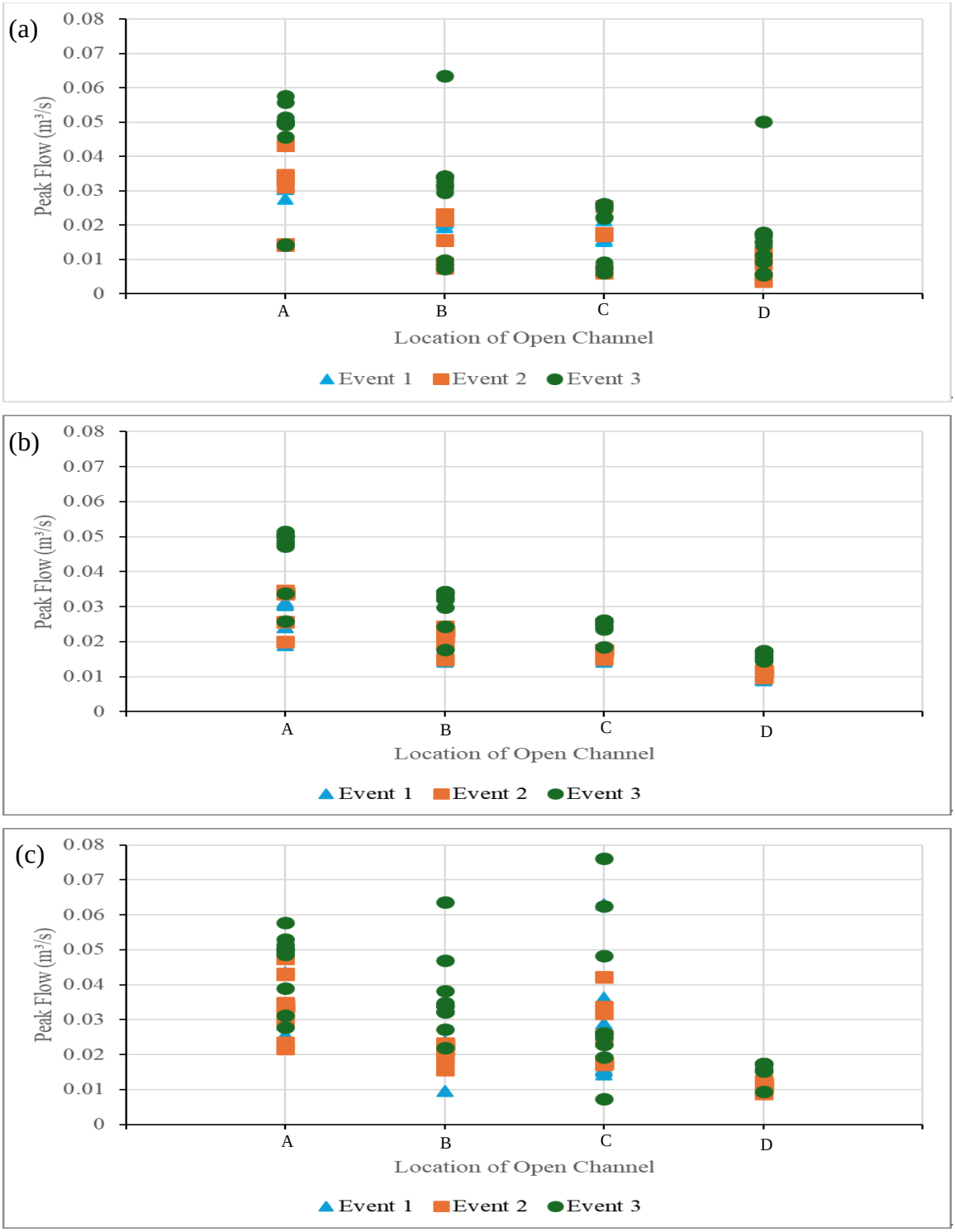


Figure 4.63: Peak Flow Plots at the Selected Locations Subjected to Historical Rainfall for (a) 0%, (b) 0.2%, and (c) 0.4% Design Slopes

0.4% slopes, respectively, with the 0.2% slope reflecting an increment of 0.4% and the 0.4% slope showing a 6.5% increase over the 0% slope.

For Event 3, peak flow rates at Location A for the 0%, 0.2%, and 0.4% slopes were estimated as 0.058 m^3/s, 0.050 m^3/s, and 0.058 m^3/s, respectively. The 0.2% slope exhibited a minor reduction of 12.9%, while the 0.4% slope remained unchanged compared to the 0% slope. At Location B, peak flow rates for the slopes were 0.064 m^3/s, 0.034 m^3/s, and 0.064 m^3/s. The 0.2% slope showed a significant reduction of 46.3%, while the 0.4% slope remained constant compared to the 0% slope. In Location C, the peak flow rates were 0.026 m^3/s, 0.026 m^3/s, and 0.076 m^3/s, where the 0.2% slope remained unchanged, while the 0.4% slope indicated a substantial increase of 192.2% relative to the 0% slope. Lastly, at Location D, the peak flow rates were 0.050 m^3/s, 0.017 m^3/s, and 0.017 m^3/s for the 0%, 0.2%, and 0.4% slopes, respectively, both the 0.2% and 0.4% slopes reflecting similar reductions of 65.3% compared to the 0% slope.

The results demonstrate a distinct influence of drain slope on peak flow responses under both design and historical rainfall events, although the effects vary with rainfall intensity. Under design rainfall conditions, the 0.2% slope consistently produced higher peak flows than both the 0% and 0.4% slopes across most locations. This pattern suggests that a moderate slope accelerates runoff more rapidly than a flat base, with insufficient time or capacity for flow attenuation, thereby leading to elevated peak discharges. However, at the 0.4% slope, peak flows showed a modest reduction. This is likely due to enhanced flow turbulence and greater energy dissipation resulting from the steeper gradient, particularly when interacting with flow-restricting structures like orifices. This aligns with the findings of Düz (2021) and Assran et al. (2024), who observed that steeper slopes can induce higher

head loss and disrupt uniform flow, thus attenuating peak discharges in systems with hydraulic controls.

In contrast, under historical rainfall events characterised by lower intensity and volume the 0.4% slope tended to yield the highest peak flows at several locations, exceeding those observed for the 0% and 0.2% slopes. This outcome may stem from the limited flow volumes being insufficient to engage dissipation mechanisms such as hydraulic jumps or full orifice submergence. As reported by Zheng et al. (2020) and Araoye et al. (2017), the performance of hydraulic structures like orifices is highly dependent on upstream head and flow regime; in low-flow conditions, steeper slopes tend to accelerate flow with minimal energy loss, whereas flatter slopes encourage temporary in-channel storage, which helps mitigate peak discharge.

These contrasting behaviours highlight the need to align drain slope design with anticipated rainfall conditions. While a 0.2% slope may intensify peak flows during extreme rainfall, it tends to perform more consistently under moderate events. Conversely, a 0.4% slope may be beneficial during high-intensity storms due to increased energy loss, but could exacerbate flow rates during lower-intensity events if not paired with effective flow control. This conclusion is supported by Kuok and Chiu (2022) and Ma et al. (2024), who stressed the importance of integrating slope design with hydraulic controls to optimize stormwater performance. Furthermore, the findings of Sembiring et al. (2022) and Assran et al. (2024) reinforce that slope effects cannot be evaluated in isolation; they must be considered alongside orifice geometry, rainfall variability, and pressure losses to ensure robust and adaptive drainage system design.

4.4 Implications of Water Level Control

The peak water levels for three different slopes (0%, 0.2%, and 0.4%) at the four selected locations under Design Rainfall conditions are presented in Figure 4.64. Figure 4.64a shows the peak water levels for exiting condition without the orifice flow restrictor whereas Figure 4.64b to Figure 4.64f show the peak water levels interrupted by orifices with diameter ranging from 0.1m to 0.5m at the four selected locations.

For the existing condition, the 0% slope had increases in peak water levels from the downstream end (Location A) to the upstream end (Location D). In contrast, with slopes of 0.2% and 0.4%, the peak water levels decreased from the downstream end to the upstream end. At 0% slope, peak water levels were recorded at 0.315 m, 0.471 m, 0.497 m, and 0.500 m for Locations A, B, C, and D, respectively. For 0.2% slope, the values were 0.339 m, 0.290 m, 0.261 m, and 0.205 m, respectively. The 0.4% slope showed further reduction in peak water levels, with 0.296 m, 0.212 m, 0.182 m, and 0.129 m at the respective locations. When comparing the 0.2% slope to the 0% slope at Location A, there was 7.6% increase in peak water level. In contrast, reductions of 8.0%, 17.2%, and 35.1% were observed at Locations B, C, and D, respectively. The 0.4% slope, in comparison to the 0% slope, showed reductions of 6.0% at Location A, 32.9% at Location B, 42.2% at Location C, and 59.0% at Location D. Overall, increasing the slope of the open channel without any flow restrictor results in a decrease in water levels.

With the implementation of orifice flow restrictors, orifice diameters ranging from 0.1 m to 0.5 m, had been evaluated. For 0% slope, peak water levels at Location A reached the maximum channel capacity of 0.5 m when using orifice diameters of 0.1 m and 0.2 m. For larger orifice diameters of 0.3 m, 0.4 m, and 0.5 m, the maximum peak water levels were

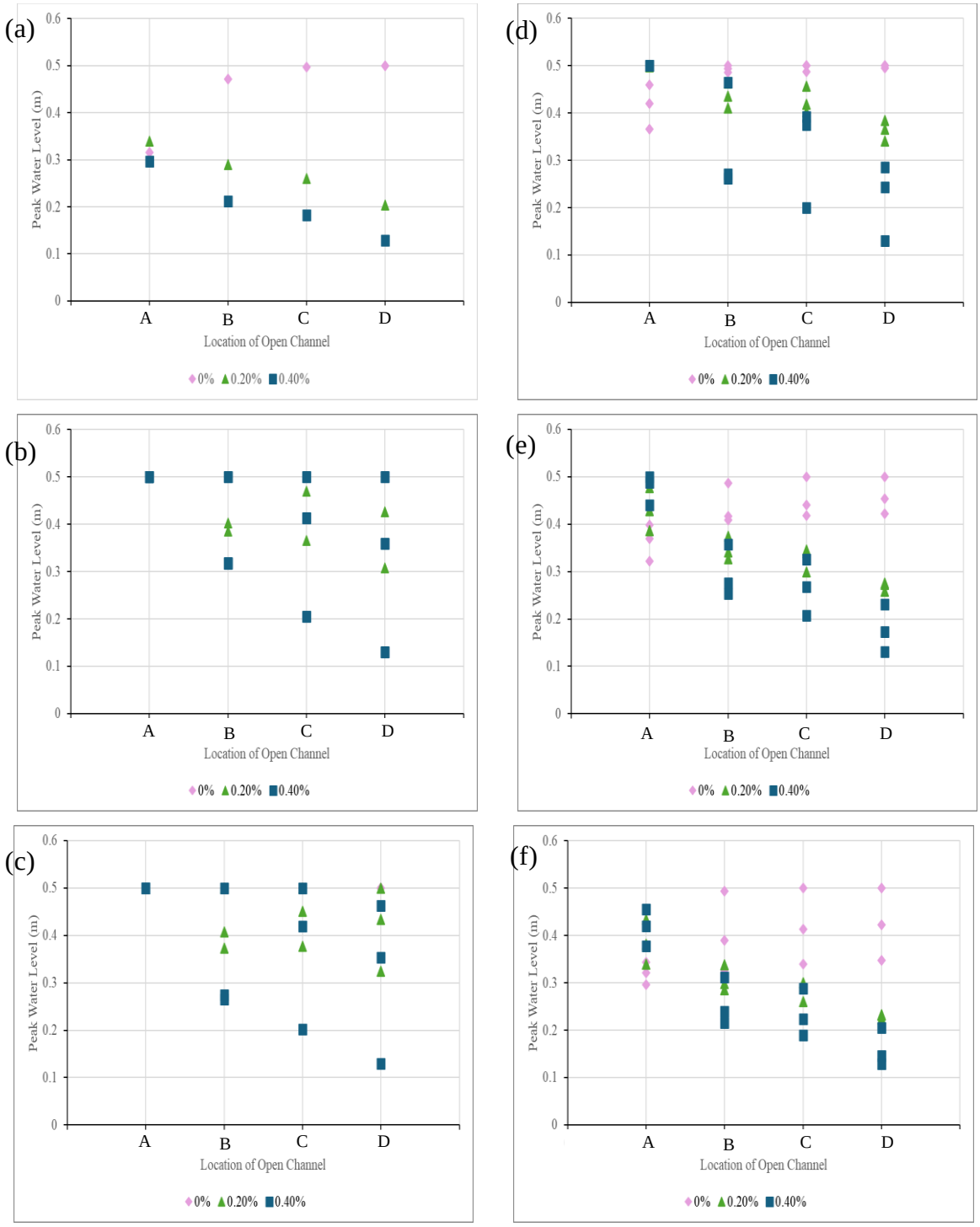


Figure 4.64: Peak Water Level Plots at the Selected Location Subjected to Design Rainfall for (a) No Orifice, (b) 0.1m, (c) 0.2m, (d) 0.3m, (e) 0.4m, and (f) 0.5m Diameter of Orifice

0.46 m, 0.40 m, and 0.34 m, respectively. As the orifice diameter decreased, the peak water level increased compared to the existing condition. The percentage differences for orifice diameters of 0.1 m to 0.5 m, compared to the existing condition, were 58.6%, 58.6%, 45.9%, 26.9%, and 7.80%, respectively. For 0.2% slope, the highest peak water levels of 0.5 m were observed for 0.1 m and 0.2 m diameter orifices, representing 47.3% increases compared to the existing condition. Peak water levels for 0.3 m, 0.4 m, and 0.5 m orifice diameters increased by 40.7%, 27.9%, and 27.9%, respectively, compared to the existing condition. For 0.4% slope, the peak water levels for 0.1 m to 0.4 m diameter orifices reached the maximum channel capacity, indicating 68.7% increases compared to the existing condition. For orifice in 0.5 m diameter, the peak water level increased by 53.6%.

At Location B, the 0% slope with 0.1 m to 0.3 m diameter orifices resulted in peak water levels of 0.5 m increased by 6.2% compared to the existing condition. Orifice diameters of 0.4 m and 0.5 m showed peak water levels increased by 3.3% and 4.8%, respectively. For 0.2% slope, 0.1 m and 0.2 m diameter orifices reached the maximum level of 0.5 m, representing 72.4% increases compared to the no-orifice condition. Peak water levels for 0.3 m, 0.4 m, and 0.5 m orifices were 0.46 m, 0.38 m, and 0.34 m, corresponding to increases of 59.8%, 29.4%, and 16.8% compared to the existing condition. At 0.4% slope, the 0.1 m and 0.2 m orifices also reached the maximum water level, showing 136.2% increases over the existing condition. Peak water levels for 0.3 m, 0.4 m, and 0.5 m orifices were 0.46 m, 0.36 m, and 0.31 m, indicating increases of 119.5%, 68.7%, and 47.2%, respectively.

At Location C, for 0% slope, all orifice diameters reached the maximum water level of 0.5 m, representing a slight increase of 0.6% compared to the existing condition. For 0.2%

slope, the 0.1 m and 0.2 m orifices reached the maximum water level, showing 91.6% increases. For orifices of 0.3 m, 0.4 m, and 0.5 m, peak levels were 0.46 m, 0.35 m, and 0.30 m, indicating increases of 75.1%, 32.3%, and 15.1%, respectively. For 0.4% slope, 0.1 m and 0.2 m orifices again reached the maximum water level, reflecting 174.3% increase. Peak water levels for 0.3 m, 0.4 m, and 0.5 m orifices were 0.39 m, 0.33 m, and 0.29 m, representing increases of 115.4%, 78.7%, and 58.4%, respectively.

At Location D, for 0% slope, all orifice diameters reached the maximum water level of 0.5 m, with no difference from the existing condition. For 0.2% slope, the 0.1 m and 0.2 m orifices also reached the maximum water level, showing 144.4% increases compared to the existing condition. Orifice diameters of 0.3 m, 0.4 m, and 0.5 m resulted in peak water levels of 0.38 m, 0.28 m, and 0.23 m, indicating increases of 88.1%, 34.9%, and 14.1%, respectively. At 0.4% slope, the 0.1 m orifice reached the maximum water level, showing a 286.9% increases. Peak water levels for 0.2 m, 0.3 m, 0.4 m, and 0.5 m orifices were 0.46 m, 0.29 m, 0.23 m, and 0.21 m, reflecting increases of 120.7%, 78.5%, and 58.9%, respectively, compared to the existing condition.

For 0% slope, all orifice diameters failed to meet the objective due to flooding at Locations C and D, particularly with the 0.1 m and 0.2 m orifices. Flooding also occurred at Location A with the 0.1 m and 0.2 m orifices and at Location B with the 0.1 m to 0.3 m orifices. For 0.2% slope, only the 0.4 m and 0.5 m orifices prevented flooding across all locations, while flooding occurred throughout the open channel for the 0.1 m and 0.2 m orifices and at Location A for the 0.3 m orifice. For 0.4% slope, only the 0.5 m orifice met the objective of preventing flooding throughout the open channel. The 0.1 m orifice caused flooding throughout the channel, while the 0.2 m orifice caused flooding everywhere except

at Location D. The 0.3 m and 0.4 m orifices only caused flooding at the downstream end (Location A).

Under historical rainfall conditions, the peak water levels for three different slopes of 0%, 0.2%, and 0.4% at four selected locations are presented in Figure 4.65. The figure illustrates the peak water levels without an orifice flow restrictor for the existing condition (Figure 4.65 (a)), and with orifice flow restrictor diameters ranging from 0.1 m to 0.5 m at the selected locations (Figure 4.65 (b)–(f)).

For existing condition at 0% slope, the water levels consistently increased from the downstream end (Location A) to the upstream end (Location D). However, a different pattern emerged at steeper slopes of 0.2% and 0.4%, where peak water levels decreased as the slope increased from Locations A to D. For Event 1, the water levels recorded at Locations A, B, C, and D were 0.132 m, 0.154 m, 0.157 m, and 0.159 m, respectively. For Event 2, the levels increased slightly to 0.140 m, 0.162 m, 0.166 m, and 0.168 m, while Event 3 saw the highest levels of 0.176 m, 0.203 m, 0.207 m, and 0.210 m. This consistent rise indicates a direct correlation between location and water accumulation in a flat slope condition. For existing condition at 0.2% slope, the pattern reversed, with peak water levels were decreasing from Locations A to D. For Event 1, the water levels measured at Locations A, B, C, and D were 0.090 m, 0.070 m, 0.061 m, and 0.046 m, respectively. In Event 2, these values increased to 0.097 m, 0.075 m, 0.065 m, and 0.049 m, while Event 3 showed a further rise to 0.129 m, 0.098 m, 0.086 m, and 0.064 m. For existing condition at 0.4% slope, the reductions in water levels were even more pronounced. For Event 1, the peak water levels at Locations A, B, C, and D were 0.071 m, 0.057 m, 0.049 m, and 0.011 m, respectively. In Event 2, the levels slightly increased to 0.076 m, 0.060 m, 0.052 m, and 0.037 m, while Event 3 yielded 0.101

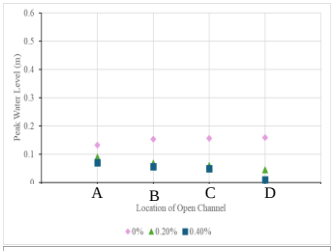
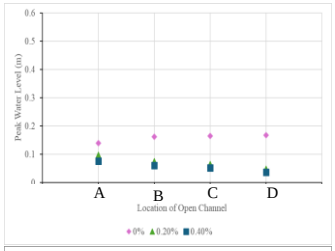
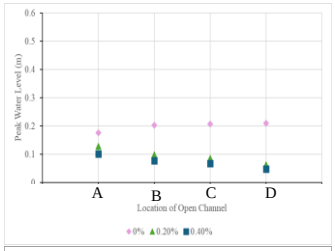
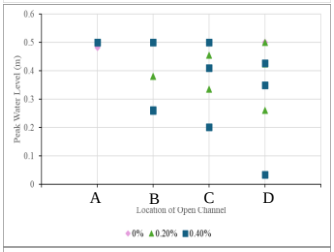
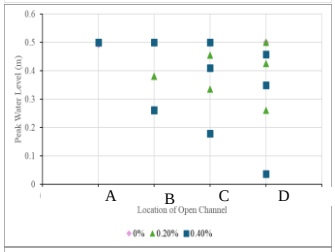
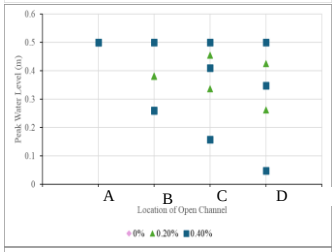
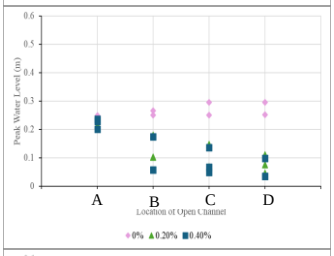
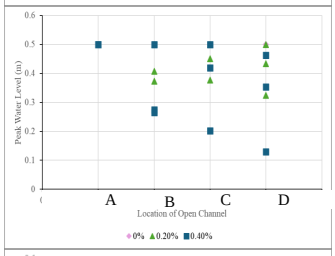
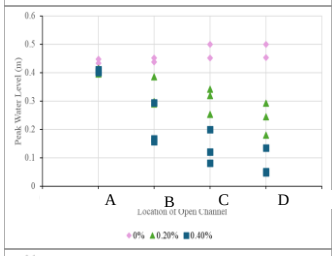
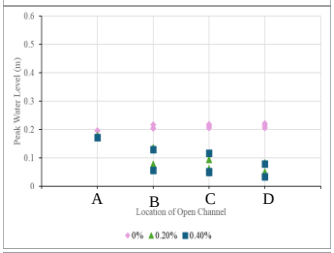
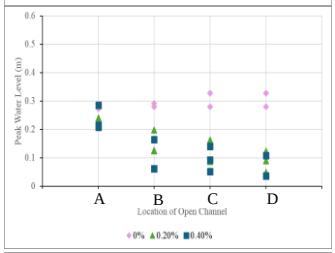
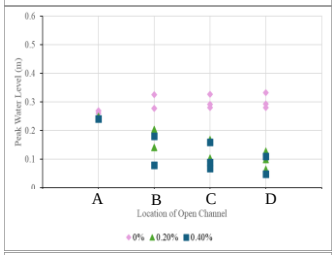
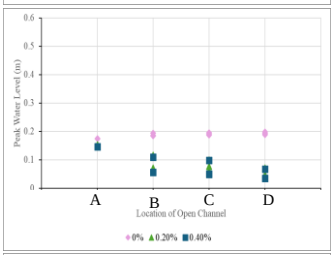
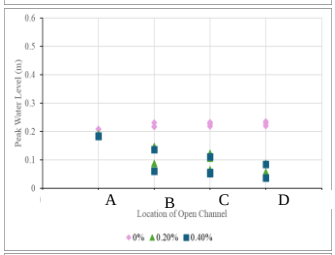
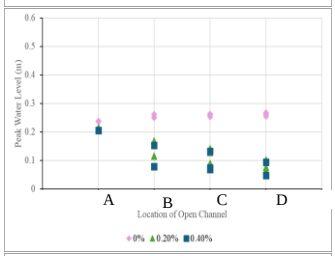
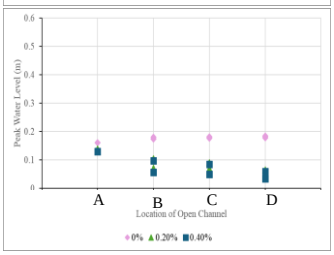
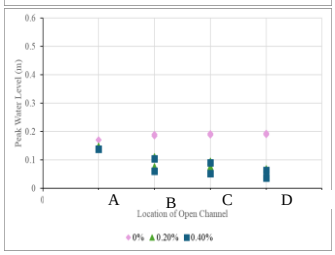
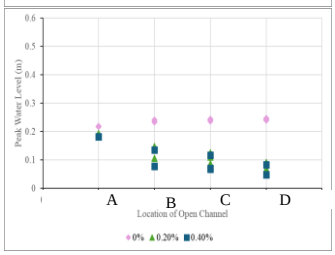
Size of Orifice	Historical Event		
	Event 1	Event 2	Event 3
(a) No Orifice			
(b) 0.1m			
(c) 0.2m			
(d) 0.3m			
(e) 0.4m			
(f) 0.5m			

Figure 4.65: Peak Water Level Plots at the Selected Location Subjected to Historical Rainfall Events for (a) No Orifice, (b) 0.1m, (c) 0.2m, (d) 0.3m, (e) 0.4m, and (f) 0.5m Diameter of Orifice

m, 0.078 m, 0.068 m, and 0.048 m. The decreasing trends in water levels with increased slope suggests the running waters in the drain with 0.2% and 0.4% were flowing faster than the 0% slope.

The performance of orifice flow restrictors, with diameters ranging from 0.1 m to 0.5 m, was assessed under varying slope conditions. For Event 1 at 0% slope, flooding occurred throughout the open channel when using 0.1 m diameter orifice, with the maximum water level reaching 0.5 m. As the orifice diameter increased, the peak water levels reduced. For 0.2 m diameter orifice, the highest peak water levels at Locations A to D were 0.249 m, 0.266 m, 0.296 m, and 0.296 m, respectively, representing increases of 88.6%, 72.9%, 89.0%, and 86.1% over no orifice. For 0.3 m diameter orifice, the peak levels were 0.196 m, 0.218 m, 0.219 m, and 0.222 m, indicating increases of 48.5%, 41.7%, 39.7%, and 39.4%. For the 0.4 m diameter orifice, the peak levels dropped further to 0.174 m, 0.194 m, 0.195 m, and 0.198 m, corresponding to increases of 31.7%, 26.0%, 24.6%, and 24.3%. For 0.5 m diameter orifice, it resulted in peak water levels of 0.161 m, 0.180 m, 0.182 m, and 0.184 m, representing increases of 21.9%, 17.2%, 16.2%, and 16.0%, respectively. For Event 1 at 0.2% slope, flooding was also observed with 0.1 m diameter orifice, reaching the same maximum water level of 0.5 m. As the orifice diameter increased, the peak water levels continued to decrease. For 0.2 m diameter orifice, the peak water levels at Locations A to D were 0.215 m, 0.179 m, 0.148 m, and 0.111 m, reflecting increases of 138%, 157%, 142.4%, and 142.6% compared to no orifice. For 0.3 m diameter orifice, the peak levels further reduced to 0.179 m, 0.137 m, 0.116 m, and 0.083 m, representing increases of 98.5%, 96.1%, 90.5%, and 82.4%. For 0.4 m diameter orifice, the peak levels decreased to 0.154 m, 0.116 m, 0.100 m, and 0.072 m, indicating increases of 70.4%, 66.3%, 63.9%, and 57.3%. For 0.5 m diameter orifice, it resulted in peak water levels of 0.138 m, 0.104 m, 0.090 m, and 0.065 m, reflecting

increases of 52.9%, 49.0%, 47.5%, and 42.1%, respectively. For Event 1 at 0.4% slope, flooding was again observed with 0.1 m diameter orifice, reaching the same maximum water level of 0.5 m at all locations except Location D, where the peak water level was slightly lower at 0.43 m. As the orifice diameter increased, the peak water levels decreased further. For 0.2 m diameter orifice, the peak water levels at Locations A to D were 0.237 m, 0.175 m, 0.137 m, and 0.098 m, corresponding to increases of 233.5%, 208.7%, 179.6%, and 825.9% over the baseline. For 0.3 m diameter orifice, the peak levels were 0.173 m, 0.129 m, 0.118 m, and 0.079 m, indicating increases of 143.6%, 128.5%, 141.2%, and 645.9%. For 0.4 m diameter orifice, the peak levels were 0.147 m, 0.110 m, 0.098 m, and 0.067 m, reflecting increases of 106.6%, 94.7%, 100.5%, and 534.4%. For 0.5 m diameter orifice, peak water levels were 0.130 m, 0.010 m, 0.085 m, and 0.060 m, corresponding to increases of 83.1%, 73.1%, 72.9%, and 463.5%, respectively.

For Event 2 at 0% slope, it was observed with flooding throughout the drain with 0.1 m diameter orifice, where the water level reached a maximum of 0.5 m. As the orifice diameter increased, the peak water levels decreased. For 0.2 m diameter orifice, the highest peak water levels at Locations A to D were 0.286 m, 0.291 m, 0.327 m, and 0.328 m, corresponding to increases of 104.5%, 79.1%, 97.8%, and 95.1% compared to the baseline. For 0.3 m diameter orifice, the peak water levels dropped to 0.209 m, 0.231 m, 0.233 m, and 0.236 m, representing increases of 49.4%, 42.5%, 40.5%, and 40.2%. For 0.4 m diameter orifice, the peak levels were 0.185 m, 0.205 m, 0.207 m, and 0.210 m, indicating increases of 32.4%, 26.5%, 25.1%, and 24.8%. For 0.5 m diameter orifice, it resulted in peak levels of 0.171 m, 0.191 m, 0.193 m, and 0.195 m, showing increases of 22.4%, 17.4%, 16.4%, and 16.2%. For Event 2 at 0.2% slope, flooding occurred with 0.1 m diameter orifice, reaching the same maximum water level of 0.5 m. As the orifice size increased, peak water levels

declined. For 0.2 m orifice, the peak water levels at Locations A to D were 0.240 m, 0.198 m, 0.162 m, and 0.124 m, with respective increases of 147.7%, 165.1%, 149.1%, and 154.0% above the baseline. For 0.3 m diameter orifice, it produced peak levels of 0.191 m, 0.148 m, 0.124 m, and 0.089 m, corresponding to increases of 97.7%, 98.1%, 90.8%, and 83.4%. For 0.4 m diameter orifice, the levels were 0.164 m, 0.125 m, 0.107 m, and 0.077 m, representing increases of 69.8%, 66.8%, 63.9%, and 57.6%. For 0.5 m diameter orifice, the peak levels were 0.148 m, 0.111 m, 0.096 m, and 0.069 m, showing increases of 52.4%, 49.1%, 47.4%, and 42.3%. For Event 2 at 0.4% slope, flooding occurred across the open channel with 0.1 m diameter orifice, reaching the maximum water level of 0.5 m at all locations except Location D, where the peak was 0.459 m. As the orifice diameter increased, peak levels dropped. For 0.2 m orifice, the peak water levels at Locations A to D were 0.286 m, 0.164 m, 0.141 m, and 0.107 m, representing increases of 276.0%, 171.8%, 171.2%, and 193.3%. For 0.3 m diameter orifice, it yielded levels of 0.185 m, 0.137 m, 0.111 m, and 0.085 m, showing increases of 142.4%, 126.6%, 114.1%, and 130.8%. For 0.4 m orifice, the peak levels were 0.157 m, 0.118 m, 0.103 m, and 0.072 m, indicating increases of 105.8%, 95.1%, 97.9%, and 96.3%. For 0.5 m diameter orifice, it resulted in peak levels of 0.139 m, 0.105 m, 0.090 m, and 0.064 m, showing increases of 82.3%, 73.4%, 73.8%, and 74.4%.

For Event 3 at 0% slope, flooding occurred across the drain with 0.1 m diameter orifice, reaching a maximum water level of 0.5 m. As the orifice diameter increased, the peak water levels decreased. For 0.2 m diameter orifice, the highest peak water levels at Locations A to D were 0.448 m, 0.452 m, 0.5 m, and 0.5 m, corresponding to increases of 154.3%, 122.7%, 141.9%, and 138.5% compared to the baseline condition. For 0.3 m diameter orifice, peak levels were 0.269 m, 0.326 m, 0.327 m, and 0.332 m, showing increases of 52.8%, 60.9%, 58.2%, and 58.5%. For 0.4 m diameter orifice, peak levels

dropped to 0.238 m, 0.262 m, 0.263 m, and 0.267 m, representing increases of 35.0%, 28.9%, 27.5%, and 27.3%. For 0.5 m diameter orifice, it resulted in peak water levels of 0.219 m, 0.241 m, 0.243 m, and 0.247 m, with increases of 24.4%, 18.9%, 17.8%, and 17.6%, respectively. For Event 3 at 0.2% slope, flooding was again observed with 0.1 m diameter orifice, reaching the same maximum water level of 0.5 m. As the orifice diameter increased, peak water levels dropped. For 0.2 m diameter orifice, the highest peak water levels at Locations A to D were 0.416 m, 0.386 m, 0.342 m, and 0.292 m, marking increases of 222.9%, 291.2%, 297.8%, and 358.1% compared to the baseline. For 0.3 m diameter orifice, peak levels were 0.250 m, 0.205 m, 0.169 m, and 0.129 m, corresponding to increases of 94.2%, 107.4%, 96.7%, and 101.6%. For 0.4 m orifice, it yielded peak levels of 0.215 m, 0.169 m, 0.141 m, and 0.103 m, indicating increases of 67.1%, 70.8%, 64.3%, and 60.8%. For 0.5 m orifice, the peak levels were 0.193 m, 0.149 m, 0.126 m, and 0.091 m, reflecting increases of 50.1%, 50.4%, 46.9%, and 42.9%, respectively. For Event 3 at 0.4% slope, flooding occurred throughout the drain for 0.1 m diameter orifice, with a maximum water level of 0.5 m at all locations. As orifice diameters increased, peak water levels steadily decreased. For 0.2 m diameter orifice, the highest peak water levels at Locations A to D were 0.411 m, 0.293 m, 0.200 m, and 0.134 m, reflecting increases of 305.8%, 275.6%, 194.2%, and 180.1% compared to the baseline. For 0.3 m diameter orifice, peak levels were 0.241 m, 0.180 m, 0.159 m, and 0.110 m, showing increases of 138.1%, 130.4%, 134.6%, and 129.8%. For 0.4 m diameter orifice, the peak levels were 0.205 m, 0.153 m, 0.132 m, and 0.094 m, representing increases of 102.2%, 95.9%, 94.4%, and 95.5%. For 0.5 m orifice, it resulted in peak water levels of 0.181 m, 0.136 m, 0.118 m, and 0.083 m, reflecting increases of 79.3%, 74.0%, 73.2%, and 73.7%, respectively.

The evaluation of open channel drainage under both design and historical rainfall conditions revealed significant effects of slope and orifice diameter on peak water levels and flooding mitigation. For both rainfall scenarios, increasing the longitudinal slope from 0% to 0.4% led to a reduction in peak water levels, attributed to the enhanced flow velocity and reduced detention time, as supported by Zheng et al. (2020) who found that steeper slopes improve hydraulic performance by accelerating discharge and reducing surface ponding.

Under design rainfall conditions, channels with a 0% slope exhibited higher water levels progressively from downstream to upstream (Locations A to D), especially in the absence of orifices. Conversely, 0.2% and 0.4% slopes displayed the opposite trend, with water levels decreasing upstream, consistent with findings from Düz (2021), which highlighted that steeper slopes create a self-cleansing effect, minimizing upstream accumulation. Introducing orifice flow restrictors generally increased water levels due to the added head loss (Araoye et al., 2017), particularly when smaller diameters (0.1 m and 0.2 m) were used. Flooding was observed at all locations for smaller orifices, especially at lower slopes, while only larger orifices (0.4 m and 0.5 m) succeeded in controlling water levels below channel capacity, demonstrating the critical balance between flow retardation and storage capacity.

Similarly, under historical rainfall, trends mirrored those of the design scenario. The 0% slope caused a backwater effect with rising peak levels from downstream to upstream, consistent across all three rainfall events. This aligns with the observations of Kuok and Chiu (2022), who noted that flat gradients in stormwater systems tend to promote water accumulation and backflow. As slope increased to 0.2% and 0.4%, peak levels fell notably, confirming that enhanced drainage gradients reduce the extent of inundation. Orifice

installations led to elevated water levels relative to the no-orifice scenario, with smaller orifices again causing flooding due to their restricted outflow capacity. Assran et al. (2024) emphasized that while orifices are effective in delaying flow, undersized orifices can cause upstream surcharging under intense rainfall.

Overall, the results validate the function of orifices as flow control devices, where smaller diameters are effective for delay but risk flooding, while larger diameters offer better flood mitigation but may not adequately slow peak discharges.

4.5 Implications of Number of Orifice and Location

The water level (WL) to peak flow (Q) relationships were evaluated under various conditions, including the existing condition (no orifice) and scenarios with different orifice sizes (0.1 to 0.5 m). The results showed how water levels responded to changes in peak flow and which scenario provided the most consistent and effective control of water levels. WL-Q plots for design rainfall are illustrated in Figure 4.66. Figure 4.66a shows the relationships for the existing condition whereas Figure 4.66b to Figure 4.66**f** show the relationships for flow through Scenarios 1 to 3 with orifice diameters from 0.1 to 0.5 m.

In the existing condition without any orifice, the WL-Q relationships were found to have wider range peak flow rates. For 0% slope, the range of peak flow rates was 0.05 to 0.16 m^3/s corresponding to peak water levels between 0.3 to 0.5 m. The subsequent steeper slopes, the ranges of peak flow rates were growing wider corresponding to narrower range of peak water levels. For 0.2% slope, the range of peak flow rates was 0.06 to 0.2 m^3/s corresponding to peak water levels between 0.2 to 0.4 m. For 0.4%, it had the same range of peak flow rates, but the corresponding peak water levels were even lower between 0.1 to 0.3 m. This reiterates the fact the running flow discharged faster with increasing steeper slopes.

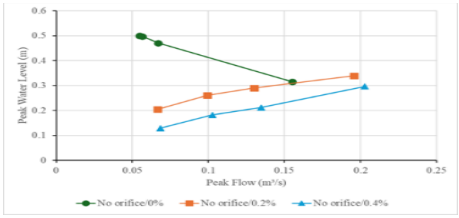
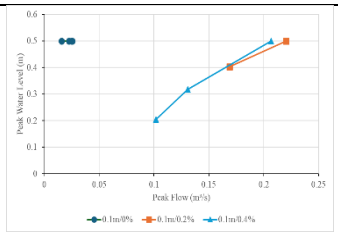
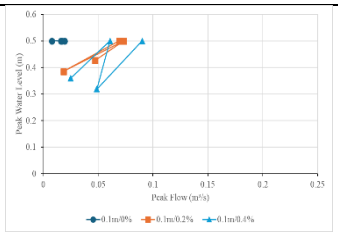
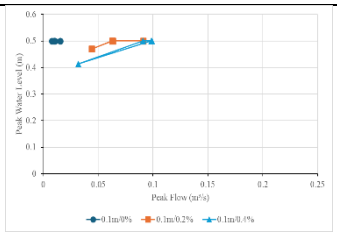
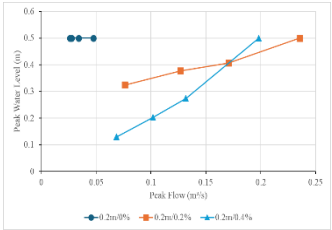
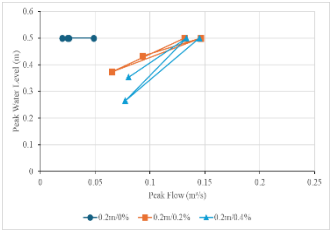
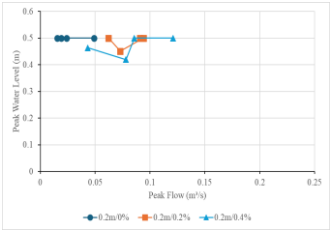
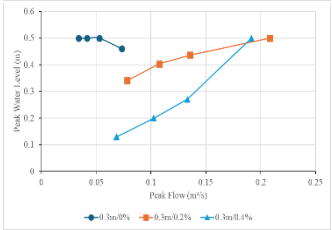
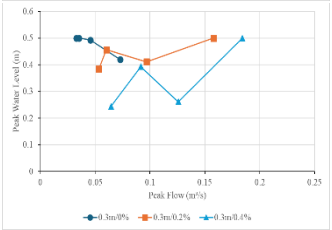
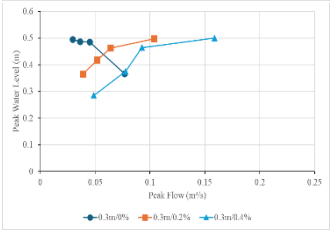
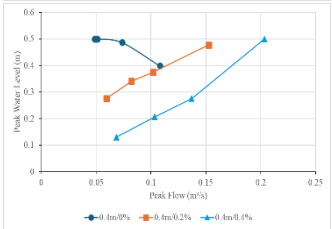
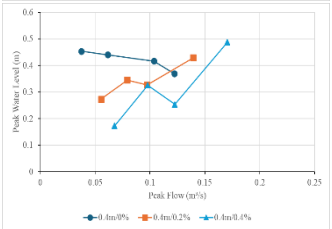
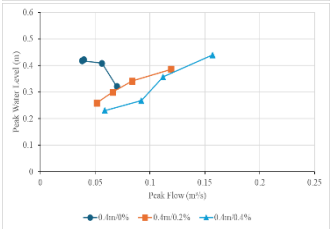
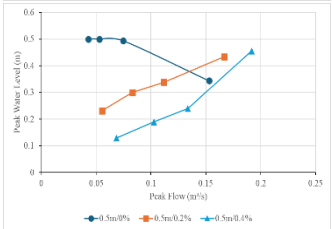
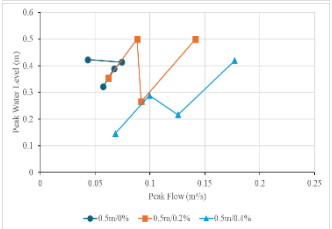
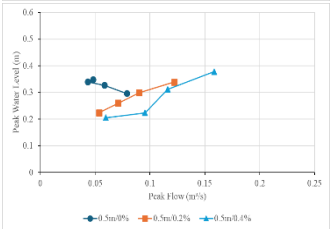
Size of Orifice	Scenario 1	Scenario 2	Scenario 3
(a) No Orifice			
(b) 0.1m			
(c) 0.2m			
(d) 0.3m			
(e) 0.4m			
(f) 0.5m			

Figure 4.66: Relationship between Peak Water Level and Flow Plots Subjected to Design Rainfall, (a) No Orifice, (b) 0.1m, (c) 0.2m, (d) 0.3m, (e) 0.4m, and (f) 0.5m Diameter of Orifice

With the insert of orifices, it was evident that the peak water levels were higher in rest of the sub-figures. It indicates that water detention occurred within the compound of the drain to raise the water levels. Scenario 1, which introduced single orifice ranging from 0.1 m to 0.5 m, showed reduction in the ranges of peak flow rates. It was the most obvious for 0% slope, in which 0.1 m being the smallest orifice diameter had the closest range between 0.01 to 0.03 m^3/s, 0.2 m diameter had between 0.02 to 0.05 m^3/s, 0.3 m diameter had between 0.03 to 0.08 m^3/s, 0.4 m diameter had between 0.04 to 0.11 m^3/s, and 0.5 diameter had between 0.04 to 0.16 m^3/s. The corresponding peak water levels for all diameter sizes range from 0.3 to 0.5 m. For 0.2% slope, 0.1 m diameter had between 0.16 to 0.22 m^3/s, 0.2 m diameter had between 0.07 to 0.25 m^3/s, 0.3 m diameter had between 0.08 to 0.21 m^3/s, 0.4 m diameter had between 0.06 to 0.16 m^3/s, and 0.5 diameter had between 0.05 to 0.17 m^3/s. The corresponding peak water levels for all diameter sizes range from 0.2 to 0.5 m. For 0.4% slope, 0.1 m diameter had between 0.1 to 0.21 m^3/s, 0.2 m diameter had between 0.06 to 0.2 m^3/s, 0.3 m diameter had between 0.06 to 0.20 m^3/s, 0.4 m diameter had between 0.06 to 0.2 m^3/s, and 0.5 diameter had between 0.06 to 0.2 m^3/s. The corresponding peak water levels for all diameter sizes range from 0.1 to 0.5 m.

In Scenario 2, which introduced a double orifice system with diameters ranging from 0.1 m to 0.5 m, a reduction in peak flow rates was observed. This reduction was most evident for a 0% slope. For the smallest orifice diameter of 0.1 m, the peak flow ranged between 0 and 0.02 m^3/s. The 0.2 m diameter orifice had a flow range of 0.01 to 0.05 m^3/s, while the 0.3 m diameter showed a range of 0.03 to 0.08 m^3/s. The 0.4 m diameter orifice produced flow rates between 0.03 and 0.13 m^3/s, and the 0.5 m diameter orifice had a flow range from 0.04 to 0.07 m^3/s. The corresponding peak water levels for all diameter sizes ranged from 0.3 m to 0.5 m. For 0.2% slope, 0.1 m diameter orifice yielded flow rates between 0.01 and

0.08 m³/s, while the 0.2 m diameter produced a range of 0.06 to 0.15 m³/s. The 0.3 m diameter orifice again showed a flow range of 0.05 to 0.16 m³/s, with the 0.4 m diameter yielding flows between 0.05 and 0.14 m³/s. The 0.5 m diameter orifice maintained a flow range of 0.06 to 0.15 m³/s. The corresponding peak water levels for all diameters remained consistent, ranging from 0.2 m to 0.5 m. In the case of the 0.4% slope, similar trends were noted. The 0.1 m diameter orifice had peak flow rates between 0.02 and 0.09 m³/s, the 0.2 m diameter ranged from 0.07 to 0.15 m³/s, and the 0.3 m diameter showed flow rates between 0.06 and 0.18 m³/s. The 0.4 m diameter orifice produced flow rates ranging from 0.06 to 0.17 m³/s, while the 0.5 m diameter orifice exhibited a flow range of 0.06 to 0.18 m³/s. The corresponding peak water levels for all diameter sizes again ranged from 0.2 m to 0.5 m.

In Scenario 3, which introduced a triple orifice ranging in diameter from 0.1 m to 0.5 m, a reduction in peak flow rates was observed. This reduction was most prominent for a 0% slope. For the smallest orifice diameter of 0.1 m, the peak flow ranged between 0 and 0.02 m³/s. As the orifice diameter increased, the peak flow ranges expanded, with the 0.2 m diameter showing a flow range between 0.01 and 0.05 m³/s, the 0.3 m diameter showing a range between 0.03 and 0.08 m³/s, the 0.4 m diameter ranging from 0.03 to 0.07 m³/s, and the 0.5 m diameter ranging from 0.04 to 0.08 m³/s. The corresponding peak water levels for all orifice diameters ranged from 0.3 m to 0.5 m. For the 0.2% slope, similar trends were observed. The orifice with a 0.1 m diameter resulted in peak flow rates between 0.04 and 0.1 m³/s, while the 0.2 m diameter produced flows between 0.06 and 0.09 m³/s. The 0.3 m diameter orifice had a range from 0.03 to 0.11 m³/s, the 0.4 m diameter had a range from 0.05 to 0.12 m³/s, and the 0.5 m diameter showed flow rates between 0.05 and 0.13 m³/s. The peak water levels across all diameters also ranged from 0.2 m to 0.5 m. For the 0.4% slope, the results were consistent with the other slopes. The 0.1 m diameter orifice had peak

flows between 0.03 and 0.1 m^3/s, the 0.2 m diameter between 0.04 and 0.12 m^3/s, the 0.3 m diameter between 0.04 and 0.16 m^3/s, the 0.4 m diameter between 0.05 and 0.16 m^3/s, and the 0.5 m diameter ranged between 0.05 and 0.16 m^3/s. The corresponding peak water levels for all diameters again ranged from 0.1 m to 0.5 m.

The number of orifice plays a significant role in regulating water levels and reducing flow rates within the open channel. For all scenarios, orifice diameters ranging from 0.1 m to 0.5 m showed a reduction in peak flow when compared to conditions without an orifice. In Scenarios 1 and 2, the highest peak water levels reached the maximum drain depth of 0.5 m. However, Scenario 3 demonstrated an improvement in water depth regulation as the orifice diameter increased from 0.1 m to 0.5 m. Notably, a clear decrease in water level was observed once the orifice diameter exceeded 0.3 m. Based on the relationship between peak water level and peak flow, it was determined that an orifice size ranging from 0.3 m to 0.5 m in Scenario 3, utilizing a triple orifice restrictor, exhibited the greatest consistency. This configuration successfully reduced the peak flow while maintaining the peak water level below the 0.5 m threshold.

This study focused on three significant historical rainfall events, labeled Event 1, Event 2, and Event 3, to evaluate the system's performance under real-world conditions. The relationship between peak water levels and peak flow rates for these historical rainfall events is illustrated in Figure 4.67. Specifically, Figure 4.67a shows the relationship for the existing condition, while Figure 4.67b to Figure 4.67d depict the implementation of orifices based on the three scenarios.

In the existing condition without any orifice, for Event 1, the range of peak flow rates was between 0.01 and 0.032 m^3/s, corresponding to peak water levels between 0.01 m and

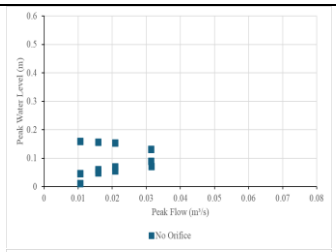
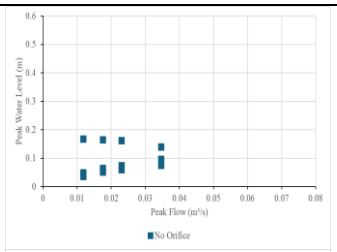
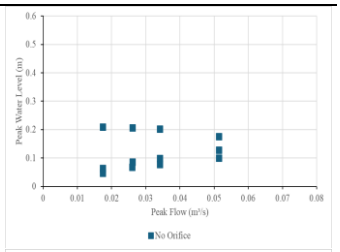
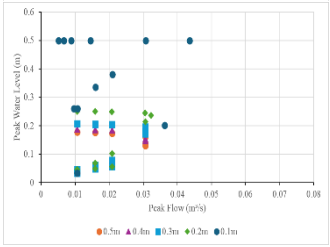
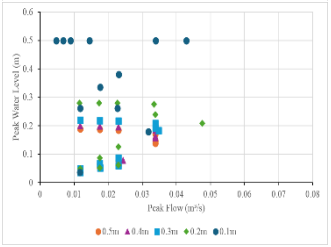
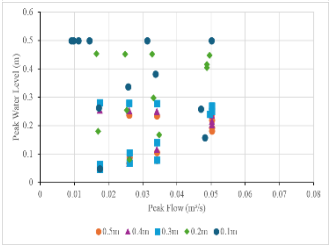
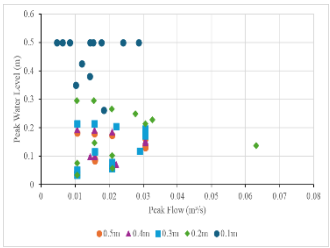
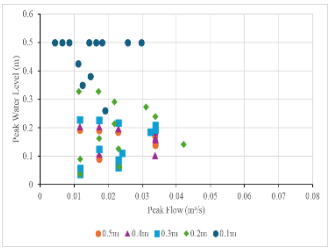
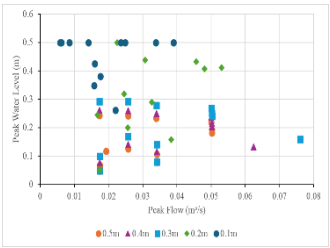
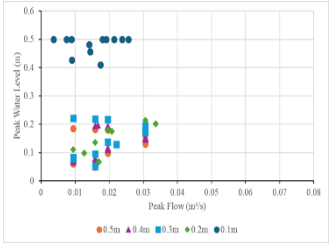
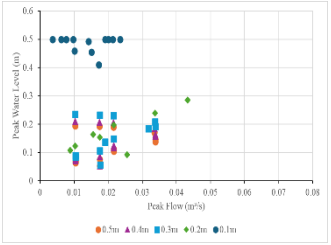
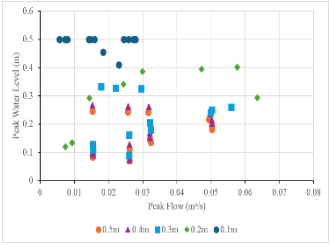
Size of Orifice	Historical Event		
	Event 1	Event 2	Event 3
(a) No Orifice			
(b) Scenario 1			
(c) Scenario 2			
(d) Scenario 3			

Figure 4.67: Relationship between Peak Water Level and Flow Plots Subjected to Event 2, (a) No Orifice, (b) Scenario 1, (c) Scenario 2, and (d) Scenario 3

0.16 m. For Event 2, the range of peak flow rates was between 0.01 and 0.035 m^3/s, corresponding to peak water levels from 0.035 m to 0.17 m. For Event 3, the range of peak flow rates was between 0.017 and 0.05 m^3/s, with peak water levels ranging from 0.045 m to 0.21 m.

The insertion of orifices significantly influenced the observed peak water levels, which were higher in the remaining sub-figures. This indicates that water detention occurred within the

compound of the drain, contributing to elevated water levels. For Event 1, Scenario 1 introduced a single orifice with diameters ranging from 0.1 m to 0.5 m, resulting in a reduction in the range of peak flow rates. For the smallest orifice diameter of 0.1 m, the peak flow rate exhibited a substantial range between 0 and 0.45 m^3/s. The 0.2 m diameter orifice had a flow rate range of 0.01 to 0.035 m^3/s, while the 0.3 m to 0.5 m diameter orifices showed a similar range of 0.01 to 0.03 m^3/s. The corresponding peak water levels for all diameter sizes ranged from 0 to 0.5 m. In Scenario 2, which introduced a double orifice system with diameters also ranging from 0.1 m to 0.5 m, a similar reduction in the range of peak flow rates was observed. For the 0.1 m diameter orifice, the peak flow rate showed a significant range between 0 and 0.03 m^3/s. The 0.2 m diameter orifice had a flow range of 0.01 to 0.065 m^3/s, while the 0.3 m to 0.5 m diameter orifices exhibited a similar range of 0.01 to 0.03 m^3/s. The corresponding peak water levels for all diameter sizes again ranged from 0 to 0.5 m. In Scenario 3, which introduced a triple orifice system with diameters ranging from 0.1 m to 0.5 m, a reduction in the range of peak flow rates was also noted. For the 0.1 m diameter orifice, the peak flow rate ranged from 0 to 0.03 m^3/s. The 0.2 m diameter orifice had a flow range of 0.009 to 0.035 m^3/s, while the 0.3 m to 0.5 m diameter orifices exhibited a similar range of 0.009 to 0.03 m^3/s. The corresponding peak water levels for all diameter sizes continued to range from 0 to 0.5 m.

For Event 2, Scenario 1 introduced a single orifice system with diameters ranging from 0.1 m to 0.5 m, resulting in a reduction in the range of peak flow rates. For the smallest orifice diameter of 0.1 m, the peak flow rate exhibited a substantial range between 0 and 0.045 m^3/s. The 0.2 m diameter orifice had a flow range of 0.01 to 0.05 m^3/s, while the 0.3 m to 0.5 m diameter orifices displayed a similar range of 0.01 to 0.035 m^3/s. The corresponding peak water levels for all diameter sizes ranged from 0 to 0.5 m. In Scenario

2, which introduced a double orifice system ranging from 0.1 m to 0.5 m, a reduction in the ranges of peak flow rates was again observed. For the 0.1 m diameter orifice, the peak flow rate showed a significant range between 0 and 0.03 m^3/s. The 0.2 m diameter orifice exhibited a flow range of 0.01 to 0.045 m^3/s, while the 0.3 m to 0.5 m diameter orifices had a similar flow range of 0.01 to 0.035 m^3/s. The corresponding peak water levels for all diameter sizes continued to range from 0 to 0.5 m. In Scenario 3, which introduced a triple orifice system with diameters ranging from 0.1 m to 0.5 m, a reduction in the ranges of peak flow rates was also noted. For the 0.1 m diameter orifice, the peak flow rate ranged from 0 to 0.025 m^3/s. The 0.2 m diameter orifice had a flow range of 0.009 to 0.045 m^3/s, while the 0.3 m to 0.5 m diameter orifices displayed a similar range of 0.01 to 0.035 m^3/s. The corresponding peak water levels for all diameter sizes remained consistent, ranging from 0 to 0.5 m.

For Event 3, Scenario 1 introduced a single orifice system with diameters ranging from 0.1 m to 0.5 m, resulting in a reduction in the ranges of peak flow rates. For the smallest orifice diameter of 0.1 m, the peak flow rate exhibited a substantial range between 0 and 0.5 m^3/s. The 0.2 m diameter orifice had a flow range of 0.01 to 0.05 m^3/s, while the 0.3 m to 0.5 m diameter orifices displayed a similar range of 0.015 to 0.03 m^3/s. The corresponding peak water levels for all diameter sizes ranged from 0 to 0.5 m. In Scenario 2, which introduced a double orifice system ranging from 0.1 m to 0.5 m, a similar reduction in the ranges of peak flow rates was observed. For the 0.1 m diameter orifice, the peak flow rate showed a significant range between 0 and 0.04 m^3/s. The 0.2 m diameter orifice exhibited a flow range of 0.015 to 0.055 m^3/s, while the 0.3 m diameter had a similar flow range of 0.015 to 0.08 m^3/s. The 0.4 m diameter orifice also demonstrated a flow range of 0.015 to 0.065 m^3/s, while the 0.5 m diameter orifice showed a flow range of 0.015 to 0.05 m^3/s. The

corresponding peak water levels for all diameter sizes continued to range from 0 to 0.5 m. In Scenario 3, which introduced a triple orifice system with diameters ranging from 0.1 m to 0.5 m, a reduction in the ranges of peak flow rates was again noted. For the 0.1 m diameter orifice, the peak flow rate ranged from 0 to 0.03 m^3/s. The 0.2 m diameter orifice had a flow range of 0.007 to 0.065 m^3/s, while the 0.3 m diameter exhibited a similar range of 0.015 to 0.06 m^3/s. The 0.4 m and 0.5 m diameter orifices both displayed a similar range of 0.015 to 0.05 m^3/s. The corresponding peak water levels for all diameter sizes remained consistent, ranging from 0 to 0.5 m.

Under existing conditions, results showed that increasing channel slope enhanced flow velocity, leading to wider ranges of peak flow and lower water levels. This observation aligns with the findings of Sembiring et al. (2022), who noted that steeper slopes contribute to increased conveyance and faster runoff, thereby reducing ponding but potentially increasing peak discharge.

The introduction of orifices significantly affected flow dynamics by reducing peak discharge and increasing water detention, which raised peak water levels. These results support studies by Araoye et al. (2017) and Zheng et al. (2020), who demonstrated that orifices act as effective flow regulators, delaying peak runoff and enhancing water retention.

Among the scenarios, Scenario 3 (triple orifice system) consistently delivered the best balance effectively reducing peak flow while keeping water levels within the maximum drain depth of 0.5 m. This outcome mirrors the conclusions of Düz (2021), who emphasized that multiple orifices help spread out the discharge over time, reducing peak flows more effectively than single-orifice designs.

Scenario 1 (single orifice) showed decreasing peak flow ranges with larger diameters but often reached the maximum drain depth, indicating limited detention performance. Scenario 2 (double orifice) improved performance further but still exhibited higher water levels. Scenario 3 achieved the most stable outcomes; particularly, orifice sizes between 0.3 m and 0.5 m provided consistent control of peak flows and water levels. This supports the findings by Ma et al. (2024), who stressed the importance of selecting appropriate orifice diameters to optimize stormwater control.

When validated against real-world historical rainfall events, Scenario 3 again demonstrated superior performance. It consistently moderated peak flow rates and maintained water levels below the 0.5 m threshold, confirming its effectiveness as a flow-regulating mechanism. This echoes the recommendation of Kuok and Chiu (2022), who advocated for model validation using historical data to ensure practical performance reliability.

Overall, the study confirms that orifice configuration and number play a critical role in optimizing open channel performance for stormwater management. Triple orifice systems, especially with diameters of 0.3 m to 0.5 m, offer the most reliable and effective solution in both design and historical rainfall conditions supporting the integrated insights of all referenced studies.

CHAPTER 5

CONCLUSION AND RECOMMENDATION

5.1 Introduction

Due to the increasing volume and runoff rate of stormwater runoff in urban areas, modifications to stormwater infrastructure are necessary to address the urban flooding problem. In this project, a compartmentalized drain was introduced as a method to detain water. The drain was divided into one, two, and three sections. Furthermore, an orifice was inserted into each thin wall to create compartments. SWMM 5.0 software was used to analyze the stormwater runoff. As stated in Chapter 1, there are three objectives for the current study: the implications of drain slope, water level control, and the number and location of orifices which are concluded in Section 5.2 to Section 5.4 respectively.

5.2 Hydrological Responses due to Drain Slope

The peak flows of the urban drain depended on the proposed drain slope. In general, the steeper the slope, the higher the flow rate of the open channel at the downstream end, which caused congestion in that area. The implementation of orifice in this study was used to control the flow rate of the open channel. For design rainfall, the 0.2% design slope had the highest peak flow rates compared to the 0% and 0.4% slopes at the downstream end of the open channel, while the highest peak flow rate for the 0.4% design slope was found at the upstream end. For historical events, the highest peak flow rates at the downstream end occurred at the 0% and 0.4% design slopes, whereas the highest peak flow rate at the upstream end was observed at the 0.4% slope.

5.3 Water Level Control due to Orifice Sizes

The sizes of the orifice played a significant role in regulating water levels within the urban drain, particularly in the context of flooding. For design rainfall, all orifice diameters at the 0% slope failed to meet the objective of flood prevention, particularly with the 0.1 m and 0.2 m orifices, despite the implementation of the orifice plate flow restrictor. At 0.2% slope, only the 0.4 m and 0.5 m orifices successfully prevented flooding across all locations, while flooding occurred throughout the open channel for the 0.1 m and 0.2 m orifices and at the downstream end for the 0.3 m orifice. At the 0.4% slope, only the 0.5 m orifice met the objective of preventing flooding throughout the open channel, whereas flooding was observed for orifices sized from 0.1 m to 0.4 m. For historical events, the 0.1 m diameter resulted in the highest water levels, with peak levels frequently reaching the maximum threshold of 0.5 m, indicating significant flow restrictions and a higher potential for flooding. Conversely, as the orifice diameter increased to 0.2 m, 0.3 m, 0.4 m, and eventually 0.5 m, the peak water levels decreased progressively.

5.4 Number of Orifices and its Location

The number of orifices played a significant role in regulating water levels and reducing flow rates within the open channel. In all scenarios, orifice diameters ranging from 0.1 m to 0.5 m showed a reduction in peak flow compared to conditions without an orifice. For design rainfall, the highest peak water levels reached the maximum drain depth of 0.5 m in Scenarios 1 and 2. However, Scenario 3 demonstrated an improvement in water depth regulation as the orifice diameter increased from 0.1 m to 0.5 m. Notably, a clear decrease in water level was observed once the orifice diameter exceeded 0.3 m. For historical events, the smallest orifice diameter of 0.1 m resulted in the highest water levels, frequently reaching

the maximum threshold of 0.5 m. This indicates significant flow restrictions and a higher potential for flooding. Conversely, as the orifice diameter increased to 0.2 m, 0.3 m, 0.4 m, and ultimately 0.5 m, the peak water levels decreased progressively. Based on the relationship between peak water level and peak flow, it was determined that an orifice size ranging from 0.3 m to 0.5 m in Scenario 3, utilizing a triple orifice restrictor, exhibited the greatest consistency. The stated configuration successfully reduced the peak flow while maintaining the peak water level below the 0.5 m threshold.

5.5 Significance of Findings

This study contributes significantly to the field of urban stormwater management by providing a practical and validated approach to optimizing open channel flow control through the use of orifices. The findings demonstrate that the integration of a triple orifice system can effectively reduce peak discharge while maintaining water levels within the safe operational depth of 0.5 m, even under intense rainfall events. This contributes directly to improved flood mitigation strategies in urban environments, particularly where space and infrastructure capacity are limited. By analyzing the effects of different orifice configurations and channel slopes, this research offers practical design guidance for engineers aiming to enhance the performance of stormwater drainage systems. Moreover, the validation of simulation results against historical rainfall data reinforces the real-world applicability and reliability of the proposed design configurations. The study supports the broader goal of sustainable urban planning by promoting controlled stormwater detention and discharge, which reduces pressure on existing infrastructure and enhances overall system resilience. In summary, the outcomes of this thesis provide a valuable reference for future stormwater system designs, contributing towards more adaptive and effective solutions in managing urban flood risks.

5.6 Recommendations for Future Study

In this thesis, model verification was performed based on the principles of hydrological processes. To determine the degree to which the models accurately represent the actual system, it is recommended that future research includes a field test for model validation. Additionally, further research can be conducted to address the issue of blockage at the orifice openings, as the thin wall that houses the orifice also functions as a weir. Runoff will be able to flow over the weir once the water level reaches its crest level.

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APPENDICES

Appendix A: Urban Drain Design

(Design based on MSMA Chapter 14)

Design Rainfall : 5-minute, 10-year ARI

Reference	Calculation	Output
	Determine design flows for the drain	
	Using design rainfall of 5-minute, 10-year ARI	
	Rainfall Intensity based on Appendix D = 278 mm/hr	
Equation 2.3	$Q = \frac{CIA}{360}$	
	Where: Q = peak flow (m^3/s);	
Table 2.5	C = dimensionless runoff coefficient;	0.80
	I = average rainfall intensity over time of concentration, t_c (mm/hr); and	278mm/hr
	A = drainage area (ha)	0.422 ha
	$Q_{10} = \frac{0.80 \times 278 \times 0.422}{360}$	0.261 m^3/s
	Calculate size of lined drain section	
Table 2.3	Manning's n for concrete lining	0.015

Assuming:

Drain longitudinal slope — 0.4% (1 in 250)

Width, B — 0.50 m

Side Slope, Z — 0

$Q = v \times A$

Where:

$$v = \frac{1}{n} S_o^{1/2} R^{2/3}$$

$$A = (B + ZD)D \; = \; 0.50D$$

$$P = B + 2D\sqrt{1 + Z^2} \; = \; 0.50 + 2D$$

D (m)	**A (m^2)**	**P (m)**	**R (m)**	**v (m/s)**	**Q (m^3/s)**
0.10	0.050	0.7	0.0714	0.7259	0.0363
0.15	0.075	0.8	0.0938	0.8701	0.0653
0.20	0.100	0.9	0.1111	0.9745	0.0974
0.25	0.125	1.0	0.1250	1.0541	0.1318
0.30	0.150	1.1	0.1364	1.1170	0.1676
0.35	0.175	1.2	0.1458	1.1682	0.2044
0.40	0.200	1.3	0.1538	1.2106	0.2421
0.45	0.225	1.4	0.1607	1.2464	0.2804

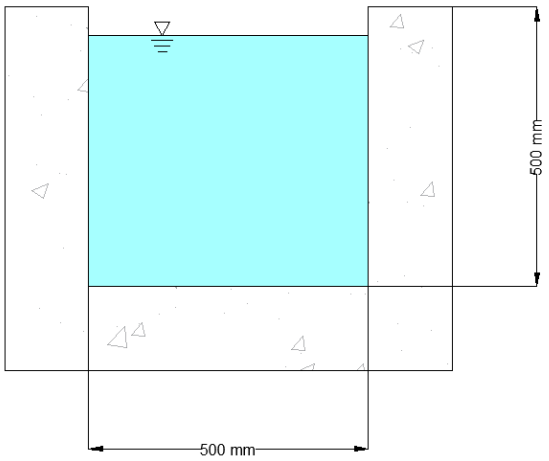
↑ Check — ↑ OK

(> Q_{10} = 0.26m^3/s)

Allowing a minimum freeboard — 50 mm

Hence, required depth, $D_{minimum}$ — 0.45 m

Section 14.2.4.2	Provided dimension of lined drain (Refer to precast concrete manufacturers):	
	Width, B	0.50 m
	Depth, D	0.50 m
	Velocity, v	1.28 m/s
	Discharge, Q	0.319 m^3/s
Section 14.2.4.3	The drain dimensions are 0.50 m wide x 0.50 m deep, which is within the recommended limits.	
Section 14.2.4.1	**Check flow velocity for lined drain**	1.28 m/s (0.6<v<2 m/s) OK!
	Depth maximum of 0.6 m, lined drain shall be open.	



Appendix B: Invert Level of the Open Drain



Figure B1: Elevation for the First Junction of an Open Channel (Google Earth)

Based on Google Earth, the elevation of first upstream node for the front and back open channel are 7m. Interpolation method is used to calculate the following elevation of the junction with design slope of 0.2% and 0.4%. Figure B2, Figure B4, and Figure B6 below show the elevation in front of housing area for 0%, 0.2% and 0.4% respectively. Figure B3, Figure B5, and Figure B7 shows the invert level of the open channel at the downstream and upstream end for for 0%, 0.2% and 0.4% respectively.

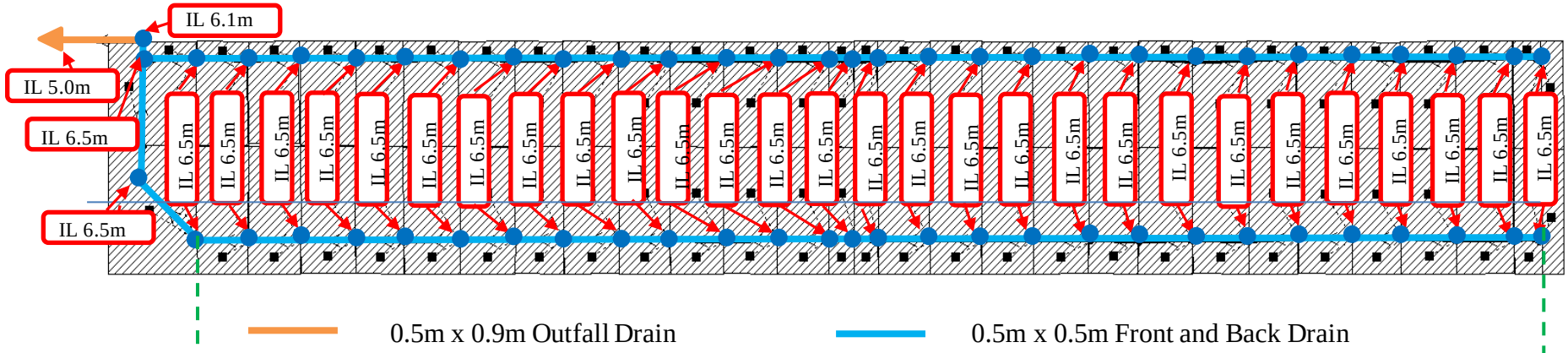


Figure B1: Elevation in Front of the Housing Area for 0%

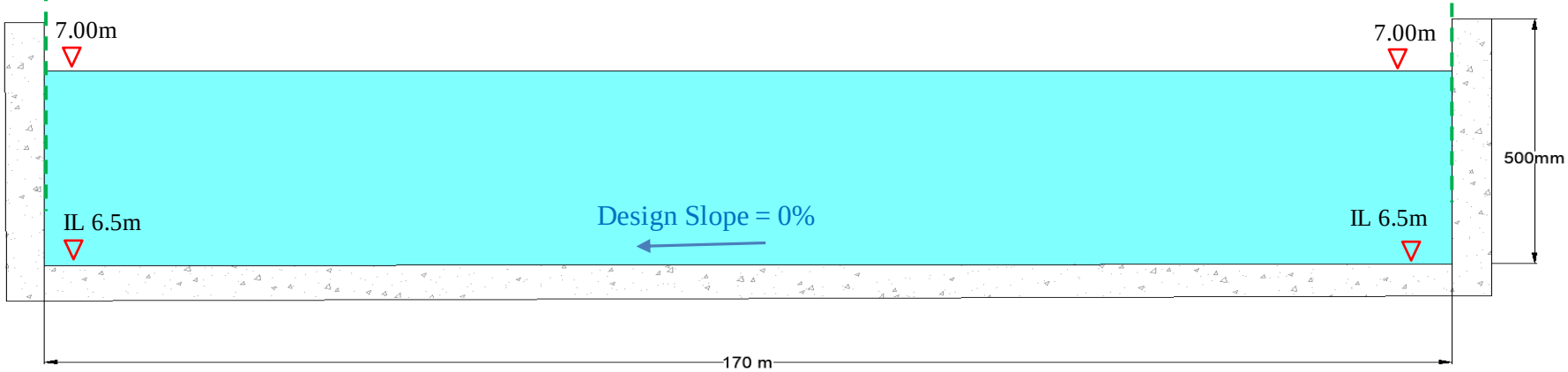


Figure B2: Longitudinal Sectional View of the Open Channel for 0%

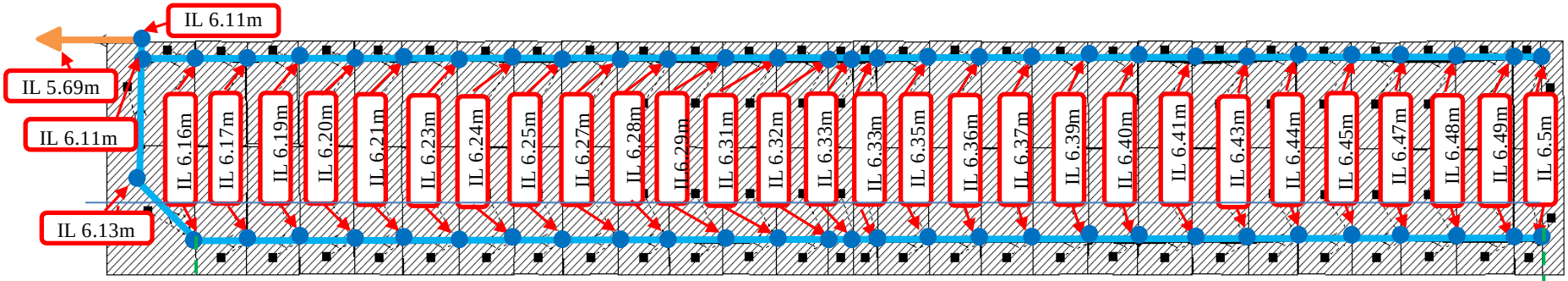


Figure B3: Elevation in Front of the Housing Area for 0.2%

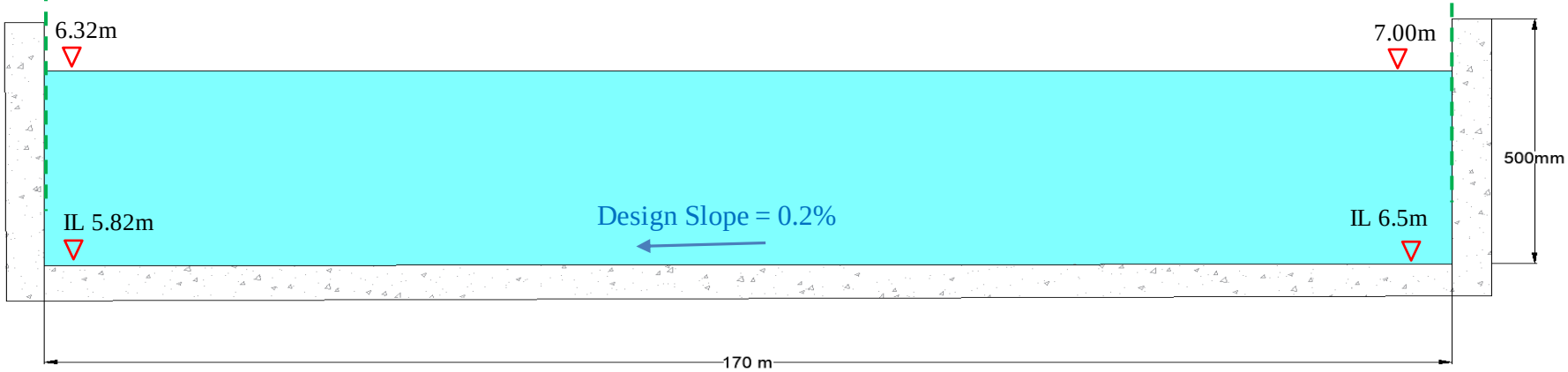


Figure B4: Longitudinal Sectional View of the Open Channel for 0.2%

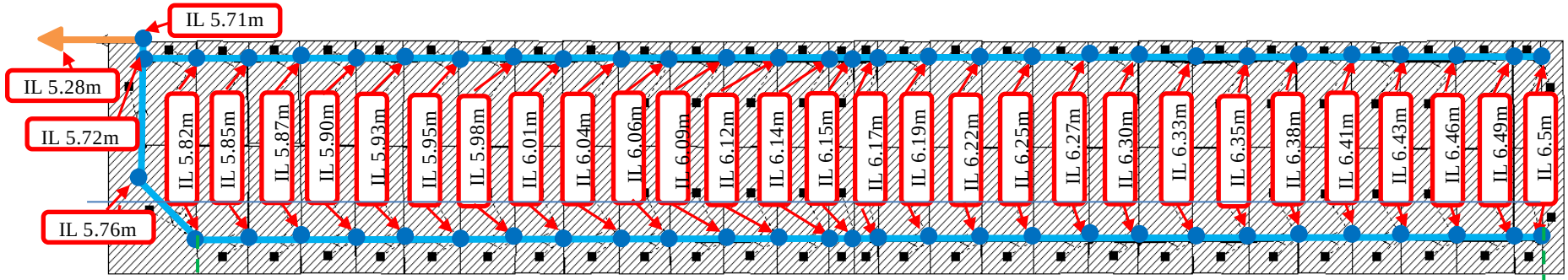


Figure B6: Elevation in Front of the Housing Area for 0.4%

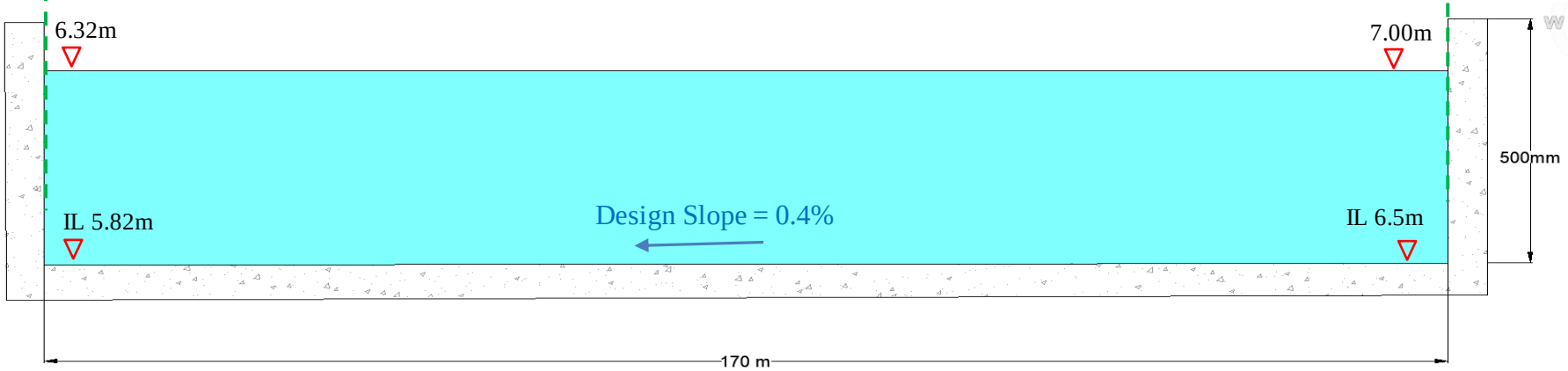


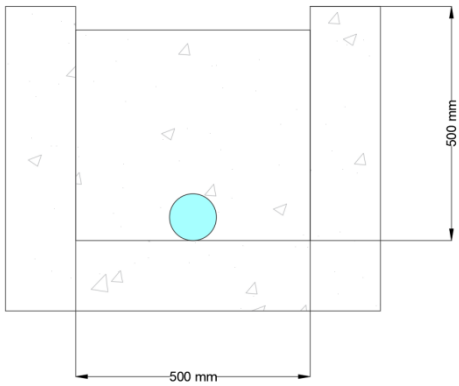
Figure B7: Longitudinal Sectional View of the Open Channel for 0.4%

Appendix C: Flow Through Orifice

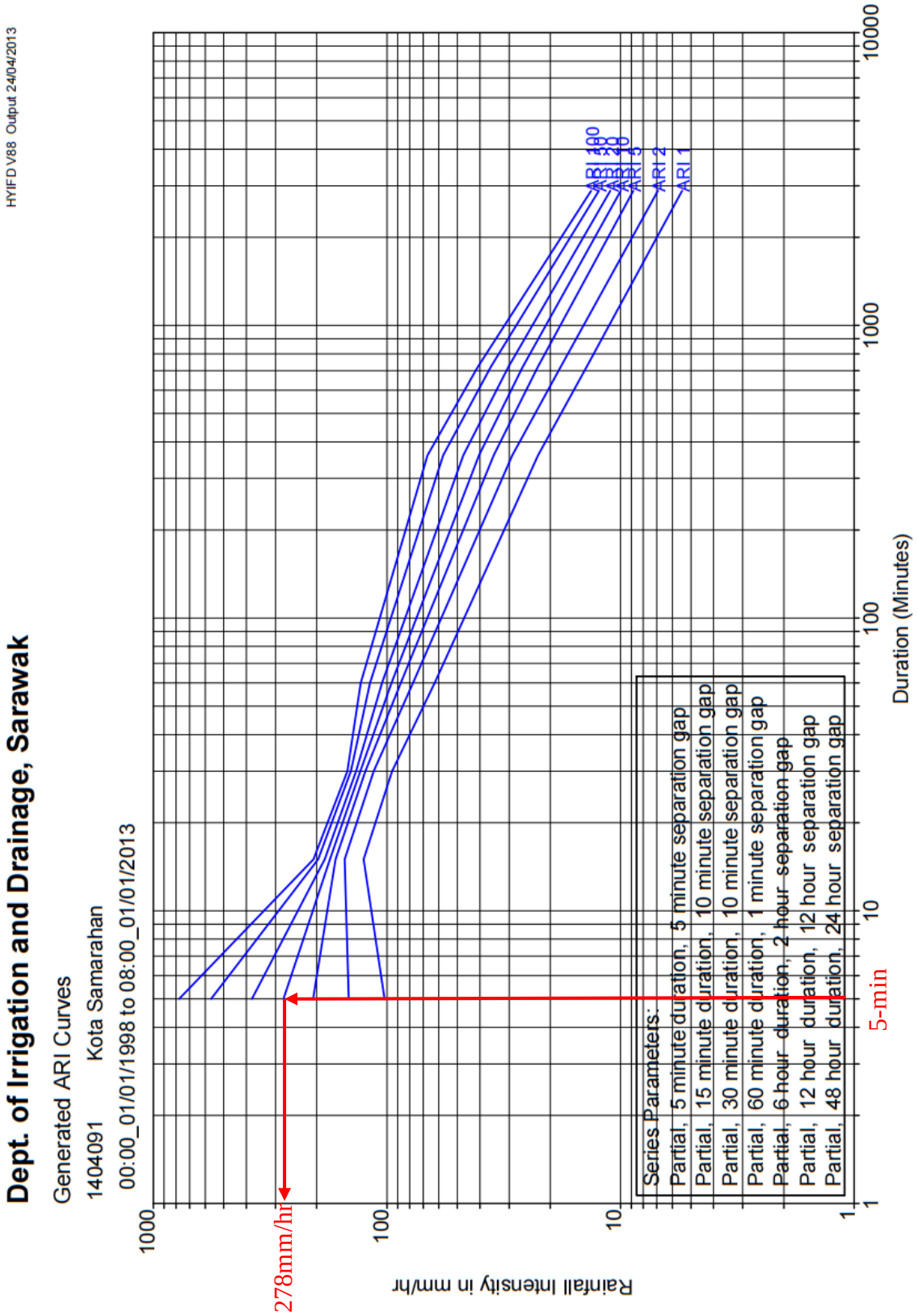
Reference	Calculation	Output
Equation 2.6	**Determine design flows for the 0.1m, 0.2m, 0.3m, 0.4m and 0.5m diameter of orifice** $Q = A_0 C_O \sqrt{2hg}$ Where: Q = Orifice flow rate (m^3/s); A_o = X-sectional area of orifice (m^2); C_o = Orifice coefficient (circular orifice=0.60); h = Efficient head of the orifice measured from the centroid of the opening (m); and g = Gravitational acceleration ($9.81 m/s^2$)	

Diameter of Orifice (m)	A_o (m^2)	h (m)*	Q (m^3/s)
0.1	0.0079	0.45	0.0140
0.2	0.0314	0.40	0.0528
0.3	0.0707	0.35	0.1112
0.4	0.1257	0.30	0.1829
0.5	0.1964	0.25	0.2609

* Note: Assuming drain-full flow



Appendix D: Intensity-Duration-Frequency Curve for Kota Samarahan



Appendix E: Evidences Of Historical Extreme Rainfall Events

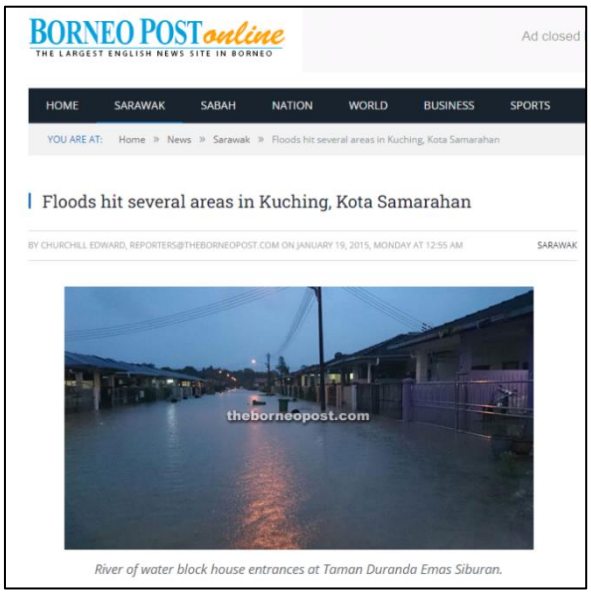


Figure E 1: Flooding at Taman Duranda Emas on 19 Jan 2015

(extracted from Borneo Post online)

(https://www.theborneopost.com/2015/01/19/floods-hit-several-areas-in-kuching-kota-samarahan/)

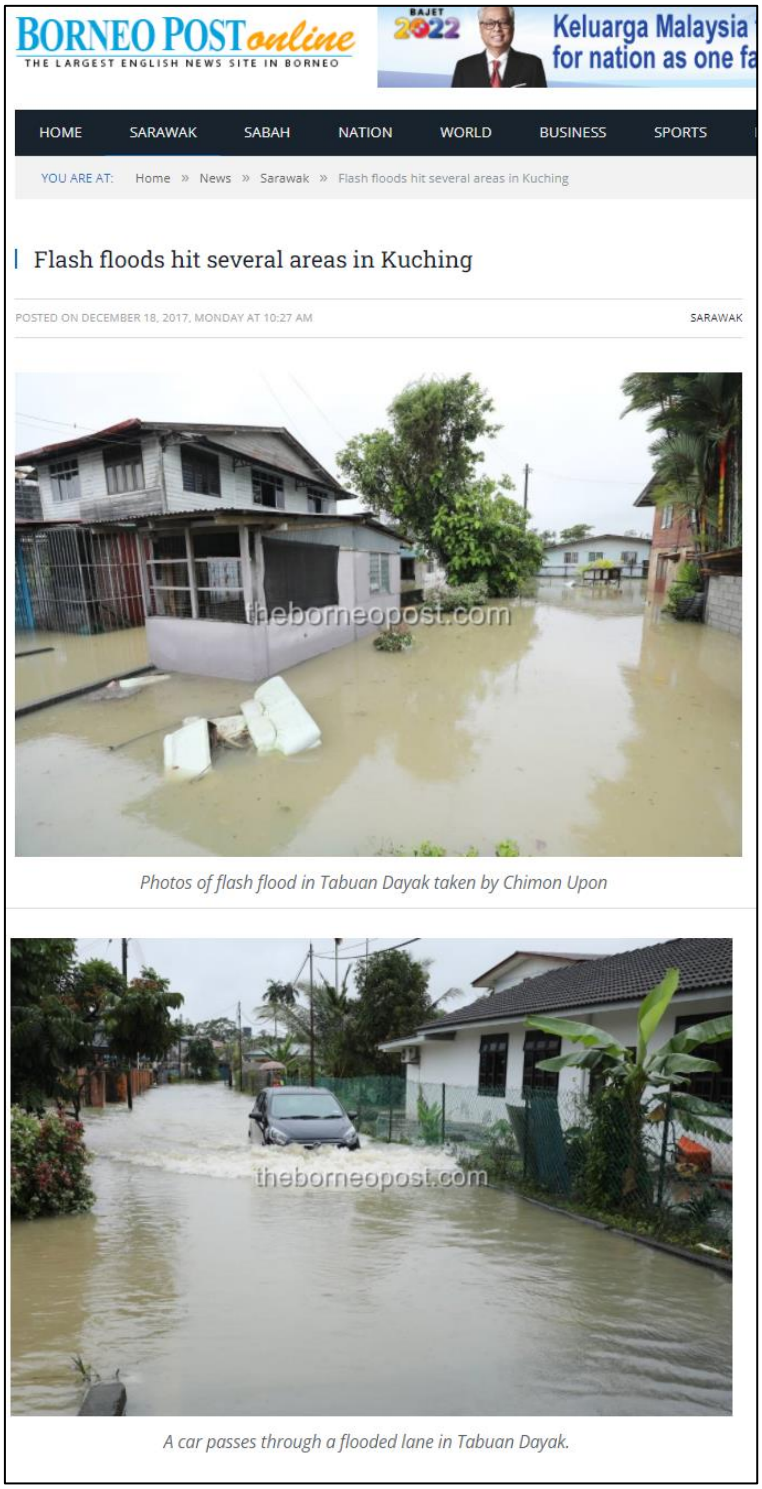


Figure E 2: Flooding at Taman Dayak on 18 Dec 2017

(extracted from Borneo Post online)

(https://www.theborneopost.com/2017/12/18/flash-floods-hit-several-areas-in-kuching/)



Figure E 3: Flooding at Jalan Tong Wei Tah on 11-12 Dec 2019

(extracted from MySarawak FB Page)

Appendix F: Historical Extreme Rainfall Data

Table F 1: Historical Extreme Rainfall Events 1 On 19 Jan 2015 (14 Hours Storm)

Time (hr)	Rainfall (mm)
1200-1300	1.5
1300-1400	3.5
1400-1500	1.5
1500-1600	1
1600-1700	23
1700-1800	21
1800-1900	8.5
1900-2000	5
2000-2100	6.5
2100-2200	4.5
2200-2300	39
2300-0000	43
0000-0100	26
0100-0200	4

Table F 2: Historical Extreme Rainfall Events 2 On 17-18 Dec 2017 (15 Hours Storm)

Time (hr)	Rainfall (mm)
1800-1900	0.5
1900-2000	0.5
2000-2100	0.5
2100-2200	7.5
2200-2300	6
2300-0000	15
0000-0100	10
0100-0200	9
0200-0300	10
0300-0400	19.5
0400-0500	32
0500-0600	47.5
0600-0700	16
0700-0800	4
0800-0900	0.5

Table F 3: Historical Extreme Rainfall Events 3 On 11-12 Dec 2019 (15 Hours Storm)

Time (hr)	Rainfall (mm)
1345 – 1445	8.0
1445 – 1545	70.5
1545 – 1645	20.0
1645 – 1745	6.0
1745 – 1845	5.5
1845 – 1945	4.5
1945 – 2045	4.5
2045 – 2145	3.5
2145 – 2245	2.5
2245 – 2345	0.5
2345 – 0045	0.5
0045 – 0145	1.0
0145 – 0245	1.5
0245 – 0345	1.5
0345 – 0445	1.0

Appendix G: Model Verification Data

Appendix G1: Catchment Flows

		1	2		3	(1 x 2 x 3)/360	
Catchment	No of Polygon	A (ha)	C	Storm Event	I (mm/hr)	Q Rational Method (m^3/s)	Q SWMM Catchment (m^3/s)
A	2	0.004	0.8	Design rainfall	278	0.002471	0.003072
		0.004	0.8	Event 1	43.0	0.000382	0.000474
		0.004	0.8	Event 2	47.5	0.000422	0.000524
		0.004	0.8	Event 3	70.5	0.000627	0.000779
B	2	0.005	0.8	Design rainfall	278	0.003089	0.003840
		0.005	0.8	Event 1	43.0	0.000478	0.000592
		0.005	0.8	Event 2	47.5	0.000528	0.000655
		0.005	0.8	Event 3	70.5	0.000783	0.000974
C	27	0.001	0.8	Design rainfall	278	0.000618	0.000768
		0.001	0.8	Event 1	43.0	0.000096	0.000118
		0.001	0.8	Event 2	47.5	0.000106	0.000131
		0.001	0.8	Event 3	70.5	0.000157	0.000195
D	24	0.00513	0.8	Design rainfall	278	0.003169	0.003962
		0.00513	0.8	Event 1	43.0	0.000490	0.000613
		0.00513	0.8	Event 2	47.5	0.000542	0.000677
		0.00513	0.8	Event 3	70.5	0.000804	0.001005
E	24	0.00787	0.8	Design rainfall	278	0.004862	0.006078
		0.00787	0.8	Event 1	43.0	0.000752	0.000940
		0.00787	0.8	Event 2	47.5	0.000831	0.001038
		0.00787	0.8	Event 3	70.5	0.001233	0.001541

		1	2		3	(1 x 2 x 3)/360	
Catchment	No of Polygon	A (ha)	C	Storm Event	I (mm /hr)	Q Rational Method (m^3/s)	Q SWMM Catchment (m^3/s)
F	10	0.00787	0.8	Design rainfall	278	0.004862	0.006078
		0.00787	0.8	Event 1	43.0	0.000752	0.000940
		0.00787	0.8	Event 2	47.5	0.000831	0.001038
		0.00787	0.8	Event 3	70.5	0.001233	0.001541
F	10	0.0005	0.8	Design rainfall	278	0.000309	0.000384
		0.0005	0.8	Event 1	43.0	0.000048	0.000059
		0.0005	0.8	Event 2	47.5	0.000053	0.000065
		0.0005	0.8	Event 3	70.5	0.000078	0.000097
G	2	0.00108	0.8	Design rainfall	278	0.000667	0.000834
		0.00108	0.8	Event 1	43.0	0.000103	0.000129
		0.00108	0.8	Event 2	47.5	0.000114	0.000142
		0.00108	0.8	Event 3	70.5	0.000169	0.000212
H	24	0.002412	0.8	Design rainfall	278	0.001490	0.001863
		0.002412	0.8	Event 1	43.0	0.000230	0.000288
		0.002412	0.8	Event 2	47.5	0.000255	0.000318
		0.002412	0.8	Event 3	70.5	0.000378	0.000472

Appendix G2: Existing Drain Flows

Location	Storm Event	Water Level (m)	b (m)	n	A (m^2)	R	S	Q (Manning Equation) (m^3/s)	Q (SWMM Drain) (m^3/s)
Back house Downstream	5-min 10-ARI	0.2812	0.5	0.018	0.1406	0.13234	0.004	0.12829	0.13420
	Event 1	0.4538	0.5	0.018	0.2269	0.16120	0.004	0.23614	0.25090
	Event 2	0.2844	0.5	0.018	0.1422	0.13305	0.004	0.13021	0.12010
	Event 3	0.3681	0.5	0.018	0.1841	0.14888	0.004	0.18166	0.18980
Front House Downstream	5-min 10-ARI	0.1769	0.5	0.018	0.0884	0.10358	0.004	0.06853	0.06650
	Event 1	0.2609	0.5	0.018	0.1305	0.12767	0.004	0.11621	0.10660
	Event 2	0.1963	0.5	0.018	0.0982	0.10996	0.004	0.07915	0.07610
	Event 3	0.1893	0.5	0.018	0.0947	0.10773	0.004	0.07529	0.07960

Appendix G3: Orifice Flows

Type of Orifice	Storm Event	Scenario	Location of Orifice	A (m^2)	Water Level at Upstream (m)	h (m)	C	Q (Orifice Formula) (m^3/s)	Q (SWMM Orifice) (m^3/s)
0.1m Ø Circular Orifice	5-min 10-ARI	S1	A	0.00786	0.34783	0.29783	0.65	0.01234	0.01221
		S2	A	0.00786	0.31944	0.26944	0.65	0.01174	0.01196
			B	0.00786	0.41365	0.36365	0.65	0.01364	0.01383
		S3	A	0.00786	0.34808	0.29808	0.65	0.01235	0.01264
			C	0.00786	0.29246	0.24246	0.65	0.01114	0.01117
			D	0.00786	0.36327	0.31327	0.65	0.01266	0.01288
	Event 1	S1	A	0.00786	0.40587	0.35587	0.65	0.01349	0.01356
		S2	A	0.00786	0.35230	0.30230	0.65	0.01243	0.01228
			B	0.00786	0.34552	0.29552	0.65	0.01229	0.01227
		S3	A	0.00786	0.29853	0.24853	0.65	0.01127	0.01114
			C	0.00786	0.30546	0.25546	0.65	0.01143	0.01116
			D	0.00786	0.34359	0.29359	0.65	0.01225	0.01233

Type of Orifice	Storm Event	Scenario	Location of Orifice	A (m^2)	Water Level at Upstream (m)	h (m)	C	Q (Orifice Formula) (m^3/s)	Q (SWMM Orifice) (m^3/s)
0.1m Ø Circular Orifice	Event 2	S1	A	0.00786	0.40548	0.35548	0.65	0.01348	0.01359
		S2	A	0.00786	0.34689	0.29689	0.65	0.01232	0.01216
			B	0.00786	0.40471	0.35471	0.65	0.01347	0.01354
		S3	A	0.00786	0.32394	0.27394	0.65	0.01184	0.01145
			C	0.00786	0.32044	0.27044	0.65	0.01176	0.01172
			D	0.00786	0.39208	0.34208	0.65	0.01323	0.01315
	Event 3	S1	A	0.00786	0.47668	0.42668	0.65	0.01477	0.01403
		S2	A	0.00786	0.49603	0.44603	0.65	0.01510	0.01549
			B	0.00786	0.40443	0.35443	0.65	0.01346	0.01372
		S3	A	0.00786	0.32173	0.27173	0.65	0.01179	0.01177
			C	0.00786	0.29633	0.24633	0.65	0.01122	0.01119
			D	0.00786	0.42961	0.37961	0.65	0.01393	0.01394

Type of Orifice	Storm Event	Scenario	Location of Orifice	A (m^2)	Water Level at Upstream (m)	h (m)	C	Q (Orifice Formula) (m^3/s)	Q (SWMM Orifice) (m^3/s)
0.2m Ø Circular Orifice	5-min 10-ARI	S1	B	0.00126	0.50000	0.49000	0.65	0.00253	0.00230
			C	0.00126	0.50000	0.49000	0.65	0.00253	0.00240
			D	0.00126	0.50000	0.49000	0.65	0.00253	0.00250
		S2	B	0.00126	0.50000	0.49000	0.65	0.00253	0.00243
			C	0.00126	0.50000	0.49000	0.65	0.00253	0.00233
			D	0.00126	0.50000	0.49000	0.65	0.00253	0.00232
		S3	D	0.00126	0.50000	0.49000	0.65	0.00253	0.00220
	Event 1	S1	A	0.00126	0.24500	0.23500	0.65	0.00175	0.00158
			B	0.00126	0.24980	0.23980	0.65	0.00177	0.00180
			C	0.00126	0.25060	0.24060	0.65	0.00177	0.00175
			D	0.00126	0.25120	0.24120	0.65	0.00178	0.00158
		S2	A	0.00126	0.24930	0.23930	0.65	0.00177	0.00168
			B	0.00126	0.26570	0.25570	0.65	0.00183	0.00180

Type of Orifice	Storm Event	Scenario	Location of Orifice	A (m^2)	Water Level at Upstream (m)	h (m)	C	Q (Orifice Formula) (m^3/s)	Q (SWMM Orifice) (m^3/s)
0.2m Ø Circular Orifice	Event 1	S2	C	0.00126	0.29610	0.28610	0.65	0.00194	0.00178
		S3	B	0.00126	0.17450	0.16450	0.65	0.00147	0.00162
	Event 2	S1	B	0.00126	0.27932	0.26932	0.65000	0.00188	0.00220
			C	0.00126	0.28001	0.27001	0.65000	0.00188	0.00170
		S2	C	0.00126	0.32732	0.31732	0.65	0.00204	0.00180
		S3	B	0.00126	0.16378	0.15378	0.65	0.00142	0.00180
			C	0.00126	0.09306	0.08306	0.65	0.00104	0.00150
			D	0.00126	0.10739	0.09739	0.65	0.00113	0.00130
	Event 3	S1	A	0.00126	0.44820	0.43820	0.65000	0.00240	0.00220
			B	0.00126	0.45180	0.44180	0.65000	0.00241	0.00220
			C	0.00126	0.45230	0.44230	0.65000	0.00241	0.00240
		S2	A	0.00126	0.43280	0.42280	0.65	0.00235	0.00240
			B	0.00126	0.43810	0.42810	0.65	0.00237	0.00220

Type of Orifice	Storm Event	Scenario	Location of Orifice	A (m^2)	Water Level at Upstream (m)	h (m)	C	Q (Orifice Formula) (m^3/s)	Q (SWMM Orifice) (m^3/s)
0.2m Ø Circular Orifice	Event 3	S2	C	0.00126	0.49980	0.48980	0.65	0.00253	0.00250
			D	0.00126	0.50000	0.49000	0.65	0.00253	0.00250
		S3	A	0.00126	0.40150	0.39150	0.65	0.00226	0.00230
			B	0.00126	0.29340	0.28340	0.65	0.00193	0.00220
			C	0.00126	0.12010	0.11010	0.65	0.00120	0.00130
			D	0.00126	0.13410	0.12410	0.65	0.00127	0.00130
0.3m Ø Circular Orifice	5-min 10-ARI	S1	A	0.07070	0.32710	0.17710	0.65	0.08566	0.09020
		S2	A	0.07070	0.25000	0.10000	0.65	0.06437	0.06420
			B	0.07070	0.39640	0.24640	0.65	0.10104	0.09150
		S3	A	0.07070	0.33300	0.18300	0.65	0.08707	0.08400
			C	0.07070	0.21500	0.06500	0.65	0.05189	0.04820
			D	0.07070	0.38660	0.23660	0.65	0.09901	0.09060
	Event 1	S1	A	0.07070	0.22030	0.07030	0.65	0.05397	0.05590

Type of Orifice	Storm Event	Scenario	Location of Orifice	A (m^2)	Water Level at Upstream (m)	h (m)	C	Q (Orifice Formula) (m^3/s)	Q (SWMM Orifice) (m^3/s)
0.3m Ø Circular Orifice	Event 1	S2	A	0.07070	0.20720	0.05720	0.65	0.04868	0.05340
			B	0.07070	0.16750	0.01750	0.65	0.02693	0.02480
		S3	A	0.07070	0.31670	0.16670	0.65	0.08310	0.08410
			D	0.07070	0.16490	0.01490	0.65	0.02485	0.02760
	Event 2	S1	A	0.07070	0.29680	0.14680	0.65	0.07799	0.07670
		S2	A	0.07070	0.44350	0.29350	0.65	0.11027	0.10500
		S3	A	0.07070	0.20340	0.05340	0.65	0.04704	0.05050
			D	0.07070	0.19400	0.04400	0.65	0.04270	0.04820
	Event 3	S1	A	0.07070	0.22076	0.07076	0.65	0.05414	0.05210
		S2	A	0.07070	0.21533	0.06533	0.65	0.05202	0.05106
			B	0.07070	0.17187	0.02187	0.65	0.03010	0.02927
		S3	A	0.07070	0.24250	0.09250	0.65	0.06190	0.06200
			D	0.07070	0.18730	0.03730	0.65	0.03931	0.03320

Type of Orifice	Storm Event	Scenario	Location of Orifice	A (m^2)	Water Level at Upstream (m)	h (m)	C	Q (Orifice Formula) (m^3/s)	Q (SWMM Orifice) (m^3/s)
0.4m Ø Circular Orifice	5-min 10-ARI	S1	A	0.12568	0.24107	0.04107	0.65	0.07333	0.09974
			B	0.12568	0.22266	0.02266	0.65	0.05447	0.07365
			C	0.12568	0.23259	0.03259	0.65	0.06532	0.05117
			D	0.12568	0.24627	0.04627	0.65	0.07784	0.04858
		S2	A	0.12568	0.34054	0.14054	0.65	0.13565	0.12222
			B	0.12568	0.29836	0.09836	0.65	0.11348	0.10341
			C	0.12568	0.23999	0.03999	0.65	0.07236	0.06140
			D	0.12568	0.20549	0.00549	0.65	0.02681	0.03709
		S3	A	0.12568	0.23840	0.03840	0.65	0.07091	0.06963
			B	0.12568	0.22340	0.02340	0.65	0.05535	0.05596
			C	0.12568	0.20650	0.00650	0.65	0.02917	0.02920
			D	0.12568	0.21142	0.01142	0.65	0.03867	0.03945
	Event 3	S2	C	0.12568	0.20230	0.00230	0.65	0.01735	0.01733

Type of Orifice	Storm Event	Scenario	Location of Orifice	A (m^2)	Water Level at Upstream (m)	h (m)	C	Q (Orifice Formula) (m^3/s)	Q (SWMM Orifice) (m^3/s)
0.4m Ø Circular Orifice	Event 3	S2	D	0.12568	0.20513	0.00513	0.65	0.02591	0.01170
		S3	B	0.12568	0.20541	0.00541	0.65	0.02661	0.02150
			C	0.12568	0.20300	0.00300	0.65	0.01982	0.01745
			D	0.12568	0.20270	0.00270	0.65	0.01880	0.01033
	Event 3	S1	A	0.12568	0.21960	0.01960	0.65	0.05066	0.05021
			B	0.12568	0.20940	0.00940	0.65	0.03508	0.03410
			C	0.12568	0.20534	0.00534	0.65	0.02644	0.02605
			D	0.12568	0.20314	0.00314	0.65	0.02028	0.01739
		S2	A	0.12568	0.22120	0.02120	0.65	0.05269	0.05006
			B	0.12568	0.20980	0.00980	0.65	0.03582	0.03400
			C	0.12568	0.20530	0.00530	0.65	0.02634	0.02570
			D	0.12568	0.20213	0.00213	0.65	0.01670	0.01736
		S3	A	0.12568	0.23579	0.03579	0.65	0.06846	0.04958

Type of Orifice	Storm Event	Scenario	Location of Orifice	A (m^2)	Water Level at Upstream (m)	h (m)	C	Q (Orifice Formula) (m^3/s)	Q (SWMM Orifice) (m^3/s)
0.4m Ø Circular Orifice	Event 3	S3	B	0.12568	0.20700	0.00700	0.65	0.03027	0.03172
			C	0.12568	0.20600	0.00600	0.65	0.02803	0.02573
			D	0.12568	0.20160	0.00160	0.65	0.01447	0.01527
0.5m Ø Circular Orifice	5-min 10-ARI	S1	A	0.19638	0.33748	0.08748	0.65	0.16723	0.17459
		S2	A	0.19638	0.25921	0.00921	0.65	0.05426	0.05183
		S3	A	0.19638	0.26907	0.01907	0.65	0.07808	0.07120
	Event 1	S1	A	0.19638	0.25975	0.00975	0.65	0.05583	0.05562
		S2	A	0.19638	0.26502	0.01502	0.65	0.06929	0.06358
		S3	A	0.19638	0.25529	0.00529	0.65	0.04112	0.04637
	Event 2	S1	A	0.19638	0.25427	0.00427	0.65	0.03695	0.03664
		S2	A	0.19638	0.25767	0.00767	0.65	0.04952	0.04869
		S3	A	0.19638	0.25047	0.00047	0.65	0.01226	0.01222

Type of Orifice	Storm Event	Scenario	Location of Orifice	A (m^2)	Water Level at Upstream (m)	h (m)	C	Q (Orifice Formula) (m^3/s)	Q (SWMM Orifice) (m^3/s)
0.5m Ø Circular Orifice	Event 3	S1	A	0.19638	0.26135	0.01135	0.65	0.06023	0.05908
		S2	A	0.19638	0.25040	0.00040	0.65	0.01131	0.01092
		S3	A	0.19638	0.25219	0.00219	0.65	0.02646	0.02629

Appendix H: Flow And Water Analysis

FLOW RATE FOR DESIGN RAINFALL AT LOCATION A (m^3/s)																
Time (min)	**Existing**	**SCENARIO 1**					**SCENARIO 2**					**SCENARIO 3**				
		0.1m	**0.2m**	**0.3m**	**0.4m**	**0.5m**	**0.1m**	**0.2m**	**0.3m**	**0.4m**	**0.5m**	**0.1m**	**0.2m**	**0.3m**	**0.4m**	**0.5m**
0000-0005	0.051	0.016	0.037	0.039	0.049	0.052	0.013	0.031	0.034	0.035	0.035	0.014	0.027	0.030	0.029	0.028
0005-0010	0.155	0.015	0.047	0.073	0.100	0.153	0.017	0.049	0.073	0.122	0.054	0.016	0.049	0.077	0.070	0.079
0010-0015	0.065	0.013	0.045	0.072	0.108	0.073	0.013	0.044	0.050	0.084	0.055	0.012	0.041	0.057	0.038	0.072
0015-0020	0.038	0.013	0.040	0.039	0.044	0.047	0.013	0.039	0.049	0.044	0.057	0.012	0.036	0.044	0.042	0.050
0020-0025	0.019	0.013	0.035	0.022	0.029	0.026	0.012	0.031	0.033	0.041	0.025	0.012	0.031	0.027	0.037	0.036
0025-0030	0.016	0.013	0.022	0.015	0.016	0.015	0.012	0.020	0.021	0.017	0.026	0.011	0.020	0.026	0.019	0.026
0030-0035	0.009	0.012	0.016	0.012	0.010	0.011	0.011	0.017	0.015	0.014	0.022	0.011	0.014	0.018	0.019	0.020
0035-0040	0.006	0.011	0.011	0.009	0.007	0.006	0.011	0.012	0.012	0.013	0.014	0.010	0.012	0.012	0.018	0.015
0040-0045	0.004	0.011	0.008	0.006	0.005	0.005	0.010	0.009	0.009	0.005	0.012	0.010	0.010	0.012	0.013	0.012
0045-0050	0.003	0.010	0.006	0.005	0.004	0.004	0.010	0.007	0.007	0.007	0.009	0.009	0.008	0.007	0.012	0.009
0050-0055	0.002	0.010	0.004	0.003	0.003	0.003	0.009	0.005	0.005	0.006	0.006	0.009	0.007	0.006	0.008	0.007
0055-0060	0.002	0.009	0.004	0.003	0.002	0.002	0.009	0.004	0.004	0.004	0.005	0.008	0.005	0.005	0.008	0.006

FLOW RATE FOR DESIGN RAINFALL AT LOCATION B (m^3/s)																
Time (min)	Existing	SCENARIO 1					SCENARIO 2					SCENARIO 3				
		0.1m	0.2m	0.3m	0.4m	0.5m	0.1m	0.2m	0.3m	0.4m	0.5m	0.1m	0.2m	0.3m	0.4m	0.5m
0000-0005	0.034	0.025	0.030	0.036	0.035	0.041	0.016	0.026	0.016	0.008	0.013	0.006	0.018	0.024	0.023	0.022
0005-0010	0.067	0.012	0.034	0.053	0.074	0.075	0.015	0.026	0.046	0.103	0.063	0.008	0.022	0.045	0.056	0.058
0010-0015	0.051	0.007	0.032	0.046	0.050	0.051	0.004	0.024	0.042	0.053	0.067	0.004	0.018	0.043	0.052	0.052
0015-0020	0.030	0.010	0.026	0.036	0.029	0.028	0.006	0.025	0.031	0.032	0.038	0.005	0.024	0.030	0.029	0.029
0020-0025	0.014	0.010	0.021	0.017	0.018	0.018	0.007	0.021	0.020	0.033	0.024	0.006	0.021	0.020	0.021	0.022
0025-0030	0.011	0.009	0.015	0.011	0.010	0.010	0.006	0.017	0.016	0.016	0.021	0.007	0.017	0.014	0.016	0.015
0030-0035	0.006	0.008	0.009	0.007	0.006	0.006	0.007	0.011	0.010	0.014	0.015	0.007	0.012	0.009	0.013	0.012
0035-0040	0.004	0.007	0.006	0.005	0.005	0.005	0.007	0.008	0.008	0.009	0.009	0.007	0.008	0.007	0.009	0.009
0040-0045	0.003	0.007	0.005	0.004	0.003	0.003	0.007	0.006	0.006	0.007	0.009	0.007	0.006	0.006	0.008	0.007
0045-0050	0.002	0.007	0.004	0.003	0.002	0.002	0.007	0.005	0.004	0.006	0.007	0.006	0.005	0.005	0.006	0.005
0050-0055	0.001	0.006	0.003	0.002	0.002	0.002	0.006	0.004	0.004	0.004	0.005	0.006	0.004	0.004	0.005	0.004
0055-0060	0.001	0.006	0.002	0.002	0.001	0.001	0.006	0.003	0.003	0.003	0.004	0.006	0.003	0.003	0.005	0.004

FLOW RATE FOR DESIGN RAINFALL AT LOCATION C (m^3/s)																
Time (min)	**Existing**	**SCENARIO 1**					**SCENARIO 2**					**SCENARIO 3**				
		0.1m	**0.2m**	**0.3m**	**0.4m**	**0.5m**	**0.1m**	**0.2m**	**0.3m**	**0.4m**	**0.5m**	**0.1m**	**0.2m**	**0.3m**	**0.4m**	**0.5m**
0000-0005	0.025	0.022	0.028	0.028	0.027	0.025	0.007	0.016	0.034	0.044	0.044	0.006	0.014	0.014	0.011	0.010
0005-0010	0.055	0.003	0.023	0.039	0.051	0.053	0.008	0.020	0.035	0.061	0.074	0.011	0.019	0.035	0.029	0.043
0010-0015	0.057	0.004	0.025	0.041	0.036	0.042	0.007	0.015	0.035	0.039	0.031	0.003	0.014	0.036	0.038	0.038
0015-0020	0.020	0.006	0.019	0.031	0.025	0.031	0.005	0.020	0.026	0.015	0.017	0.004	0.019	0.025	0.024	0.026
0020-0025	0.012	0.007	0.017	0.015	0.013	0.013	0.005	0.017	0.012	0.012	0.016	0.005	0.017	0.017	0.018	0.018
0025-0030	0.009	0.006	0.012	0.009	0.010	0.007	0.006	0.013	0.008	0.009	0.008	0.005	0.014	0.011	0.015	0.014
0030-0035	0.004	0.006	0.008	0.006	0.006	0.004	0.006	0.008	0.006	0.006	0.006	0.005	0.009	0.008	0.012	0.010
0035-0040	0.003	0.006	0.006	0.004	0.004	0.004	0.006	0.006	0.005	0.005	0.004	0.005	0.007	0.006	0.008	0.008
0040-0045	0.002	0.005	0.005	0.003	0.003	0.003	0.006	0.004	0.003	0.004	0.004	0.005	0.005	0.005	0.006	0.006
0045-0050	0.002	0.005	0.003	0.002	0.002	0.002	0.005	0.003	0.003	0.004	0.004	0.005	0.004	0.004	0.005	0.005
0050-0055	0.001	0.005	0.002	0.002	0.002	0.001	0.005	0.003	0.002	0.002	0.002	0.005	0.003	0.004	0.004	0.004
0055-0060	0.000	0.005	0.002	0.001	0.001	0.002	0.005	0.002	0.002	0.002	0.002	0.005	0.003	0.003	0.004	0.003

FLOW RATE FOR DESIGN RAINFALL AT LOCATION D (m^3/s)																
Time (min)	**Existing**	**SCENARIO 1**					**SCENARIO 2**					**SCENARIO 3**				
		0.1m	**0.2m**	**0.3m**	**0.4m**	**0.5m**	**0.1m**	**0.2m**	**0.3m**	**0.4m**	**0.5m**	**0.1m**	**0.2m**	**0.3m**	**0.4m**	**0.5m**
0000-0005	0.026	0.025	0.026	0.029	0.028	0.028	0.012	0.025	0.033	0.037	0.036	0.010	0.015	0.029	0.031	0.016
0005-0010	0.055	0.016	0.024	0.034	0.049	0.043	0.019	0.023	0.030	0.037	0.043	0.007	0.015	0.027	0.039	0.048
0010-0015	0.028	0.005	0.016	0.024	0.021	0.023	0.005	0.007	0.021	0.025	0.014	0.004	0.004	0.018	0.021	0.012
0015-0020	0.013	0.004	0.009	0.012	0.013	0.016	0.004	0.010	0.015	0.005	0.007	0.002	0.010	0.014	0.008	0.007
0020-0025	0.009	0.003	0.010	0.009	0.010	0.010	0.003	0.010	0.007	0.007	0.008	0.002	0.009	0.008	0.005	0.004
0025-0030	0.004	0.002	0.006	0.004	0.006	0.004	0.003	0.007	0.004	0.005	0.003	0.003	0.007	0.003	0.005	0.004
0030-0035	0.002	0.003	0.004	0.004	0.003	0.003	0.004	0.004	0.003	0.004	0.001	0.003	0.005	0.003	0.003	0.003
0035-0040	0.002	0.004	0.002	0.002	0.002	0.002	0.004	0.003	0.003	0.003	0.002	0.003	0.003	0.002	0.003	0.002
0040-0045	0.001	0.003	0.002	0.002	0.001	0.001	0.003	0.002	0.002	0.002	0.002	0.003	0.002	0.002	0.002	0.002
0045-0050	0.001	0.003	0.002	0.001	0.001	0.001	0.003	0.002	0.002	0.002	0.003	0.003	0.002	0.002	0.002	0.001
0050-0055	0.001	0.003	0.001	0.001	0.001	0.001	0.003	0.001	0.001	0.001	0.001	0.003	0.002	0.001	0.001	0.001
0055-0060	0.000	0.003	0.001	0.001	0.001	0.001	0.003	0.001	0.001	0.001	0.001	0.003	0.001	0.001	0.001	0.001

WATER LEVEL FOR DESIGN RAINFALL AT LOCATION A (m)																
Time (min)	Existing	SCENARIO 1					SCENARIO 2					SCENARIO 3				
		0.1m	0.2m	0.3m	0.4m	0.5m	0.1m	0.2m	0.3m	0.4m	0.5m	0.1m	0.2m	0.3m	0.4m	0.5m
0000-0005	0.153	0.241	0.179	0.152	0.132	0.117	0.234	0.170	0.135	0.117	0.104	0.226	0.160	0.124	0.103	0.091
0005-0010	0.315	0.500	0.500	0.460	0.399	0.344	0.500	0.500	0.420	0.370	0.299	0.500	0.500	0.366	0.280	0.260
0010-0015	0.281	0.497	0.485	0.381	0.353	0.329	0.497	0.463	0.367	0.341	0.322	0.494	0.444	0.363	0.322	0.296
0015-0020	0.177	0.470	0.367	0.270	0.241	0.233	0.449	0.343	0.261	0.244	0.233	0.434	0.321	0.254	0.248	0.235
0020-0025	0.118	0.437	0.265	0.195	0.173	0.164	0.409	0.251	0.203	0.196	0.186	0.386	0.244	0.199	0.197	0.191
0025-0030	0.095	0.405	0.195	0.147	0.130	0.121	0.373	0.195	0.159	0.157	0.151	0.347	0.194	0.171	0.169	0.157
0030-0035	0.068	0.372	0.159	0.115	0.101	0.097	0.342	0.164	0.129	0.127	0.127	0.315	0.163	0.139	0.143	0.132
0035-0040	0.056	0.342	0.127	0.092	0.083	0.077	0.315	0.135	0.109	0.104	0.104	0.288	0.140	0.118	0.120	0.112
0040-0045	0.046	0.314	0.105	0.077	0.067	0.063	0.288	0.113	0.091	0.090	0.093	0.264	0.121	0.102	0.106	0.096
0045-0050	0.038	0.286	0.088	0.065	0.056	0.053	0.264	0.097	0.079	0.079	0.083	0.243	0.105	0.090	0.092	0.083
0050-0055	0.033	0.260	0.075	0.055	0.048	0.045	0.242	0.084	0.069	0.067	0.070	0.224	0.092	0.080	0.079	0.073
0055-0060	0.028	0.235	0.065	0.047	0.041	0.039	0.221	0.074	0.060	0.058	0.060	0.206	0.082	0.069	0.075	0.065

WATER LEVEL FOR DESIGN RAINFALL AT LOCATION B (m)																
Time (min)	Existing	SCENARIO 1					SCENARIO 2					SCENARIO 3				
		0.1m	0.2m	0.3m	0.4m	0.5m	0.1m	0.2m	0.3m	0.4m	0.5m	0.1m	0.2m	0.3m	0.4m	0.5m
0000-0005	0.228	0.241	0.231	0.225	0.223	0.222	0.226	0.197	0.170	0.152	0.140	0.227	0.181	0.152	0.133	0.120
0005-0010	0.471	0.500	0.500	0.500	0.487	0.494	0.500	0.500	0.493	0.401	0.356	0.500	0.500	0.486	0.353	0.291
0010-0015	0.391	0.498	0.484	0.459	0.439	0.424	0.496	0.465	0.442	0.417	0.390	0.500	0.490	0.464	0.409	0.327
0015-0020	0.240	0.471	0.377	0.302	0.283	0.269	0.451	0.353	0.302	0.298	0.301	0.486	0.403	0.322	0.311	0.260
0020-0025	0.163	0.438	0.284	0.216	0.199	0.188	0.410	0.272	0.229	0.228	0.229	0.463	0.318	0.242	0.252	0.212
0025-0030	0.110	0.405	0.214	0.161	0.146	0.140	0.375	0.212	0.177	0.178	0.188	0.437	0.246	0.198	0.209	0.176
0030-0035	0.086	0.374	0.165	0.124	0.110	0.104	0.343	0.172	0.145	0.144	0.154	0.410	0.193	0.166	0.174	0.148
0035-0040	0.066	0.343	0.131	0.100	0.088	0.083	0.314	0.141	0.120	0.119	0.132	0.383	0.160	0.141	0.151	0.126
0040-0045	0.054	0.314	0.108	0.081	0.072	0.068	0.289	0.118	0.102	0.100	0.106	0.357	0.136	0.122	0.129	0.108
0045-0050	0.045	0.287	0.091	0.068	0.060	0.058	0.265	0.101	0.086	0.084	0.090	0.331	0.118	0.106	0.113	0.094
0050-0055	0.038	0.260	0.077	0.058	0.052	0.049	0.242	0.087	0.074	0.073	0.077	0.306	0.104	0.093	0.099	0.083
0055-0060	0.033	0.236	0.067	0.051	0.045	0.043	0.222	0.076	0.064	0.063	0.067	0.282	0.092	0.082	0.086	0.074

WATER LEVEL FOR DESIGN RAINFALL AT LOCATION C (m)																
Time (min)	Existing	SCENARIO 1					SCENARIO 2					SCENARIO 3				
		0.1m	0.2m	0.3m	0.4m	0.5m	0.1m	0.2m	0.3m	0.4m	0.5m	0.1m	0.2m	0.3m	0.4m	0.5m
0000-0005	0.239	0.240	0.239	0.235	0.233	0.234	0.251	0.222	0.185	0.167	0.152	0.231	0.185	0.156	0.144	0.133
0005-0010	0.497	0.500	0.500	0.500	0.500	0.500	0.500	0.500	0.500	0.438	0.385	0.500	0.500	0.487	0.369	0.317
0010-0015	0.413	0.500	0.486	0.463	0.446	0.430	0.500	0.496	0.474	0.441	0.414	0.498	0.492	0.465	0.418	0.340
0015-0020	0.255	0.472	0.379	0.306	0.288	0.271	0.489	0.406	0.324	0.312	0.317	0.486	0.404	0.323	0.311	0.269
0020-0025	0.168	0.438	0.285	0.215	0.200	0.193	0.467	0.311	0.236	0.240	0.245	0.463	0.318	0.245	0.253	0.217
0025-0030	0.115	0.405	0.214	0.162	0.148	0.139	0.440	0.235	0.186	0.188	0.198	0.437	0.246	0.201	0.211	0.178
0030-0035	0.089	0.374	0.165	0.124	0.112	0.107	0.411	0.181	0.150	0.152	0.161	0.410	0.193	0.166	0.175	0.150
0035-0040	0.067	0.343	0.132	0.100	0.088	0.085	0.381	0.148	0.124	0.125	0.133	0.383	0.161	0.141	0.151	0.127
0040-0045	0.055	0.314	0.108	0.082	0.072	0.070	0.352	0.124	0.104	0.105	0.112	0.357	0.137	0.122	0.129	0.109
0045-0050	0.046	0.287	0.091	0.069	0.061	0.058	0.326	0.106	0.089	0.088	0.093	0.331	0.119	0.106	0.113	0.095
0050-0055	0.039	0.260	0.078	0.059	0.052	0.050	0.298	0.091	0.077	0.075	0.079	0.306	0.104	0.093	0.099	0.084
0055-0060	0.022	0.236	0.068	0.051	0.046	0.043	0.273	0.080	0.067	0.065	0.068	0.283	0.092	0.083	0.087	0.075

WATER LEVEL FOR DESIGN RAINFALL AT LOCATION D (m)																
Time (min)	Existing	SCENARIO 1					SCENARIO 2					SCENARIO 3				
		0.1m	0.2m	0.3m	0.4m	0.5m	0.1m	0.2m	0.3m	0.4m	0.5m	0.1m	0.2m	0.3m	0.4m	0.5m
0000-0005	0.249	0.250	0.249	0.245	0.246	0.246	0.247	0.227	0.213	0.205	0.201	0.244	0.210	0.171	0.152	0.138
0005-0010	0.500	0.500	0.500	0.500	0.500	0.500	0.500	0.500	0.500	0.454	0.423	0.500	0.500	0.495	0.378	0.326
0010-0015	0.421	0.499	0.487	0.462	0.447	0.435	0.497	0.495	0.474	0.444	0.416	0.500	0.496	0.477	0.422	0.348
0015-0020	0.256	0.471	0.379	0.308	0.288	0.279	0.487	0.405	0.324	0.312	0.316	0.494	0.420	0.329	0.314	0.273
0020-0025	0.168	0.438	0.286	0.218	0.199	0.190	0.465	0.312	0.237	0.240	0.248	0.476	0.331	0.245	0.255	0.219
0025-0030	0.118	0.406	0.214	0.162	0.145	0.141	0.438	0.236	0.186	0.190	0.197	0.454	0.255	0.202	0.211	0.181
0030-0035	0.088	0.374	0.166	0.126	0.114	0.107	0.410	0.181	0.150	0.152	0.160	0.429	0.197	0.168	0.179	0.151
0035-0040	0.068	0.343	0.132	0.101	0.089	0.085	0.381	0.148	0.124	0.125	0.134	0.403	0.162	0.143	0.152	0.128
0040-0045	0.056	0.314	0.109	0.083	0.073	0.070	0.353	0.124	0.105	0.106	0.113	0.376	0.138	0.123	0.130	0.111
0045-0050	0.047	0.287	0.091	0.070	0.061	0.059	0.325	0.106	0.089	0.088	0.094	0.350	0.120	0.107	0.114	0.096
0050-0055	0.040	0.261	0.078	0.059	0.053	0.051	0.298	0.092	0.077	0.075	0.079	0.324	0.105	0.095	0.100	0.085
0055-0060	0.037	0.236	0.068	0.052	0.046	0.045	0.273	0.080	0.067	0.065	0.069	0.299	0.093	0.084	0.089	0.076

FLOW RATE FOR EVENT 1 AT LOCATION A (m^3/s)																
Time (min)	**Existing**	**SCENARIO 1**					**SCENARIO 2**					**SCENARIO 3**				
		0.1m	**0.2m**	**0.3m**	**0.4m**	**0.5m**	**0.1m**	**0.2m**	**0.3m**	**0.4m**	**0.5m**	**0.1m**	**0.2m**	**0.3m**	**0.4m**	**0.5m**
0000-0100	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0100-0200	0.001	0.000	0.000	0.000	0.000	0.006	0.001	0.000	0.000	0.000	0.000	0.001	0.001	0.001	0.000	0.000
0200-0300	0.003	0.002	0.003	0.003	0.002	0.003	0.003	0.002	0.002	0.002	0.002	0.003	0.003	0.003	0.003	0.004
0300-0400	0.002	0.002	0.002	0.001	0.002	0.002	0.002	0.002	0.002	0.001	0.001	0.003	0.001	0.005	0.001	0.005
0400-0500	0.001	0.003	0.006	0.003	0.002	0.001	0.002	0.001	0.001	0.001	0.001	0.001	0.001	0.003	0.001	0.002
0500-0600	0.017	0.011	0.016	0.016	0.017	0.016	0.011	0.016	0.016	0.016	0.016	0.010	0.003	0.016	0.016	0.016
0600-0700	0.015	0.013	0.015	0.015	0.016	0.015	0.012	0.017	0.015	0.015	0.015	0.012	0.003	0.015	0.015	0.015
0700-0800	0.006	0.010	0.006	0.006	0.006	0.006	0.010	0.007	0.006	0.006	0.006	0.010	0.006	0.007	0.007	0.007
0800-0900	0.004	0.006	0.004	0.004	0.004	0.004	0.006	0.004	0.004	0.004	0.004	0.007	0.004	0.004	0.004	0.004
0900-1000	0.005	0.005	0.005	0.005	0.005	0.005	0.005	0.005	0.005	0.005	0.005	0.006	0.005	0.005	0.005	0.005
1000-1100	0.003	0.003	0.003	0.003	0.003	0.003	0.003	0.003	0.003	0.003	0.003	0.004	0.004	0.003	0.003	0.003
1100-1200	0.028	0.014	0.028	0.028	0.028	0.028	0.014	0.024	0.028	0.028	0.027	0.014	0.028	0.027	0.027	0.027
1200-1300	0.031	0.014	0.030	0.031	0.031	0.031	0.014	0.028	0.031	0.031	0.031	0.014	0.034	0.031	0.031	0.031
1300-1400	0.019	0.015	0.019	0.019	0.019	0.019	0.014	0.020	0.019	0.019	0.019	0.013	0.010	0.019	0.019	0.019

FLOW RATE FOR EVENT 1 AT LOCATION B (m^3/s)																
Time (min)	**Existing**	**SCENARIO 1**					**SCENARIO 2**					**SCENARIO 3**				
		0.1m	**0.2m**	**0.3m**	**0.4m**	**0.5m**	**0.1m**	**0.2m**	**0.3m**	**0.4m**	**0.5m**	**0.1m**	**0.2m**	**0.3m**	**0.4m**	**0.5m**
0000-0100	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0100-0200	0.000	0.000	0.000	0.001	0.002	0.005	0.000	0.001	0.000	0.000	0.001	0.001	0.001	0.000	0.000	0.000
0200-0300	0.002	0.001	0.002	0.002	0.002	0.002	0.002	0.001	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002
0300-0400	0.004	0.002	0.001	0.001	0.001	0.001	0.001	0.001	0.000	0.001	0.001	0.002	0.001	0.003	0.001	0.004
0400-0500	0.000	0.001	0.009	0.002	0.001	0.000	0.002	0.001	0.001	0.001	0.001	0.002	0.000	0.002	0.004	0.006
0500-0600	0.011	0.008	0.011	0.011	0.011	0.011	0.007	0.011	0.011	0.011	0.011	0.006	0.015	0.010	0.010	0.010
0600-0700	0.010	0.009	0.010	0.010	0.009	0.010	0.008	0.010	0.011	0.010	0.010	0.007	0.017	0.010	0.010	0.010
0700-0800	0.004	0.006	0.004	0.004	0.004	0.004	0.007	0.004	0.005	0.004	0.004	0.006	0.002	0.004	0.004	0.004
0800-0900	0.002	0.004	0.003	0.002	0.002	0.002	0.004	0.003	0.003	0.003	0.003	0.005	0.002	0.002	0.002	0.002
0900-1000	0.003	0.003	0.003	0.003	0.003	0.003	0.004	0.003	0.003	0.003	0.003	0.004	0.003	0.003	0.003	0.003
1000-1100	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002
1100-1200	0.019	0.008	0.019	0.019	0.019	0.019	0.006	0.018	0.020	0.019	0.019	0.005	0.000	0.017	0.017	0.017
1200-1300	0.021	0.005	0.021	0.021	0.021	0.021	0.005	0.021	0.022	0.021	0.021	0.003	0.021	0.020	0.020	0.020
1300-1400	0.013	0.009	0.013	0.013	0.013	0.013	0.008	0.013	0.013	0.013	0.013	0.006	0.020	0.012	0.012	0.012

FLOW RATE FOR EVENT 1 AT LOCATION C (m^3/s)																
Time (min)	**Existing**	**SCENARIO 1**					**SCENARIO 2**					**SCENARIO 3**				
		0.1m	**0.2m**	**0.3m**	**0.4m**	**0.5m**	**0.1m**	**0.2m**	**0.3m**	**0.4m**	**0.5m**	**0.1m**	**0.2m**	**0.3m**	**0.4m**	**0.5m**
0000-0100	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0100-0200	0.001	0.001	0.001	0.001	0.002	0.004	0.001	0.002	0.001	0.002	0.001	0.021	0.001	0.003	0.001	0.000
0200-0300	0.001	0.000	0.002	0.001	0.000	0.001	0.001	0.002	0.001	0.001	0.001	0.001	0.001	0.002	0.001	0.002
0300-0400	0.003	0.003	0.000	0.000	0.001	0.000	0.000	0.001	0.001	0.003	0.003	0.002	0.001	0.003	0.000	0.003
0400-0500	0.000	0.000	0.007	0.001	0.001	0.000	0.001	0.003	0.002	0.001	0.001	0.003	0.000	0.002	0.014	0.005
0500-0600	0.008	0.006	0.008	0.008	0.009	0.008	0.005	0.008	0.008	0.008	0.008	0.005	0.008	0.008	0.008	0.008
0600-0700	0.008	0.007	0.008	0.008	0.007	0.008	0.006	0.008	0.008	0.008	0.008	0.006	0.008	0.008	0.008	0.008
0700-0800	0.003	0.005	0.003	0.003	0.003	0.003	0.005	0.003	0.003	0.003	0.003	0.005	0.003	0.003	0.003	0.003
0800-0900	0.002	0.003	0.002	0.002	0.002	0.002	0.003	0.002	0.002	0.002	0.002	0.004	0.002	0.002	0.002	0.002
0900-1000	0.002	0.002	0.002	0.002	0.002	0.002	0.003	0.002	0.002	0.002	0.002	0.003	0.002	0.002	0.002	0.002
1000-1100	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002
1100-1200	0.014	0.006	0.014	0.014	0.014	0.014	0.005	0.014	0.014	0.014	0.014	0.004	0.017	0.014	0.014	0.014
1200-1300	0.016	0.003	0.016	0.016	0.016	0.016	0.003	0.015	0.016	0.016	0.016	0.003	0.011	0.016	0.016	0.016
1300-1400	0.010	0.006	0.010	0.010	0.010	0.010	0.005	0.009	0.010	0.010	0.010	0.004	0.010	0.010	0.010	0.010

FLOW RATE FOR EVENT 1 AT LOCATION D (m^3/s)																
Time (min)	**Existing**	**SCENARIO 1**					**SCENARIO 2**					**SCENARIO 3**				
		0.1m	**0.2m**	**0.3m**	**0.4m**	**0.5m**	**0.1m**	**0.2m**	**0.3m**	**0.4m**	**0.5m**	**0.1m**	**0.2m**	**0.3m**	**0.4m**	**0.5m**
0000-0100	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0100-0200	0.001	0.000	0.001	0.000	0.002	0.003	0.000	0.000	0.000	0.000	0.000	0.003	0.000	0.000	0.000	0.000
0200-0300	0.001	0.001	0.001	0.001	0.001	0.001	0.001	0.001	0.001	0.001	0.001	0.000	0.001	0.001	0.001	0.001
0300-0400	0.002	0.003	0.000	0.001	0.001	0.001	0.002	0.000	0.003	0.001	0.002	0.001	0.000	0.001	0.002	0.002
0400-0500	0.003	0.001	0.004	0.001	0.001	0.003	0.001	0.004	0.002	0.000	0.002	0.000	0.000	0.001	0.017	0.003
0500-0600	0.006	0.004	0.006	0.006	0.006	0.006	0.004	0.006	0.006	0.006	0.006	0.003	0.005	0.005	0.005	0.005
0600-0700	0.005	0.005	0.005	0.005	0.005	0.005	0.004	0.005	0.005	0.005	0.005	0.004	0.004	0.005	0.005	0.005
0700-0800	0.002	0.003	0.002	0.002	0.002	0.002	0.003	0.002	0.002	0.002	0.002	0.003	0.002	0.002	0.002	0.002
0800-0900	0.001	0.002	0.001	0.001	0.001	0.001	0.002	0.001	0.001	0.001	0.001	0.002	0.001	0.001	0.001	0.001
0900-1000	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.001	0.001	0.001	0.001
1000-1100	0.001	0.001	0.001	0.001	0.001	0.001	0.001	0.001	0.001	0.001	0.001	0.001	0.001	0.001	0.001	0.001
1100-1200	0.010	0.005	0.010	0.010	0.010	0.010	0.005	0.010	0.010	0.010	0.010	0.003	0.006	0.008	0.008	0.008
1200-1300	0.011	0.004	0.011	0.011	0.011	0.011	0.004	0.010	0.011	0.011	0.011	0.002	0.013	0.009	0.009	0.009
1300-1400	0.006	0.003	0.006	0.006	0.006	0.006	0.002	0.006	0.006	0.006	0.006	0.002	0.006	0.006	0.006	0.006

WATER LEVEL FOR EVENT 1 AT LOCATION A (m)																
Time (min)	Existing	SCENARIO 1					SCENARIO 2					SCENARIO 3				
		0.1m	0.2m	0.3m	0.4m	0.5m	0.1m	0.2m	0.3m	0.4m	0.5m	0.1m	0.2m	0.3m	0.4m	0.5m
0000-0100	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0100-0200	0.015	0.019	0.016	0.015	0.015	0.041	0.020	0.017	0.015	0.014	0.014	0.017	0.024	0.014	0.014	0.012
0200-0300	0.034	0.068	0.055	0.046	0.039	0.038	0.076	0.048	0.041	0.038	0.037	0.071	0.044	0.051	0.040	0.048
0300-0400	0.030	0.058	0.042	0.028	0.037	0.032	0.055	0.042	0.030	0.027	0.028	0.082	0.024	0.068	0.031	0.053
0400-0500	0.026	0.088	0.054	0.047	0.034	0.026	0.069	0.031	0.025	0.034	0.026	0.058	0.018	0.042	0.029	0.030
0500-0600	0.093	0.316	0.159	0.132	0.115	0.110	0.290	0.156	0.130	0.116	0.108	0.270	0.149	0.128	0.116	0.107
0600-0700	0.089	0.406	0.151	0.125	0.113	0.104	0.382	0.152	0.125	0.112	0.104	0.360	0.132	0.125	0.112	0.104
0700-0800	0.055	0.256	0.088	0.073	0.066	0.062	0.254	0.090	0.074	0.067	0.063	0.248	0.075	0.076	0.069	0.065
0800-0900	0.042	0.128	0.063	0.053	0.049	0.046	0.137	0.064	0.054	0.049	0.047	0.145	0.053	0.055	0.050	0.048
0900-1000	0.047	0.100	0.072	0.061	0.055	0.052	0.105	0.071	0.060	0.055	0.052	0.111	0.063	0.060	0.055	0.052
1000-1100	0.039	0.085	0.059	0.050	0.046	0.043	0.087	0.059	0.050	0.046	0.044	0.089	0.048	0.051	0.046	0.044
1100-1200	0.125	0.500	0.217	0.184	0.164	0.152	0.491	0.218	0.183	0.163	0.151	0.459	0.200	0.181	0.161	0.148
1200-1300	0.132	0.500	0.245	0.196	0.174	0.161	0.500	0.249	0.196	0.174	0.161	0.482	0.201	0.196	0.174	0.161
1300-1400	0.100	0.500	0.174	0.143	0.128	0.119	0.457	0.175	0.144	0.128	0.119	0.420	0.158	0.144	0.129	0.120

WATER LEVEL FOR EVENT 1 AT LOCATION B (m)																
Time (min)	Existing	SCENARIO 1					SCENARIO 2					SCENARIO 3				
		0.1m	0.2m	0.3m	0.4m	0.5m	0.1m	0.2m	0.3m	0.4m	0.5m	0.1m	0.2m	0.3m	0.4m	0.5m
0000-0100	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0100-0200	0.022	0.020	0.019	0.021	0.029	0.069	0.022	0.021	0.019	0.016	0.018	0.024	0.018	0.019	0.017	0.014
0200-0300	0.041	0.068	0.058	0.050	0.044	0.044	0.077	0.051	0.046	0.043	0.043	0.080	0.031	0.057	0.046	0.055
0300-0400	0.048	0.052	0.046	0.037	0.041	0.036	0.059	0.044	0.032	0.032	0.034	0.095	0.018	0.077	0.038	0.064
0400-0500	0.032	0.090	0.076	0.051	0.039	0.028	0.070	0.036	0.031	0.036	0.034	0.067	0.014	0.050	0.037	0.051
0500-0600	0.109	0.316	0.163	0.138	0.137	0.121	0.291	0.165	0.137	0.125	0.119	0.337	0.113	0.143	0.130	0.121
0600-0700	0.104	0.406	0.155	0.132	0.128	0.115	0.382	0.160	0.132	0.121	0.115	0.466	0.103	0.139	0.126	0.118
0700-0800	0.065	0.256	0.091	0.078	0.073	0.070	0.254	0.093	0.080	0.074	0.071	0.331	0.056	0.085	0.078	0.075
0800-0900	0.050	0.129	0.066	0.058	0.054	0.053	0.138	0.067	0.059	0.055	0.054	0.187	0.040	0.062	0.058	0.056
0900-1000	0.057	0.101	0.075	0.066	0.062	0.060	0.106	0.074	0.065	0.062	0.060	0.138	0.047	0.068	0.063	0.060
1000-1100	0.047	0.086	0.062	0.054	0.051	0.050	0.088	0.062	0.055	0.052	0.050	0.102	0.037	0.057	0.053	0.052
1100-1200	0.146	0.500	0.222	0.192	0.174	0.164	0.491	0.237	0.191	0.173	0.163	0.500	0.154	0.201	0.179	0.166
1200-1300	0.154	0.500	0.250	0.204	0.185	0.174	0.500	0.266	0.204	0.185	0.174	0.500	0.174	0.218	0.194	0.180
1300-1400	0.117	0.500	0.178	0.150	0.137	0.130	0.457	0.186	0.150	0.137	0.130	0.500	0.122	0.161	0.144	0.136

WATER LEVEL FOR EVENT 1 AT LOCATION C (m)																
Time (min)	Existing	SCENARIO 1					SCENARIO 2					SCENARIO 3				
		0.1m	0.2m	0.3m	0.4m	0.5m	0.1m	0.2m	0.3m	0.4m	0.5m	0.1m	0.2m	0.3m	0.4m	0.5m
0000-0100	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0100-0200	0.025	0.033	0.025	0.019	0.034	0.072	0.026	0.025	0.023	0.020	0.021	0.011	0.006	0.022	0.011	0.023
0200-0300	0.045	0.067	0.058	0.051	0.043	0.045	0.082	0.056	0.047	0.044	0.044	0.079	0.010	0.058	0.047	0.056
0300-0400	0.050	0.052	0.042	0.031	0.042	0.039	0.065	0.048	0.036	0.035	0.039	0.095	0.006	0.078	0.042	0.065
0400-0500	0.033	0.091	0.085	0.052	0.040	0.040	0.075	0.048	0.035	0.037	0.035	0.069	0.005	0.050	0.072	0.057
0500-0600	0.112	0.316	0.164	0.139	0.139	0.122	0.341	0.173	0.143	0.130	0.123	0.338	0.034	0.144	0.131	0.122
0600-0700	0.106	0.406	0.156	0.133	0.129	0.116	0.458	0.168	0.137	0.125	0.118	0.466	0.032	0.140	0.127	0.120
0700-0800	0.067	0.256	0.092	0.079	0.074	0.071	0.307	0.098	0.083	0.076	0.073	0.331	0.017	0.086	0.079	0.076
0800-0900	0.051	0.129	0.067	0.059	0.055	0.054	0.162	0.070	0.061	0.057	0.055	0.187	0.012	0.063	0.059	0.057
0900-1000	0.058	0.101	0.075	0.066	0.063	0.061	0.121	0.078	0.068	0.063	0.061	0.138	0.015	0.069	0.064	0.061
1000-1100	0.048	0.086	0.062	0.055	0.052	0.051	0.094	0.065	0.057	0.053	0.052	0.102	0.012	0.058	0.054	0.052
1100-1200	0.148	0.500	0.222	0.193	0.176	0.166	0.500	0.261	0.200	0.180	0.169	0.500	0.056	0.203	0.181	0.168
1200-1300	0.157	0.500	0.251	0.205	0.187	0.176	0.500	0.296	0.214	0.192	0.180	0.500	0.067	0.219	0.195	0.182
1300-1400	0.119	0.500	0.179	0.151	0.138	0.132	0.500	0.195	0.157	0.142	0.134	0.500	0.039	0.162	0.146	0.137

WATER LEVEL FOR EVENT 1 AT LOCATION D (m)																
Time (min)	Existing	SCENARIO 1					SCENARIO 2					SCENARIO 3				
		0.1m	0.2m	0.3m	0.4m	0.5m	0.1m	0.2m	0.3m	0.4m	0.5m	0.1m	0.2m	0.3m	0.4m	0.5m
0000-0100	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0100-0200	0.031	0.032	0.043	0.028	0.039	0.074	0.024	0.023	0.023	0.023	0.019	0.039	0.011	0.023	0.022	0.025
0200-0300	0.045	0.067	0.059	0.051	0.047	0.045	0.082	0.056	0.049	0.045	0.044	0.082	0.019	0.059	0.048	0.057
0300-0400	0.056	0.071	0.048	0.047	0.044	0.039	0.072	0.049	0.051	0.048	0.043	0.097	0.011	0.078	0.042	0.067
0400-0500	0.045	0.091	0.091	0.052	0.040	0.056	0.075	0.055	0.040	0.036	0.048	0.069	0.008	0.051	0.099	0.070
0500-0600	0.113	0.316	0.164	0.140	0.138	0.124	0.341	0.173	0.143	0.131	0.124	0.353	0.067	0.146	0.133	0.124
0600-0700	0.108	0.406	0.156	0.134	0.130	0.118	0.458	0.168	0.138	0.126	0.120	0.491	0.062	0.142	0.129	0.121
0700-0800	0.068	0.257	0.092	0.080	0.075	0.072	0.307	0.098	0.083	0.077	0.074	0.350	0.034	0.087	0.080	0.077
0800-0900	0.052	0.129	0.067	0.059	0.056	0.055	0.162	0.071	0.061	0.058	0.056	0.197	0.024	0.064	0.060	0.058
0900-1000	0.059	0.101	0.076	0.067	0.064	0.062	0.121	0.078	0.068	0.064	0.062	0.144	0.029	0.070	0.065	0.062
1000-1100	0.049	0.086	0.063	0.056	0.053	0.051	0.094	0.065	0.057	0.054	0.052	0.104	0.022	0.059	0.055	0.053
1100-1200	0.151	0.500	0.223	0.194	0.177	0.168	0.500	0.261	0.201	0.181	0.170	0.500	0.096	0.205	0.183	0.171
1200-1300	0.159	0.500	0.251	0.207	0.188	0.178	0.500	0.296	0.215	0.193	0.181	0.500	0.098	0.222	0.198	0.184
1300-1400	0.121	0.500	0.179	0.152	0.140	0.133	0.500	0.196	0.158	0.143	0.136	0.500	0.072	0.164	0.148	0.139

FLOW RATE FOR EVENT 2 AT LOCATION A (m^3/s)																
Time (min)	**Existing**	**SCENARIO 1**					**SCENARIO 2**					**SCENARIO 3**				
		0.1m	**0.2m**	**0.3m**	**0.4m**	**0.5m**	**0.1m**	**0.2m**	**0.3m**	**0.4m**	**0.5m**	**0.1m**	**0.2m**	**0.3m**	**0.4m**	**0.5m**
0000-0100	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0100-0200	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0200-0300	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0300-0400	0.001	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0400-0500	0.005	0.005	0.005	0.005	0.005	0.005	0.004	0.005	0.005	0.005	0.005	0.005	0.005	0.005	0.005	0.006
0500-0600	0.004	0.004	0.004	0.004	0.004	0.004	0.004	0.004	0.004	0.004	0.004	0.004	0.004	0.004	0.004	0.004
0600-0700	0.011	0.008	0.011	0.011	0.011	0.011	0.008	0.010	0.010	0.010	0.010	0.008	0.012	0.010	0.010	0.010
0700-0800	0.007	0.008	0.007	0.007	0.007	0.007	0.008	0.007	0.007	0.007	0.007	0.008	0.007	0.007	0.007	0.007
0800-0900	0.007	0.007	0.006	0.006	0.006	0.006	0.007	0.006	0.006	0.006	0.006	0.007	0.006	0.006	0.006	0.006
0900-1000	0.007	0.007	0.007	0.007	0.007	0.007	0.007	0.007	0.007	0.007	0.007	0.007	0.007	0.007	0.007	0.007
1000-1100	0.014	0.011	0.014	0.014	0.014	0.014	0.010	0.014	0.014	0.014	0.014	0.010	0.019	0.014	0.014	0.014
1100-1200	0.023	0.014	0.023	0.023	0.023	0.023	0.014	0.023	0.023	0.023	0.023	0.013	0.029	0.023	0.023	0.022
1200-1300	0.035	0.014	0.033	0.034	0.034	0.034	0.014	0.031	0.034	0.034	0.034	0.014	0.043	0.034	0.034	0.034
1300-1400	0.012	0.013	0.013	0.012	0.011	0.011	0.013	0.013	0.012	0.012	0.012	0.012	0.009	0.012	0.012	0.012
1400-1500	0.003	0.008	0.003	0.003	0.003	0.003	0.008	0.004	0.003	0.003	0.003	0.008	0.003	0.004	0.004	0.004

FLOW RATE FOR EVENT 2 AT LOCATION B (m^3/s)																
Time (min)	Existing	SCENARIO 1					SCENARIO 2					SCENARIO 3				
		0.1m	0.2m	0.3m	0.4m	0.5m	0.1m	0.2m	0.3m	0.4m	0.5m	0.1m	0.2m	0.3m	0.4m	0.5m
0000-0100	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0100-0200	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0200-0300	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0300-0400	0.001	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.002	0.000	0.000	0.000	0.000
0400-0500	0.004	0.004	0.003	0.003	0.004	0.004	0.003	0.003	0.003	0.003	0.003	0.003	0.003	0.003	0.003	0.004
0500-0600	0.003	0.003	0.003	0.003	0.003	0.003	0.003	0.003	0.003	0.003	0.003	0.003	0.003	0.003	0.003	0.003
0600-0700	0.007	0.006	0.007	0.007	0.007	0.007	0.006	0.007	0.007	0.007	0.007	0.005	0.007	0.007	0.007	0.007
0700-0800	0.005	0.005	0.005	0.005	0.005	0.005	0.005	0.005	0.005	0.005	0.005	0.005	0.005	0.005	0.005	0.005
0800-0900	0.004	0.005	0.004	0.004	0.004	0.004	0.005	0.004	0.004	0.004	0.004	0.005	0.004	0.004	0.004	0.004
0900-1000	0.005	0.005	0.005	0.005	0.005	0.005	0.005	0.005	0.005	0.005	0.005	0.004	0.005	0.005	0.005	0.005
1000-1100	0.009	0.007	0.009	0.009	0.009	0.009	0.007	0.009	0.009	0.009	0.009	0.006	0.011	0.009	0.009	0.009
1100-1200	0.015	0.007	0.015	0.015	0.015	0.015	0.007	0.015	0.015	0.015	0.015	0.005	0.014	0.014	0.014	0.014
1200-1300	0.023	0.005	0.023	0.023	0.023	0.023	0.004	0.022	0.023	0.023	0.023	0.003	0.016	0.022	0.022	0.021
1300-1400	0.008	0.009	0.008	0.008	0.008	0.008	0.009	0.008	0.008	0.008	0.008	0.008	0.007	0.008	0.008	0.008
1400-1500	0.002	0.005	0.002	0.002	0.002	0.002	0.006	0.002	0.002	0.002	0.002	0.006	0.002	0.002	0.002	0.002

FLOW RATE FOR EVENT 2 AT LOCATION C (m^3/s)																
Time (min)	Existing	SCENARIO 1					SCENARIO 2					SCENARIO 3				
		0.1m	0.2m	0.3m	0.4m	0.5m	0.1m	0.2m	0.3m	0.4m	0.5m	0.1m	0.2m	0.3m	0.4m	0.5m
0000-0100	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0100-0200	0.001	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0200-0300	0.000	0.002	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.001
0300-0400	0.000	0.001	0.000	0.000	0.001	0.000	0.000	0.000	0.000	0.000	0.000	0.002	0.000	0.001	0.000	0.000
0400-0500	0.003	0.003	0.003	0.003	0.003	0.003	0.002	0.002	0.003	0.003	0.003	0.002	0.003	0.002	0.003	0.003
0500-0600	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002
0600-0700	0.006	0.004	0.005	0.005	0.006	0.006	0.004	0.005	0.005	0.005	0.005	0.004	0.006	0.005	0.005	0.005
0700-0800	0.004	0.004	0.004	0.004	0.004	0.004	0.004	0.004	0.004	0.004	0.004	0.004	0.004	0.004	0.004	0.004
0800-0900	0.003	0.004	0.003	0.003	0.003	0.003	0.004	0.003	0.003	0.003	0.003	0.004	0.003	0.003	0.003	0.003
0900-1000	0.004	0.004	0.004	0.004	0.004	0.004	0.004	0.004	0.004	0.004	0.004	0.004	0.004	0.004	0.004	0.004
1000-1100	0.007	0.006	0.007	0.007	0.007	0.007	0.005	0.007	0.007	0.007	0.007	0.005	0.007	0.007	0.007	0.007
1100-1200	0.012	0.006	0.012	0.012	0.012	0.012	0.004	0.011	0.012	0.012	0.012	0.003	0.012	0.012	0.012	0.012
1200-1300	0.018	0.003	0.017	0.018	0.018	0.018	0.003	0.017	0.017	0.017	0.017	0.002	0.026	0.017	0.017	0.017
1300-1400	0.006	0.007	0.006	0.006	0.006	0.006	0.007	0.006	0.006	0.006	0.006	0.006	0.006	0.006	0.006	0.006
1400-1500	0.002	0.004	0.002	0.002	0.002	0.002	0.004	0.002	0.002	0.002	0.002	0.005	0.001	0.002	0.002	0.002

FLOW RATE FOR EVENT 2 AT LOCATION D (m^3/s)																
Time (min)	Existing	SCENARIO 1					SCENARIO 2					SCENARIO 3				
		0.1m	0.2m	0.3m	0.4m	0.5m	0.1m	0.2m	0.3m	0.4m	0.5m	0.1m	0.2m	0.3m	0.4m	0.5m
0000-0100	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0100-0200	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0200-0300	0.001	0.001	0.001	0.001	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0300-0400	0.002	0.000	0.000	0.000	0.000	0.000	0.001	0.001	0.000	0.001	0.000	0.003	0.000	0.000	0.001	0.001
0400-0500	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.001	0.002	0.002	0.002	0.002
0500-0600	0.001	0.001	0.001	0.001	0.001	0.001	0.001	0.001	0.001	0.001	0.001	0.001	0.001	0.001	0.001	0.001
0600-0700	0.004	0.003	0.004	0.004	0.004	0.004	0.003	0.004	0.004	0.004	0.004	0.002	0.003	0.003	0.003	0.003
0700-0800	0.002	0.003	0.002	0.002	0.002	0.002	0.003	0.003	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002
0800-0900	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002
0900-1000	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002
1000-1100	0.005	0.004	0.005	0.005	0.005	0.005	0.004	0.005	0.005	0.005	0.005	0.003	0.004	0.004	0.004	0.004
1100-1200	0.008	0.005	0.008	0.008	0.008	0.008	0.004	0.008	0.008	0.008	0.008	0.003	0.005	0.007	0.007	0.007
1200-1300	0.012	0.004	0.012	0.012	0.012	0.012	0.003	0.011	0.012	0.012	0.012	0.002	0.009	0.010	0.010	0.010
1300-1400	0.004	0.004	0.005	0.004	0.004	0.004	0.004	0.004	0.004	0.004	0.004	0.004	0.003	0.004	0.004	0.004
1400-1500	0.001	0.003	0.001	0.001	0.001	0.001	0.003	0.001	0.001	0.001	0.001	0.003	0.001	0.001	0.001	0.001

WATER LEVEL FOR EVENT 2 AT LOCATION A (m)																
Time (min)	Existing	SCENARIO 1					SCENARIO 2					SCENARIO 3				
		0.1m	0.2m	0.3m	0.4m	0.5m	0.1m	0.2m	0.3m	0.4m	0.5m	0.1m	0.2m	0.3m	0.4m	0.5m
0000-0100	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0100-0200	0.006	0.007	0.006	0.006	0.006	0.005	0.007	0.006	0.006	0.005	0.005	0.007	0.011	0.005	0.005	0.005
0200-0300	0.010	0.012	0.011	0.010	0.010	0.010	0.010	0.010	0.010	0.009	0.010	0.012	0.011	0.009	0.009	0.008
0300-0400	0.018	0.015	0.013	0.013	0.012	0.012	0.014	0.013	0.013	0.011	0.011	0.015	0.011	0.012	0.011	0.011
0400-0500	0.051	0.103	0.076	0.065	0.059	0.057	0.098	0.073	0.063	0.058	0.055	0.100	0.069	0.062	0.057	0.060
0500-0600	0.046	0.099	0.069	0.058	0.053	0.051	0.099	0.069	0.059	0.053	0.051	0.099	0.059	0.059	0.054	0.051
0600-0700	0.074	0.204	0.121	0.101	0.091	0.085	0.196	0.120	0.100	0.090	0.084	0.189	0.109	0.099	0.089	0.083
0700-0800	0.059	0.179	0.095	0.080	0.072	0.068	0.177	0.096	0.080	0.072	0.068	0.173	0.083	0.081	0.073	0.069
0800-0900	0.056	0.153	0.089	0.074	0.067	0.063	0.154	0.089	0.074	0.067	0.064	0.154	0.078	0.075	0.068	0.064
0900-1000	0.059	0.156	0.094	0.079	0.071	0.067	0.156	0.094	0.079	0.071	0.067	0.156	0.083	0.079	0.071	0.067
1000-1100	0.085	0.295	0.143	0.119	0.107	0.100	0.285	0.143	0.118	0.106	0.099	0.275	0.131	0.117	0.105	0.098
1100-1200	0.112	0.500	0.198	0.163	0.145	0.134	0.478	0.197	0.162	0.144	0.134	0.443	0.194	0.162	0.144	0.133
1200-1300	0.140	0.500	0.276	0.209	0.185	0.171	0.500	0.272	0.209	0.185	0.171	0.492	0.286	0.209	0.185	0.170
1300-1400	0.076	0.430	0.130	0.106	0.095	0.089	0.393	0.133	0.108	0.097	0.091	0.358	0.117	0.110	0.099	0.093
1400-1500	0.038	0.197	0.059	0.049	0.044	0.042	0.196	0.062	0.051	0.046	0.044	0.191	0.045	0.054	0.049	0.047

WATER LEVEL FOR EVENT 2 AT LOCATION B (m)																
Time (min)	Existing	SCENARIO 1					SCENARIO 2					SCENARIO 3				
		0.1m	0.2m	0.3m	0.4m	0.5m	0.1m	0.2m	0.3m	0.4m	0.5m	0.1m	0.2m	0.3m	0.4m	0.5m
0000-0100	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0100-0200	0.007	0.008	0.007	0.008	0.008	0.008	0.008	0.007	0.007	0.007	0.007	0.008	0.009	0.007	0.007	0.007
0200-0300	0.014	0.016	0.014	0.013	0.012	0.013	0.012	0.011	0.015	0.013	0.013	0.015	0.009	0.012	0.011	0.011
0300-0400	0.026	0.022	0.016	0.017	0.016	0.015	0.019	0.014	0.015	0.018	0.014	0.028	0.009	0.016	0.015	0.014
0400-0500	0.061	0.104	0.079	0.070	0.066	0.065	0.099	0.076	0.068	0.064	0.062	0.118	0.051	0.070	0.065	0.069
0500-0600	0.055	0.100	0.072	0.063	0.060	0.058	0.100	0.072	0.063	0.060	0.058	0.115	0.045	0.066	0.061	0.060
0600-0700	0.087	0.205	0.124	0.107	0.099	0.095	0.197	0.123	0.106	0.098	0.094	0.236	0.082	0.110	0.100	0.095
0700-0800	0.071	0.180	0.099	0.085	0.079	0.076	0.177	0.099	0.086	0.080	0.076	0.221	0.063	0.090	0.083	0.079
0800-0900	0.067	0.153	0.092	0.080	0.074	0.072	0.154	0.092	0.080	0.074	0.072	0.194	0.058	0.084	0.077	0.074
0900-1000	0.071	0.157	0.098	0.084	0.079	0.076	0.157	0.097	0.084	0.079	0.076	0.196	0.063	0.088	0.081	0.077
1000-1100	0.100	0.296	0.147	0.125	0.115	0.110	0.285	0.146	0.125	0.115	0.109	0.350	0.100	0.131	0.118	0.111
1100-1200	0.131	0.500	0.202	0.170	0.155	0.146	0.478	0.211	0.170	0.154	0.146	0.500	0.129	0.180	0.160	0.150
1200-1300	0.162	0.500	0.279	0.217	0.196	0.185	0.500	0.291	0.217	0.196	0.185	0.500	0.164	0.231	0.205	0.191
1300-1400	0.090	0.430	0.137	0.113	0.104	0.099	0.393	0.139	0.114	0.105	0.100	0.471	0.085	0.123	0.112	0.106
1400-1500	0.045	0.198	0.062	0.053	0.050	0.048	0.197	0.065	0.055	0.052	0.051	0.256	0.034	0.061	0.056	0.055

WATER LEVEL FOR EVENT 2 AT LOCATION C (m)																
Time (min)	Existing	SCENARIO 1					SCENARIO 2					SCENARIO 3				
		0.1m	0.2m	0.3m	0.4m	0.5m	0.1m	0.2m	0.3m	0.4m	0.5m	0.1m	0.2m	0.3m	0.4m	0.5m
0000-0100	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0100-0200	0.007	0.007	0.006	0.006	0.007	0.006	0.008	0.008	0.007	0.008	0.007	0.008	0.003	0.007	0.007	0.005
0200-0300	0.017	0.014	0.014	0.015	0.014	0.013	0.016	0.015	0.016	0.014	0.014	0.019	0.003	0.010	0.012	0.009
0300-0400	0.025	0.023	0.014	0.019	0.019	0.021	0.017	0.016	0.019	0.018	0.019	0.033	0.003	0.013	0.018	0.017
0400-0500	0.062	0.104	0.080	0.071	0.067	0.066	0.108	0.080	0.070	0.066	0.064	0.119	0.016	0.071	0.066	0.070
0500-0600	0.056	0.100	0.073	0.064	0.060	0.059	0.109	0.076	0.066	0.062	0.059	0.115	0.014	0.067	0.062	0.061
0600-0700	0.089	0.205	0.125	0.108	0.100	0.096	0.229	0.130	0.110	0.101	0.096	0.236	0.025	0.111	0.101	0.096
0700-0800	0.072	0.180	0.099	0.086	0.080	0.077	0.208	0.104	0.089	0.082	0.079	0.221	0.019	0.091	0.084	0.080
0800-0900	0.068	0.154	0.093	0.081	0.075	0.073	0.180	0.097	0.083	0.077	0.074	0.195	0.018	0.085	0.078	0.075
0900-1000	0.072	0.157	0.098	0.085	0.080	0.077	0.182	0.102	0.088	0.081	0.078	0.197	0.019	0.089	0.082	0.078
1000-1100	0.102	0.296	0.148	0.126	0.117	0.111	0.337	0.154	0.130	0.119	0.113	0.350	0.030	0.132	0.119	0.113
1100-1200	0.133	0.500	0.203	0.171	0.156	0.148	0.500	0.228	0.177	0.160	0.151	0.500	0.047	0.181	0.162	0.152
1200-1300	0.166	0.500	0.280	0.219	0.198	0.187	0.500	0.327	0.227	0.204	0.191	0.500	0.093	0.233	0.207	0.193
1300-1400	0.092	0.430	0.138	0.114	0.105	0.100	0.477	0.146	0.119	0.109	0.103	0.471	0.026	0.124	0.113	0.107
1400-1500	0.046	0.198	0.062	0.054	0.051	0.049	0.238	0.068	0.057	0.054	0.052	0.256	0.011	0.061	0.057	0.056

WATER LEVEL FOR EVENT 2 AT LOCATION D (m)																
Time (min)	Existing	SCENARIO 1					SCENARIO 2					SCENARIO 3				
		0.1m	0.2m	0.3m	0.4m	0.5m	0.1m	0.2m	0.3m	0.4m	0.5m	0.1m	0.2m	0.3m	0.4m	0.5m
0000-0100	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0100-0200	0.008	0.009	0.009	0.009	0.009	0.009	0.008	0.008	0.008	0.008	0.008	0.008	0.005	0.008	0.008	0.007
0200-0300	0.014	0.018	0.016	0.018	0.016	0.017	0.017	0.018	0.018	0.012	0.013	0.018	0.005	0.015	0.016	0.015
0300-0400	0.034	0.027	0.027	0.022	0.023	0.025	0.025	0.021	0.022	0.023	0.017	0.072	0.005	0.020	0.020	0.018
0400-0500	0.063	0.105	0.080	0.071	0.068	0.067	0.109	0.080	0.071	0.067	0.065	0.123	0.032	0.072	0.067	0.071
0500-0600	0.057	0.100	0.073	0.065	0.061	0.060	0.109	0.076	0.066	0.062	0.060	0.119	0.027	0.068	0.063	0.062
0600-0700	0.091	0.205	0.125	0.109	0.101	0.097	0.229	0.130	0.111	0.102	0.098	0.246	0.051	0.113	0.103	0.097
0700-0800	0.074	0.180	0.100	0.087	0.081	0.078	0.208	0.105	0.090	0.083	0.080	0.232	0.038	0.093	0.085	0.081
0800-0900	0.070	0.154	0.093	0.081	0.076	0.074	0.180	0.097	0.084	0.078	0.075	0.204	0.036	0.086	0.079	0.076
0900-1000	0.073	0.157	0.099	0.086	0.081	0.078	0.182	0.103	0.088	0.082	0.079	0.206	0.038	0.090	0.083	0.079
1000-1100	0.104	0.296	0.148	0.127	0.118	0.113	0.337	0.155	0.131	0.120	0.114	0.367	0.059	0.134	0.121	0.114
1100-1200	0.135	0.500	0.203	0.172	0.158	0.150	0.500	0.228	0.178	0.161	0.152	0.500	0.082	0.183	0.164	0.154
1200-1300	0.168	0.500	0.281	0.220	0.200	0.189	0.500	0.328	0.228	0.205	0.192	0.500	0.107	0.236	0.210	0.195
1300-1400	0.094	0.430	0.137	0.114	0.106	0.102	0.477	0.146	0.120	0.110	0.104	0.497	0.053	0.125	0.114	0.109
1400-1500	0.047	0.198	0.063	0.054	0.051	0.050	0.238	0.068	0.058	0.054	0.053	0.270	0.021	0.062	0.058	0.057

FLOW RATE FOR EVENT 3 AT LOCATION A (m^3/s)																
Time (min)	**Existing**	**SCENARIO 1**					**SCENARIO 2**					**SCENARIO 3**				
		0.1m	**0.2m**	**0.3m**	**0.4m**	**0.5m**	**0.1m**	**0.2m**	**0.3m**	**0.4m**	**0.5m**	**0.1m**	**0.2m**	**0.3m**	**0.4m**	**0.5m**
0000-0100	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0100-0200	0.006	0.005	0.005	0.005	0.005	0.006	0.006	0.005	0.005	0.005	0.005	0.004	0.006	0.005	0.005	0.005
0200-0300	0.051	0.014	0.050	0.050	0.050	0.050	0.014	0.046	0.050	0.050	0.050	0.014	0.058	0.056	0.050	0.049
0300-0400	0.015	0.014	0.013	0.014	0.014	0.014	0.013	0.015	0.015	0.015	0.015	0.012	0.017	0.017	0.015	0.015
0400-0500	0.004	0.010	0.005	0.004	0.004	0.004	0.009	0.005	0.005	0.005	0.005	0.009	0.006	0.005	0.005	0.005
0500-0600	0.004	0.006	0.004	0.004	0.004	0.004	0.006	0.004	0.004	0.004	0.004	0.006	0.002	0.004	0.004	0.004
0600-0700	0.003	0.004	0.003	0.003	0.003	0.003	0.004	0.003	0.003	0.003	0.003	0.004	0.004	0.003	0.003	0.003
0700-0800	0.003	0.003	0.003	0.003	0.003	0.003	0.003	0.003	0.003	0.003	0.003	0.003	0.002	0.003	0.003	0.003
0800-0900	0.003	0.003	0.003	0.002	0.002	0.002	0.003	0.003	0.003	0.003	0.003	0.003	0.002	0.003	0.003	0.003
0900-1000	0.002	0.003	0.003	0.002	0.002	0.002	0.002	0.002	0.002	0.003	0.002	0.002	0.002	0.002	0.001	0.001
1000-1100	0.002	0.002	0.002	0.001	0.002	0.001	0.003	0.001	0.001	0.002	0.001	0.002	0.000	0.001	0.002	0.001
1100-1200	0.007	0.003	0.002	0.002	0.001	0.001	0.000	0.001	0.001	0.001	0.001	0.001	0.000	0.002	0.001	0.002
1200-1300	0.005	0.002	0.002	0.002	0.002	0.002	0.002	0.001	0.001	0.001	0.000	0.000	0.001	0.004	0.002	0.002
1300-1400	0.002	0.002	0.003	0.001	0.003	0.002	0.001	0.001	0.001	0.001	0.001	0.002	0.001	0.002	0.001	0.002
1400-1500	0.003	0.003	0.002	0.001	0.001	0.003	0.003	0.002	0.001	0.001	0.001	0.002	0.001	0.002	0.003	0.002

FLOW RATE FOR EVENT 3 AT LOCATION B (m^3/s)																
Time (min)	**Existing**	**SCENARIO 1**					**SCENARIO 2**					**SCENARIO 3**				
		0.1m	**0.2m**	**0.3m**	**0.4m**	**0.5m**	**0.1m**	**0.2m**	**0.3m**	**0.4m**	**0.5m**	**0.1m**	**0.2m**	**0.3m**	**0.4m**	**0.5m**
0000-0100	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0100-0200	0.004	0.004	0.004	0.004	0.004	0.004	0.004	0.003	0.003	0.003	0.004	0.003	0.003	0.003	0.003	0.003
0200-0300	0.034	0.007	0.033	0.034	0.034	0.034	0.005	0.031	0.034	0.034	0.034	0.004	0.064	0.030	0.032	0.032
0300-0400	0.010	0.010	0.011	0.010	0.010	0.010	0.009	0.010	0.010	0.010	0.010	0.007	0.014	0.010	0.010	0.010
0400-0500	0.003	0.007	0.003	0.003	0.003	0.003	0.006	0.003	0.003	0.003	0.003	0.006	0.003	0.003	0.003	0.003
0500-0600	0.003	0.004	0.003	0.003	0.003	0.003	0.004	0.003	0.003	0.003	0.003	0.004	0.002	0.003	0.003	0.003
0600-0700	0.002	0.002	0.002	0.002	0.002	0.002	0.003	0.002	0.002	0.002	0.002	0.003	0.002	0.002	0.002	0.002
0700-0800	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002
0800-0900	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002
0900-1000	0.001	0.002	0.002	0.001	0.001	0.001	0.000	0.001	0.003	0.002	0.001	0.003	0.001	0.002	0.001	0.002
1000-1100	0.001	0.000	0.001	0.001	0.007	0.004	0.002	0.001	0.001	0.001	0.000	0.003	0.000	0.019	0.004	0.001
1100-1200	0.005	0.001	0.001	0.001	0.001	0.001	0.002	0.001	0.000	0.001	0.001	0.002	0.000	0.002	0.004	0.001
1200-1300	0.004	0.001	0.001	0.002	0.001	0.001	0.002	0.001	0.001	0.001	0.001	0.001	0.000	0.004	0.002	0.001
1300-1400	0.001	0.000	0.002	0.000	0.002	0.001	0.000	0.003	0.001	0.001	0.001	0.000	0.001	0.001	0.000	0.001
1400-1500	0.002	0.002	0.002	0.001	0.002	0.002	0.002	0.002	0.001	0.001	0.001	0.000	0.001	0.001	0.003	0.000

FLOW RATE FOR EVENT 3 AT LOCATION C (m^3/s)																
Time (min)	Existing	SCENARIO 1					SCENARIO 2					SCENARIO 3				
		0.1m	0.2m	0.3m	0.4m	0.5m	0.1m	0.2m	0.3m	0.4m	0.5m	0.1m	0.2m	0.3m	0.4m	0.5m
0000-0100	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0100-0200	0.003	0.003	0.003	0.003	0.003	0.003	0.003	0.003	0.003	0.003	0.003	0.002	0.003	0.002	0.002	0.003
0200-0300	0.026	0.005	0.025	0.026	0.026	0.026	0.004	0.023	0.026	0.026	0.026	0.003	0.003	0.022	0.026	0.026
0300-0400	0.007	0.007	0.008	0.007	0.007	0.007	0.006	0.008	0.007	0.007	0.007	0.005	0.007	0.008	0.008	0.008
0400-0500	0.002	0.005	0.002	0.002	0.002	0.002	0.005	0.002	0.002	0.002	0.002	0.005	0.002	0.003	0.003	0.003
0500-0600	0.002	0.003	0.002	0.002	0.002	0.002	0.003	0.002	0.002	0.002	0.002	0.003	0.002	0.002	0.002	0.002
0600-0700	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002
0700-0800	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002
0800-0900	0.001	0.001	0.001	0.001	0.002	0.001	0.001	0.001	0.001	0.001	0.001	0.002	0.001	0.001	0.001	0.001
0900-1000	0.001	0.004	0.001	0.002	0.001	0.001	0.000	0.001	0.002	0.001	0.001	0.006	0.001	0.002	0.000	0.000
1000-1100	0.001	0.009	0.000	0.000	0.010	0.006	0.002	0.000	0.000	0.000	0.000	0.003	0.000	0.016	0.004	0.002
1100-1200	0.006	0.000	0.000	0.000	0.000	0.003	0.000	0.001	0.000	0.000	0.000	0.002	0.000	0.006	0.007	0.001
1200-1300	0.004	0.000	0.001	0.002	0.001	0.004	0.004	0.001	0.001	0.000	0.000	0.008	0.000	0.004	0.001	0.002
1300-1400	0.001	0.000	0.001	0.002	0.002	0.000	0.002	0.003	0.002	0.001	0.000	0.000	0.001	0.002	0.000	0.000
1400-1500	0.001	0.001	0.001	0.001	0.004	0.002	0.002	0.001	0.003	0.001	0.001	0.002	0.001	0.000	0.003	0.000

FLOW RATE FOR EVENT 3 AT LOCATION D (m^3/s)																
Time (min)	**Existing**	**SCENARIO 1**					**SCENARIO 2**					**SCENARIO 3**				
		0.1m	**0.2m**	**0.3m**	**0.4m**	**0.5m**	**0.1m**	**0.2m**	**0.3m**	**0.4m**	**0.5m**	**0.1m**	**0.2m**	**0.3m**	**0.4m**	**0.5m**
0000-0100	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0100-0200	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.002	0.001	0.002	0.002	0.002	0.002
0200-0300	0.017	0.005	0.016	0.017	0.017	0.017	0.005	0.014	0.017	0.017	0.017	0.003	0.009	0.014	0.015	0.015
0300-0400	0.005	0.005	0.005	0.005	0.005	0.005	0.004	0.005	0.005	0.005	0.005	0.002	0.006	0.005	0.005	0.005
0400-0500	0.001	0.003	0.002	0.002	0.002	0.002	0.003	0.002	0.002	0.002	0.002	0.003	0.001	0.001	0.001	0.001
0500-0600	0.001	0.002	0.001	0.001	0.001	0.001	0.002	0.001	0.001	0.001	0.001	0.002	0.001	0.001	0.001	0.001
0600-0700	0.001	0.001	0.001	0.001	0.001	0.001	0.001	0.001	0.001	0.001	0.001	0.001	0.001	0.001	0.001	0.001
0700-0800	0.001	0.001	0.001	0.001	0.001	0.001	0.001	0.001	0.001	0.001	0.001	0.001	0.001	0.001	0.001	0.001
0800-0900	0.001	0.001	0.001	0.001	0.001	0.001	0.001	0.001	0.001	0.001	0.001	0.001	0.001	0.001	0.001	0.001
0900-1000	0.001	0.000	0.001	0.000	0.001	0.002	0.001	0.001	0.001	0.001	0.001	0.001	0.001	0.001	0.001	0.000
1000-1100	0.001	0.011	0.001	0.001	0.010	0.004	0.001	0.007	0.004	0.000	0.000	0.001	0.000	0.008	0.002	0.001
1100-1200	0.004	0.003	0.008	0.000	0.000	0.050	0.005	0.003	0.001	0.001	0.000	0.002	0.000	0.001	0.004	0.000
1200-1300	0.002	0.000	0.001	0.001	0.001	0.001	0.006	0.000	0.002	0.000	0.000	0.006	0.000	0.001	0.000	0.000
1300-1400	0.001	0.000	0.001	0.002	0.001	0.001	0.001	0.003	0.001	0.000	0.000	0.001	0.000	0.001	0.001	0.001
1400-1500	0.001	0.001	0.001	0.002	0.015	0.001	0.001	0.001	0.003	0.001	0.000	0.000	0.000	0.018	0.001	0.000

WATER LEVEL FOR EVENT 3 AT LOCATION A (m)																
Time (min)	Existing	SCENARIO 1					SCENARIO 2					SCENARIO 3				
		0.1m	0.2m	0.3m	0.4m	0.5m	0.1m	0.2m	0.3m	0.4m	0.5m	0.1m	0.2m	0.3m	0.4m	0.5m
0000-0100	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0100-0200	0.052	0.107	0.078	0.066	0.061	0.058	0.125	0.074	0.064	0.059	0.056	0.097	0.072	0.061	0.056	0.055
0200-0300	0.176	0.500	0.448	0.269	0.238	0.219	0.500	0.433	0.269	0.237	0.219	0.500	0.401	0.260	0.236	0.217
0300-0400	0.086	0.494	0.151	0.122	0.109	0.102	0.425	0.152	0.124	0.111	0.103	0.386	0.126	0.132	0.113	0.106
0400-0500	0.046	0.264	0.073	0.060	0.055	0.052	0.239	0.075	0.062	0.056	0.054	0.224	0.061	0.066	0.059	0.056
0500-0600	0.044	0.136	0.066	0.055	0.051	0.048	0.136	0.066	0.056	0.051	0.048	0.138	0.057	0.056	0.051	0.049
0600-0700	0.039	0.088	0.058	0.049	0.045	0.043	0.091	0.058	0.050	0.046	0.043	0.095	0.050	0.050	0.046	0.044
0700-0800	0.039	0.083	0.058	0.049	0.045	0.043	0.084	0.058	0.049	0.045	0.043	0.085	0.049	0.049	0.045	0.043
0800-0900	0.034	0.072	0.050	0.042	0.039	0.037	0.073	0.050	0.043	0.040	0.038	0.073	0.042	0.043	0.040	0.038
0900-1000	0.032	0.054	0.052	0.038	0.037	0.033	0.060	0.045	0.035	0.036	0.031	0.062	0.033	0.034	0.032	0.030
1000-1100	0.032	0.037	0.048	0.032	0.028	0.023	0.073	0.027	0.033	0.032	0.024	0.067	0.011	0.028	0.027	0.030
1100-1200	0.052	0.054	0.034	0.037	0.027	0.026	0.048	0.026	0.024	0.023	0.018	0.053	0.011	0.030	0.028	0.033
1200-1300	0.042	0.058	0.049	0.037	0.028	0.031	0.043	0.028	0.024	0.022	0.025	0.047	0.018	0.051	0.033	0.035
1300-1400	0.030	0.052	0.053	0.031	0.041	0.034	0.058	0.033	0.029	0.029	0.027	0.062	0.024	0.040	0.032	0.031
1400-1500	0.035	0.078	0.050	0.031	0.029	0.047	0.073	0.046	0.029	0.029	0.027	0.058	0.024	0.032	0.042	0.032

WATER LEVEL FOR EVENT 3 AT LOCATION B (m)																
Time (min)	Existing	SCENARIO 1					SCENARIO 2					SCENARIO 3				
		0.1m	0.2m	0.3m	0.4m	0.5m	0.1m	0.2m	0.3m	0.4m	0.5m	0.1m	0.2m	0.3m	0.4m	0.5m
0000-0100	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0100-0200	0.062	0.108	0.081	0.071	0.067	0.066	0.125	0.077	0.069	0.065	0.064	0.110	0.055	0.069	0.065	0.064
0200-0300	0.203	0.500	0.452	0.278	0.250	0.235	0.500	0.438	0.278	0.250	0.234	0.500	0.293	0.326	0.262	0.241
0300-0400	0.102	0.494	0.159	0.129	0.118	0.112	0.426	0.163	0.130	0.119	0.114	0.488	0.097	0.149	0.127	0.120
0400-0500	0.055	0.264	0.076	0.065	0.061	0.059	0.239	0.079	0.067	0.063	0.061	0.300	0.045	0.074	0.067	0.065
0500-0600	0.052	0.137	0.069	0.060	0.057	0.055	0.137	0.069	0.061	0.057	0.055	0.177	0.042	0.064	0.059	0.057
0600-0700	0.047	0.090	0.061	0.054	0.051	0.050	0.092	0.061	0.054	0.051	0.050	0.110	0.037	0.057	0.053	0.051
0700-0800	0.047	0.084	0.060	0.054	0.051	0.049	0.085	0.061	0.054	0.051	0.049	0.095	0.037	0.056	0.052	0.050
0800-0900	0.042	0.073	0.053	0.047	0.044	0.043	0.074	0.053	0.047	0.045	0.044	0.081	0.031	0.049	0.046	0.045
0900-1000	0.038	0.060	0.055	0.041	0.041	0.039	0.060	0.047	0.045	0.044	0.039	0.056	0.025	0.041	0.038	0.038
1000-1100	0.039	0.050	0.050	0.036	0.052	0.049	0.075	0.031	0.036	0.035	0.026	0.085	0.009	0.088	0.047	0.037
1100-1200	0.076	0.049	0.039	0.039	0.035	0.029	0.050	0.031	0.029	0.030	0.022	0.070	0.009	0.044	0.050	0.039
1200-1300	0.057	0.064	0.051	0.044	0.030	0.037	0.053	0.031	0.032	0.027	0.026	0.066	0.014	0.069	0.041	0.040
1300-1400	0.036	0.056	0.055	0.035	0.046	0.039	0.059	0.041	0.035	0.034	0.030	0.070	0.018	0.046	0.036	0.036
1400-1500	0.042	0.079	0.053	0.036	0.030	0.052	0.073	0.049	0.036	0.033	0.030	0.066	0.018	0.035	0.054	0.038

WATER LEVEL FOR EVENT 3 AT LOCATION C (m)																
Time (min)	Existing	SCENARIO 1					SCENARIO 2					SCENARIO 3				
		0.1m	0.2m	0.3m	0.4m	0.5m	0.1m	0.2m	0.3m	0.4m	0.5m	0.1m	0.2m	0.3m	0.4m	0.5m
0000-0100	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0100-0200	0.064	0.108	0.082	0.072	0.068	0.067	0.144	0.081	0.072	0.068	0.066	0.111	0.017	0.070	0.065	0.065
0200-0300	0.207	0.500	0.452	0.280	0.252	0.237	0.500	0.500	0.292	0.260	0.242	0.500	0.120	0.327	0.263	0.243
0300-0400	0.104	0.494	0.159	0.130	0.119	0.114	0.500	0.171	0.136	0.123	0.117	0.488	0.030	0.151	0.128	0.122
0400-0500	0.056	0.264	0.077	0.066	0.062	0.060	0.291	0.082	0.070	0.065	0.062	0.300	0.014	0.075	0.068	0.066
0500-0600	0.053	0.138	0.069	0.061	0.058	0.056	0.160	0.072	0.063	0.059	0.057	0.177	0.013	0.064	0.060	0.058
0600-0700	0.048	0.090	0.062	0.055	0.052	0.051	0.100	0.064	0.056	0.053	0.051	0.110	0.012	0.057	0.054	0.052
0700-0800	0.048	0.084	0.061	0.054	0.052	0.050	0.090	0.063	0.056	0.052	0.051	0.095	0.012	0.056	0.053	0.051
0800-0900	0.045	0.073	0.053	0.051	0.044	0.043	0.078	0.056	0.049	0.046	0.045	0.082	0.010	0.050	0.047	0.046
0900-1000	0.038	0.064	0.055	0.041	0.041	0.041	0.067	0.050	0.051	0.046	0.040	0.068	0.008	0.040	0.045	0.037
1000-1100	0.040	0.064	0.046	0.029	0.067	0.060	0.085	0.038	0.040	0.036	0.028	0.086	0.003	0.122	0.051	0.037
1100-1200	0.080	0.065	0.045	0.040	0.039	0.000	0.060	0.038	0.033	0.034	0.023	0.067	0.003	0.055	0.078	0.042
1200-1300	0.061	0.063	0.051	0.045	0.038	0.046	0.067	0.042	0.035	0.033	0.026	0.089	0.005	0.070	0.038	0.041
1300-1400	0.036	0.061	0.056	0.039	0.047	0.033	0.065	0.059	0.034	0.036	0.030	0.077	0.006	0.047	0.043	0.039
1400-1500	0.043	0.079	0.053	0.039	0.060	0.053	0.081	0.051	0.044	0.034	0.030	0.066	0.006	0.063	0.055	0.040

WATER LEVEL FOR EVENT 3 AT LOCATION D (m)																
Time (min)	Existing	SCENARIO 1					SCENARIO 2					SCENARIO 3				
		0.1m	0.2m	0.3m	0.4m	0.5m	0.1m	0.2m	0.3m	0.4m	0.5m	0.1m	0.2m	0.3m	0.4m	0.5m
0000-0100	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0100-0200	0.065	0.109	0.082	0.073	0.069	0.068	0.145	0.081	0.072	0.068	0.067	0.114	0.033	0.071	0.067	0.066
0200-0300	0.210	0.500	0.453	0.281	0.254	0.239	0.500	0.500	0.293	0.261	0.244	0.500	0.134	0.332	0.267	0.247
0300-0400	0.105	0.494	0.161	0.131	0.120	0.115	0.500	0.171	0.136	0.124	0.118	0.500	0.062	0.153	0.130	0.124
0400-0500	0.057	0.265	0.077	0.067	0.063	0.061	0.291	0.083	0.070	0.065	0.063	0.317	0.027	0.076	0.069	0.067
0500-0600	0.054	0.138	0.070	0.062	0.058	0.057	0.160	0.073	0.063	0.059	0.058	0.185	0.026	0.065	0.061	0.059
0600-0700	0.049	0.090	0.062	0.055	0.053	0.051	0.100	0.065	0.057	0.053	0.052	0.114	0.022	0.058	0.055	0.053
0700-0800	0.049	0.084	0.061	0.055	0.052	0.051	0.090	0.064	0.056	0.053	0.051	0.097	0.022	0.057	0.054	0.052
0800-0900	0.045	0.074	0.054	0.049	0.047	0.046	0.079	0.056	0.050	0.047	0.046	0.084	0.019	0.051	0.048	0.047
0900-1000	0.041	0.067	0.055	0.046	0.044	0.045	0.070	0.052	0.051	0.047	0.045	0.077	0.015	0.048	0.045	0.044
1000-1100	0.041	0.155	0.049	0.040	0.090	0.070	0.083	0.043	0.048	0.040	0.025	0.088	0.005	0.131	0.061	0.040
1100-1200	0.083	0.065	0.060	0.041	0.047	0.098	0.065	0.040	0.047	0.036	0.023	0.071	0.005	0.048	0.084	0.041
1200-1300	0.065	0.064	0.052	0.045	0.053	0.063	0.075	0.043	0.044	0.045	0.027	0.077	0.008	0.072	0.044	0.043
1300-1400	0.039	0.060	0.056	0.048	0.047	0.041	0.067	0.061	0.049	0.036	0.031	0.075	0.011	0.048	0.043	0.041
1400-1500	0.044	0.079	0.054	0.047	0.057	0.053	0.081	0.053	0.051	0.034	0.031	0.072	0.011	0.054	0.057	0.040

Appendix I: Simulation Result of Flow Patterns

A. Drain with 0% Slope

Design Rainfall (Flow Rate)

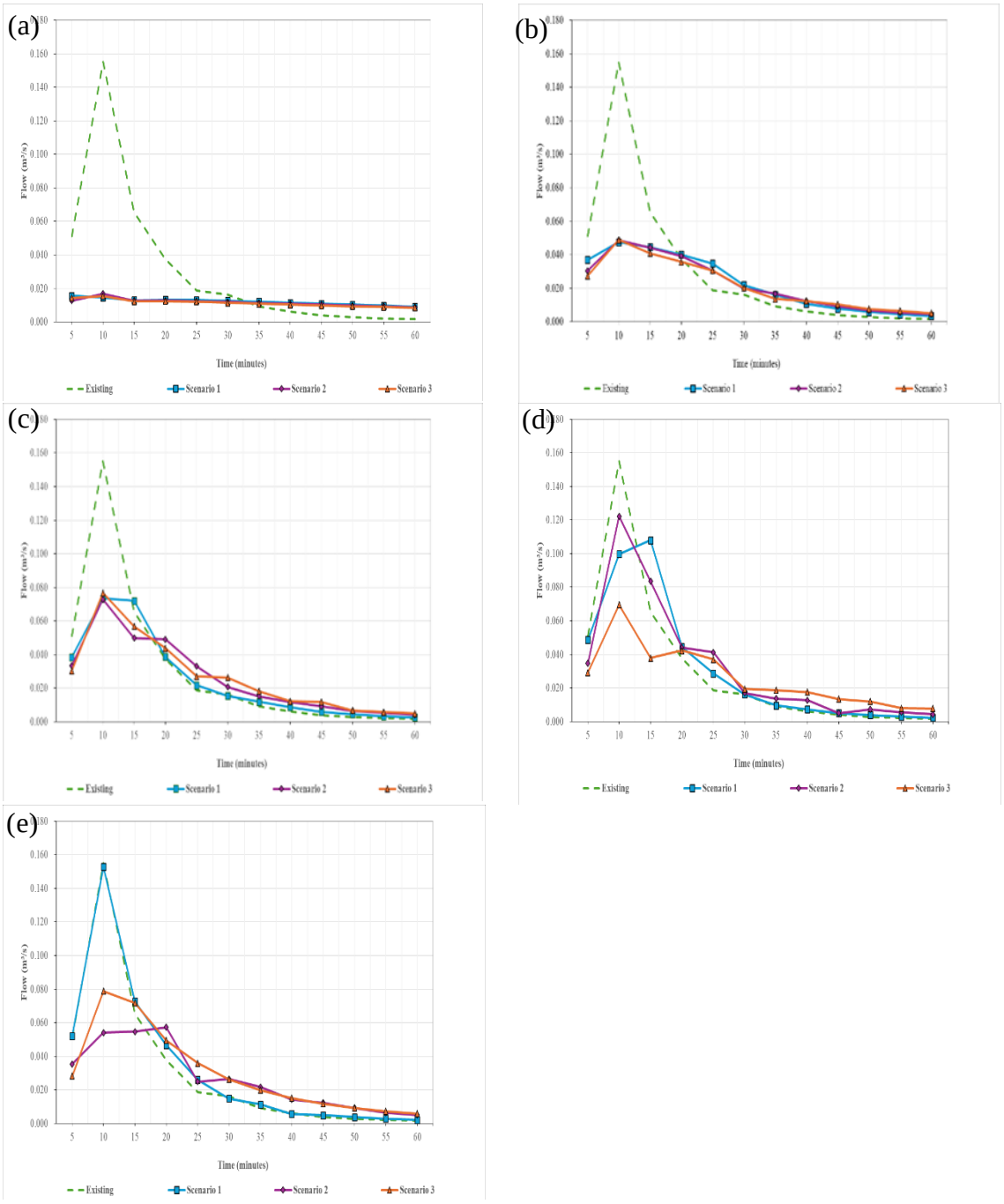


Figure AI.1: Flow Plots for Drain with 0% Slope at Location A Subjected to Design Rainfall, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

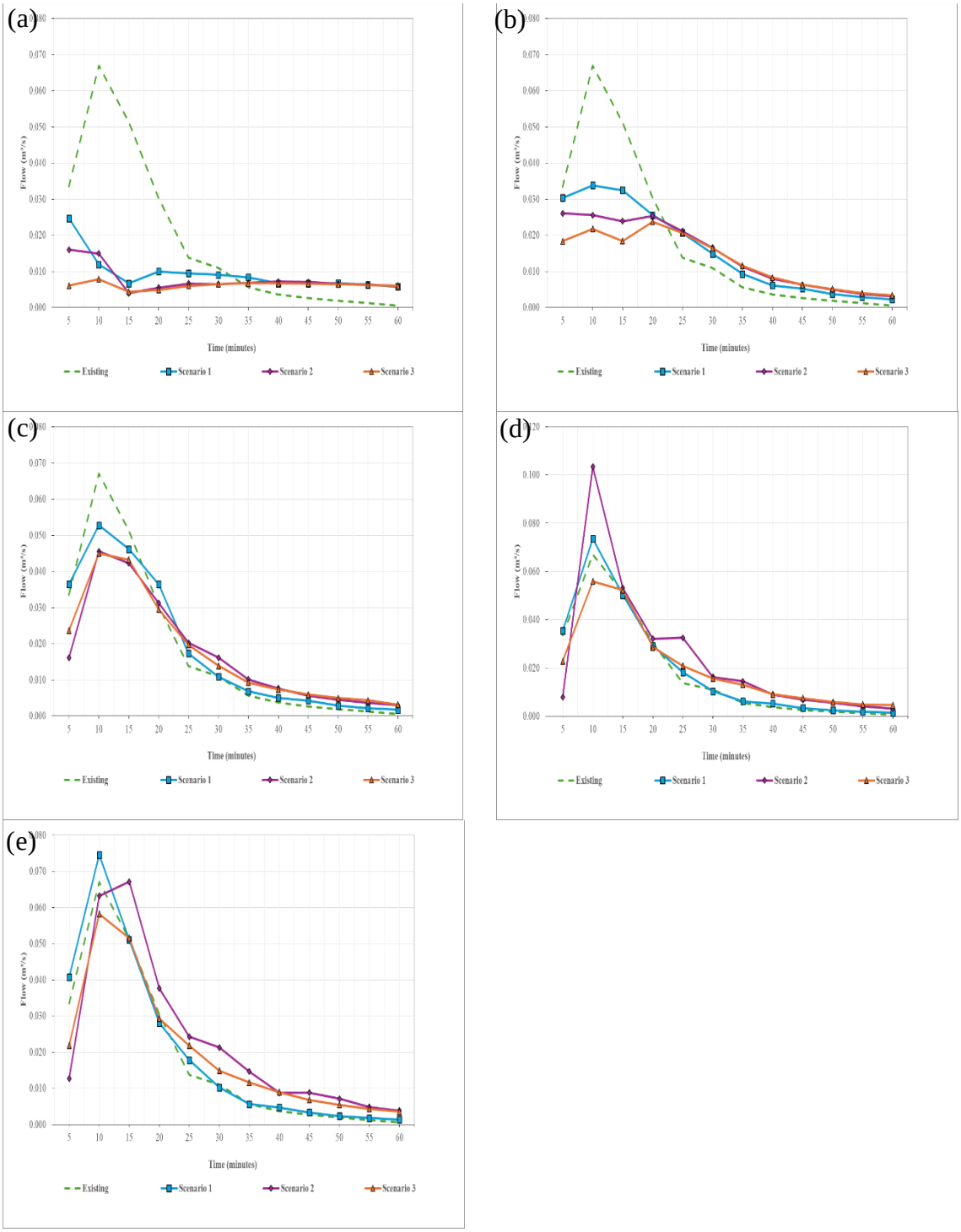


Figure AI.2: Flow Plots for Drain with 0% Slope at Location B Subjected to Design Rainfall, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

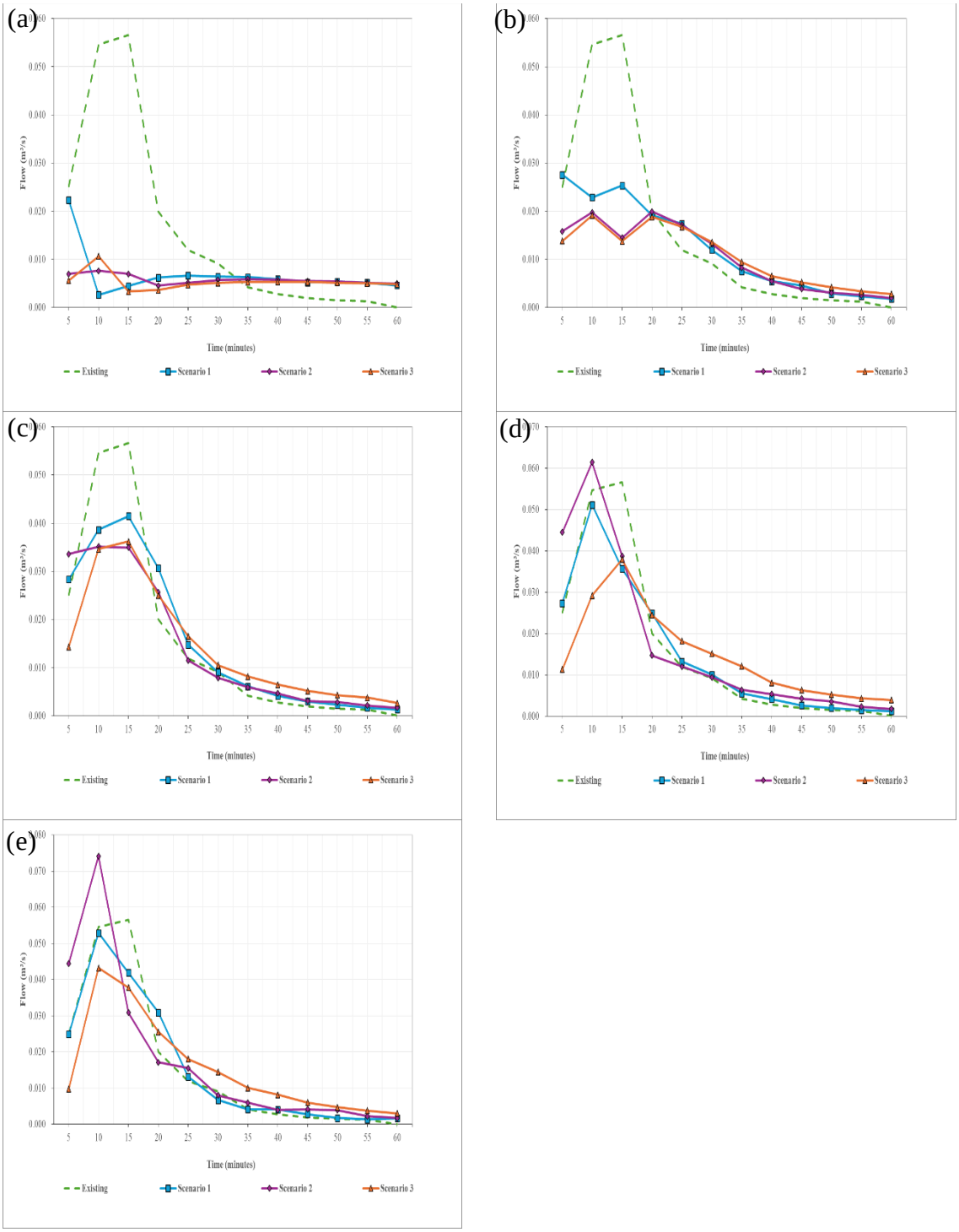


Figure AI.3: Flow Plots for Drain with 0% Slope at Location C Subjected to Design Rainfall, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

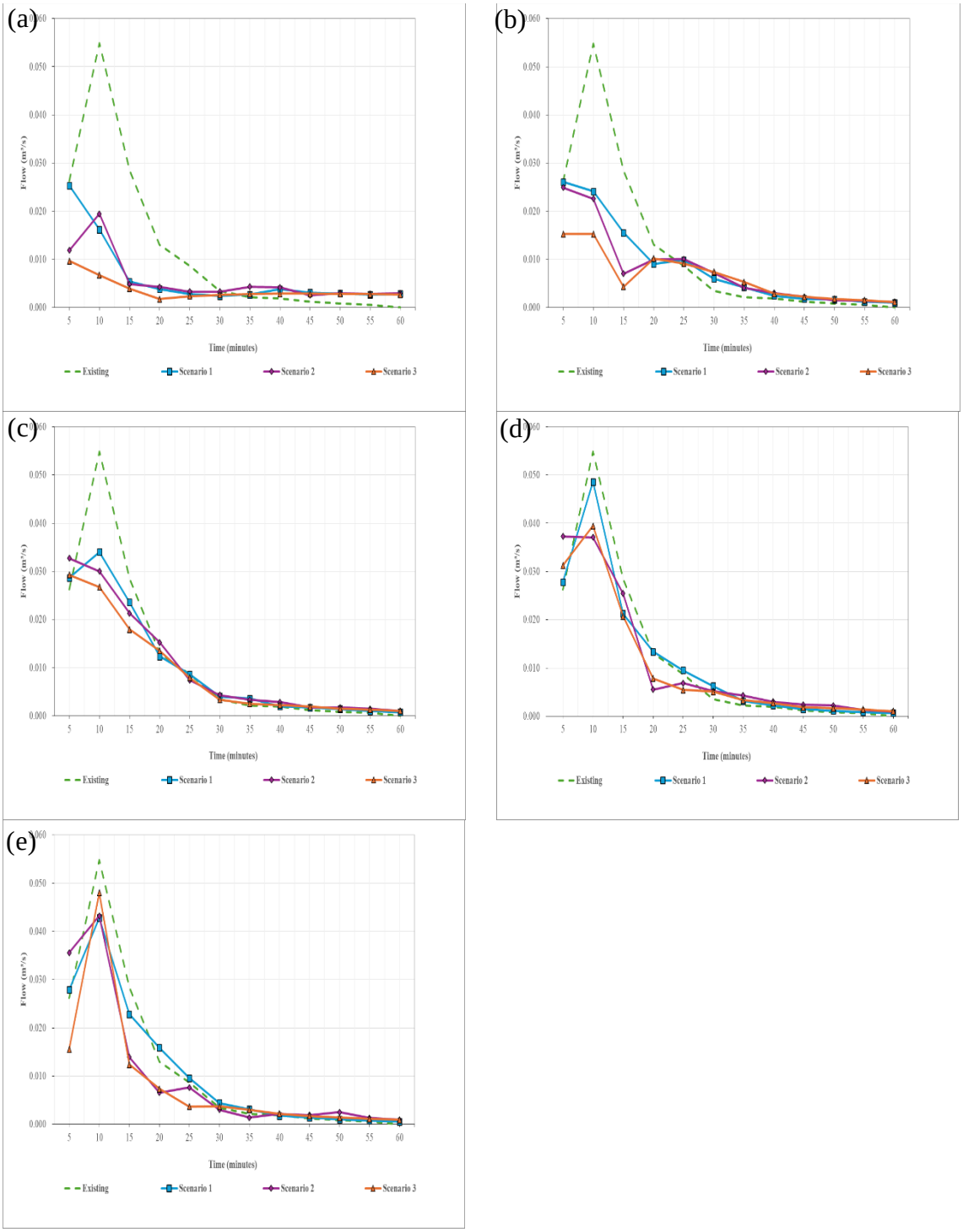


Figure AI.4: Flow Plots for Drain with 0% Slope at Location D Subjected to Design Rainfall, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

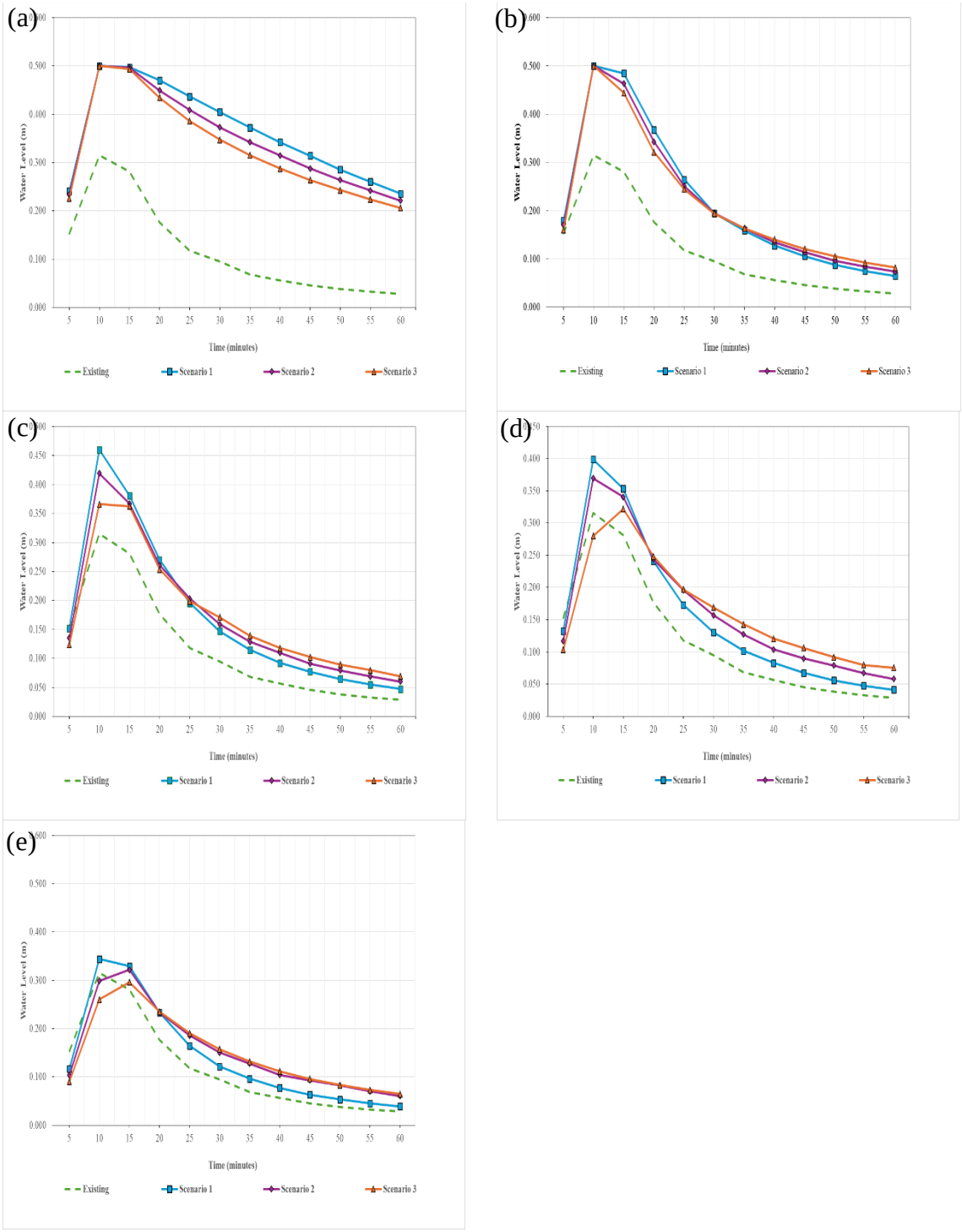


Figure AI.5: Water Level Plots for Drain with 0% Slope at Location A Subjected to Design Rainfall, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

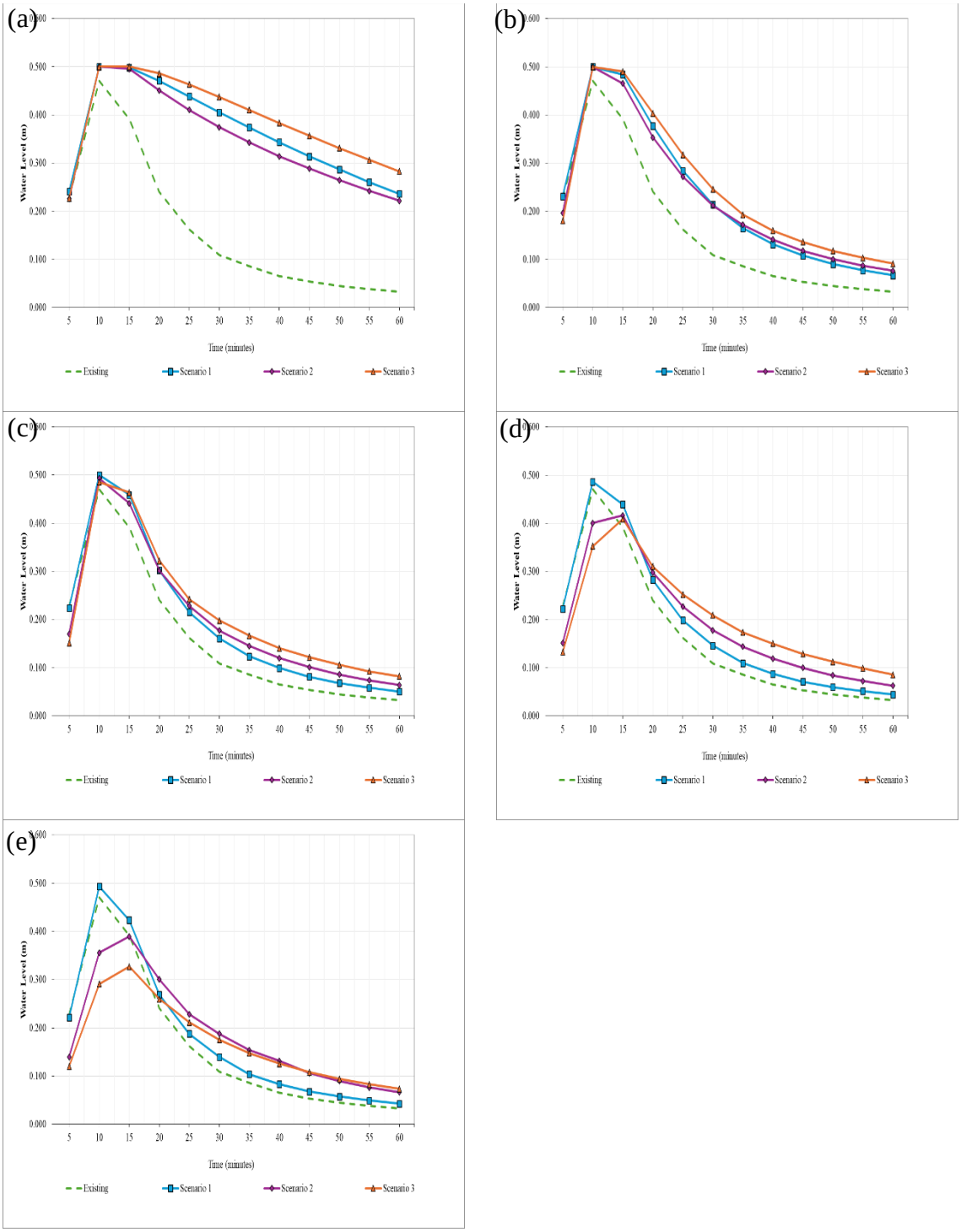


Figure AI.6: Water Level Plots for Drain with 0% Slope at Location B Subjected to Design Rainfall, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

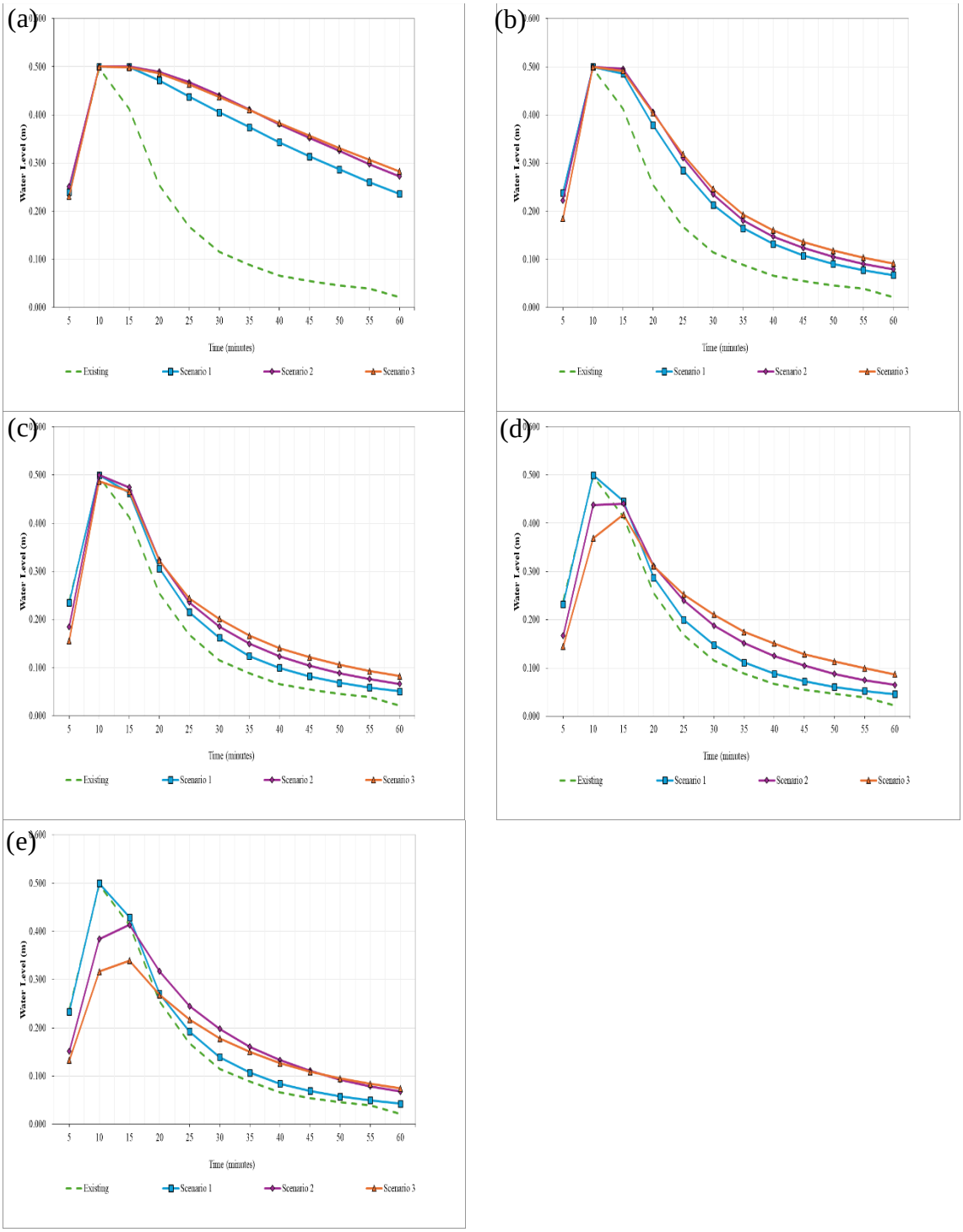


Figure AI.7: Water Level Plots for Drain with 0% Slope at Location C Subjected to Design Rainfall, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

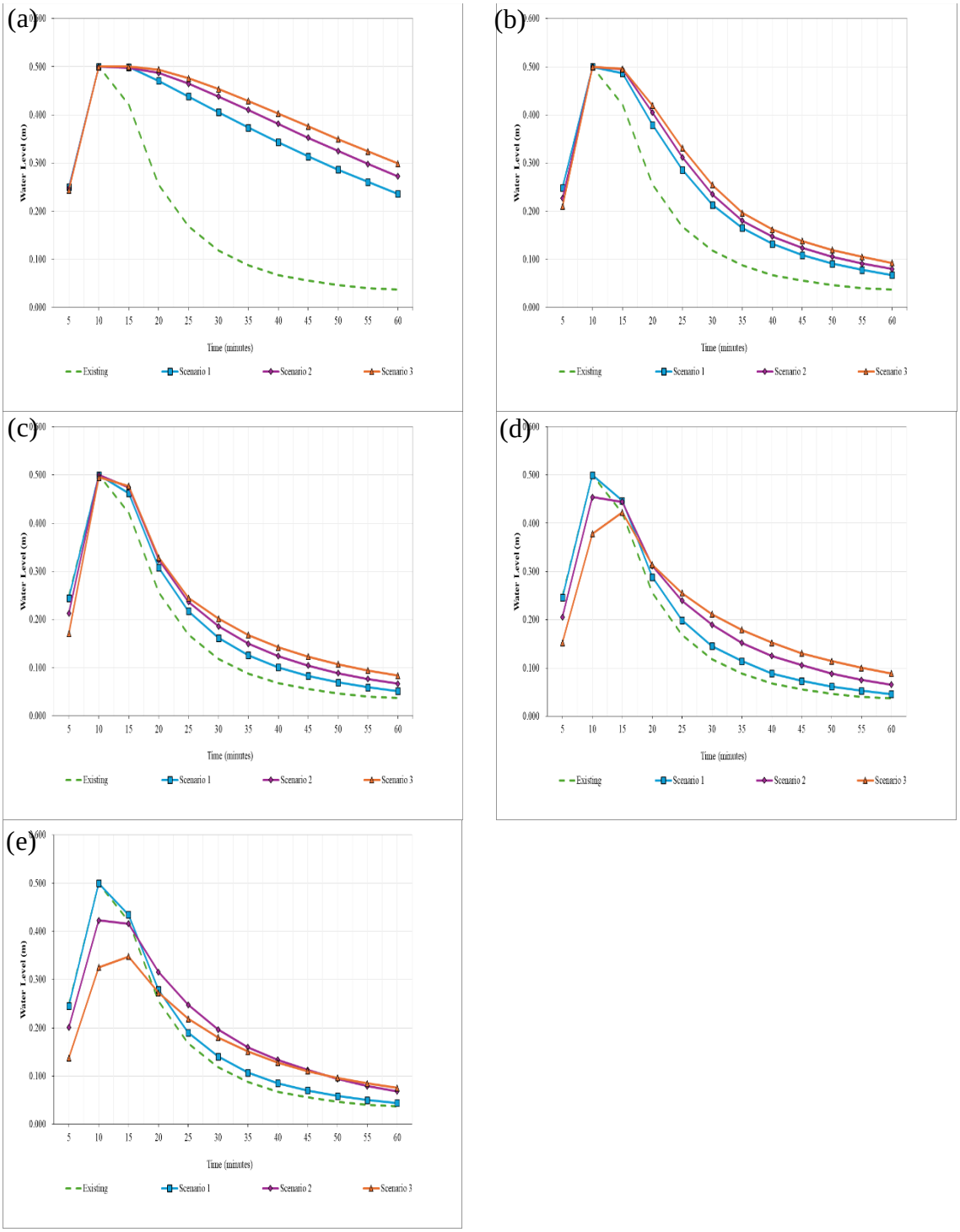


Figure AI.8: Water Level Plots for Drain with 0% Slope at Location D Subjected to Design Rainfall, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

ii. Drain With 0% Slope - Historical Rainfall

Event 1 (Flow Rates)

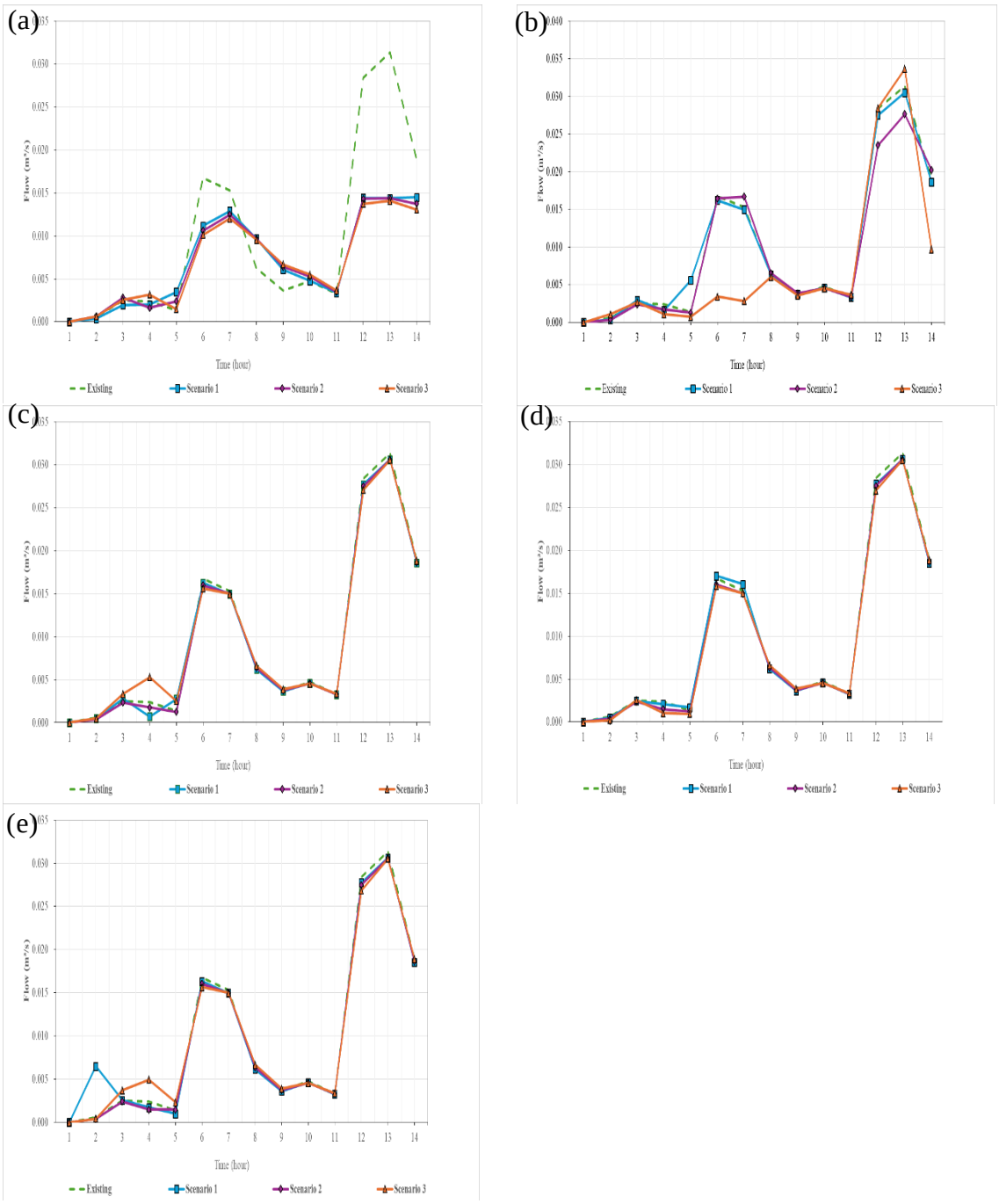


Figure AI.9: Flow Plots for Drain with 0% Slope at Location A Subjected to Event 1, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice



Figure AI.10: Flow Plots for Drain with 0% Slope at Location B Subjected to Event 1, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice



Figure AI.11: Flow Plots for Drain with 0% Slope at Location C Subjected to Event 1, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

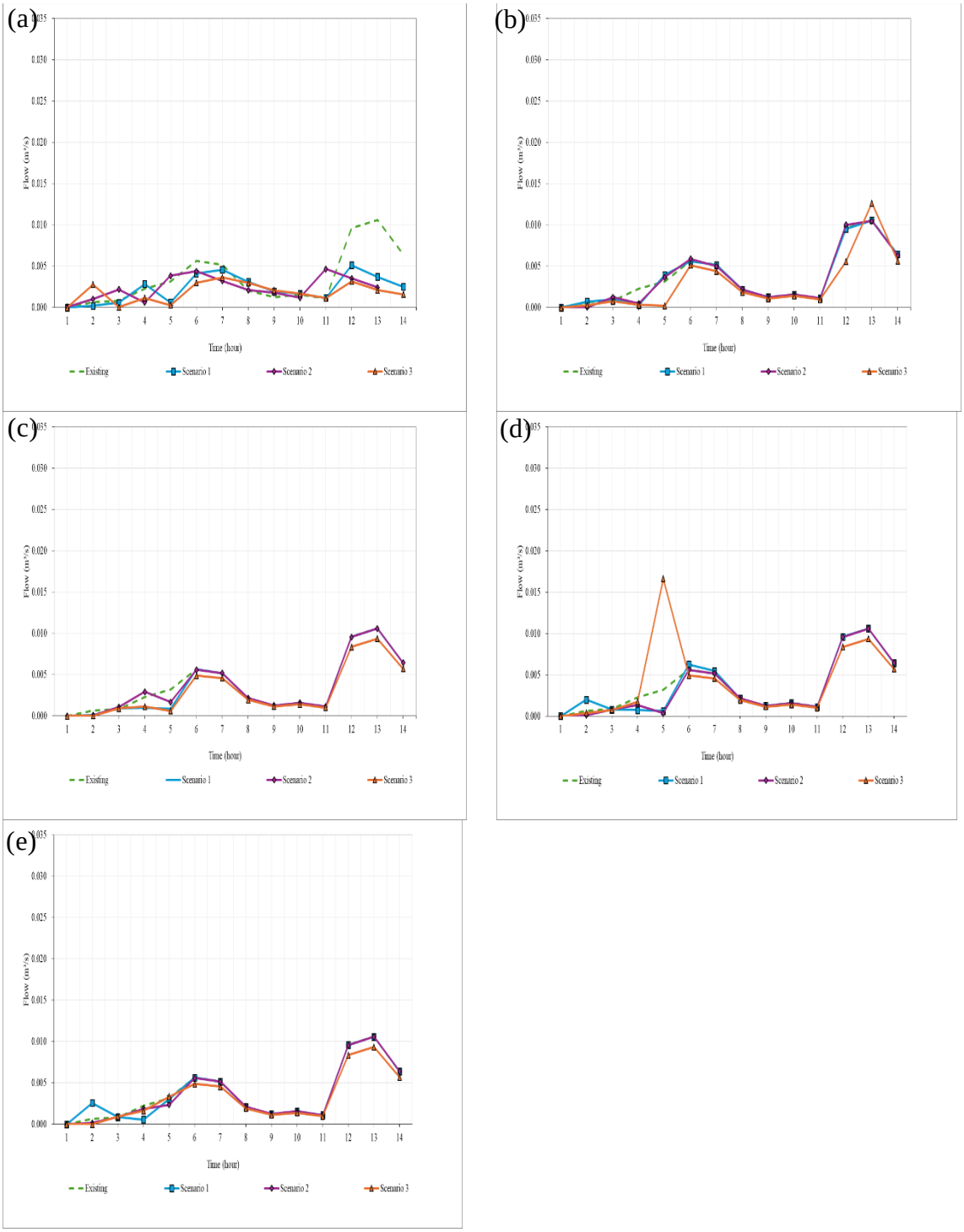


Figure AI.12: Flow Plots for Drain with 0% Slope at Location D Subjected to Event 1, (a) 0.1m, (b) 0. 2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

Event 1 (Water Levels)

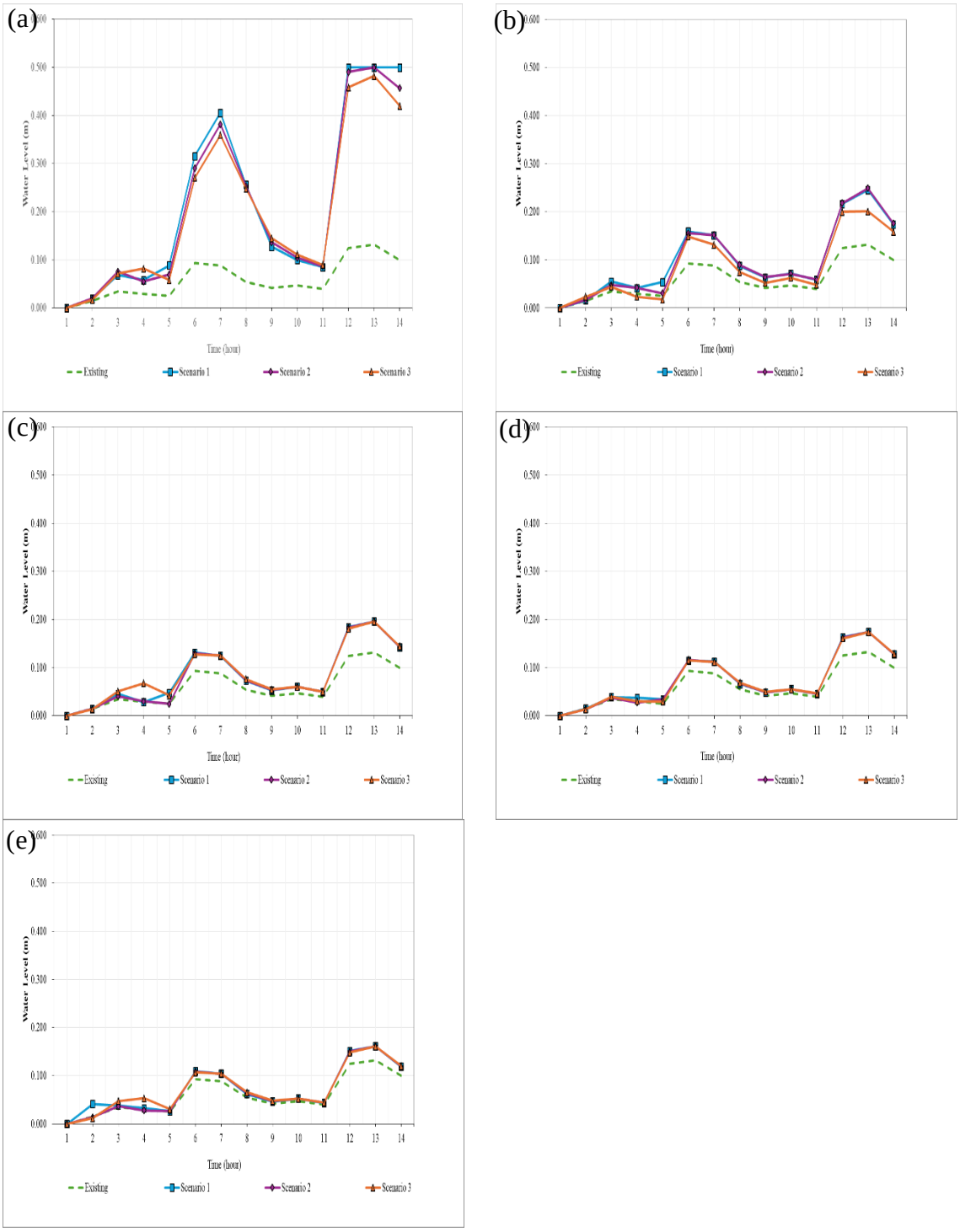


Figure AI.13: Water Level Plots for Drain with 0% Slope at Location A Subjected to Event 1, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

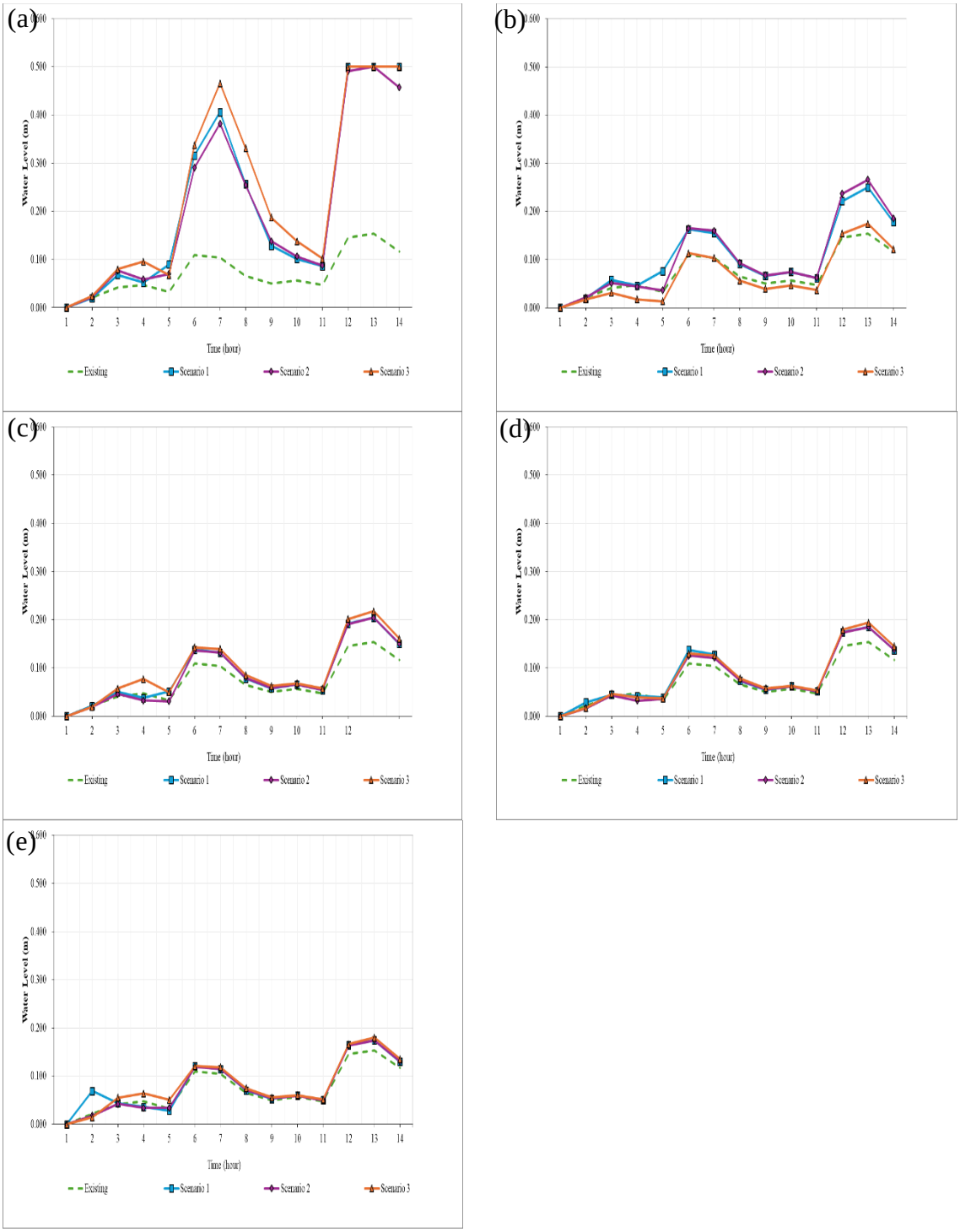


Figure AI.14: Water Level Plots for Drain with 0% Slope at Location B Subjected to Event 1, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

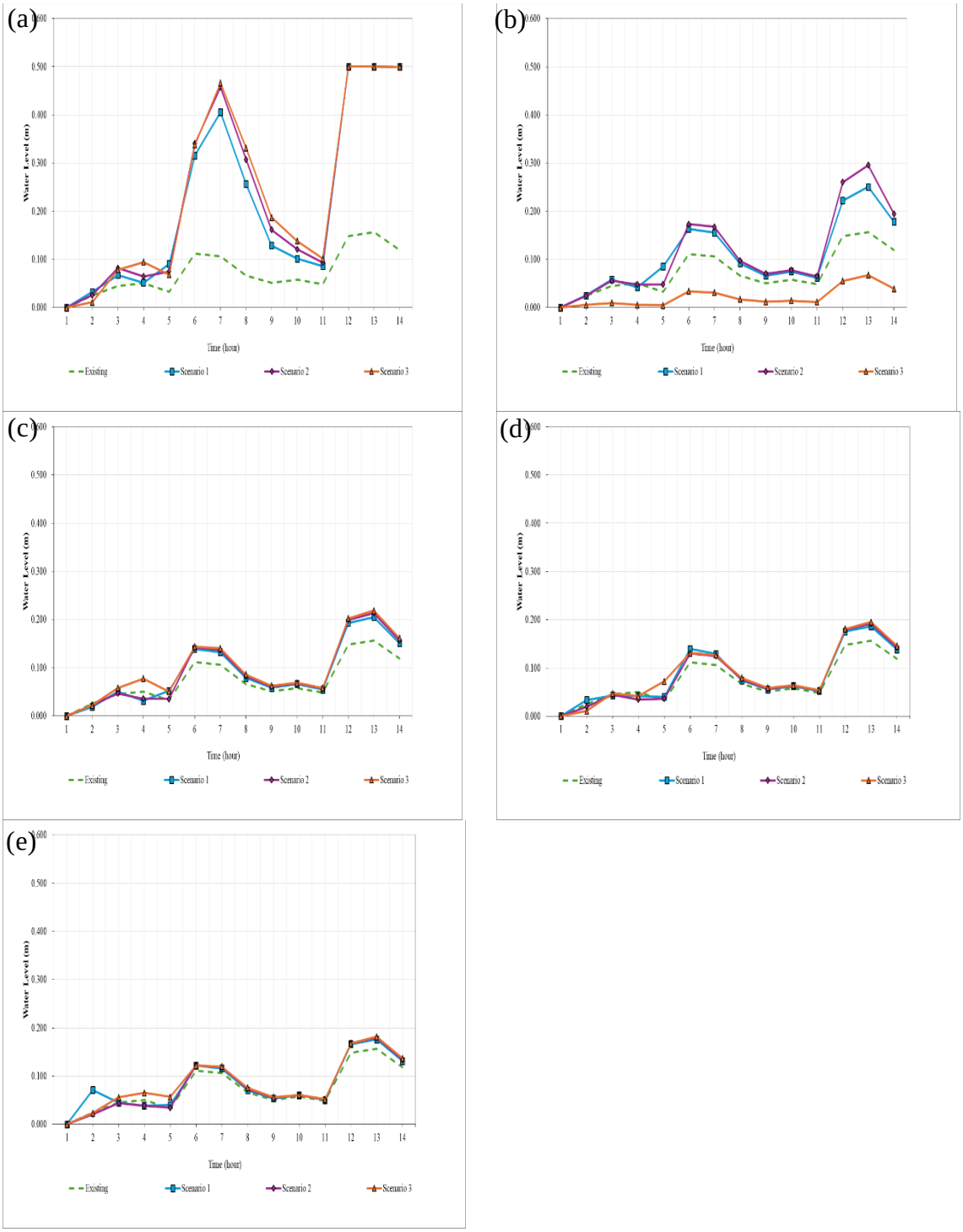


Figure AI.15: Water Level Plots for Drain with 0% Slope at Location C Subjected to Event 1, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

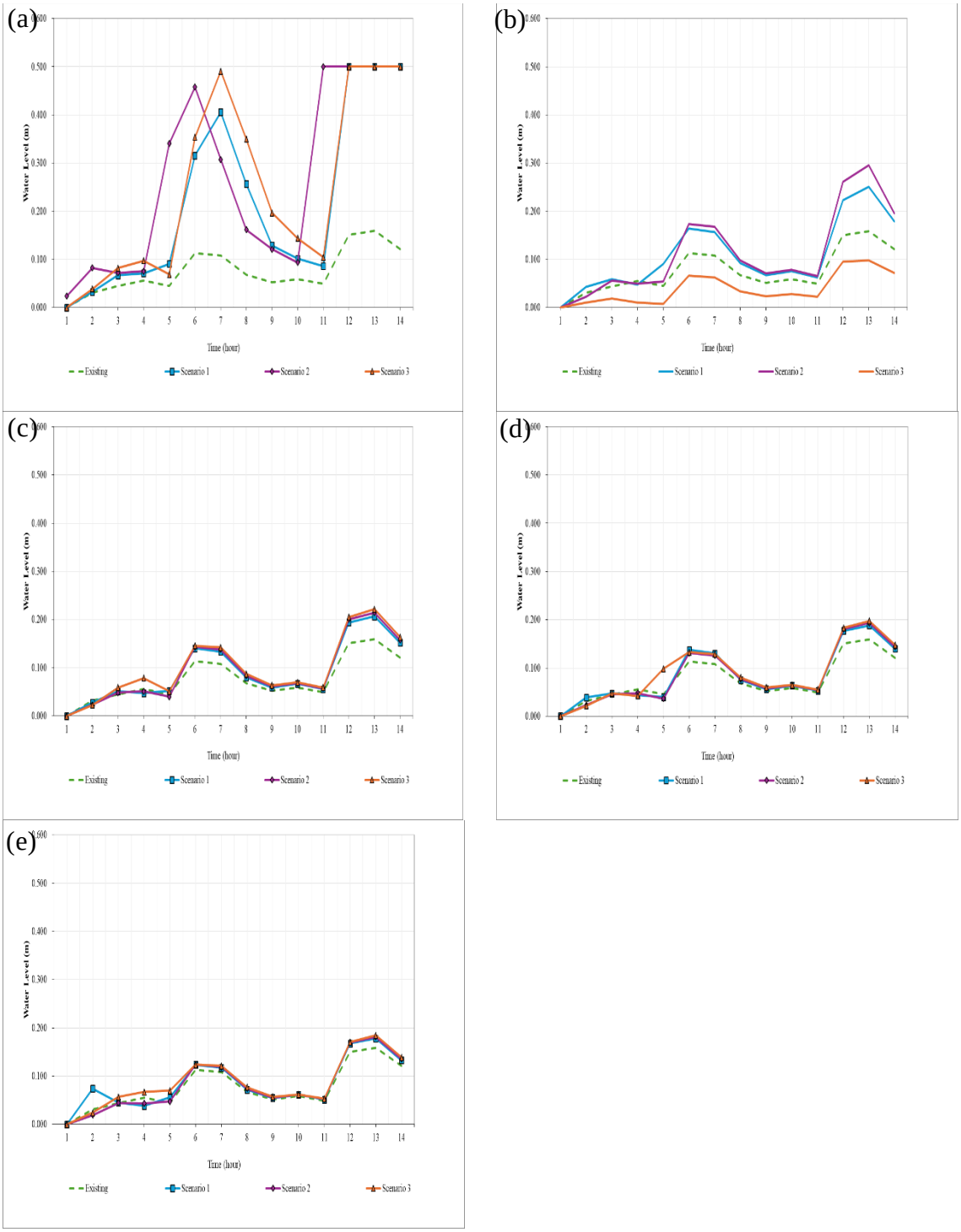


Figure AI.16: Water Level Plots for Drain with 0% Slope at Location D Subjected to Event 1, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

Event 2 (Flow Rates)

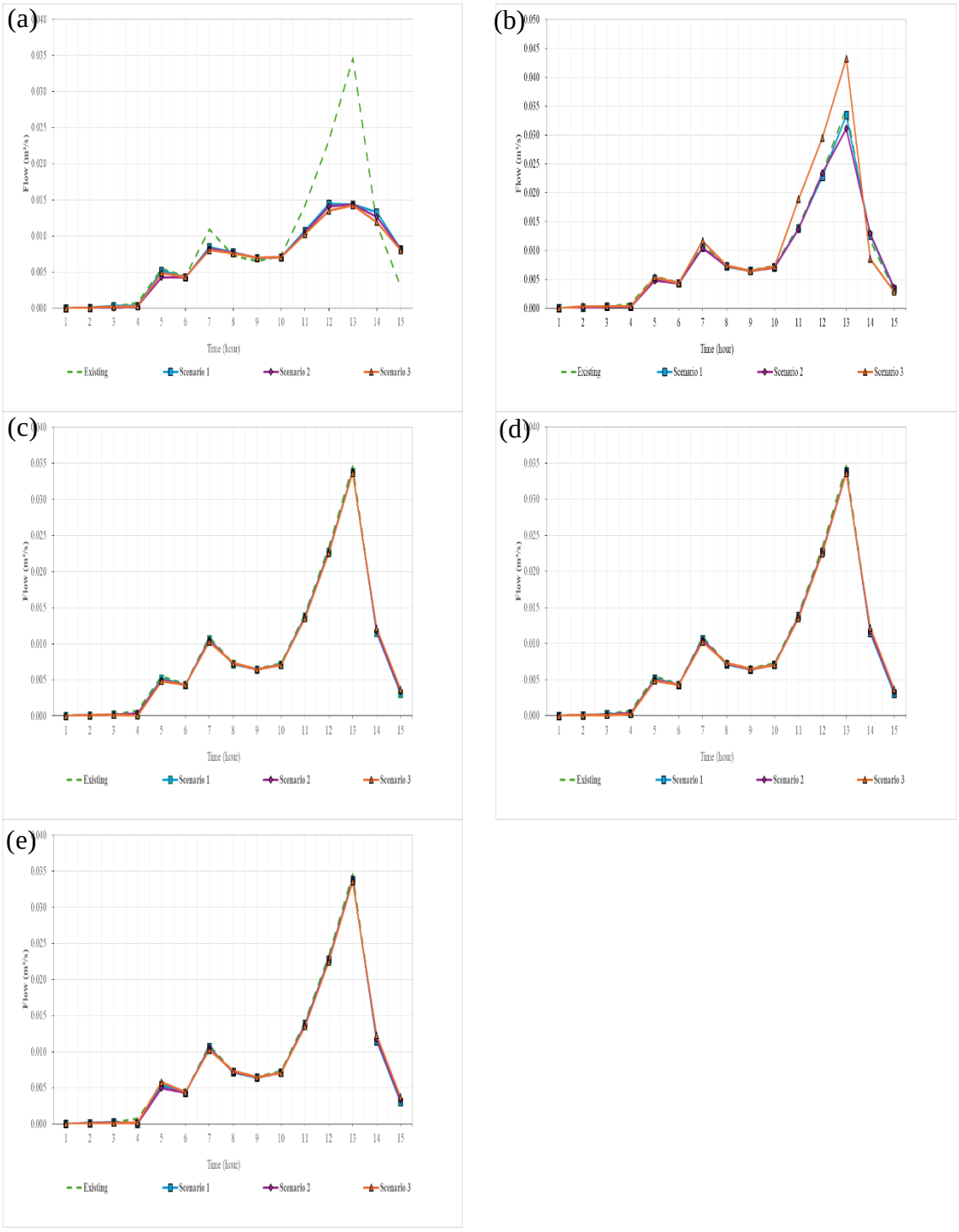


Figure AI.17: Flow Plots for Drain with 0% Slope at Location A Subjected to Event 2, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

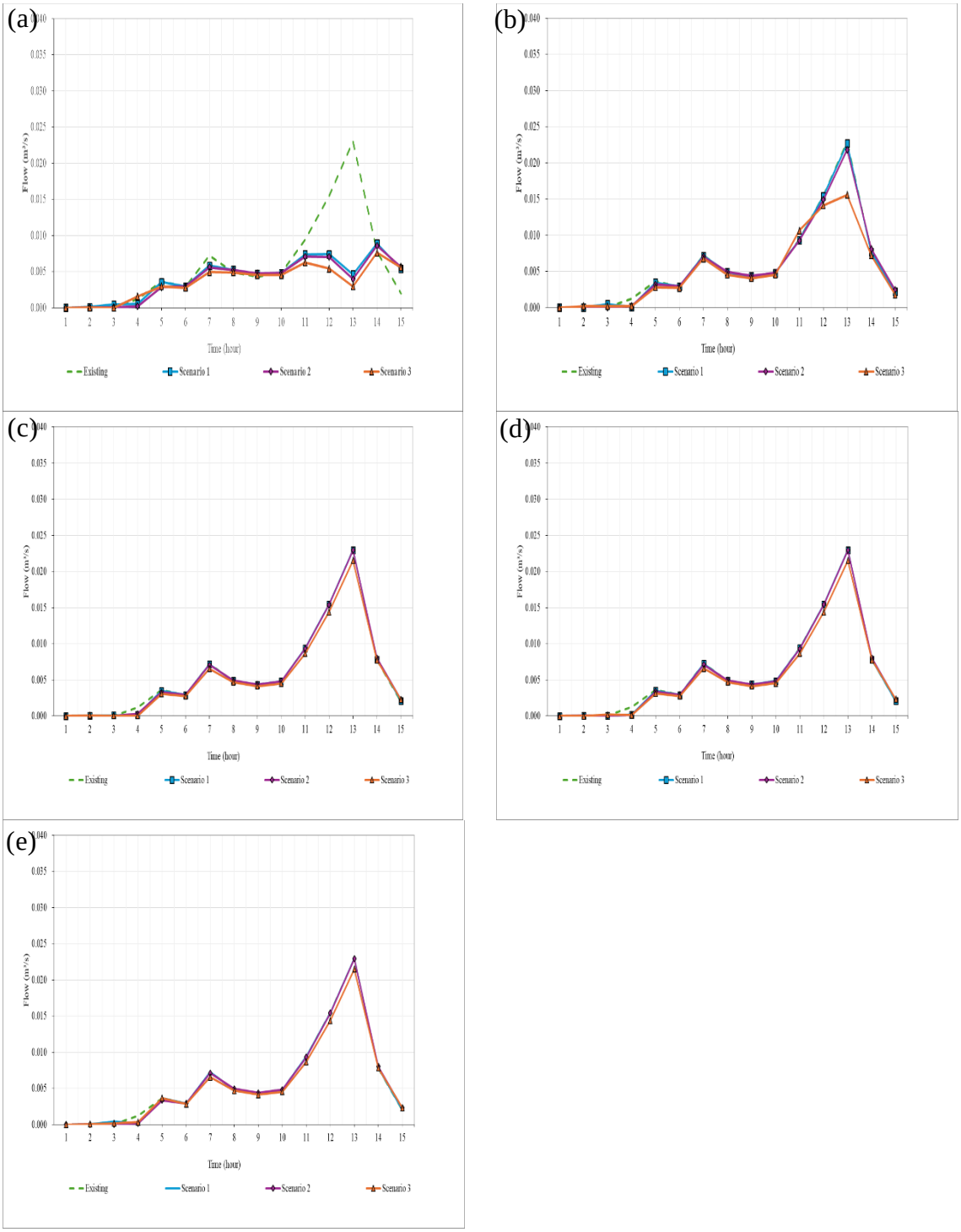


Figure AI.18: Flow Plots for Drain with 0% Slope at Location B Subjected to Event 2, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

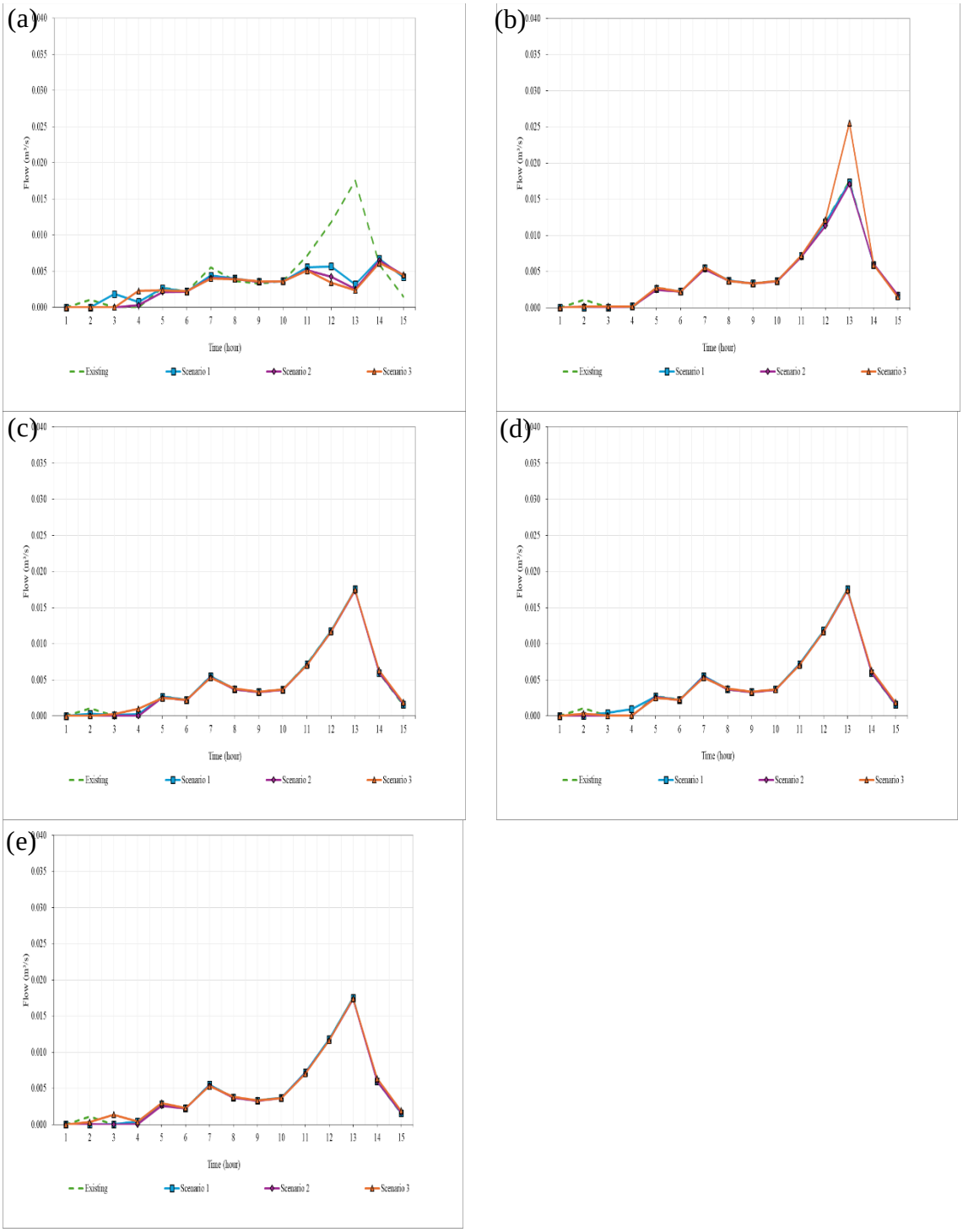


Figure AI.19: Flow Plots for Drain with 0% Slope at Location C Subjected to Event 2, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

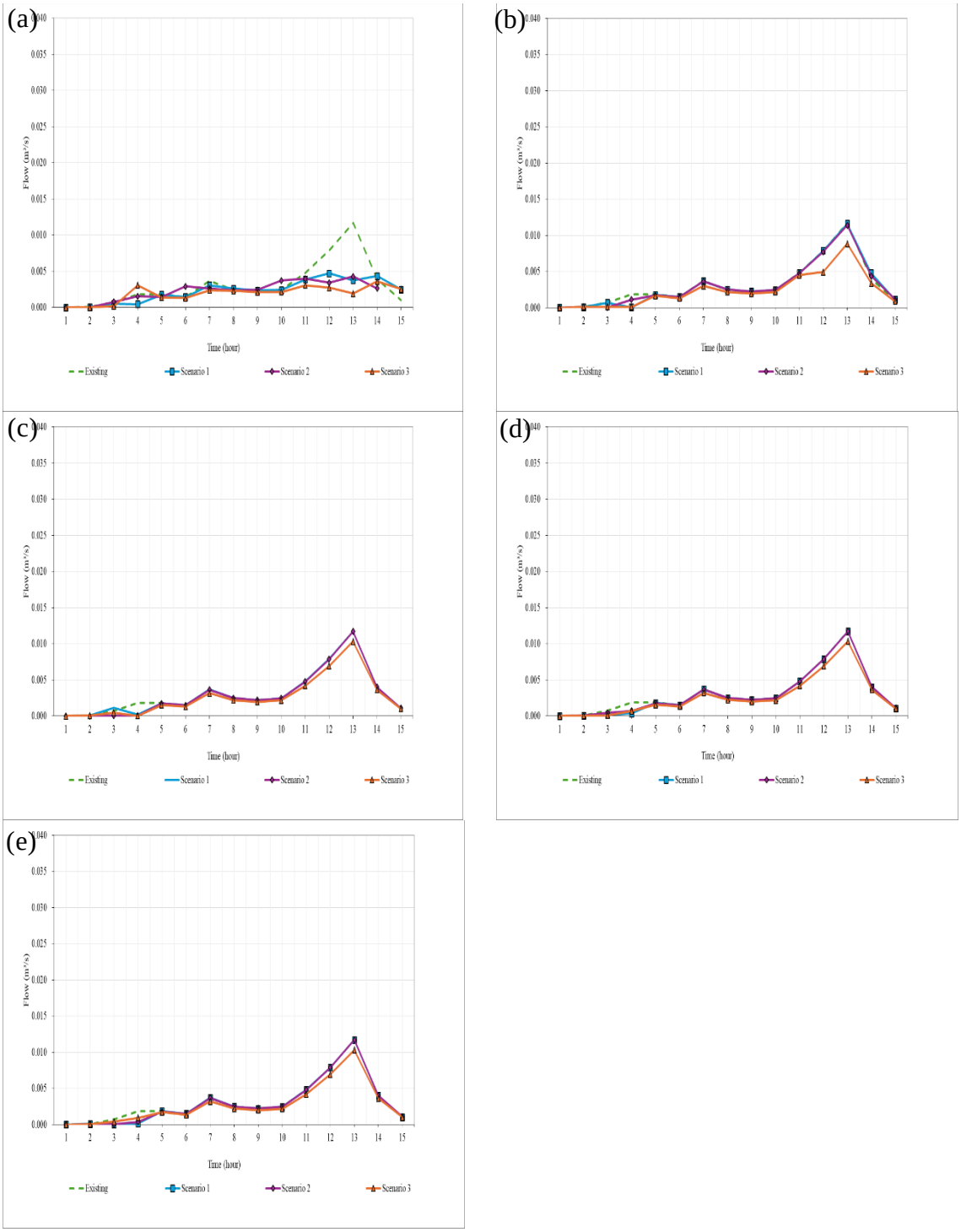


Figure AI.20: Flow Plots for Drain with 0% Slope at Location D Subjected to Event 2, (a) 0.1m, (b) 0. 2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

Event 2 (Water Levels)

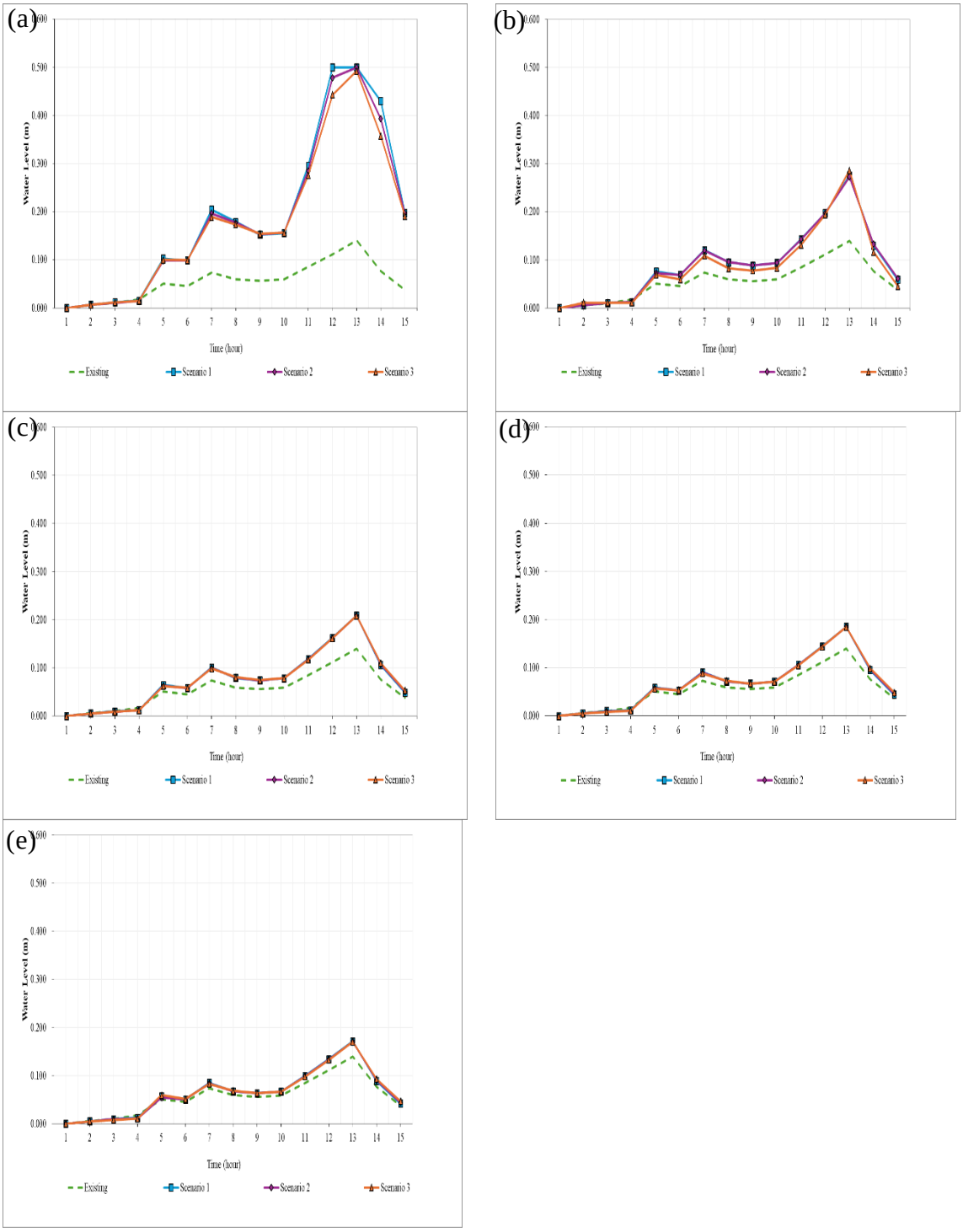


Figure AI.21: Water Level Plots for Drain with 0% Slope at Location A Subjected to Event 2, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

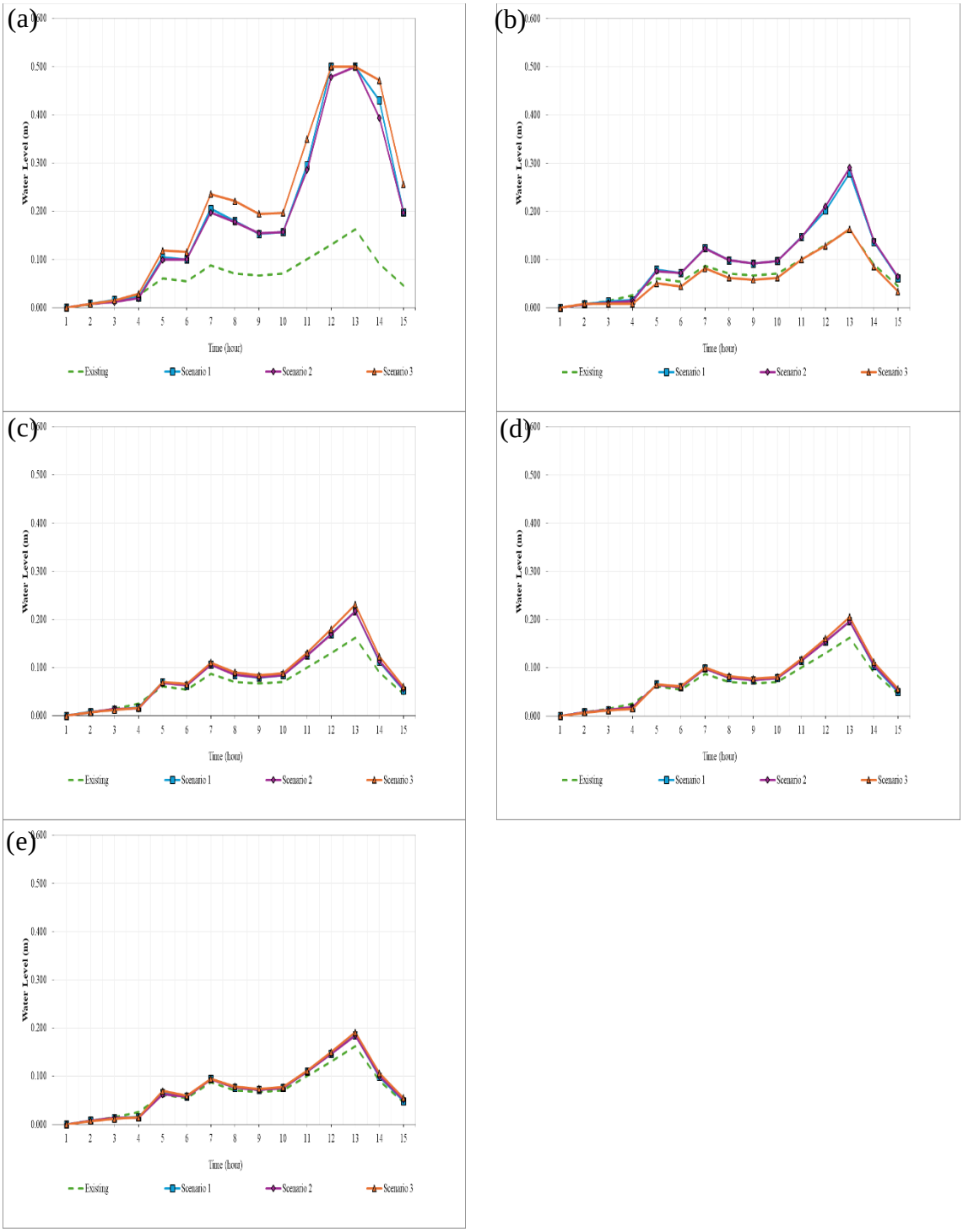


Figure AI.22: Water Level Plots for Drain with 0% Slope at Location B Subjected to Event 2, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

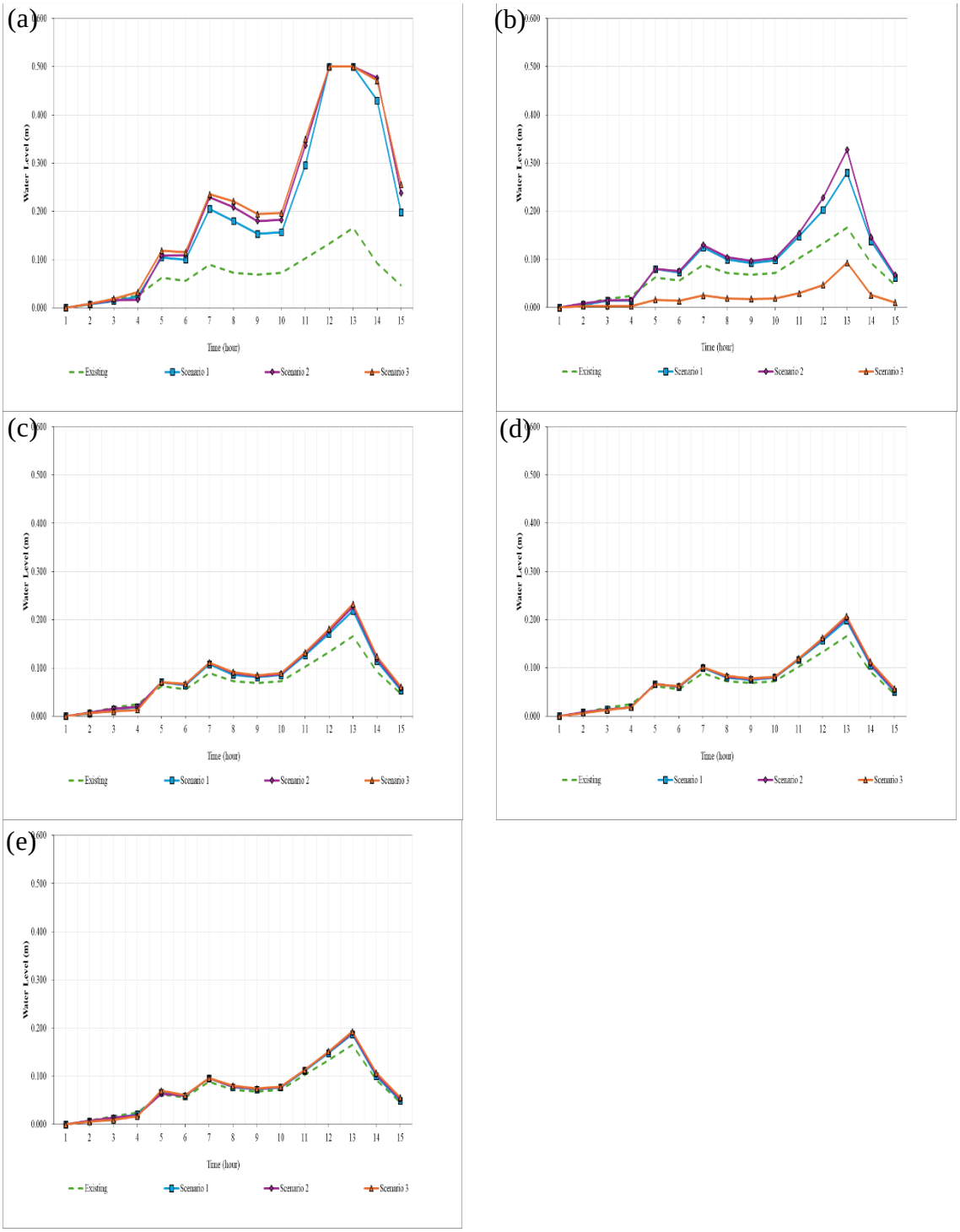


Figure AI.23: Water Level Plots for Drain with 0% Slope at Location C Subjected to Event 2, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

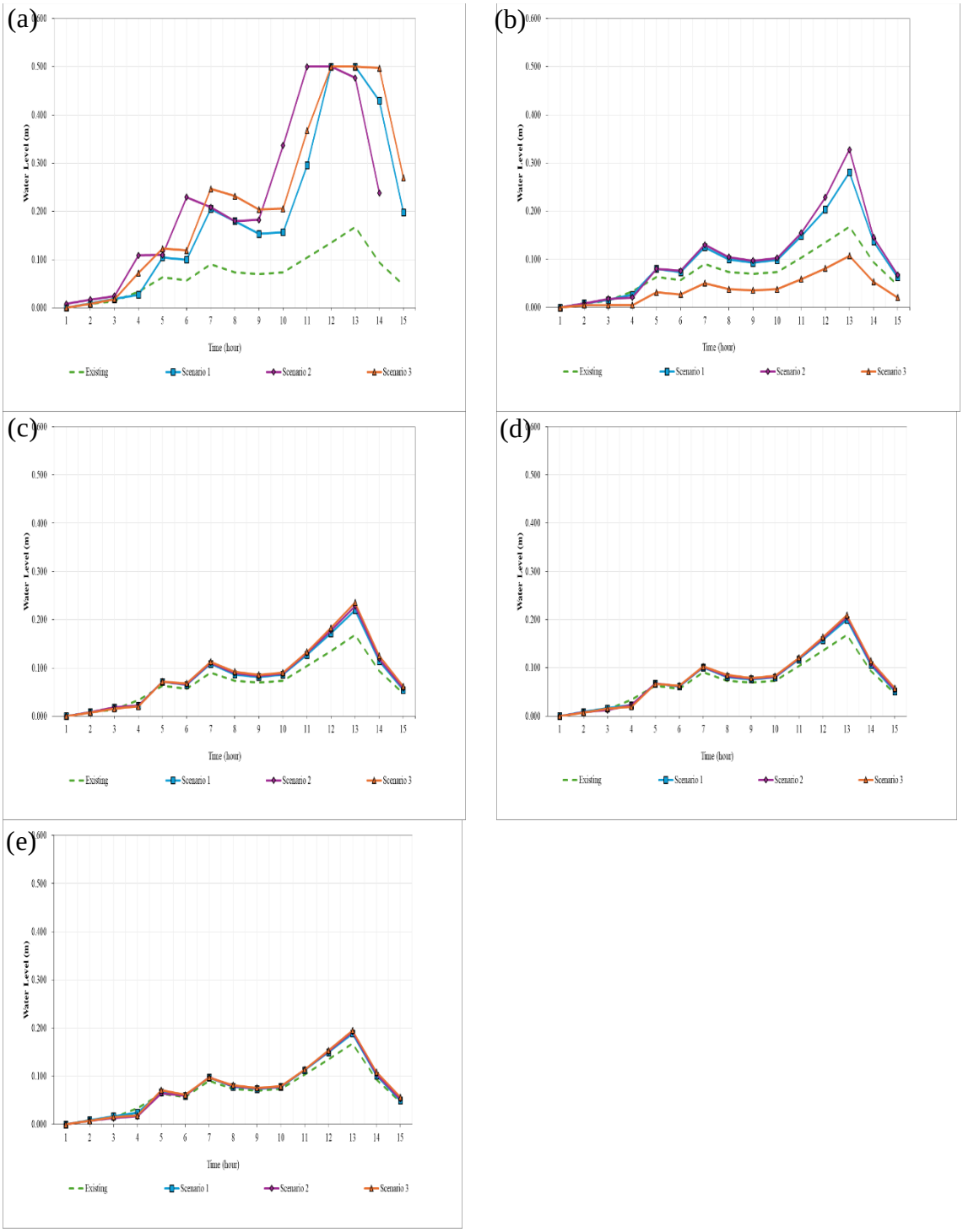


Figure AI.24: Water Level Plots for Drain with 0% Slope at Location D Subjected to Event 2, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

Event 3 (Flow Rates)

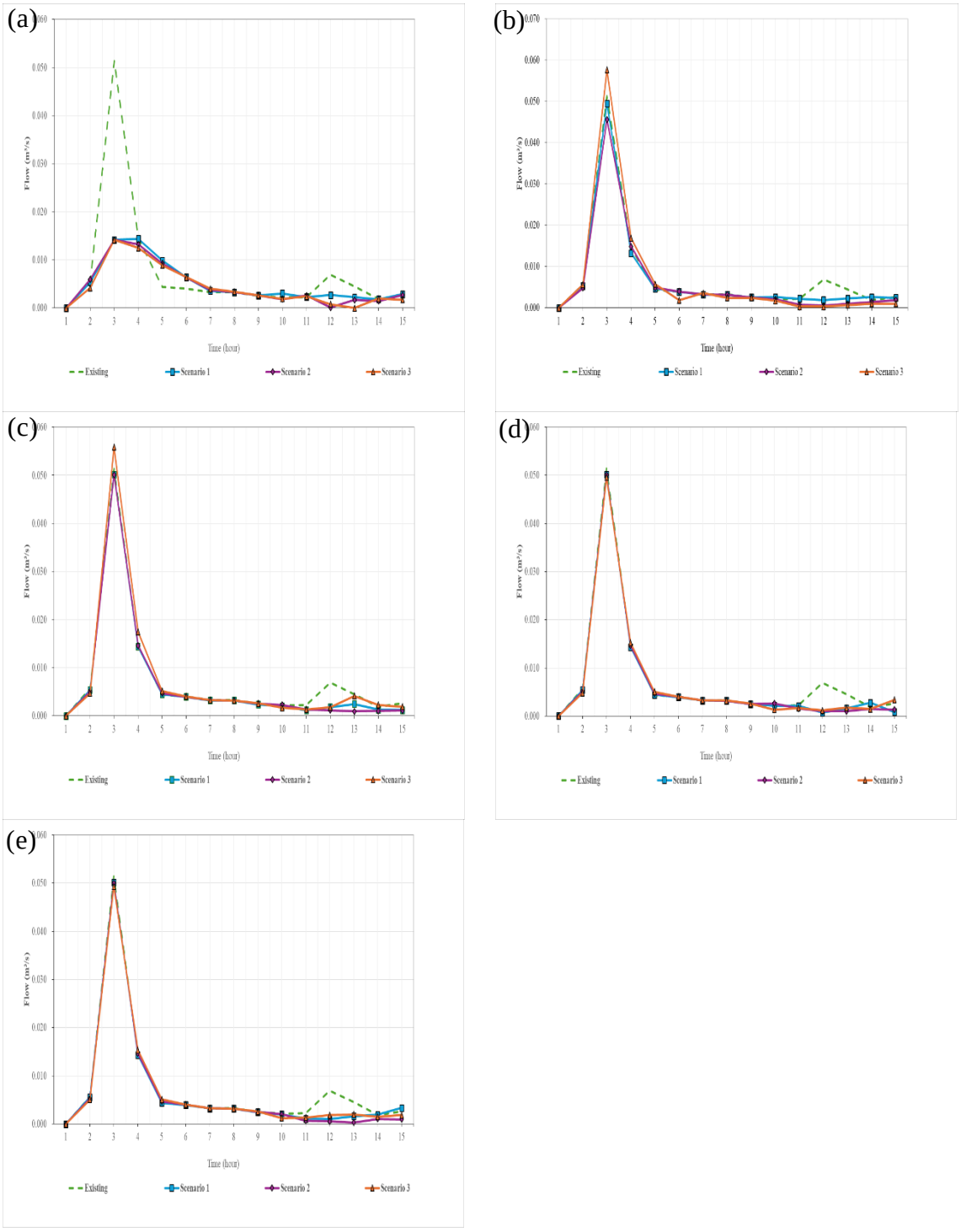


Figure AI.25: Flow Plots for Drain with 0% Slope at Location A Subjected to Event 3, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

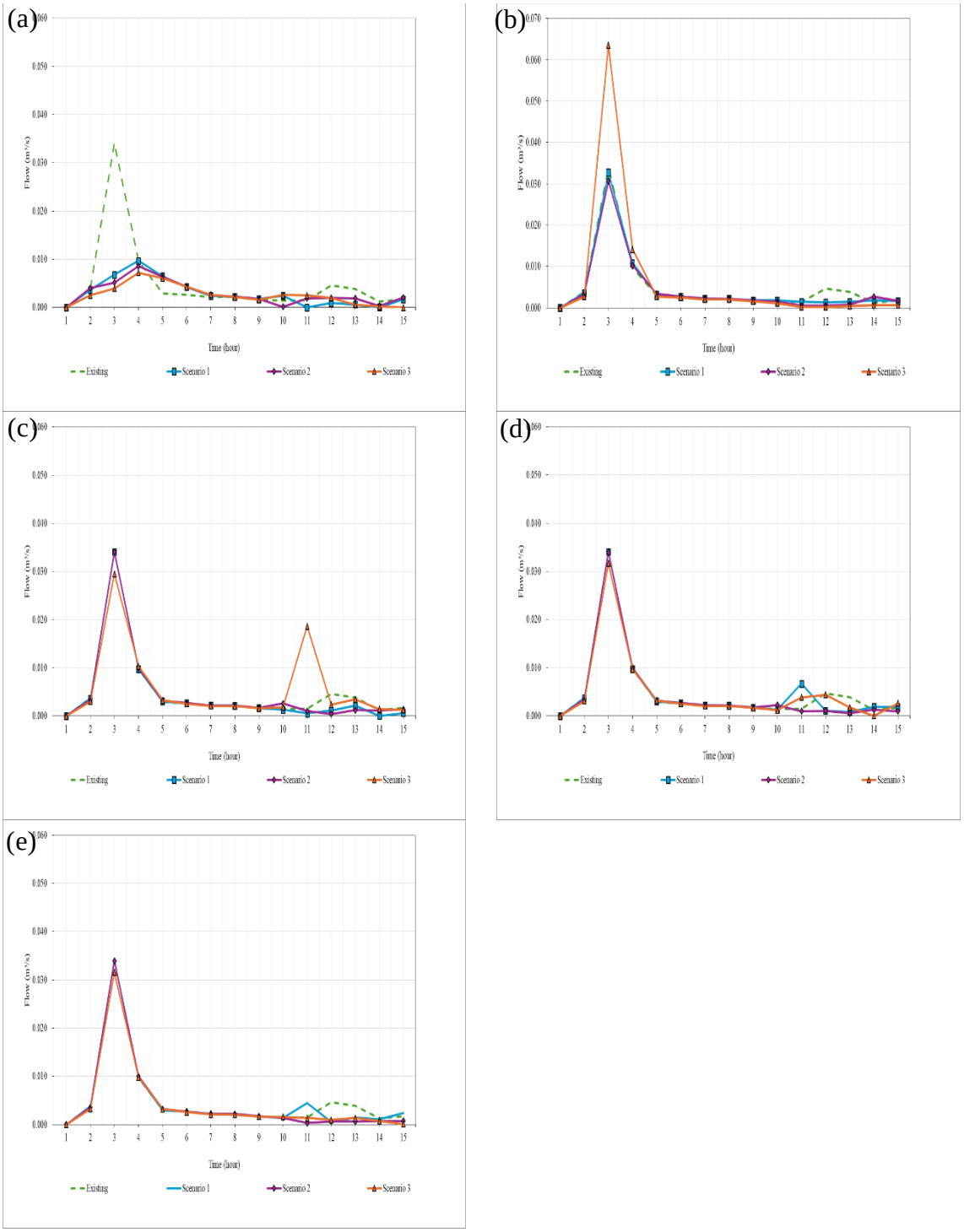


Figure AI.26: Flow Plots for Drain with 0% Slope at Location B Subjected to Event 3, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

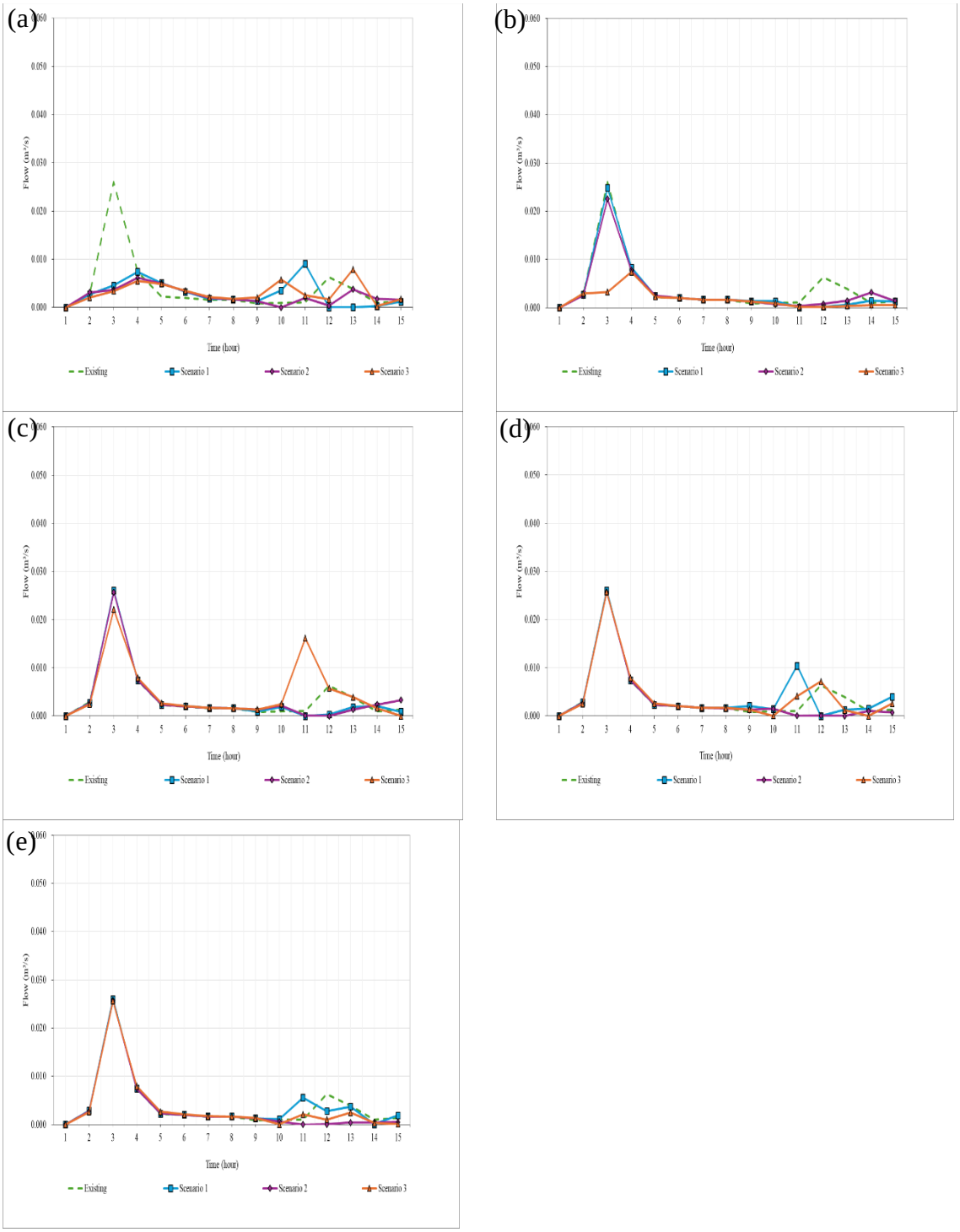


Figure AI.27: Flow Plots for Drain with 0% Slope at Location C Subjected to Event 3, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

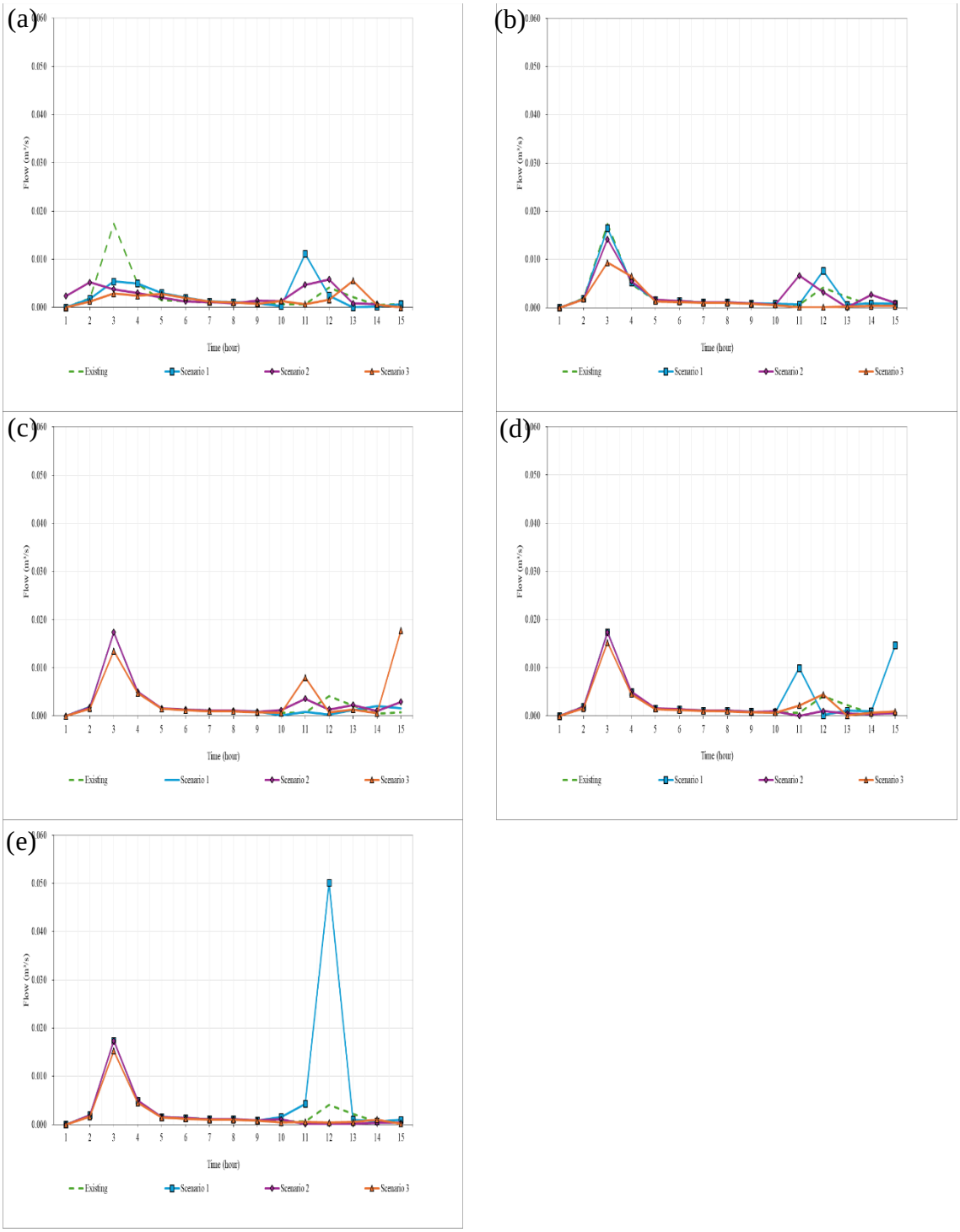


Figure AI.28: Flow Plots for Drain with 0% Slope at Location D Subjected to Event 3, (a) 0.1m, (b) 0. 2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

Event 3 (Water Levels)

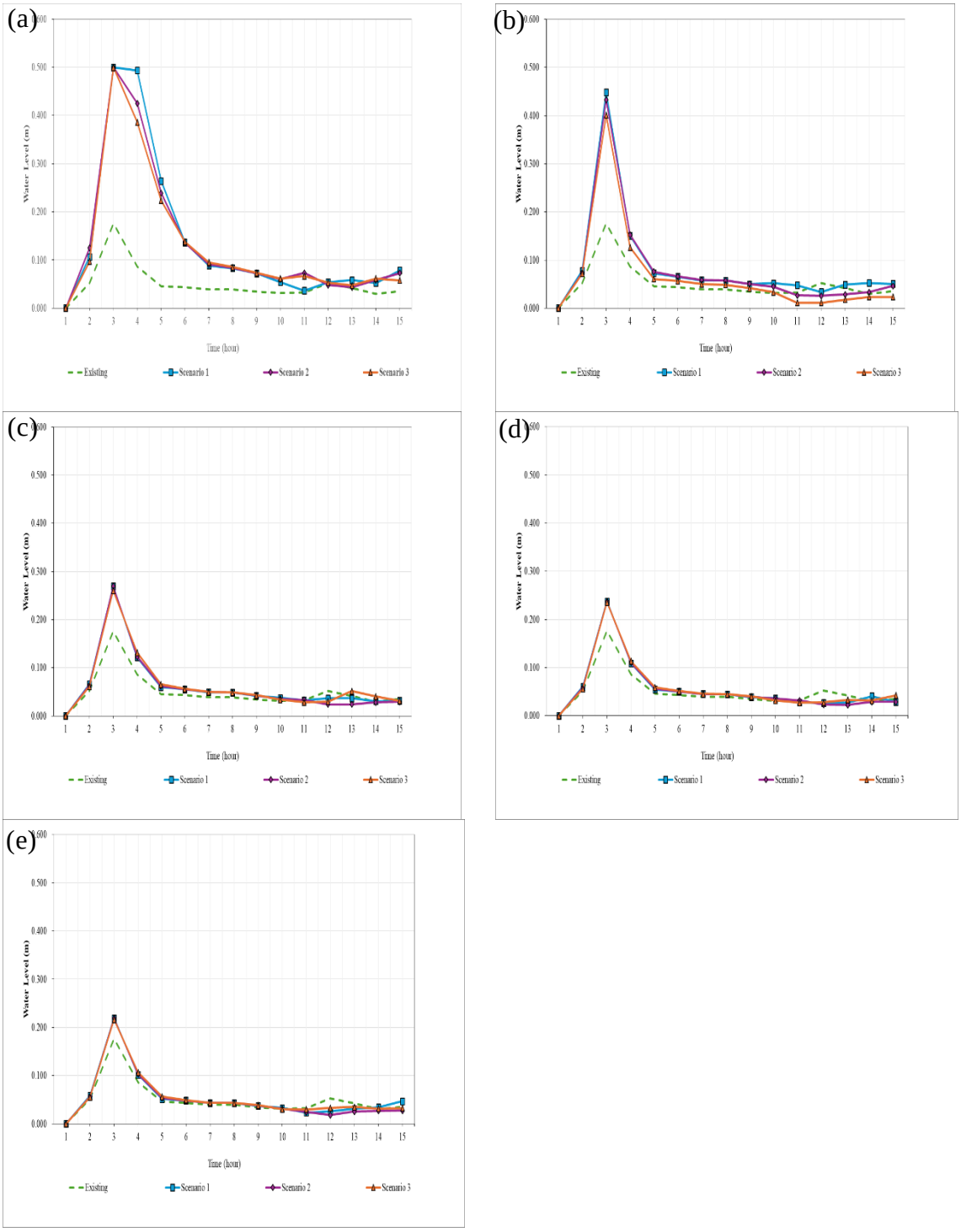


Figure AI.29: Water Level Plots for Drain with 0% Slope at Location A Subjected to Event 3, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

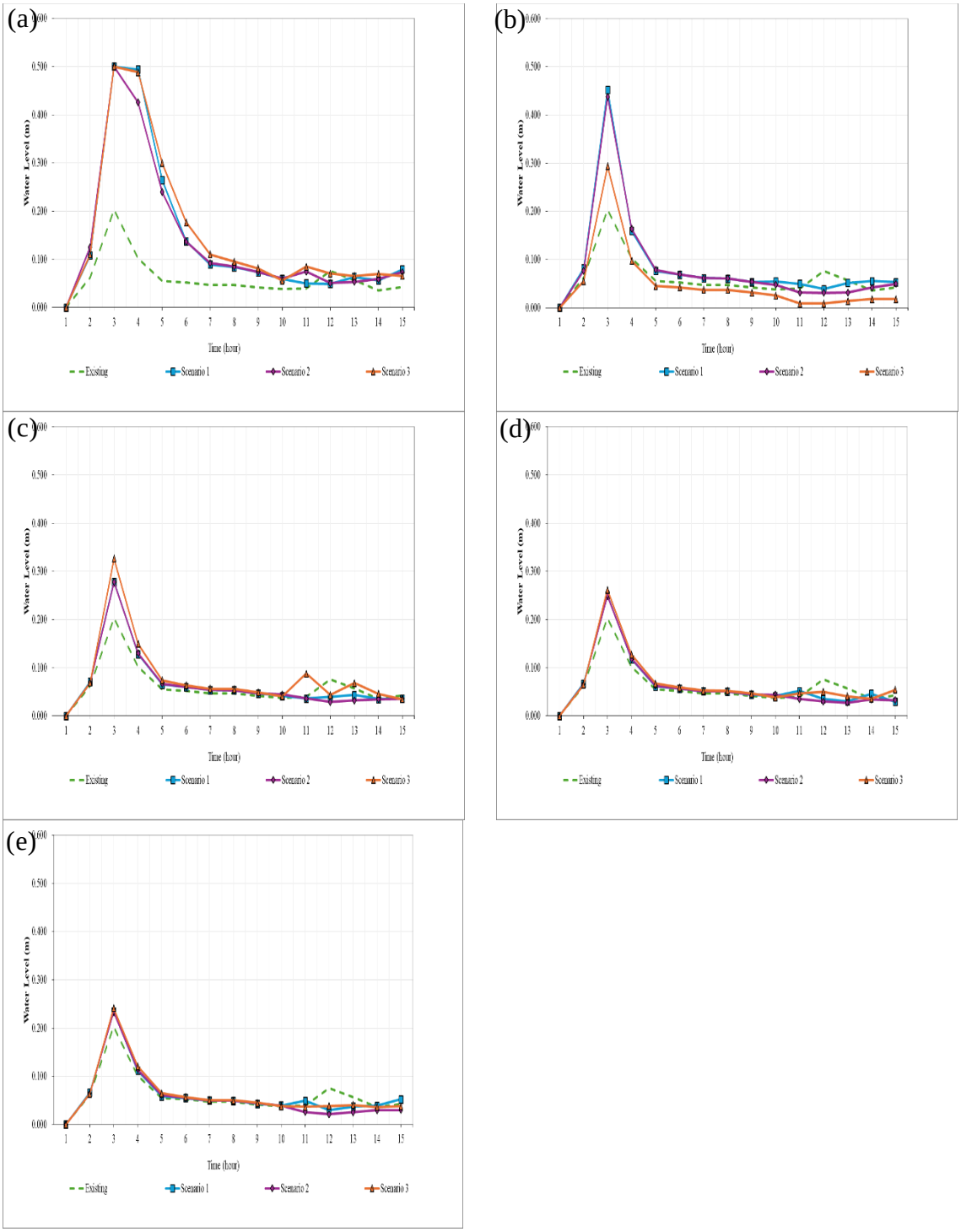


Figure AI.30: Water Level Plots for Drain with 0% Slope at Location B Subjected to Event 3, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

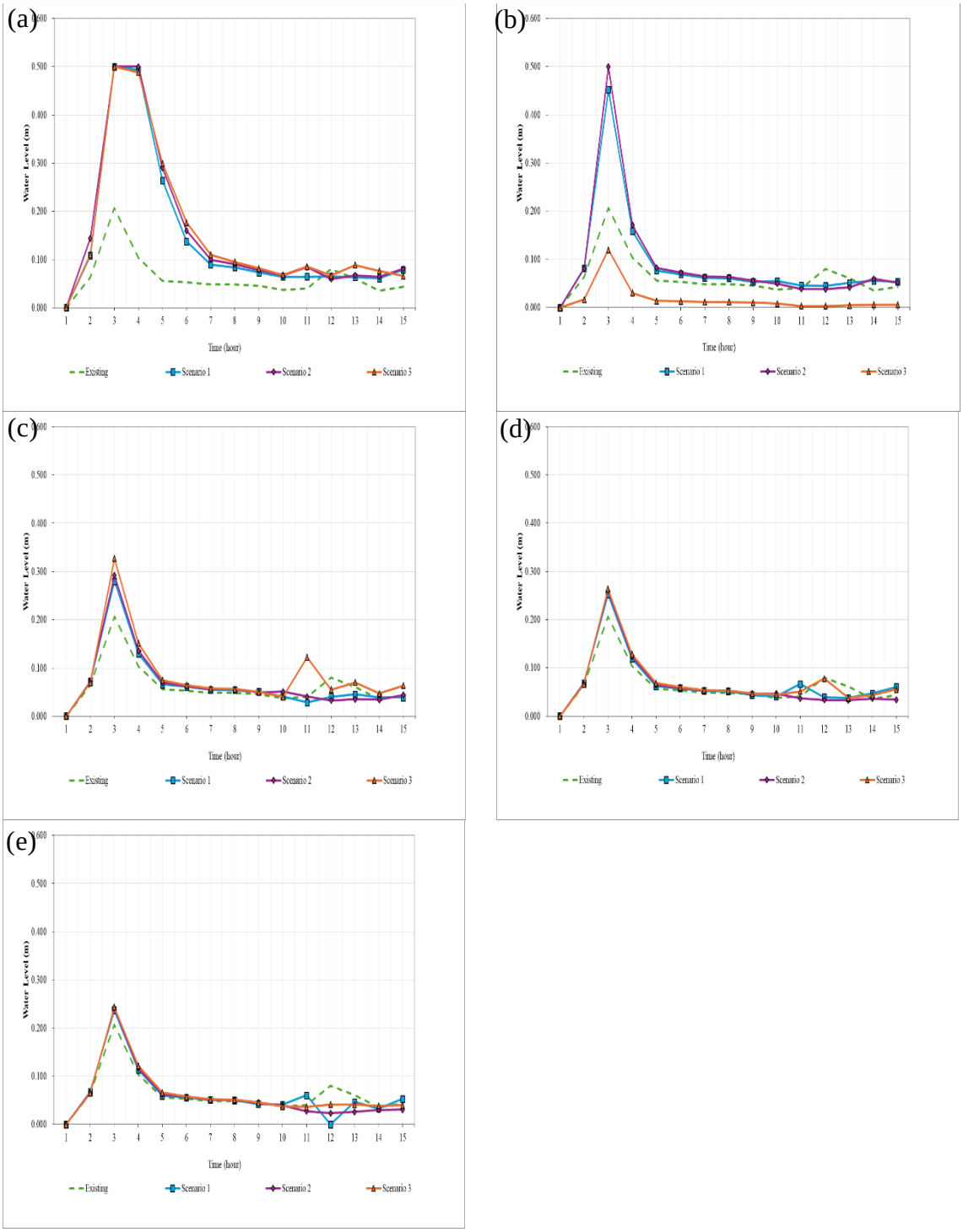


Figure AI.31: Water Level Plots for Drain with 0% Slope at Location C Subjected to Event 3, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

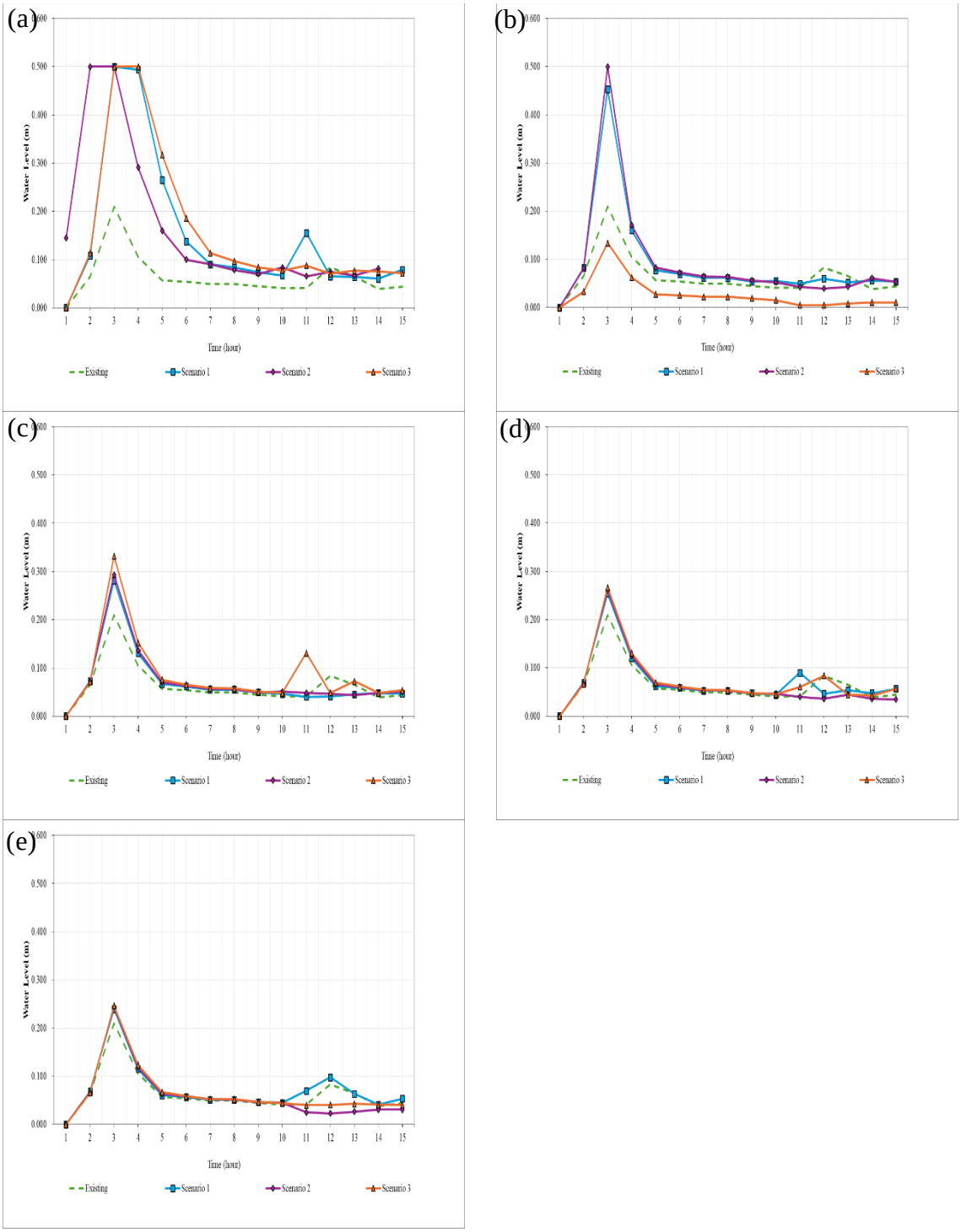


Figure AI.32: Water Level Plots for Drain with 0% Slope at Location D Subjected to Event 3, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

iii. Drain With 0.2% Slope - Design Rainfall

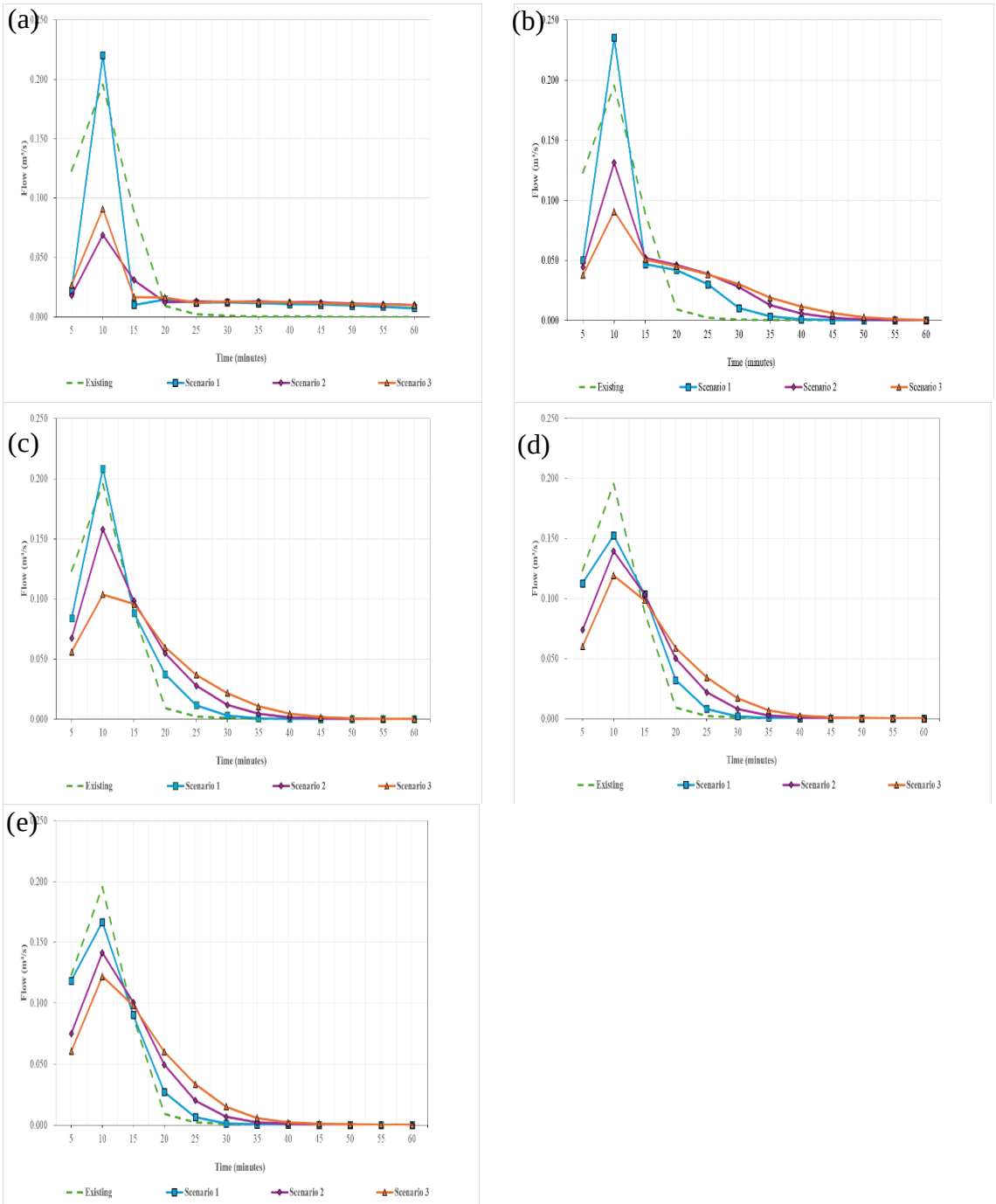


Figure AI.33: Flow Plots for Drain with 0.2% Slope at Location A Subjected to Design Rainfall, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

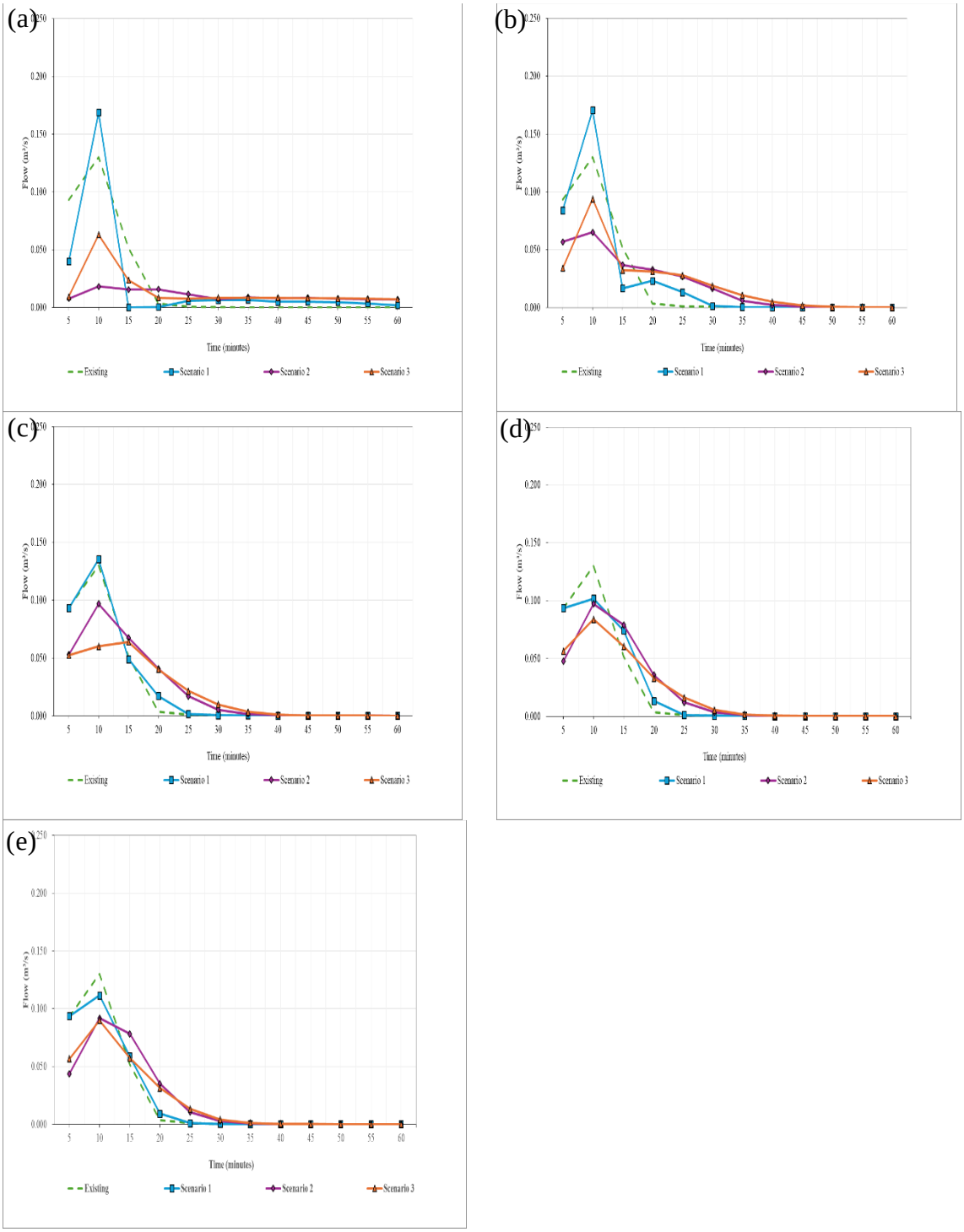


Figure AI.34: Flow Plots for Drain with 0.2% Slope at Location B Subjected to Design Rainfall, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

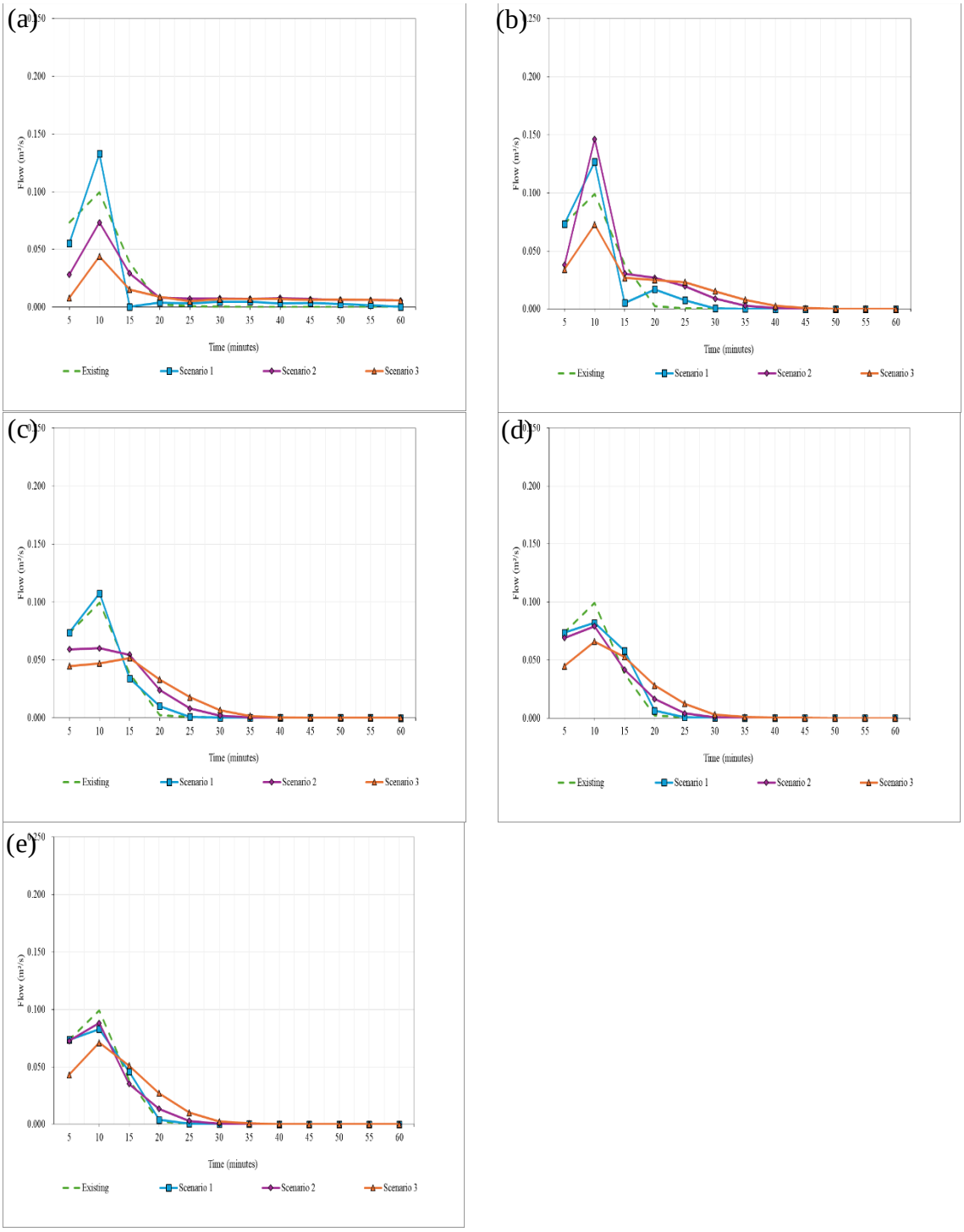


Figure AI.35: Flow Plots for Drain with 0.2% Slope at Location C Subjected to Design Rainfall, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

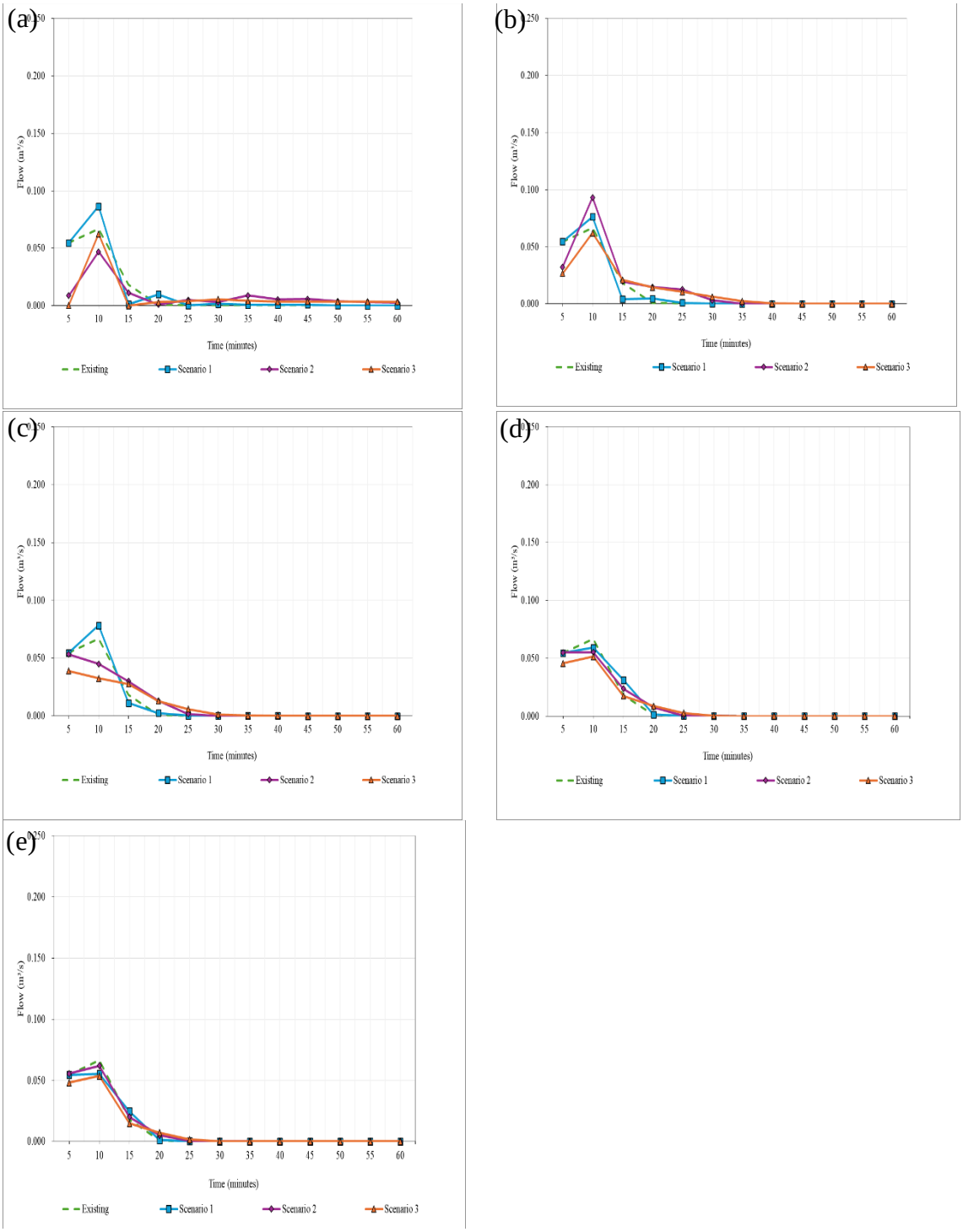


Figure AI.36: Flow Plots for Drain with 0.2% Slope at Location D Subjected to Design Rainfall, (a) 0.1m, (b) 0. 2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

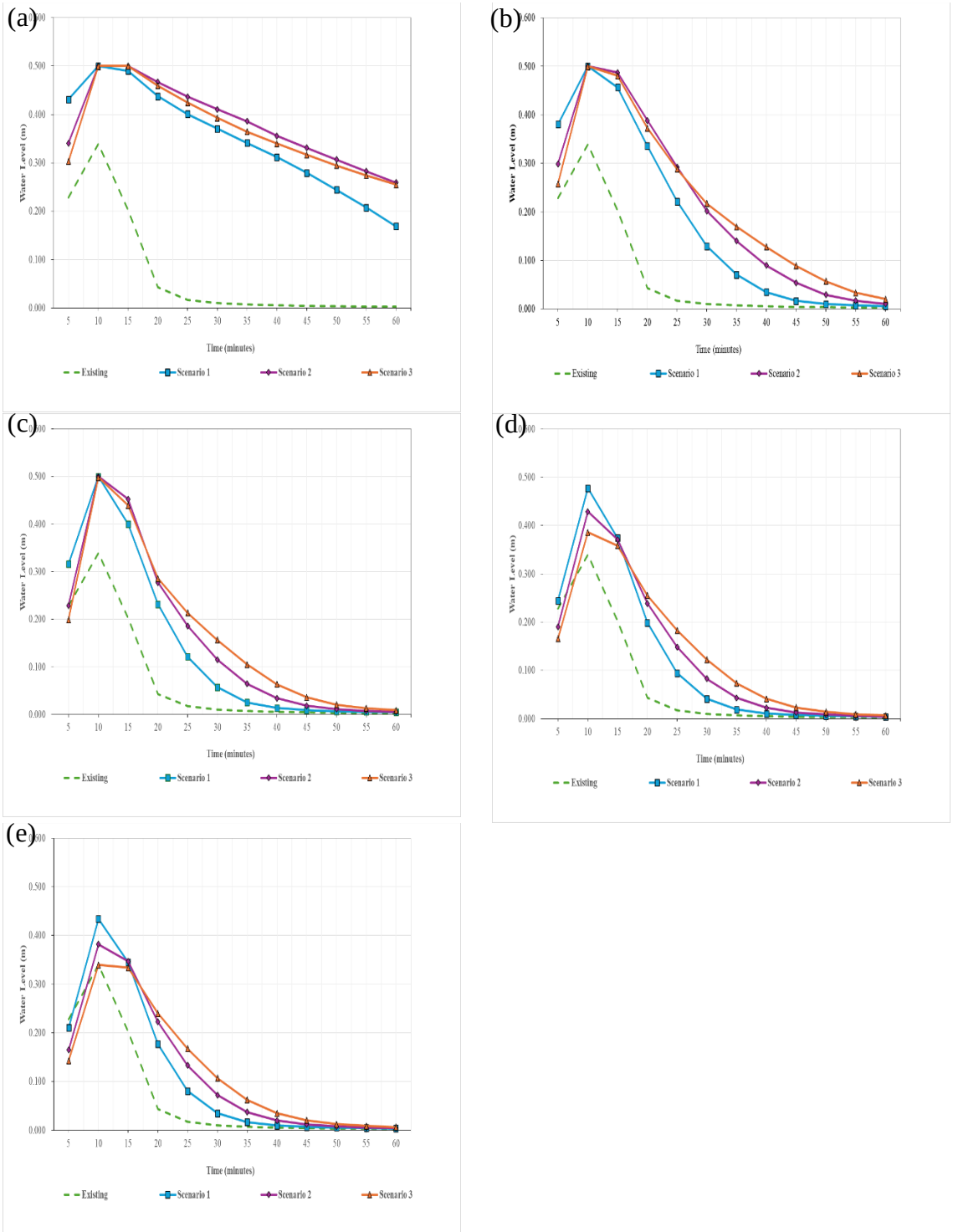


Figure AI.37: Water Level Plots for Drain with 0.2% Slope at Location A Subjected to Design Rainfall, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

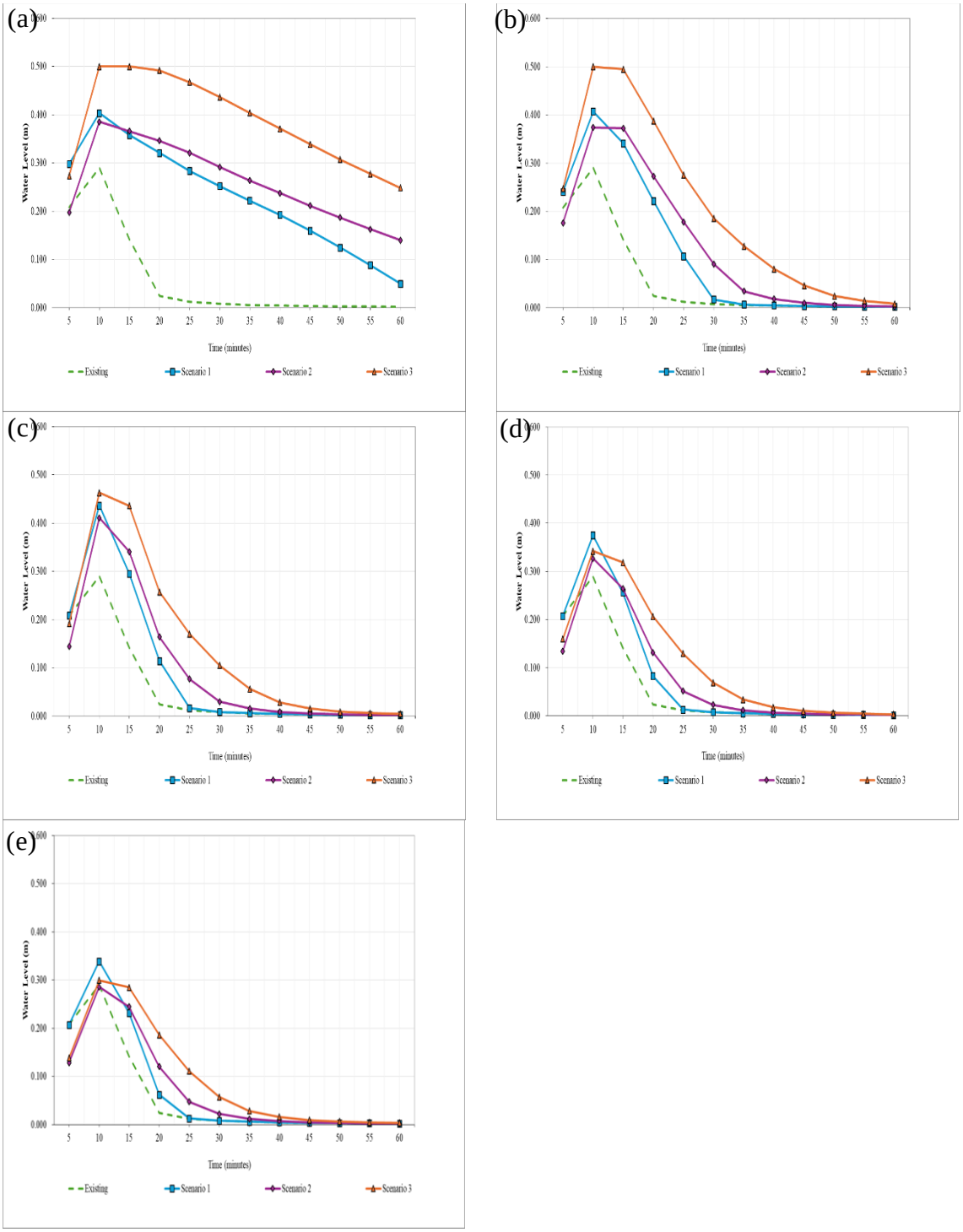


Figure AI.38: Water Level Plots for Drain with 0.2% Slope at Location B Subjected to Design Rainfall, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

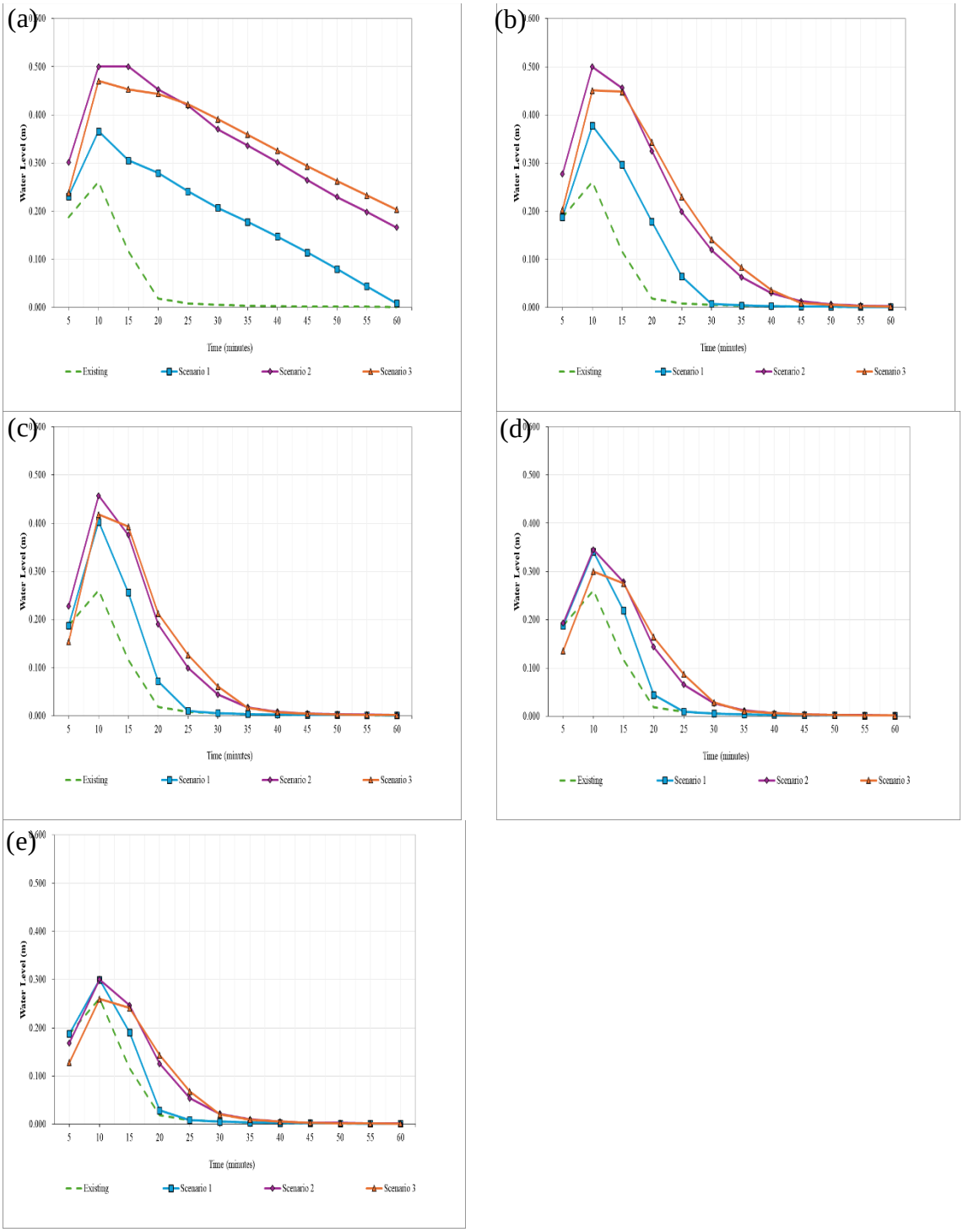


Figure AI.39: Water Level Plots for Drain with 0.2% Slope at Location C Subjected to Design Rainfall, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

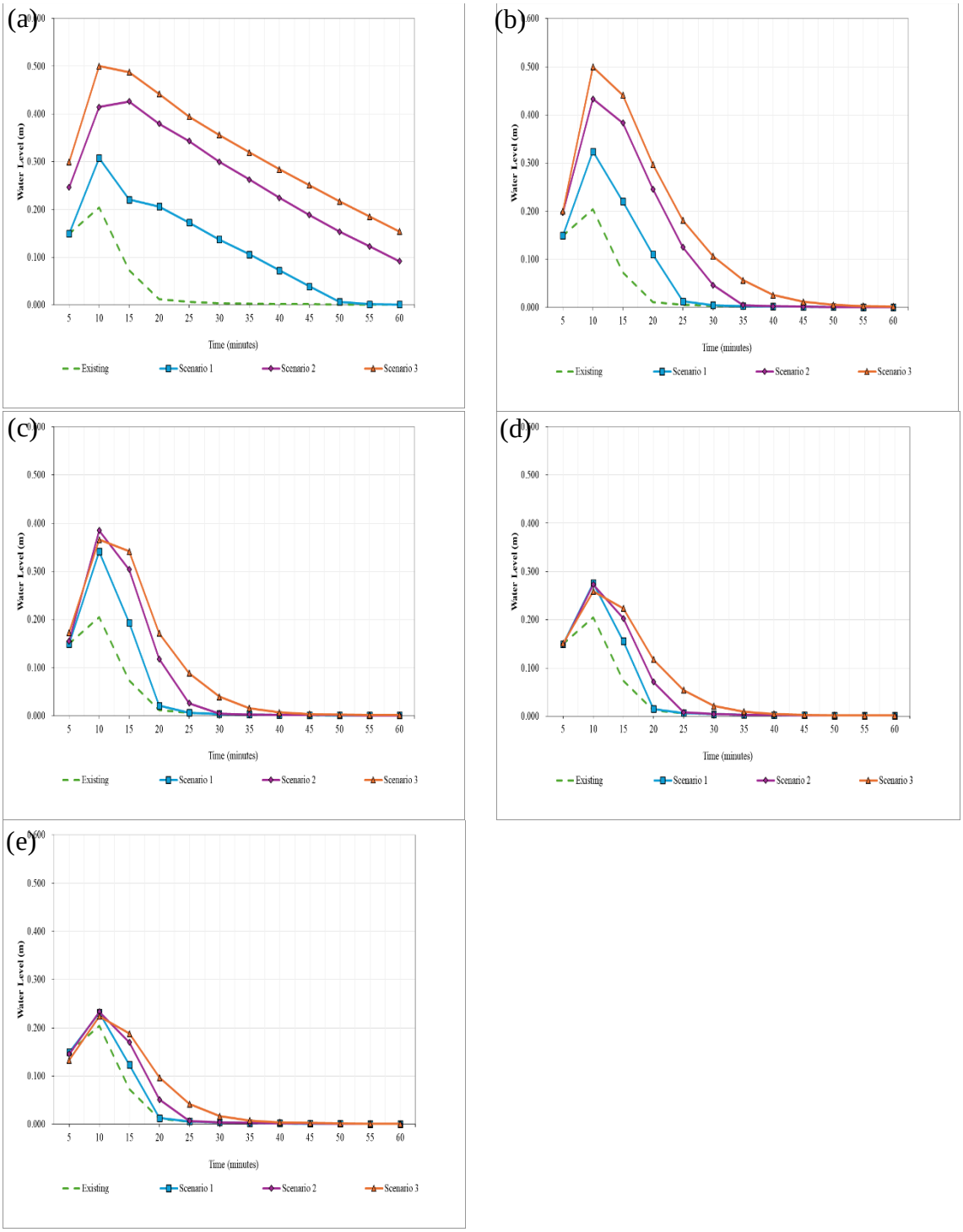


Figure AI.40: Water Level Plots for Drain with 0.2% Slope at Location D Subjected to Design Rainfall, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

iv. Drain With 0.2% Slope - Historical Rainfall

Event 1 (Flow Rates)

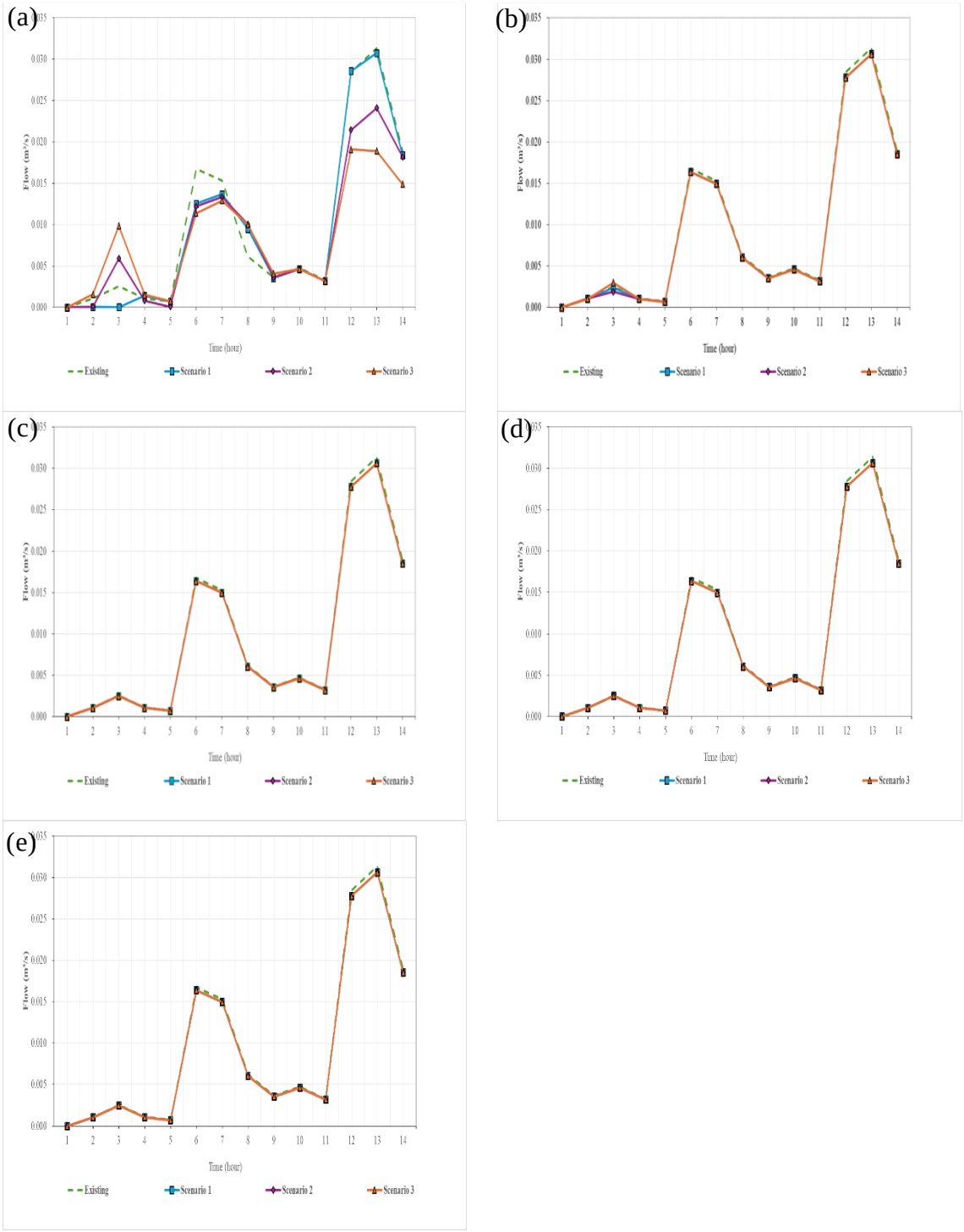


Figure AI.41: Flow Plots for Drain with 0.2% Slope at Location A Subjected to Event 1, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

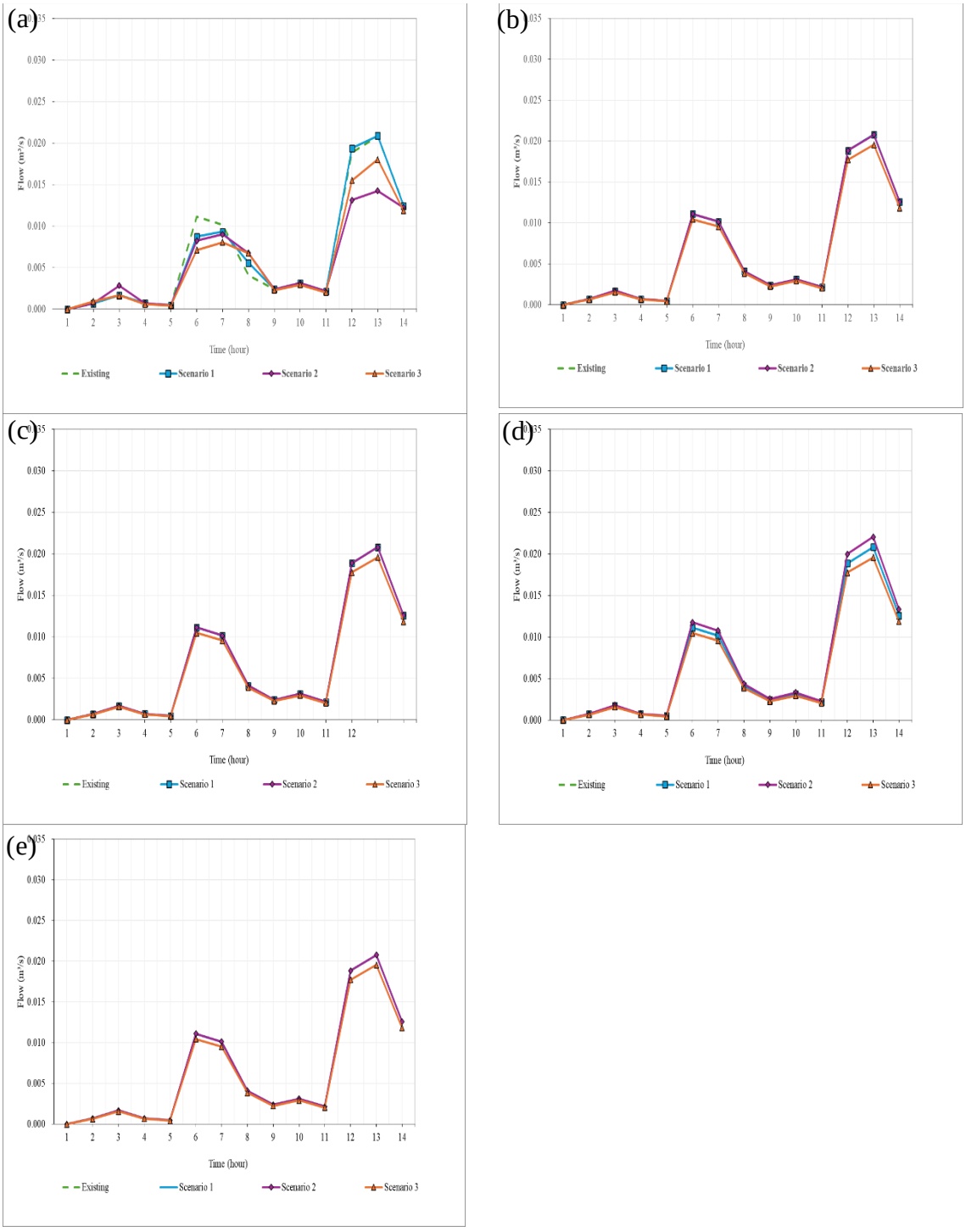


Figure AI.42: Flow Plots for Drain with 0.2% Slope at Location B Subjected to Event 1, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

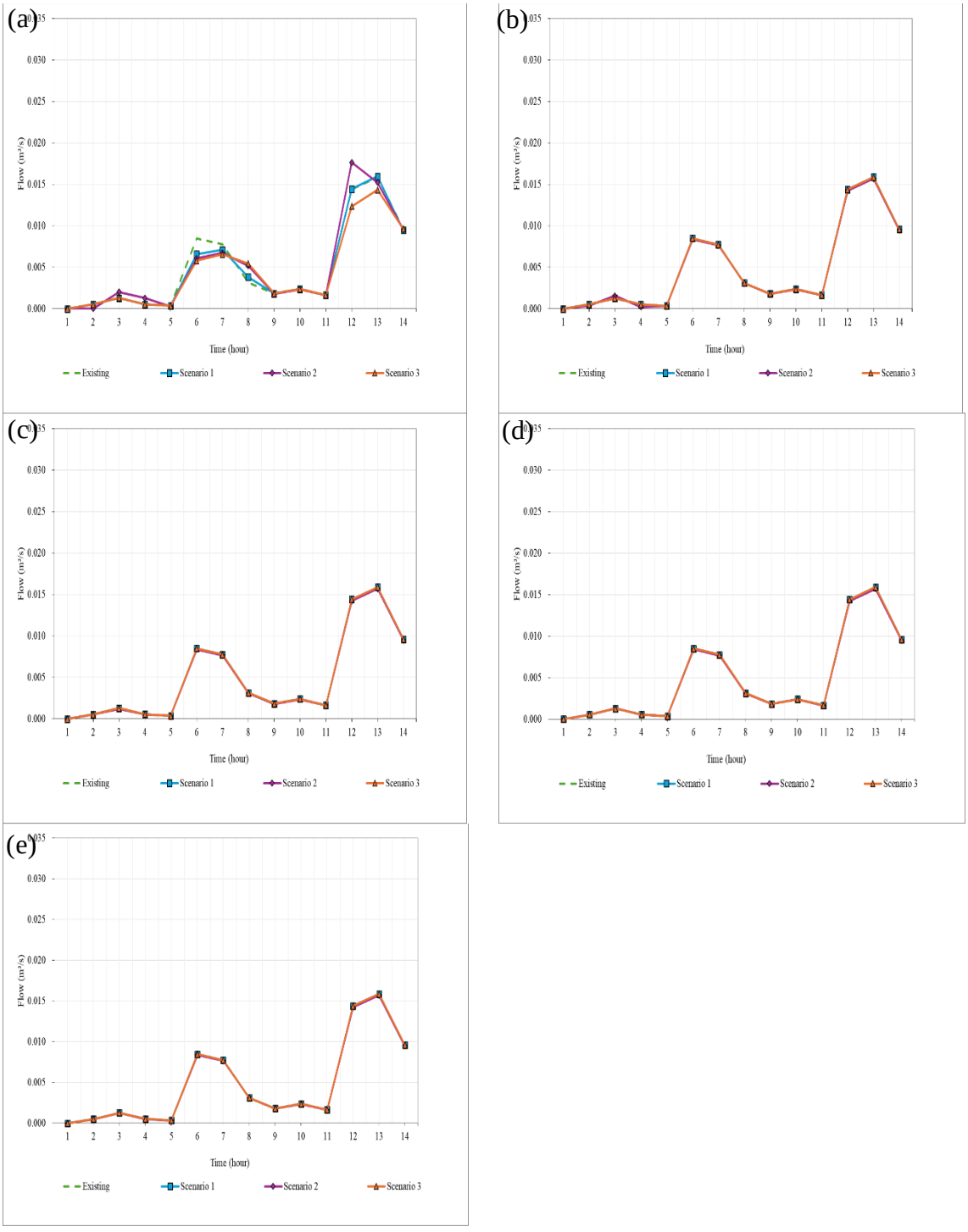


Figure AI.43: Flow Plots for Drain with 0.2% Slope at Location C Subjected to Event 1, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

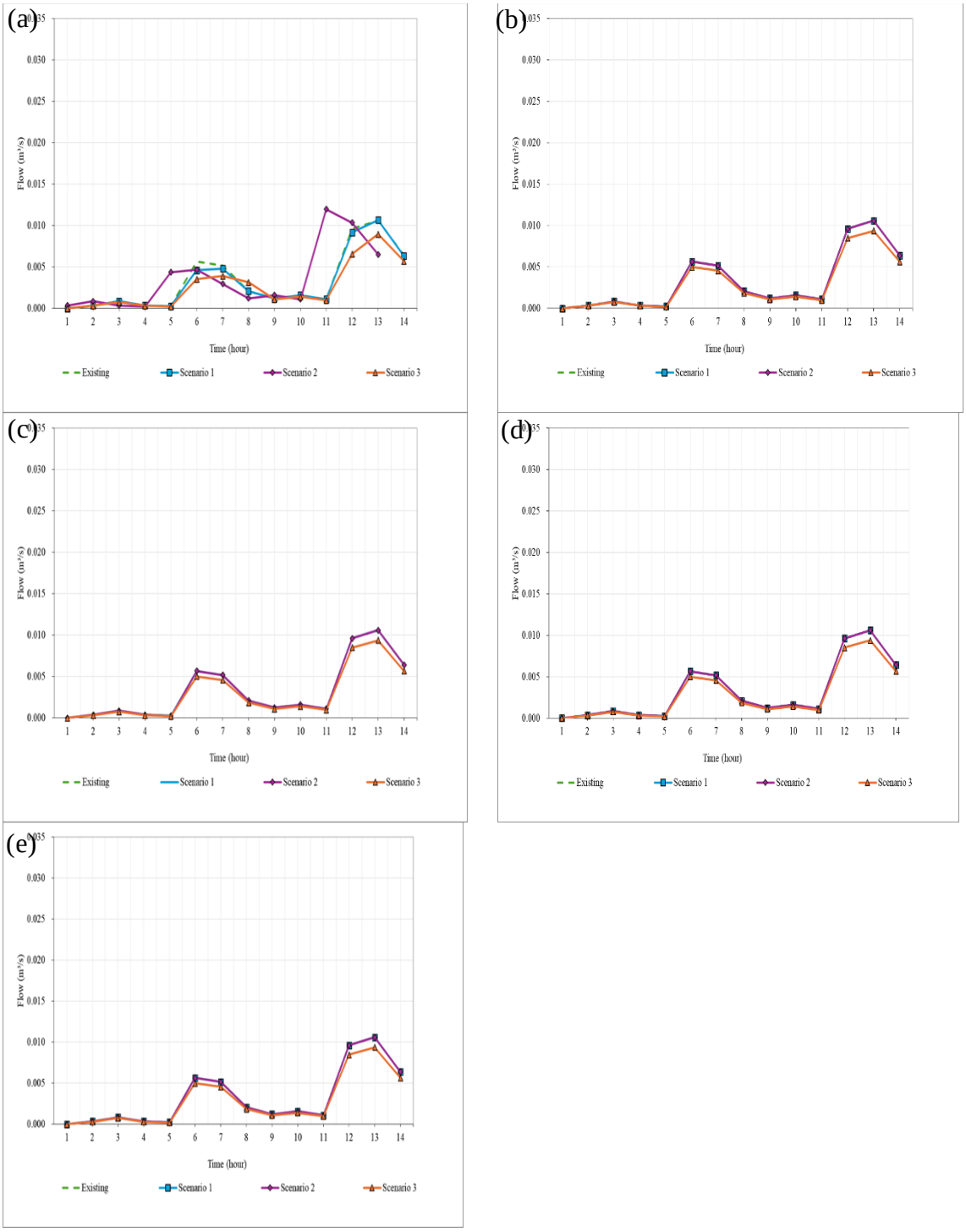


Figure AI.44: Flow Plots for Drain with 0.2% Slope at Location D Subjected to Event 1, (a) 0.1m, (b) 0. 2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

Event 1 (Water Levels)

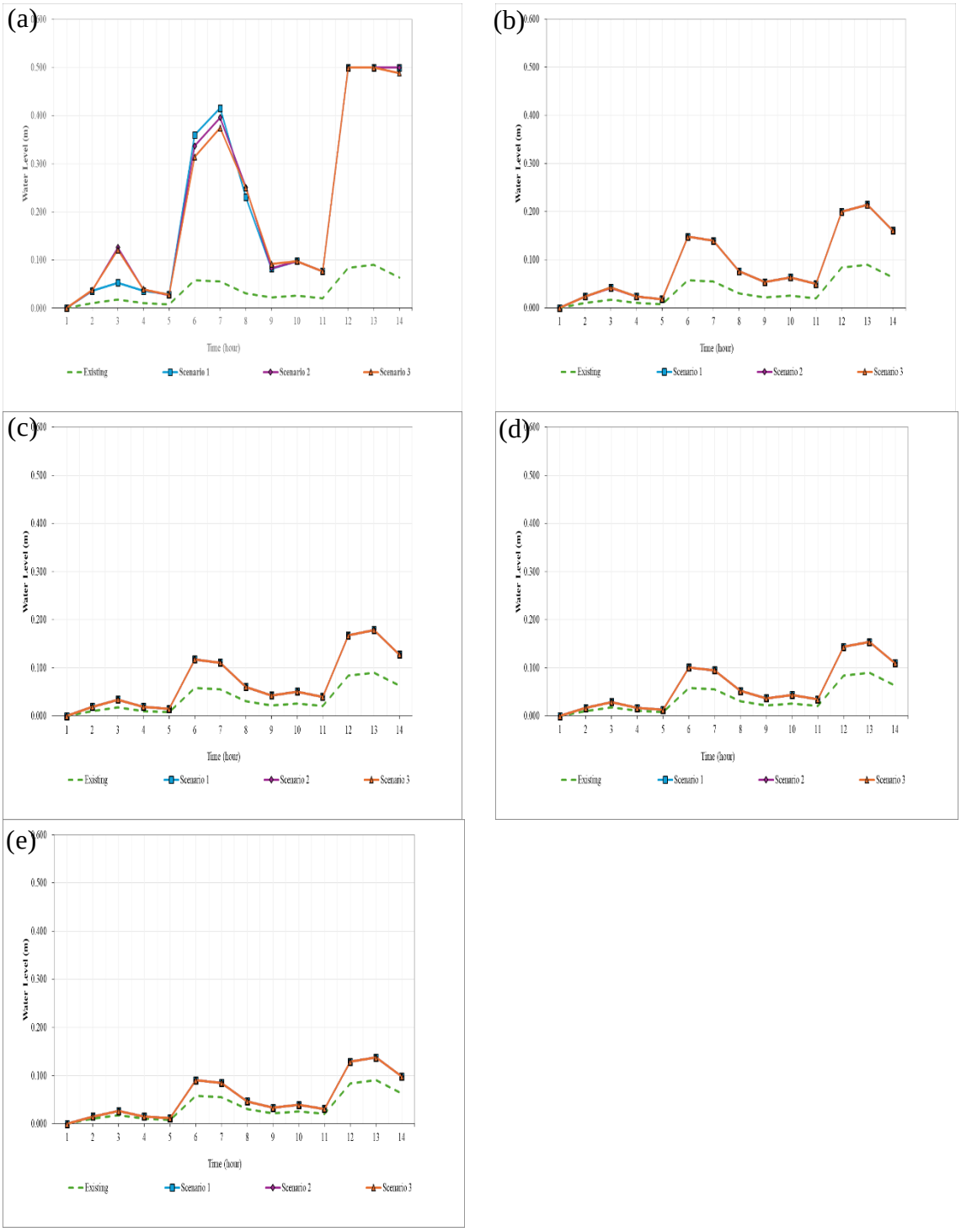


Figure AI.45: Water Level Plots for Drain with 0.2% Slope at Location A Subjected to Event 1, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

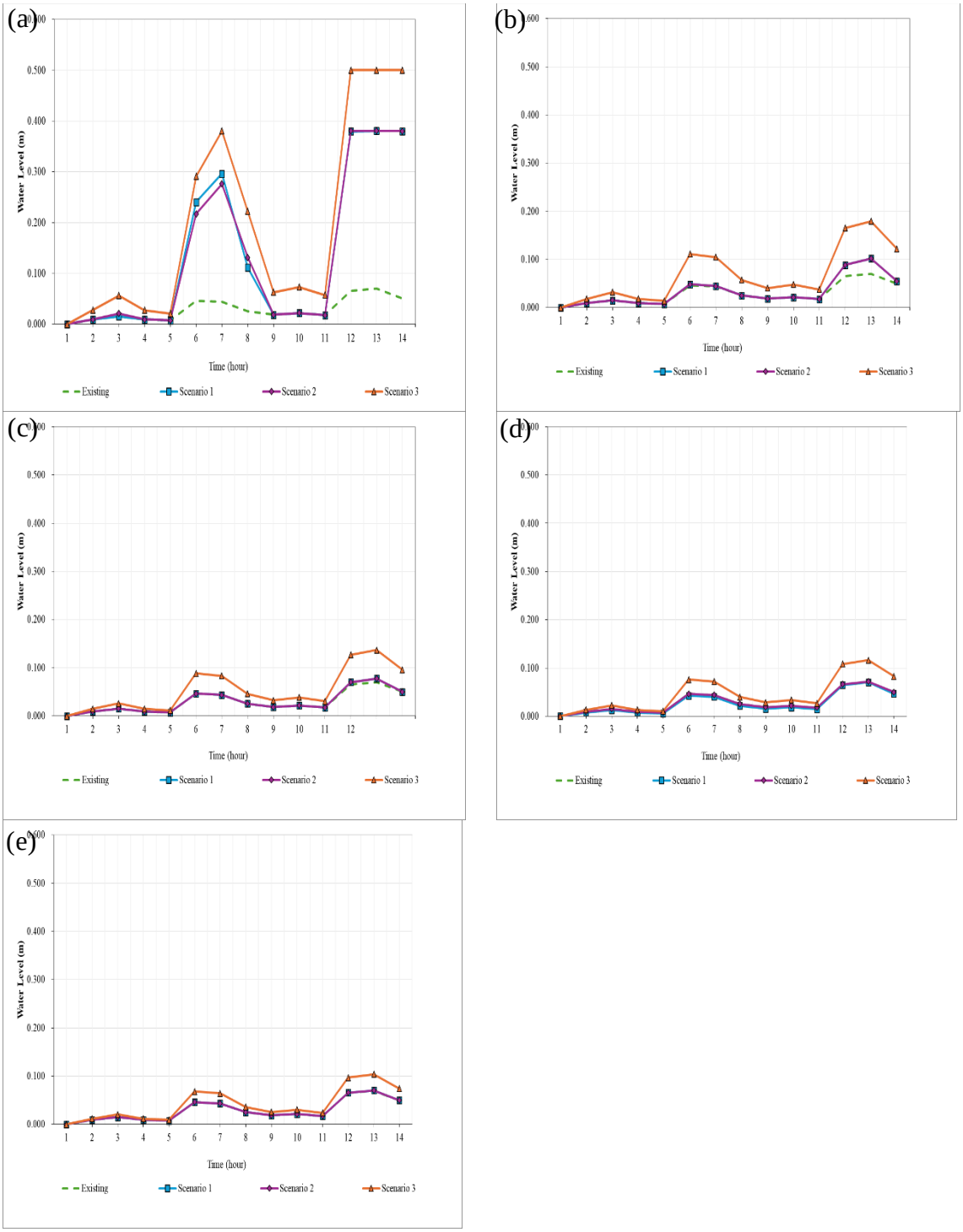


Figure AI.46: Water Level Plots for Drain with 0.2% Slope at Location B Subjected to Event 1, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

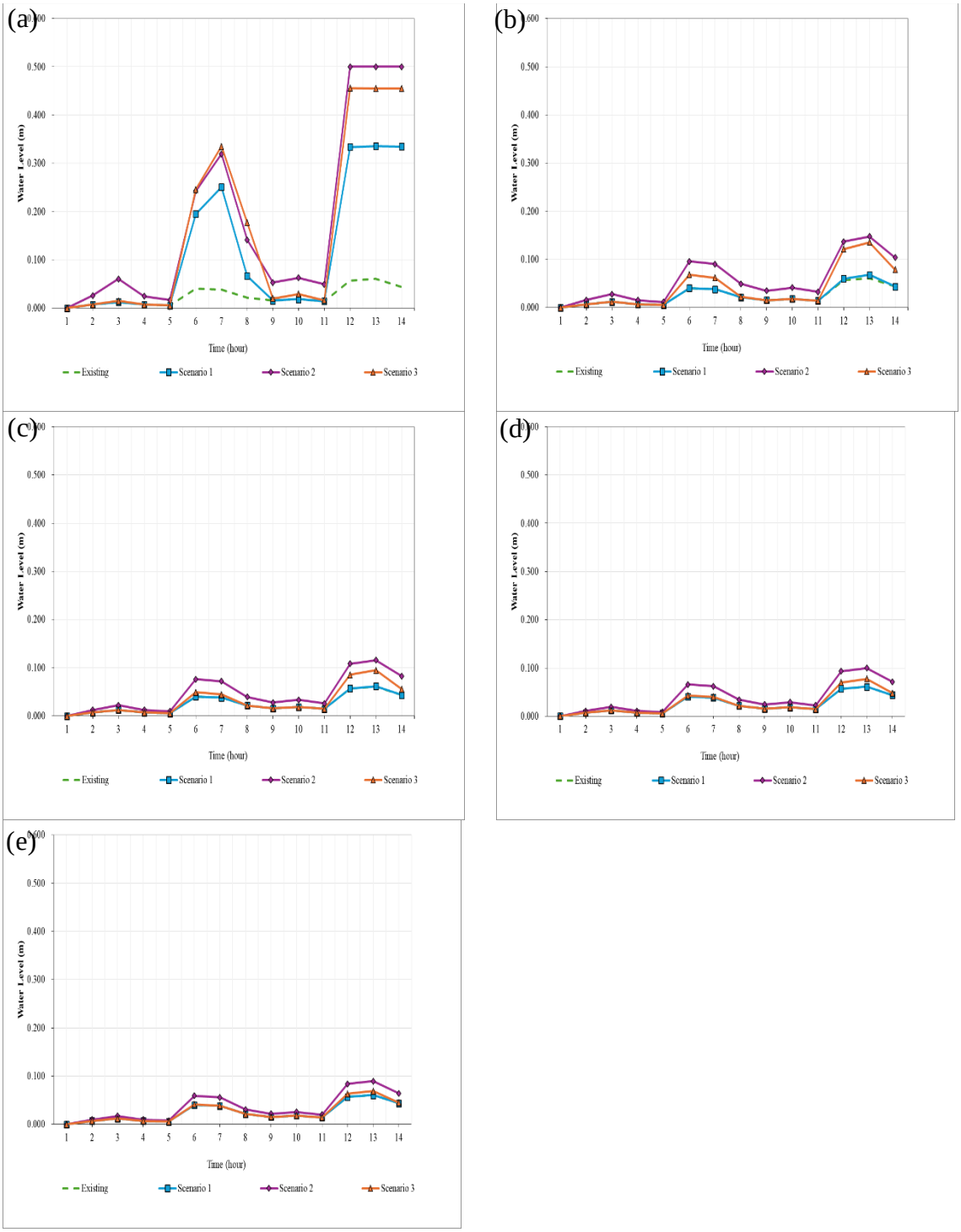


Figure AI.47: Water Level Plots for Drain with 0.2% Slope at Location C Subjected to Event 1, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

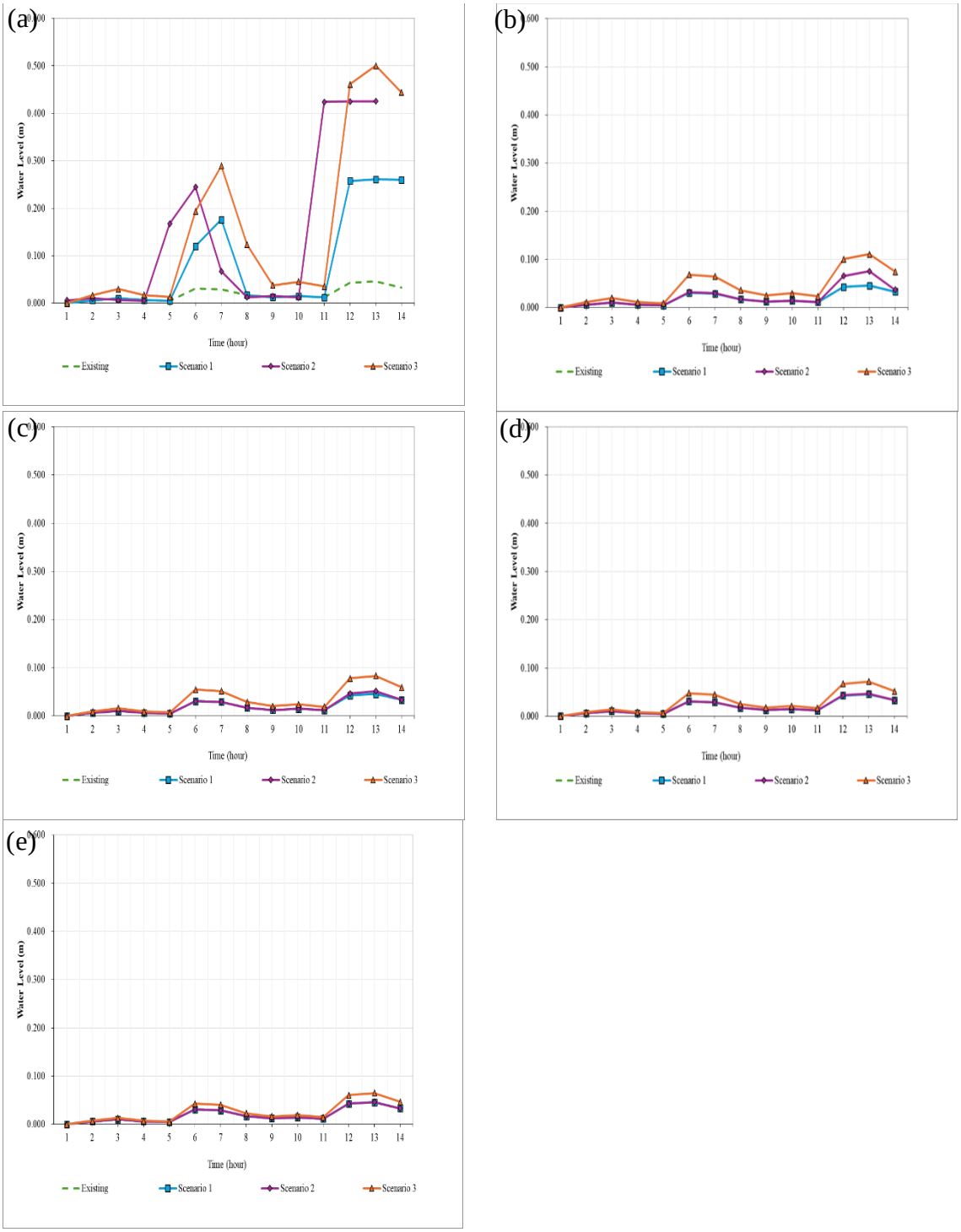


Figure AI.48: Water Level Plots for Drain with 0.2% Slope at Location D Subjected to Event 1, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

Event 2 (Flow Rates)

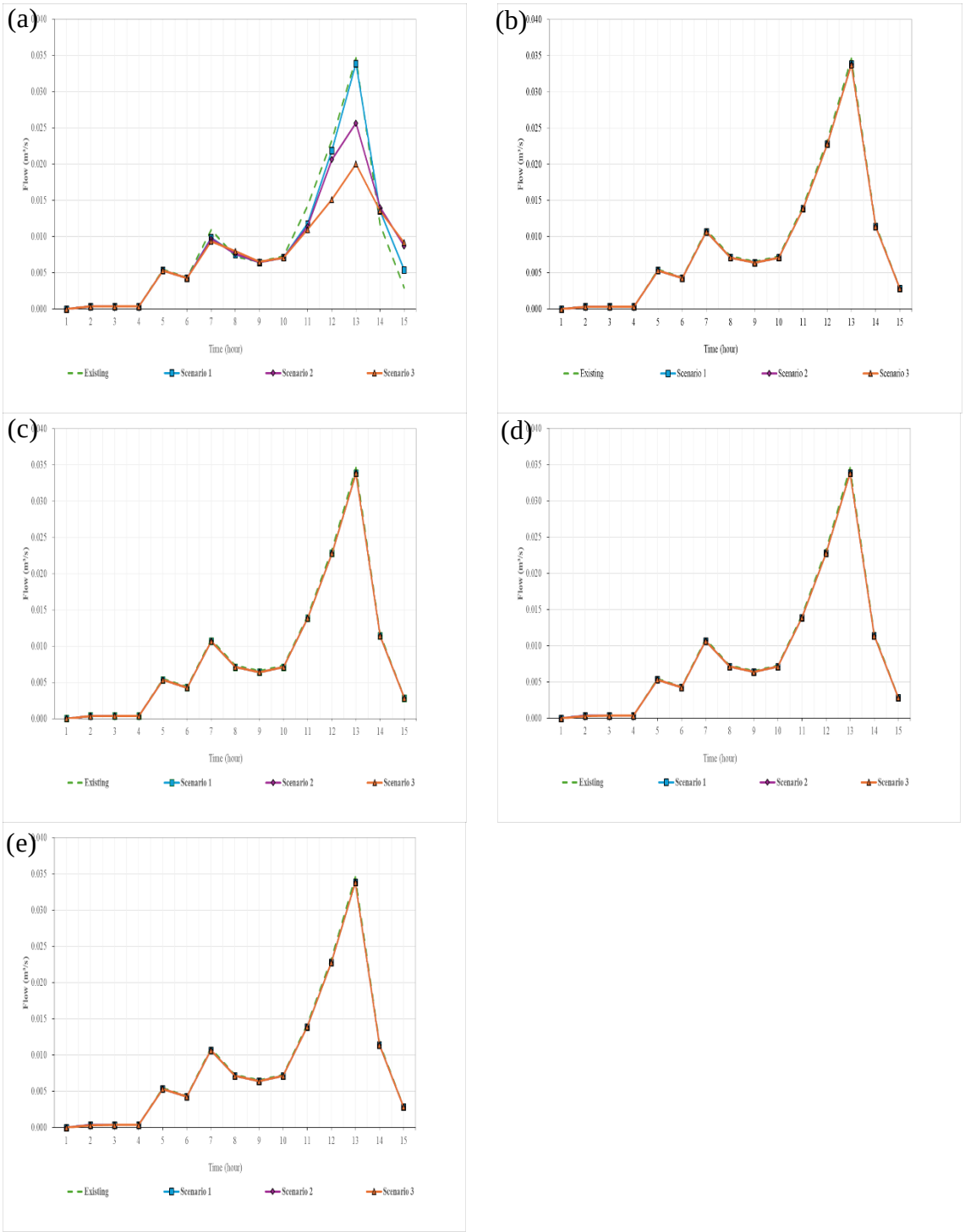


Figure AI.49: Flow Plots for Drain with 0.2% Slope at Location A Subjected to Event 2, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

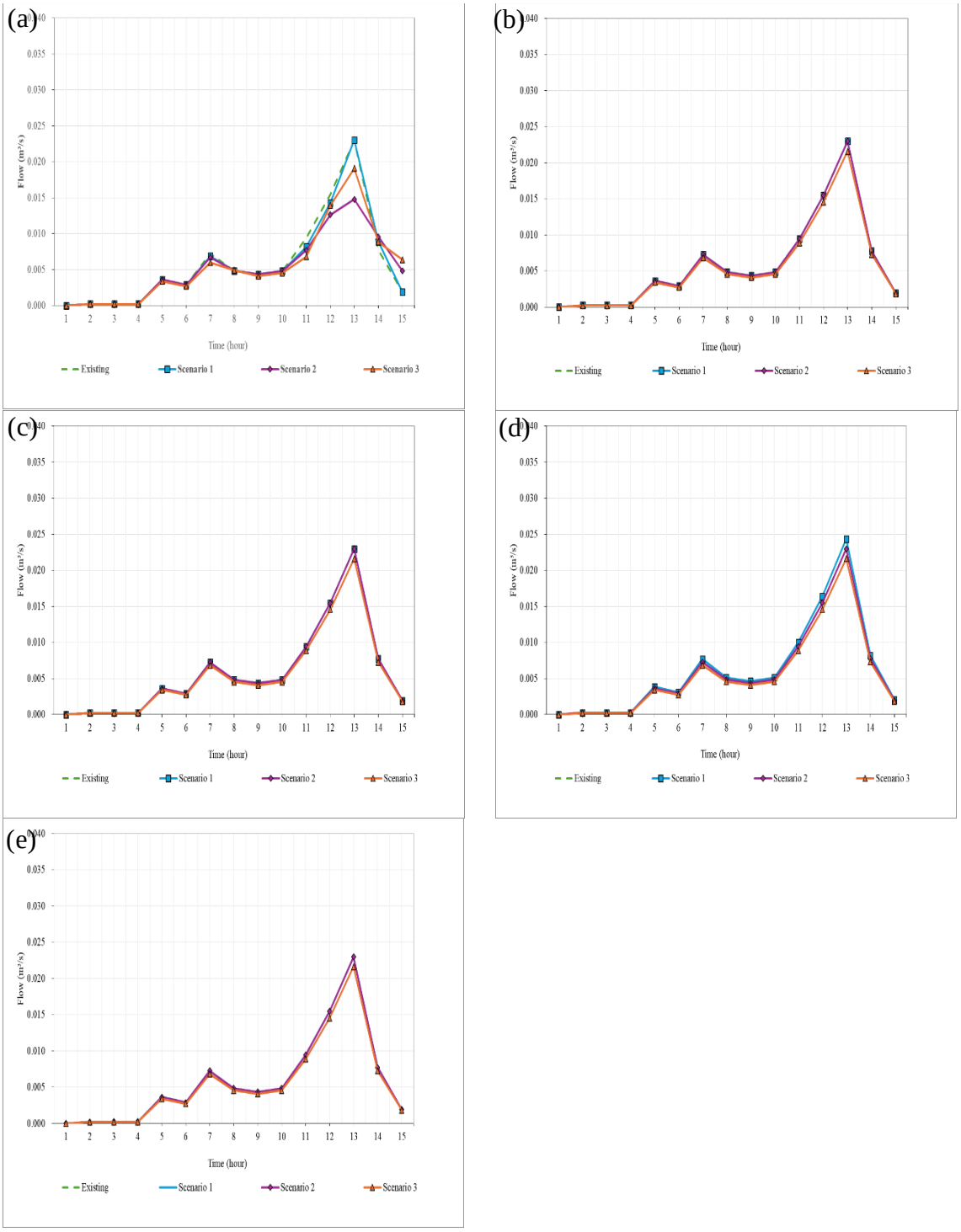


Figure AI.50: Flow Plots for Drain with 0.2% Slope at Location B Subjected to Event 2, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

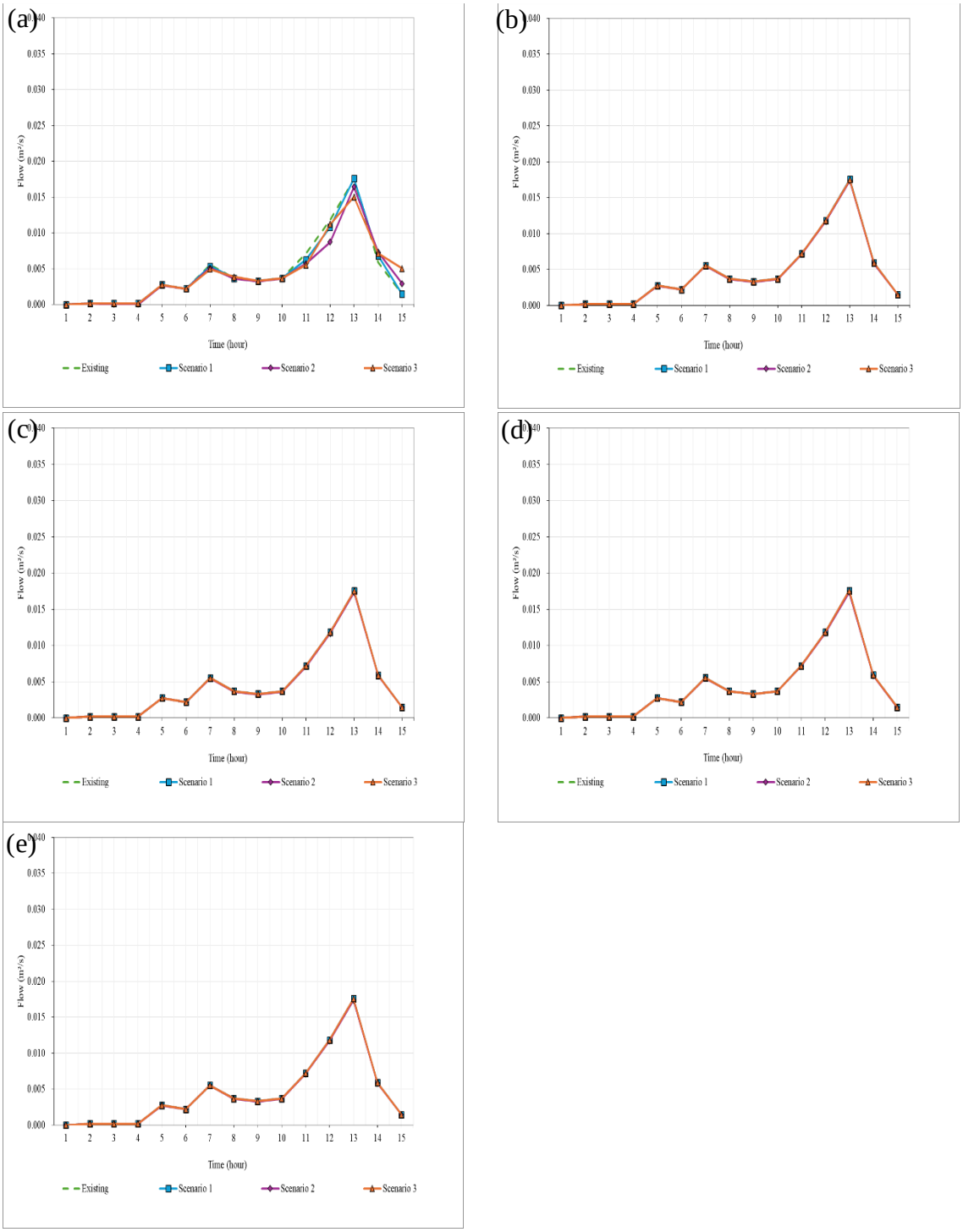


Figure AI.51: Flow Plots for Drain with 0.2% Slope at Location C Subjected to Event 2, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

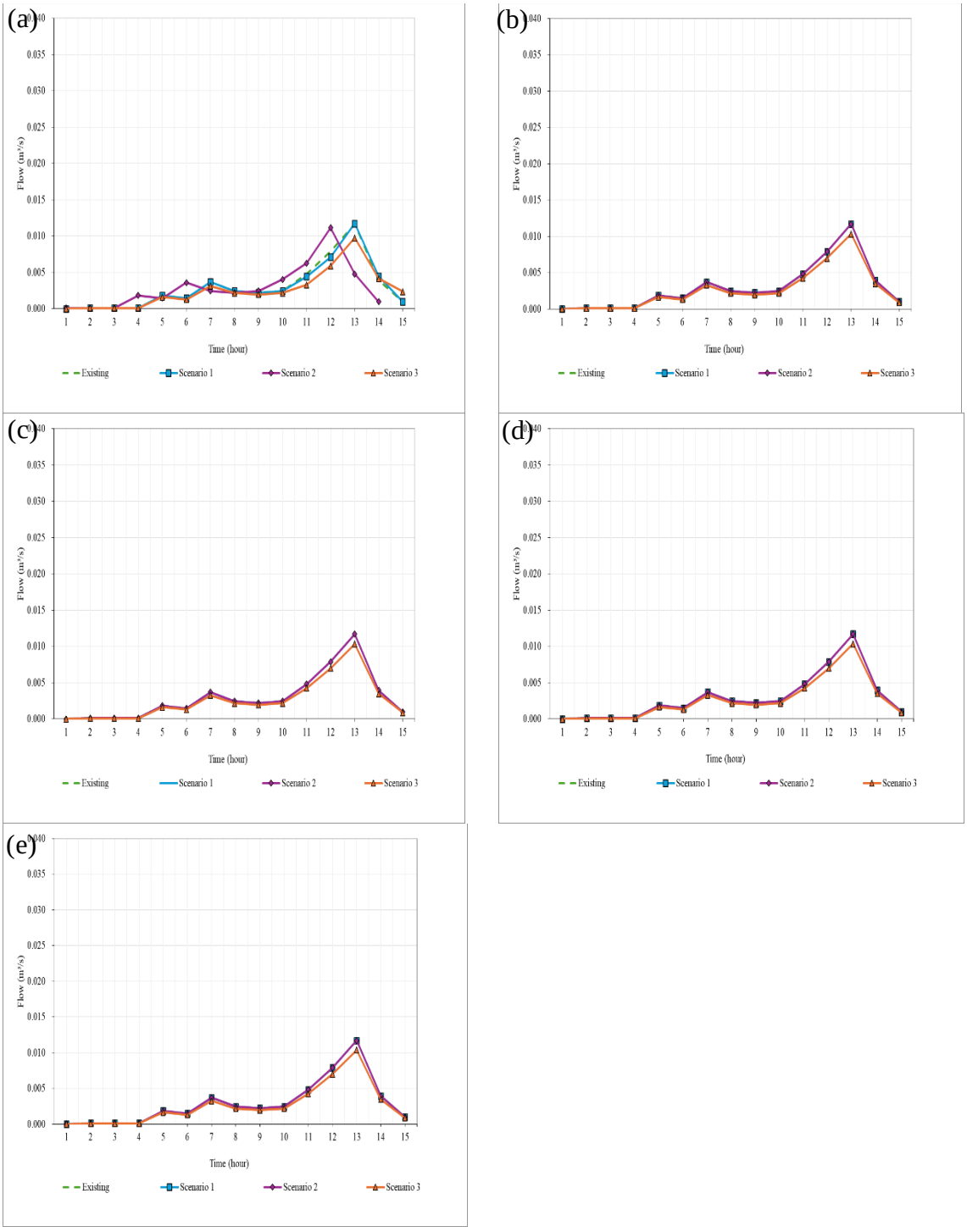


Figure AI.52: Flow Plots for Drain with 0.2% Slope at Location D Subjected to Event 2, (a) 0.1m, (b) 0. 2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

Event 2 (Water Levels)

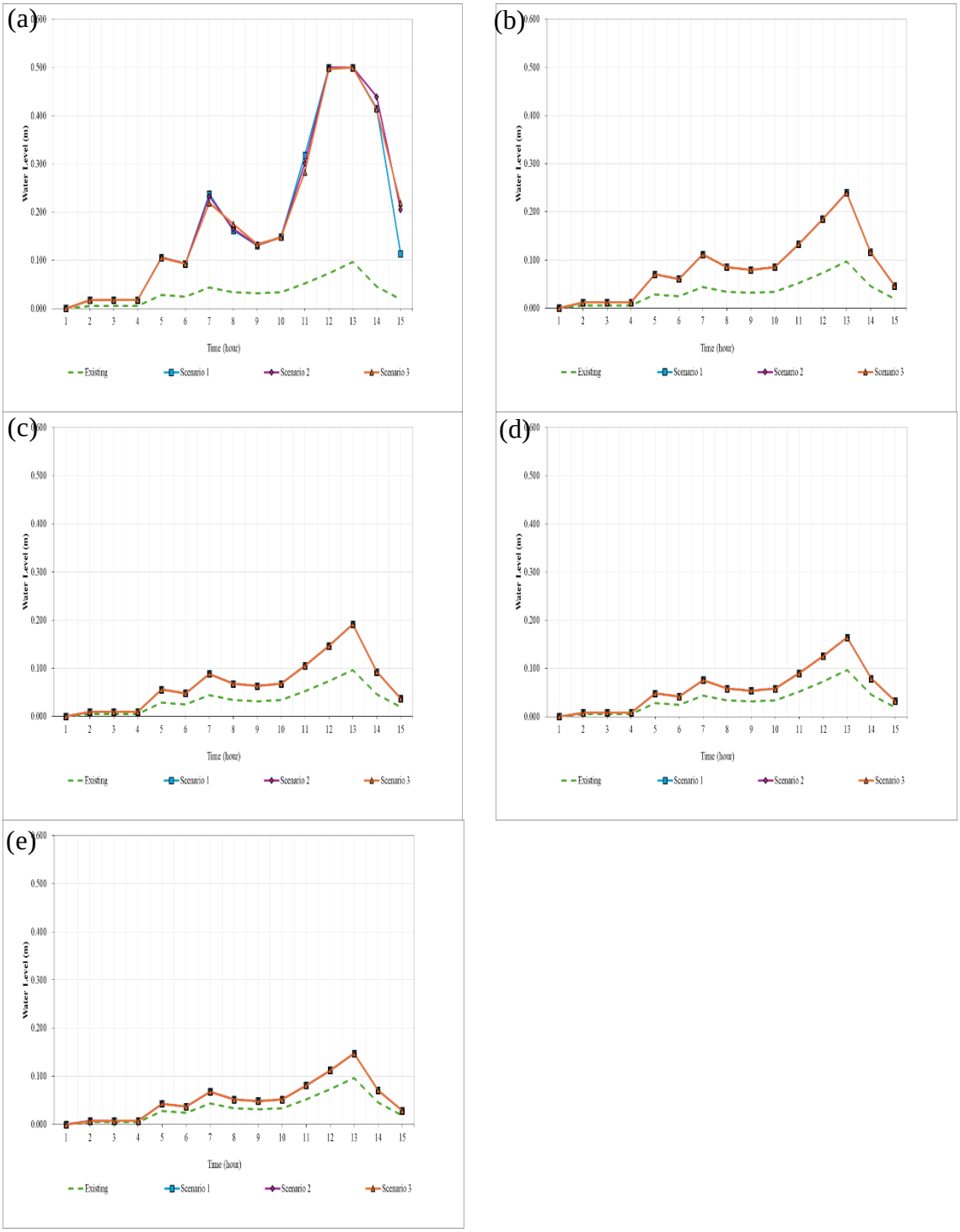


Figure AI.53: Water Level Plots for Drain with 0.2% Slope at Location A Subjected to Event 2, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

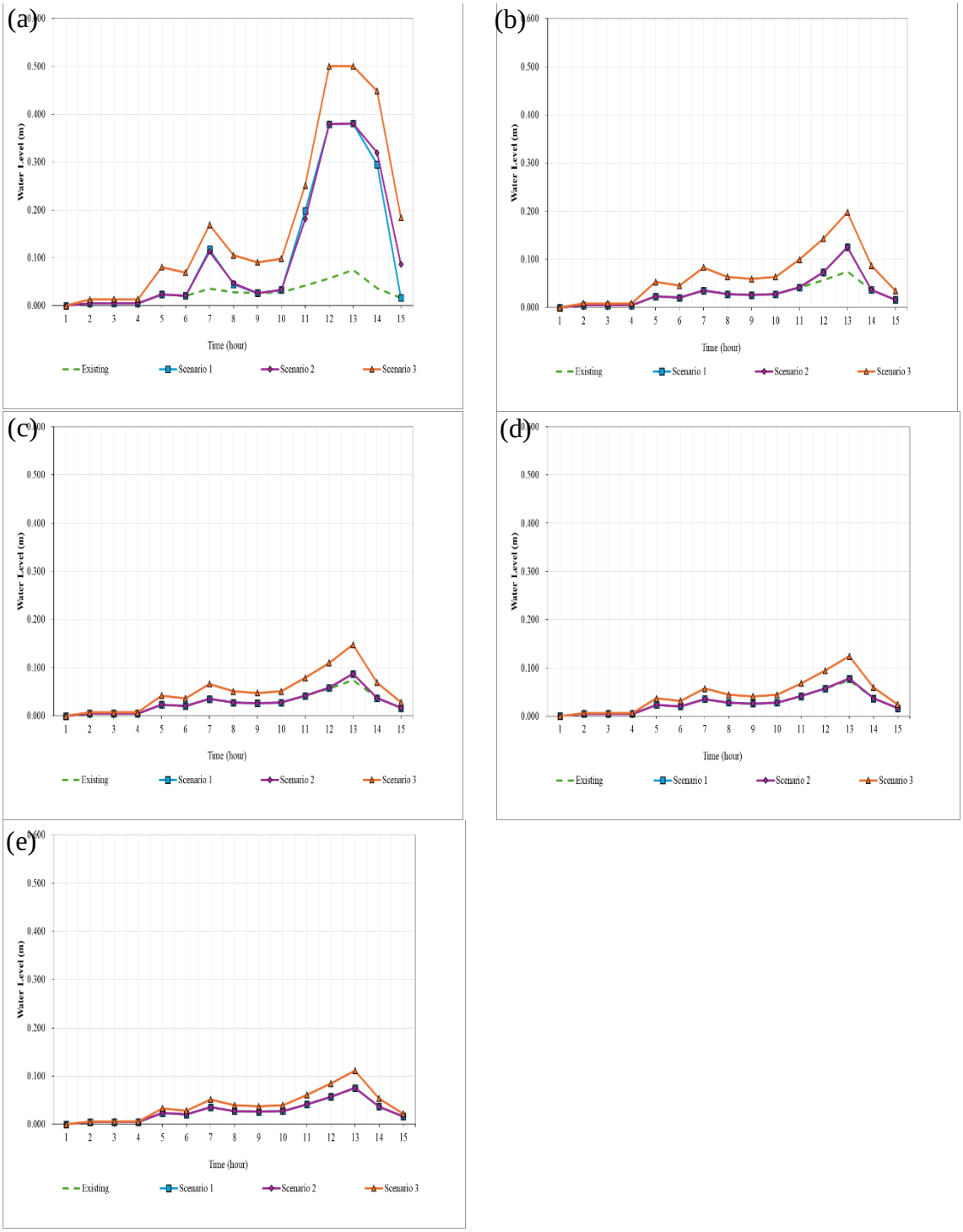


Figure AI.54: Water Level Plots for Drain with 0.2% Slope at Location B Subjected to Event 2, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

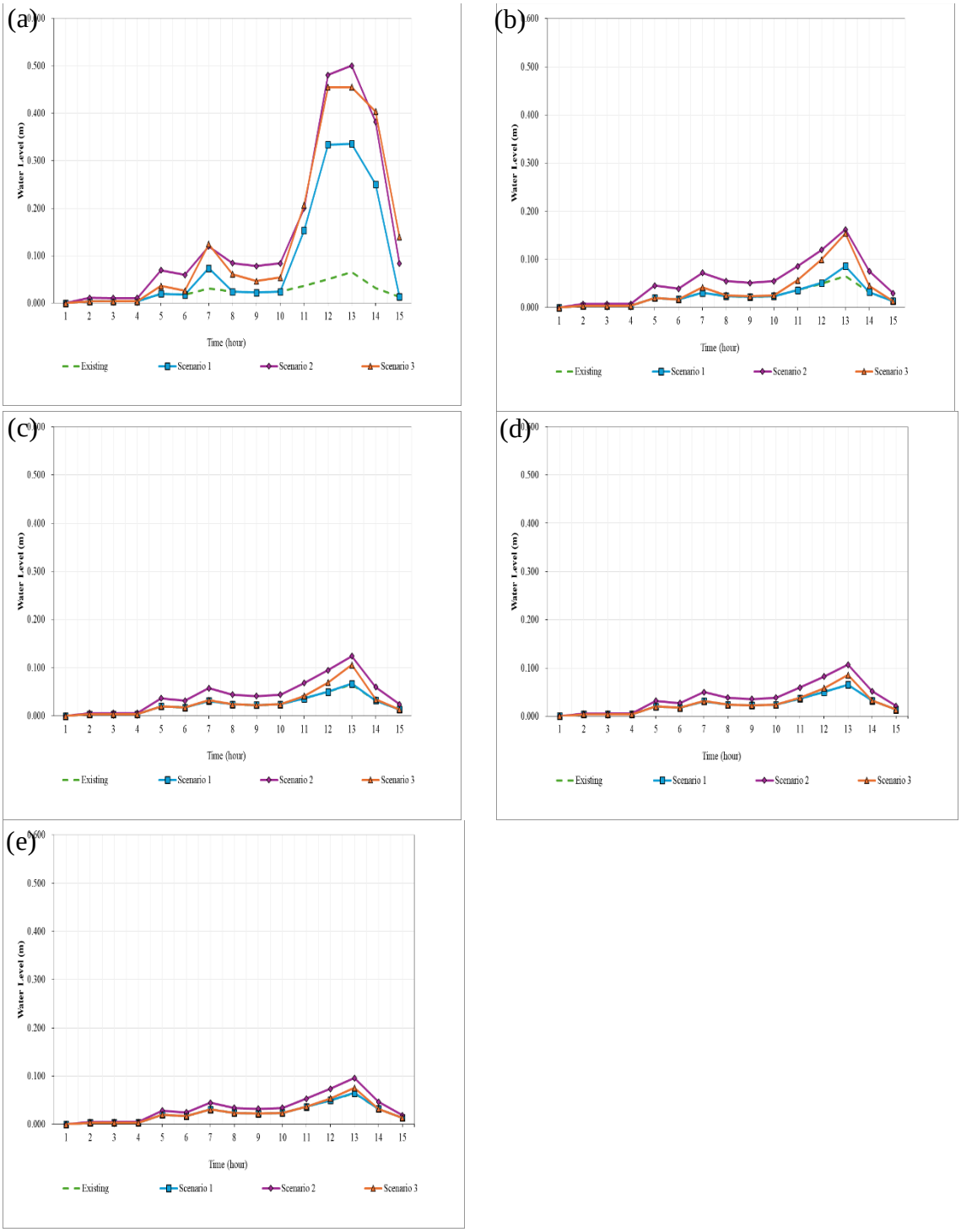


Figure AI.55: Water Level Plots for Drain with 0.2% Slope at Location C Subjected to Event 2, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

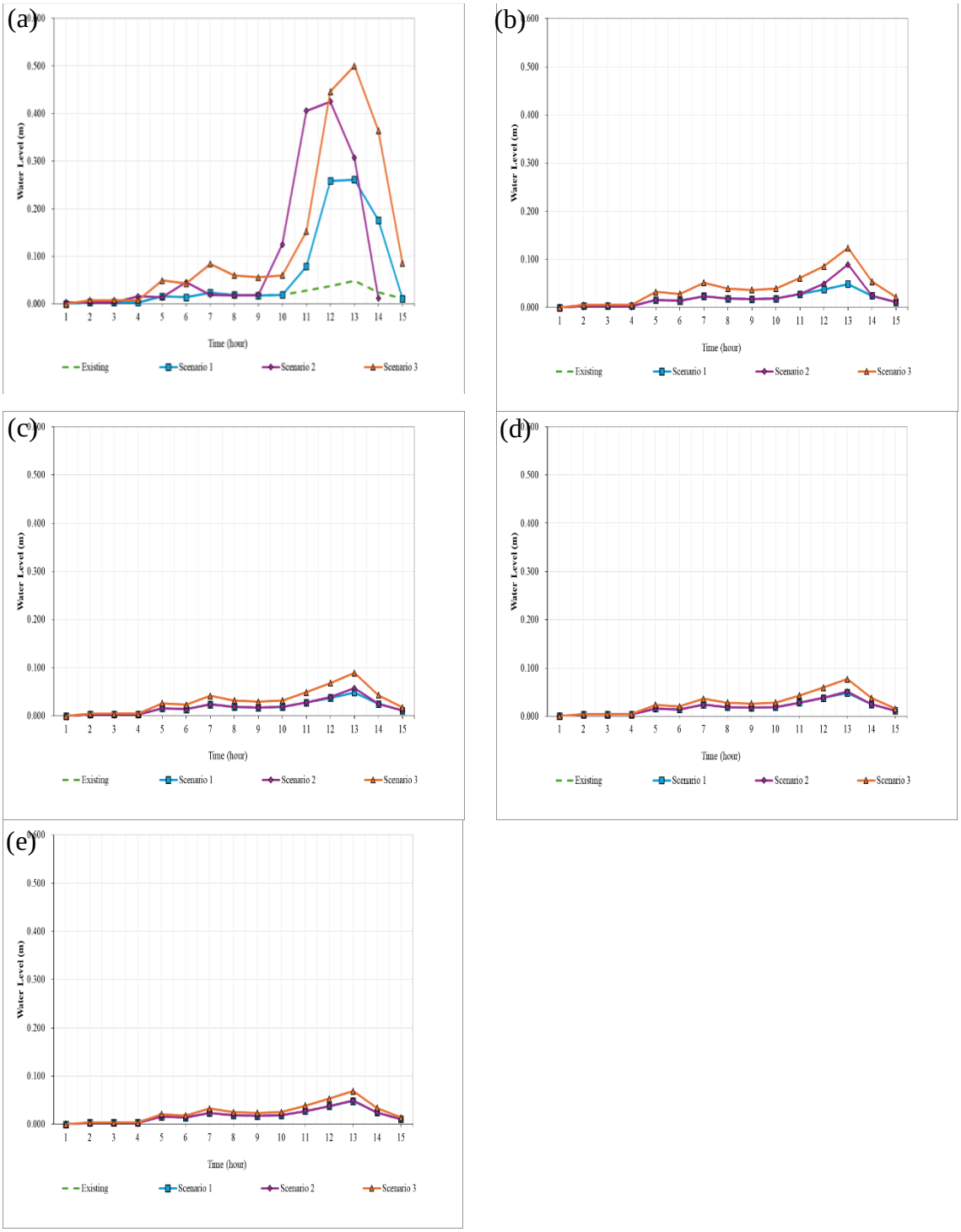


Figure AI.56: Water Level Plots for Drain with 0.2% Slope at Location D Subjected to Event 2, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

Event 3 (Flow Rates)

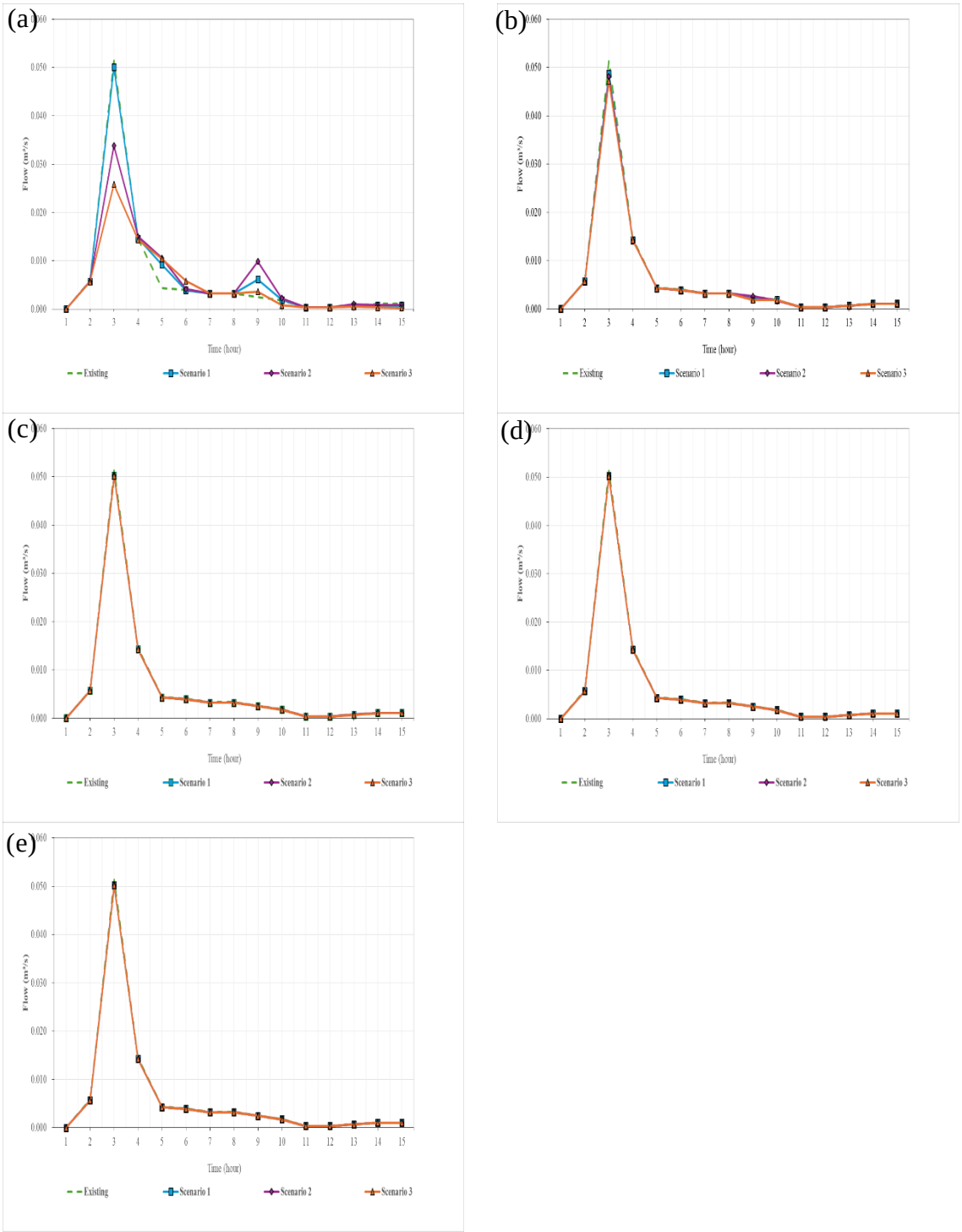


Figure AI.57: Flow Plots for Drain with 0.2% Slope at Location A Subjected to Event 3, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

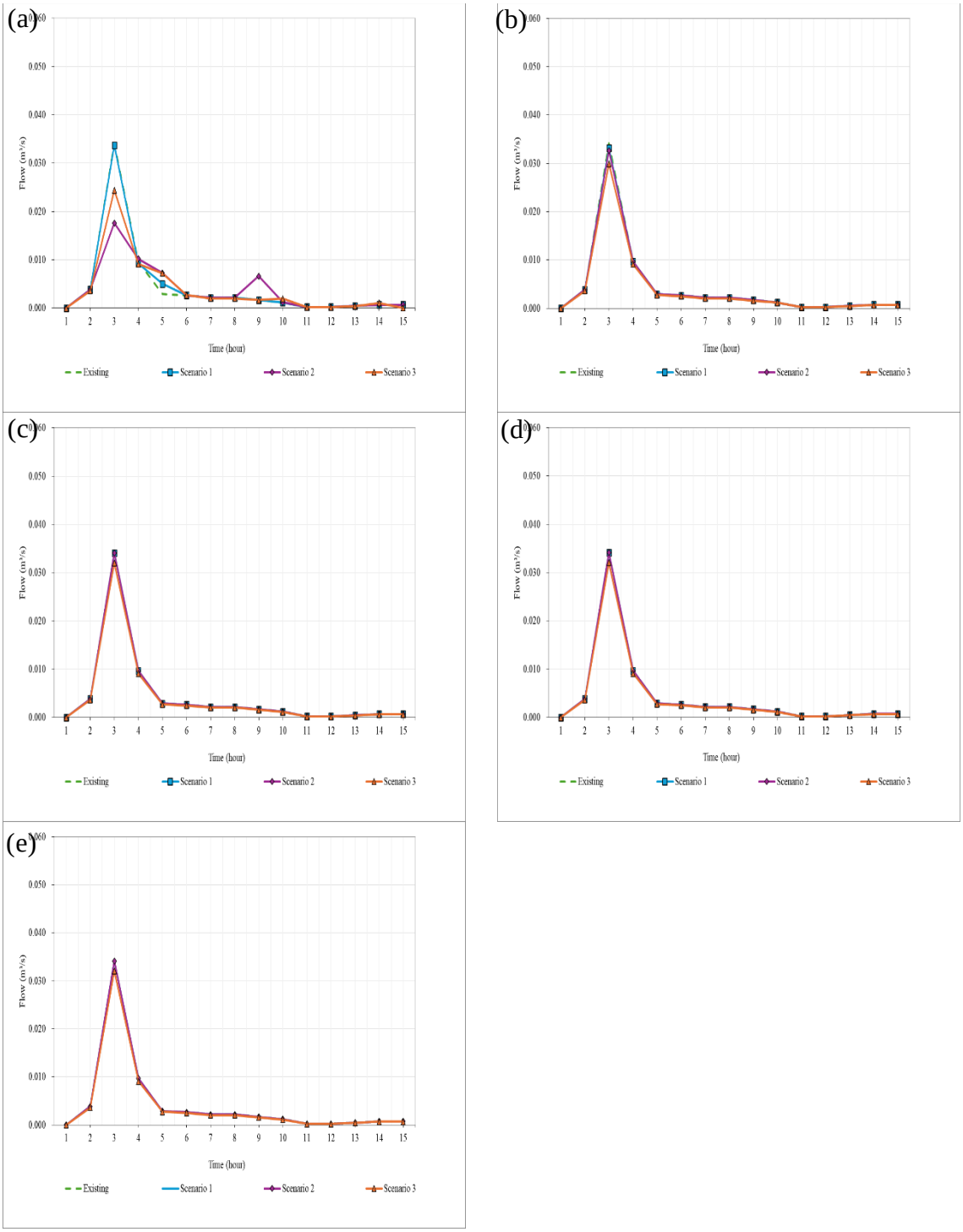


Figure AI.58: Flow Plots for Drain with 0.2% Slope at Location B Subjected to Event 3, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

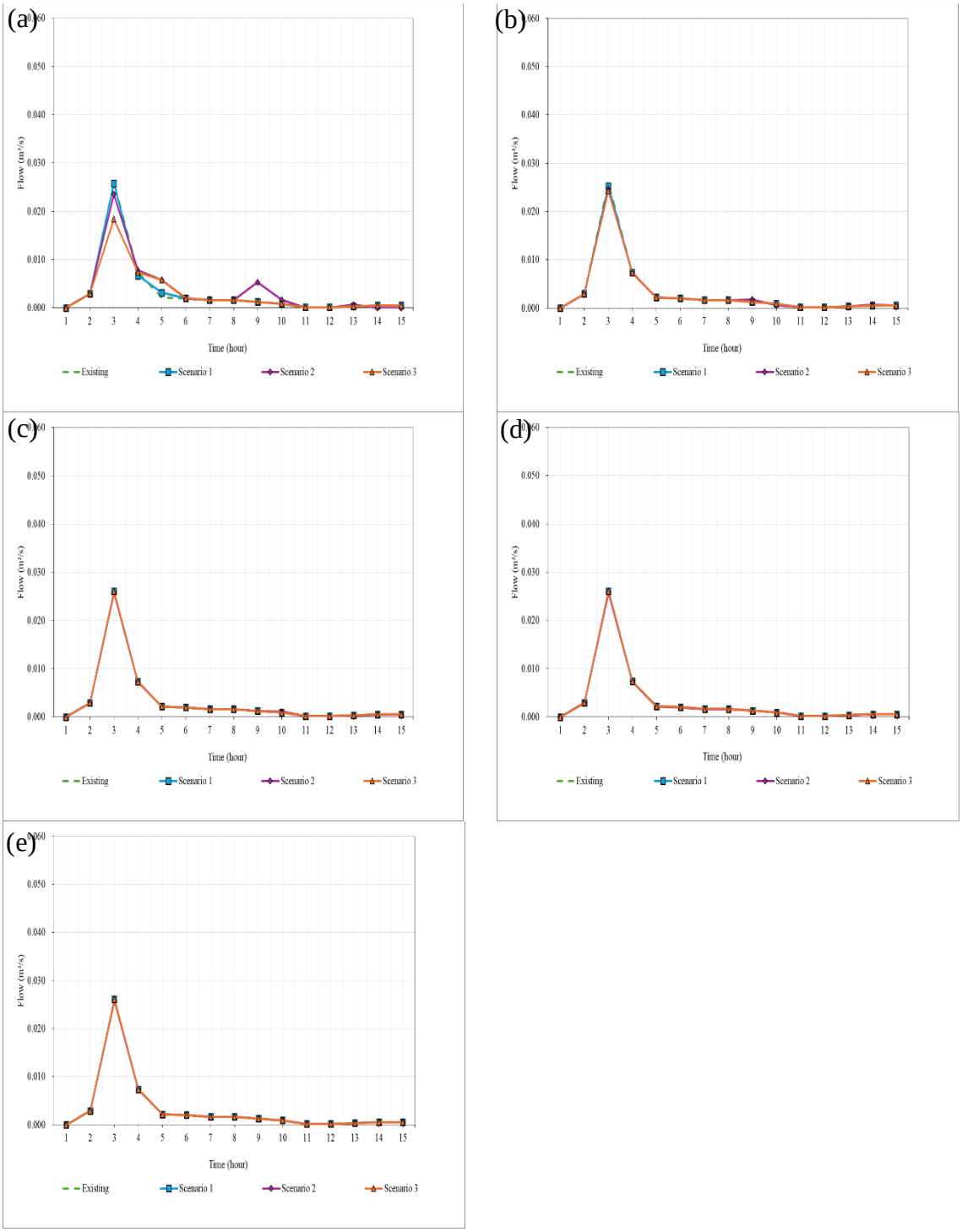


Figure AI.59: Flow Plots for Drain with 0.2% Slope at Location C Subjected to Event 3, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

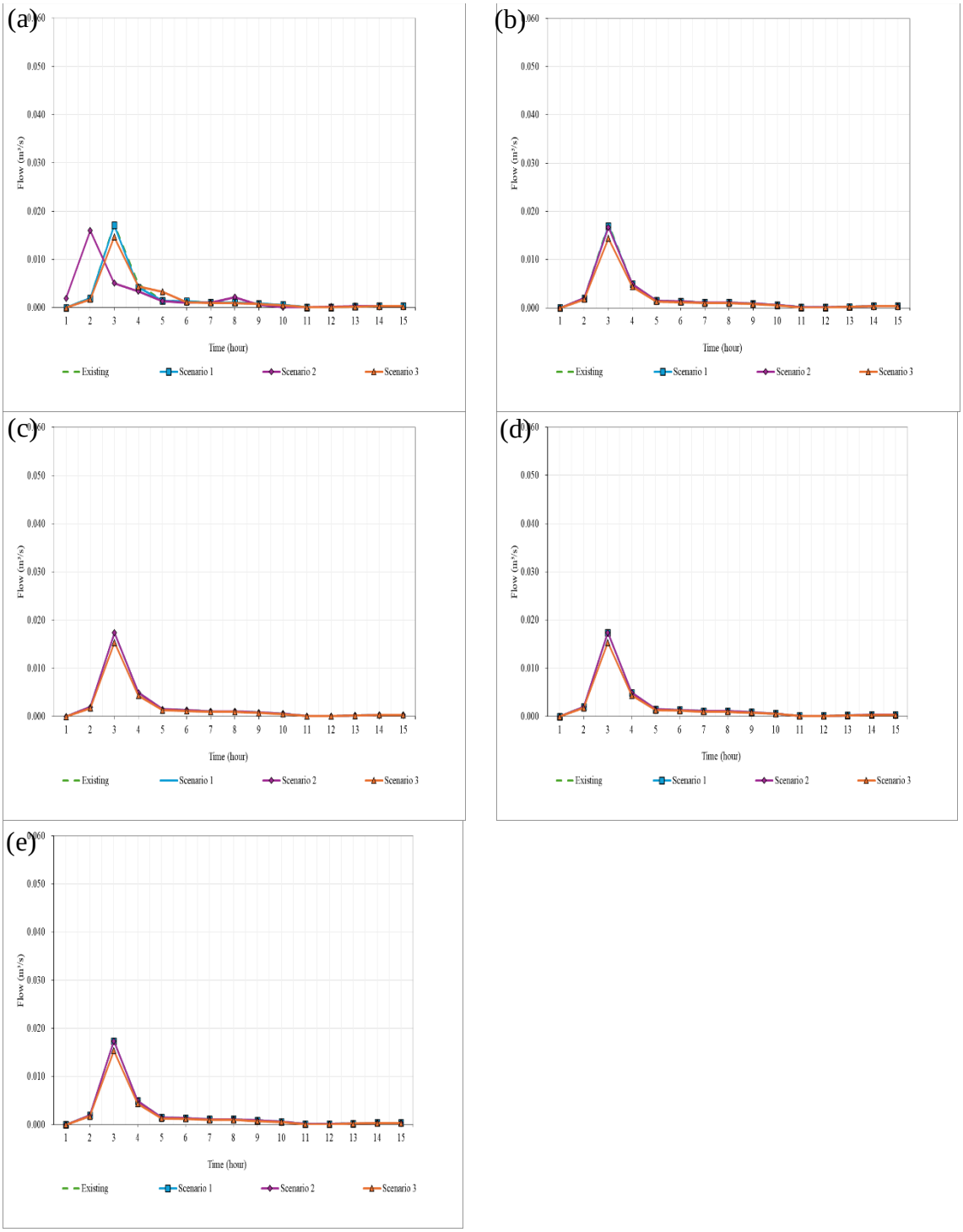


Figure AI.60: Flow Plots for Drain with 0.2% Slope at Location D Subjected to Event 3, (a) 0.1m, (b) 0. 2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

Event 3 (Water Levels)

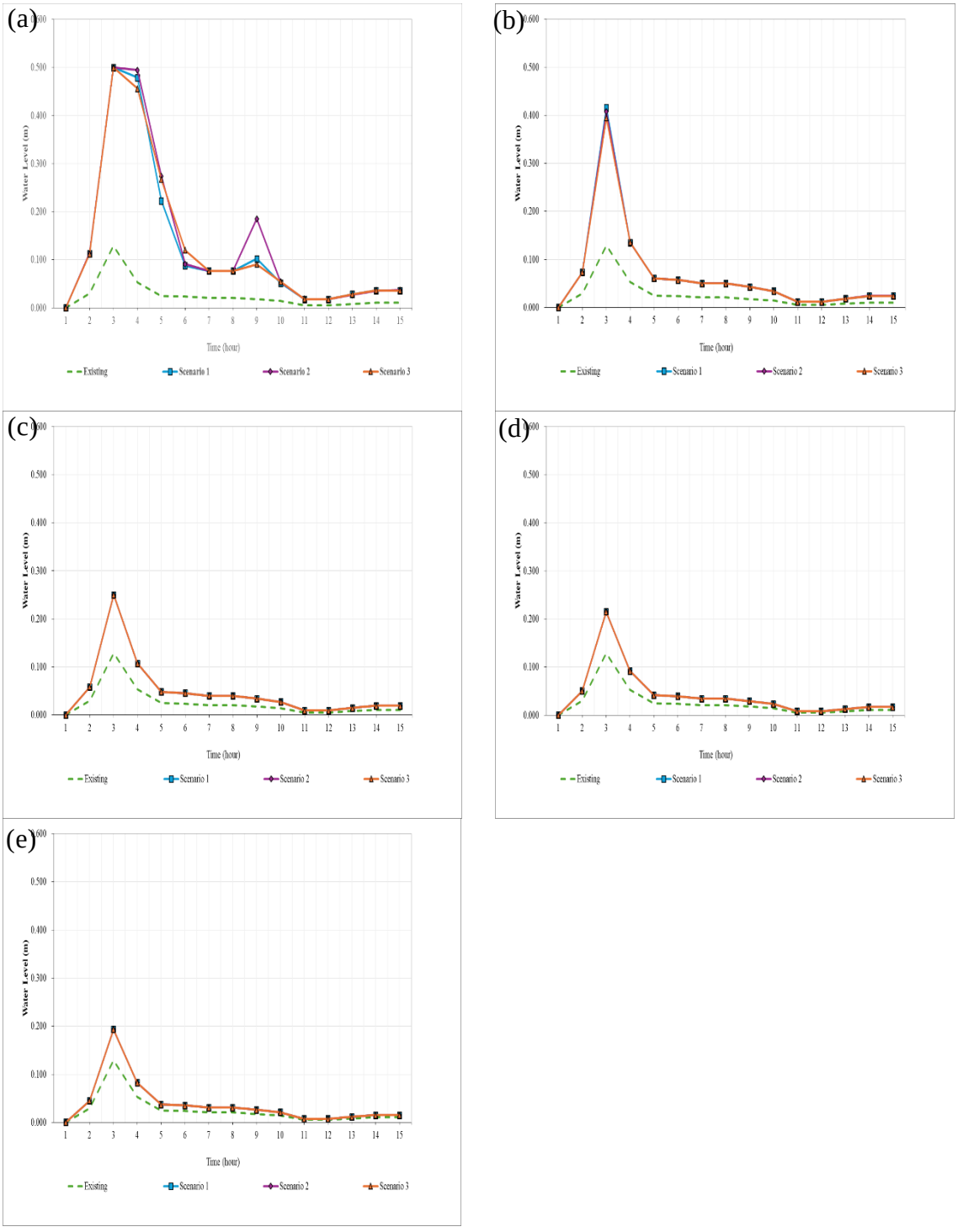


Figure AI.61: Water Level Plots for Drain with 0.2% Slope at Location A Subjected to Event 3, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

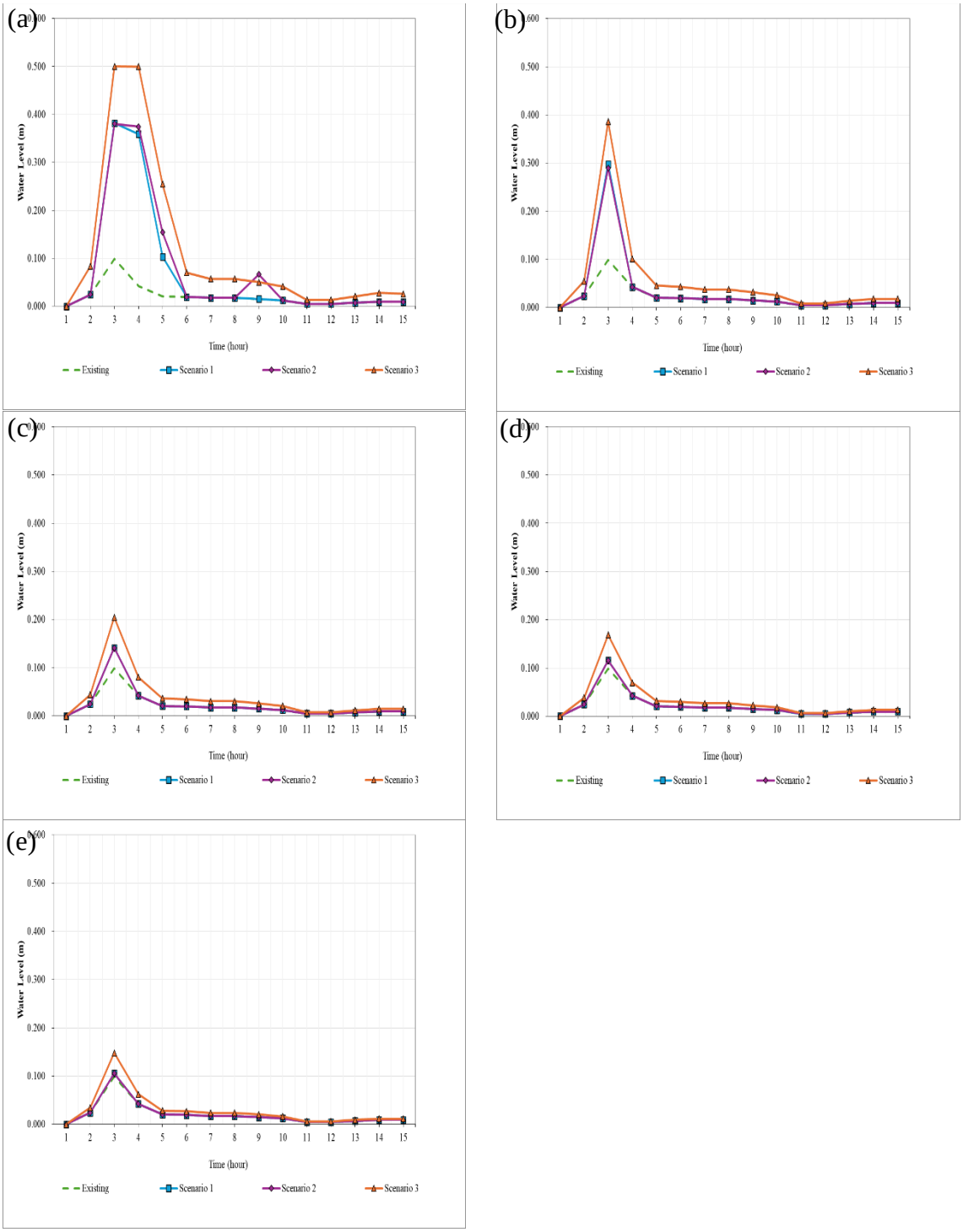


Figure AI.62: Water Level Plots for Drain with 0.2% Slope at Location B Subjected to Event 3, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

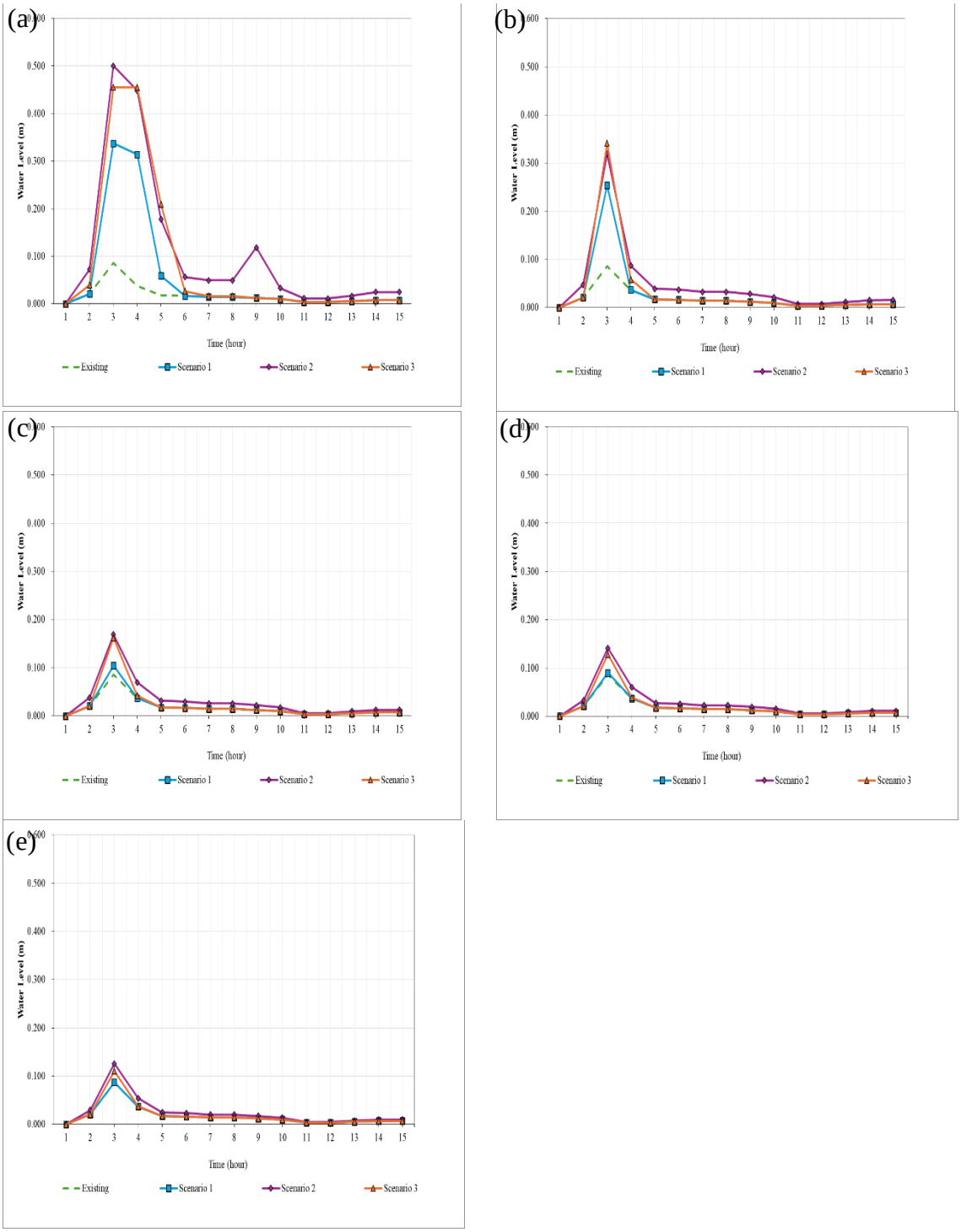


Figure AI.63: Water Level Plots for Drain with 0.2% Slope at Location C Subjected to Event 3, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

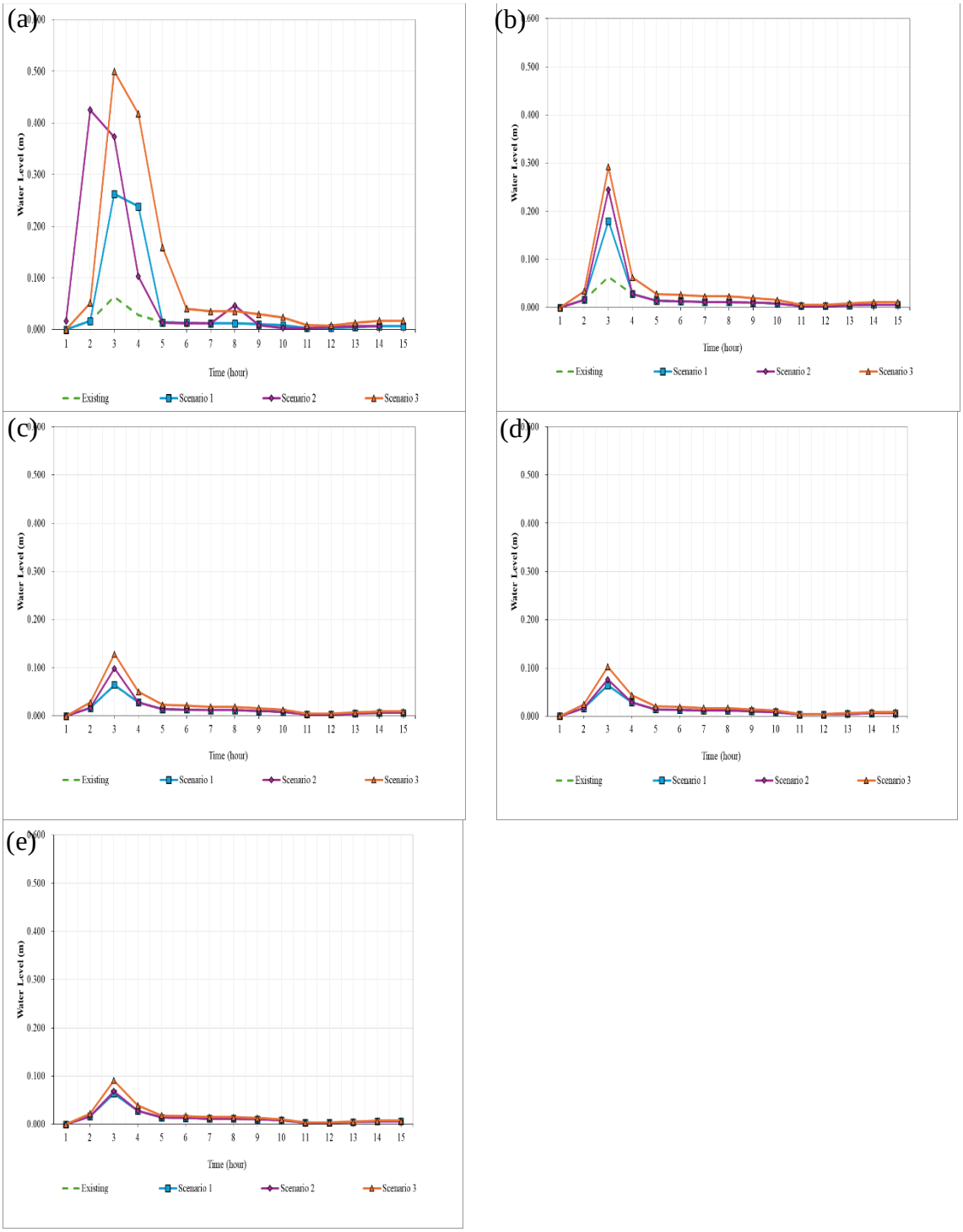


Figure AI.64: Water Level Plots for Drain with 0.2% Slope at Location D Subjected to Event 3, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

v. Drain With 0.4% Slope - Design Rainfall

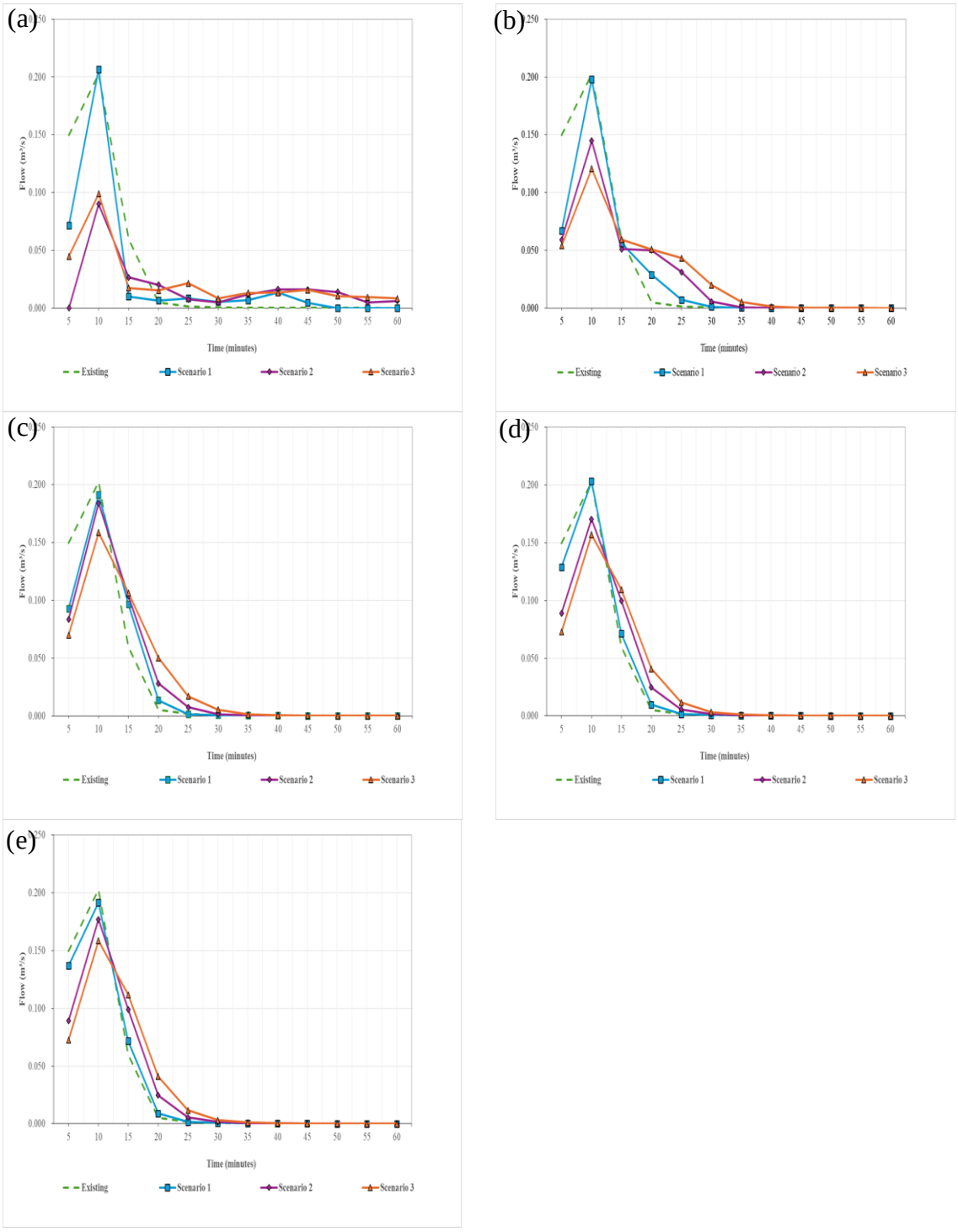


Figure AI.65: Flow Plots for Drain with 0.4% Slope at Location A Subjected to Design Rainfall, (a) 0.1m, (b) 0. 2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

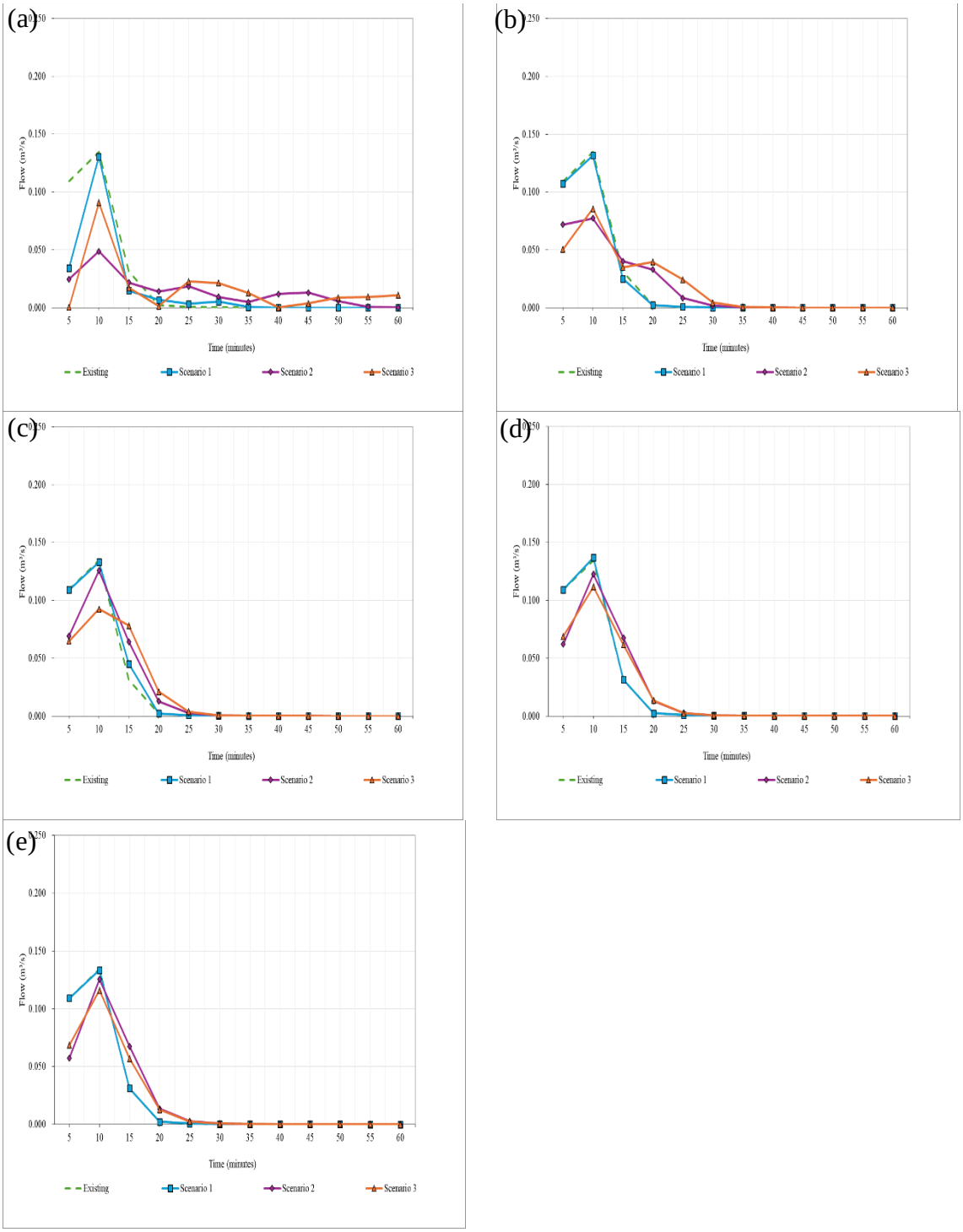


Figure AI.66: Flow Plots for Drain with 0.4% Slope at Location B Subjected to Design Rainfall, (a) 0.1m, (b) 0. 2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

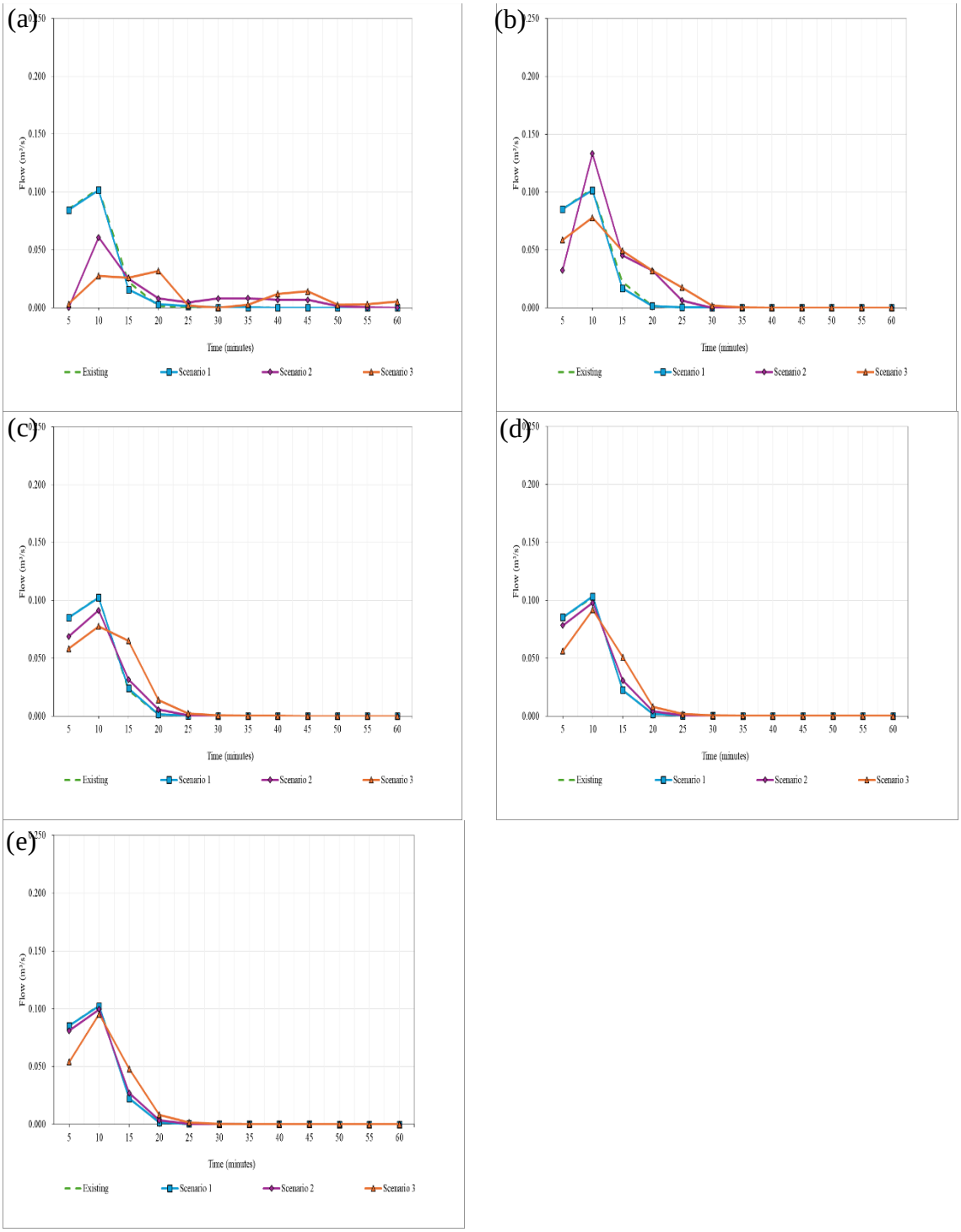


Figure AI.67: Flow Plots for Drain with 0.4% Slope at Location C Subjected to Design Rainfall, (a) 0.1m, (b) 0. 2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

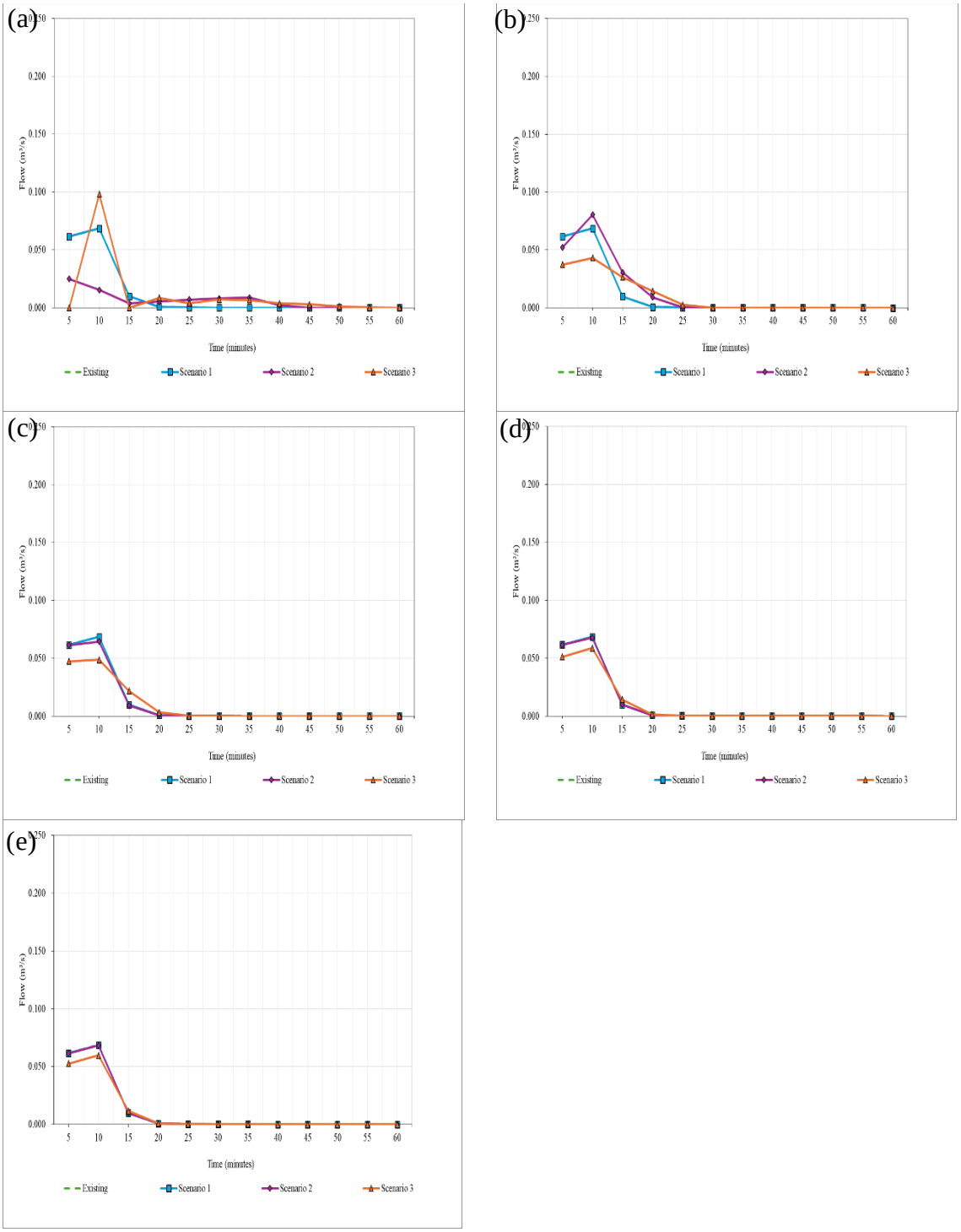


Figure AI.68: Flow Plots for Drain with 0.4% Slope at Location D Subjected to Design Rainfall, (a) 0.1m, (b) 0. 2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

Design Rainfall (Water Levels)

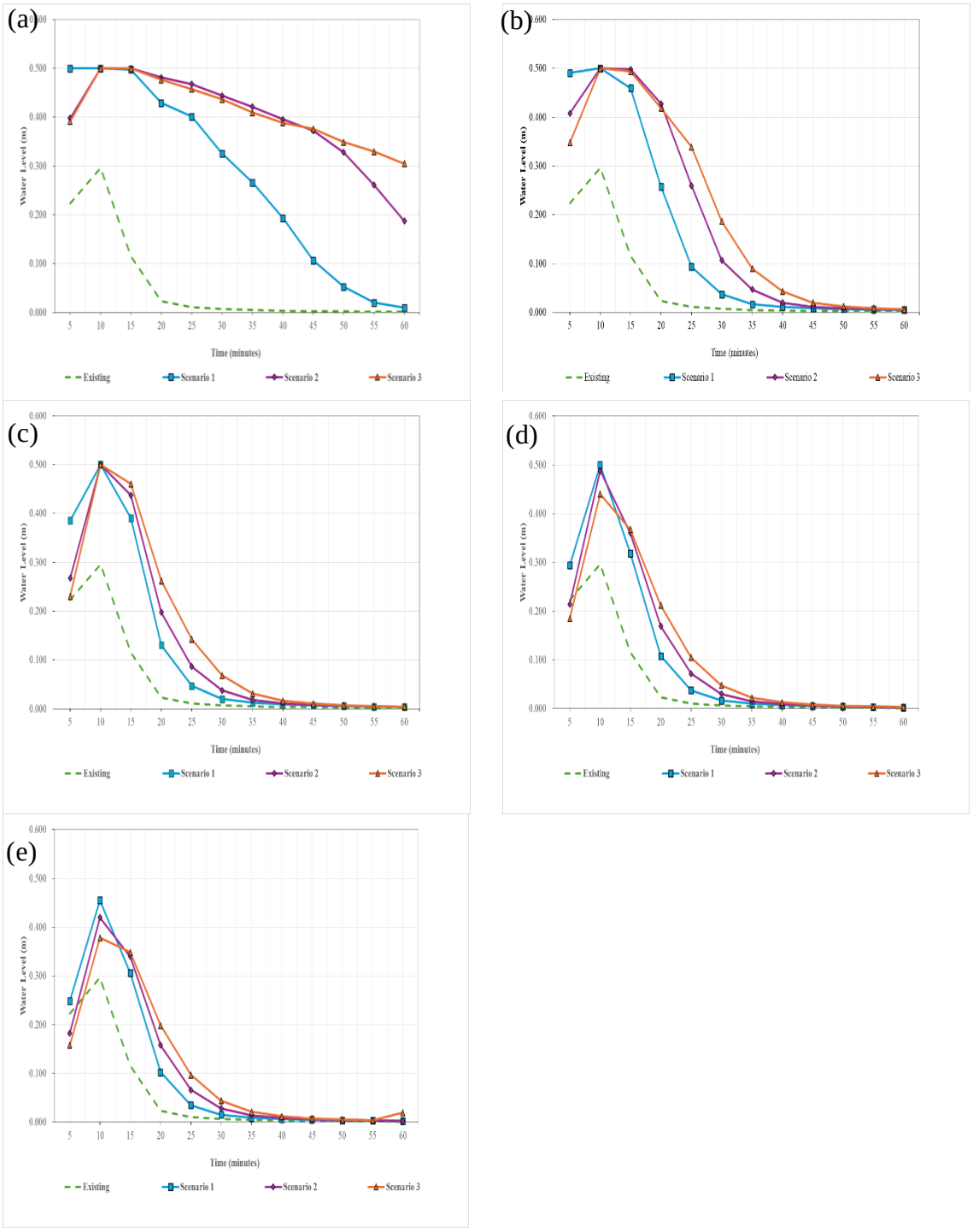


Figure AI.69: Water Level Plots for Drain with 0.4% Slope at Location A Subjected to Design Rainfall, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

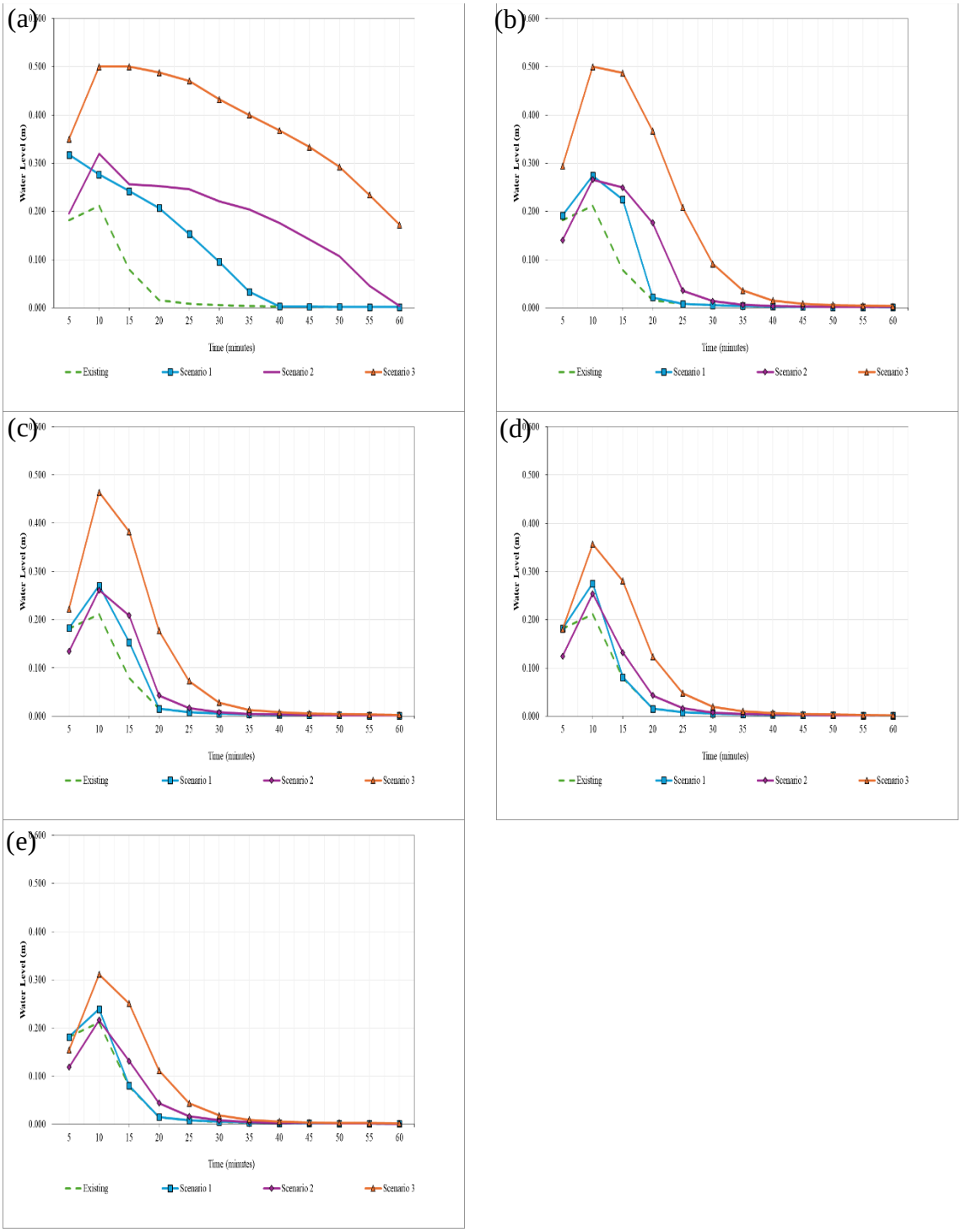


Figure AI.70: Water Level Plots for Drain with 0.4% Slope at Location B Subjected to Design Rainfall, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

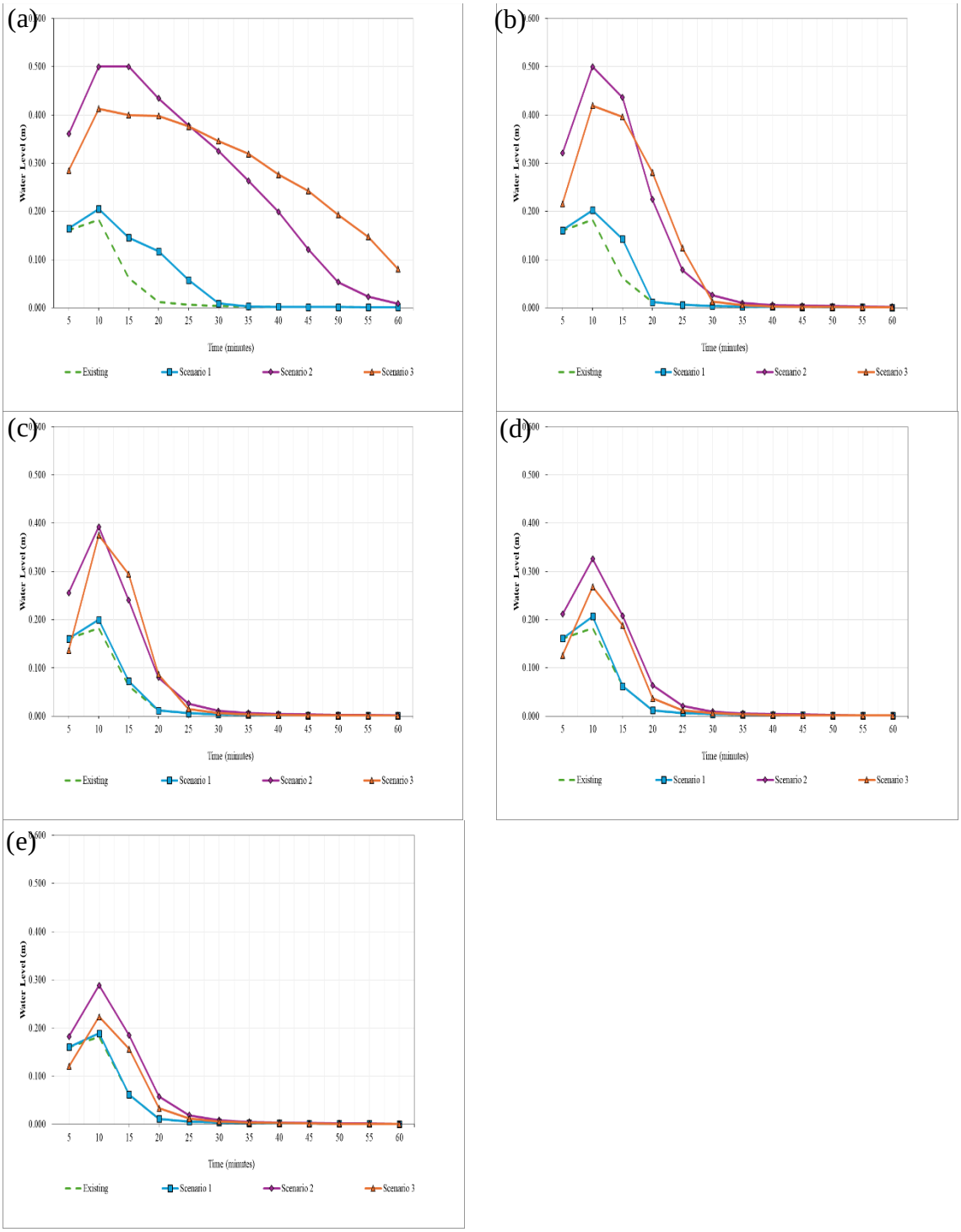


Figure AI.71: Water Level Plots for Drain with 0.4% Slope at Location C Subjected to Design Rainfall, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

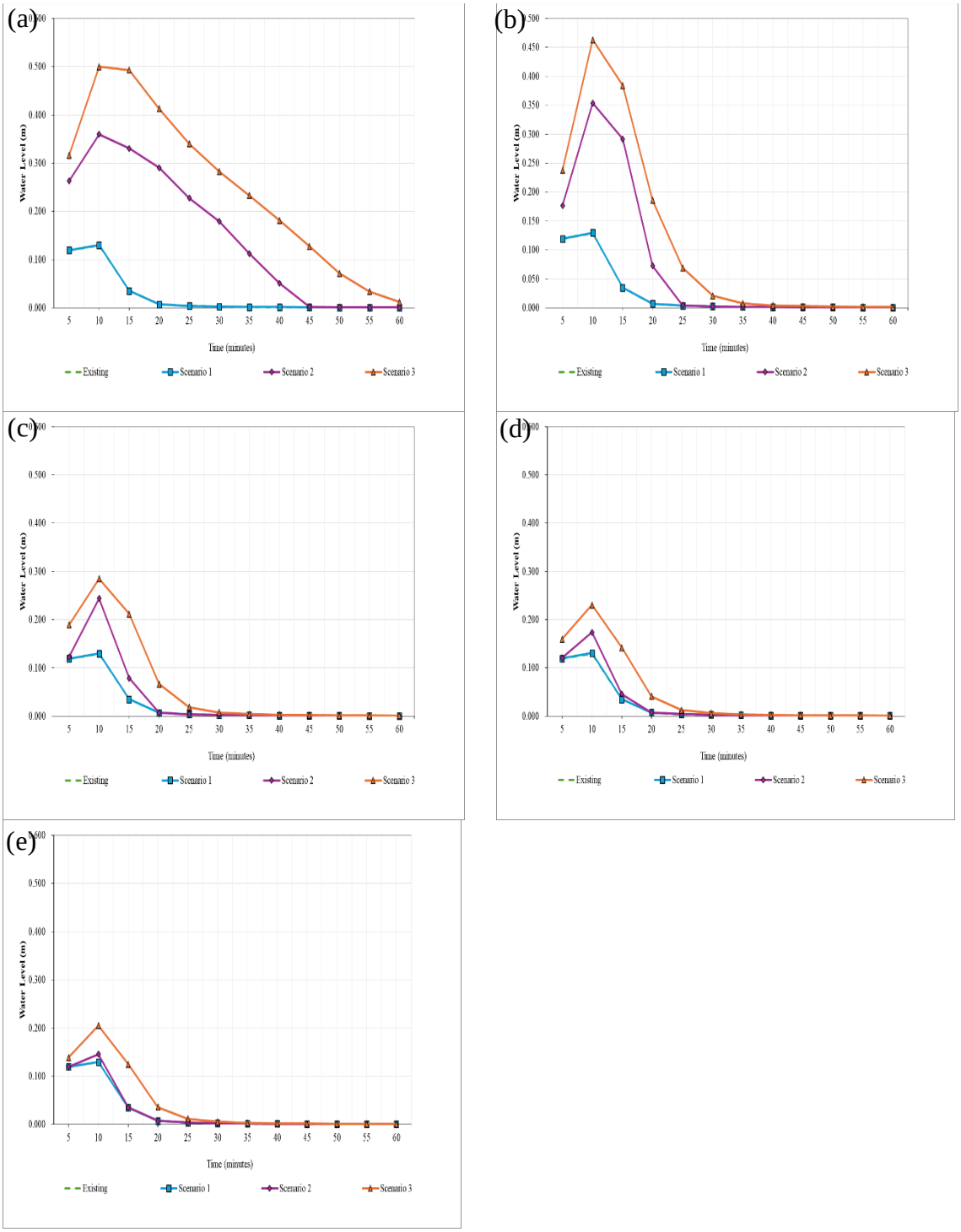


Figure AI.72: Water Level Plots for Drain with 0.4% Slope at Location D Subjected to Design Rainfall, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

vi. Drain With 0.4% Slope – Historical Event

Event 1 (Flow Rates)

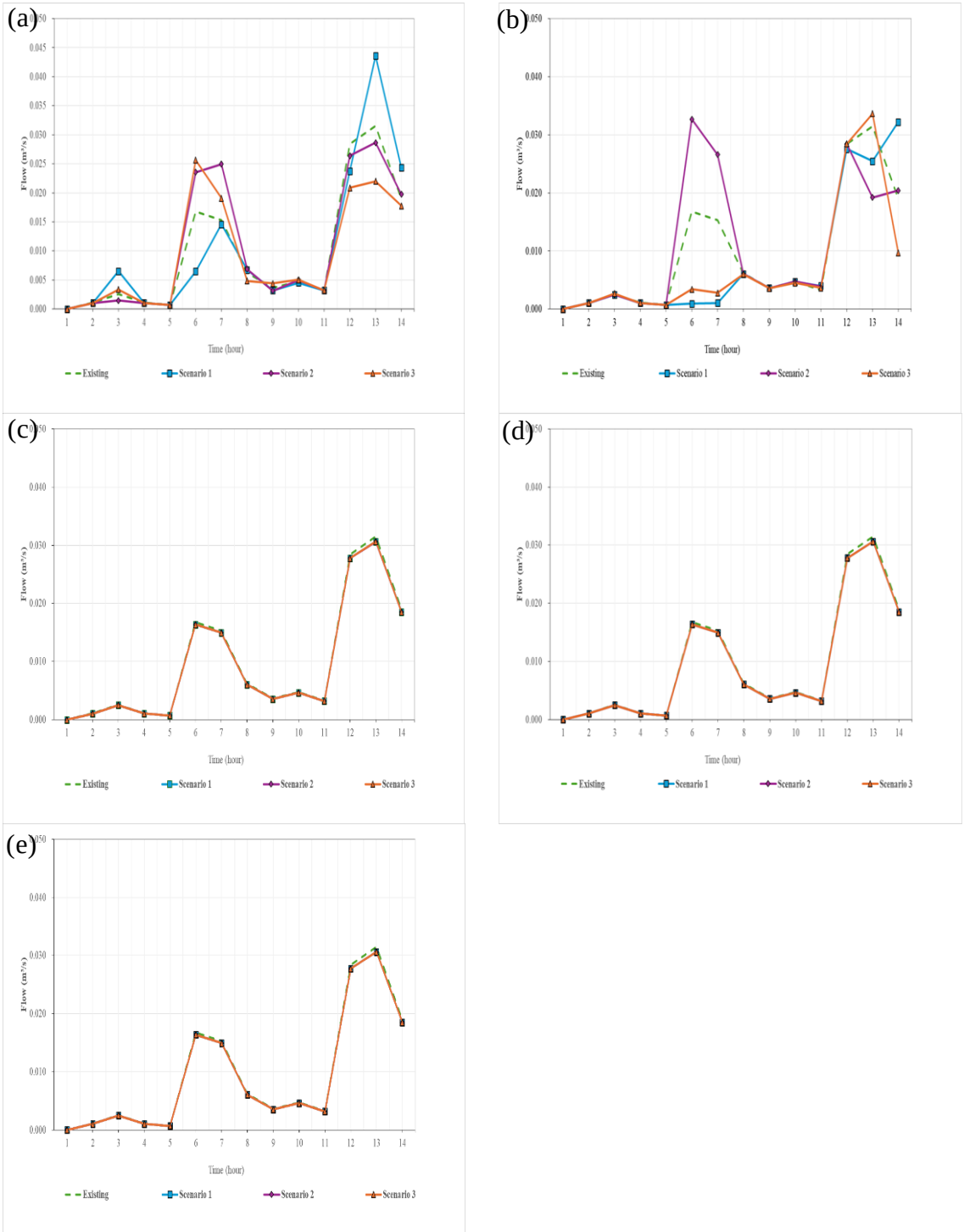


Figure AI.73: Flow Plots for Drain with 0.4% Slope at Location A Subjected to Event 1, (a) 0.1m, (b) 0. 2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

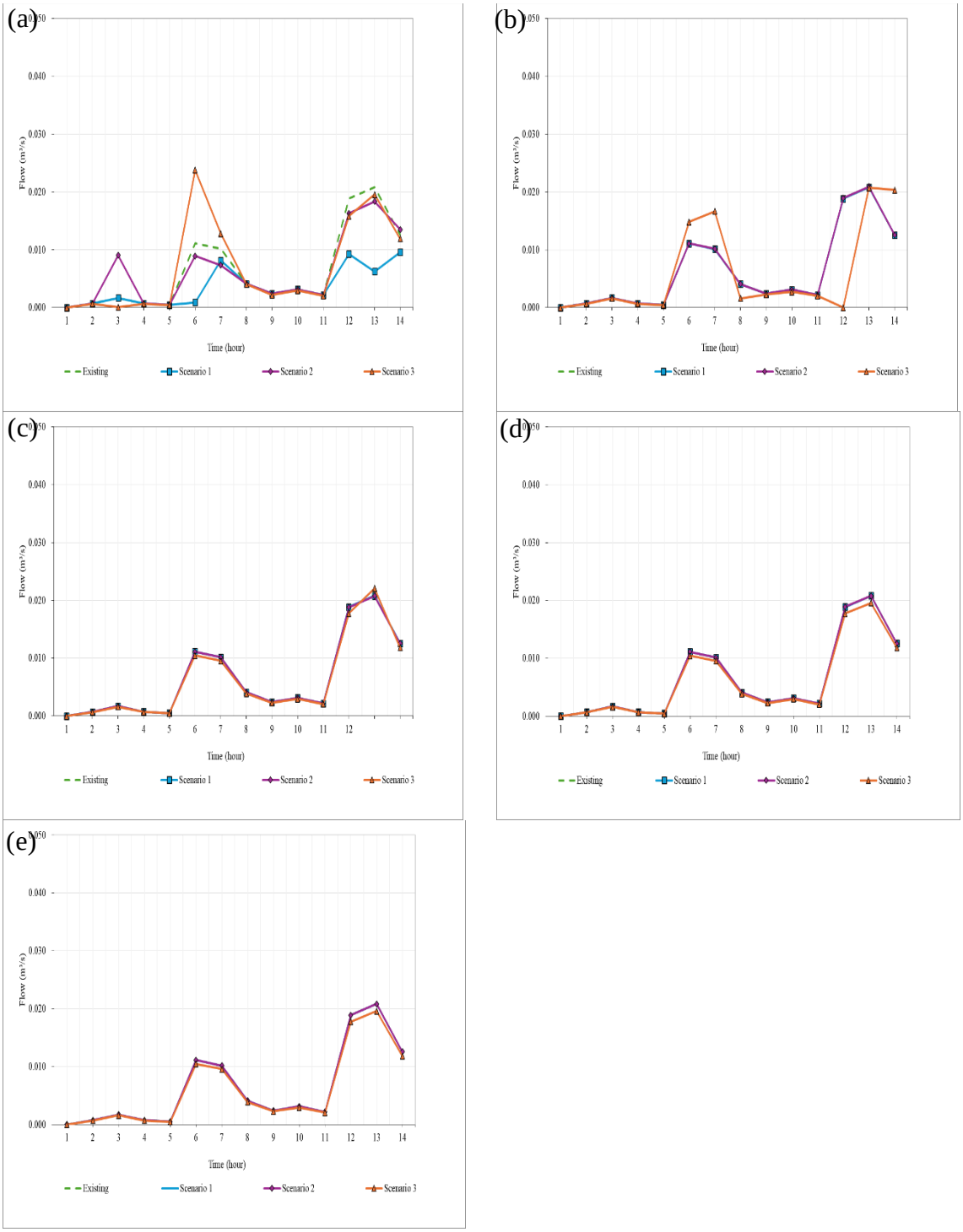


Figure AI.74: Flow Plots for Drain with 0.4% Slope at Location B Subjected to Event 1, (a) 0.1m, (b) 0. 2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice



Figure AI.75: Flow Plots for Drain with 0.4% Slope at Location C Subjected to Event 1, (a) 0.1m, (b) 0. 2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

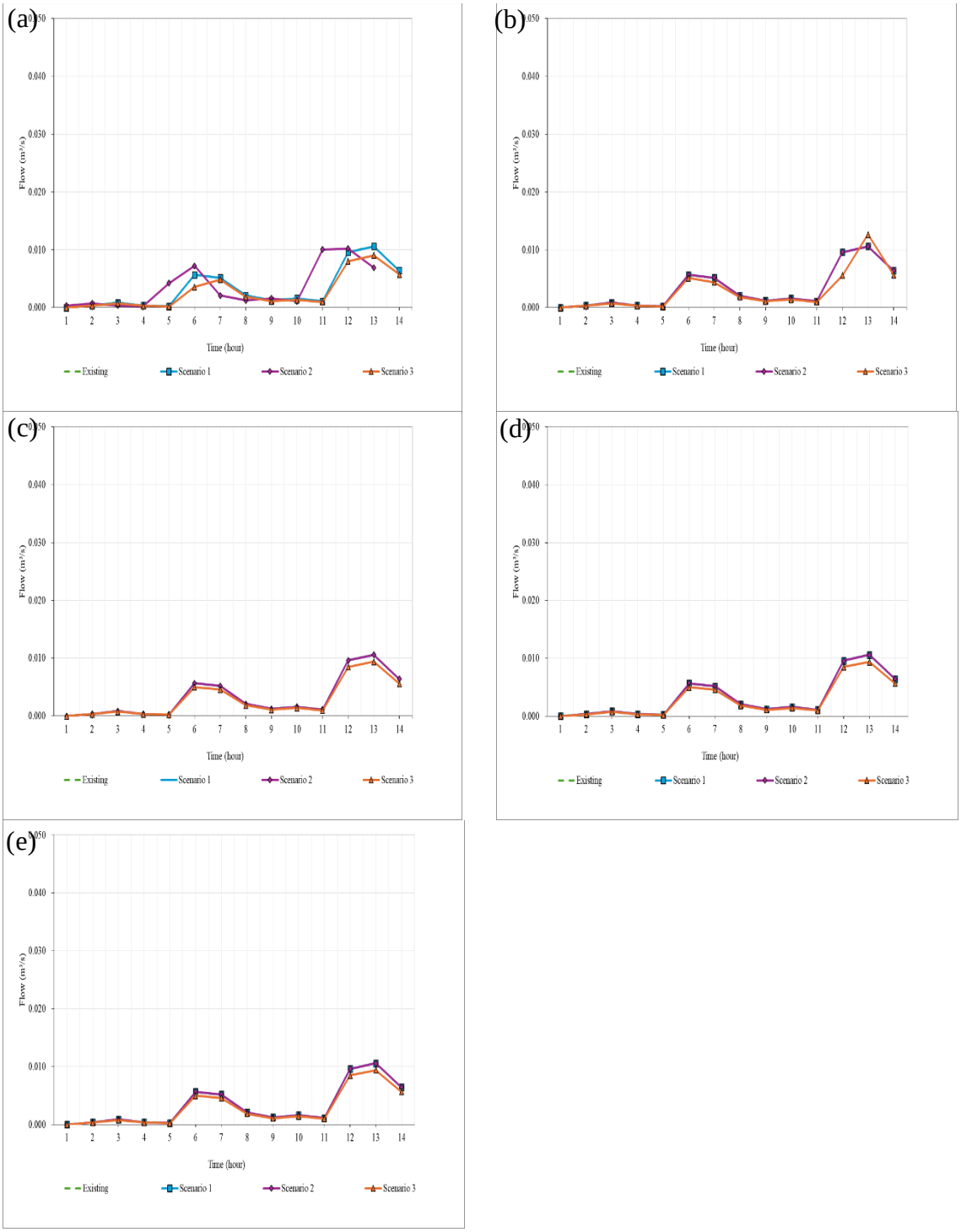


Figure AI.76: Flow Plots for Drain with 0.4% Slope at Location D Subjected to Event 1, (a) 0.1m, (b) 0. 2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

Event 1 (Water Levels)

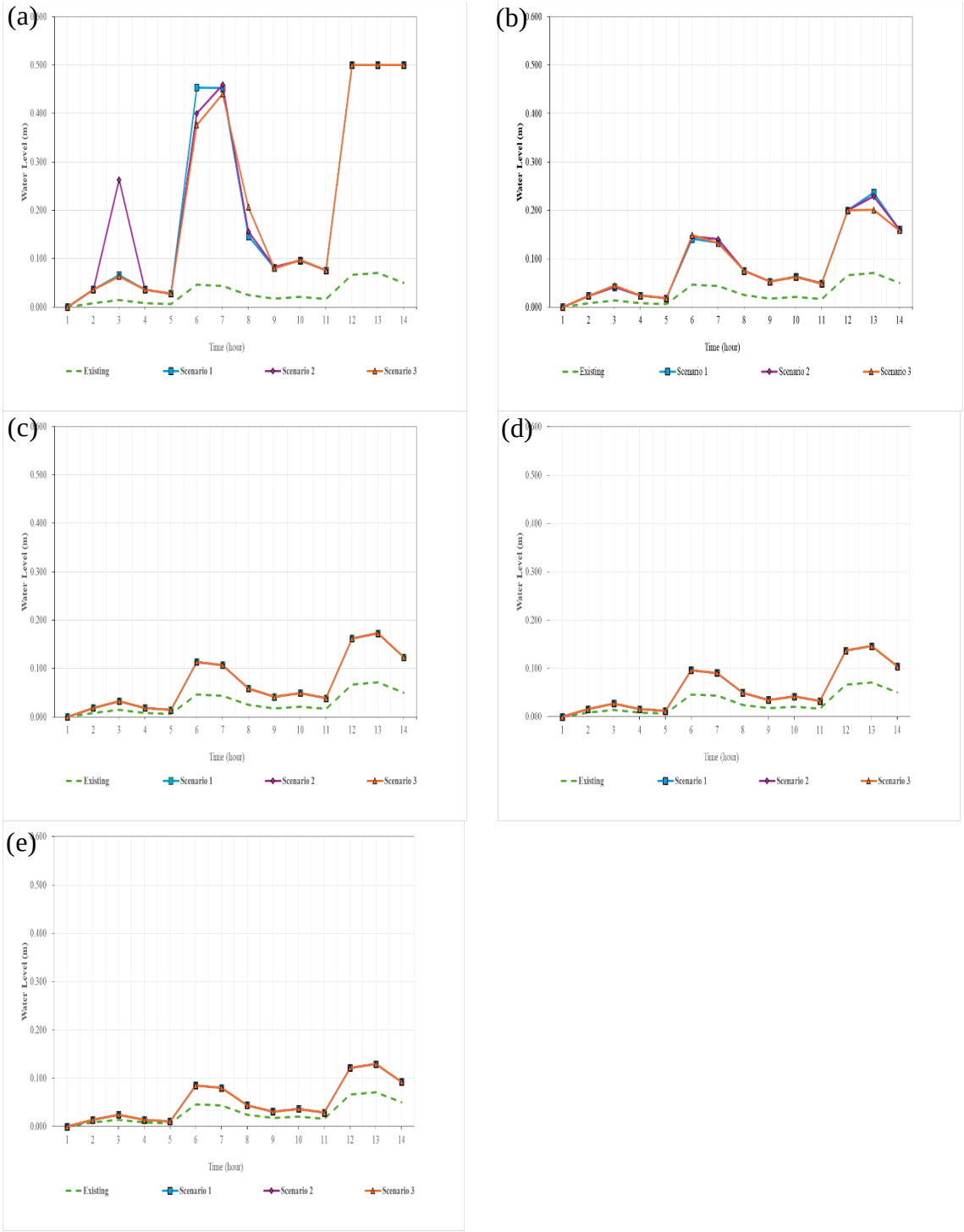


Figure AI.77: Water Level Plots for Drain with 0.4% Slope at Location A Subjected to Event 1, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

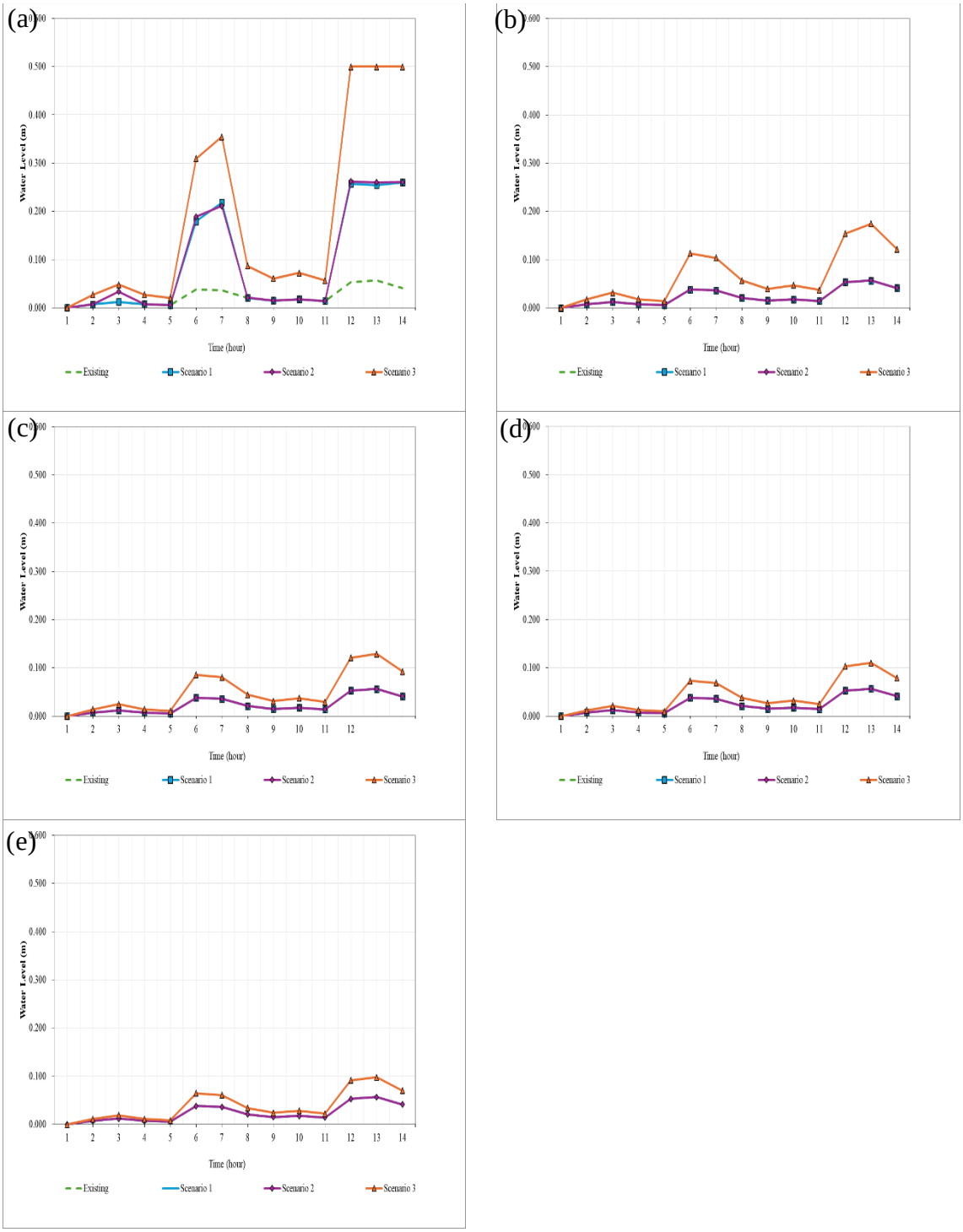


Figure AI.78: Water Level Plots for Drain with 0.4% Slope at Location B Subjected to Event 1, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

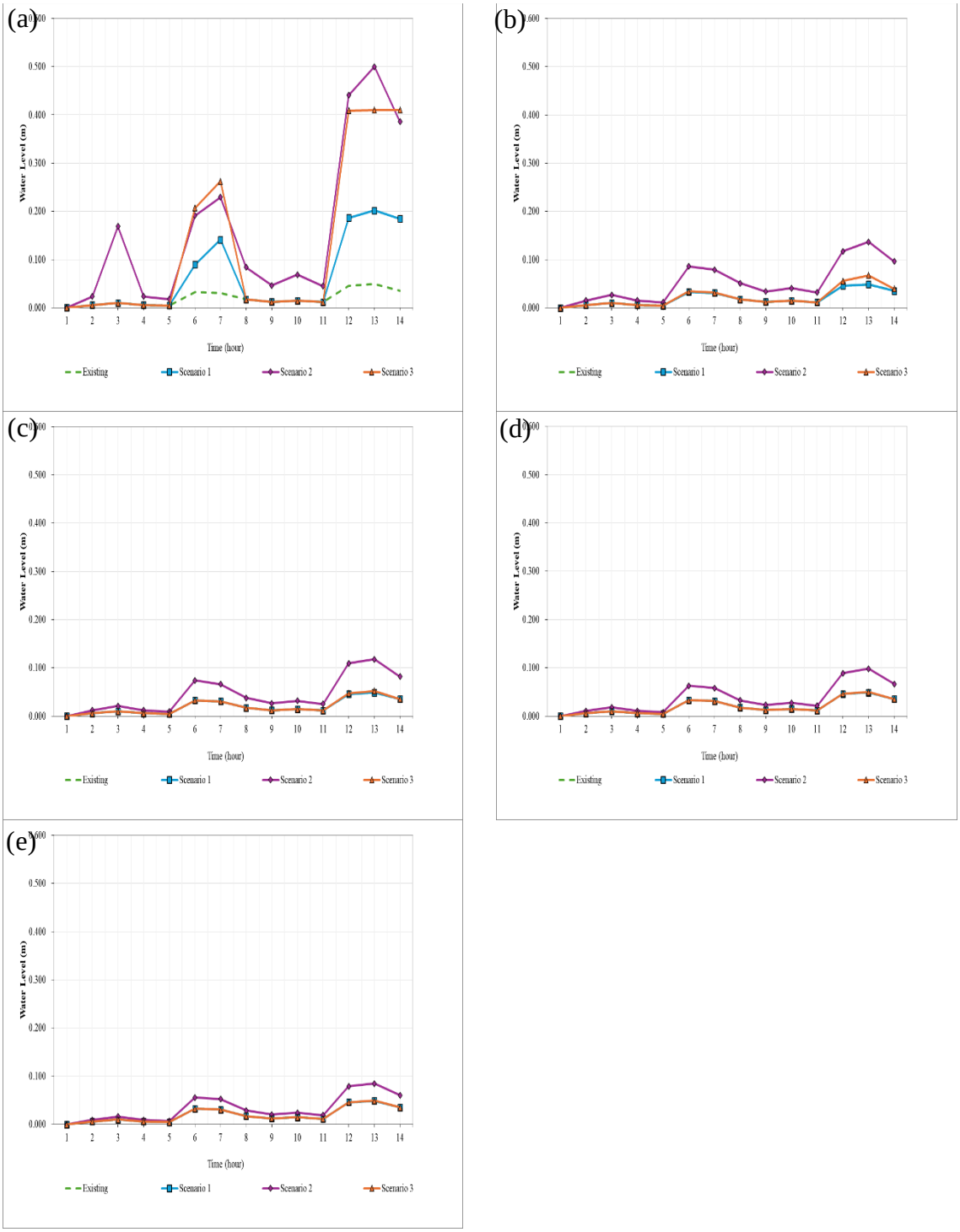


Figure AI.79: Water Level Plots for Drain with 0.4% Slope at Location C Subjected to Event 1, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

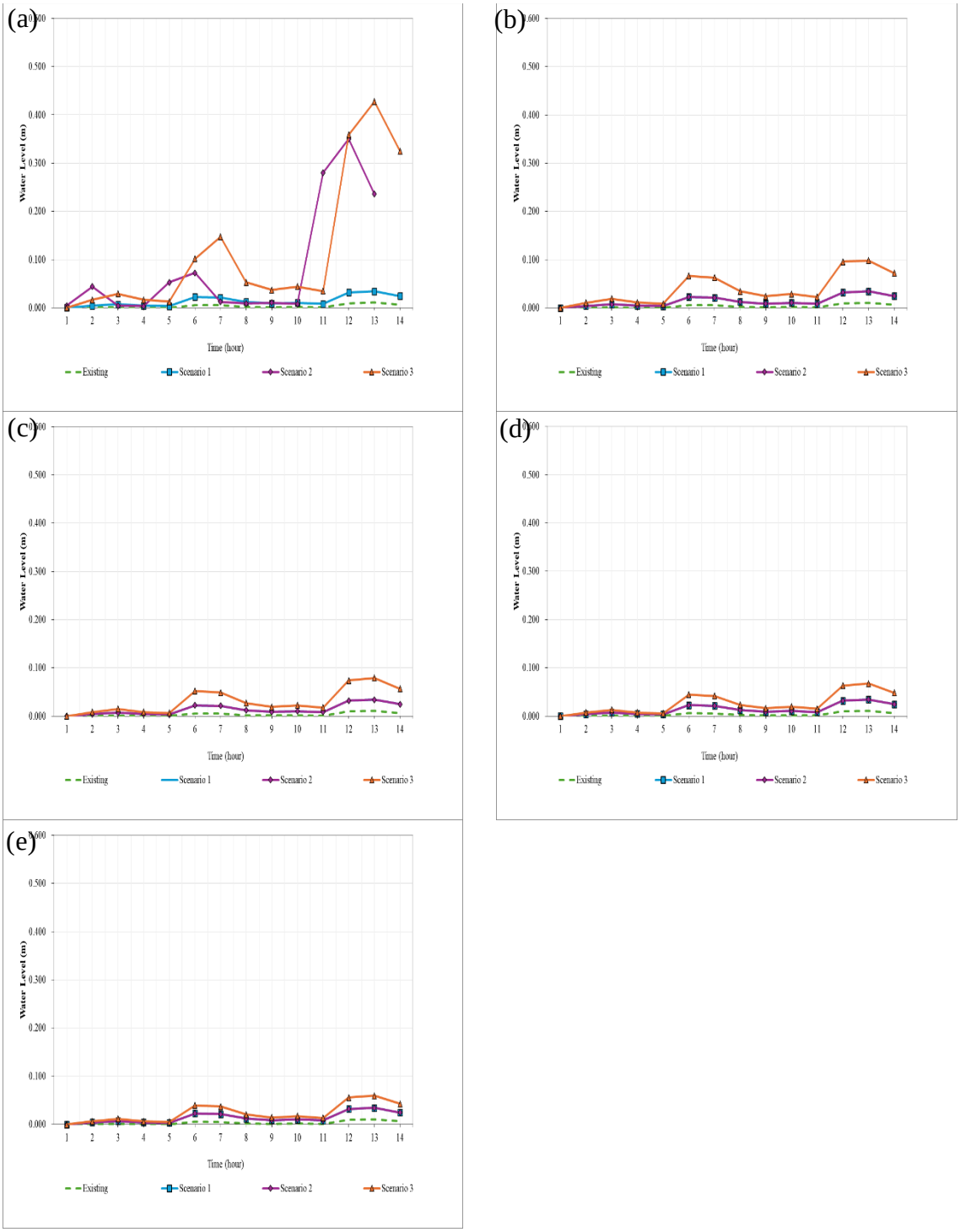


Figure AI.80: Water Level Plots for Drain with 0.4% Slope at Location D Subjected to Event 1, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

Event 2 (Flow Rates)

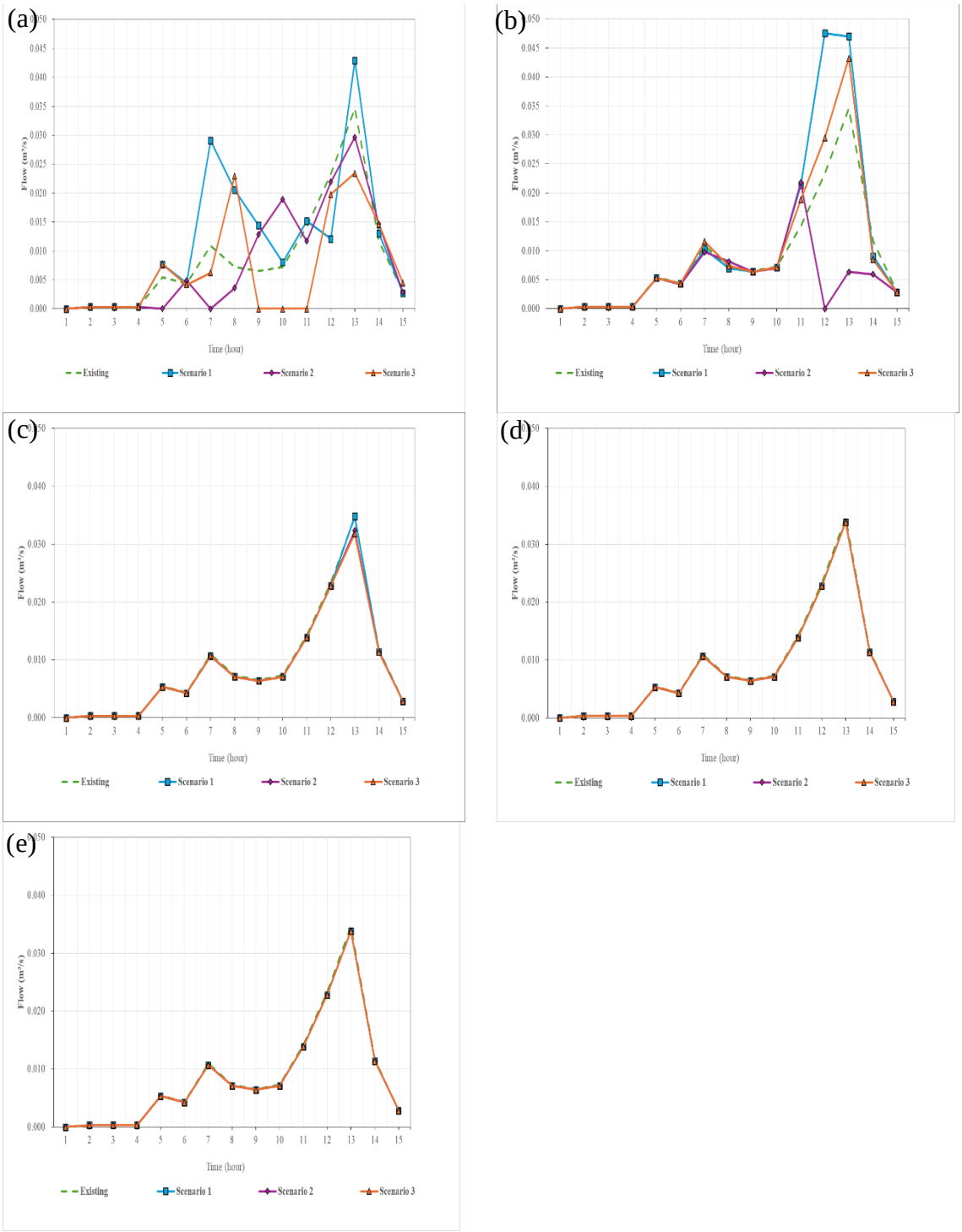


Figure AI.81: Flow Plots for Drain with 0.4% Slope at Location A Subjected to Event 2, (a) 0.1m, (b) 0. 2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

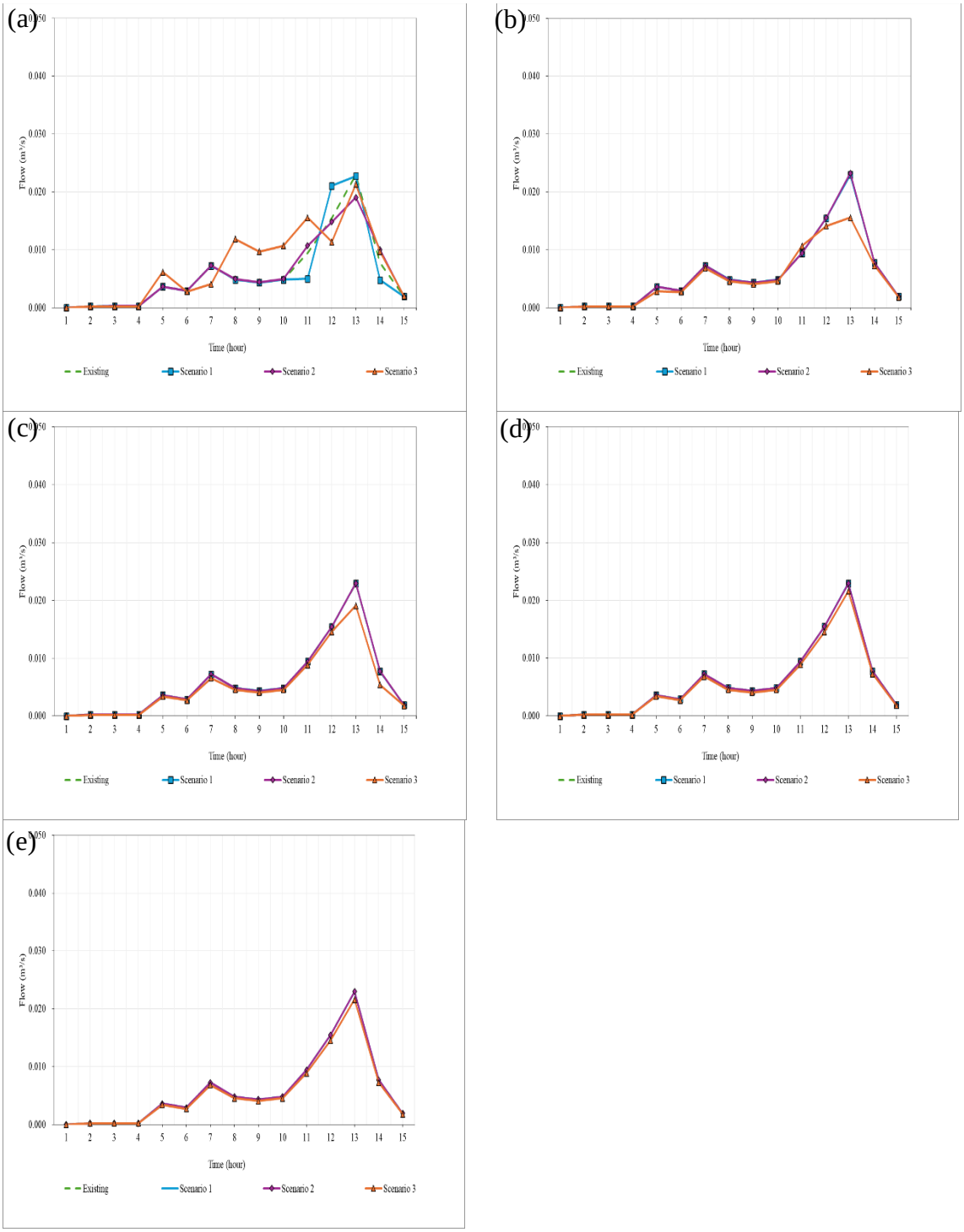


Figure AI.82: Flow Plots for Drain with 0.4% Slope at Location B Subjected to Event 2, (a) 0.1m, (b) 0. 2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

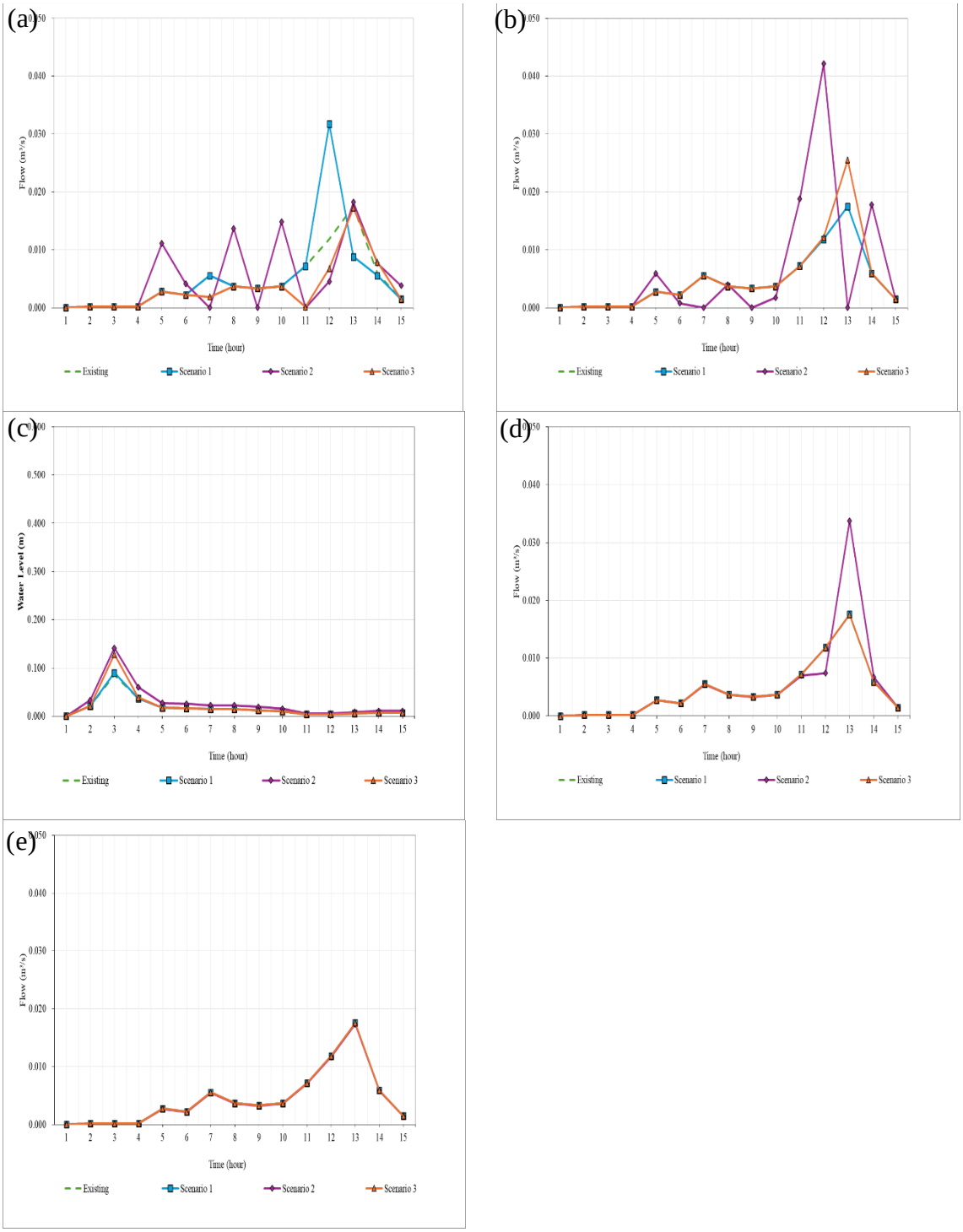


Figure AI.83: Flow Plots for Drain with 0.4% Slope at Location C Subjected to Event 2, (a) 0.1m, (b) 0. 2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

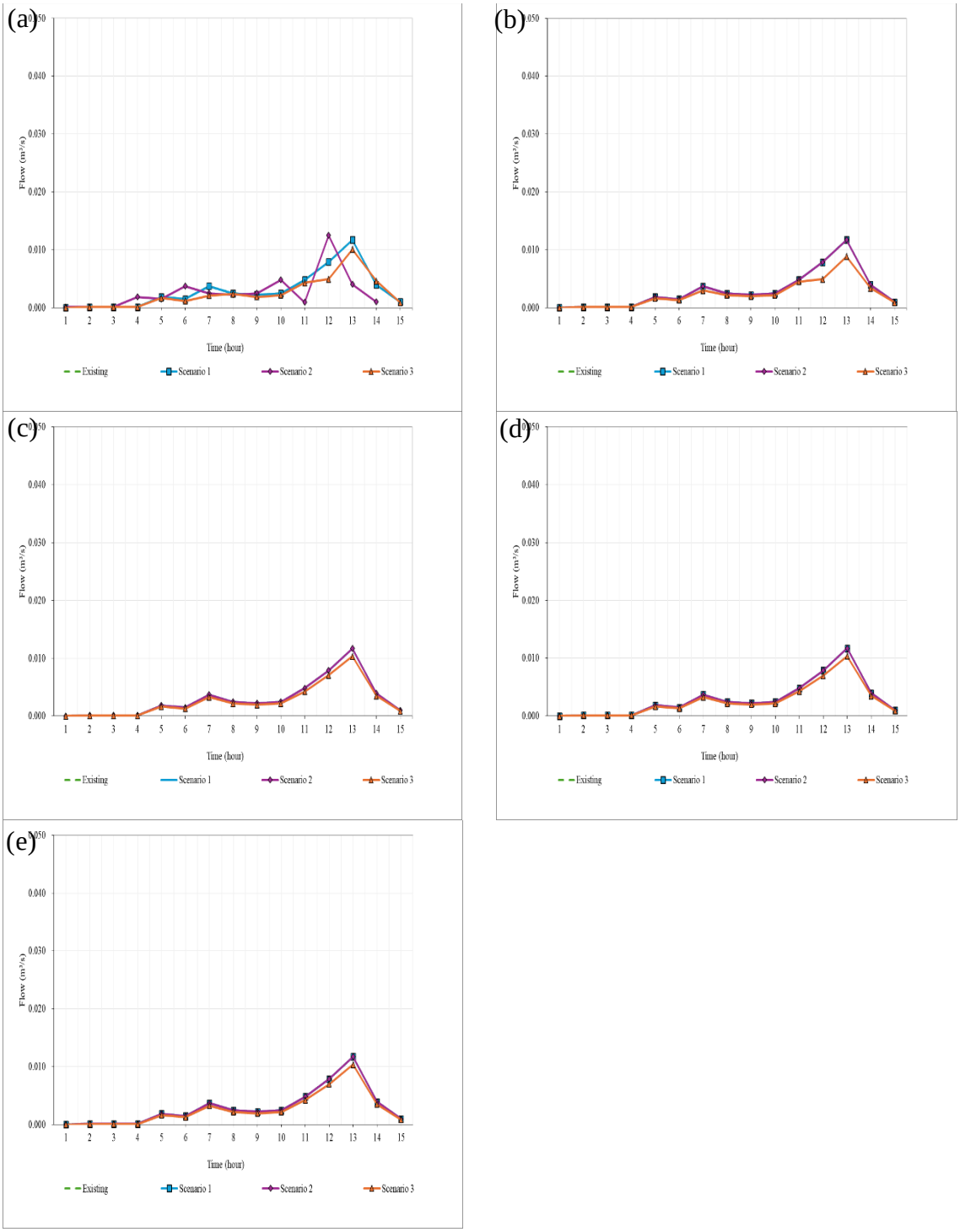


Figure AI.84: Flow Plots for Drain with 0.4% Slope at Location D Subjected to Event 2, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

Event 2 (Water Levels)

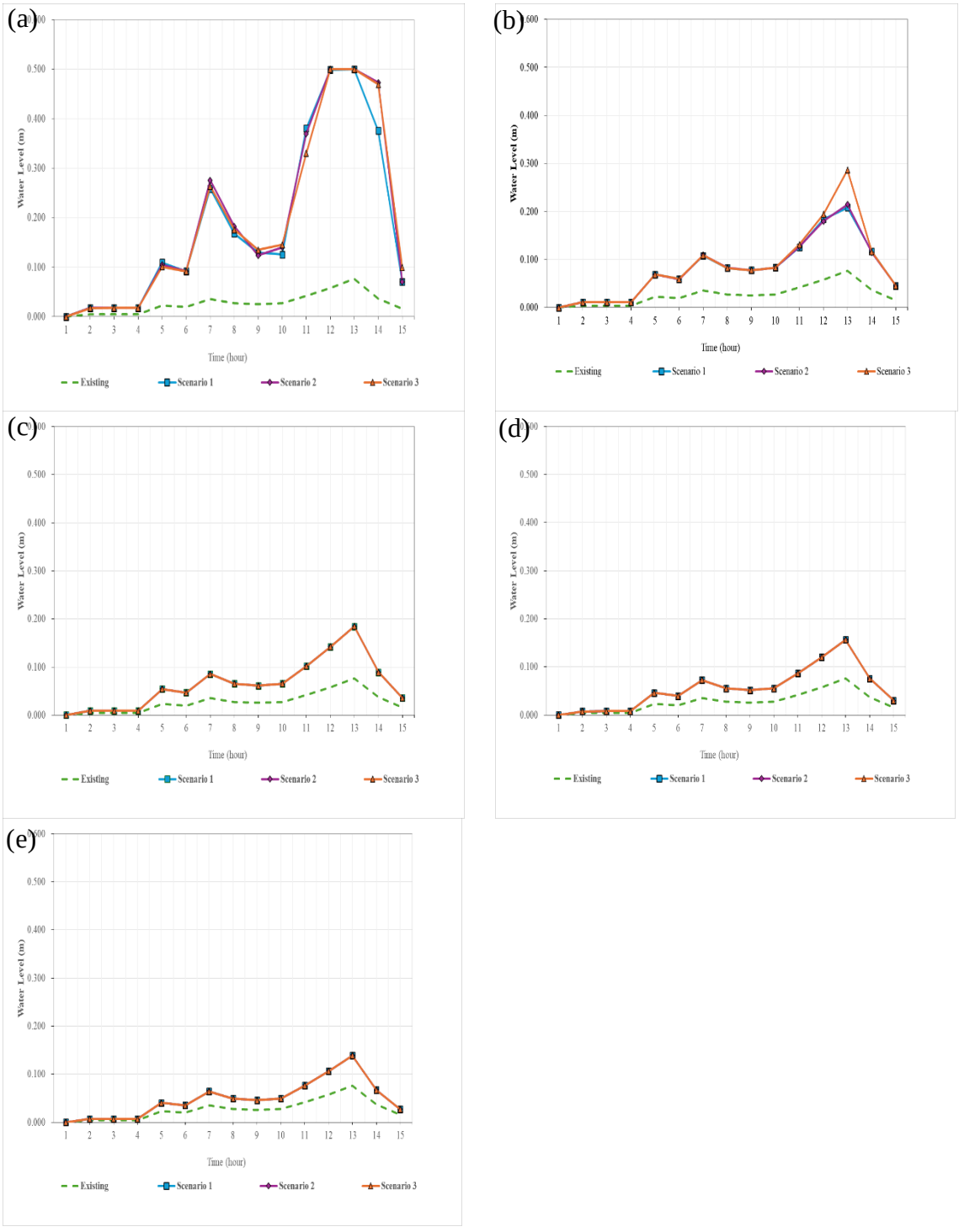


Figure AI.85: Water Level Plots for Drain with 0.4% Slope at Location A Subjected to Event 2, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

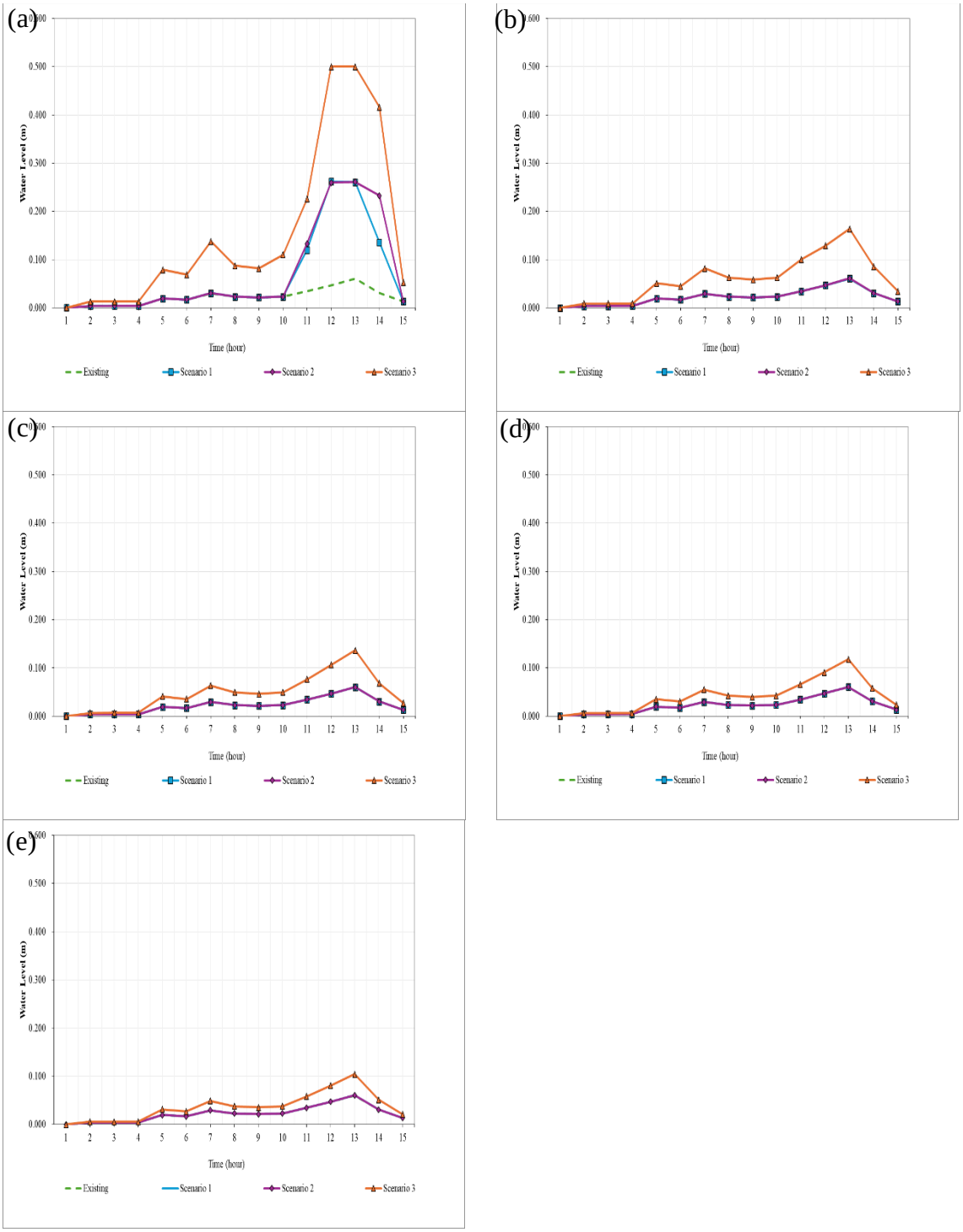


Figure AI.86: Water Level Plots for Drain with 0.4% Slope at Location B Subjected to Event 2, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

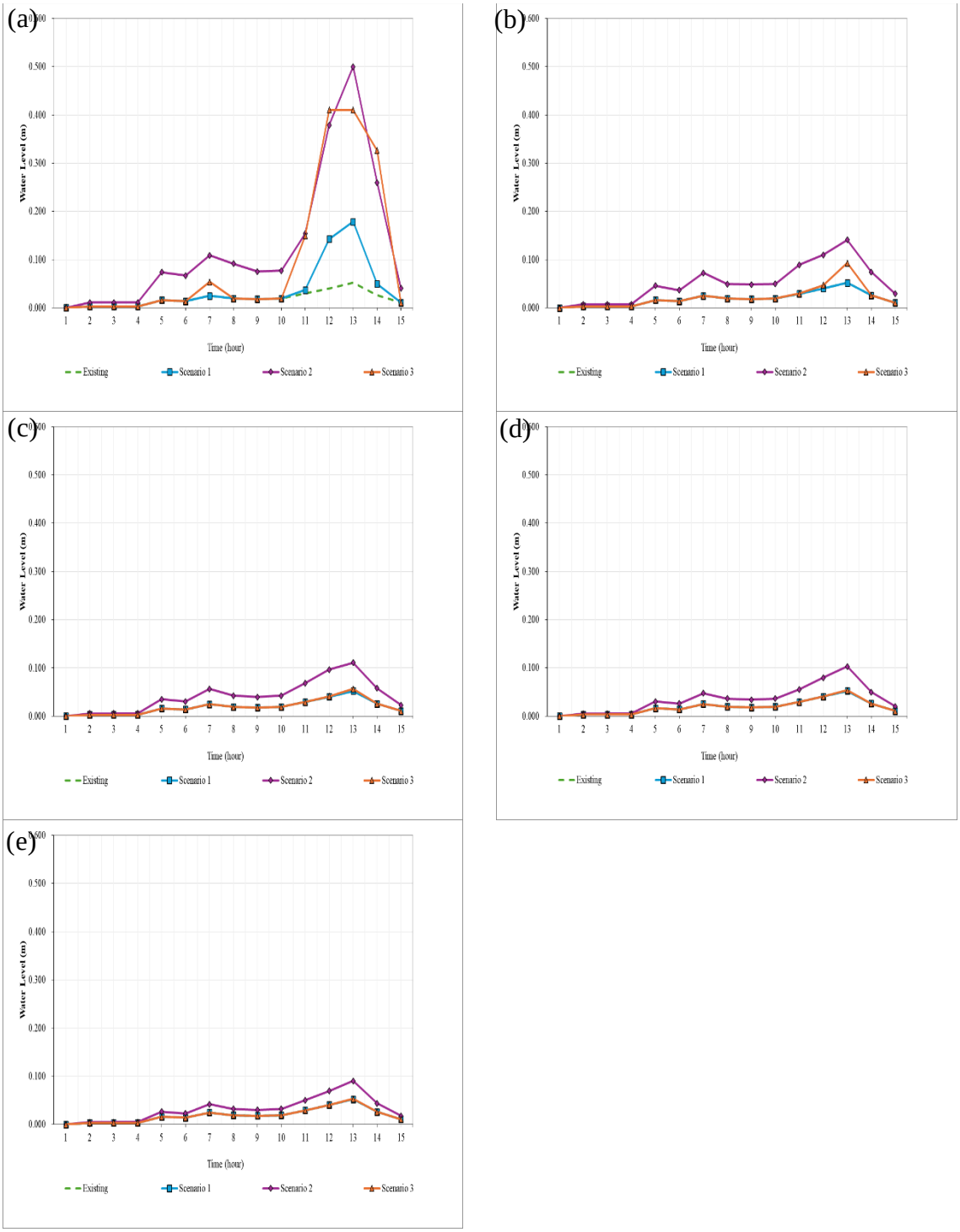


Figure AI.87: Water Level Plots for Drain with 0.4% Slope at Location C Subjected to Event 2, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

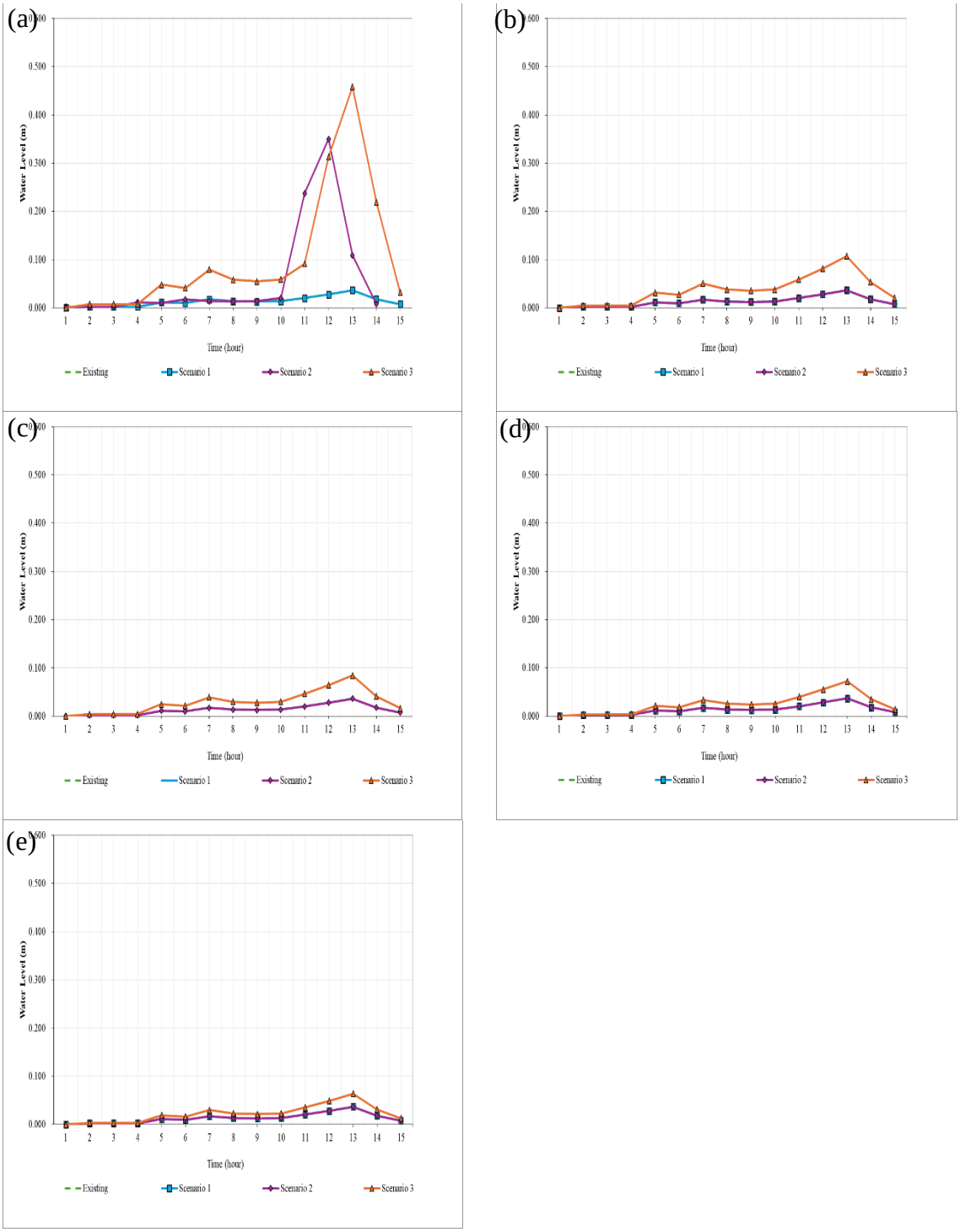


Figure AI.88: Water Level Plots for Drain with 0.4% Slope at Location D Subjected to Event 2, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

Event 3 (Flow Rates)

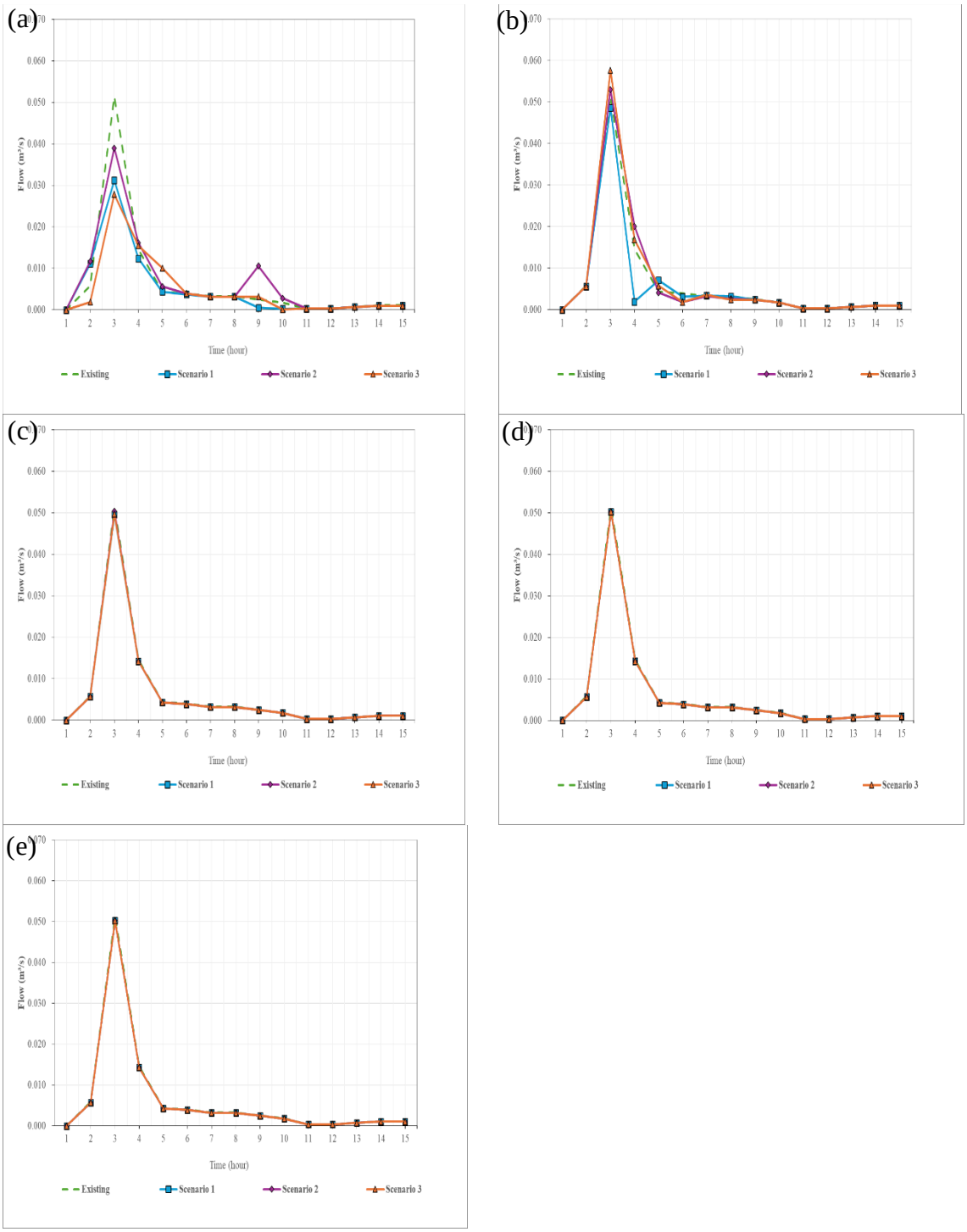


Figure AI.89: Flow Plots for Drain with 0.4% Slope at Location A Subjected to Event 3, (a) 0.1m, (b) 0. 2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

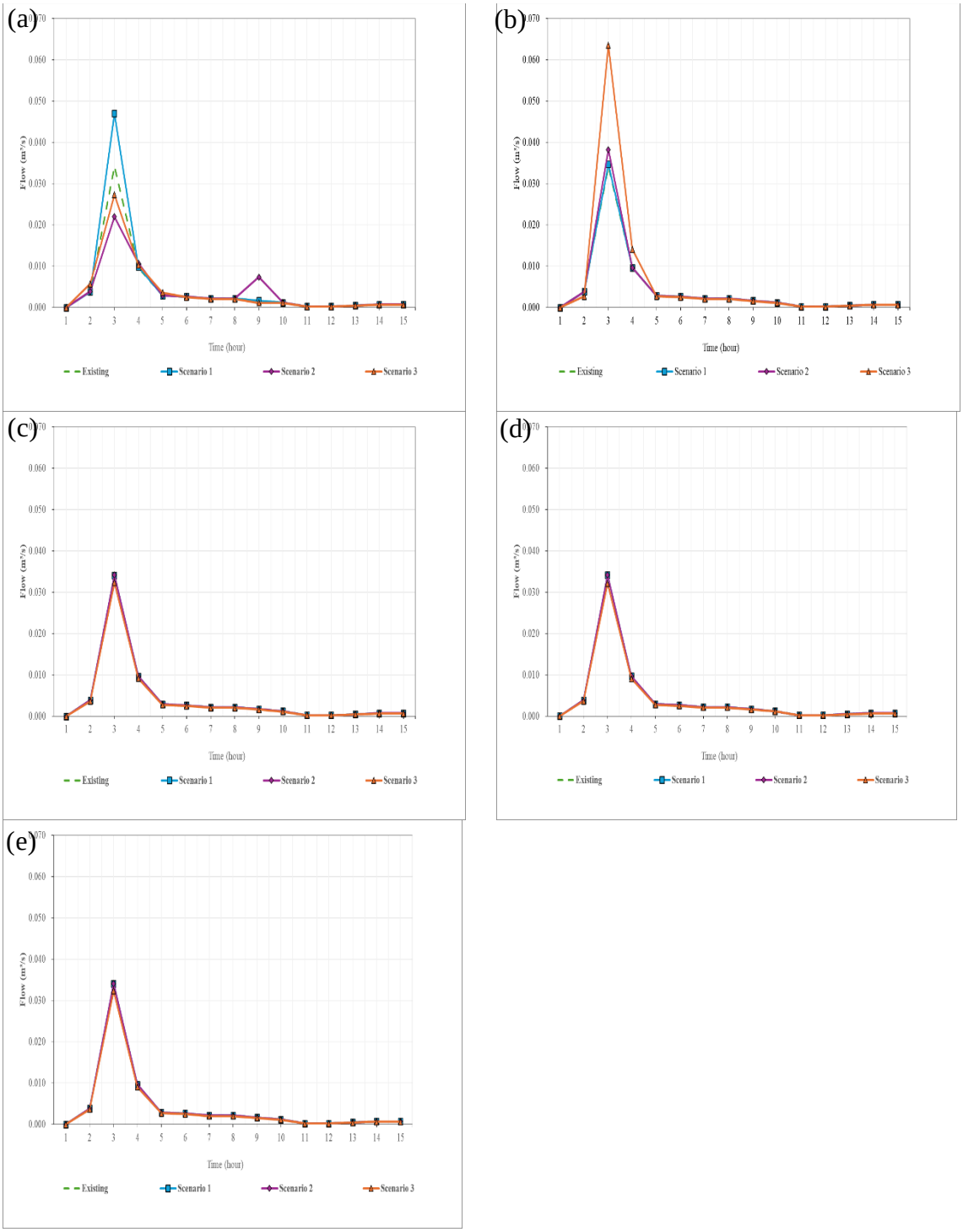


Figure AI.90: Flow Plots for Drain with 0.4% Slope at Location B Subjected to Event 3, (a) 0.1m, (b) 0. 2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

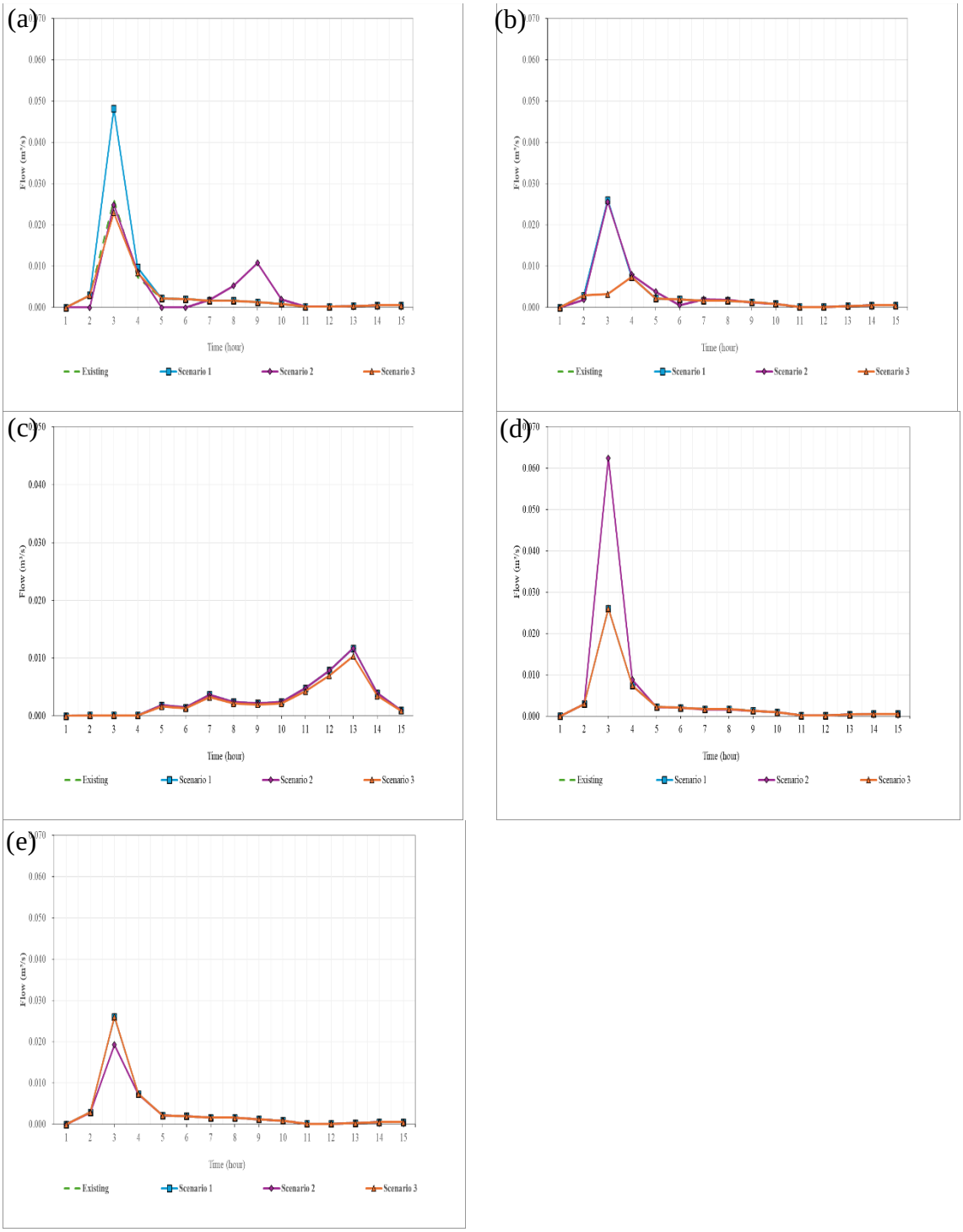


Figure AI.91: Flow Plots for Drain with 0.4% Slope at Location C Subjected to Event 3, (a) 0.1m, (b) 0. 2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

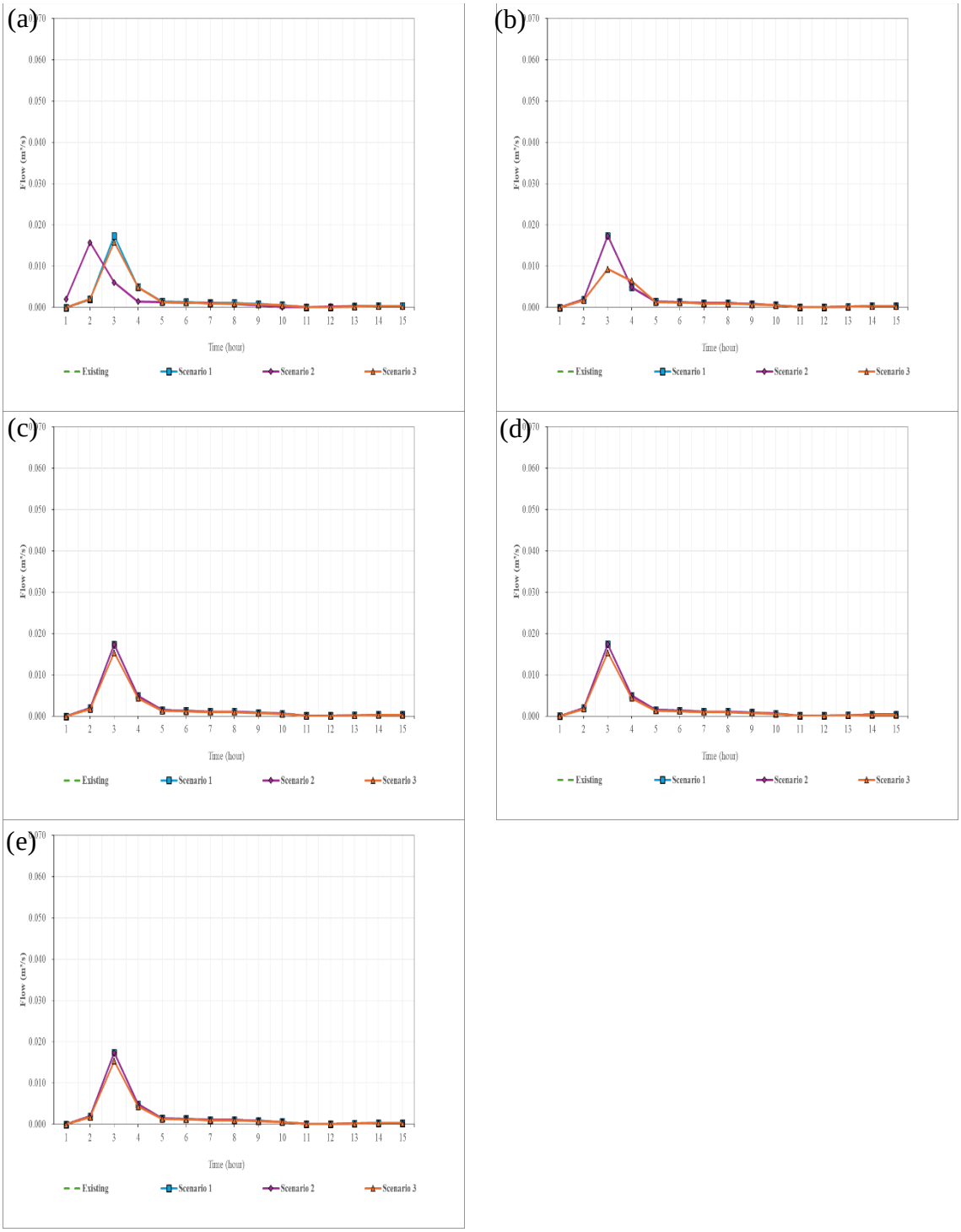


Figure AI.92: Flow Plots for Drain with 0.4% Slope at Location D Subjected to Event 3, (a) 0.1m, (b) 0. 2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

Event 3 (Water Levels)

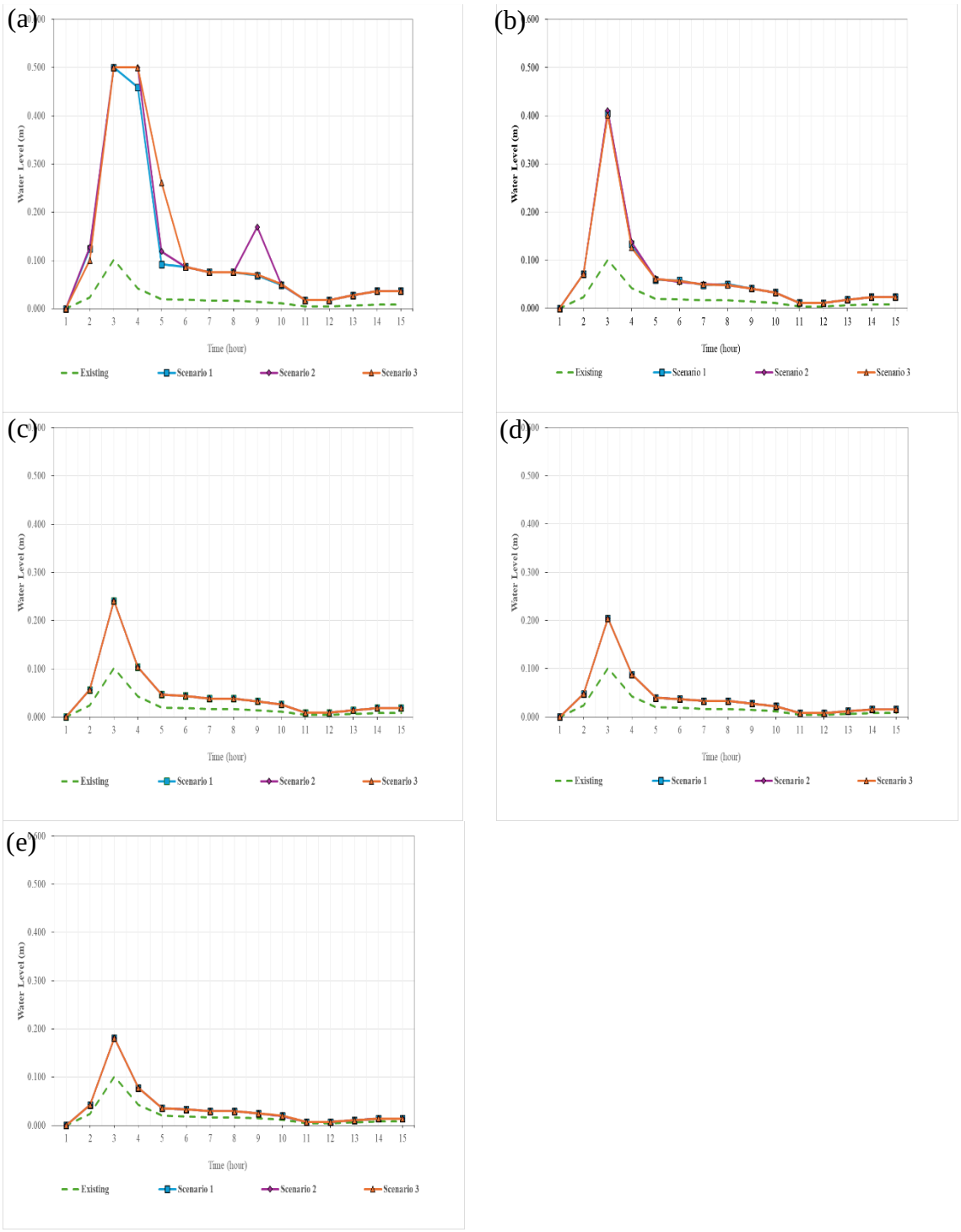


Figure AI.93: Water Level Plots for Drain with 0.4% Slope at Location A Subjected to Event 3, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

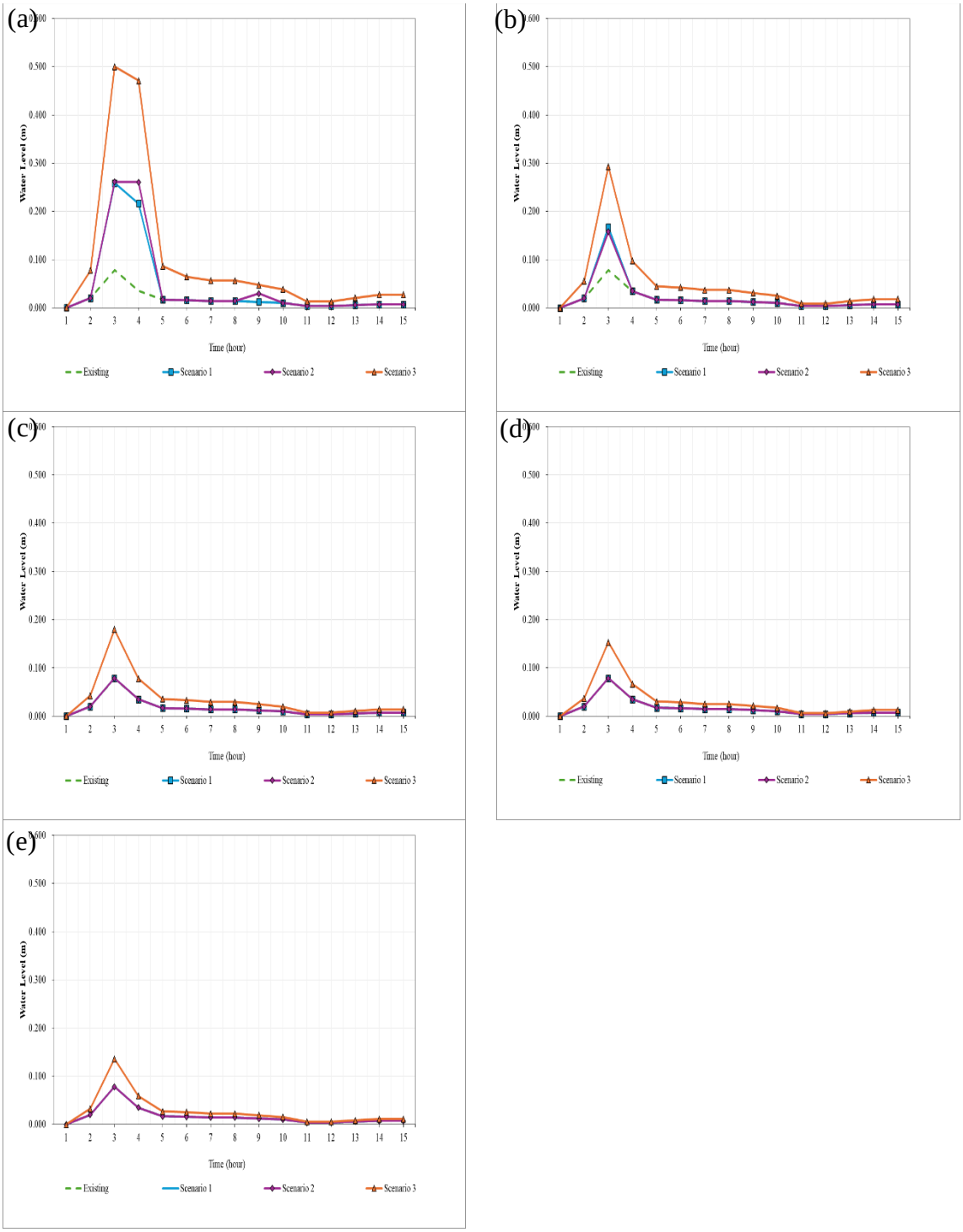


Figure AI.94: Water Level Plots for Drain with 0.4% Slope at Location B Subjected to Event 3, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

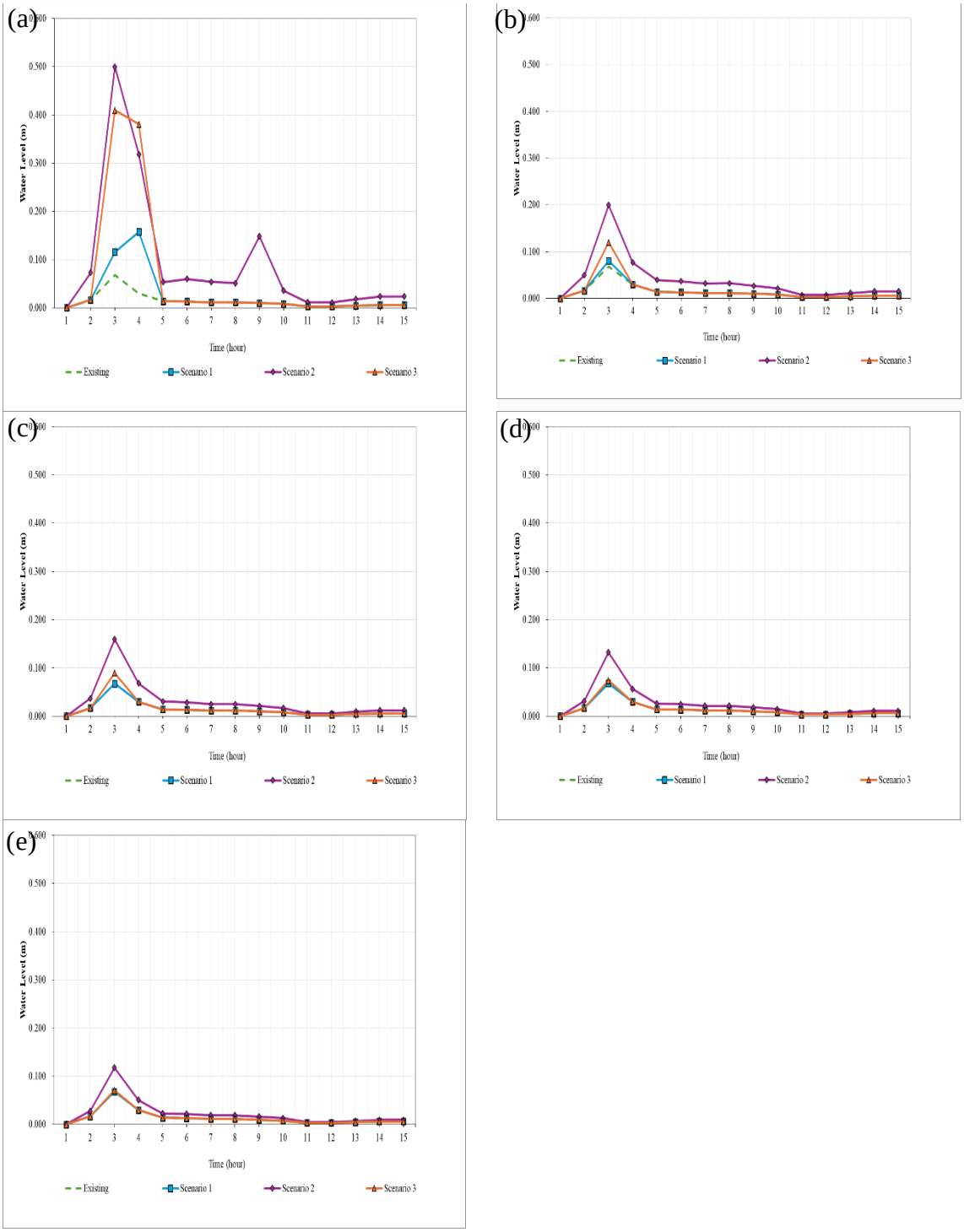


Figure AI.95: Water Level Plots for Drain with 0.4% Slope at Location C Subjected to Event 3, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice

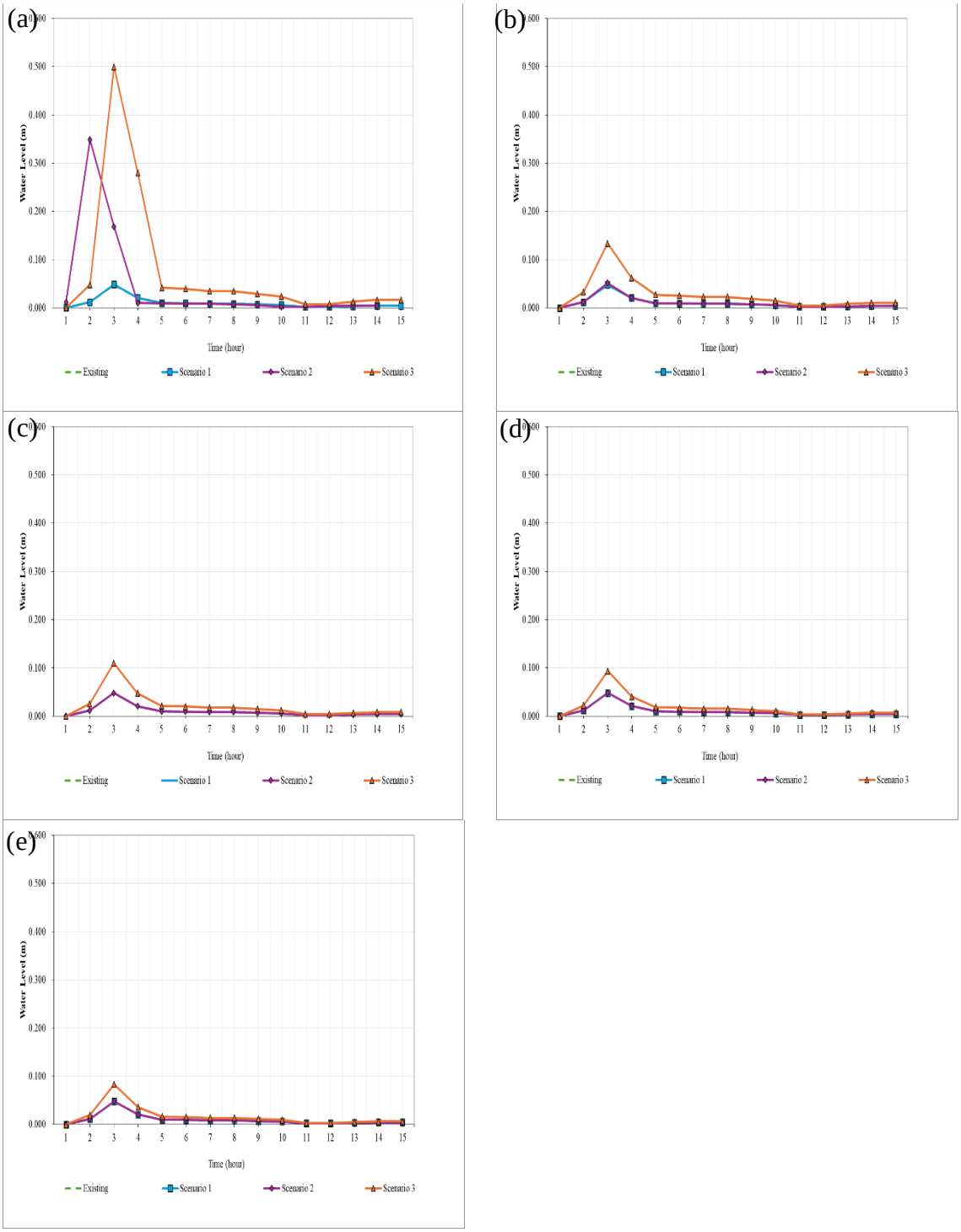


Figure AI.96: Water Level Plots for Drain with 0.4% Slope at Location D Subjected to Event 3, (a) 0.1m, (b) 0.2m, (c) 0.3m, (d) 0.4m, and (e) 0.5m Diameter of Orifice