การบรรเทาน้ำท่วมอันเนื่องมาจากฝนที่ตกในพื้นที่เขตบางกอกน้อยและบางกอกใหญ่ โดยใช้แบบจำลอง MIKE 11

นายปวีร์ คล่องเวสสะ

วิทยานิพนธ์นี้เป็นส่วนหนึ่งของการศึกษาตามหลักสูตรปริญญาวิทยาศาสตรมหาบัณฑิต

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ปีการศึกษา 2555

FLOOD MITIGATION FOR LOCAL RAINFALL
IN BANGKOK NOI AND BANGKOK YAI DISTRICTS
USING MIKE 11 MODEL

Mr.Pawee Klongvessa

A Thesis Submitted in Partial Fulfillment of the Requirements
for the Degree of Master of Science Program in Earth Sciences

Department of Geology
Faculty of Science
Chulalongkorn University

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By	Mr. Pawee Klongvessa
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ในช่วงหลายสิบปีที่ผ่านมา อิทธิพลของการเปลี่ยนแปลงสภาพภูมิอากาศและการเจริญเติบโตอย่างรวดเร็วของเมืองเป็นประเด็นที่ก่อให้เกิดการเปลี่ยนแปลงปริมาณฝนในหลายพื้นที่ ซึ่งอาจส่งผลกระทบต่อการประเมินและบรรเทาปัญหาน้ำท่วม งานวิจัยนี้มีจุดประสงค์เพื่อศึกษาการเปลี่ยนแปลงปริมาณฝนที่ส่งผลกระทบต่อการเปลี่ยนแปลงระดับน้ำในคลอง และหาแนวทางการบรรเทาน้ำท่วมในเขตบางกอกน้อยและบางกอกใหญ่ กรุงเทพมหานคร

การศึกษานี้ใช้วิธีการหาแนวโน้มค่าเฉลี่ยเคลื่อนที่ 10 ปี และวิธีแมน-เคนดอลในการหาแนวโน้มของปริมาณฝนรายปีและฝนสูงสุดรายวันในช่วงปี พ.ศ. 2525 ถึง 2553 และเปรียบเทียบปริมาณฝนสูงสุดรายวันสองช่วงเวลา โดยช่วงแรกระหว่างช่วงปี พ.ศ. 2525 ถึง 2539 และช่วงที่สองระหว่างปี พ.ศ. 2540 ถึง 2553 ที่คาบอุบัติซ้ำ 2, 5, 10, 25, 50 และ 100 ปี และประยุกต์ใช้แบบจำลอง MIKE 11 ในการประเมินการเปลี่ยนแปลงของระดับน้ำในคลองต่างๆ อันเนื่องจากการเปลี่ยนแปลงปริมาณน้ำฝนออกแบบที่คาบอุบัติซ้ำต่างๆ พร้อมเสนอแนวทางบรรเทาน้ำท่วมโดยกำหนดปริมาณการสูบน้ำและระดับน้ำเริ่มต้นที่ค่าต่างๆ ร่วมกับการสร้างแนวป้องกันน้ำท่วมและประตูระบายน้ำ

การศึกษานี้พบว่าปริมาณฝนรายปีมีแนวโน้มเพิ่มขึ้นขณะที่ปริมาณฝนสูงสุดรายวันมีแนวโน้มลดลงซึ่งส่งผลให้ระดับน้ำสูงสุดในคลองลดลงโดยเฉพาะสำหรับฝนที่คาบอุบัติซ้ำ 25, 50 และ 100 ปี บริเวณที่มีความเสี่ยงต่อน้ำท่วมมากที่สุดได้แก่บริเวณคลองชักพระ คลองบางขุนนนท์ และส่วนตะวันออกของคลองจักรทอง บริเวณที่มีความเสี่ยงต่อน้ำท่วมน้อยที่สุดได้แก่เขตบางกอกใหญ่ การสร้างแนวป้องกันน้ำท่วมสูง 0.75 เมตรเหนือระดับน้ำทะเลและการควบคุมระดับน้ำเริ่มต้นให้สูง 0.70 เมตรเหนือระดับน้ำทะเลสามารถบรรเทาน้ำท่วมที่เกิดจากฝนที่คาบอุบัติซ้ำ 10 ปีได้ และจากการศึกษาพบว่าการสร้างประตูระบายน้ำเพื่อป้องกันไม่ให้น้ำไหลย้อนกลับเข้าคลองบริเวณคลองวัดยางสุทธารามจะสามารถบรรเทาน้ำท่วมที่เกิดจากฝนที่คาบอุบัติซ้ำ 25 ปีได้

ภาควิชา ธรณีวิทยา ลายมือชื่อนิสิต ____________

สาขาวิชา โลกศาสตร์ ลายมือชื่อ อ.ที่ปรึกษาวิทยานิพนธ์หลัก ____________

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KEYWORDS : BANGKOK / FLOOD / FREQUENCY ANALYSIS / MANN-KENDALL TEST / MIKE 11 MODEL / RAINFALL

PAWEE KLONGVESSA : FLOOD MITIGATION FOR LOCAL RAINFALL IN BANGKOK NOI AND BANGKOK YAI DISTRICTS USING MIKE 11 MODEL. ADVISOR : ASSIST. PROF. SRILERT CHOTPANTARAT, Ph.D., 208 pp.

In the past decades, the influence of climate change and rapid growth rate of urbanization have caused changes in the amount of rainfall in many areas which may affect the flood assessment and mitigation. This research aims to investigate the change in the amount of rainfall which impacts on changes of the water levels in canals and evaluate the appropriate mitigation measures for floods in Bangkok Noi and Bangkok Yai districts of Bangkok.

Ten-year moving average and Mann-Kendall test were applied to determine the trend of the annual rainfall and maximum 1-day rainfall during the period of 1982-2010. The maximum 1-day rainfall in two periods, the 1982-1996 and the 1997-2010, were compared under different return periods of 2-, 5-, 10-, 25-, 50- and 100-year. The MIKE 11 model was then applied to assess changes of the water levels in canals caused by changes of amounts of design rainfall events for various return periods. The flood mitigation was also proposed by applying various pumping capacities and initial water levels, incorporating with building dykes and a floodgate.

This study has found that the annual rainfall has increased while the maximum 1-day rainfall has decreased which causes the maximum water level being lower, especially for 25-, 50- and 100-year rainfalls. The highest flood risk areas are along Chakphra and Bangkhunnon canals and the eastern part of Jakthong Canal while the lowest flood risk area is Bangkok Yai district. Flood caused from the 10-year rainfall can be mitigated by building dykes with the height of 0.75 m (MSL) and maintaining the initial water level of 0.70 m (MSL). Furthermore, it has also been found that flood caused from the 25-year rainfall can be mitigated by building the floodgate to prevent the flowing back water at Wat Yangsuttharam Canal.

Department : Geology Student's Signature ....

Field of Study : Earth Science Advisor's Signature ....

Academic Year : 2012

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I also would like to thank the Graduate School of Chulalongkorn University for providing the research support scholarship which gave me much convenience for the field observations and data gathering.

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CHAPTER I

INTRODUCTION

1.1 Rationale

The terrain of Bangkok is the floodplain with the average elevation of not more than 1 meters above the mean sea level (Camp, Dresser & McKee Consulting Engineers [CDM], 1968). There have been floods in Bangkok which have been caused by the runoff from upstream in the Chao Phraya basin, tidal floods from the Gulf of Thailand, and storm water from local rainfalls (ACE Consultco CAE and Asian Institute of Technology [AIT], 1986). Severe floods have occurred in 1942, 1983 and 1995 (AIT, Danish Hydraulic Institute [DHI] and Acres International Limited, 1996)

In 1987, flood mitigation plans in the area to the west of the Chao Phraya River in Bangkok and Samut Prakarn have been proposed by the Netherlands Engineering Consultant (NEDECO) and Span Company Limited (SPAN). This proposal aimed to protect the area from flooding with a return period of 100 years by building flood barriers to prevent flooding from the rainfall with a return period of 2 years by changing small irrigation canals and unnecessary irrigation canals to drainage canals, and using pumps in some areas (Netherlands Engineering Consultant [NEDECO] and Span Company Limited [SPAN], 1987).

However, the Intergovernmental Panel on Climate Change (IPCC) reported that the global temperature from 1956 to 2005 increased for 0.13^{o}C per decade in average. This increasing in the temperature might affect wind patterns and cause the amount of precipitation in some areas to increase or decrease (Intergovernmental Panel on Climate Change [IPCC], 2007). Besides, the shift in the Walker Circulation is believed to cause the negative relationship between the amount of the summer rainfall in Thailand and the El Niño-Southern Oscillation (ENSO) since 1980 (Singhratina et al., 2005). The annual mean temperature of Thailand has increased over this period while the annual rainfall from 1951 to 2005 decreased (Bangkok Metropolitan Administration [BMA], Green Leaf Foundation [GLF] and United Nations Environment Programme [UNEP], 2009).

An urbanization may also be a significant contributory factor that causes the amount of rainfall to change since it can cause an urban heat island (UHI), more surface roughness, and condensation nuclei from pollution (Chandler, 1965). The data from the Survey and Mapping division, Department of City Planning, BMA revealed that from 1968 to 2002, the residential area in Bangkok had expanded from 181 to 336 km^2, and the commercial area had expanded from 543 to 370 km^2 while the agricultural and vacant areas were reduced from 543 to 370 km^2 and from 624 to 379 km^2, respectively (BMA et al., 2009). The urbanization of the Bangkok metropolitan area has possibly caused the local precipitation patterns to change.

The Bangkok Noi and Bangkok Yai districts have the landuse type of high-density residential areas with Thai art and cultural conservation areas along the Chao Phraya River (Department of City Planning, BMA, 2006). According to the field investigations in 2012, there were many drainage canals in the area with poor water quality and some traces of flooding were also found along the west part of the area. The flood and poor water quality can affect the health and well-being of the people living in this area. However, these problems can be solved by proper flood mitigation (บริษัท ทีม คอนซัลติ้ง เอนจิเนียริ่ง แอนด์ แมเนจเมนท์ จำกัด, 2546).

This study aims to determine the trend of rainfall in the area of the Bangkok Noi and Bangkok Yai districts surrounded by the Bangkok Noi Canal, Chakphra Canal, Bangkok Yai Canal, and Chao Phraya River, and to apply the MIKE 11 model to determine effects of the changing rainfall on canals water levels and evaluate the flood mitigation for the local rainfall with the return period of 2, 5, 10, 25, 50, and 100 years which can account for these effects.

1.2 Objectives

1. To determine of the trend of rainfall in the Bangkok Noi and Bangkok Yai districts.
2. To determine of the effects of the changing rainfall on canals water levels in the Bangkok Noi and Bangkok Yai districts.
3. To evaluate the flood mitigation for the local rainfall with the return period of 2, 5, 10, 25, 50, and 100 years in the Bangkok Noi and Bangkok Yai districts.

1.3 Hypothesis

Amount of rainfall in Bangkok has been subjected to change because of the climate change and urbanization. The change in rainfall amount may affect water levels in canals. Hence, the flood mitigation should be adapted to be able to account for the present effects of urbanization and climate change.

1.4 Scope

The determination of the trends of annual and maximum 1-day rainfall was done in the inner Bangkok which covers both 2 sides of Chao Phraya River with a total area of approximately 130 km^2. The period of the study was from 1982 to 2010.

Amounts of maximum 1-day rainfall with the return periods of 2, 5, 10, 25, 50, and 100 years were compared between the periods of 1982-1996 and 1997-2010 in the study area which is the part of the Bangkok Noi and Bangkok Yai districts covering the total area of approximately 15 km^2.

The canal water level from the maximum 1-day rainfall with the return period of 2, 5, 10, 25, 50, and 100 years were compared between the periods of 1982-1996 and 1997-2010 for each pumping capacity in the study area.

The determination of the flood mitigation involved controlling initial the water level before the rainfall event, adjusting the pumping capacity, building dykes, and building a floodgate.

1.5 Expected outcomes

1. Knowing the trend of rainfall in the inner Bangkok.
2. Knowing the effects of the changing rainfall on canals water levels in the Bangkok Noi and Bangkok Yai districts.
3. Knowing the flood mitigation for the local rainfall with the return period of 2, 5, 10, 25, 50, and 100 years in the Bangkok Noi and Bangkok Yai districts.

CHAPTER II

LITERATURE REVIEWS

2.1 The study area

2.1.1 Location

The location of the study area is shown in Figure 2-1. The area is in the western part of Bangkok and covers a total area of 15 km^2. The northern part of the area is in the Bangkok Noi District and the southern part of the area is in the Bangkok Yai District. With reference to the WGS84 datum, the area is located in zone 47N in the northern hemisphere at an easting of 657075 – 661284 of and a northing of 1517502 – 1523856. The Bangkok Noi District is bounded by the Boromrajchonnee Road in the north, Bangkok Noi and Chakphra canals in the west, the Mon Canal in the south, and the Chao Phraya River in the east. The Bangkok Noi Canal also passed through this district from the west to the east. The Bangkok Yai District is located to the south of the Bangkok Noi District. This district is bounded by the Mon Canal in the north, the Bangkok Yai Canal in the west and the south, and the Chao Phraya River in the east. The study area consists of the Bangkok Yai District and the part of the Bangkok Noi District on the south side of the Bangkok Noi Canal.

2.1.2 Characteristics

The study area has the characteristics of a low elevation floodplain. The elevation across the area ranges from 0.0 to 3.5 m (MSL) with the highest elevation in the east and the lowest elevation in the northwest. However, the landform has been transformed by the flow of fill materials (Hara, Takeuchi and Thaitakoo, 2008). Figure 2-2 shows the elevation of the study area interpolated from the bank elevation data (see Table 5-2 and the Appendix E).

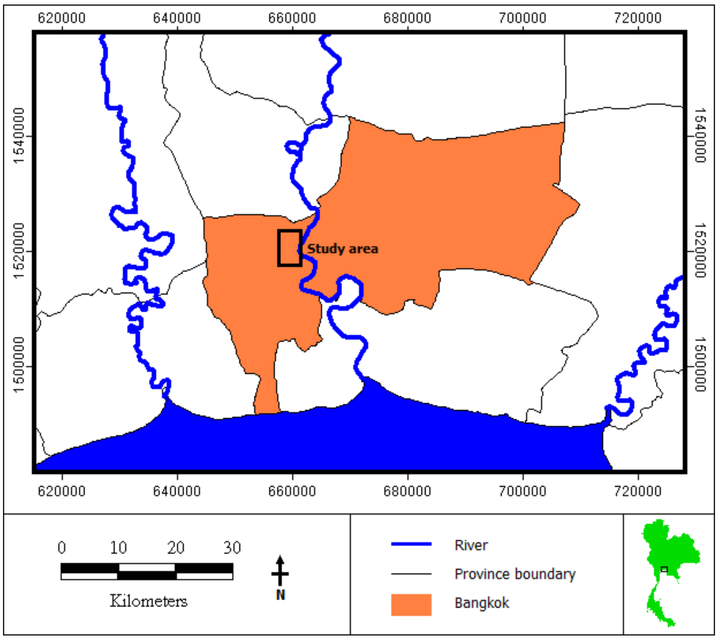
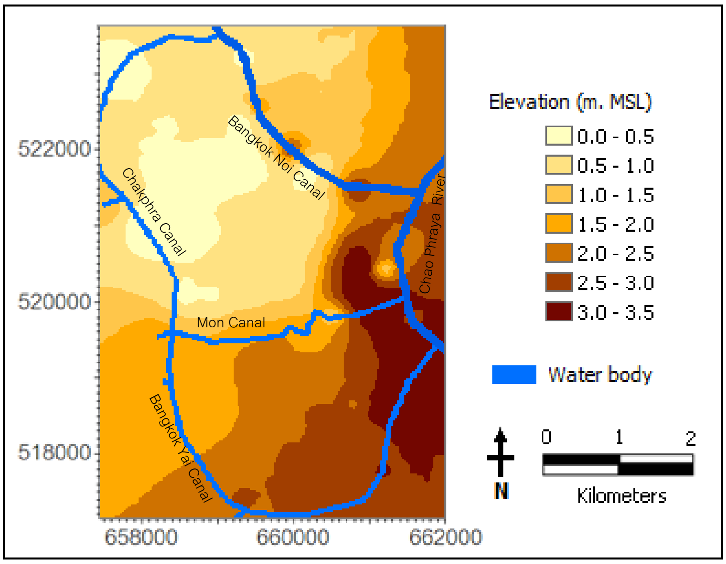


Figure 2-1 Location of the study area



Source : Interplated from the bank elevation of the Chao Phraya River obtained from the Royal Irrigation Department and elevation data at the survey points of this study which were obtained from the digital elevation model derived by the Land Development Department and the benchmarks of the Public Works Department

Figure 2-2 Elevation of the study area

According to the Bangkok Land Use Comprehensive Plan B.E. 2549 (Department of City Planning, BMA, 2006), the study area is mainly characterized by high-density residential areas. Along the Chao Phraya River, there are some Thai art and cultural conservation areas and government, institute, infrastructure areas, and there is also a small commercial area at the center of the Bangkok Noi District. Figure 2-3 shows the land-use plan of the area.

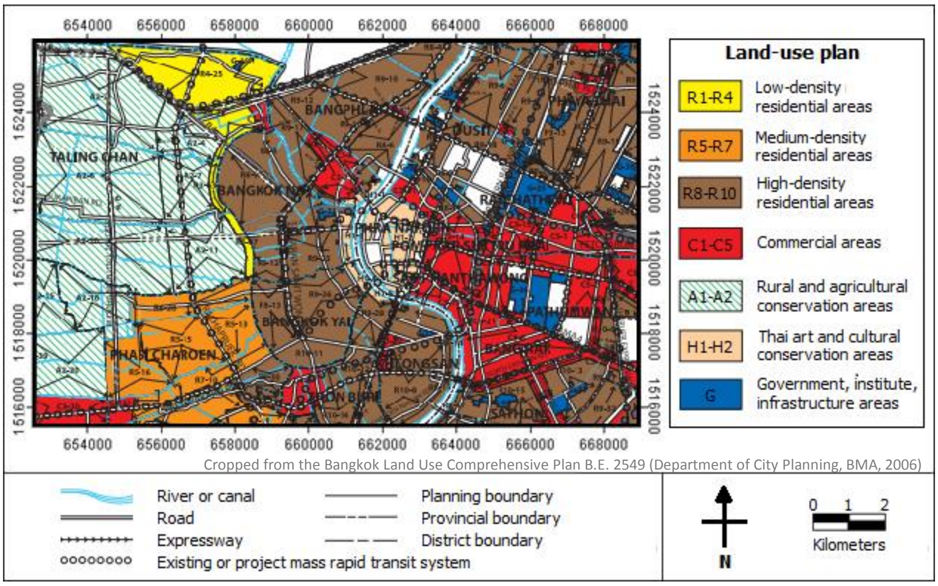


Figure 2-3 Land-use plan of the study area (Department of City Planning, BMA, 2006)

2.1.2 Meteorological data

The climate in the area is dominated by the southwest monsoon, which brings warm and wet air from the Indian Ocean between May and October, and the northeast monsoon, which brings cold and dry air from China between October and February. As a result, the amount of rainfall is high between May and October. Moreover, additional tropical storms from the Bay of Bengal around May and from the South China Sea around October can result in heavy rainfall in these two months. From April to May, there is also a high possibility of thunderstorm occurrences caused by the meeting of hot the air mass in the area with the upper cold air mass from China (กรมอุตุนิยมวิทยา, 2550).

In 1984, the Japan International Cooperation Agency (JICA) derived the rainfall intensity-duration-frequency curve (IDF curve) in the eastern suburban-Bangkok using the rainfall data from 1951 to 1982 in average among the stations of Don Muang, Bang Khen, Bangkok, Bang Na, Bang Kapi, Minburi, and Lat Krabang. The relationships between rainfall intensity and duration for each return period were shown in Table 2-1 (Japan International Cooperation Agency [JICA], 1984).

Table 2-1 Rainfall intensity (mm/h) for various durations and return periods in the eastern suburban-Bangkok (JICA, 1984)

Duration	Return period (years)				
	2	5	7	10	20
5 minutes	11.3	14.1	14.9	15.7	17.1
10 minutes	20.2	25.3	26.9	28.4	31.0
15 minutes	25.0	31.7	33.7	35.7	39.2
30 minutes	42.5	54.3	58.0	61.5	67.9
1 hour	58.7	76.0	81.5	86.8	96.5
2 hours	72.4	95.0	102.2	109.2	122.4
6 hours	85.8	114.0	123.0	132.0	149.4
12 hours	90.0	120.0	129.6	139.2	157.2
24 hours	93.6	122.4	134.4	144.0	163.2

In 1996, the NEDECO, SPAN, and Water Development Consultant Company Limited (WDC) derived IDF curves from the rainfall data during 1951 – 1982 at two stations at the Royal Irrigation Department and Thai Meteorological Department. The relationships between rainfall intensity and duration for each return period were shown in Tables 2-2 and 2-3 for the stations at the Royal Irrigation Department and Thai Meteorological Department, respectively (เนเธอร์แลนด์ เอนจิเนียริง คอนซัลแต็นซ์, บริษัท วอเตอร์ ดีเว็ลลัฟเม็นท์ คอนซัลเท็นส์ จำกัด และ บริษัท สแปน จำกัด, 2539).

Table 2-2 Rainfall intensity (mm/h) for various durations and return periods at the Royal Irrigation Department (เนเธอร์แลนด์ เอนจิเนียริง คอนซัลแต็นซ์ และคณะ, 2539)

Duration	Return period (years)								
	2	5	10	25	50	100	200	500	1000
15 minutes	108	130	144	163	177	190	204	222	235
30 minutes	85	105	118	135	147	159	172	188	200
1 hour	59	75	85	98	108	117	127	139	149
2 hours	34.6	44.2	50.5	58.5	64.5	70.4	76.2	84.8	89.8
6 hours	12.5	15.8	18.0	20.8	22.9	24.9	27.0	29.7	31.7
12 hours	6.6	8.2	9.2	10.6	11.6	12.6	13.6	14.9	15.8
24 hours	3.4	4.2	4.8	5.5	6.0	6.5	7.0	87.6	8.1

Table 2-3 Rainfall intensity (mm/h) for various durations and return periods at the Thai Meteorological Department (เนเธอร์แลนด์ เอนจิเนียริง คอนซัลแต็นซ์ และคณะ, 2539)

Duration	Return period (years)								
	2	5	10	20	50	100	200	500	1000
5 minutes	157	256	321	384	465	525	586	666	726
10 minutes	134	196	237	277	328	366	404	455	493
15 minutes	120	166	196	225	263	291	319	356	384
30 minutes	92	119	138	155	178	195	212	234	251
1 hour	62	78	89	100	113	124	134	147	158
2 hours	36.6	48.7	56.7	64.4	74.4	81.9	89.3	99.2	106.6
6 hours	13.6	18.3	21.3	24.3	28.1	30.9	33.8	37.5	40.3
12 hours	7.2	9.5	11.0	12.4	14.3	15.7	17.1	19.0	20.4
24 hours	4.0	5.3	6.1	6.9	8.0	8.8	9.6	10.6	11.4

Sungkhanuam (2003) had found that about 50% of rainfalls had one center while 40% of them moved southward and most of the rainfall occurred between 6 pm and 12 am with the duration of 1-3 hours.

2.1.3 Flood

Bangkok can be flooded by high tides in the Chao Phraya River and local rainfalls. In 1968, CDM proposed the plan to mitigate the flood in Bangkok and Thonburi by dividing the area into small polders. Each polder was surrounded by either the river or multipurpose canal to carry and drain the water from the internal rainfall which was drained out from the polder by drainage canals and pumps. Each polder was also protected from the external flood by flood barriers, floodgates and navigation locks (CDM, 1968). In 1987, the NEDECO designed the flood mitigation in Thonburi and Samut Prakan west which aimed to prevent the area from 100-year flood due to the tides in the Chao Phraya River by barriers and to prevent the area from flood due to 2-year local rainfall using drainage canals and pumps (NEDECO and SPAN, 1987). Sudpuang (1999) had calculated that the drainage system could drain the water for 1,057,482 m^3 per hour by regulating gates and 500,778 m^3 per hour by pumping stations. These mitigations could reduce the water level from the critical level of 1.9 m (MSL) to 1.0 m (MSL) within 14.62 days when 5-year rainfall with the duration of 24 hours occurred and 10.46 days when there was no rainfall.

In 2003, the TEAM used the MIKE 11 model to design the drainage system and suggested that the floodgates could be opened to drain the water when the tide in the Chao Phraya River was below 1.2 m (MSL). When the tide was above 1.2 m (MSL), the floodgates must be closed and the water could be drained by pumps (บริษัท ทีม คอนซัลติ้ง เอนจิเนียริ่ง แอนด์ แมเนจเมนท์ จำกัด, 2546). However, flood can still occur when the structures are not operated appropriately, the tides exceed the barrier, or rainfall intensity exceeds the pumping capacity combined with high tides in the Chao Phraya River (สำนักการระบายน้ำ กรุงเทพมหานคร, 2553). Pictures of flood protection structures, canals, and traces of flood are shown in Figures 2-4, 2-5, and 2-6 respectively.

(a) (b)

Figure 2-4 (a) Main floodgate and pump at the Chakphra Canal, and (b) dyke along the Chakphra Canal with the floodgate at the Jaoarm Canal

(a) (b)

Figure 2-5 (a) Bangkok Yai Canal, the main canal and (b) Wat Dongmullek Canal, the drainage canal

(a) (b)

Figure 2-6 Traces of flood near (a) the Bangkhunnon Canal and (b) the Wat Deeduad Canal

There has also been Klong Mahachai - Klong Sanamchai Monkey Cheek project which aimed to reduce flood from the local rainfall by building detention storage with the capacity of 6,000,000 m^3 in the west bank of the Chao Phraya River (สำนักการระบายน้ำ กรุงเทพมหานคร, 2553). Klong Sanamchai or the Sanamchai Canal was a canal lying from the Bangkok Yai Canal to the southwest. Klong Mahachai or the Mahachai Canal was the canal continuing from the Sanamchai Canal to the Tha Chin River, another river draining water from the floodplain to the Gulf of Thailand apart from the Chao Phraya River. Klungsupavipat (2000) studied the potential of this project and suggested that this project by the MIKE 11 model combined with canal excavation could mitigate flood with a return period of 2 years but could not mitigate the flood with a return period of 5 years or longer.

Jutanka (2004) used the MIKE 11 model to study the effect of this project combined with other flood mitigations including floodgates and pumps operation, and building dykes. The result suggested that this project combined with building dykes along the Mahachai Canal with a height of 2.75 m (MSL) could protect the area from flood with a return period of 100 but pumps with the capacity of 300 m^3 per second should be used while the dyke was not completed.

Boonya-aroonnet et al. (2009) applied different floodgates operations at the Mahachai Canal and Luang Canal, the canal which divert the water from the Mahachai canal to the Chao Phraya River in the MIKE 11 model. The result suggested that when there was a high tide, closing the floodgate at the Mahachai Canal could lower the water level in the Mahachai Canal to prevent flood while opening the floodgate at the Luang Canal could improve the water quality. These two mitigations could be done together but the effectiveness would be lower than when only one of these mitigations was applied.

2.2 Regional changes in rainfall

It had been shown in the report of the IPCC that the global temperature had increased with the average rate of 0.13^{o}C per year from 1956 to 2005 in average (IPCC, 2007). The increasing temperature tends to increase a frequency of El Niño (Bacher et al., 1999). Because the amount of monsoon rainfall in Thailand has had a negative relationship with the ENSO since 1980 (Singhratina et al., 2005), it is inferred that the global warming causes Thailand having more frequency of the year with less amount of monsoon rainfall. There have been many studies of rainfall trends in the surrounding areas of Thailand.

Wu, Yang and Yu (2002) studied the impacts of climate change on rainfall in the southern of Taiwan. About 90% of rainfall in the area occurred between May and October. The transition probabilities of dry days and wet days were determined by the Markov chain. Mean daily precipitation, number of wet days, and transition probabilities of dry days and wet days from 1932 to 1998 were determined for each month by the Mann-Kendall test. The result was that the mean daily precipitation increased from January to May and from August to September and decreased from October to November, the number of wet days increased in January and decreased from June to December. The probability that the day was dry when the previous day had been dry increased in March, from June to October and in December, and the probability that the day was wet when the previous day had been wet increased in January and August and decreased from February to April, from June to July and from August to December. The study also applied the HBV hydrological model designed by the Swedish Meteorological and Hydrological Institute to determine the effect of the change in transition probabilities of dry days and wet days on the runoff amount. The input of wet or dry day was determined by the Markov chain and the amount of daily rainfall was determined by the

Weibull distribution. Data from 1981 to 1990 was used for the calibration and data from 1991 to 1992 was used for the verification. The model was run in two cases, when the trends of the transition period of wet or dry day were determined and when they were not. In both two cases, mean daily runoff increased between May and September and decreased in other months. However, when the trends of transition period of wet or dry day were determined, the mean daily runoff appeared to be lower than when they were not. It was concluded that the transition period of wet or dry day affected the runoff in the area.

Hayashi el al. (2003) determined the relationship between changes in mean rainfall intensity (MRI) and the southern oscillation index (SOI) during the southwest monsoon period in Sri Lanka. Data from 1960 to 1996 collected in 187 stations in Sri Lanka was used for the analysis. The mean of rainy days, the mean of MRI, the standard deviation of MRI and the slope of a linear regression of MRI were also determined for each station. The zone of high rainy days and mean of MRI was the southwestern part of the country. The zones of high standard deviation of MRI were the northern, northwestern and eastern parts. The negative slope of the linear regression of MRI was found in the south western part while other parts had positive slopes. The principal component analysis among the selected 77 stations in an equal-sized grid showed three dominant principle components. The first one showed the pattern of MRI in the north half of the country while the second one and the third one showed that in the south half and southwestern coastal belt, respectively. SOI was calculated from archives of the Commonwealth Scientific and Industrial Research Organisation (CSIRO) of Australia. The correlation coefficient between SOI and the change in MRI was significantly negative at the significance level of 0.05 for the first component, but were not significant for the second and third components.

Goswami et al. (2006) studied trends of daily rainfall variance, number of days with moderate rainfall (5 mm - 100 mm), number of days with heavy rainfall (100 mm - 150 mm), and number of days with very heavy rainfall (>150 mm) in India. The linear trends were determined and the t-test was done. The result showed an increasing in daily rainfall variance at the significance level of 0.01. Number of days with moderate rainfall decreased at the significance level of 0.1 while both number of days with heavy rainfall and number of days with very heavy rainfall increased at the significance level of 0.01.

Al-Tabbaa and Pal (2009) studied trends of number of dry days and total rainfall on wet days, the day with the rainfall amount of more than 1 mm, in autumn, spring, and winter from 1954 to 2003 in Kerala, India by the Mann-Kendall test with the significance level of 0.05. The results showed that the total rainfall on wet days significantly decreased in spring and increased for some part of the area in winter and autumn. The significant decreasing in total rainfall on the wet days in spring was also shown. Possibility of the delay in monsoon onset was suggested by the result.

Al-Tabbaa and Pal (2010) studied frequency and magnitude of extreme monsoon rainfall excesses and deficits in India. The area was divided into five regions, the northwest, the central northwest, the northeast, the west central, and the peninsular. The lower and upper quartiles of rainfall amount between June and September from 1871 to 2005 were calculated for each region, and then trends of the frequency and average magnitude of rainfall excess and deficits for each 15 years were determined by the Mann-Kendall test. Deficits frequency increased in every region except in the peninsular region where it had no trend. Deficits magnitude increased in every region except in the peninsular region where it decreased and in the northeast where it had no trend. Excesses frequency decreased in every region except in the northeast where it increased. Excesses magnitude decreased in every region except in the peninsular where it increased.

Hu, Maskey and Uhlenbrook (2012) determined trends of seven rainfall indices in the Yellow River Basin for each season during the period of 1960-2006. The seven indices included total precipitation, maximum number of consecutive dry days, maximum number of consecutive wet days, number of events the rainfall amount of which were more than the long-term 90^{th} percentile, fraction of total rainfall from those events, mean rainfall on wet days, and maximum rainfall in 5 consecutive days. The trends of these indices were determined by the Mann-Kendall test. Changes in indices were found in winter, spring, and summer. In winter there were increasing trends in the maximum rainfall in 5 consecutive days and the total precipitation. In spring, there were decreasing trends in the maximum number of consecutive dry days and the fraction of total rainfall from events the rainfall amount of which were more than the long-term 90^{th} percentile and an increasing trend in the total precipitation in spring. In summer, there were decreasing trends in the number of events the rainfall amount of which was more than the long-term 90^{th} percentile and the fraction of total rainfall from those events. It was concluded that the rainfall increased in winter and spring while frequency of extreme events and amount of rainfall from the extreme events decreased in summer.

2.3 Effects of urbanization on rainfall

Urbanization can enhance precipitation in many ways. Pollutions in cities can be the condensation nuclei. Furthermore, surface roughness of the land due to the urbanization can cause the air mass turbulence. Moreover, the urban heat islands can cause the thermal convection (Chandler, 1965). There has been much research which support that urbanizations have affected rainfall in urban areas and also their downwind areas.

Reality and causes of urban rainfall was first studied as the major field program in the Metropolitan Meteorological Experiment (METROMEX) with the objectives of identifying the reality of the urban rainfall anomaly, determination for its causes, and development of a means to predict it (Changnon, Huff and Semonin, 1976). The study concentrated on summer time in the location of St. Louis where there were two major urban areas, St. Louis and the industrial area to north of the city at the Alton-Wood River. The area to the east of them was their downwind area. The average summer rainfall from 1941 to 1968 in the available stations (Changnon and Huff, 1973 cited in Changnon et al., 1976) and from 1971 to 1974 in the METROMEX network of 250 stations increased in Edwardsville, the downwind area located 25-30 km away to the northeast of St. Louis, and decreased in the area to the west and the southwest of St. Louis. In comparison of water outputs of raincells from 1971 to 1973, the highest output was from the raincell initiated or exposed by the urban of St. Louis and the Alton-Wood River followed by those from hills raincell, bottomlands raincell, and rural raincell (Schickedaz, 1973 cited in Changnon et al., 1976). The FPS-18 radar data was taken to study 17 storms between 1972 and 1973, and the result showed that strong echoes developped more frequently over urban and hill areas (Huff and Schlessman, 1974 cited in Changnon et al., 1976). The distribution of afternoon rains studied by 3 cm range-height-indicator (RHI) TPS 10 also showed the initiations of raincells over the urban area of St. Louis (Changnon and Semonin, 1975). It was proposed that when additional raincells were produced, there could be the merging which caused the heavier rainfall. It was concluded that urban area could be the point for both rain initiation and enhancement.

Buishand (1979) investigated urbanization effects on rainfall in the western part of the Netherlands. The cumulative sum technique was used among the sequences of differences between monthly rainfall in rural and urban areas from 1923 to 1970. The rainfall data in the Amsterdam area was compared with that in the north of the North Holland Province. The rainfall data in the Rotterdam area and Zeeuws-Vlaanderen were compared with that in the islands and peninsulas in the south of the South Holland Province. At the significance level of 0.1, there were increasing trends in differences between monthly rainfall in the Amsterdam area and the north of the North Holland Province and between that in the Rotterdam area and the islands and peninsulas in the south of the South Holland Province during summer and winter. It was concluded that the urbanization caused the summer and winter rainfall in the Amsterdam and Rotterdam regions increasing.

Alpert and Shafir (1990) applied the orographic model (Alpert and Shafir, 1989 cited in Alpert and Shafir, 1990) to simulate the rainfall in the area of the Judean Mountains, Israel. Five experiments consisted of the simulations of the annual rainfall from 1931 to 1960, the annual rainfall from 1951 to 1980, and 3 case studies of orographic rainfall events, 16-21 February 1983, 31 December 1982 - 2 January 1983, and 4-5 March 1983, were conducted in the model. The rainfall in the urban area of Jerusalem and most of the surrounding cities appear to be higher than model prediction unlike at the rural area. Correlation coefficients between observed and modeled data when all stations were calculated appeared to be poorer than when only rural stations were calculated. It was concluded that the urbanization enhanced the rainfall for about 20% during the rainfall events in Jerusalem and about 10% in the annual rainfall averages; moreover, the increasing in rainfall appeared to be higher at the downwind urban stations.

Subbiah, Vishwanath and Devi (1991) investigated trends of rainfall in 20 stations in Tamil Nadu at the south of India from 1901 to 1987. The area was divided into three zones, the rainy north coastal plain, the dry south coastal plain, and the dry interior. In the dry interior, there were decreasing trends in rainfall at all stations except at the industrial town of Coimbatore where there was a significant increasing trend at the significance level of 0.05

Çiçek and Turkoglu (2005) studied trends of precipitation in the period between May and September which was the warm period in two stations in Ankara, Turkey. One station was the Ankara Meteorology Station (AMS) which located in an area with the urban characteristic, and another one was the Esenboga Meteorology Station (EMS) which located in an area with the rural characteristic. The day with the precipitation amount of not less than 0.1 was considered rainy day, and the days with the precipitation amount of not less than 2.5 mm, 6.25 mm, and 12.5 mm were considered light, moderate, and heavy precipitation days, respectively. From 1956 to 2001, the ratio of the number of precipitation days at the AMS to that at the EMS was 0.9 as same as that for the number of light precipitation days. However, the ratios of the numbers of moderate precipitation days and heavy precipitation days at the AMS to that at the EMS were 1.1 and 1.2, respectively. Variation of precipitation in the week was also studied. At the AMS, Sundays was the day with highest precipitation in the period of 1920-1930, but in the period of 1996 – 2000, Wednesday was the day with the highest precipitation. At the EMS, there were no trends in both 2 periods. According to the Mann-Kendall rank correlation test with the significance level of 0.95, there was an increasing trend of the number of precipitation day from 1983 at the AMS and 1998 at the EMS. The number of light precipitation days at the AMS had increased since 1988 while there was no significant trend at the EMS. There had been non-significant increasing trends in the number of moderate and heavy precipitation days since 1980 at AMS, and at the EMS, there was no trend in the number of moderate precipitation days and also a decreasing trend in the number of heavy precipitation days after 1983. It was concluded that urbanization caused an increasing in the number of heavy precipitation days.

Fu, Guo and Wang (2006) used the Mesoscale Modeling System (MM5) to study effects of urbanization on a convective storm in Beijing. The convective precipitation on 4 June 2003 was simulated in two different land surface conditions in the area of 600×600 km^2. The first condition was the unmodified condition. The land surface was original and partly urbanized. The second condition was when roughness length was adjusted to 50 cm, albedo was adjusted to 18%, thermal inertia was adjusted to 0.03 cal. cm^{-2} K^{-1} s$^{-1/2}$ and availability moisture was adjusted to 10% in the 200×200 km^2 of Beijing area while the outside area was still non-modified. These adjusted values represented the characteristics of the typical urban surface. The result showed that the urbanization caused surface sensitive heat flux increasing and concentrating because of the higher storing of solar energy and caused the surface latent heat flux decreasing because of the decreasing of moisture availability. The lower moisture availability also caused the convective cells developing earlier and distributing dispersedly. The dryer air caused the evaporative cooling which enhanced the downdraft. The surface roughness also strengthened the lower convection and weakened the upper convection which enhanced the lower convergence. In conclusions, precipitation decreased in total, especially in the urban area, and was intense along the boundary between urban and non-urban area after the urbanization.

Chen et al. (2007) used the MM5 model to study the effects of the urban heat island on the thunderstorm in the island of Taiwan where there was a complicate landscape with the north-south mountain range. The habitable flat land was in less than one-third of the western plain area. The thunder storm on 22 June 1994 was simulated in four cases, the control case which used the land cover data from the United States Geological Survey (USGS), the cases when the urban sizes in the center of Taiwan were 15 × 15 km^2, and 30 × 30 km^2, and the case when the urban area covered latitude of from 23.5°N to 24.0°N where the elevation of the topography was less than 500 m. In the case that the urban size in the center of Taiwan was 15 × 15 km^2, the urbanization enhanced the precipitation over both mountainous and plain areas in the downwind of the city. In the case that the urban size was larger, the urbanization also enhanced the precipitation over the plain area in the upwind of the city. This enhancement was distinct from many cases of other cities on the large plain areas (Negri, Pierce and Shepherd, 2002 cited in Chen et al., 2007; Huffines, Orville and Steiger, 2002 cited in Chen et al., 2007). It was explained that urban area enhanced sea breeze and vertical velocity. Then the sea breeze circulation moved to the downwind mountainous area where the vertical motion was enhanced. When the level of free convection was reached, the precipitation was formed over the plain area.

Mohanty et al. (2008) used the MM5 model with the input of soil moisture and multi-level ground temperature calculated by the NOAH land surface model (Chen and Dudhia, 2001 cited in Mohanty et al., 2008) to investigate effects of urbanization on sea breeze induced convection and precipitation in the area of Chennai which was the large city located on the east coastline of India. The sea breeze circulation on 28 June 2003 was simulated in two cases, the control case, which represented the urban characteristics, and case when Chennai was replaced by irrigated croplands and pastures, which represented the non-urban cahracteristics. This modifying caused the roughness length decreasing from 0.80 to 0.15 cm, the albedo increasing from 0.15 to 0.18, and the surface emissivity increasing from 0.880 to 0.985. The result showed that the urbanization caused the surface temperature over Chennai increasing by 3.0 K in the early morning. This increasing in the surface temperature enhanced the onshore flow by 4.0 m per second and the precipitation increased by 25 mm over the large area 150 km west of Chennai.

Shem and Shepherd (2009) studied the urbanization effect on thunderstorm in summertime in Atlanta using the Weather Research and Forecast (WRF) model combined with the NOAH land surface model (Ek et al., 2003 cited in Shem and Shepherd, 2009). The convective storm with minimal large scale urban forcing on 17 August 2002 and the convective storm induced by the convergence zone from the urban heat island on 26 July 1996 were simulated in comparison between the urban condition which was based the 30" 1994 USGS land cover data set and the non-urban condition when Atlanta was replaced by the dominant landcover type in the surrounding rural area. In the urban condition, within the strip of 20-50 km east of the city, the rainfall in the downwind of the city was larger in amount by 10-13% than that in the non-urban condition. The different initiations of the convective system seemed not likely to affect the amount of rainfall as much as the urban characteristics did.

Ooka and Yamanaka (2009) used the MM5 model and the TERRAIN program modified in the previous work (Kawamoto and Ooka, 2008 cited in Ooka and Yamanaka, 2009) to study the storm event on 15 August 2005 in Tokyo, Japan. Three wind systems which converged to the urban area of Tokyo were observed during the event. They were the dry and cold northwesterly wind, warm southeasterly wind, and cold northeasterly wind. The model could reproduce these three winds by the inputs of land-use data from the ministry of land, transport, infrastructure, and tourism (MLIT) of Japan and anthropogenic heat data from the Advanced Industrial Science and Technology, Japan (AIST). The result showed that the northeasterly wind had most water vapor mixing ratio among those three winds. It was concluded that the northeasterly wind was the main factor that bring the moisture to the convective area. The model was also applied to estimate effects of urbanization on rainfall by three simulations. The first one was the control simulation which used the land-use data and anthropogenic heat data that were used to reproduce the three wind systems mentioned above. The second one was the simulation which used the same land-use data as the first one, but the anthropogenic heat data was not included. The third one was the simulation when the urban area was replaced by farmlands. The result of the second simulation showed an easterly shift of the heavy rainfall region in comparison to the control simulation. The result of the third simulation, showed less rainfall than others simulation. In conclusions, the urbanization in Tokyo seemed to induce the rainfall.

Kishtawal et al. (2010) studied the amount of rainfall and its relation to the urbanization over India using daily rainfall dataset and population data. According to the rainfall data in the previous work (Sinha Ray and Srivastava, 2000 cited in Kishtawal et al., 2010), frequency of the heavy rainfall, the rainfall with the amount of more than 70 mm, during monsoon season from 151 stations in the period of 1901-1990 showed increasing trends in 23 stations and decreasing trends in 48 stations at the significance level of 0.01. Out of 23 stations with the increasing trends, 12 of them were in the areas with fast urbanizations where the population within 0.25° radius from the stations location growth for more than 280 during the period of 1990-2000. Out of 48 stations with the decreasing trends, 41 of them were in the areas with slow urbanizations where the population within 0.25° radius from the stations location growth for less than 55 during the period of 1990-2000. Out of most 15 populated cities in India, there were increasing trends in rainfalls in Mumbai, New Delhi, Hyderabad, Ahmedabad, Pune, and Indore, while there was a decreasing trend only in Nagpur. Another analysis used 1°

gridded rainfall data in the period of 1950-2003 from the previous work (Bhate et al., 2005 cited in Kishtawal et al., 2010). The grid was considered urban if the population density was more than 3000 people / km^2 and was considered rural if the population density was less than 600 people / km^2. With these criteria, 11% of the grids were urban grids while 62% were rural grids. Number of heavy rainfall aggregated over urban grids showed an increasing trend for about 18% per decade while there was no significant trend of heavy rainfall aggregated over rural grids. The other analysis used the data from the Tropical Rain Measurement Mission (TRMM) combined rain rate product (3G68) in the monsoon season from 1998 to 2007. The ratio of the number of the heavy rain events with the near surface rain rate of more than 5.0 mm/h to the number of all rain events was computed at each 0.5° grid cells. Population density data was converted to 0.5° grid cells, and then a variation of population density grid cells with different ratio of the number of heavy rain events to the number of all rain events was determined. This ratio was found to increase with the increasing of population density in a non-liner trend. The normalized histogram of the rain rates for urban and rural grids also showed that urban area had less light rainfall, the rainfall with the rain rate of below 2 mm/h, but more heavy rainfall than rural area.

2.4 Flood modeling

Flood has been defined as “an overflow or inundation that comes from a river or other body of water and causes or threatens damage” and “any relatively high streamflow overtopping the natural or artificial banks in any reach of a stream” (United States Geological Survey [USGS], 2011). Computer models used to determine effects of flood need four parts which are the hydrodynamic model which is used to determine runoff from a storm, the hydraulic model which is used to route the runoff through channels, the floodplain mapping and visualization tools, and the extraction of spatial data used in the model (Snead, 2000). There have been many studies that use models to study about flood problems.

Kronborg, Mark and Tomicic (1999) studied the flood in the Playa de Gandia Resort, Valencia, Spain. Gandia was the city on the floodplain near the Mediterranean Sea with low elevation and high groundwater level, and there was a sand dune which obstructed gravitational drainage, so pumps were required. The problem of flood was most severe in the southeastern part of the area. At that time, pumps and pipe network were very small and the flood drainage system was not separated from the waste water drainage system. The MOUSE model was applied to find the solution to the flood problem in the area. The event in September, 1944 was used for the calibration. The designed storm with a return period of 8 years and duration of 1 hour was simulated in three cases. The first case was the situation that the drainage system was as same as it was at that time. In the second case, the detention storage with pump was added to the drainage system, a partial replacement of sewer networks was done, and the flood drainage system was separated from the waste water drainage system in the southeastern part of the area. The third case was the case extent from the second case with more storage pumps and pipes but there was no separation between flood and waste water drainage systems. In the first case, 6% of total hydraulic load during the 24 hours simulation period infiltrated into the ground, while 40% of the load went to the treatment plant and 54% of the load was discharged to the environment. There was an inundation with the maximum depth of 40 cm and the period of about 4 hours. In the second case, 4% of total hydraulic load infiltrated into the ground, while 45% of the load got evacuated and 51% of the load was discharged to the environment. The separation between flood and waste water drainage systems caused the inundation duration being longer for 1 hour but the maximum inundation was reduced by around 6 cm. In the third case, 2% of total hydraulic load infiltrated into the ground, while 66% of the load got evacuated and 32% of the load was discharged to the environment. The inundation duration was reduced by more than 1 hour and 30 minutes and the maximum inundation was reduced by around 10 cm from the first case. In conclusions, more storage basins and pumps were proposed as the solution to the flood problem.

Tucci and Villanueva (1999) used the hydrodynamic model (Tucci, 1978 cited in Tucci and Villanueva, 1999) to study the flood problem in União da Victoria and Proto União, Brazil. There had not been big floods in that area during the period of 1930-1983, but after the building of the Foz do Areia Dam in the early of 199th decade, there were severe floods in 1983 and 1992. The storage function was used to represent the floodplain, and it was assumed that the floodplain had an infinite surface roughness. In the river, the Lateral Distribution Method was used. Those two events in 1983 and 1992 were used for the model calibration. The result of the study was that the dam operations had only little effect on the flood, but the main cause of the flood was the narrowing of the water way from the dam to the city.

Ahmed and Shah-Newaz (2001) studied about flood problem in the Young Brahmaputra Floodplain, Bangladesh. This area was the flat area bounded by a horseshoe embankment, and had the problem of riverine flood. The study used the MIKE 11 model calibrated by water level and discharge in the period of 1995-1996 hydrological years. In the model, there was a river re-excavation and different operation rules of the main inlet were applied to find the way to solve the flood problems. It was found that the re-excavation could enhance the flow during high flow situation, and the excess water from the river and floodplain could be drained quickly. The model was also applied to determine the effect of compartmentalization, the mitigation measures involving the constructions of embankments to divide a floodplain into parts to slowdown the runoff and store the water when there was a possibility that the flood peak exceeded the danger level. It was found that compartmentalization caused water level in most rivers to increase and inundation maps showed a significant improvement of the flood situation.

Apirumanekul and Mark (2001) studied the flood problem in Dhaka, Bangledesh. This city was surrounded by flood barriers but have the problem of flood when there was a heavy rainfall combined with a high water level in the river. The drainage system in the area involved draining the water to canals by pipes and pumping the water out to the river. The MOUSE model was used to determine if that drainage system could solve the flood problem in 1996. The result was that the flood problem could not be solved even there was a real time control system in the model. However, when the drainage capacity was enhanced by increasing the diameter of pipes and realignment of pipes, the flood problem was relieved.

Marfai (2003) developed the flood model for river flood and tidal flood in Semarang, Indonesia. The model for river flood was developed using the HEC-RAS model combined with the HEC-GeoRAS model and the model for tidal flood was developed using the neighborhood function. The evaluation of the model was done by comparing the simulated floods with reliable source maps using the confusion matrix concept. The flood simulated by the river flood model was compared with the flood map from the river flood with a return of 100 years from the JICA in 2003. The flood simulated by the tidal flood model was compared with the map from the Semarang Public Works Office developed in 2001. The accuracy of the river flood model was 77%, and the reliability was 76%. The accuracy of the tidal flood model was 89%, and the reliability was 83%. Inundated areas from the flood in November 2000 were evaluated by this model. The result was that the river flood inundated 510.57 hectares of built up area, 164.12 hectares of fishponds, 1.34 hectares of agricultural area, and 711.68 hectares of others, and the tidal flood inundated 814.56 hectares of built up area, 2.26 hectares of fishponds, 231.94 hectares of agricultural area, and 461.86 hectares of others.

Rungsipanodorn (2005) used the Hydrowork model to study the local drainage system in the Buengkum Area, Bangkok. The area was separated into 5 polder systems. The flood in 2004 was used for the calibration. Rainfall with return periods of 2 and 5 years were simulated in 2 conditions. The first condition used the landuse data in 1999 while the second condition used the predicted landuse data. The result suggested that most of the canals had no problem of overflow but the pipe system was too small to alleviate the flood with a return period of 5 years and also 2 years for the 10 villages in the area. Given the landuse data in 1999, if the smallest diameter of pipes was changed from 0.3 m to 0.6 m, the flood duration would be reduced from 5 hours to 1 hour and the depth would be reduced from 20 cm to 15 cm when 5-year rainfall occurred.

Thien (2005) used the MIKE 11 model to study the flood problem in the Yom Basin, Thailand. The area had the problem of flood in the wet season and the problem of drought in the dry season. The objective of the study was to find the way to relieve the flood problem. Data during the period of 1994-1995 was used for the calibration and verification. Dieu Tiet Lu model, which was widely used in Vietnam and approved by the Ministry of Agriculture and Rural Development of Vietnam, was also used to simulate the dam operation. Two storage basins with the total storage of 346.64 million m^3 were built in the model, and it was founded that an inundation depth in Changwat Phrae was reduced by 0.3 m. Moreover, the inundation area was reduced by 10 km^2 during the peak discharge in 1995. However, for the rainfall with a return period of 100 years in 1995, those two storage basins could reduce the inundated area for only 10%. In this case, the Kaeng Suea Ten Dam which had the useful storage of 665.5 million m^3 was built in the model, and it was found that the inundated area from that rainfall was reduced by 48%. It was concluded that the two small reservoirs could alleviate normal floods, but in the case of long return period flood, a large reservoir was required.

Wongwiwat (2005) used the distributed flood model to analyze the characteristics of flood from extreme rainfalls and develop a vulnerability database in the Bangkapi and Buengkum districts, Bangkok. The flow in the river network was simulated by the 1-D St. Venant's equation with the diffusive approximation and the overland flow was simulated by the 2-D St. Venant's equation with the diffusive approximation. Loss functions were determined from the linear regression between observed flood characteristics and losses. It was found that the most three factors which affect the loss were flood depth, flood duration, and building area. The model could simulate the flow in canals well. But for the overland flow, the effect of building grids in the digital elevation model (DEM) caused some discretion. It was suggested that the DEM should have a high resolution and the effect of building grids should be reduced.

Visutimeteegorn (2006) studied the flood problem in the upstream and downstream area of the Chao Phraya Dam which was built to raise the water level and distribute the water to irrigation canals. The study aimed to find the operation rule of the dam which could alleviate flood problems in the area using the HEC-RAS model and data of events in 1995, 1996, 2002, and 2006. Different operation rules were simulated in the model. The result showed that flood problems could be alleviated by diverting the water to the canals until the full capacity was reached ten days prior to the coming of the flood peak, diverting the water to the side canals until the full capacity was reached for one side and half capacity was reached for another side afterwards, and swapping the sides for the diversion every five days.

Indra et al. (2007) studied the flood events in February 2007 in Jakarta, Indonesia which caused most activities to stop for four days. The study aimed to develop the flood control system by building a flood model which included rainfall-rate, infiltration, delay, and control system modeling. In the simulation, the area was separated into three parts from upstream to downstream which were Bogor, Depok, and Jakarta, respectively. The results showed that the problem of flood was caused by the low infiltration rate and rain aggregated in Bogor, Depok, and Jakarta. It was suggested that more infiltration rate was required and the control system was also important to alleviate the flood problem.

Pawattana, Tripathi and Weesakul (2007) studied the way to solve problems of flood and drought in the Chi River Basin. Data of salt crust, soil drainage, slope, and geological formation were used to find the proper location to build reservoirs for diverting the water to. It was found that two reservoirs with the total storage capacity of 15.84 million m^3 could be built. Then the MIKE 11 model calibrated with the data in 2000 was applied to determine the flood when these reservoirs were included. The result showed that flood depth from events in the period of 14 August - 3 October 2001 and 28 October – 16 November 2001 at the diverted location could be reduced by 11 cm.

Sakol (2010) used the MIKE 11 model to study the flood management in 9 polder areas in the Lower East Chao Phraya Basin, Thailand. The simulation was run in 4 cases. The first case was the simulation of the situation in 2006, the second case was the simulation when the polder area where flood would cause less damage was chosen to store the excess water before the one where flood would cause more damage, the third case was the simulation when the upper polder area was used to store the excess water before the lower one and the last case was similar to the third one but 4 pumping stations were applied. The result suggested that applying pumping stations could lower the water level in both upper and lower parts of the study area while choosing the polder area to store the excess water upon the damage which the flood would cause could not be used to manage the excess water but could be used to decide which area should store the excess water for each flood depth.

2.5 MIKE11 model

The MIKE11 Model is the production of the Danish Hydraulic Institute (DHI). It is a fully dynamic one-dimensional model for simulation of river and channel systems. The hydrodynamic (HD) module is considered the nucleus of the model (DHI, 2009b). It is used for the computation of flows by solving the equations for the conservation of continuity and momentum, the Saint Venant equations (see Eqs. 3.33 – 3.34), which is based on following assumptions (DHI, 2009a):

1. Water cannot be compressed and is homogeneous.
2. The bottom slopes of rivers and channels are small enough to assume that the cosine of the angle between the bottom and horizontal is approximately 1.
3. In comparison to the water depth, the wave lengths are large enough to assume that the flow is parallel to the bottom without vertical accelerations.
4. The flow is subcritical.

Flow over hydraulic structures can also be computed by the advanced computational module and lateral inflow from the rainfall to rivers and channels can be calculated by the Rainfall-Runoff (RR) module (DHI, 2009a). The detail of the calculation will be discussed in the section 3.3.

CHAPTER III

METHODOLOGY

3.1 Data collection

This study involved rainfall trend analysis and the MIKE 11 model application. For the rainfall trend analysis, daily rainfall data was required. For the MIKE 11 model application, data of channel network, cross-sections, floodgate and pump operation, catchments, and water level and rainfall time series were required. Some of the data could be obtained for the external organizations while others were obtained from the field observations.

Table 3-1 Data obtained from the external organizations

Data	Source	Resolution
Rainfall	Thai Meteorological Department	1 day
Rainfall	Department of Drainage and Sewerage (SCADA system)	15 min
Rainfall	Department of Drainage and Sewerage (Districts Offices)	5 min
Digital elevation model[a]	Land Development Department	5 m
Benchmark[b]	Public Works Department	-
Canal network	Department of Drainage and Sewerage	-
Cross-sections[c]	Royal Irrigation Department	-
Canal excavation depth	Department of Drainage and Sewerage	-
Water level	Royal Irrigation Department	15 min
Water level	Department of Drainage and Sewerage	15 min
Pump operation	Department of Drainage and Sewerage	15-60 min[d]
Floodgate operation	Department of Drainage and Sewerage	15-60 min[d]

[a] Digital elevation model from the Land Development Department was in the scale of 1:400 and derived in 2008.

[b] Benchmark data from the Public Works Department was as of 2007

[c] Cross-sectional data from the Royal Irrigation Department was surveyed in 2004.

[d] Pumps and floodgates operation data was noted manually and the intervals of the data could be varies.

3.1.1 Data from external organizations

The data obtained from the external organizations were rainfall, elevation, channel network, cross-sections of the Chao Phraya River, excavation depths of drainage canals, water level, pump operation and floodgate operation. These data were obtained from the Thai Meteorological Department (TMD), the Public Works Department, the Department of Drainage and Sewerage (DDS), the Land Development Department (LDD) and the Royal Irrigation Department (RID) as shown in Table 3-1.

3.1.2 Data from the field observations

Since the cross-sectional data of others channels apart from the Chao Phraya River were not available from the RID, they were derived from the widths, bed levels and bank elevations of channels. The field observations aimed to get the data of widths and bed levels of the channels. They were conducted in the period of April – August 2012. There was also another objective of the field observations which was identifying the characteristics of the catchments. Followings were data obtained during the field observations:

1. Channel widths which were measured using the laser distance meter DISTO E5.
2. Water depths data which were measured by a sonar radiometer along Bangkok Noi Canal. This data combined with the observed water level data could be used to calculate for the bed level of the canal.
3. Characteristics of the area.

3.2 Rainfall change analysis

Rainfall characteristics were studied using the daily rainfall data from the TMD. Before analysis, the missing data should be filled and the consistency analysis should be done to check and adjust the data for reliability. In the analysis, the three techniques were carried out, the moving average, the Mann-Kendall test, and the frequency analysis comparison.

3.2.1 Filling the missing data

There were someday that the daily rainfall data at some stations were missing. The method used to fill the missing data was the inverse distance method (Singh, 1992). This method used the data from nearby stations to fill the missing data described by Eqs. 3.1 – 3.2;

$$x = \frac{\sum_{i=1}^{n} w_i x_i}{\sum_{i=1}^{n} w_i} \tag{3.1}$$

$$w_i = \frac{1}{d_i^2} \tag{3.2}$$

where n was the number of nearby stations used for filling the data, x_i was the data at the i^{th} station i, and d_i was the distance from the i^{th} station to the station with the missing data.

3.2.2 Consistency technique

There could be many factors that affect the consistency of rainfall data, so the data should be checked for its consistency. The technique used to check for the consistency was the double-mass analysis (Singh, 1992). For each station, the accumulated annual rainfall was compared with the accumulated average annual rainfall from surrounding stations.

The accumulated annual rainfall data from the tested station which was represented by x was plotted against the accumulated average annual rainfall from surrounding stations which was represented by y. The inconsistency was represented by a change in the slope of the plot data series. The inconsistent data could be made consistent using Eq. 3.3;

$$x' = \frac{m'}{m}x \tag{3.3}$$

where x' was the corrected data, x was the non-corrected data, m' was the slope of the non-corrected data, and m was the slope of the correct data.

3.2.3 Annual rainfall and maximum 1-day rainfall

The annual rainfall for each year could be determined by sum of daily rainfall within the year as described by Eq. 3.4;

$$P = \sum_{i=1}^{n} P_i \tag{3.4}$$

where P represented annual rainfall, n represented number of days in the year, and P_i represented amount of rainfall in i^{th} day of the year.

The maximum 1-day rainfall for each year could be determined by the maximum of daily rainfall within the year as described by Eq. 3.5;

$$P_{x1d} = \max(P_1, P_2, \ldots, P_n) \tag{3.5}$$

where P_{x1d} represented maximum 1-day rainfall, n represented number of days in the year, and P_i represented amount of rainfall in i^{th} day of the year.

3.2.4 Moving average technique

The moving average (Subramanya, 2009) was the statistical technique used to find the magnitude of the increasing or decreasing of the rainfall. In general, the best fit line for the time series could explain the trend of rainfall through its slope.

The best fit of the linear equation was described by Eq. 3.6;

$$y = mx + k \tag{3.6}$$

where x and y were variables represent time and rainfall data, respectively, m was the slope, and k was the value of y when $x = 0$ determined by Eqs. 3.7 – 3.8;

$$m = \frac{\sum_{i=1}^{n}(x_i - \bar{x})(y_i - \bar{y})}{\sum_{i=1}^{n}(x_i - \bar{x})^2} \tag{3.7}$$

$$k = \bar{y} - m\bar{x} \tag{3.8}$$

where n was the number of data in the series, x_i and y_i represented i^{th} data of x and y, respectively, and $\bar{x}$ and $\bar{y}$ represented averages of x and y, respectively.

The annual data in the time series were usually varies and showed the weak linear trend, so the slope of the annual data in the time series might be too rough to represent the trend. In order to have the stronger linear trend, a new time series consists of average data in each consecutive specified duration was generated as described by Eq. 3.9;

$$x_i' = \frac{1}{m}\sum_{k=0}^{m-1} x_{i+k} \text{ for } i \le n - m + 1 \qquad (3.9)$$

where x_i' was the i^{th} data of x in the new time series, m was the length of consecutive years and n was the length of the former series.

The new time series would have the stronger linear trend, so the slope of the best fit line of the new time series could represent the trend of the data better than that of the old one.

3.2.5 Mann-Kendall test

The Mann-Kendall (MK) Test (Mann, 1945; Kendall, 1975) was the technique used to find the trend of rainfalls in the time series whether it increased, decreased or neither. The principle of the MK Test was the comparison among every pair of data in the time series. If the data increased in most of the pairs, the rainfall increased. Conversely, if the data decreased in most of the pairs, the rainfall decreased. However, the increasing or decreasing would not be significant if the numbers of increasing pairs and decreasing pairs were not very different.

The null hypothesis for the test was that the amount of rainfall did not change. In order to test the hypothesis, let x_i and x_j be the data for i^{th} and j^{th} year, respectively. The test statistic S was determined from Eq. 3.10;

$$S = \sum_{i=1}^{n-1}\sum_{j=i+1}^{n} \sigma(x_j - x_i) \qquad (3.10)$$

where n was the number of records and σ was the sign function defined by Eq. 3.11;

$$\sigma(x) = \begin{cases} +1 \ \ if\ x > 0 \\ 0 \quad if\ x = 0 \\ -1\ if\ x < 0 \end{cases} \tag{3.11}$$

If S was positive, the number of the increasing pairs was more than that of the decreasing pairs. Conversely, if S was negative, the number of decreasing pairs was more than that of the increasing pairs. The value of $|S|$ showed the difference between the increasing pairs and decreasing pairs.

The test statistics S was assumed to be normally distributed with the mean of 0 and variance ($Var(S)$) determined by Eq. 3.12;

$$Var(S) = \frac{n(n-1)(2n+5) - \sum_{i=1}^{m} t_i(t_i-1)(2t_i+5)}{18} \tag{3.12}$$

If there were repetitions of the data, the repeated data must have been tied into groups. t_i in Eq. 3.12 was the number of data in of i^{th} tied group.

The test statistics S was then normalized to Z by Eq. 3.13;

$$Z = \frac{S - \sigma(S)}{\sqrt{Var(S)}} \tag{3.13}$$

The normal distribution with the mean of 0 and variance of 1 which was called standard normal distribution was applied to test the null hypothesis. At a significance level of α, if the area under the standard normal distribution density function from $-\infty$ to Z was less than $1-0.5\alpha$, the null hypothesis would be accepted. In other words, there was no significant change in amount of rainfall. Conversely, if the area under the standard normal distribution density function from $-\infty$ to Z was more than $1-0.5\alpha$, the null hypothesis would be rejected. In other words, there was a significant change in amount rainfall. In this case, the positive and negative values of Z represented the increasing and decreasing, respectively, in amount of rainfall. The area under the standard normal distribution density function from -∞ to Z was shown in table 3-2.

Table 3-2 Area under the standard normal distribution density function from -∞ to Z

z	0.00	0.01	0.02	0.03	0.04	0.05	0.06	0.07	0.08	0.09
-2.6	0.0047	0.0048	0.0049	0.0051	0.0052	0.0054	0.0055	0.0057	0.0059	0.0060
-2.5	0.0062	0.0064	0.0066	0.0068	0.0069	0.0071	0.0073	0.0075	0.0078	0.0080
-2.4	0.0082	0.0084	0.0087	0.0089	0.0091	0.0094	0.0096	0.0099	0.0102	0.0104
-2.3	0.0107	0.0110	0.0113	0.0116	0.0119	0.0122	0.0125	0.0129	0.0132	0.0136
-2.2	0.0139	0.0143	0.0146	0.0150	0.0154	0.0158	0.0162	0.0166	0.0170	0.0174
-2.1	0.0179	0.0183	0.0188	0.0192	0.0197	0.0202	0.0207	0.0212	0.0217	0.0222
-2.0	0.0228	0.0233	0.0239	0.0244	0.0250	0.0256	0.0262	0.0268	0.0274	0.0281
-1.9	0.0287	0.0294	0.0301	0.0307	0.0314	0.0322	0.0329	0.0336	0.0344	0.0351
-1.8	0.0359	0.0367	0.0375	0.0384	0.0392	0.0401	0.0409	0.0418	0.0427	0.0436
-1.7	0.0446	0.0455	0.0465	0.0475	0.0485	0.0495	0.0505	0.0516	0.0526	0.0537
-1.6	0.0548	0.0559	0.0571	0.0582	0.0594	0.0606	0.0618	0.0630	0.0643	0.0655
-1.5	0.0668	0.0681	0.0694	0.0708	0.0721	0.0735	0.0749	0.0764	0.0778	0.0793
-1.4	0.0808	0.0823	0.0838	0.0853	0.0869	0.0885	0.0901	0.0918	0.0934	0.0951
-1.3	0.0968	0.0985	0.1003	0.1020	0.1038	0.1056	0.1075	0.1093	0.1112	0.1131
-1.2	0.1151	0.1170	0.1190	0.1210	0.1230	0.1251	0.1271	0.1292	0.1314	0.1335
-1.1	0.1357	0.1379	0.1401	0.1423	0.1446	0.1469	0.1492	0.1515	0.1539	0.1562
-1.0	0.1587	0.1611	0.1635	0.1660	0.1685	0.1711	0.1736	0.1762	0.1788	0.1814
-0.9	0.1841	0.1867	0.1894	0.1922	0.1949	0.1977	0.2005	0.2033	0.2061	0.2090
-0.8	0.2119	0.2148	0.2177	0.2206	0.2236	0.2266	0.2296	0.2327	0.2358	0.2389
-0.7	0.2420	0.2451	0.2483	0.2514	0.2546	0.2578	0.2611	0.2643	0.2676	0.2709
-0.6	0.2743	0.2776	0.2810	0.2843	0.2877	0.2912	0.2946	0.2981	0.3015	0.3050
-0.5	0.3085	0.3121	0.3156	0.3192	0.3228	0.3264	0.3300	0.3336	0.3372	0.3409
-0.4	0.3446	0.3483	0.3520	0.3557	0.3594	0.3632	0.3669	0.3707	0.3745	0.3783
-0.3	0.3821	0.3859	0.3897	0.3936	0.3974	0.4013	0.4052	0.4090	0.4129	0.4168
-0.2	0.4207	0.4247	0.4286	0.4325	0.4364	0.4404	0.4443	0.4483	0.4522	0.4562
-0.1	0.4602	0.4641	0.4681	0.4721	0.4761	0.4801	0.4840	0.4880	0.4920	0.4960
-0.0	0.5000	0.4960	0.4920	0.4880	0.4840	0.4801	0.4761	0.4721	0.4681	0.4641
0.0	0.5000	0.5040	0.5080	0.5120	0.5160	0.5199	0.5239	0.5279	0.5319	0.5359
0.1	0.5398	0.5438	0.5478	0.5517	0.5557	0.5596	0.5636	0.5675	0.5714	0.5753
0.2	0.5793	0.5832	0.5871	0.5910	0.5948	0.5987	0.6026	0.6064	0.6103	0.6141
0.3	0.6179	0.6217	0.6255	0.6293	0.6331	0.6368	0.6406	0.6443	0.6480	0.6517
0.4	0.6554	0.6591	0.6628	0.6664	0.6700	0.6736	0.6772	0.6808	0.6844	0.6879
0.5	0.6915	0.6950	0.6985	0.7019	0.7054	0.7088	0.7123	0.7157	0.7190	0.7224
0.6	0.7257	0.7291	0.7324	0.7357	0.7389	0.7422	0.7454	0.7486	0.7517	0.7549
0.7	0.7580	0.7611	0.7642	0.7673	0.7704	0.7734	0.7764	0.7794	0.7823	0.7852
0.8	0.7881	0.7910	0.7939	0.7967	0.7995	0.8023	0.8051	0.8078	0.8106	0.8133
0.9	0.8159	0.8186	0.8212	0.8238	0.8264	0.8289	0.8315	0.8340	0.8365	0.8389
1.0	0.8413	0.8438	0.8461	0.8485	0.8508	0.8531	0.8554	0.8577	0.8599	0.8621
1.1	0.8643	0.8665	0.8686	0.8708	0.8729	0.8749	0.8770	0.8790	0.8810	0.8830
1.2	0.8849	0.8869	0.8888	0.8907	0.8925	0.8944	0.8962	0.8980	0.8997	0.9015
1.3	0.9032	0.9049	0.9066	0.9082	0.9099	0.9115	0.9131	0.9147	0.9162	0.9177
1.4	0.9192	0.9207	0.9222	0.9236	0.9251	0.9265	0.9279	0.9292	0.9306	0.9319
1.5	0.9332	0.9345	0.9357	0.9370	0.9382	0.9394	0.9406	0.9418	0.9429	0.9441
1.6	0.9452	0.9463	0.9474	0.9484	0.9495	0.9505	0.9515	0.9525	0.9535	0.9545
1.7	0.9554	0.9564	0.9573	0.9582	0.9591	0.9599	0.9608	0.9616	0.9625	0.9633
1.8	0.9641	0.9649	0.9656	0.9664	0.9671	0.9678	0.9686	0.9693	0.9699	0.9706
1.9	0.9713	0.9719	0.9726	0.9732	0.9738	0.9744	0.9750	0.9756	0.9761	0.9767
2.0	0.9772	0.9778	0.9783	0.9788	0.9793	0.9798	0.9803	0.9808	0.9812	0.9817
2.1	0.9821	0.9826	0.9830	0.9834	0.9838	0.9842	0.9846	0.9850	0.9854	0.9857
2.2	0.9861	0.9864	0.9868	0.9871	0.9875	0.9878	0.9881	0.9884	0.9887	0.9890
2.3	0.9893	0.9896	0.9898	0.9901	0.9904	0.9906	0.9909	0.9911	0.9913	0.9916
2.4	0.9918	0.9920	0.9922	0.9925	0.9927	0.9929	0.9931	0.9932	0.9934	0.9936

3.2.6 Frequency analysis comparison

The frequency analysis (Singh, 1992) was the technique used to determine the probability (P) that the rainfall exceeded some specific amount. That probability was explained in terms of return period in years (T) by Eq. 3.14;

$$T = \frac{1}{P} \tag{3.14}$$

A frequency curve was the curve that showed the relation between that probability and amount of the rainfall.

The principal of the frequency analysis comparison technique was separating the data series into two parts at a chosen breakpoint. Hence, the first part would be the series of the rainfall before the breakpoint and the second one would be that after the breakpoint. The frequency curves which were derived from each part could be used to determine the rainfall for each return period in that part. For each return period, if the amount of the rainfall in the second part was more than that in the first part, the amount of rainfall would increase. Conversely, if that in the second part was less than that in the first part, the amount of rainfall would decrease.

The frequency curve could be derived by finding the curve of the distribution to fit the observed data. The distributions that were commonly used in the frequency analysis were the followings,

1. Normal Distribution, the distribution where the probability (P) that the rainfall could exceed each specific amount (X) was defined by Eq. 3.15;

$$P(\mathrm{X}) = 1 - \frac{1}{S\sqrt{2\pi}} \int_{-\infty}^{\mathrm{X}} e^{-\frac{(x-\bar{x})^2}{2S^2}} dx \qquad (3.15)$$

where $\bar{x}$ and S were the average and standard deviation of the distribution of the amount of rainfall (x), respectively.

2. Log-normal Distribution, the distribution where the probability (P) that the rainfall could exceed each specific amount (X) was defined by Eqs. 3.16 – 3.17;

$$P(\mathrm{X}) = 1 - \frac{1}{S\sqrt{2\pi}} \int_{-\infty}^{\log \mathrm{X}} e^{-\frac{(y-\bar{y})^2}{2S^2}} dy \qquad (3.16)$$

$$y = \log x \qquad (3.17)$$

where $\bar{y}$ and S were the average and standard deviation of the distribution of the logarithm of the amount of rainfall (x) to base 10 (y), respectively.

3. Gumbel Distribution, the distribution where the probability (P) that the rainfall could exceed each specific amount (X) was defined by Eqs. 3.18 – 3.20;

$$P(\mathrm{X}) = 1 - e^{-e^{-\frac{\mathrm{X}-\alpha}{}}} \qquad (3.18)$$

$$= \frac{S\sqrt{6}}{\pi} \qquad (3.19)$$

$$\alpha = \bar{x} - \lim_{n \quad \infty} \left(\sum_{k=1}^{n} \frac{1}{k} - \ln n \right) \quad 0.5772 \qquad (3.20)$$

where $\bar{x}$ and S were the average and standard deviation of the distribution of the amount of rainfall (x), respectively.

4. Gamma Distribution, the distribution where the probability (P) that the rainfall could exceed each specific amount (X) was defined by Eqs. 3.21 – 3.23;

$$P(\mathrm{X}) = 1 - \frac{1}{a\ \ (b)} \int_{-\infty}^{\mathrm{X}} \left(\frac{x}{a}\right)^{b-1} e^{-\frac{x}{a}}\, dx \tag{3.21}$$

$$a = \frac{S^2}{\bar{x}} \tag{3.22}$$

$$b = \frac{\bar{x}^2}{S^2} \tag{3.23}$$

where $\bar{x}$ and S were the average and standard deviation of the distribution of the amount of rainfall (x), respectively.

5. Pearson Type III Distribution, the distribution where the probability (P) that the rainfall could exceed each specific amount (X) was defined by Eqs. 3.24 – 3.27;

$$P(\mathrm{X}) = 1 - \frac{1}{a\ \ (b)} \int_{-\infty}^{\mathrm{X}} \left(\frac{x-c}{a}\right)^{b-1} e^{-\frac{x-c}{a}}\, dx \tag{3.24}$$

$$b = \left(\frac{2}{C_s}\right)^2 \tag{3.25}$$

$$a = \frac{S}{\sqrt{b}} \tag{3.26}$$

$$c = \bar{x} - ba \tag{3.27}$$

where $\bar{x}$, S, and C_s were the average, standard deviation, and skewness of the distribution of the amount of rainfall (x), respectively.

The skewness C_s was sometimes suggested to be adjusted to account for the number of observed data using Eq. 3.28 (Hazen, 1930);

$$\widehat{C_s} = C_s\left(1 + \frac{8.5}{n}\right) \tag{3.28}$$

where $\widehat{C_s}$ was the adjusted skewness, and n was the number of observed data.

6. Log-Pearson Type III Distribution, the distribution where the probability (P) that the rainfall could exceed each specific amount ($\mathcal{X}$) was defined by Eq. 3.29;

$$P(\mathrm{X}) = 1 - \frac{1}{a\ \ (b)}\int_{-\infty}^{\log \mathrm{X}} \left(\frac{y-c}{a}\right)^{b-1} e^{-\frac{y-c}{a}}\, dy \tag{3.29}$$

where b and a could be calculated from Eqs. 3.25 and 3.26, respectively, c could be calculated from Eq. 3.30, and $\bar{y}$, S, and C_s were the average, standard deviation, and skewness of the distribution of logarithm of the amount of rainfall (x) to base 10 (y), respectively.

$$c = \bar{y} - ba \tag{3.30}$$

The skewness C_s was sometimes suggested to be adjusted to account for the number of observed data using Eq. 3.28.

The frequency curve was used to calculate for the theoretical occurrence probability of an equal or higher magnitude rainfall. However an observed one could be calculated from Eq. 3.31;

$$P = \frac{m}{N+1} \tag{3.31}$$

where P was the observed occurrence probability of an equal or higher magnitude the rainfall, m was the order of rainfall amount in the series sorted form the highest to the lowest, and N was the total number of data in the series.

One of the well-known tests for the goodness of fit for the frequency curve was the Kolmogorov-Smirnov (KS) test (Haan,1977). The test value D was defined by Eq. 3.32;

$$D = \max(|F(x) - S_n(x)|) \tag{3.32}$$

where $F(x)$ was the theoretical occurrence probability of the rainfall with the amount of x or lower, and $S_n(x)$ was the observed occurrence probability of the rainfall with the amount of x or lower.

Table 3-3 Criteria value for the Kolmogorov-Smirnov test (Birnbaum, 1952)

n*	α				
	0.2	0.15	0.1	0.05	0.01
1	0.900	0.925	0.950	0.975	0.995
2	0.684	0.726	0.776	0.842	0.929
3	0.565	0.597	0.642	0.708	0.829
4	0.494	0.525	0.564	0.624	0.734
5	0.446	0.474	0.510	0.563	0.669
6	0.410	0.136	0.470	0.521	0.618
7	0.381	0.405	0.438	0.486	0.577
8	0.358	0.381	0.411	0.457	0.543
9	0.339	0.360	0.388	0.432	0.514
10	0.322	0.342	0.368	0.409	0.486
11	0.307	0.326	0.352	0.391	0.468
12	0.295	0.313	0.338	0.375	0.450
13	0.284	0.302	0.325	0.361	0.433
14	0.274	0.292	0.314	0.349	0.418
15	0.266	0.283	0.304	0.338	0.404
16	0.258	0.274	0.295	0.328	0.391
17	0.250	0.266	0.286	0.318	0.380
18	0.244	0.259	0.278	0.309	0.370
19	0.237	0.252	0.272	0.301	0.361
20	0.231	0.246	0.264	0.294	0.352

* n represents the number of data.

If D was less than or equal to the criteria value which was shown in table 3-3, the frequency curve was acceptable. On the other hand, if D was larger than the critical value, it was unacceptable.

3.3 Computation by the MIKE11 model

3.3.1 Computation of channel flow

The Hydrodynamic (HD) module was used to simulate the flow in channels by applying an implicit finite difference method to the Saint Venant equations which consisted of the equations for the conservation of mass (Eq. 3.33) and the conservation of momentum (Eq. 3.34). The flow at the point x and the time step t was described by these equations (DHI, 2009a).

$$\frac{\partial A}{\partial t}+\frac{\partial Q}{\partial x}=q \tag{3.33}$$

$$\frac{\partial Q}{\partial t}+\frac{\partial\left(\alpha\frac{Q^2}{A}\right)}{\partial x}+gA\frac{\partial h}{\partial x}+\frac{gQ|Q|}{C^2AR}=0 \tag{3.34}$$

where Q was discharge, A was flow area, q was lateral inflow, h was stage above datum, C was Chezy resistant coefficient which could be determined from the manning's n (n) by Eq. 3.35, R was hydraulic radius, g was the acceleration due to gravity and α was momentum distribution coefficient the default value of which was 1.

$$C=\frac{R^{1/6}}{n} \tag{3.35}$$

when the flow was supercritical, the conservation of momentum would be reduced to Eq. 3.36;

$$\frac{\partial Q}{\partial t}+\frac{\partial\left(\alpha\frac{Q^2}{A}\right)}{\partial x}+gA\frac{\partial h}{\partial x}=0 \tag{3.36}$$

Water levels and discharges were calculated alternatively along the channel as shown in Figure 3-1 (DHI, 2009a) using the implicit finite difference scheme (Abbott and Ionescu, 1967 cited in DHI, 2009a). When points along the channel were specified by users and time series of either water level or discharge was applied at each boundary, the model would generated more point in the middle way between every 2 adjacent users specified points to compute for time series of another parameter (DHI, 2009a).

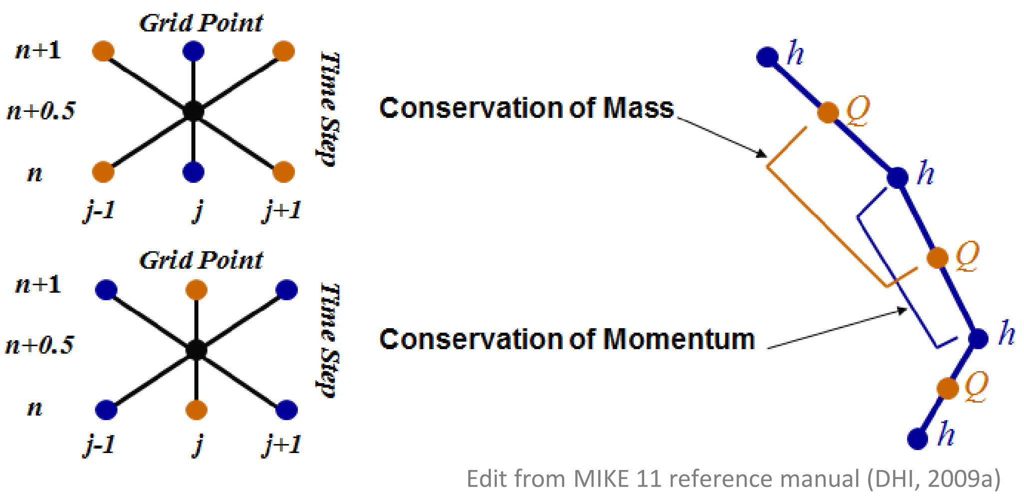


Figure 3-1 Solution scheme and computational grid used in the MIKE 11 model

In the computation for the water level, Eq. 3.33 was applied. The terms in the equation were calculated from Eqs. 3.37 – 3.40 (DHI, 2009a);

$$\frac{\partial A}{\partial t}=b_s\frac{\partial h}{\partial t} \tag{3.37}$$

$$b_s=\frac{A_{0,j}+A_{0,j+1}}{\Delta 2x_j} \tag{3.38}$$

$$\frac{\partial h}{\partial t} = \frac{h_j^{n+1} - h_j^n}{\Delta 2x_j} \tag{3.39}$$

$$\frac{\partial Q}{\partial x} = \frac{\frac{\left(Q_{j+1}^{n+1} + Q_{j+1}^n\right)}{2} - \frac{\left(Q_{j-1}^{n+1} + Q_{j-1}^n\right)}{2}}{\Delta 2x_j} \tag{3.40}$$

where b_s was the storage width, $A_{0,j}$ was the surface area between the point $j-1$ and the point j, $A_{0,j+1}$ was the surface area between the point j and the point $j+1$, and $\Delta 2x_j$ was the distance between the point $j-1$ and the point $j+1$, superscripts n and $n+1$ over the variables represented values of the variables at the time level n and $n+1$, respectively, and subscripts $j-1$, j and $j+1$ under the variables represented values of the variables at the points $j-1$, j and $j+1$, respectively.

In the computation for the discharge, Eq. 3.34 was applied. The terms in the equation were calculated from Eqs. 3.41 – 3.43 (DHI, 2009a);

$$\frac{\partial Q}{\partial t} = \frac{Q_j^{n+1} - Q_j^n}{\Delta t} \tag{3.41}$$

$$\frac{\partial\left(\alpha \frac{Q^2}{A}\right)}{\partial x} = \frac{\left[\alpha \frac{Q^2}{A}\right]_{J+1}^{n+1/2} - \left[\alpha \frac{Q^2}{A}\right]_{J-1}^{n+1/2}}{\Delta 2x_j} \tag{3.42}$$

$$\frac{\partial h}{\partial x} = \frac{\frac{\left(h_{j+1}^{n+1} + h_{j+1}^n\right)}{2} - \frac{\left(h_{j-1}^{n+1} + h_{j-1}^n\right)}{2}}{\Delta 2x_j} \tag{3.43}$$

where Δt was the length of a time step, $\Delta 2x_j$ was the distance between the point $j-1$ and the point $j+1$, superscripts n, $n+1/2$ and $n+1$ over the variables represented values of the variables at the time level n, $n+1/2$ and $n+1$, respectively, and subscripts $j-1$, j and $j+1$ under the variables represented values of the variables at the points $j-1$, j and $j+1$, respectively.

3.3.2 Computation of structure operation

Structures in this study involved pumps and sluice gates. When the structures were placed in the channel, the discharge in that point would be calculated by the equation upon the structure instead of Eq. 3.34 (DHI, 2009a). For the pumps, this study fixed the discharge, so the discharge at the point where this structure located would be that fixed discharge value.

For the sluice floodgate, the discharge (Q) was controlled by Eq. 3.44 (DHI, 2009a);

$$Q = L\sqrt{gy_c^3} \tag{3.44}$$

where L was the spillway width, g was the acceleration due to gravity, and y_c was critical depth determined upon the flow condition.

Let a be the flow area through the gate, b be the width of the gate, H_g be the height of the gate, G_0 be the gate opening height, and H and h be the upstream and downstream water height, respectively, measured above the sill level. For a controlled submerged flow condition, y_c was calculated from Eq. 3.45;

$$y_c = aG_0\left(\frac{H-h}{G_0}\right)^b \tag{3.45}$$

For a controlled free flow condition, y_c was calculated from Eq. 3.46;

$$y_c = aG_0\left(\frac{H}{G_0}\right)^b \tag{3.46}$$

For an un-controlled submerged flow condition, y_c was calculated from Eq. 3.47;

$$y_c = aH\left(1 - \frac{h}{H}\right)^b \tag{3.47}$$

For an un-controlled free flow condition, y_c was calculated from Eq. 3.48;

$$y_c = aH \tag{3.48}$$

For an over-the-top flow condition, y_c was calculated from Eq. 3.49;

$$y_c = a(H - H_g - G_0) \tag{3.49}$$

3.3.3 Computation of rainfall-runoff

Lateral inflows from the rainfall to the channel could be determined by the Rainfall-Runoff (RR) module. There were 4 types of models available, the Nedbør-Afstrømings-Model (NAM), Unit Hydrograph Method (UHM), Soil Moisture Accounting Model (SMAP), and Urban which used either the time area method or the kinematic wave method upon the selection of the user (DHI, 2009a). This study used the model type of NAM.

The rainfall for each basin was calculated by the Theissen polygon method (Singh, 1992) described by Eq. 3.50;

$$P = \frac{\sum_{i=1}^{n} A_i P_i}{\sum_{i=1}^{n} A_i} \tag{3.50}$$

where P was the catchment rainfall, n was the number of Theissen polygons, A_i was the size of the intersection area between the catchment and i^{th} polygon, and P_i was the amount of rainfall in i^{th} polygon.

Components of the NAM model were surface storage, root zone storage, evapotranspiration, overland flow, interflow, interflow and overland flow routing, groundwater recharge, soil moisture content, and baseflow (DHI, 2009a).

Surface storage (U_{max}) was the maximum amount of water that could be stored on the surface while root zone storage (L_{max}) was the maximum amount of water could be stored in the rootzone. As far as the amount of water in the surface storage (U) had not reached the maximum surface storage, the incoming rainfall would be stored on the surface. The amount of water in the surface storage was gradually lost by an evapotranspiration which could be calculated from Eq. 3.51 (DHI, 2009a);

$$E_a = \left(E_p - U\right)\frac{L}{L_{max}} \tag{3.51}$$

where E_a was an actual rate of evapotranspiration, E_p was a potential evapotranspiration, and L was the soil moisture content.

However if the amount of water exceeded the maximum surface storage, some of the excess water (P_N) would become an overland flow as described by Eq. 3.52 or infiltrated as described by Eq. 3.53 and the remaining water would become soil moisture content as described by Eq. 3.54 (DHI, 2009a);

$$QOF = \begin{cases} CQOF\dfrac{L/L_{max} - TOF}{1 - TOF}P_N\ ;\ L/L_{max} > TOF \\ 0\qquad\qquad\qquad\qquad\quad ;\ L/L_{max} \leq TOF \end{cases} \tag{3.52}$$

$$G = \begin{cases} (P_N - QOF)\dfrac{L/L_{max} - TG}{1 - TG}\ ;\ L/L_{max} > TG \\ 0\qquad\qquad\qquad\qquad\quad ;\ L/L_{max} \leq TG \end{cases} \tag{3.53}$$

$$\Delta L = P_N - QOF - G \tag{3.54}$$

where QOF was the amount of overland flow, $CQOF$ was the overland flow runoff coefficient, TOF was the threshold value for overland flow, G was the amount of infiltrating moisture, TG was the rootzone threshold value for groundwater recharge, and ΔL was the amount of increasing soil moisture content.

There was also an interflow contribution (QIF) which could be calculated from Eq. 3.55 (DHI, 2009a);

$$QIF = \begin{cases} (CKIF)^{-1}\dfrac{L/L_{max} - TIF}{1 - TIF}U \; ; \; L/L_{max} > TIF \\ 0 \qquad ; \; L/L_{max} \leq TIF \end{cases} \quad (3.55)$$

where $CKIF$ was the time constant for routing interflow, and TIF was the threshold value for interflow.

In the flow routing, overland flow and interflow were routed by the same time constant (CK_{12}). The interflow was routed as a linear reservoir while the routing of overland flow was kinematic. The time constant for routing the overland flow was determined by Eq. 3.56 (DHI, 2009a);

$$CK = \begin{cases} CK_{12} \qquad ; \; OF < OF_{min} \\ CK_{12}\left(\dfrac{OF}{OF_{min}}\right)^{-0.4} \; ; \; OF \geq OF_{min} \end{cases} \quad (3.56)$$

where CK was the variable time constant for routing overland flow, OF was the overland flow, and OF_{min} was an upper limit overland flow for linear routing which was 0.4 mm/h.

Another flow that was routed was the baseflow from groundwater which was routed by another time constant ($CKBF$). It was routed as linear reservoir.

The routed flows from overland flow, interflow, and baseflow would be applied to Eq. 3.33 as the lateral flow to the channel.

3.4 Rainfall design

In this study, the dimensionless mass curve method (Guo and Hargadin, 2009) was applied to design the hyetograph. A dimensionless mass curve was the dimensionless curve showing the relation between cumulative rainfall depth and time. In this method, rainfall events the amounts of which reach a specific criteria value were selected and hyetographs of these rainfalls were then converted to dimensionless mass curves by converting the duration and cumulative rainfall depth in their unit to those in dimensionless units. The cumulative rainfall depths in the dimensionless unit were averaged for each time step to get the dimensionless mass curve of the designed rainfall. This dimensionless mass curve was then converted to the hyetograph of the design rainfall by multiplying the dimensionless rainfall depth with an appropriate depth, multiplying the dimensionless duration with an appropriate duration, and converting the cumulative rainfall depth to the non-cumulative one.

3.5 Model sensitivity analysis

The sensitivity analysis was used to determine which parameters the result was sensitive to. The sensitivity index (I) was calculated from Eq. 3.57 (Eckhardt et al., 2002);

$$I = \frac{y_2 - y_1}{y_0} \div \frac{x_2 - x_1}{x_0} \qquad (3.57)$$

where x_0 was an initial value of the input, x_1 was $x_0 - \Delta x$ where Δx was the specified difference of input values, x_2 was $x_0 + \Delta x$, and y_0, y_1, and $y_2$were outputs calculated with x_0, x_1, and x_2, respectively.

The magnitude of I was used to determine the sensitivity. If it was less than 0.05, the sensitivity was small. If it was not less than 0.05 but less than 0.20, the sensitivity was medium. If it was not less than 0.20 but less than 1.00, the sensitivity was high. And if it was 1.00 or more, the sensitivity was very high.

Followings were main parameters the sensitivities of which were determined:

1. **Manning coefficient** (n) which was the value indicating the roughness of the channel. The value of Manning coefficient was between 0.025 and 0.060 for major streams with no boulder or brush and the top width of which at flood stage was more than 100 feet, between 0.025 and 0.050 for minor streams with not many ineffective slopes or sections and the top width of which at flood stage was less than 100 feet, and between 0.011 and 0.016 for concrete canals (Chow, 1959).

2. **Groundwater leakage coefficient** which was the coefficient of loss of water from the river to the groundwater. Groundwater leakage coefficient was between 5.3×10^{-8} s^{-1} and 3.6×10^{-7} s^{-1} for drift, sand, and gravel with some clay and silt, between 9.4×10^{-9} s^{-1} and 8.0×10^{-7} s^{-1} for drift, clay, and silt with considerable sand and gravel, and between 1.3×10^{-10} s^{-1} and 7.7×10^{-9} s^{-1} for drift, clay, and silt with some sand and gravel (Walton, 1965).
3. **Maximum water content in surface storage** (U_{max}) which was the maximum water content of the interception, surface depression, and uppermost soil layer storages. The value of maximum water content in surface storage was normally between 10 and 20 mm (DHI, 2009a).

4. **Maximum water content in root zone storage** (L_{max}) which was the maximum soil moisture content in the root zone. The value of Maximum water content in root zone storage was normally between 50 and 300 mm (DHI, 2009c).

5. **Overland flow runoff coefficient** ($CQOF$) which was the ratio of runoff from the rainfall to the amount of rainfall water that reached the ground. The value of overland flow runoff coefficient was normally between 0.70 and 0.95 for downtown areas, between 0.40 and 0.75 for multi-units residential areas, and between 0.30 and 0.40 for single-family residential areas. (Chow, 1962).

6. **Time constant for routing interflow** ($CKIF$) which was the constant used to determine the interflow contribution. The value of time constant for routing interflow was normally between 500-1000 hours (DHI, 2009a).

7. **Time constant for routing overland flow** (CK_{12}) which was the value used to determine the peak of the hydrograph. The value of time constant for routing overland flow was normally between 3-48 hours (DHI, 2009a).

8. **Root zone threshold value for overland flow** (TOF) which was the value indicating the threshold value of relative soil moisture for the generating of overland flow. The value of time constant for routing overland flow was normally between 0-0.7 (DHI, 2009a).

9. **Root zone threshold value for interflow** (TIF) which was the value indicating the threshold value of relative soil moisture for the generating of interflow. This parameter was rarely important and could be assumed to be 0 in most cases (DHI, 2009a).

10. **Root zone threshold value for groundwater recharge** (TG) which was the value indicating the threshold value of relative soil moisture for the occurrence of groundwater recharge. The value of root zone threshold value for groundwater recharge was normally between 0-0.7 (DHI, 2009c).

11. **Time constant for routing baseflow** $(CKBF)$ which was the value used to determine the peak of hydrograph in dry periods. The value of time constant for routing baseflow was normally between 500-1000 hours (DHI, 2009a).

Besides these parameters, sensitivities of the pump starting time and additional water from the initial overland flow and upstream discharge were also studied.

3.6 Model calibration and verification

Rainfall events in the period of May – August 2010 during the time floodgates must be closed due to the high water level outside the dyke were chosen for the calibration and verification.

In the calibration, each parameter mentioned in the section 3.5 was assigned to the model by trial and then simulations were done to compare the modeled water level with the observed water level. Each parameter was then adjusted and the simulations were done on and on until the error between the modeled and observed data was acceptable. The error could be measured by the root mean square error $(RMSE)$ which could be calculated from Eq. 3.58;

$$RMSE = \sqrt{\frac{\sum_{i=1}^{n}(x_{oi} - x_{ei})^2}{n}} \tag{3.58}$$

where n was the number of data, x_{oi} was the i^{th} observed data, and x_{ei} was the i^{th} modeled data.

The correlation coefficient (CC) was another index that could be used to determine the reliability of the modeled data. It showed the linearity of the relation between modeled data and observed data. The correlation coefficient could be calculated from Eq. 3.59;

$$CC = \frac{\sum_{i=1}^{n}(x_{oi} - \overline{x_o})(x_{ei} - \overline{x_e})}{\sqrt{\sum_{i=1}^{n}(x_{oi} - \overline{x_o})^2 \sum_{i=1}^{n}(x_{ei} - \overline{x_e})^2}} \tag{3.59}$$

where n was the number of data, x_{oi} was the i^{th} observed data, x_{ei} was the i^{th} modeled data, and $\overline{x_o}$ and $\overline{x_e}$ were the averages of observed data and modeled data, respectively.

The magnitude of the correlation coefficient was between 0 and 1. The magnitude of 0 indicated that there was no linearity of the relation between the modeled data and observed data. There would be more linearity between these data when the magnitude of the correlation coefficient was closer to 1.

In the verification, the simulation was done using the calibrated parameters and the modeled data was compared with the observed data to determine if the model was reliable. Root mean square error and correlation coefficient were determined to check for that reliability.

3.7 Uncertainty analysis

The uncertainty analysis was done to determine the confidence limit of the data. The measurement errors were assumed to be random variables that follow a probability distribution (National Astronautics and Space Administration [NASA], 2010). In this study, they were assumed to follow the normal distribution. The measurement error of i^{th} data ($_i$) were calculated from Eq. 3.60;

$$_i = x_{oi} - x_{ei} \tag{3.60}$$

where x_{oi} was the i^{th} observed data, and x_{ei} was the i^{th} expected data.

Mean of errors ( ) and standard deviation of errors ((x)) were calculated from Eqs. 3.61 – 3.62;

$$= \frac{\sum_{i=1}^{n} \ _i}{n} \tag{3.61}$$

$$(\ ) = \sqrt{\frac{\sum_{i=1}^{n}\left(\ _i - \ \right)^2}{n}} \tag{3.62}$$

where n was the number of data.

At the significance level of α, $100(1-\alpha)$ % confidence interval for the measurement error had the lower boundary of $(\ )Z_{0.5\alpha} +$ and upper boundary of $(\ )Z_{1-0.5\alpha} +$ where Z_x was the value such that the area under the standard normal distribution from $-\infty$ to Z_x was x (see Table 3-2).

3.8 Model application

3.8.1 Effects of changing rainfall on canal water level

The studying of effects of changing rainfall on canal water level was done using the MIKE 11. For each pump usage, the design rainfalls with return periods of 2, 5, 10, 25, 50, and 100 years in both the periods of 1982 – 1996 and

1997 – 2010 were simulated in the model and results of water levels from the rainfall were compared between these 2 periods.

Since besides the rainfall, inlet and outlet discharges could also affect the simulated water level, they should be controlled. Discharges for every boundary were set to 0 to cut off the effects of the inlet and outlet discharges and all floodgates along the Bangkok Noi canal and Chao Phraya River were closed to represent the situation that the water could not be drained out by gravity drainage; however, it could still be drained out by pumps which were controllable.

For each pump usage, the design rainfalls mentioned in the section 3.4 with returns period of 2, 5, 10, 25, 50, and 100 years in both the periods of 1982 – 1996 and 1997 – 2010 were simulated in the model and results of water levels from the rainfall were compared between these 2 periods.

3.8.2 Flood mitigation development

Besides studying effects of changing rainfall on canals water levels, this study also aimed to find the appropriate flood mitigation which involved pumping, reducing an initial water level, building dykes in the low-lying flood prone area, and building floodgates in the flood prone area where there was the water flowing back. These mitigations were applied to the MIKE 11 model with simulated rainfall mentioned in the section 3.8.1 to determine the flood characteristics and find out the mitigation which could alleviate the flood.

CHAPTER IV

RAINFALL CHARACTERISTICS

4.1 Rainfall stations

There have been a lot of TMD rainfall stations in the area and nearby. Some of them are still in used while others has been closed. The location of only 32 stations can be verified which are shown in Table 4-1.

Table 4-1 Locations of TMD rainfall stations in inner Bangkok

Station	Easting	Northing	Availability[a]
455201[b]	668682	1518012	92%
455203[b]	669567	1515866	55%
455002	666366	1518396	78%
455003	667691	1517852	52%
455004	666958	1519691	85%
455005	666525	1521747	49%
455006	665890	1522296	86%
455007	666963	1523655	75%
455009	663298	1523816	75%
455010	663368	1522127	70%
455011[b]	664473	1527972	67%
455012	664606	1521151	74%
455014[b]	666337	1513418	89%
455015	662244	1519323	91%
455016	661820	1519966	26%
455017	664961	1517220	83%
455024[b]	669579	1523242	81%
455042[b]	668807	1526525	35%
455049	657443	1523319	91%
455050	656797	1520981	86%
455051	657386	1522858	75%

Table 4-1 Locations of TMD rainfall stations in inner Bangkok (continue)

Station	Easting	Northing	Availability[a]
455052	655554	1522908	63%
455055	659683	1522226	58%
455056	662418	1525194	83%
455058	661152[c]	1522589[c]	79%
455060	660717	1523401	58%
455061	663030	1518498	47%
455063	655441	1516700	86%
455065	658221	1518991	92%
455066 [b]	656094	1512986	78%
455086	664509	1522318	9%
455088 [b]	655838	1510464	30%

[a] "Availability" refers to percent of days daily rainfall data of which are available in the period of 1982-2010.

[b] Stations 455201, 455203, 455011, 455014, 455042, 455042, 455058, 455066, and 455068 are located outside but near the study area.

[c] An easting and a northing of the station 455058 are based on the field investigations in 2011.

The stations chosen as representative rainfall stations for the study were those in the area which provided daily rainfall data for 75% during the period of 1982 – 2010 in the districts to the east of the Chao Phraya River or 60% during the period of 1982 – 2010 in the districts to the west of the river. With these criteria, 15 stations were chosen. Their locations were shown in Figure 4-1.

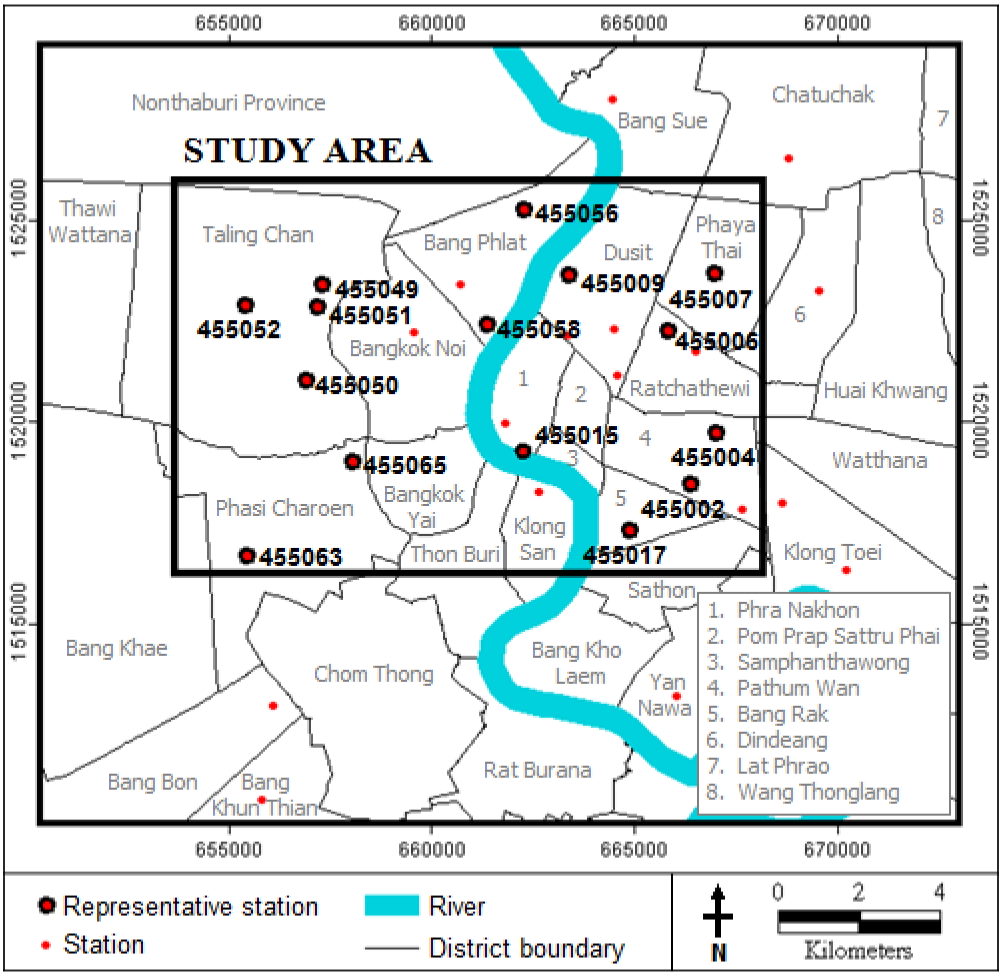


Figure 4-1 Locations of TMD rainfall stations in inner Bangkok

4.2 Rainfall data preparation

For each chosen station, data form the nearest 5 stations in the area (see Table 4-1) were used to fill the missing data by the inverse distance squared method (see Eqs. 3.1 - 3.2). These nearest stations and distances were shown in Table 4-2

Table 4-2 Nearest 5 stations for representative stations

Station	Nearest 5 stations (distance in km)
455002	455004 (1.42), 455003 (1.43), 455017 (1.83), 455201 (2.35), 455012 (3.27)
455004	455002 (1.42), 455003 (1.98), 455005 (2.10), 455201 (2.41), 455012 (2.77)
455006	455005 (0.84), 455086 (1.38), 455012 (1.72), 455007 (1.73), 455010 (2.53)
455007	455006 (1.73), 455005 (1.96), 455024 (2.65), 455086 (2.79), 455042 (3.41)
455009	455058 (1.38), 455056 (1.63), 455010 (1.69), 455086 (1.93), 455060 (2.63)
455015	455016 (0.77), 455061 (1.14), 455012 (2.99), 455010 (3.02), 455017 (3.44)
455017	455002 (1.83), 455061 (2.32), 455003 (2.80), 455004 (3.18), 455015 (3.44)
455049	455051 (0.46), 455052 (1.93), 455050 (2.43), 455055 (2.49), 455060 (3.27)
455050	455051 (1.97), 455052 (2.29), 455049 (2.43), 455065 (2.45), 455055 (3.14)
455051	455049 (0.46), 455052 (1.83), 455050 (1.97), 455055 (2.38), 455060 (3.37)
455052	455051 (1.83), 455049 (1.93), 455050 (2.29), 455055 (4.19), 455065 (4.74)
455056	455009 (1.63), 455058 (1.73), 455060 (2.47), 455010 (3.21), 455011 (3.46)
455058	455060 (1.24), 455009 (1.38), 455056 (1.73), 455010 (2.00), 455055 (2.61)
455063	455065 (3.60), 455066 (3.77), 455050 (4.49), 455052 (6.21), 455088 (6.25)
455065	455050 (2.45), 455055 (3.55), 455063 (3.60), 455016 (3.73), 455051 (3.96)

However, in some stations, there were still some days the data of which were not available both at the representative station and its surrounding stations which caused the daily rainfall missing and being unable to be interpolated at that station. In this case, the years with such days were excluded from the study for each station. Numbers of days the data of which were available at neither the representative station nor its surrounding stations were shown in Table 4-3. Moreover, the years with more than 10% of data interpolated from only 1 surrounding station were considered to have lack of reliable data. As a result, they were also excluded for each station. Numbers of days the data of which were interpolated from only 1 surrounding station were shown in Table 4-4.

Table 4-3 Number of days rainfall data unavailable and unable to be interpolated

Year	Station (455xxx)														
	002	004	006	007	009	015	017	049	050	051	052	056	058	063	065
1982	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
1983	0	0	0	0	6	0	0	9	1	9	1	4	20	0	2
1984	0	0	0	0	2	0	0	0	0	0	0	0	5	0	0
1985	365	365	365	365	365	365	365	6	6	6	6	365	30	15	30
1986	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
1987	0	0	0	0	6	0	0	0	0	0	0	0	0	0	0
1988	0	0	0	0	0	0	0	1	1	1	1	0	0	0	0
1989	0	0	0	0	1	0	0	0	0	0	0	0	0	0	0
1990	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
1991	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
1992	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
1993	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
1994	0	0	2	9	0	0	0	0	0	0	0	0	0	0	0
1995	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
1996	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
1997	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
1998	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
1999	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
2000	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
2001	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
2002	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
2003	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
2004	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
2005	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
2006	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
2007	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
2008	0	4	0	0	0	0	0	0	0	0	0	0	0	0	0
2009	3	7	0	0	0	0	0	0	0	0	0	0	0	0	0
2010	10	10	0	0	0	0	1	0	0	0	0	0	31	0	0

Table 4-4 Number of days rainfall data were interpolated from 1 surrounding station

Year	Station (455xxx)														
	002	004	006	007	009	015	017	049	050	051	052	056	058	063	065
1982	0	0	0	4	1	0	0	10	1	13	1	0	0	0	0
1983	0	0	0	1		1	0							2	
1984	0	0	0	0		90	0	7	1	10	1	20		0	1
1985															
1986	0	0	123	0	8	0	0	3	2	3	4	0	0	8	2
1987	0	0	0	0		0	0	3	0	4	0	6	0	0	0
1988	0	0	0	0	1	0	0					0	0	0	0
1989	0	0	0	0		0	0	1	0	1	0	1	0	0	0
1990	0	0	0	0	0	0	0	0	0	0	0	0	1	0	0
1991	0	0	2	0	0	0	0	0	0	0	0	0	1	0	0
1992	0	0	0	1	0	0	0	0	0	0	0	0	0	0	0
1993	0	0	0	0	0	0	0	1	1	1	1	0	0	0	0
1994	0	0			1	0	0	0	0	0	0	0	1	0	0
1995	0	0	0	1	0	0	0	0	0	0	0	0	0	0	0
1996	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
1997	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
1998	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
1999	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
2000	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
2001	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
2002	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
2003	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
2004	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
2005	0	0	1	0	0	0	0	0	0	0	0	0	0	0	0
2006	0	0	0	0	0	2	0	0	0	0	0	0	0	0	0
2007	0	1	0	0	0	0	0	0	0	0	0	0	0	0	0
2008	4		0	0	0	0	0	0	0	0	0	0	0	0	0
2009			0	5	0	0	212	31	0	31	0	0	243	0	0
2010			150	150	31	1		28	0	28	0	0		0	0

represents years with days the data of which were unavailable and unable to be interpolated.

represents years that more than 10% of data were interpolated from only 1 surrounding station.

After filling the missing data, the double mass analysis was then done to check and adjust the data for its consistency with those from surrounding stations (see the section 3.2.2 for further details about the method). For each representative stations, data from others representative station within the distance of 5 kilometers were used for the analysis. Except for the station 455063, where there was only 1 representative station within the distance of 5 kilometers, the distance was extended to 7 kilometers. Stations within those distances were shown in Table 4-5.

Table 4-5 Stations used for the double mass analysis

Station	Stations used for double mass analysis
455002	455004, 455006, 455015, 455017
455004	455002, 455006, 455007, 455015, 455017
455006	455002, 455004, 455007, 455009, 455015, 455056, 455058
455007	455004, 455006, 455009, 455056
455009	455006, 455007, 455015, 455056, 455058
455015	455002, 455004, 455006, 455009, 455017, 455058, 455065
455017	455002, 455004, 455015
455049	455050, 455051, 455052, 455058, 455065
455050	455049, 455051, 455052, 455063, 455065
455051	455049, 455050, 455052, 455058, 455065
455052	455049, 455050, 455051, 455065
455056	455006, 455007, 455009, 455058, 455060
455058	455006, 455009, 455015, 455049, 455051, 455056
455063	455049, 455051, 455052, 455065
455065	455049, 455051, 455052, 455063

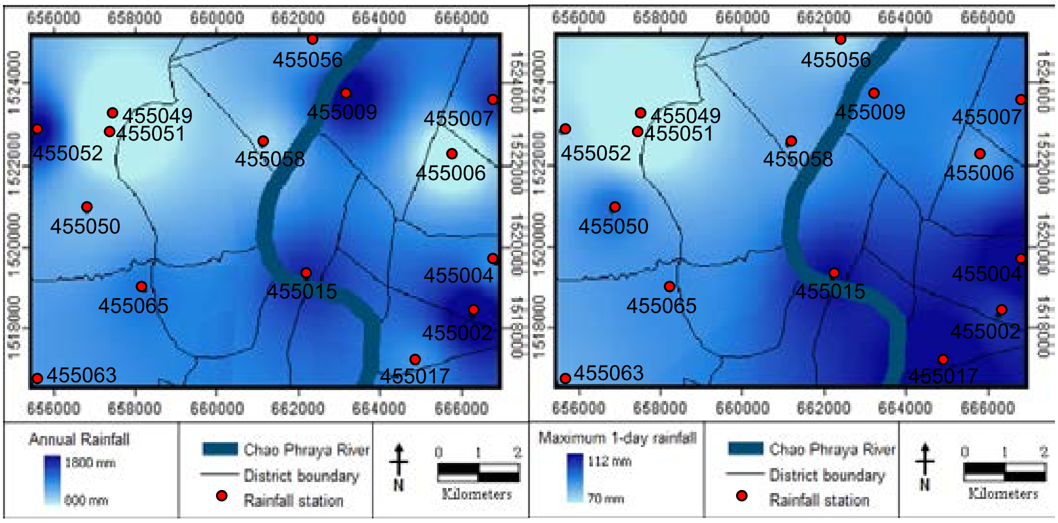
Since the data at the end of the series was more available than that at the beginning of the series (see the table 4-4), the data at beginning of the series was adjusted to be consistent with that at the end of the time series. The double mass curves for each station was shown in the Appendix A.

4.3 Average annual and maximum 1-day rainfall

The mean annual rainfall at each station in inner Bangkok is between 1,248 and 1,601 mm upon the location. The mean annual rainfall from all stations is 1,437 mm. For the eastern part of the Chao Phraya River, the mean annual rainfall is 1,481 mm while for the western part of the river, it is 1,399 mm. The average maximum 1-day rainfall is between 78 and 112 mm upon the location. The average maximum 1-day rainfall from all stations is 95 mm. For the eastern part of the Chao Phraya River, the average maximum 1-day rainfall is 102 mm, while for the western part of the river, it is 88 mm. Hence, the eastern part of the area seems to have more amount of both the annual and maximum 1-day rainfall than the western part. The amounts of the average annual and maximum 1-day rainfall for each station are shown in Table 4-6 and Figure 4-2.

Table 4-6 Amounts of the annual and maximum 1-day rainfall

Station	Annual rainfall (mm)	Maximum 1-day rainfall (mm)
455002	1,601	101
455004	1,439	112
455006	1,270	92
455007	1,515	97
455009	1,581	96
455015	1,527	105
455017	1,432	110
455049	1,309	82
455050	1,396	97
455051	1,248	78
455052	1,569	87
455056	1,409	86
455058	1,369	91
455063	1,427	91
455065	1,461	94

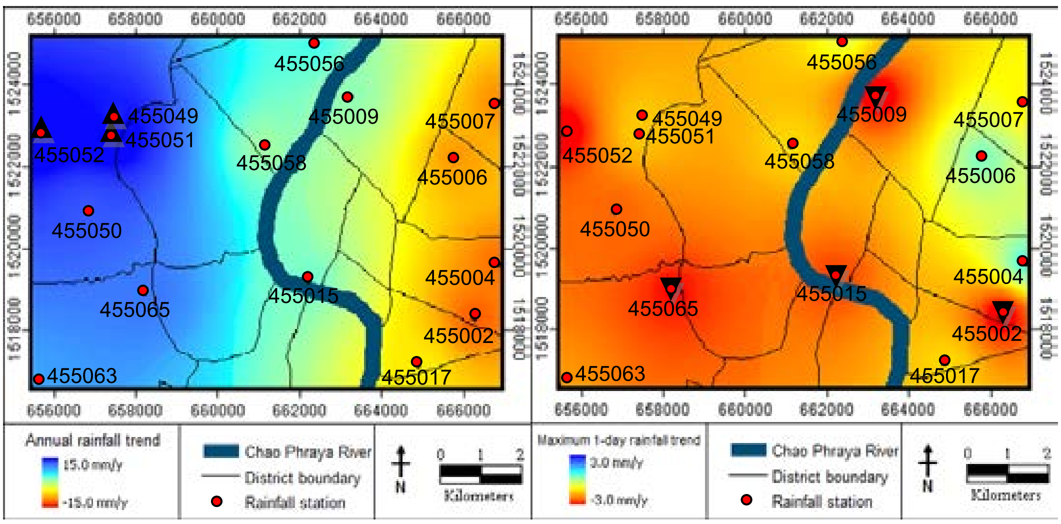


(a) (b)

Figure 4-2 The average (a) annual and (b) maximum 1-day rainfall in inner Bangkok

4.4 Annual and maximum 1-day rainfall trends

The annual and maximum 1-day rainfall trends determined by 10-year moving average are shown in Figure 4-3 and Table 4-7. Up and down arrows in Figure 4-3 show significant increasing and decreasing trends, respectively, at a 0.1 significance level determined by the Mann-Kendall test (see the section 3.2.5). The annual and maximum 1-day rainfall at each station is shown in the Appendix B and Appendix C, respectively.



(a) (b)

Figure 4-3 Trends of the (a) annual and (b) maximum 1-day rainfall in inner Bangkok

Table 4-7 Trends of 10-year average annual and maximum 1-day rainfall (mm/year)

Station	Trend of the annual rainfall		Trend of the maximum 1-day rainfall	
	10-year average Trend	Mann-Kendall value	10-year average trend	Mann-Kendall value
455002	– 8.2 (0.53%)	– 1.45	– 2.2 (2.27%)	– 2.51*
455004	– 7.7 (0.55%)	– 1.10	+ 0.5 (0.46%)	– 0.77
455006	– 6.4 (0.52%)	– 1.00	+ 0.3 (0.37%)	– 0.26
455007	– 6.7 (0.45%)	– 1.10	– 0.7 (0.74%)	– 1.23
455009	+ 1.1 (0.07%)	– 0.12	– 2.2 (2.33%)	– 1.76*
455015	+ 1.9 (0.12%)	– 0.33	– 1.8 (1.78%)	– 1.83*
455017	– 3.9 (0.28%)	– 0.75	– 0.9 (0.86%)	– 1.10
455049	+ 9.4 (0.72%)	+ 2.20*	– 1.0 (1.23%)	– 1.32
455050	+ 8.0 (0.57%)	+ 1.41	– 1.6 (1.64%)	– 1.50
455051	+ 11.8 (0.95%)	+ 2.73*	– 1.0 (1.24%)	– 1.41
455052	+ 11.9 (0.76%)	+ 1.68*	– 2.2 (2.45%)	– 1.54
455056	+ 2.8 (0.20%)	+ 0.29	– 0.5 (0.58%)	– 0.38
455058	+ 0.7 (0.05%)	– 0.22	– 0.9 (0.99%)	– 0.65
455063	+ 5.8 (0.41%)	+ 0.93	– 1.3 (1.38%)	– 1.21
455065	+ 7.1 (0.49%)	+ 1.38	– 2.0 (2.15%)	– 2.42*

* MK values marked by * show significant increasing or decreasing trends at the 0.1 significance level.

Annual rainfall has decreasing trends for 0.28-0.55 % per year in the eastern part of the Chao Phraya River, increasing trends for 0.41-0.95 % per year in the western part of the river, and also increasing trends for 0.05-0.20 % per year along the river. The results of the Mann-Kendall test with the confidence interval of 90% show significant increasing trends at the stations 455049, 455051, and 455052 in the northwestern part of the area.

These changes in amount of the annual rainfall seems to be the effect of urbanization which have enhanced the rainfall in many cities such as Amsterdam and Rotterdam (Buishand, 1979), Jerusalem (Alpert and Shafir, 1990), Ankara (Çiçek and

Turkoglu, 2005) and many cities in India (Kishtawal et al., 2010). In this study, the city core of Bangkok in the eastern part of the Chao Phraya River have the higher amount of annual rainfall than the outskirts in the western part, but the annual rainfall trend in the eastern part is slightly decreasing while the trend in the western part is increasing. These trends are believed to be caused by the expansion of the city. At the beginning, the eastern part was the urbanized city core where the rainfall was subjected to be enhanced by condensation nuclei from pollution, turbulence from surface roughness, and thermal convection from the urban heat island (Chandler, 1965). Then the expansion of the city which let the people move from the city core to the outskirts causes the outskirts being urbanized, so the rainfall is enhanced in the out skirts. Another interesting point is that the trend coincided with another research in Beijing (Fu et al., 2006) where the rainfall during the convective storm was intense along the boundary between urban and non-urban area after the urbanization.

Maximum 1-day rainfall has decreasing trends for almost all over the area. Only station 455004 and 455006 are an exception with the increasing trends of 0.46 and 0.37 % per year, respectively. Others station show decreasing trends for 0.58-2.45 % per year. The northeastern part of the area tends to have less decreasing trends than others part. The results of the Mann-Kendall test show significant decreasing trends at the stations 455002, 455009, 455015 and 455065.

These changes in the amount of maximum 1-day rainfall seem to be the effects of regional climate change. The rising in the global temperature has increased a frequency of El Niño (Bacher et al., 1999). It has been found that the amount of monsoon rainfall in Thailand have a negative relationship with the ENSO (Singhratina et al., 2005). Amount of rainfall in Thailand is decreasing (BMA et al., 2009). In this study, the decreasing in the maximum 1-day rainfall in inner Bangkok which happens almost all over the area is believed to be related to the decreasing of the rainfall in Thailand. Some decreasing trends in extreme monsoon rainfall are observed in the neighboring areas such as in the northeastern part of India (Al-Tabbaa and Pal, 2010) and the Yellow River Basin in China (Hu et al., 2012).

4.5 Change of rainfall frequency distribution

4.5.1 Frequency distributions analysis and testing the goodness of fit

The Frequency analysis comparison between the periods of 1982-1996 and 1997-2010 was done to determine the change of the maximum 1-day rainfall for each return period in the Bangkok Noi and Bangkok Yai districts. Since stations 455015, 455049, 455050, 455051, 455058, and 455065 were located nearby these 2 districts, such stations were selected to explain the rainfall characteristics in the area. The locations of these districts and stations were shown in Figure 4-4

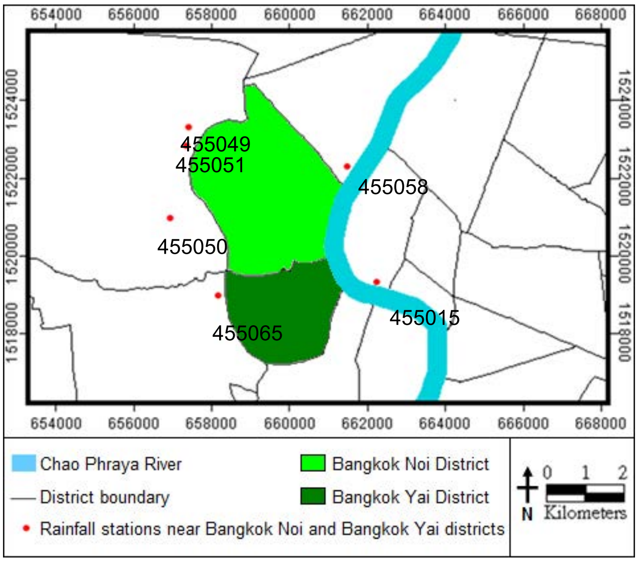


Figure 4-4 Bangkok Noi and Bangkok Yai districts and nearby TMD rainfall stations

For each of these stations, exceedance probabilities of the observed data in each time series of 1982-1996 and 1997-2010 were assigned according to their order of magnitude using Eq. 3.31. In order to find the distribution that best described the maximum 1-day rainfall for each exceedance probability, the following distributions: normal, lognormal, Gumbel, gamma, Pearson type III, and log Pearson type III distributions were tried and then the Kolmogorov-Smirnov test was done to determine the proper one which gave the curve that was closest to the observed data. The Kolmogorov-Smirnov values calculated from Eq. 3.32 were shown in Table 4-8.

Table 4-8 Kolmogorov-Smirnov values of the maximum 1-day rainfall frequency

Distribution type	Period	Stations					
		455015	455049	455050	455051	455058	455065
Normal	1982-1996	0.122	0.100	0.097	0.163	0.122	0.182
	1997-2010	0.107	0.113	0.184**	0.100	0.108	0.114
Lognormal	1982-1996	0.063	0.072*	0.071	0.105	0.142**	0.141
	1997-2010	0.108	0.079	0.131	0.083*	0.065	0.109*
Gumbel	1982-1996	0.055	0.086	0.084	0.097	0.161**	0.125
	1997-2010	0.094	0.071	0.114	0.095	0.059	0.127
Gamma	1982-1996	0.074	0.079	0.063*	0.126	0.132	0.140
	1997-2010	0.087	0.088	0.143**	0.089	0.069	0.114
Pearson type III	1982-1996	0.048*	0.115	0.068	0.101	0.112*	0.116**
	1997-2010	0.084*	0.058*	0.096	0.092	0.053*	0.114
Log Pearson type III	1982-1996	0.066	0.097	0.073	0.087*	0.136**	0.105
	1997-2010	0.095	0.062	0.080	0.086	0.060	0.110
Pearson type III[a]	1982-1996	0.099	0.179**	0.120	0.174	0.122	0.082
	1997-2010	0.143	0.109	0.079*	0.123	0.112	0.114
Log Pearson type III[a]	1982-1996	0.071	0.114	0.074	0.115	0.131**	0.082*
	1997-2010	0.105	0.072	0.109	0.087	0.070	0.111

[a] The skewness coefficients of distributions marked [a] are adjusted using Eq. 3.28.

* For each time series, the minimum KS value among those from every distribution in the table is marked by *.

** For each distribution, the maximum KS value among those from every time series in the study is marked by **.

In comparison with criteria values for the Kolmogorov-Smirnov test at the 0.1 significance level, every value in the table is less than the criteria values which are upon the number of available data in the series as of Table 3-3. Hence, using any of these distributions to describe the exceedance probability of the maximum 1-day rainfall is acceptable. However, most of the lowest values among these distributions for each time series are found for the Pearson type III distribution without adjusting the skewness coefficient. Moreover, among these distributions, the maximum Kolmogorov-Smirnov value when this distribution is applied is also lowest. Hence, the cumulative function of the Pearson type III distribution without adjusting the skewness coefficient has been chosen to describe the frequency distribution of the maximum 1-day rainfall amount in these 2 districts. Frequency curves when this distribution is applied for each station are shown in the Appendix D.

4.5.2 Change of the maximum 1-day rainfall

The frequency curve of the maximum 1-day rainfall in the periods of 1982-1996 and 1997-2010 were calculated from Eqs. 3.24 – 3.27. The result was shown in Table 4-9.

The maximum 1-day rainfall at every return period in the period of 1997-2010 is lower than that in the period of 1982-1996. Except at the station 455058, where the maximum 1-day rainfall with a return periods of 100 years in the period of 1997-2010 is higher than that in the period of 1982-1996, but the difference is only 1 mm. Hence, it may conclude that the overall maximum 1-day rainfall have decreased at every return period in these 2 districts.

Table 4-9 The maximum 1-day rainfall for various return periods (mm)

Return period	Time series	Station					
		455015	455049	455050	455051	455058	455065
2 Years	1982-1996	107	82	105	79	93	102
	1997-2010	87	75	81	69	82	80
5 Years	1982-1996	149	109	133	104	119	134
	1997-2010	118	88	106	84	107	92
10 Years	1982-1996	178	128	151	122	133	154
	1997-2010	139	97	123	93	123	98
25 Years	1982-1996	214	151	171	145	149	178
	1997-2010	163	106	144	104	142	104
50 Years	1982-1996	240	168	185	162	159	196
	1997-2010	181	113	159	112	156	108
100 Years	1982-1996	266	185	199	179	169	213
	1997-2010	198	120	173	119	170	111

4.6 Summary

The mean annual rainfall from 1982 to 2010 in inner Bangkok is 1,437 mm. That in the eastern part of the Chao Phraya River is 1,481 mm while that in the western part is 1,399 mm. Trends of the annual rainfall are decreasing for 0.28-0.55 % per year in the eastern part of the Chao Phraya River, increasing for 0.41-0.95 % per year in the western part and increasing for 0.05-0.20 % per year along the river which seems to be the effect of the expansion of the city.

The average maximum 1-day rainfall from 1982 to 2010 in inner Bangkok is 95 mm. That in the eastern part of the Chao Phraya River is 102 mm while that in the western part is 88 mm. The overall trend of the maximum 1-day rainfall is decreasing which seems to be the effect of the regional climate change. There are only 2 stations with increasing trends with the magnitudes of less than 0.50% per year. Other stations have decreasing trends for 0.58-2.45% per year.

In the Bangkok Noi and Bangkok Yai Districts, it has been found that the cumulative function of the Pearson type III distribution without the adjusting the skewness coefficient is the best function to describe the frequency distribution of the maximum 1-day rainfall. In comparison between the periods of 1982 – 1996 and 1997 – 2010, the maximum 1-day rainfall decreases from 79 – 107 mm to 69 – 87 mm for a return period of 2 years, from 104 – 149 mm to 84 – 118 mm for a return period of 5 years, from 122 – 178 mm to 93 – 139 mm for a return period of 10 years, 145 – 214 mm to 104 – 163 mm for a return period of 25 years, from 159 – 240 mm to 108 – 181 mm for a return period of 50 years, and from 169 – 266 mm to 119 – 198 mm for a return period of 100 years.

CHAPTER V

CANAL WATER LEVEL AND FLOOD MITIGATION

5.1 Model development

In model development, model inputs were the followings:

5.1.1 River and Channel Network

In this study rivers and canals in the area (Figure 5-1) are categorized as followings:

1. Main channel outside the dyke, in which the water levels were not controlled by floodgates. They were the Chao Phraya River and Bangkok Noi Canal.
2. Main channel inside the dyke, in which the water levels could be controlled by floodgates. They were the Chakphra Canal, Mon Canal and Bangkok Yai Canal. However, the Bangwak Canal, Bangcheuknang Canal, Bangphrom Canal and Phasicharoen Canal were also included even though they were not located in the study area since their water levels in these canals were the boundary conditions for the MIKE 11 model. The details of these canals were shown in Table 5-1.
3. Small drainage canals which were others canal in the Bangkok Noi and Bangkok Yai districts except those in the part to the North of the Bangkok Noi Canal. The details of these canals were shown in Table 5-1.

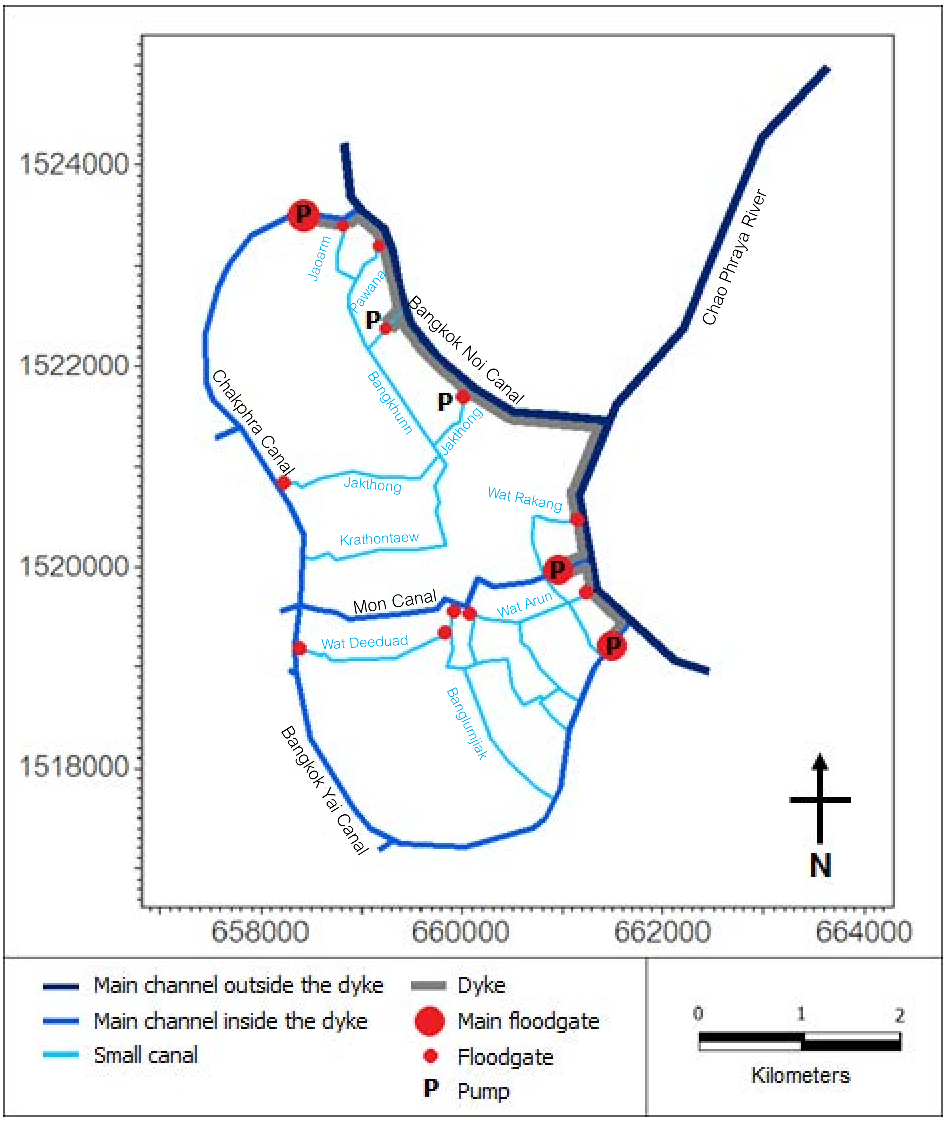


Figure 5-1 Canals and flood control structures in the study area

Table 5-1 Main channel inside the dyke and small canals in the study area (DDS)

Canal	Flow (from – to)	Excavation depth (m, MSL)
Mon	Mon Canal Junction[a] – Chao Phraya River	-2.5
Bangkok Yai	Mon Canal Junction[a] – Bangkok Noi Canal	-2.5
Chakphra	Mon Canal Junction[a] – Chao Phraya River	-2.5
Phasicharoen	Paricharoen District – Bangkok Yai Canal	-2.5
Bangcheuknang	Paricharoen District – Mon Canal Junction[a]	-2.0
Bangphrom	Talingchan District – Chakphra Canal	-2.0
Bangwak	Paricharoen District – Bangkok Yai Canal	-1.5
Jakthong	Chakphra Canal – Bangkok Noi Canal	-1.5
Bangkhunnon	Jakthong Canal – Bangkok Noi Canal	-1.5
Jaoarm	Bangkhunnon Canal – Chakphra Canal	-1.5
Pawana	Bangkhunnon Canal – Bangkok Noi Canal	-1.5
Wangderm	Mon Canal – Bangkok Yai Canal	-1.5
Wat Arun	Mon Canal – Chao Phraya River	-1.5
Banglamjiak	Mon Canal – Bangkok Yai Canal	-1.0
Wat Rachasittharam	Wat Arun Canal – Bangkok Yai Canal	-1.0
Wat Sankkrajai	Wat Rachasittharam Canal – Bangkok Yai Canal	-1.5
Lang Wat Rachasittharam	Wat Sankkrajai Canal – Banglamjiak Canal	-1.0
Yak Wat Arun	Lang Wat Rachasittharam Canal - Wat Arun Canal	-1.5
Wat Deeduad	Bangkok Yai Canal – Banglamjiak Canal	-1.5
Bankhamin	Sutthawas Road[b] – Mon Canal	-1.0
Wat Rakang	Bankhamin – Chao Phraya River	-1.0
Wat Yangsuttharam	Jakthong Canal – Chakphra Canal[c]	-1.0
Wat Dongmullek	Wat Yangsuttharam Canal – Krathontaew Canal	-1.0
Krathontaew	Wat Dongmullek Canal – Chakphra Canal	-1.0

[a] Mon Canal Junction is the intersection among the Mon, Chakphra, Bangkok Yai, and Bangcheuknang Canals.

[b] Sutthawas Road is the road in the Bangkok Noi District. However, in this study the Bankhamin Canal started from the intersection between the Bankamin and Wat Rakang Canals.

[c] In this study, the Wat Yangsuttharam Canal ended at the junction between the Wat Yangsuttharam and Wat Dongmullek Canals.

5.1.2 Cross-sections of channels

This study used the cross-sectional data surveyed in 2004 by the RID. The cross-sections were shown in the Appendix E. However, in the study area, the data from the RID was available only along the Chao Phraya River.

For other channels, cross-sections were derived in rectangular shapes using canal widths, bed levels and bank elevations. The widths were measured during the field surveys in 2012 using the laser distance meter DISTO E5. The bed levels were determined from excavation depths obtained from the DDS (see Table 5-1) except for the Bangkok Noi Canal where the excavation depth data from the DDS was not available. The bed levels of this canal were calculated from the observed water level and water depth measured during the field surveys in 2012 using a sonar radiometer. The bank elevations were as of benchmarks data from Public Works Department surveyed in 2007 and the digital elevation model from the LDD derived in 2008. The widths, bed levels and bank elevations were shown in Table 5-2 and location for the points of cross-sectional data were shown in Figure 5-2.

Table 5-2 Widths, bed levels and bank elevations of channels (DDS, LDD, and Public Works Department)

No.	Canal	Easting	Northing	Width (m)	Bed level (m, MSL)	Bank elevation (m, MSL)
1	Wangderm	660860	1519271	5.4	-1.5	3.39
2	Wangderm	660997	1519489	3.6	-1.5	3.39
3	Wangderm	661005	1519481	3.8	-1.5	3.39
4	Bangkok Yai	661034	1519449	25.6	-2.5	3.39
5	Wat Arun	660703	1519938	3.5	-1.5	3.39
6	Wangderm	660605	1520052	4.0	-1.5	3.39

Table 5-2 Widths, bed levels and bank elevations of channels (Continue)

No.	Canal	Easting	Northing	Width (m)	Bed level (m, MSL)	Bank elevation (m, MSL)
7	Wangderm	660511	1520181	4.0	-1.5	3.39
8	Mon	660753	1520328	14.0	-2.5	3.39
9	Bankhamin	660420	1520361	5.0	-1.0	3.39
10	Bankhamin	660346	1520831	8.9	-1.0	3.39
11	Bangkok Noi	660302	1521788	66.8	-7.0	3.00
12	Bangkok Yai	660608	1518084	28.7	-2.5	3.02[b]
13	Banglumjeak	660130	1518367	3.3[a]	-1.0	2.86[b]
14	Lang Wat Rachasittharam	660309	1518944	4.2	-1.0	2.34[b]
15	Wat Sankkrajai	660520	1518849	4.8	-1.5	3.12[b]
16	Wat Sankkrajai	660515	1519017	4.2	-1.5	3.08[b]
17	Wat Rachasittharam	660668	1519044	5.5	-1.0	3.12[b]
18	Bangkok Yai	660838	1519029	31.9	-2.5	3.24[b]
19	Wat Rachasittharam	660524	1519176	6.2	-1.0	2.98[b]
20	Wat Rachasittharam	660327	1519448	3.7	-1.0	2.74[b]
21	Wat Rachasittharam	660230	1519598	4.1[a]	-1.0	2.61[b]
22	Wat Rachasittharam	660230	1519765	3.6	-1.0	2.49[b]
23	Wat Arun	660230	1519765	3.3	-1.5	2.49[b]
24	Wat Arun	660269	1519789	3.4	-1.5	2.57[b]
25	Mon	660023	1520118	19.6	-2.5	0.82
26	Wat Dongmullek	659448	1520737	3.6	-1.0	0.95
27	Wat Rakang	660398	1520800	6.2	-1.0	3.39
28	Wangderm	660668	1519994	4.7	-1.5	3.39
29	Wat Arun	660765	1519973	5.9	-1.5	3.39
30	Bangkok Yai	658606	1517870	38.5	-2.5	2.47
31	Bangkok Yai	658899	1517631	20	-2.5	2.75
32	Phasicharoen	658877	1517465	15.6	-2.5	1.85
33	Bangkok Yai	659675	1517516	30	-2.5	2.73
34	Banglumjeak	659846	1518856	3.3	-1.0	2.70
35	Lang Wat Rachasittharam	660181	1519061	4.4	-1.0	2.54[b]

Table 5-2 Widths, bed levels and bank elevations of channels (Continue)

No.	Canal	Easting	Northing	Width (m)	Bed level (m, MSL)	Bank elevation (m, MSL)
36	Bangcheuknang	657838	1519850	16.3	-2.0	1.86
37	Bangwak	657923	1519279	15.1	-1.5	1.77
38	Bangkok Yai	658024	1519528	11.9	-2.5	1.54
39	Watdeeduad	658092	1519461	4.3	-1.5	1.58
40	Watdeeduad	658256	1519419	3.3	-1.5	1.63
41	Watdeeduad	658378	1519394	3.5	-1.5	1.70
42	Watdeeduad	658477	1519383	2.7	-1.5	1.70
43	Watdeeduad	658560	1519360	2.5	-1.5	1.74
44	Watdeeduad	658792	1519372	3.5	-1.5	2.00
45	Watdeeduad	659000	1519408	3.9	-1.5	2.25
46	Mon	658957	1519791	16.8	-2.5	1.96
47	Watdeeduad	659025	1519406	2.8	-1.5	2.24
48	Banglumjeak	660365	1518149	3.8	-1.0	2.95[b]
49	Watdeeduad	659252	1519512	3	-1.5	2.14
50	Watdeeduad	659345	1519567	3	-1.5	2.09
51	Banglumjeak	659528	1519382	3.1[a]	-1.0	2.22
52	Banglumjeak	659535	1519325	2.6	-1.0	2.25
53	Banglumjeak	659603	1519303	5.7[a]	-1.0	2.24
54	Banglumjeak	659667	1519311	4.4	-1.0	2.23
55	Banglumjeak	659608	1519244	3.6	-1.0	2.25
56	Lang Wat Rachasittharam	659769	1519330	4.3[a]	-1.0	2.19
57	Yak Wat Arun	659769	1519330	3.2	-1.5	2.19
58	Yak Wat Arun	659771	1519393	3.5	-1.5	2.16
59	Lang Wat Rachasittharam	659946	1519367	3.9	-1.0	2.14
60	Yak Wat Arun	659732	1519538	4.5	-1.5	2.07
61	Yak Wat Arun	659743	1519688	6.5	-1.5	1.95
62	Wat Arun	659752	1519892	5.7	-1.5	1.65
63	Wat Arun	659857	1519811	4.2	-1.5	1.84
64	Banglumjeak	659558	1519699	3.4	-1.0	1.97

Table 5-2 Widths, bed levels and bank elevations of channels (Continue)

No.	Canal	Easting	Northing	Width (m)	Bed level (m, MSL)	Bank elevation (m, MSL)
65	Banglumjeak	659567	1519602	4.3[a]	-1.0	2.03
66	Watdeeduad	659567	1519602	4.2	-1.5	2.03
67	Krathontaew	658898	1520448	4.0[a]	-1.0	0.91
68	Krathontaew	658849	1520445	4.4	-1.0	0.80
69	Krathontaew	658542	1520393	4.5	-1.0	0.33
70	Chakphra	658282	1523772	18.0	-4.7	0.70
71	Bangkok Yai	658192	1518569	14.1	-2.5	1.81
72	Krathontaew	658214	1520374	7.6	-1.0	0.33
73	Krathontaew	659030	1520466	3.3	-1.0	0.95
74	Krathontaew	659265	1520480	3.8	-1.0	0.95
75	Krathontaew	659488	1520513	4.0[a]	-1.0	0.95
76	Wat Dongmullek	659461	1520628	4.7	-1.0	0.95
77	Wat Yangsuttharam	659444	1520908	3.4	-1.0	0.95
78	Wat Yangsuttharam	659418	1520857	5.2	-1.0	0.95
79	Wat Dongmullek	659418	1520857	3.3	-1.0	0.95
80	Wat Yangsuttharam	659410	1521037	3.0	-1.0	0.95
81	Wat Yangsuttharam	659429	1521136	3.5	-1.0	0.95
82	Wat Yangsuttharam	659440	1521162	4.2	-1.0	0.95
83	Jakthong	659086	1521186	6.8	-1.5	0.95
84	Jakthong	659261	1521196	6.6[a]	-1.5	0.95
85	Jakthong	659431	1521416	6.0[a]	-1.5	0.95
86	Jakthong	659391	1521493	6.2	-1.5	0.95
87	Bangkhunnon	659391	1521493	6.3	-1.5	0.95
88	Bangkhunnon	659386	1521508	3.3	-1.5	0.95
89	Wat Yangsuttharam	659508	1521304	5.1	-1.0	0.95
90	Bangkhunnon	659331	1521560	4.8	-1.5	0.84
91	Bangkhunnon	659272	1521648	5.9	-1.5	0.57
92	Jakthong	659630	1521735	7.1	-1.5	0.53
93	Bangphrom	657185	1521585	15.4	-2.5	1.40

Table 5-2 Widths, bed levels and bank elevations of channels (Continue)

No.	Canal	Easting	Northing	Width (m)	Bed level (m, MSL)	Bank elevation (m, MSL)
94	Chakphra	657747	1521287	12.4	-2.5	0.04
95	Jakthong	658536	1521228	6.1	-1.5	0.33
96	Jakthong	658317	1521182	6.7	-1.5	0.33
97	Jakthong	658862	1521203	6.8[a]	-1.5	0.84
98	Wat Arun	660499	1519859	4.0	-1.5	3.00[b]
99	Bankhamin	660388	1520408	6.4	-1.0	3.39
100	Bankhamin	660373	1520486	6.8	-1.0	3.39
101	Bankhamin	660356	1520566	9.0	-1.0	3.39
102	Bangkok Noi	659497	1522314	45.7	-6.0	3.00
103	Jakthong	659660	1521866	7.0	-1.5	0.18
104	Jakthong	659727	1522042	8.8	-1.5	0.19
105	Bangkhunnon	658885	1522660	5.5	-1.5	0.09
106	Bangkok Yai	659081	1522857	55.9	-2.5	2.80
107	Bangkhunnon	659033	1522819	6.9	-1.5	0.21
108	Bangkhunnon	658825	1522546	8.1	-1.5	0.08
109	Bangkhunnon	658727	1522442	7.5[a]	-1.5	0.08
110	Jaoarm	658727	1522442	3.9	-1.5	0.55
111	Jaoarm	658641	1522612	3.2	-1.5	0.77
112	Bangkhunnon	659119	1521855	4.8	-1.5	0.53
113	Bangkhunnon	659057	1521951	4.2	-1.5	0.20
114	Bangkhunnon	658965	1522075	5.7[a]	-1.5	0.14
115	Bangkhunnon	658845	1522276	6.1	-1.5	0.07
116	Bangkhunnon	658804	1522312	2.8	-1.5	0.24
117	Jaoarm	658601	1523126	6.5[a]	-1.5	0.77
118	Pawana	658668	1523268	5.5	-1.5	0.77
119	Jaoarm	658445	1523481	4.4[a]	-1.5	0.77
120	Jaoarm	658537	1523737	4.1	-1.5	0.77
121	Jaoarm	658514	1523675	7.6	-1.5	0.77
122	Jaoarm	658505	1523201	4.9	-1.5	0.75

Table 5-2 Widths, bed levels and bank elevations of channels (Continue)

No.	Canal	Easting	Northing	Width (m)	Bed level (m, MSL)	Bank elevation (m, MSL)
123	Pawana	658893	1523515	6.9	-1.5	0.27
124	Wat Rakang	660653	1520735	4.2	-1.0	0.89
125	Chakphra	657449	1523409	30	-2.5	0.19
126	Bangkok Noi	658472	1524509	54.9	-6.3	2.44

[a] The widths of the channels at the points 13, 21, 51, 53, 56, 65, 67, 75, 84, 85, 97, 109, 114, 117 and 119 were shown in terms of average.

[b] The bank elevations of the channels at the points 12, 13, 14, 15, 16, 17, 18, 19, 20, 21, 22, 23, 24, 35, 48 and 98 were obtained from the interpolation.

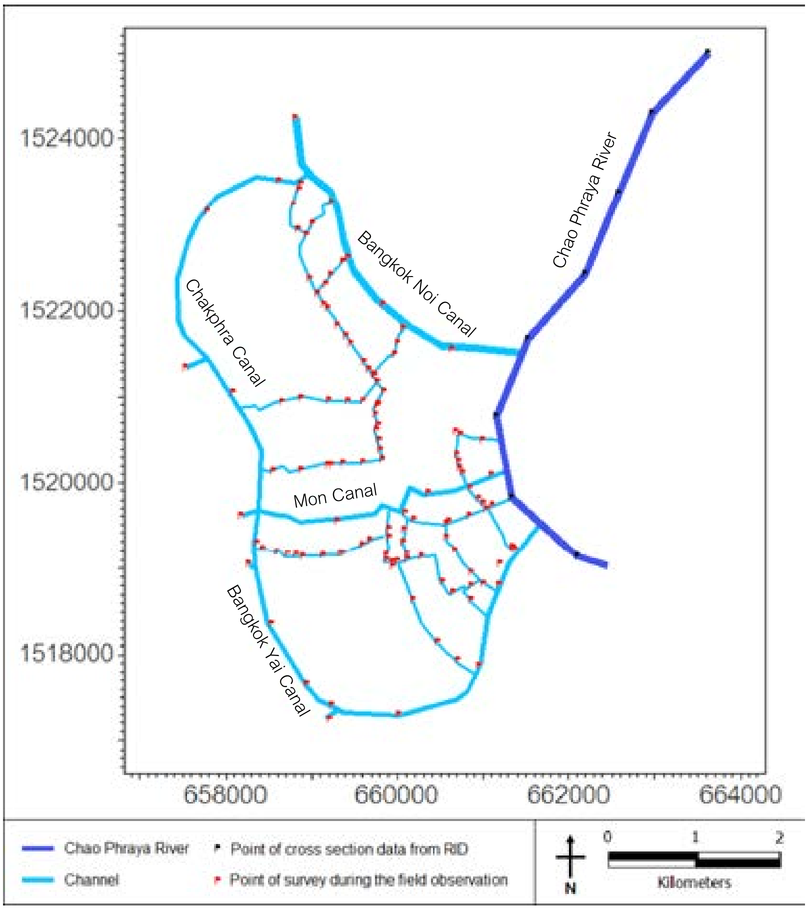


Figure 5-2 Points of cross-sectional data in the MIKE 11 model

5.1.3 Flood control structures

There were 11 floodgates along the Chao Phraya River and Bangkok Noi Canal. Some of them were also equipped with pumps. There were 3 of the gates which were equipped with large numbers of pumps, so they were considered as main pumping stations. These stations included the pumping stations located at the connection points between the Chakphra and Bangkok Noi canals, between the Mon Canal and Chao Phraya River, and between the Bangkok Yai Canal and Chao Phraya River. There were also floodgates along the Chakphra, Mon, and Bangkok Yai Canals. Locations of the gates were shown in Figure 5-1 and sizes of the gates and pumping capacities were shown in Table 5-3.

Table 5-3 Floodgates and pumps in the study area (DDS)

No.	Inner channel	Outer channel	Width[b]	Pumping capacity
1	Chakphra Canal	Bangkok Noi Canal	15.5 m	45.0 m^3/s
2	Mon Canal	Chao Phraya River	12.0 m	24.0 m^3/s
3	Bangkok Yai Canal	Chao Phraya River	18.0 m	54.0 m^3/s
4	Jaoarm Canal	Bangkok Noi Canal	4.0 m[c]	-
5	Pawana Canal	Bangkok Noi Canal	4.0 m[c]	-
6	Bangkhunnon Canal	Bangkok Noi Canal	4.0 m[c]	0.3 m^3/s
7	Jakthong Canal	Bangkok Noi Canal	4.0 m[c]	6.0 m^3/s
8	Wat Rakang Canal	Chao Phraya River	4.0 m[c]	-
9	Wat Arun Canal	Chao Phraya River	4.0 m[c]	-
10[a]	Jakthong Canal	Chakphra Canal	2.0 m[c]	2.0 m^3/s
11[a]	Wat Deeduad Canal	Bangkok Yai Canal	3.0 m[c]	-
12[a]	Banglumjeak Canal	Mon Canal	3.0 m[c]	-
13[a]	Wat Arun Canal	Mon Canal	4.0 m[c]	-
14[a]	Wat Deeduad Canal	Bangkumjeak Canal	3.0 m[c]	-

[a] The floodgates no. 10, 11, 12, 13, and 14 were not used in the model.

[b] "Width" refers to the sum of width of all gates along the width of the canal.

[c] The widths of the floodgates no. 4, 5, 6, 7, 8, 9, 10, 11, 12, 13, and 14 were obtained from the field observations.

The sizes of the gates were obtained from the DDS for 3 main floodgates and measured during the field investigations using the laser distance meter DISTO E5 for others gate. Pumping capacities were obtained from the DDS.

5.1.4 Catchments

In this study, catchments were determined with the assumption that storm water could not cross main roads and canals. Hence, the roads and canals were considered catchments boundaries. The storm water of such catchment would be led to the canals, which were the boundary, uniformly along their lengths. Locations of the catchment boundaries, which were main roads and canals, were shown in Figure 5-3.

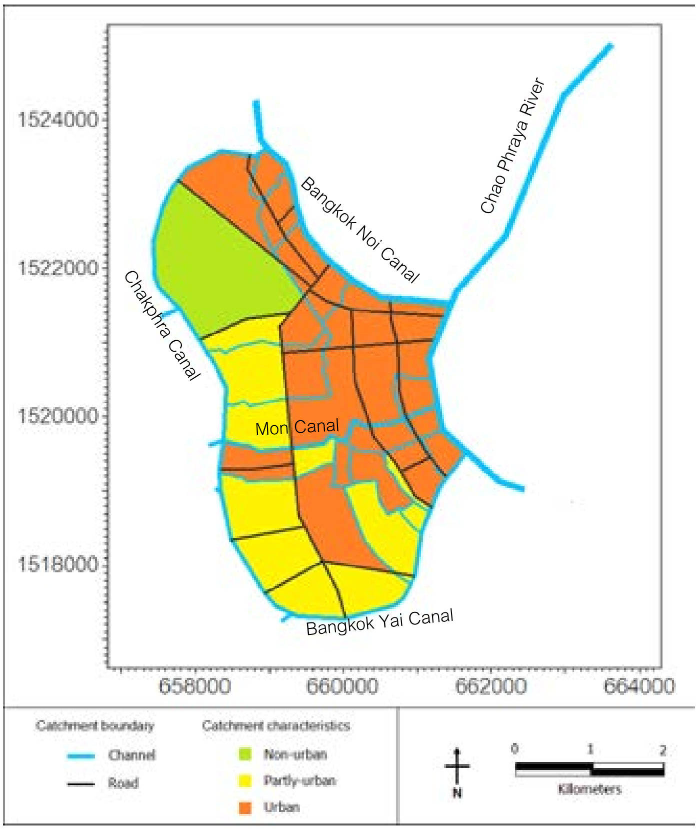


Figure 5-3 Catchment boundaries and characteristics in the MIKE 11 model

This study classified characteristics of catchments into 3 types, urban, non-urban, and partly-urban, as shown in Figure 5-3. The catchment would be urban if most parts of it were urban areas. If most parts of it were agricultural areas or undeveloped lands, it would be non-urban. If it had the area of urban lands which was not very different from the area of agricultural or undeveloped lands, it would be partly-urban. The characteristics of the land were determined from the field observations in 2012.

Every catchment had an area of not more than 1.0 km^2 except a non-urban catchment in the northwestern part which had an area of approximately 2.4 km^2. As a result, this non-urban catchment was considered a large catchment while others were considered small catchments in this study.

5.1.5 Network boundary

There were 9 points where the water levels were observed which were shown in Table 5-4 and Figure 5-4.

Table 5-4 Points where water levels were measured in the study area (DDS and RID)

Station	Channel	Easting	Northing	Time Step (minutes)
BP01	Bangphrom Canal	-[a]	-[a]	15
BW01	Bangwak Canal	-[a]	-[a]	15
C4	Chao Phraya River	662100	1519066	15
C12	Chao Phraya River	663620	1524970	15
W03	Chakphra Canal	658365	1523761	15
W04	Mon Canal	660566	1520252	15
W06	Bangkok Yai Canal	661132	1519543	15
W07	Phasicharoen Canal	658811	1517474	15
W22	Bangcheuknang Canal	-[a]	-[a]	15

[a] Locations of the stations BP01, BW01, and W22 were not available from the DDS

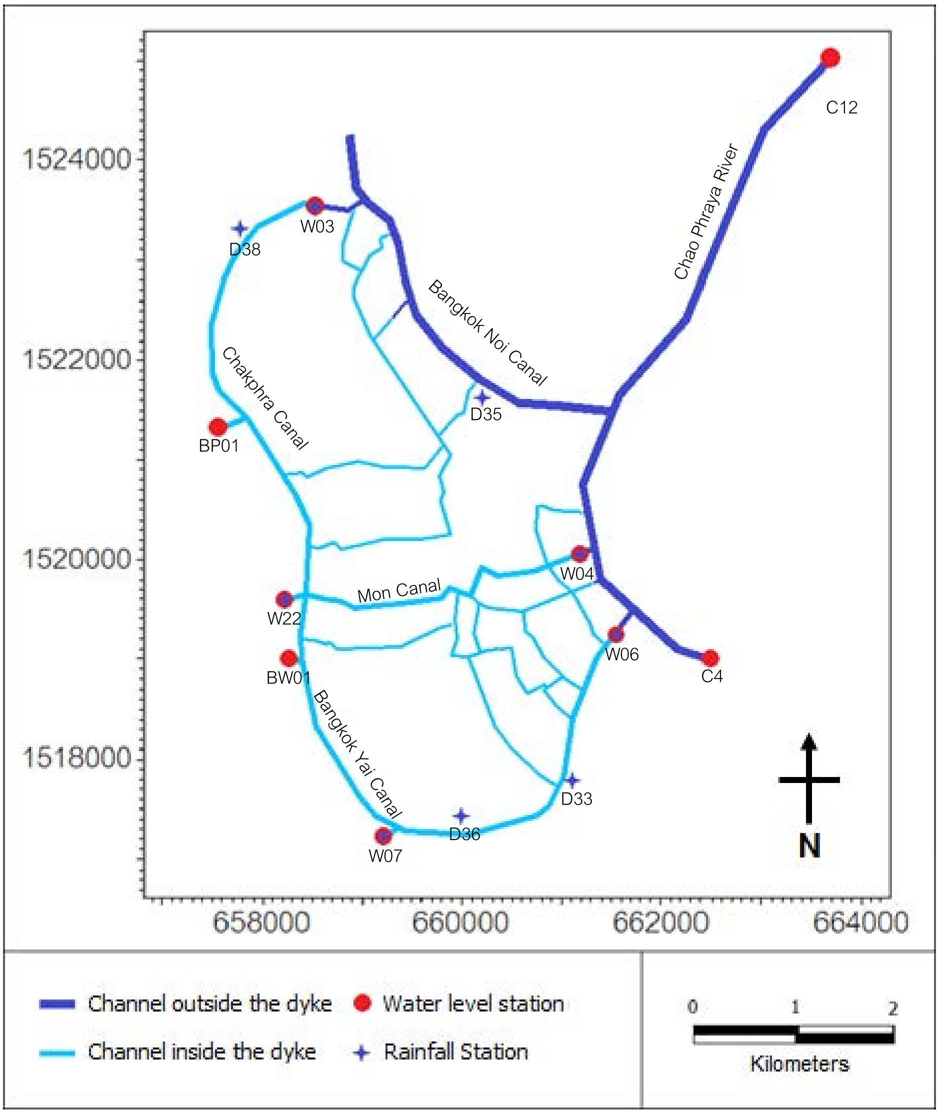


Figure 5-4 Locations of water level and rainfall stations in the study area (DDS and RID)

Since the water levels at the Bangkok Yai Canal floodgate (W06) and the Mon Canal floodgate (W04) were not the end point of the canal network in the study area, they were not considered network boundary in the MIKE 11 model but were used for the calibration and verification, and since the upstream boundary of the Bangkok Noi Canal was not available, the water level outside the Chakphra Canal floodgate (W03) was applied to that boundary instead and the water level inside this gate was used for the calibration and verification. The network boundaries were shown in Figure 5-5 and Table 5-5.

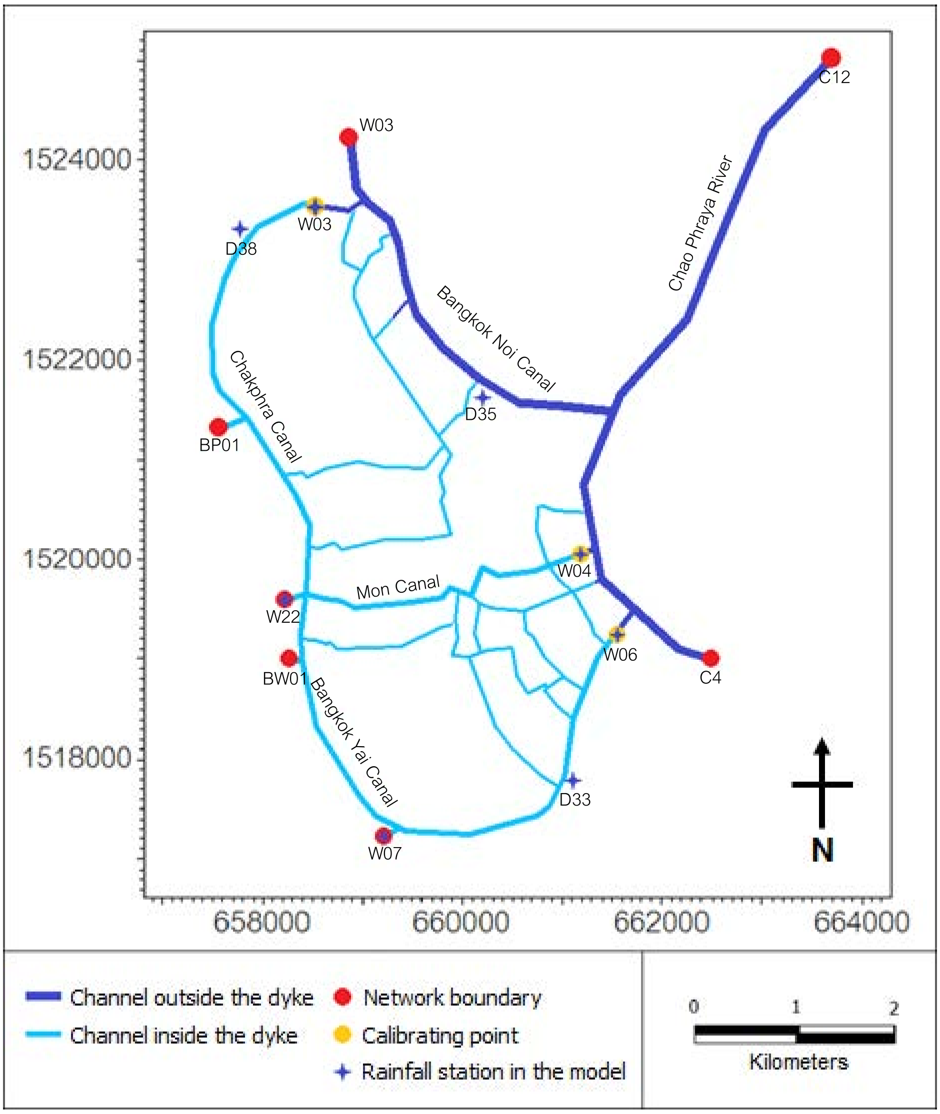


Figure 5-5 Network boundaries, calibrating points and rainfall stations in the MIKE 11 model

Table 5-5 Points of network boundary conditions in the MIKE 11 model

Point	Channel	Easting	Northing
BP01	Bangphrom Canal	657185[a]	1521585[a]
BW01	Bangwak Canal	657923[a]	1519279[a]
C4	Chao Phraya River	662100	1519066
C12	Chao Phraya River	663620	1524970
W03	Chakphra Canal	658472[b]	1524509[b]
W07	Phasicharoen Canal	658811	1517474
W22	Bangcheuknang Canal	657838[a]	1519850[a]

[a] Locations of the points BP01, BW01 and W22 were points in canals obtained from the field investigations.

[b] The point W03 in the table was moved from its real location to represent the upstream boundary of the Bangkok Noi Canal which was located nearby

5.1.6 Rainfall station

There were 9 locations where the rainfall data were observed which were shown in Figure 5-4 and Table 5-6.

Table 5-6 Locations of rainfall stations in the study area (DDS)

Station	Location	Northing	Easting	Time step (minutes)
D33	Thonburi District	661059	1517765	5
D35	Bangkok Noi District	659772	1521980	5
D36	Bangkok Yai District	659614	1517627	5
D38	Talingchan District	657548	1523500	5
W03	Chakphra Canal	658365	1523761	15
W04	Mon Canal	660566	1520252	15
W06	Bangkok Yai Canal	661132	1519543	15
W07	Phasicharoen Canal	658811	1517474	15
W22	Bangcheuknang Canal	-[a]	-[a]	15

[a] Location of the station W22 was not available

However, the rainfall data at the station D36 appeared to be error. In many rainfall events, there was no rainfall observed at this station while they were observed at the surrounding stations. Hence, the station D36 was excluded from the study. The rainfall stations used in the MIKE 11 model were shown in Figure 5-5 and Table 5-7.

Table 5-7 Locations of rainfall stations in the MIKE 11 model

Station	Location	Easting	Northing
D33	Thonburi District	661059	1517765
D35	Bangkok Noi District	659772	1521980
D38	Talingchan District	657548	1523500
W03	Chakphra Canal	658365	1523761
W04	Mon Canal	660566	1520252
W06	Bangkok Yai Canal	661132	1519543
W07	Phasicharoen Canal	658811	1517474
W22	Bangcheuknang Canal	657838[a]	1519850[a]

[a] Location of the station W22 was the point in the Bangcheuknang Canal obtained from the field investigations.

5.2 Rainfall design

For each station of W03, W04, W06, W07, W22, D33, D35, and D38, rainfall hyetographs with the amount of more than 58.7 millimeters of rain, which was the amount of rainfall with a return period of 2 years and duration of 1 hour (see Table 2-1), in the period between May 2008 and September 2010 were converted to the dimensionless mass curves by converting the duration and cumulative rainfall depth in their units to those in dimensionless units. After that, for each station, the dimensionless mass curves from all events were averaged to determine the dimensionless mass curve of the design rainfall. The dimensionless mass curves at each station were shown in the Appendix F.

The dimensionless mass curve of the design rainfall was then converted to the hyetograph of the design rainfall by multiplying the dimensionless rainfall depth with an appropriate depth, multiplying the dimensionless duration with an appropriate duration, and converting the cumulative rainfall depth to the non-cumulative rainfall hyetograph. The appropriate rainfall depth for various return periods was determined by the maximum 1-day rainfall at the nearest TMD station (see Table 4-9) which was shown in Table 5-8.

Table 5-8 Rainfall depths (mm) for each return period in the stations used in the MIKE 11 model

Return period	Period of study	Station used in the model *Nearest TMD station (455xxx)*							
		W03	W04	W06	W07	W22	D33	D35	D38
		049	*015*	*015*	*065*	*065*	*065*	*058*	*049*
2-year	1982-1996	82	107	107	102	102	102	93	82
	1997-2010	75	87	87	80	80	80	82	75
5-year	1982-1996	109	149	149	134	134	134	119	109
	1997-2010	88	118	118	92	92	92	107	88
10-year	1982-1996	128	178	178	154	154	154	133	128
	1997-2010	97	139	139	98	98	98	123	97
25-year	1982-1996	151	214	214	178	178	178	149	151
	1997-2010	106	163	163	104	104	104	142	106
50-year	1982-1996	168	240	240	196	196	196	159	168
	1997-2010	113	181	181	108	108	108	156	113
100-year	1982-1996	185	266	266	213	213	213	169	185
	1997-2010	120	198	198	111	111	111	170	120

The appropriate rainfall duration for this study was 3-hour duration because the Department of Drainage and Sewerage planned to build the capacity of the area in Bangkok to store the storm water of the rainfall with that duration (สำนักการระบายน้ำ กรุงเทพมหานคร, 2553). The design hyetograph at each station was shown in Figures 5-6 – 5-13.

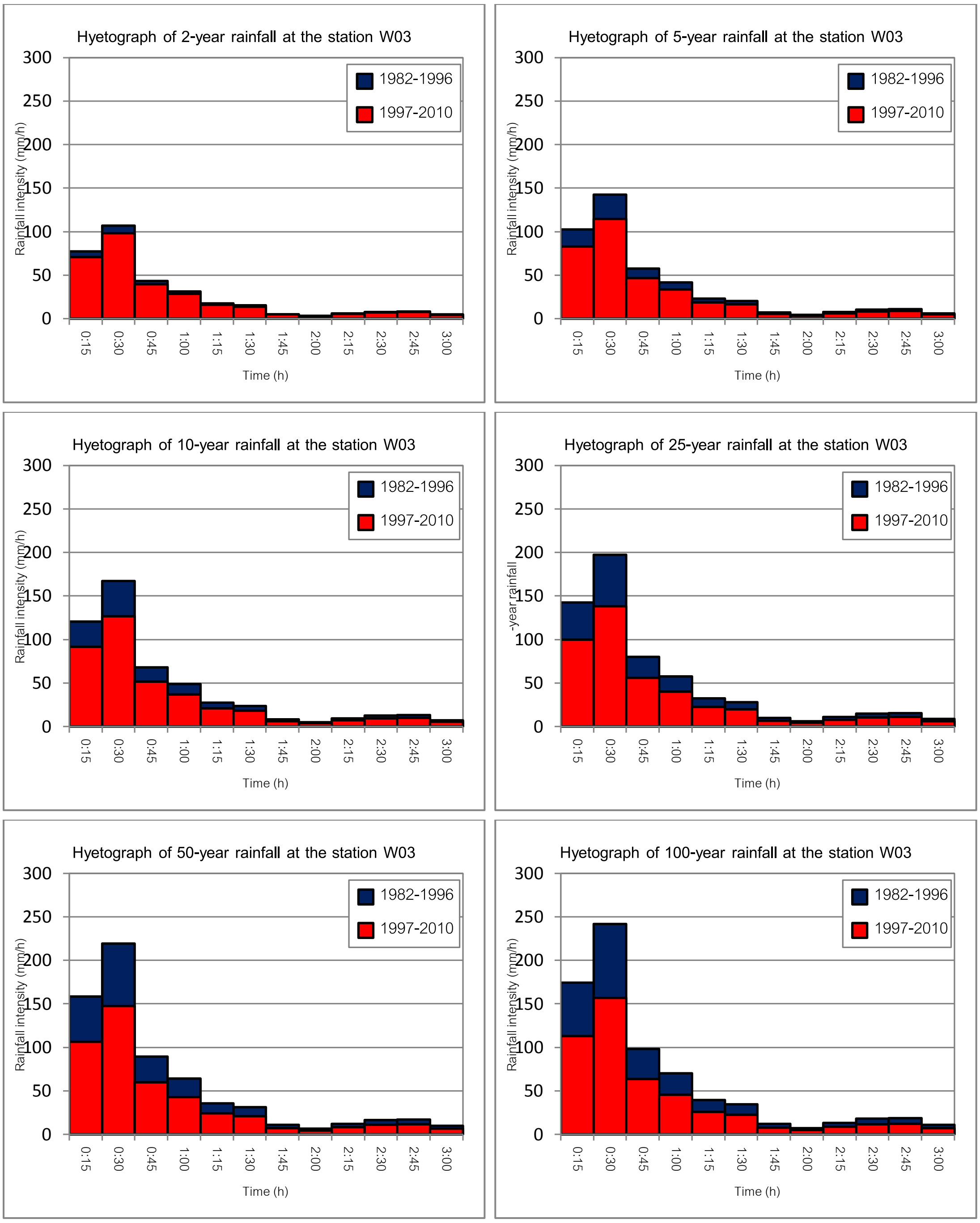


Figure 5-6 Design hyetograph at the station W03

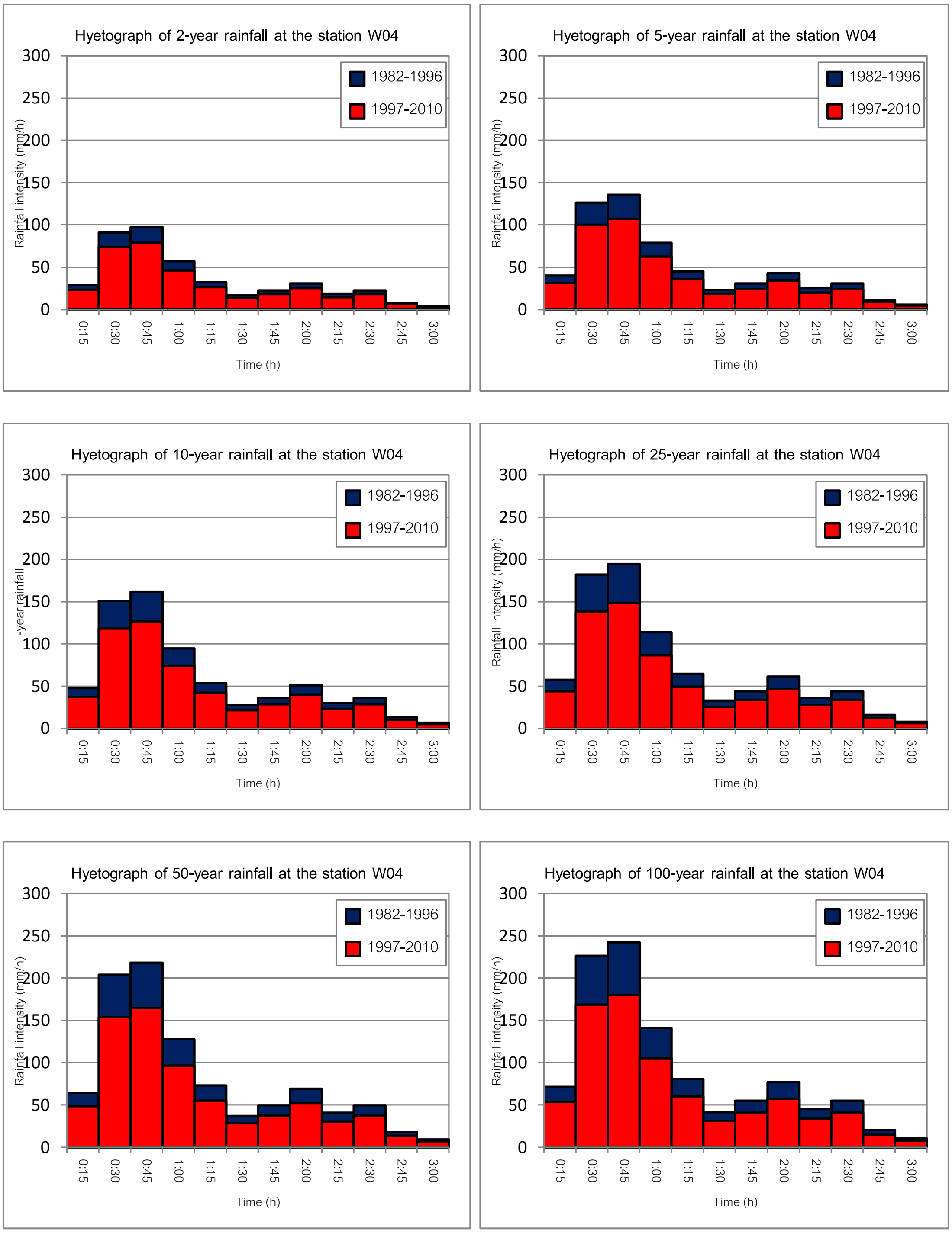


Figure 5-7 Design hyetograph at the station W04

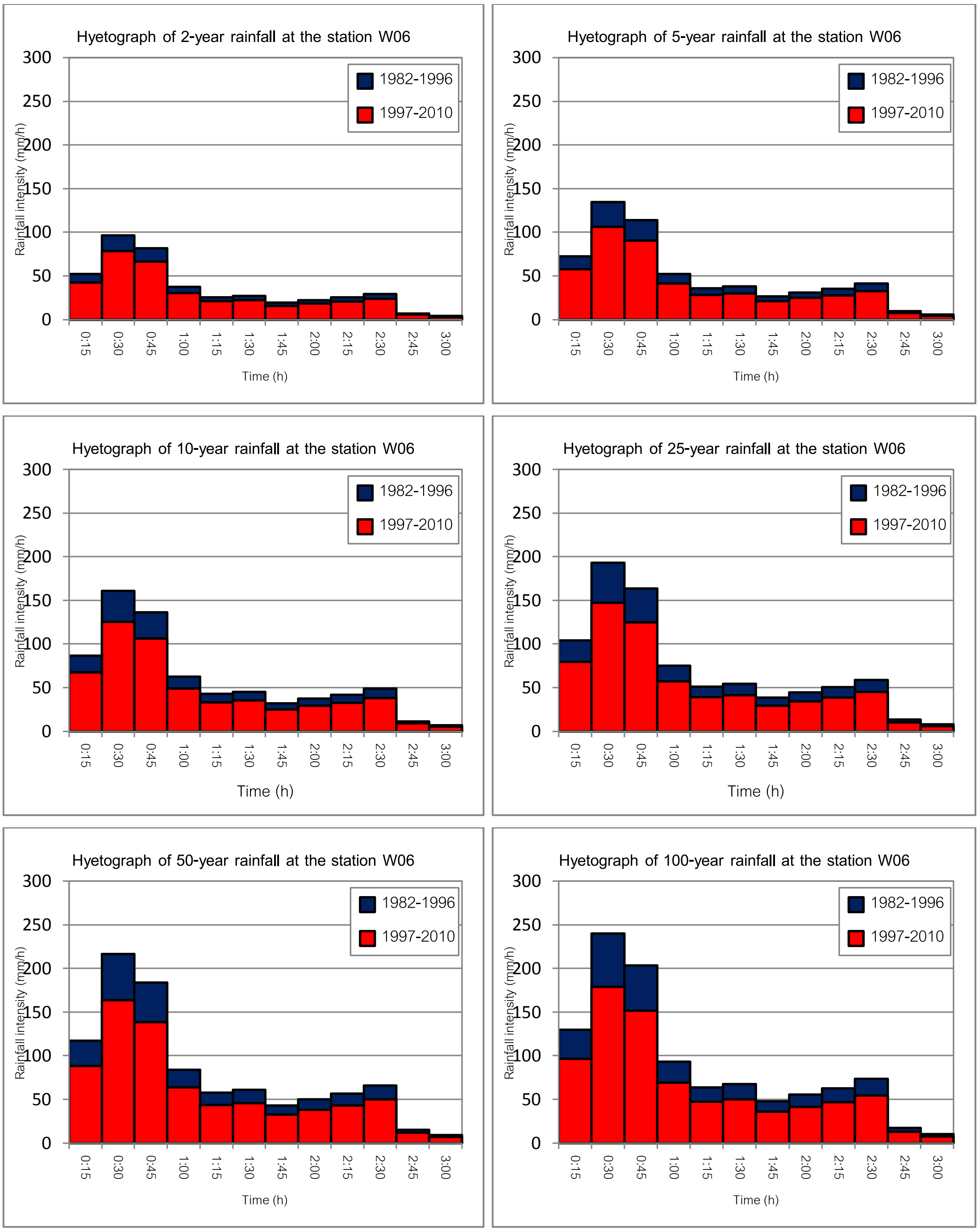


Figure 5-8 Design hyetograph at the station W06

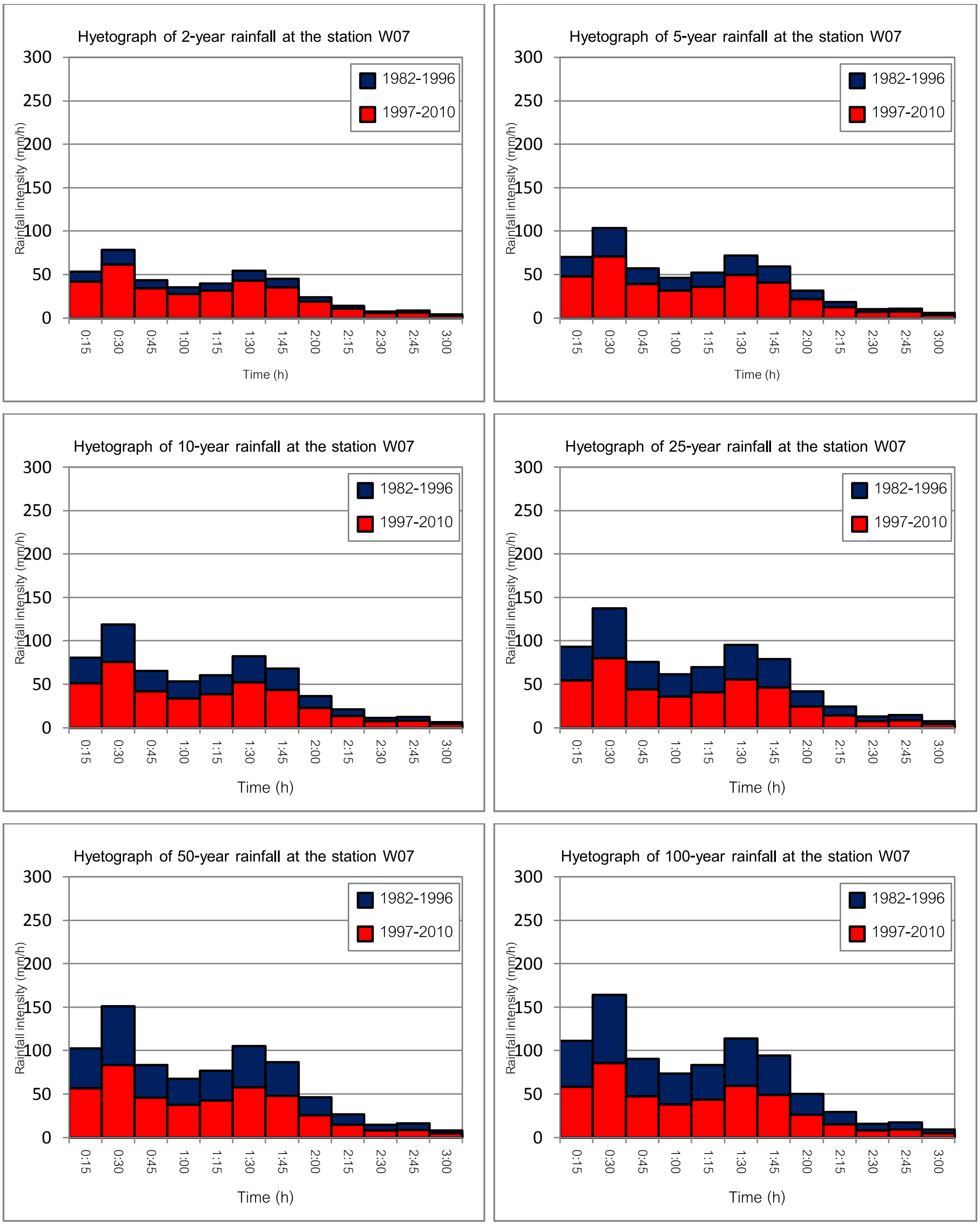


Figure 5-9 Design hyetograph at the station W07

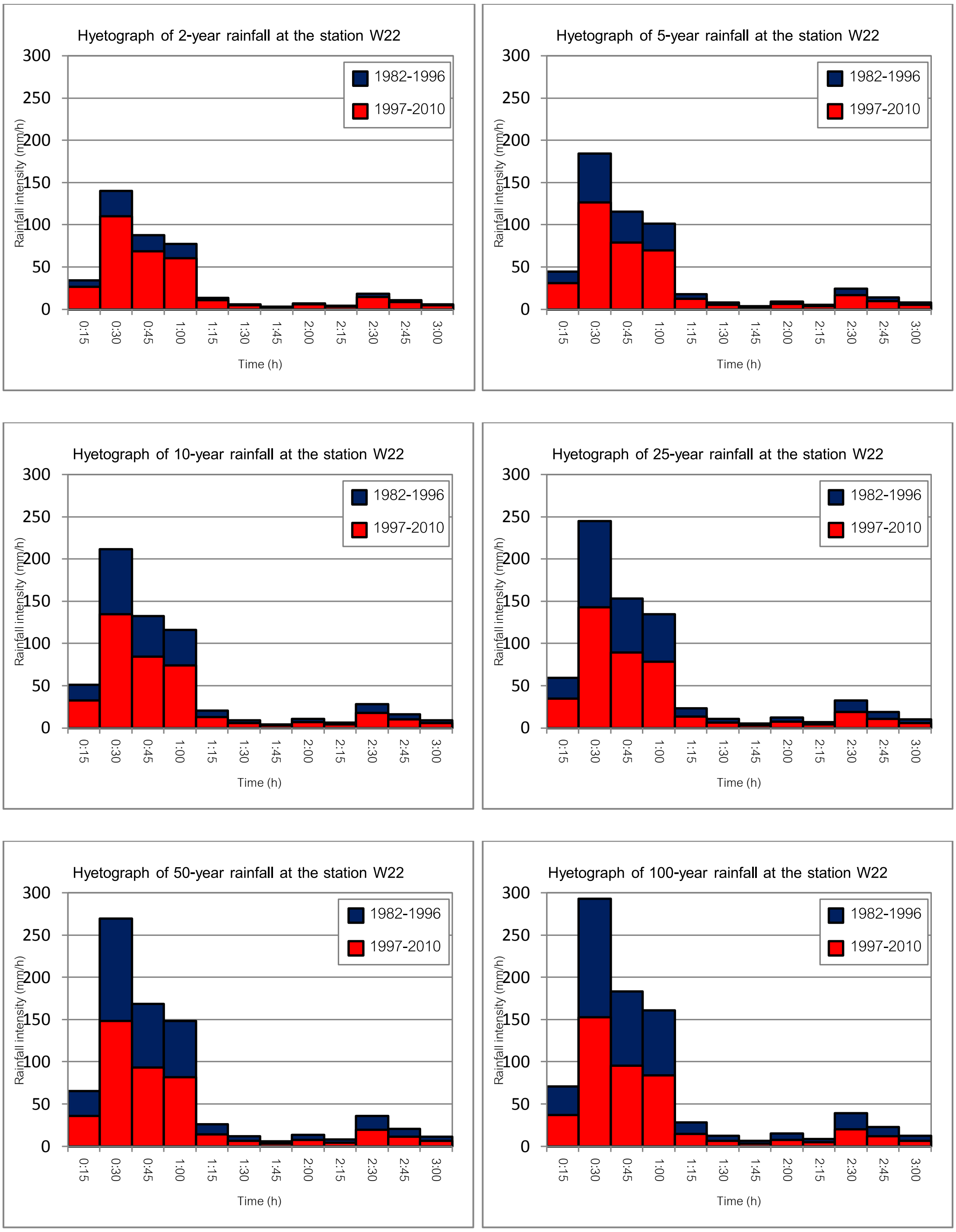


Figure 5-10 Design hyetograph at the station W22

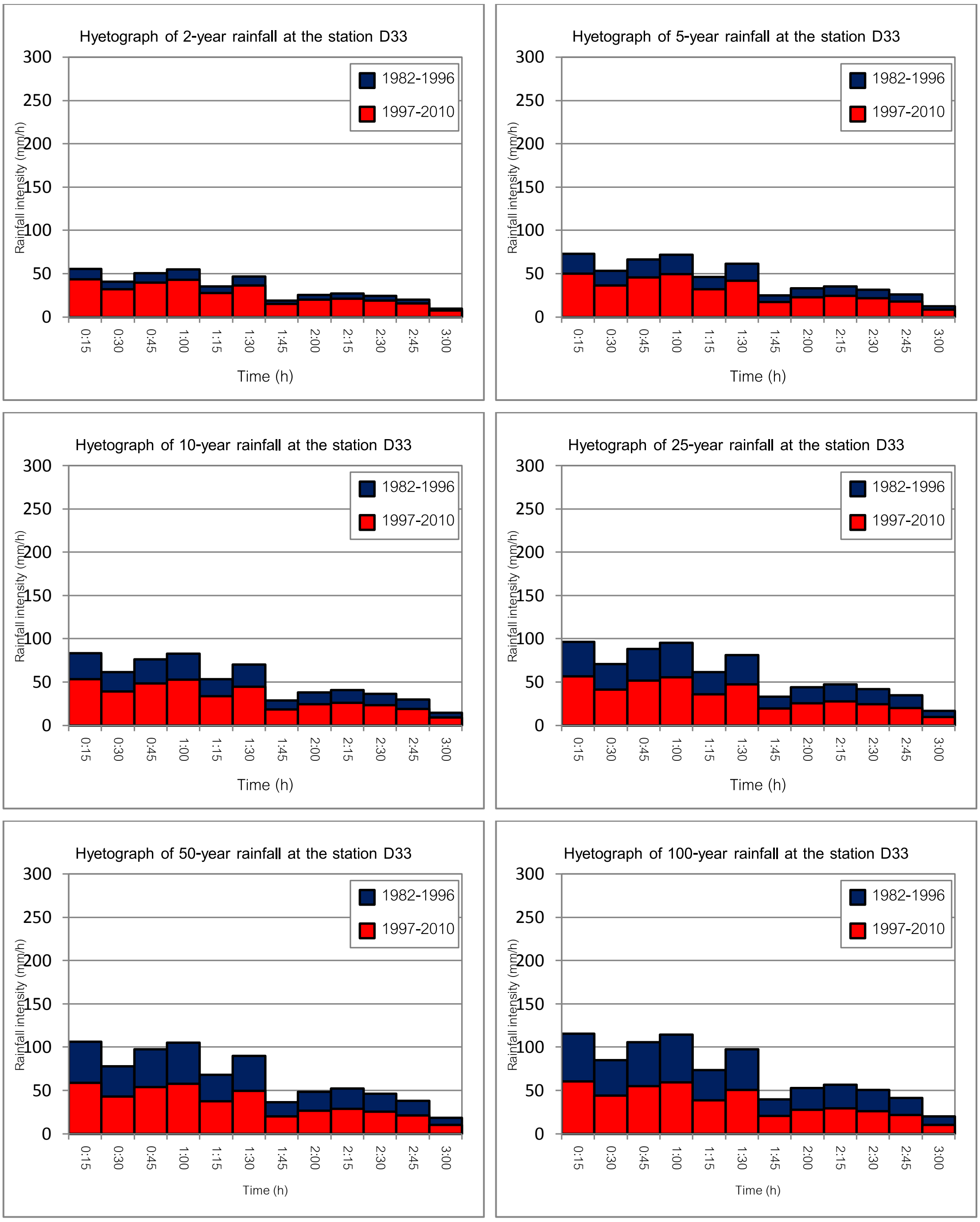


Figure 5-11 Design hyetograph at the station D33

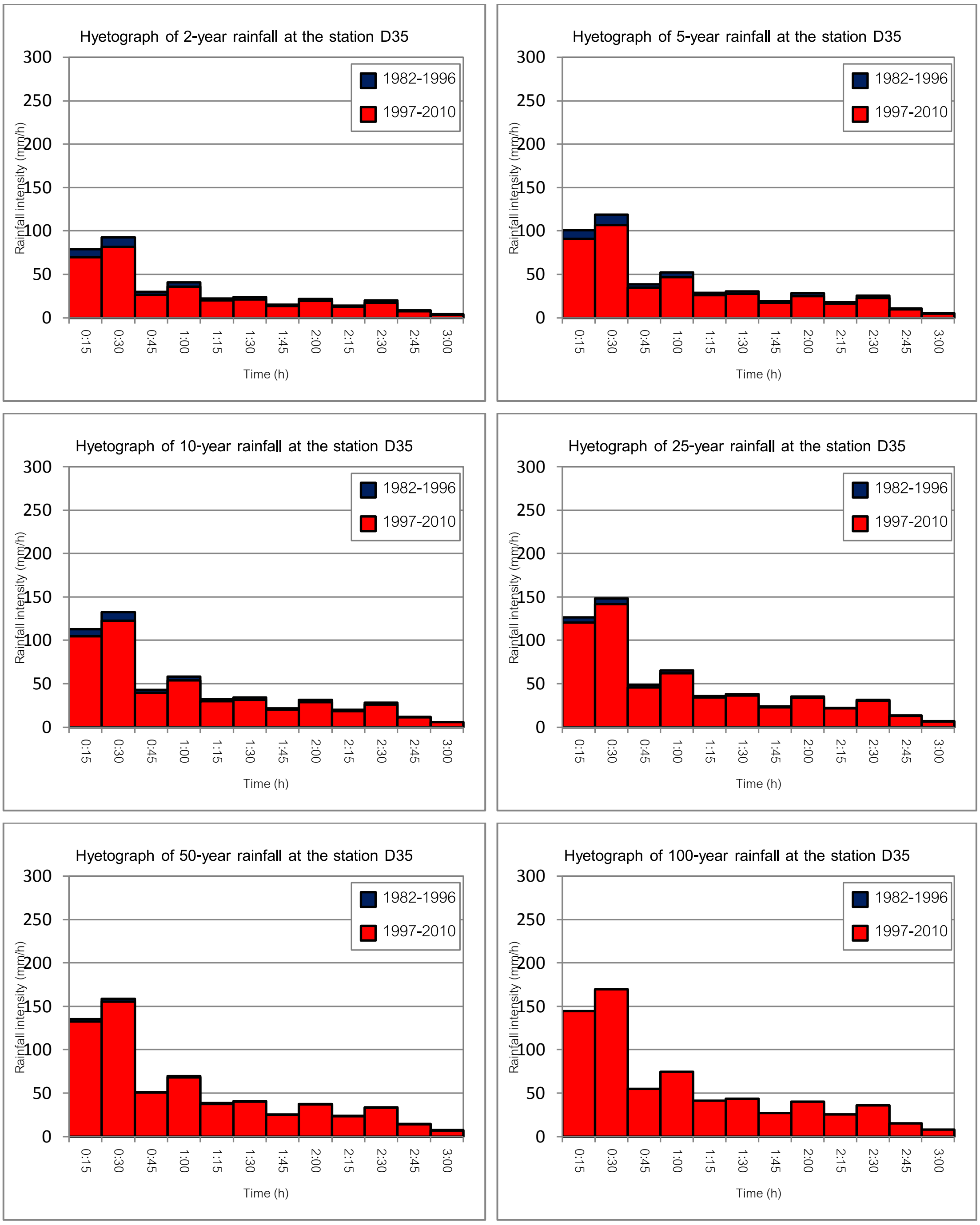


Figure 5-12 Design hyetograph at the station D35

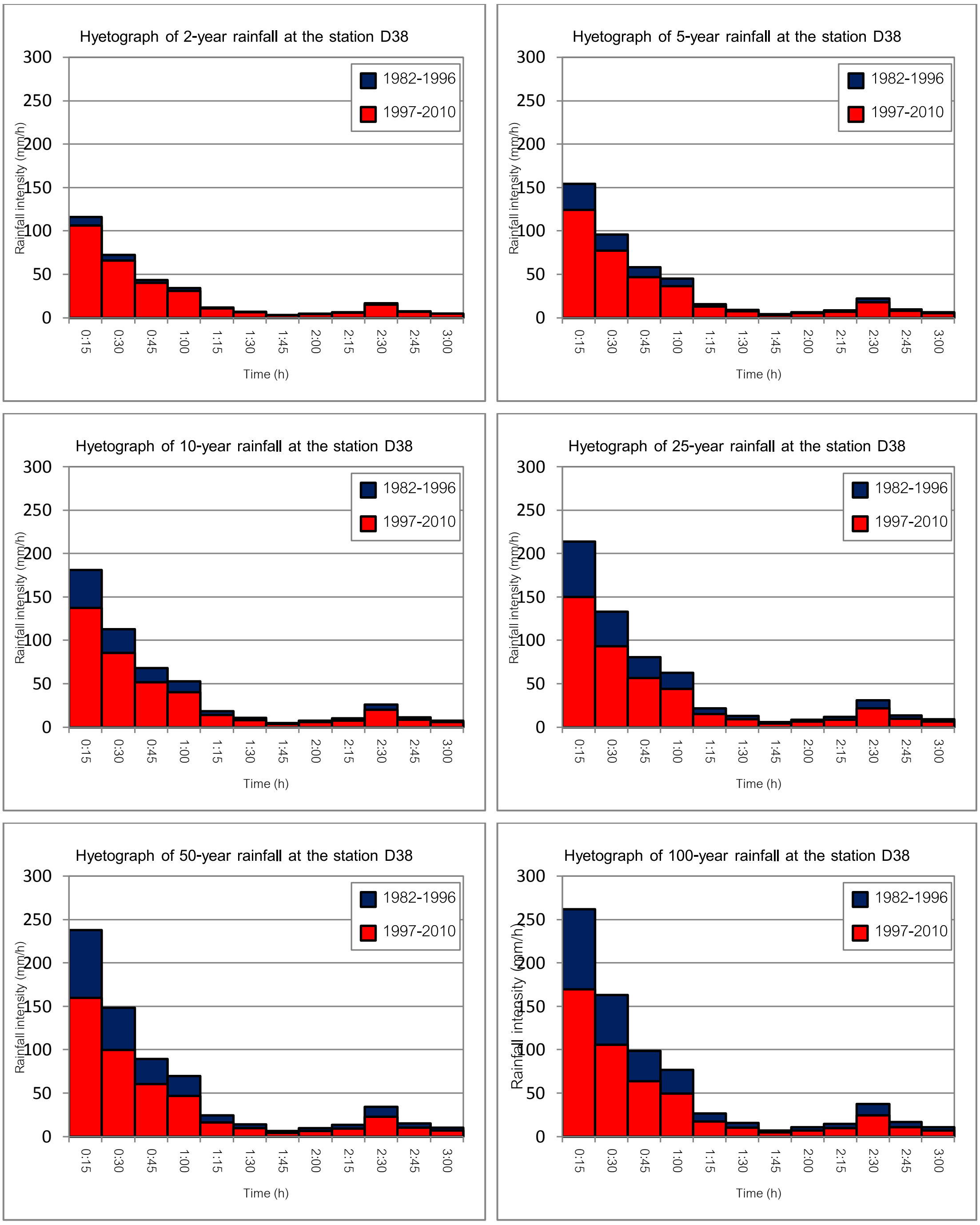


Figure 5-13 Design hyetograph at the station D38

5.3 Model sensitivity analysis

The sensitivity of each parameter in the model was determined using Eq. 3.57. The simulation was run using the rainfall with a return period of 50 years according to the frequency curve in the period of 1982 – 1996. All floodgates were closed to represent the scenario that the water levels outside the gates were high, and the initial water level in the network was set to 0.7 m (MSL) which was believed to be the suitable initial water level for the flood mitigation for this study (the further details is in the section 5.7). The initial values of input parameters were shown in Table 5-9. These values were derived from the calibration results discussed later in the section 5.4.

Table 5-9 Input parameters used in the MIKE 11 model

Input parameter	Channel or area	Value
Manning coefficient	Chao Phraya River	0.020
Manning coefficient	Bangkok Noi Canal	0.033
Manning coefficient	Phasicharoen Canal	0.033
Manning coefficient	Inner main earth canals[a]	0.045
Manning coefficient	Small drainage canals	0.015
Groundwater leakage coefficient	Earth canals	3×10^{-7} s^{-1}
Groundwater leakage coefficient	Paved canals	0 s^{-1}
Maximum water content in surface storage	All catchments	10 mm
Maximum water content in root zone storage	All catchments	100 mm
Overland flow runoff coefficient	Urban catchments	0.85
Overland flow runoff coefficient	Partly-urban catchments	0.75
Overland flow runoff coefficient	Non-urban catchments	0.60
Time constant for routing interflow	All catchments	500 h
Time constant for routing overland flow	Large catchment	6 h
Time constant for routing overland flow	Small catchments	3 h
Rootzone threshold value for overland flow	All catchments	0
Rootzone threshold value for interflow	All catchments	0
Rootzone threshold value for groundwater recharge	All catchments	0
Time constant for routing baseflow	All catchments	2000 h

[a] "Inner main earth canals" refers to the Bangcheuknang, Bangkok Yai, Bangphrom, Bangwak Chakphra, and Mon canals.

In the sensitivity analysis of each parameter, the simulation was performed for 3 times. The first one was when the analyzed parameter was set to a specific value (x_0), the second one was when the value of the analyzed parameter was reduced to x_1 which is two-thirds of x_0, and the third one was when the value of the analyzed parameter was increased to x_2 which is four-thirds of x_0. The output of each simulation was the water level at the point in the Bangkhunnon Canal where there was the maximum flood depth in the simulation. The location of this point is shown in Figure 5-14. Since the value of x_2 is the double of that of x_1, the difference of outputs between when the value of the input parameter was x_1 and x_2 could show how much the value of the output would change when the value of the input parameter was doubled or increased for 100%.

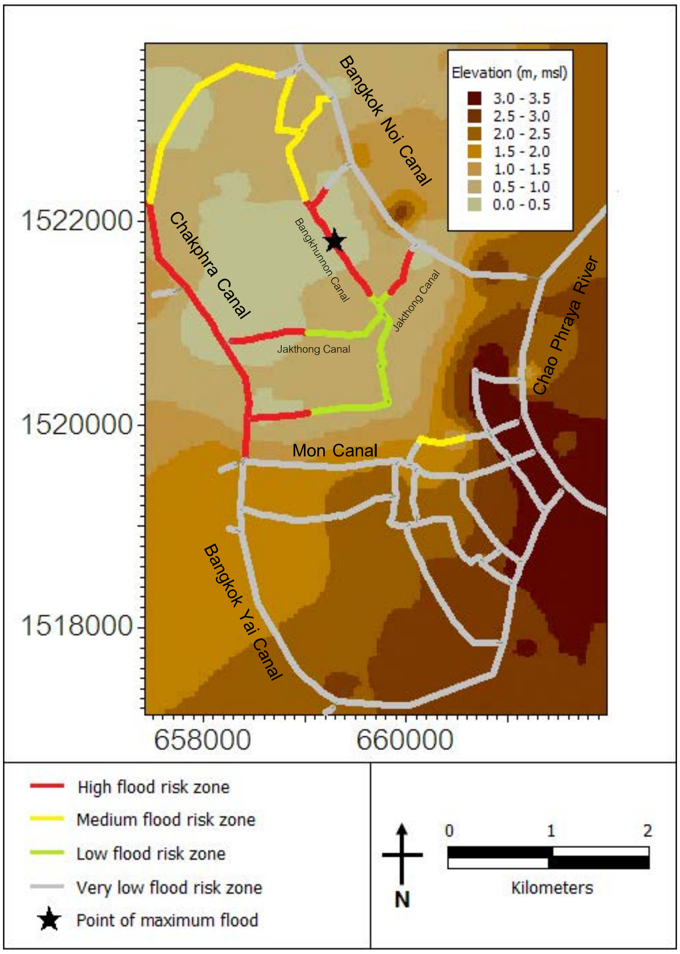


Figure 5-14 Flood prone zones in the simulation

The results of the sensitivity analysis show that the water level is sensitive to parameters involving the overland flow. The overland flow runoff coefficient has a very high sensitivity index. The time constant for routing overland flow has a high sensitivity index. The root zone threshold value for overland flow has a medium sensitivity index. The others parameter have small sensitivity indices. Those parameters include the parameters involving groundwater, interflow, abstraction, and Manning coefficient of the channel. The report from the TEAM in 2003 has also suggested that the water level is not sensitive to the Manning coefficient but sensitive to the physical conditions and to the floodgate and pump operation (บริษัท ทีม คอนซัลติ้ง เอนจิเนียริ่ง แอนด์ แมเนจเมนท์ จำกัด, 2546).

The followings are results from the sensitivity analysis of each parameter:

5.3.1 Manning coefficient

Table 5-10 shows the results of the sensitivity analysis of Manning coefficient.

Table 5-10 Results of the sensitivity analysis of Manning coefficient

n	Input value (x_n)	Output value (y_n)
0	0.03	2.1429 m (MSL)
1	0.02	2.1432 m (MSL)
2	0.04	2.1425 m (MSL)

When the Manning coefficient was increased for 100%, the water level at the Bangkhunnon Canal would decrease for 0.03%. The sensitivity index is 0.00. As a result, the effect of the Manning coefficient is small.

5.3.2 Groundwater leakage coefficient

Table 5-11 shows the results of the sensitivity analysis of the groundwater leakage coefficient.

Table 5-11 Results of the sensitivity analysis of groundwater leakage coefficient

n	Input value (x_n)	Output value (y_n)
0	3×10^{-7} second^{-1}	2.0827 m (MSL)
1	2×10^{-7} second^{-1}	2.1035 m (MSL)
2	4×10^{-7} second^{-1}	2.0620 m (MSL)

When the groundwater leakage coefficient was increased for 100%, the water level at the Bangkhunnon Canal would decrease for 1.97%. The sensitivity index is -0.03. As a result, the effect of the groundwater leakage coefficient is small.

5.3.3 Maximum water content in surface storage

Table 5-12 shows the results of the sensitivity analysis of the maximum water content in surface storage.

Table 5-12 Results of the sensitivity analysis of maximum water content in surface storage

n	Input value (x_n)	Output value (y_n)
0	15 mm	2.1295 m (MSL)
1	10 mm	2.1433 m (MSL)
2	20 mm	2.1156 m (MSL)

When the maximum water content in surface storage was increased for 100%, the water level at the Bangkhunnon Canal would decrease for 1.29%. The sensitivity index is -0.02. As a result, the effect of the maximum water content in surface storage is small.

5.3.4 Maximum water content in root zone storage

Table 5-13 shows the results of the sensitivity analysis of the maximum water content in root zone storage.

Table 5-13 Results of the sensitivity analysis of maximum water content in root zone storage

n	Input value (x_n)	Output value (y_n)
0	150 mm	2.1170 m (MSL)
1	100 mm	2.1433 m (MSL)
2	200 mm	2.1027 m (MSL)

When the maximum water content in root zone storage was increased for 100%, the water level at the Bangkhunnon Canal would decrease for 1.89%. The sensitivity index is -0.03. As a result, the effect of the maximum water content in root zone storage is small.

5.3.5 Overland flow runoff coefficient

Table 5-14 shows the results of the sensitivity analysis of the overland flow runoff coefficient.

Table 5-14 Results of the sensitivity analysis of overland flow runoff coefficient

n	Input value (x_n)	Output value (y_n)
0	0.75	2.0178 m (MSL)
1	0.50	0.8421 m (MSL)
2	1.00	3.4213 m (MSL)

When the overland flow runoff coefficient was increased for 100%, the water level at the Bangkhunnon canal would increase for 306.28%. The sensitivity index is 1.92. As a result, the effect of the overland flow runoff coefficient is very high.

5.3.6 Time constant for routing interflow

Table 5-15 shows the results of the sensitivity analysis of the time constant for routing interflow.

Table 5-15 Results of the sensitivity analysis of time constant for routing interflow

n	Input value (x_n)	Output value (y_n)
0	600 hours	2.1435 m (MSL)
1	400 hours	2.1431 m (MSL)
2	800 hours	2.1436 m (MSL)

When the time constant for routing interflow was increased for 100%, the water level at the Bangkhunnon Canal would increase for 0.02%. The sensitivity index is 0.00. As a result, the effect of the time constant for routing interflow is small.

5.3.7 Time constant for routing overland flow

Table 5-16 shows the results of the sensitivity analysis of the time constant for routing overland flow.

Table 5-16 Results of the sensitivity analysis of time constant for routing overland flow

n	Input value (x_n)	Output value (y_n)
0	3 hours	2.3144 m (MSL)
1	2 hours	2.7585 m (MSL)
2	4 hours	1.8571 m (MSL)

When the time constant for routing overland flow was increased for 100%, the water level at the Bangkhunnon Canal would decrease for 32.68%. The sensitivity index is -0.58. As a result, the effect of the time constant for routing overland flow is high.

5.3.8 Root zone threshold value for overland flow

Table 5-17 shows the results of the sensitivity analysis of the root zone threshold value for overland flow.

Table 5-17 Results of the sensitivity analysis of root zone threshold value for overland flow

n	Input value (x_n)	Output value (y_n)
0	0.3	1.9960 m (MSL)
1	0.2	2.0566 m (MSL)
2	0.4	1.9168 m (MSL)

When the root zone threshold value for overland flow was increased for 100%, the water level at the Bangkhunnon Canal would decrease for 6.8%. The sensitivity index is -0.11. As a result, the effect of the root zone threshold value for overland flow is medium.

5.3.9 Root zone threshold value for interflow

Table 5-18 shows the results of the sensitivity analysis of the root zone threshold value for interflow.

Table 5-18 Results of the sensitivity analysis of root zone threshold value for interflow

n	Input value (x_n)	Output value (y_n)
0	0.3	2.1433 m (MSL)
1	0.2	2.1433 m (MSL)
2	0.4	2.1433 m (MSL)

When the root zone threshold value for interflow was increased for 100%, the water level at the Bangkhunnon Canal would not change. The sensitivity index is 0.00. As a result, the effect of the root zone threshold value for interflow is small.

5.3.10 Root zone threshold value for groundwater recharge

Table 5-19 shows the results of the sensitivity analysis of the root zone threshold value for groundwater recharge.

Table 5-19 Results of the sensitivity analysis of root zone threshold value for groundwater recharge

n	Input value (x_n)	Output value (y_n)
0	0.3	2.1734 m (MSL)
1	0.2	2.1612 m (MSL)
2	0.4	2.1886 m (MSL)

When the root zone threshold value for groundwater recharge was increased for 100%, the water level at the Bangkhunnon Canal would increase for 1.27%. The sensitivity index is 0.02. As a result, the effect of the root zone threshold value for groundwater recharge is small.

5.3.11 Time constant for routing baseflow

Table 5-20 shows the results of the sensitivity analysis of the time constant for routing baseflow.

Table 5-20 Results of the sensitivity analysis of time constant for routing baseflow

n	Input value (x_n)	Output value (y_n)
0	3000 hours	2.1429 m (MSL)
1	2000 hours	2.1433 m (MSL)
2	4000 hours	2.1426 m (MSL)

When the time constant for routing baseflow was increased for 100%, the water level at the Bangkhunnon Canal would decrease for 0.03%. The sensitivity index is 0.00. As a result, the effect of the time constant for routing baseflow is small.

5.3.12 Initial Overland flow

Table 5-21 shows the results of the sensitivity analysis of the total initial overland flow from every basin.

Table 5-21 Results of the sensitivity analysis of initial overland flow

n	Input value (x_n)	Output value (y_n)
0	150,000 m^3 per day	2.2472 m (MSL)
1	100,000 m^3 per day	2.2130 m (MSL)
2	200,000 m^3 per day	2.2667 m (MSL)

When the time initial overland flow was increased for 100%, the water level at the Bangkhunnon Canal would increase for 2.43%. The sensitivity index is 0.04. As a result, the effect of the initial overland flow is small.

5.3.13 Upstream Discharge

Table 5-22 shows the results of the sensitivity analysis of the total discharge from upstream boundaries.

Table 5-22 Results of the sensitivity analysis of upstream discharge

n	Input value (x_n)	Output value (y_n)
0	15 m^3 per second	2.4891 m (MSL)
1	10 m^3 per second	2.3702 m (MSL)
2	20 m^3 per second	2.6093 m (MSL)

When the upstream discharge was increased for 100%, the water level at the Bangkhunnon Canal would increase for 10.09%. The sensitivity index is 0.14. As a result, the effect of the upstream discharge is medium.

5.3.13 Pump starting time

Table 5-23 shows the results of the sensitivity analysis of the pump starting time.

Table 5-23 Results of the sensitivity analysis of pump starting time

n	Input value (x_n)	Output value (y_n)
0	15 minutes	2.1483 m (MSL)
1	10 minutes	2.0943 m (MSL)
2	20 minutes	2.1914 m (MSL)

When the pump starting time was increased for 100%, the water level at the Bangkhunnon Canal would increase for 4.64%. The sensitivity index is 0.07. As a result, the effect of the pump starting time is medium.

5.4 Model calibration and verification

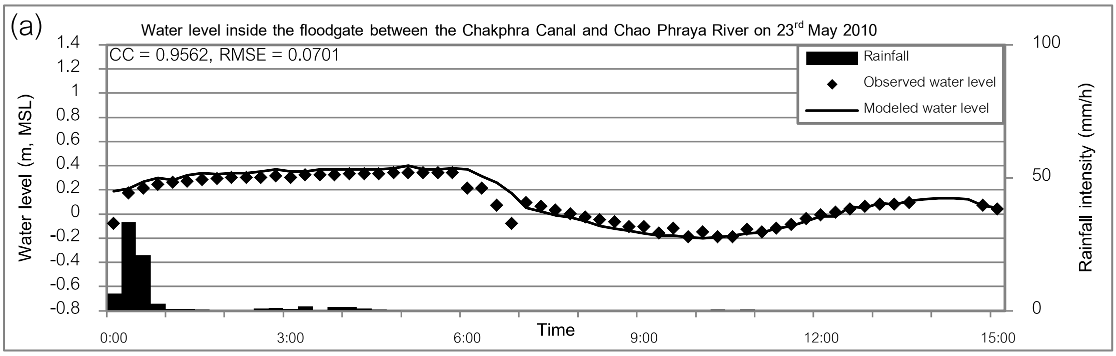
Six rainfall events during the period of May – July 2010 which occurred when the floodgates were closed were chosen for the calibration and verification. These 6 events were on 23rd May, 24th – 25th May, 8th – 9th June, 14th – 15th June, 16th July, and 26th – 27th July 2010. The first 3 ones were used for the calibration, and the others were used for the verification. The boundary conditions in these events were shown in the Appendix G, the rainfall data were shown in the Appendix H, and the floodgate and pump operation were shown in the Appendix I. The water levels inside the Chakphra Canal floodgate (W03), Mon Canal floodgate (W04) and Bangkok Yai Canal floodgate (W06) were simulated and then the observed and modeled water levels were compared.

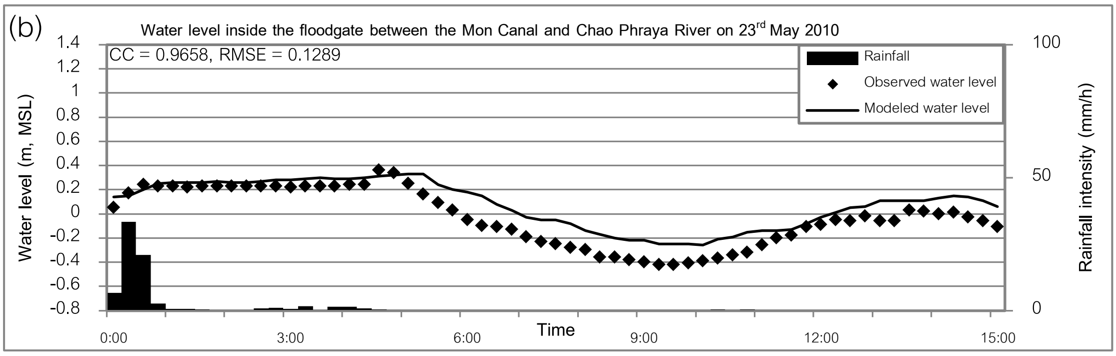
The values of the calibrated parameters are shown in Table 5-9. Most of them are in the range mentioned in the section 3.5 except the Manning coefficient of the Chao Phraya River which is 0.02, and the overland flow runoff coefficient of the non-urban catchment which is 0.6. However, the Manning coefficient is as same as that suggested by the public domain model for the Chao Phraya River (ธีรพล เจริญสุข และหรรษา วัฒนานุกิจ, 2553). About the runoff coefficient of the area with non-urban characteristics, there are also some residential areas and concrete roads in the non-urban catchment which causes the runoff coefficient being higher than that of the full agricultural land. The runoff coefficient of Bangkok depends on the area such as 0.4 for the Bangkapi and Buengkhum districts (Wongwiwat, 2005), 0.65 for most part of the eastern part of the Chao Phraya River, and 0.98 for the area of Suvarnabhumi Airport (Sakol, 2010).

For the verification, the simulated water levels were obtained, and the root mean square error and correlation coefficient were determined (see the section 3.6) The results from simulations for both the calibration and verification are as the followings:

5.4.1 Calibration with the event on 23rd May 2010

Figure 5-15 shows the result of the calibration with the event on 23th May 2010. The overall correlation coefficient (CC) is 0.9466 and the overall root mean square error (RMSE) is 0.1042 meters.





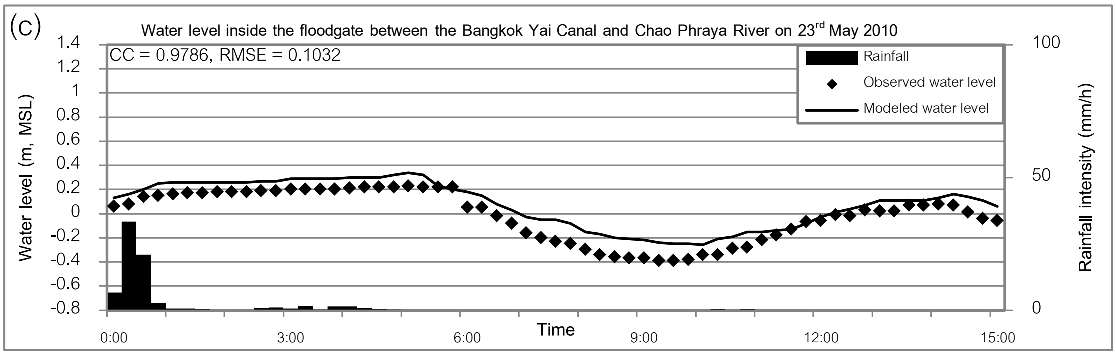


Figure 5-15 Water levels inside floodgates at the stations (a)W03, (b)W04, and (c)W06 on 23th May 2010

5.4.2 Calibration with the event on 24th – 25th May 2010

Figure 5-16 shows the result of the calibration with the event on 24th – 25th May 2010. The overall correlation coefficient (CC) is 0.9790 and the overall root mean square error (RMSE) is 0.1154.

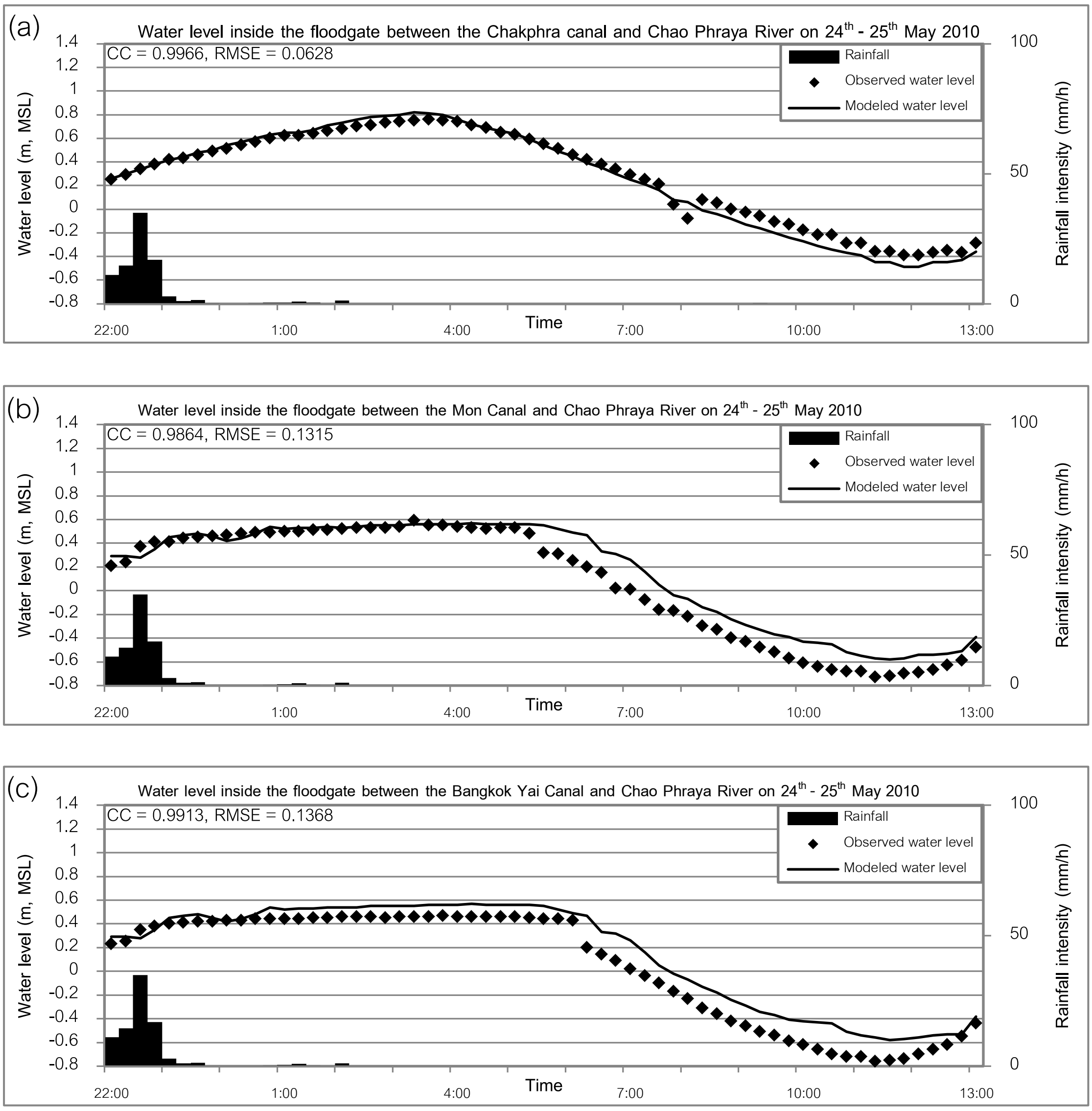


Figure 5-16 Water levels inside floodgates at the stations (a)W03, (b)W04, and (c)W06 on 24th – 25th May 2010

5.4.3 Calibration with the event on 8th – 9th June 2010

Figure 5-17 shows the result of the calibration with the event on 8th – 9th June 2010. The overall correlation coefficient (CC) is 0.9510 and the overall root mean square error (RMSE) is 0.1502.

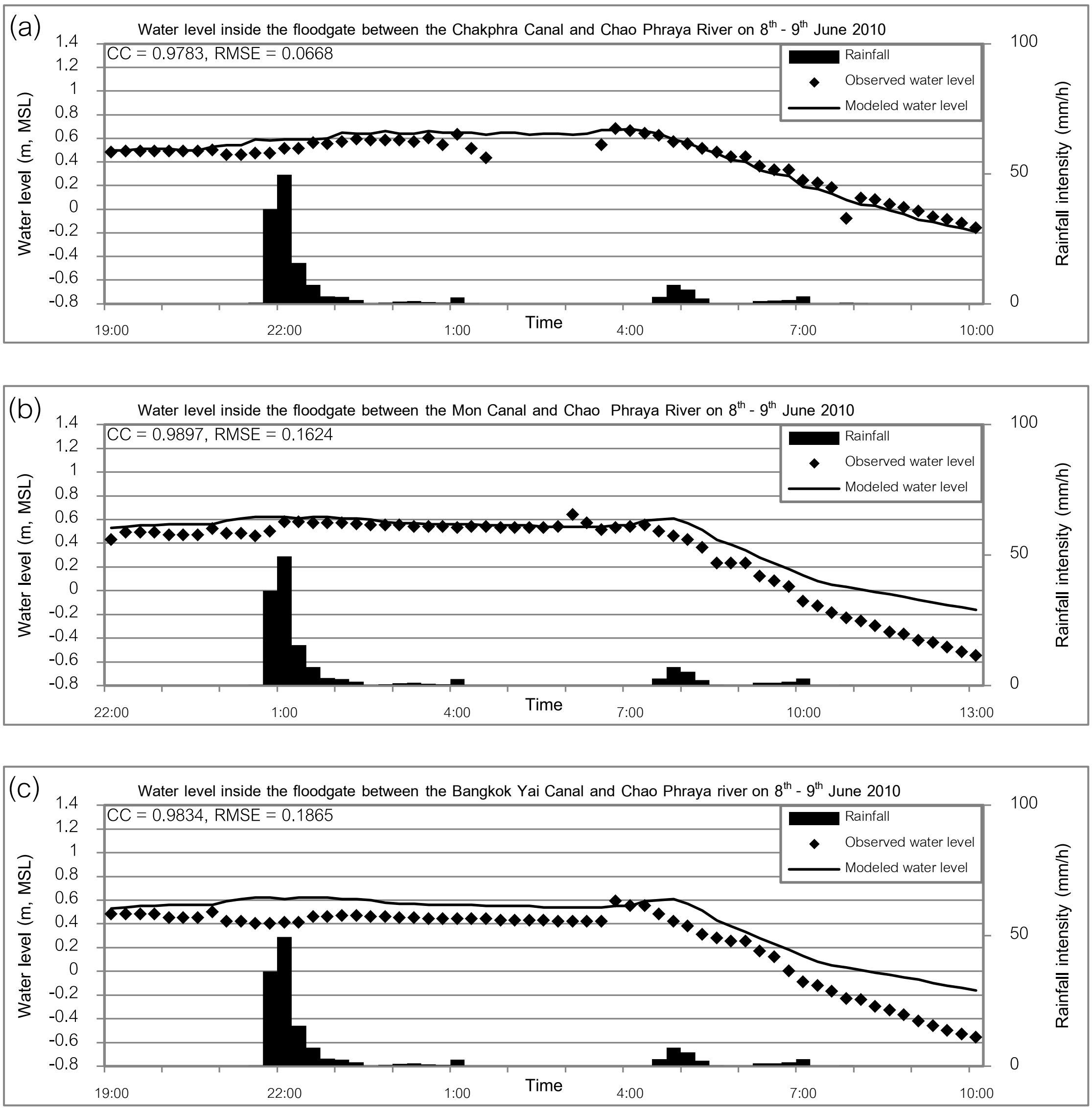
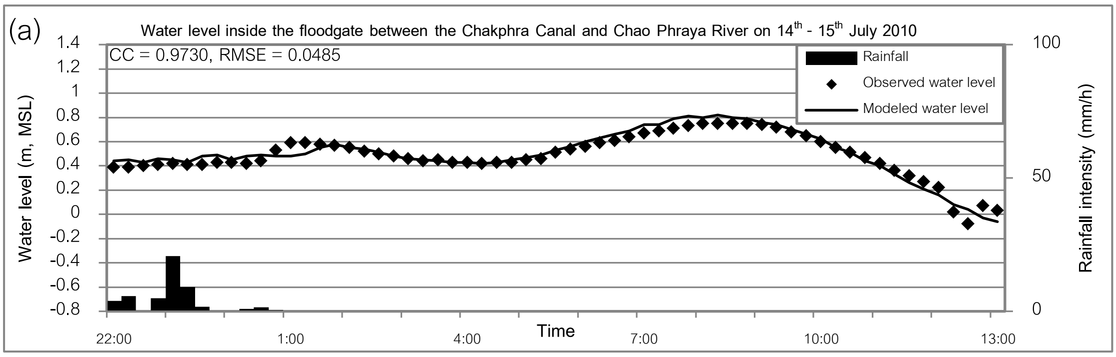


Figure 5-17 Water levels inside floodgates at the stations (a)W03, (b)W04, and (c)W06 on 8th - 9th June 2010

5.4.4 Verification with the event on 14th – 15th July 2010

Figure 5-18 shows the result of the verification with the event on 14th – 15th July 2010. The overall correlation coefficient (CC) is 0.9354 and the overall root mean square error (RMSE) is 0.1220. However, the water level data inside the floodgate between the Mon Canal and Chao Phraya River (W04) was abnormally low and considered unreliable at this verification period, so it was excluded from the verification.



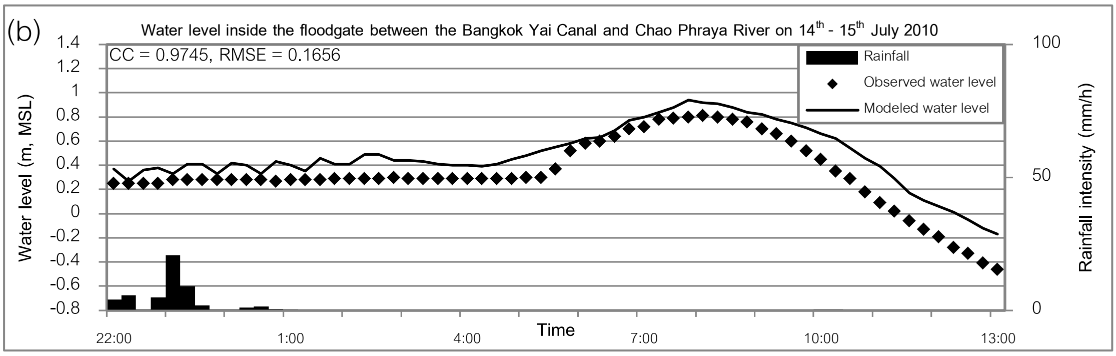
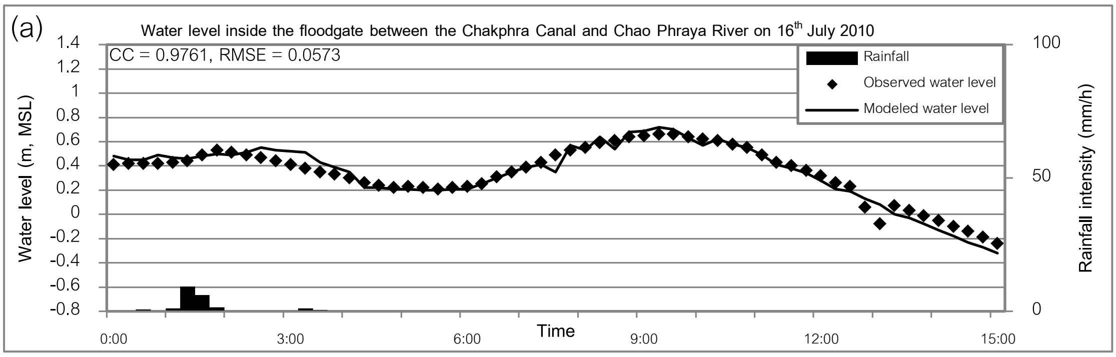


Figure 5-18 Water levels inside floodgates at the stations (a)W03, and (b)W06 on 14th – 15th July 2010

5.4.5 Verification with the event on 16th July 2010

Figure 5-19 shows the result of the verification with the event on 16th July 2010. The overall correlation coefficient (CC) is 0.9477 and the overall root mean square error (RMSE) is 0.1102. However, the water level data inside the floodgate between the Mon Canal and Chao Phraya River (W04) was abnormally low and considered unreliable at this verification period, so it was excluded from the verification.



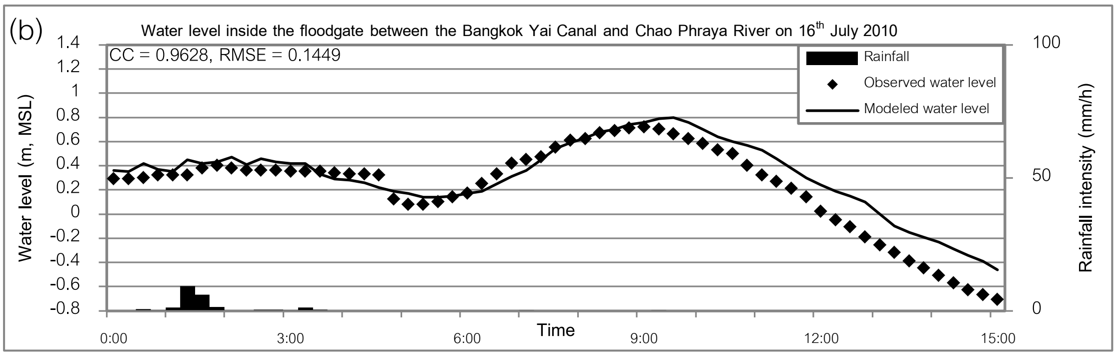
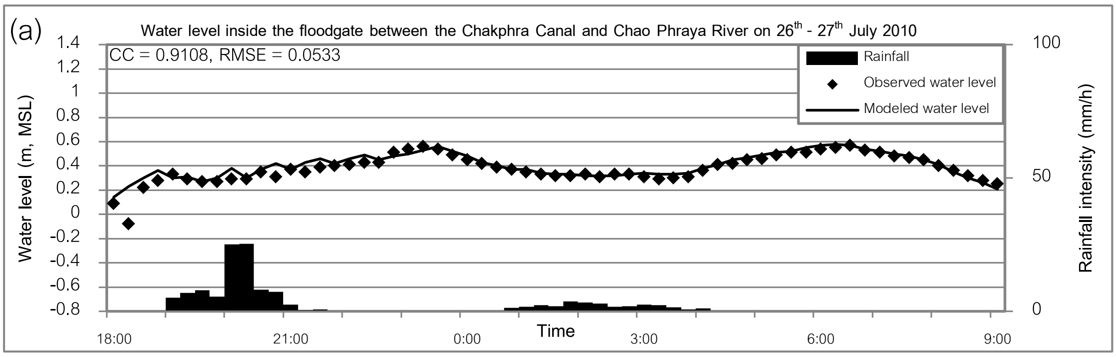


Figure 5-19 Water levels inside floodgates at the stations (a)W03, and (b)W06 on 16th July 2010

5.4.6 Verification with the event on 26th – 27th July 2010

Figure 5-20 shows the result of the verification with the event on 26th – 27th July 2010. The overall correlation coefficient (CC) is 0.8679 and the overall root mean square error (RMSE) is 0.0898 meters. However, the water level data inside the floodgate between the Mon Canal and Chao Phraya River (W04) was abnormally low and considered unreliable at this verification period, so it was excluded from the verification.



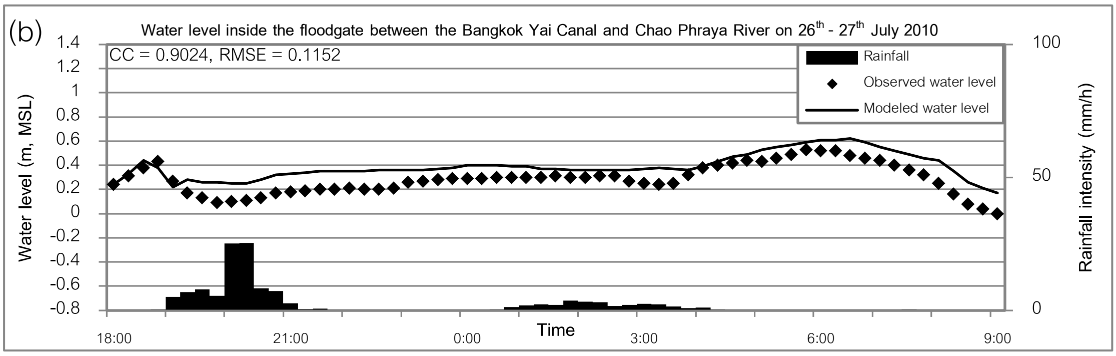


Figure 5-20 Water levels inside floodgates at the stations (a)W03, and (b)W06 on 26th – 27th July 2010

With the exception of the unreliable or error observed data, the overall CC from all events is 0.9634, and the overall RMSE from all events is 0.1182 meters.

5.5 Uncertainty analysis

For all events mentioned in the section 5.4, the errors were calculated using Eq. 3.60. The mean error was -0.07 meters and the standard deviation of the errors was 0.09 meters. With an assumption that the errors were normally distributed, the 95% confidence interval for the errors was determined and the result shows that the upper and lower boundary of the interval is 0.11 meters and -0.25 meters, respectively. Hence, with the 95% confidence interval, the observed water level can range from 0.25 meters lower to 0.11 meters higher than the modeled water level.

5.6 Water level under different return periods of rainfall

The MIKE 11model was applied to determine the maximum water levels and floods, which were defined as the elevation heads above the bank height of the channel, in the canals due to different return periods of rainfall, different frequency curves mentioned in the section 4.5.2, and different pump usages. The pump starting time was assumed to be 15 minutes. All floodgates were closed to represent the scenario that the water levels outside the gates were high, and the initial water level in the network was set to 0.7 m (MSL) which was believed to be the suitable initial water level for the flood mitigation for this study (the further details is in the section 5.7). Tables 5-24 and 5-25 show the maximum water level and flood, respectively, in the Bangkhunnon Canal where the maximum flood usually occurred in the simulations. Blank cells in these tables represent the cases the simulation could not be carried out because the water level goes lower than the canal bed which is resulted from too high pumping. Water level time series were shown in the Appendix J.

Table 5-24 Maximum water levels (m, MSL) at the Bangkhunnon Canal when an initial water level is 0.7 m (MSL)

Return period	Period of study	Pump usages							
		50%	60%	70%	80%	90%	100%	110%	120%
2-year	1982-1996	1.15	0.89	0.68	0.53	0.39			
	1997-2010	0.76	0.55	0.40	0.25				
5-year	1982-1996	1.98	1.66	1.38	1.14	0.95	0.82		
	1997-2010	1.22	0.96	0.74	0.59	0.44			
10-year	1982-1996	2.54	2.20	1.89	1.61	1.37	1.18		
	1997-2010	1.54	1.25	1.00	0.81	0.65			
25-year	1982-1996	3.24	2.89	2.56	2.25	1.96	1.71	1.50	
	1997-2010	1.92	1.60	1.32	1.09	0.91	0.78		
50-year	1982-1996	3.77	3.40	3.06	2.73	2.42	2.14	1.90	1.69
	1997-2010	2.20	1.87	1.57	1.31	1.10	0.96		
100-year	1982-1996	4.27	3.90	3.55	3.22	2.90	2.60	2.32	2.08
	1997-2010	2.46	2.13	1.82	1.53	1.30	1.13	1.00	

Table 5-25 Maximum floods (m) at the Bangkhunnon Canal when an initial water level is 0.7 m (MSL)

Return period	Period of study	Pump usages							
		50%	60%	70%	80%	90%	100%	110%	120%
2-year	1982-1996	1.08	0.82	0.61	0.46	0.32			
	1997-2010	0.69	0.48	0.33	0.18				
5-year	1982-1996	1.91	1.59	1.31	1.07	0.88	0.75		
	1997-2010	1.15	0.89	0.67	0.52	0.37			
10-year	1982-1996	2.47	2.13	1.82	1.54	1.30	1.11		
	1997-2010	1.47	1.18	0.93	0.74	0.58			
25-year	1982-1996	3.17	2.82	2.49	2.18	1.89	1.64	1.43	
	1997-2010	1.85	1.53	1.25	1.02	0.84	0.71		
50-year	1982-1996	3.70	3.33	2.99	2.66	2.35	2.07	1.83	1.62
	1997-2010	2.13	1.80	1.50	1.24	1.03	0.89		
100-year	1982-1996	4.21	3.83	3.48	3.15	2.83	2.53	2.25	2.01
	1997-2010	2.39	2.06	1.75	1.46	1.23	1.06	0.93	

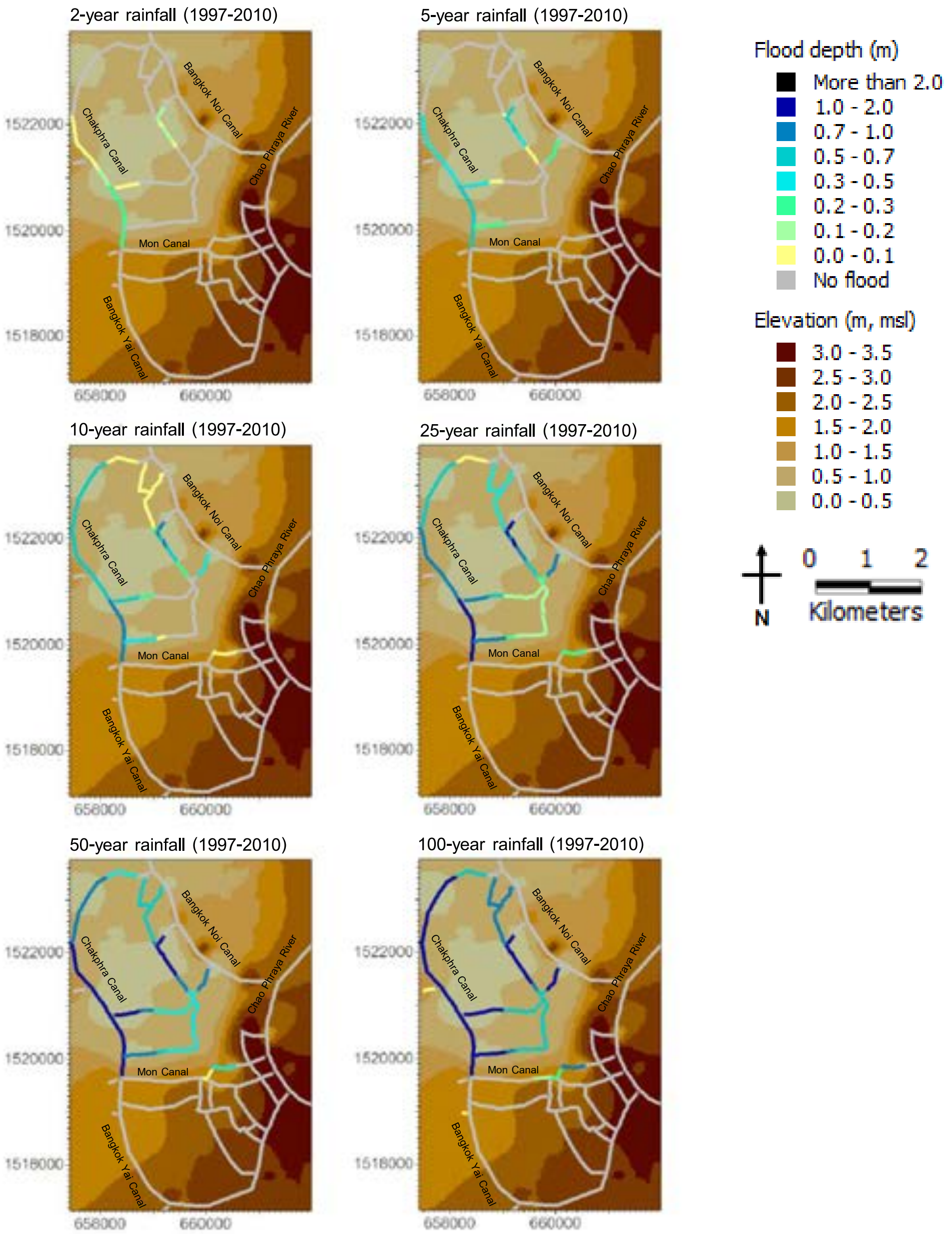


Figure 5-21 Simulation-derived flood map when the pump usage was at 80% maximum capacity

Flooded zones in the channel network due to various return periods of rainfall and pump usages are shown in the Appendix K. Figure 5-21 shows the flood in the case that the pump usage is at 80% of the capacity. With this pump usage, the inundated zones when 5-year rainfall occurs are along the Chakphra and Bangkhunnon canals and the eastern part of the Jakthong Canal. This study defines these zones as the high flood risk zones. Moreover, most of these zones are inundated even when 2-year rainfall occurs. When 10-year rainfall occurs, the additional inundated zones are along the Jaoarm and Pawana canals and the small middle part of the Mon Canal. These zones are defined as the medium flood risk zones. The zones defined as low flood risk zones are the additional inundated zones when 25-year rainfall occurs which are along the Jakthong, Wat Yangsuttharam, Dongmullek and Krathontaew canals. Other zones do not have the problem of flood even when 100-year rainfall occurs. They are defined as very low flood risk zones. Locations of high, medium, low and very low flood risk zones are shown in Figure 5-14.

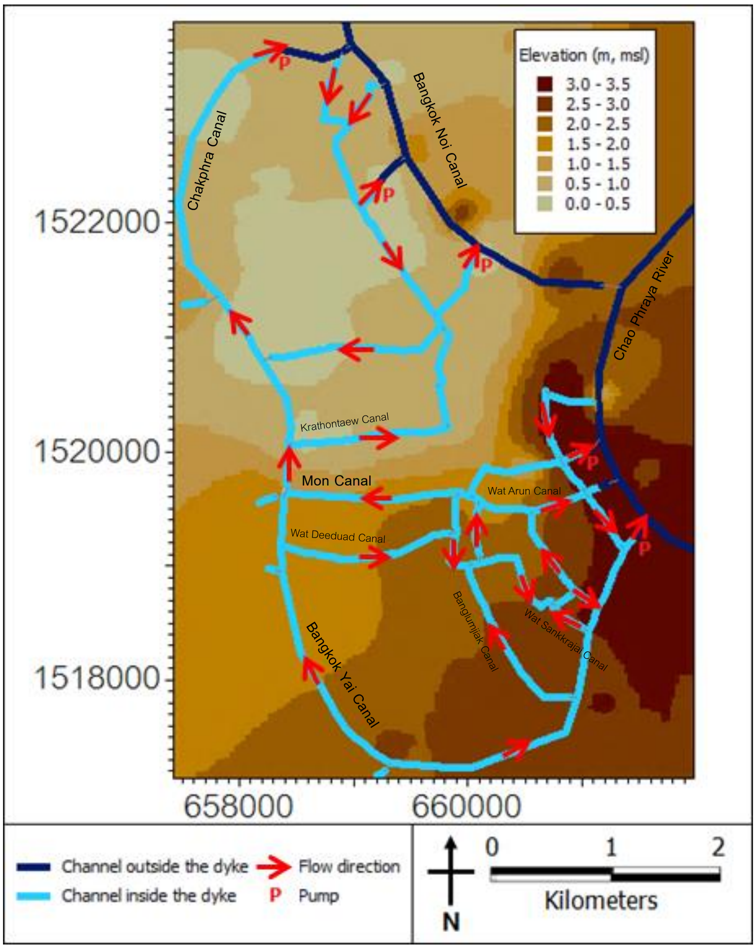


Figure 5-22 Flow in the canal network during the simulated rainfall events

Figure 5-22 shows the flow in the canal network. Normally, the water flows from small drainage canals to main canals and evacuate to the Bangkok Noi Canal or Chao Phraya River. The exceptions where the water flows back to drainage canals are the Krathontaew Canal where the water from the Chakphra Canal flows to, the Wat Arun Canal where the water from the Mon Canal flows to, and the Wat Deeduad, Banglumjiak, and Wat Sankkrajai canals where the water from the Bangkok Yai Canal flows to.

5.7 Mitigation measures

This study focused on the high flood risk zones since they were much more sensitive to flood than others area. However, along the Chakphra Canal, most of the area was non-urban area. As a result, the flood was allowed to happen there for this study, so the high flood risk zones that need to have the mitigation were only along the Bangkhunnon Canal and the eastern part of the Jakthong Canal.

Four mitigations were simulated in the MIKE 11 model. They were building flood dykes, controlling the initial water level before the rainfall, pumping, and applying the floodgate in the point of a canal where the water flew backwards to the flood risk area. However, the first 3 measures could be adopted with one another to obtain the optimal mitigation.

5.7.1 Using dykes, pumping, and controlling initial water level

Since most of the area considered medium flood risk for this study have the bank elevation of not less than 0.75 m (MSL), it is suggested that there should be dykes with a height of 0.75 m (MSL) along the Bangkhunnon Canal and the eastern part of the Jakthong Canal and the maximum water level should be controlled not to exceed 0.75 m (MSL).

The simulation was done for different pump usages and the initial water level was adjusted with the step of 0.05 m to determine the optimum one which was the highest one that did not cause flood. The results are shown in Table 5-26.

Table 5-26 Optimum initial water levels (m, MSL) for the flood mitigation

Return period	Period of study	Pump usage							
		50%	60%	70%	80%	90%	100%	110%	120%
2-year	1982-1996	0.25	0.55	0.70	0.70	0.70			
	1997-2010	0.65	0.70	0.70	0.70				
5-year	1982-1996	-0.55[b]	-0.25[b]	0.00[a]	0.25	0.40[a]	0.60[a]		
	1997-2010	0.20	0.45	0.70	0.70	0.70			
10-year	1982-1996	-1.10[b]	-0.75[b]	-0.45[b]	-0.20[b]	0.05[b]	0.25[b]		
	1997-2010	-0.15	0.15	0.40	0.60	0.70			
25-year	1982-1996	-1.80[b]	-1.45[b]	-1.15[b]	-0.80[b]	-0.55[b]	-0.30[b]	-0.05[b]	
	1997-2010	-0.50[b]	-0.20[a]	0.05	0.30	0.45	0.65		
50-year	1982-1996	-2.35[b]	-0.95[b]	-1.30[b]	-1.00[b]	-0.70[b]	-0.45[b]	-0.25[b]	
	1997-2010	-0.75[b]	-0.45[b]	-0.15[b]	0.10[b]	0.30[b]	0.45[b]		
100-year	1982-1996	-2.85[b]	-2.45[b]	-2.10[b]	-1.80[b]	-1.45[b]	-1.15[b]	-0.90[b]	-0.65[b]
	1997-2010	-1.05[b]	-0.70[b]	-0.40[b]	-0.10[b]	0.15[b]	0.30[b]	0.45[b]	

[a] Values marked by [a] were obtained from the simulation with the higher initial water level than that value in the table for 0.05 m and that simulation caused the flood with the depth of less than 0.05 m.

[b] Values marked by [b] were obtained from differences between flood depth in Table 5-24 and the bank elevation. They were considered to have low reliabilities.

According to Table 5-26, floods from 2-year, 5-year and 10-year rainfalls can be relieved by the dykes, the initial water level of 0.70 m (MSL), and the current pump. The flood from 25-year rainfall cannot be relieved with the current pump unless the initial water level is reduced to 0.65 m (MSL) or lower. For 50-

year and 100-year rainfalls, the lower initial water level or more pumps are required.

5.7.2 Applying floodgates

Since along the Bangkhunnon Canal, the high flood risk zone, the water flows from the Pawana and Wat Yangsuttharam canals in the north and the south, respectively, while it can evacuate the area by the Jakthong Canal in the west, building the floodgate between the Wat Yangsuttharam and Jakthong canals to prevent the flow as shown in Figure 5-23 may reduce the flood peak.

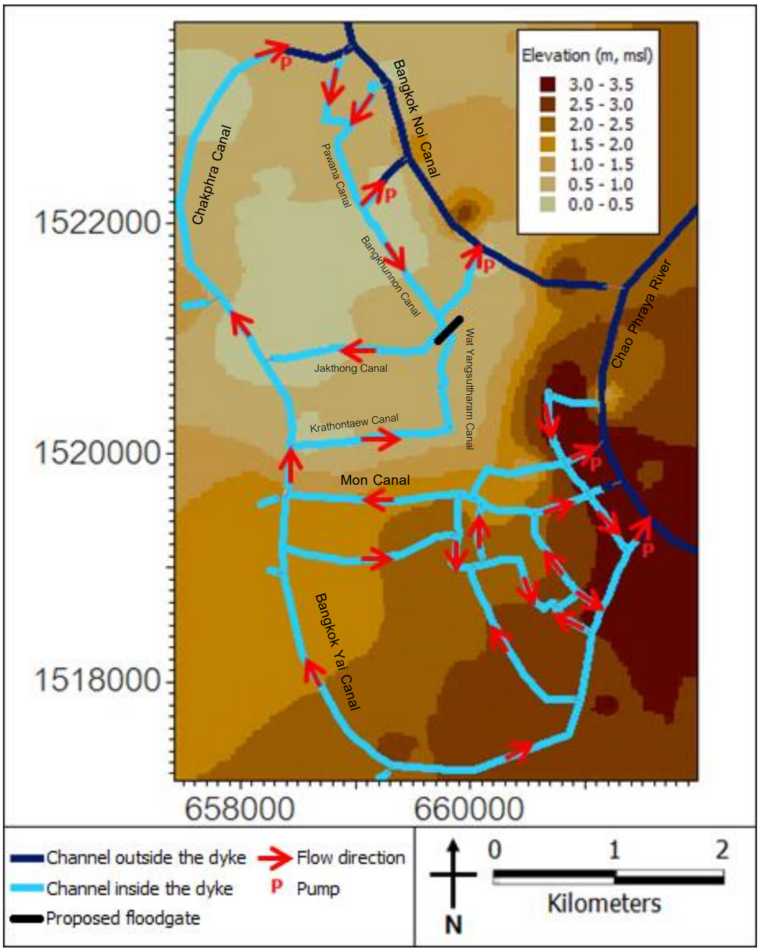


Figure 5-23 Location of the proposed floodgate

Water levels at the Bangkhunnon Canal due to different return periods of rainfall in the period of 1997-2010, and different pump usages were simulated with this floodgate. The result is shown in Table 5-27. However, in some cases, high pumping caused the water level going lower than the bed of the canal, so the simulation could not be carried out. Those cases are represented by blank cells in the table.

Table 5-27 Maximum water levels (m, MSL) at the Bangkhunnon Canal when an initial water level is 0.7 m (MSL) with and without the floodgate between the Wat Yangsuttharam Canal and Jakthong Canal applied

Return period	Proposed Floodgate	Pump usage						
		50%	60%	70%	80%	90%	100%	110%
2-year	Yes	0.74	0.52	0.35				
	No	0.76	0.55	0.39				
5-year	Yes	1.21	0.92	0.69	0.52	0.38		
	No	1.22	0.96	0.74	0.58	0.43		
10-year	Yes	1.54	1.23	0.95	0.74	0.59		
	No	1.54	1.24	1.00	0.81	0.65		
25-year	Yes	1.91	1.59	1.29	1.02	0.83	0.69	
	No	1.92	1.60	1.32	1.09	0.91	0.76	
50-year	Yes	2.20	1.86	1.55	1.26	1.02	0.87	
	No	2.20	1.87	1.56	1.31	1.10	0.95	
100-year	Yes	2.47	2.12	1.81	1.51	1.23	1.03	0.90
	No	2.46	2.12	1.81	1.53	1.30	1.13	0.99

Water level time series are shown in the Appendix L. From Table 5-27, the proposed floodgate can reduce the maximum flood peak along the Bangkhunnon Canal for up to approximately 0.10 meters. The more capacity the pumping and the shorter the return period the rainfall are, the more this floodgate can reduce the flood peak. With this floodgate, the dykes with a height of 0.75 m (MSL), and initial water level of 0.70 m (MSL), current pumping capacity can prevent this area from flood due to the rainfall with return periods of up to 25 years. However, this floodgate seems not to relieve the flood from 50- and 100-year rainfall well.

5.8 Summary

The MIKE 11 model was developed to study the effect of decreasing maximum 1-day rainfall on maximum water level from the rainfall and to determine the appropriate flood mitigation. The CC of the model is 0.9634 and the RMSE of the model is 0.1182 m. With the confidence interval of 95%, the observed water level can range from 0.25 m lower to 0.11 higher than the modeled water level.

The water levels in channels are sensitive to the parameters involving the overland flow, especially the overland flow runoff coefficient and the time constant for routing overland flow. The overland flow runoff coefficient of the area ranges from 0.60 for the catchment with non-urban characteristics to 0.85 for the catchment with urban characteristics. The time constant for routing overland flow is 3 hours for most of the area except one large catchment with the area of 2.4 km^2 which has the time constant for routing overland flow of 6 hours. The parameters involving the groundwater, interflow, abstraction and Manning coefficient do not have much effect on the water levels.

The maximum water level from the rainfall in the period of 1997 – 2010 is lower than that from the rainfall in the period of 1982 – 1996. For the Bangkhunnon Canal, the high flood risk area, when the pump usage is 50%, the difference will be 0.39, 0.76, 1.00, 1.32, 1.57, 1.81 m for 2-, 5-, 10-, 25-, 50-, and 100-year rainfalls, respectively. When the pump usage is 80%, the difference will be 0.28, 0.55, 0.80, 1.16, 1.42, 1.69 m for 2-, 5-, 10-, 25-, 50-, 100- year rainfalls, respectively.

Given the pump usage of 80% and initial water level of 0.70 m (MSL), the inundated zones when 5-year rainfall occurs are along the Chakphra and Bangkhunnon canals and the eastern part of the Jakthong Canal. These zones are defined as the high flood risk zones. When 10-year rainfall occurs, the additional inundated zones are along the Jaoarm and Pawana canals and the small middle part of the Mon Canal. These zones are defined as the medium flood risk zones. The low flood risk zones are along the Jakthong, Wat Yangsuttharam, Dongmullek and Krathontaew canals where there is the flood when 25-year rainfall occurs. Other zones are the very low flood risk zones where there is no problem of flood even when 100-year rainfall occurs.

During the rainfall events, the water flows from small drainage canals to main canals and evacuate to the Bangkok Noi Canal or Chao Phraya River except for the Krathontaew, Chakphra, Wat Arun, Wat Deeduad, Banglumjiak, and Wat Sankkrajai canals where the water from main canals flows back to drainage canals.

Given the current pumping capacity, building dykes with a height of 0.75 m (MSL) along the Bangkhunnon Canal and the eastern part of the Jakthong Canal and setting an initial water level to 0.70 m (MSL) can mitigate the flood from the rainfall with a return period of 10 years. If the floodgate between the Wat Yangsuttharam and Jakthong canals is built, flood from the rainfall with the return of 25 years can also be mitigated. However, for the rainfall with return periods of 50 and 100 years, more pumping capacity or less initial water level are required.

CHAPTER VI

CONCLUSIONS AND RECOMMENDATIONS

6.1 Conclusions

In inner Bangkok, amount of both the annual and maximum 1-day rainfall in the eastern part of the Chao Phraya River is higher than that in the western part. However, the annual rainfall is increasing in the western part and slightly decreasing in the eastern part while the maximum 1-day rainfall decreases for almost all over the area.

In the Bangkok Noi and Bangkok Yai districts, the annual rainfall is increasing for 0.7 – 11.8 mm per year (0.05 – 0.95 %). The increasing seems to be low in the eastern part and high in the northwestern part of these districts. The maximum 1-day rainfall is decreasing for 0.9 – 2.0 mm per year (0.99 – 2.15 %). The decreasing seems to be low in the northern part and high in the southern part of these districts.

At every return period, the overall maximum 1-day rainfall in the period of 1997 – 2010 has changed from that in the period of 1982 – 1996 in a negative direction. The maximum 1-day rainfall has changed from 79 – 107 mm to 69 – 87 mm for a return period of 2 years, from 104 – 149 mm to 84 – 118 mm for a return period of 5 years, from 122 – 178 mm to 93 – 139 mm for a return period of 10 years, 145 – 214 mm to 104 – 163 mm for a return period of 25 years, from 159 – 240 mm to 108 – 181 mm for a return period of 50 years, and from 169 – 266 mm to 119 – 198 mm for a return period of 100 years.

That decreasing in the maximum 1-day rainfall causes the maximum water level when the rainfall occurs being lower than in the past. The difference in maximum water level is larger for the longer return period rainfall and lower pump usages. For the Bangkhunnon Canal, the most flood risk area, when the pump usage is 50%, the difference will be 0.39, 0.76, 1.00, 1.32, 1.57, 1.81 m for 2-, 5-, 10-, 25-, 50-, and 100-year rainfalls, respectively. When the pump usage is 80%, the difference will be 0.28, 0.55, 0.80, 1.16, 1.42, 1.69 m for 2-, 5-, 10-, 25-, 50-, and 100-year rainfalls, respectively.

In the study area, the high flood risk zones are along the Chakphra and Bangkhunnon canals and the eastern part of the Jakthong Canal. Given the pump usage of 80% and the initial water level of 0.70 m (MSL), these zones have the problem of flood when 5-year rainfall occurs. When 10-year rainfall occurs, the additional inundated zones are along the Jaoarm and Pawana canals and the small middle part of the Mon Canal. These zones are the medium flood risk zone. The low flood risk zones are along the Jakthong, Wat Yangsuttharam, Dongmullek and Krathontaew canals where there is the flood when 25-year rainfall occurs. Other zones are the very low flood risk zones where there is no problem of flood even when 100-year rainfall occurs.

This study recommends that building dykes with a height of 0.75 m (MSL) along the Bangkhunnon Canal and the eastern part of the Jakthong Canal and setting an initial water level to 0.70 m (MSL) combined with 90 % of the current pumping capacity can mitigate the flood from 10-year rainfall. However, for the 25-year rainfall, less initial water level is required. Another interesting mitigation is building a floodgate between the Wat Yangsuttharam and Jakthong canals. With this floodgate, the initial water level of 0.70 m (MSL) and dykes with a height of 0.75 m (MSL) combined with 80% of the current pumping capacity can mitigate the flood from 10-year rainfall. Moreover, full pumping capacity is also enough to prevent this area from flood due to 25-year rainfall. However, for the rainfall with return periods of 50 and 100 years, more pumping capacity or less initial water level are required.

6.2 Recommendations

This study has found that the annual rainfall increases in the outskirt of the inner Bangkok and slightly decreases in the city core and believed that this trend is the result of the expansion of the city. If it is true, others area with high rate of urbanization, especially around the city core, should also have the increasing trend of rainfall. Studying trends of rainfall in other parts of Bangkok is recommended in order to prove this assumption. Mechanisms behind the change of rainfall are also recommended for the further study. Moreover, in some coastal cities such as Taipei (Chen et al., 2007) and Chennai (Mohanty at al., 2008), urbanization can enhance the sea breezes and this enhancement causes more inland precipitation in the downwind areas. Bangkok is also the coastal city with the high rate of urbanization, so it is believed that the downwind area may also have the increasing trend of rainfall. As the result, the further study in the downwind area of Bangkok is also suggested.

In flood modeling, this study was conducted only in the area of the Bangkok Noi and Bangkok Yai districts. The source of flood in this study was only the local rainfall in these 2 districts while the runoff from the upstream area was still excluded. In this study, the sensitivity of the upstream discharge has been found to be medium. Expansion of the study area until the western boundary of the Tha Chin River is reached can help simulating the runoff from the upstream area.

Pump starting time is also another factor which possibly affects the water level. This factor depends on both the pumps themselves and the decision when to start the pumps, so it may be difficult to specify this factor precisely. This study assumed that the pumps reached their capacities in 15 minutes after the rainfall began. This duration was believed to be long enough for the pumps to react against the onset of the rainfall.

However, applying different pump starting time may affect the result since the sensitivity analysis suggests that the pump starting time has medium sensitivity to the water level.

Another limitation of this study is that this study focused on the flow in the open channels, while pipes and the floodplain were still excluded. The MIKE 11 model itself can simulate the flow only in the channel. When the water level is higher than the bank defined in the cross-section, the width of the water flow over the bank will still be equal to the top width of the cross-section (DHI, 2009a). However, including pipes and floodplain will both give more accurate result and allow the modeling of flood on the floodplain. They will also allow the studying of many more mitigations such as increasing the infiltration rate which could reduce flood in Jarkata, Indonesia (Indra et al., 2007), increasing basin storages which could reduce flood in the Playa de Gandia Resort in Valencia, Spain (Kronborg at al., 1999), the Young Brahmaputra Floodplain in Bangladesh (Ahmed and Shah-Newaz, 2001), and the Yom River Basin (Thien, 2005) and Chi River Basin (Pawattana et al., 2007), Thailand. In fact, increasing the infiltration rate and slowing the overland flow are the interesting mitigation measures for the area since it has been found that the water level in the area is sensitive to the overland flow runoff coefficient and time constant for routing overland flow. However, the elevation data should be revised frequently since the elevation of the area of Bangkok is subjected to change by the land subsidence (Giao, Nutalaya, and Phien-wej, 2006) and the filling of materials (Hara et al., 2008).

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APPENDICES

APPENDIX A

DOUBLE MASS CURVE

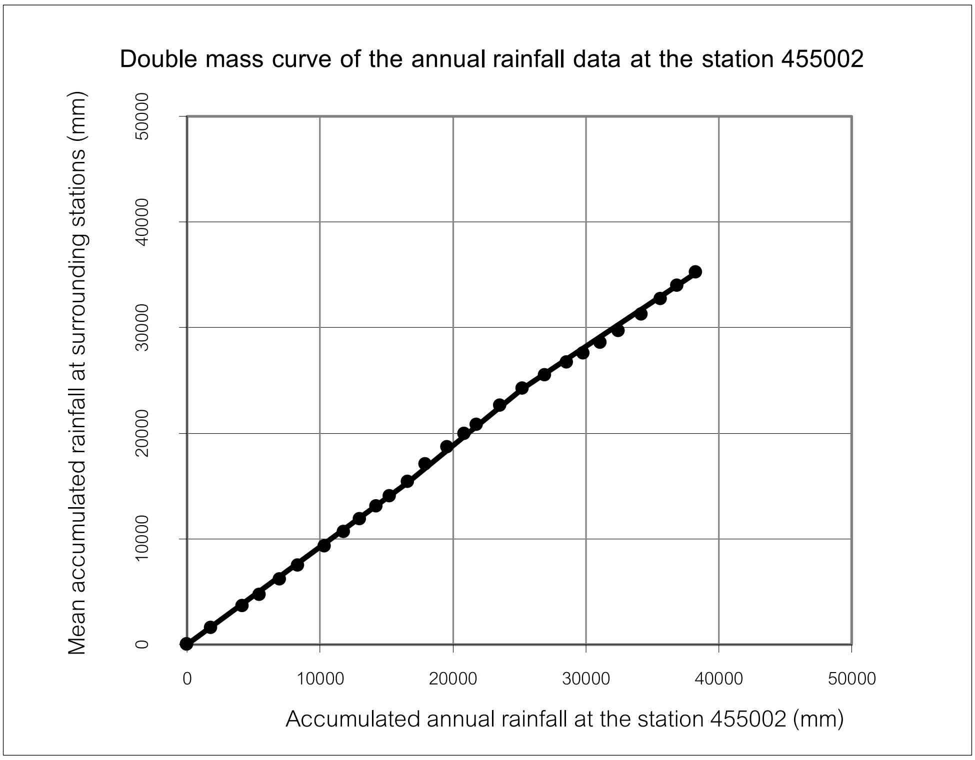


Figure A-1 Double mass curve of the annual rainfall data at the station 455002

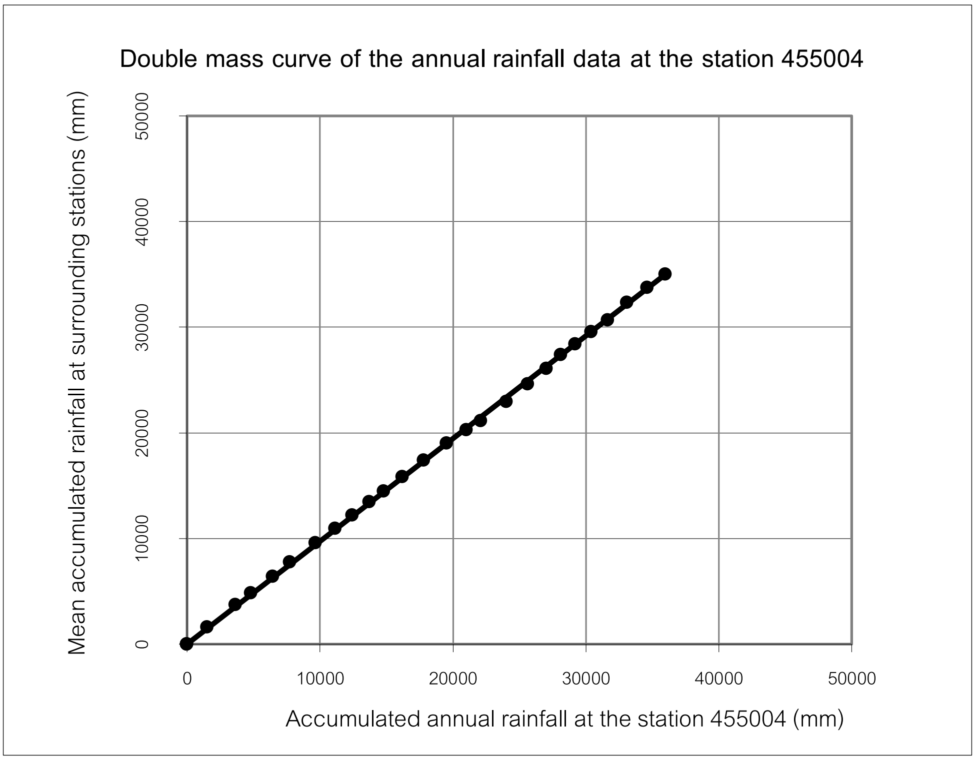


Figure A-2 Double mass curve of the annual rainfall data at the station 455004

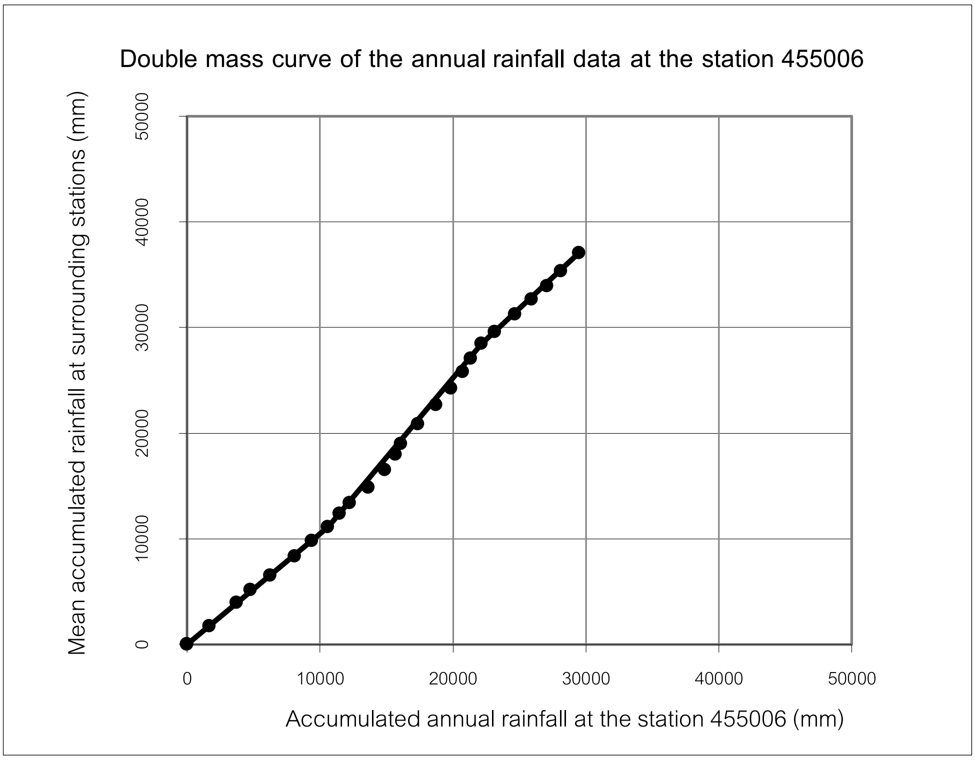


Figure A-3 Double mass curve of the annual rainfall data at the station 455006

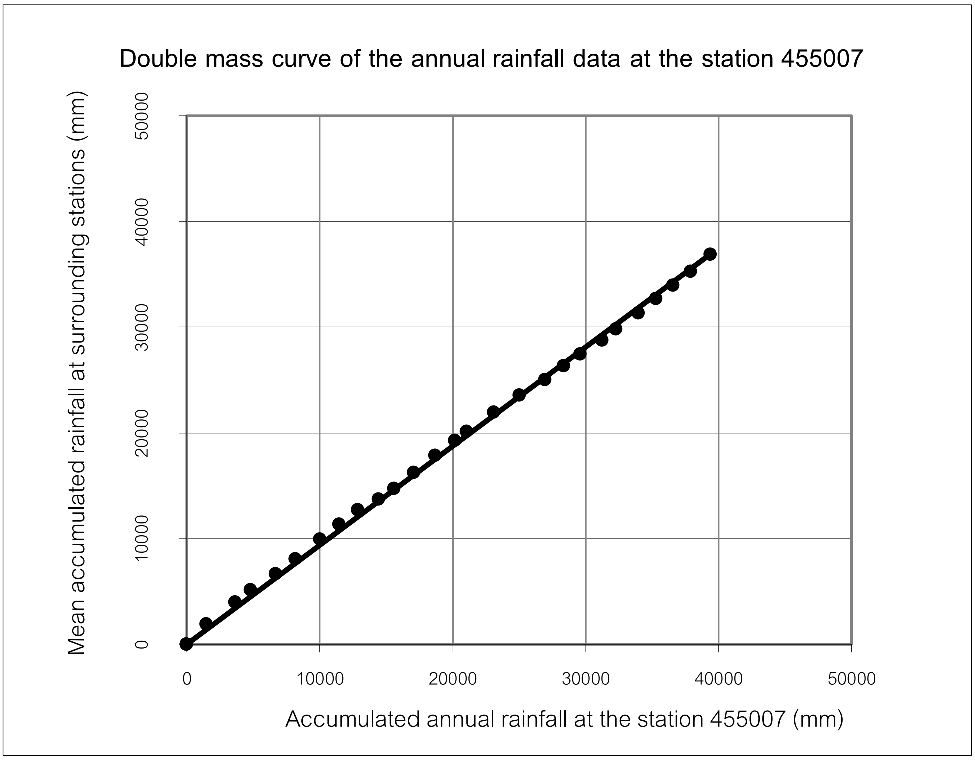


Figure A-4 Double mass curve of the annual rainfall data at the station 455007

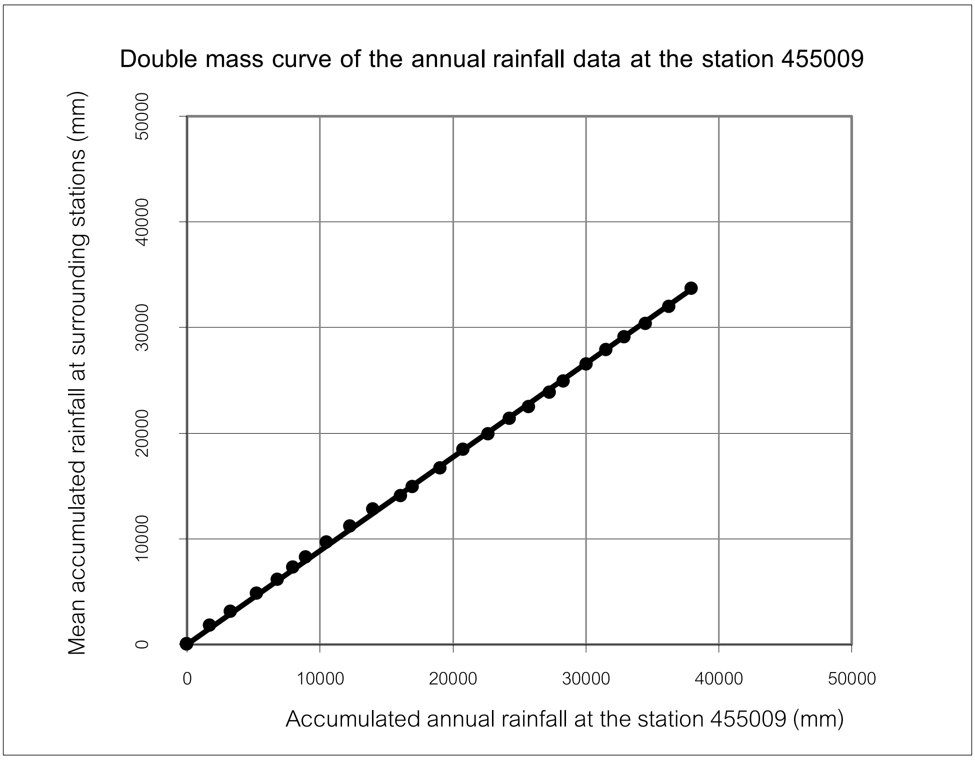


Figure A-5 Double mass curve of the annual rainfall data at the station 455009

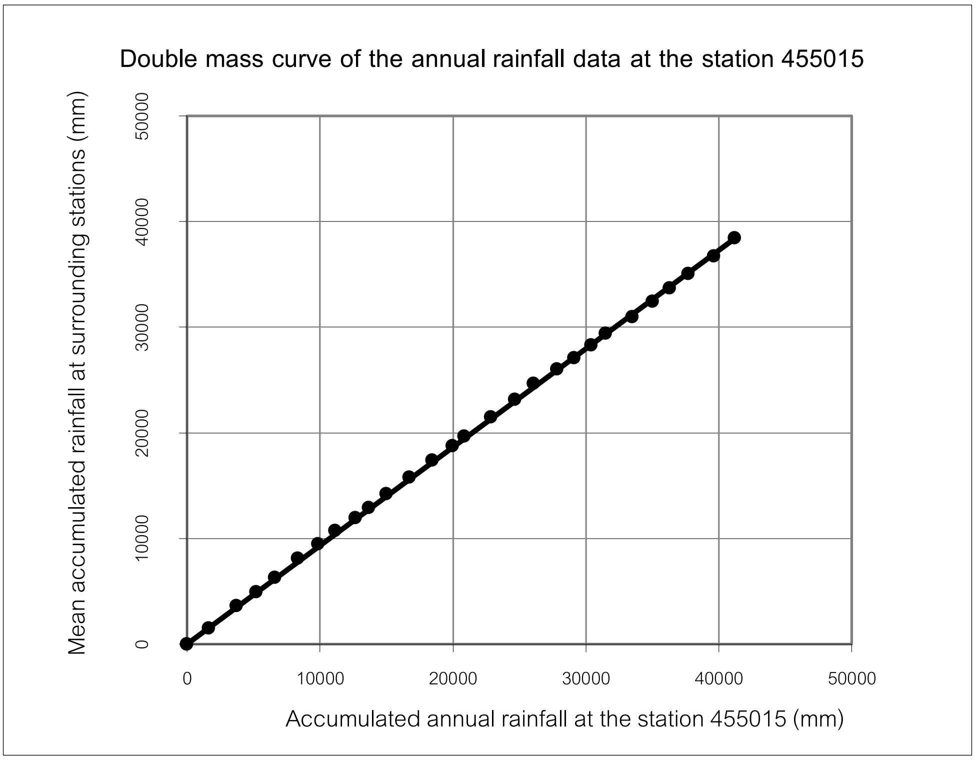


Figure A-6 Double mass curve of the annual rainfall data at the station 455015

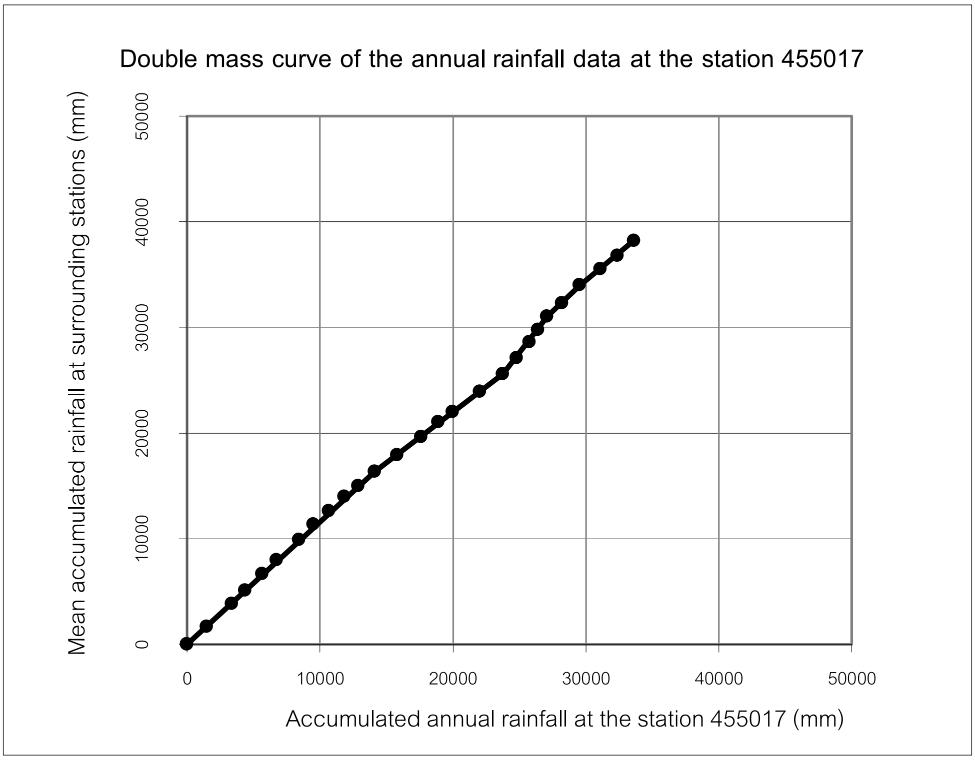


Figure A-7 Double mass curve of the annual rainfall data at the station 455017

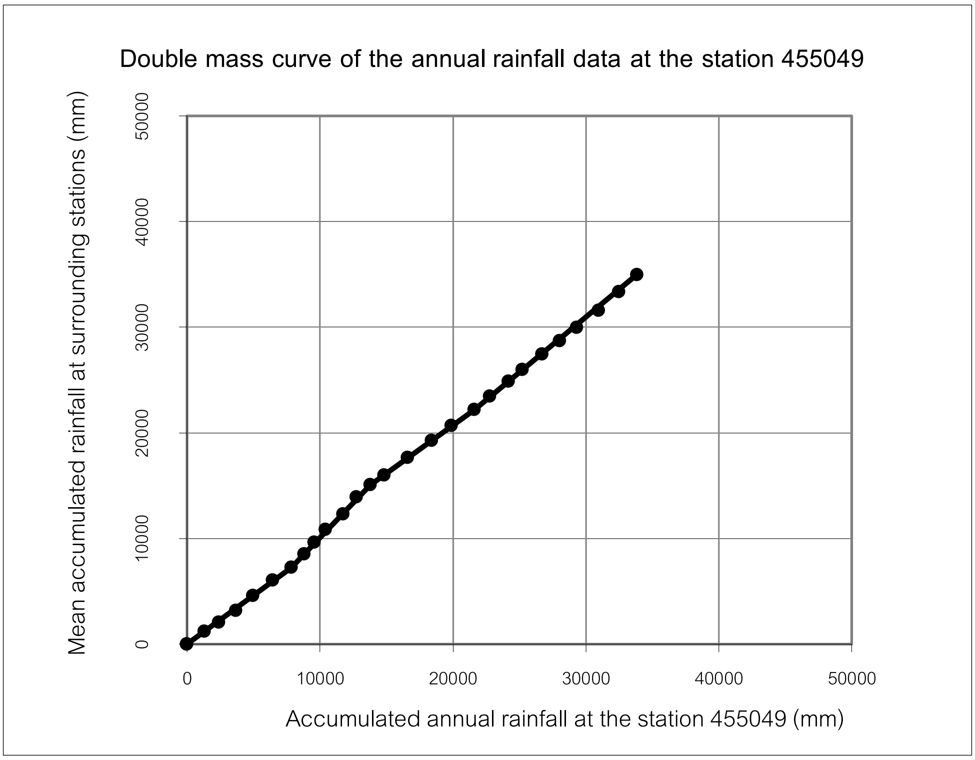


Figure A-8 Double mass curve of the annual rainfall data at the station 455049

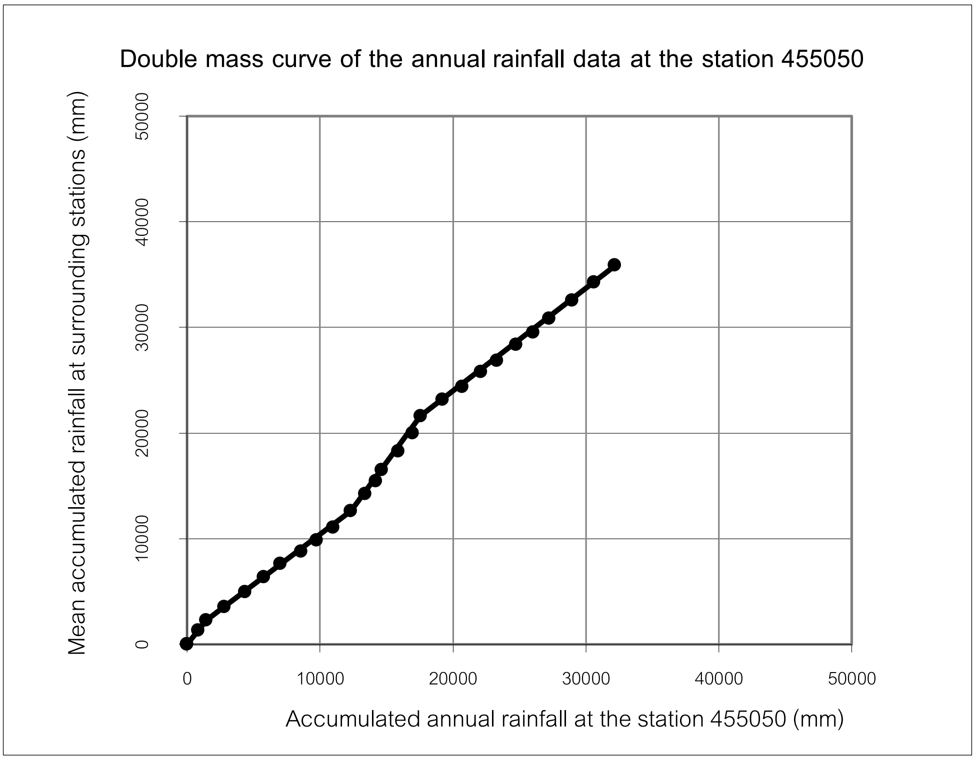


Figure A-9 Double mass curve of the annual rainfall data at the station 455050

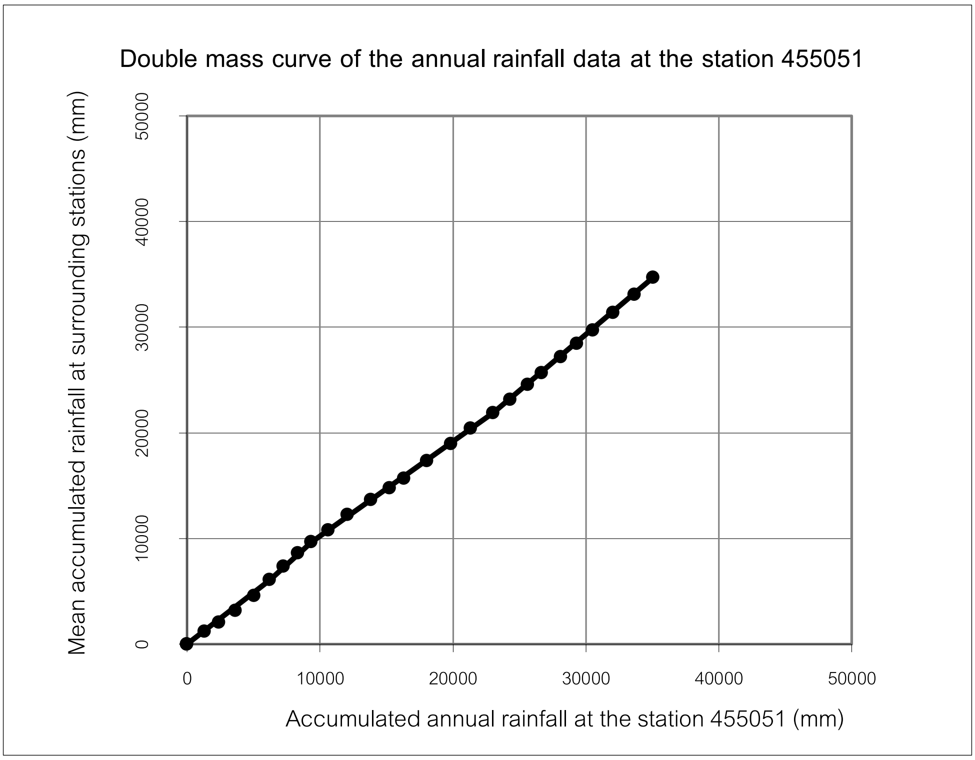


Figure A-10 Double mass curve of the annual rainfall data at the station 455051

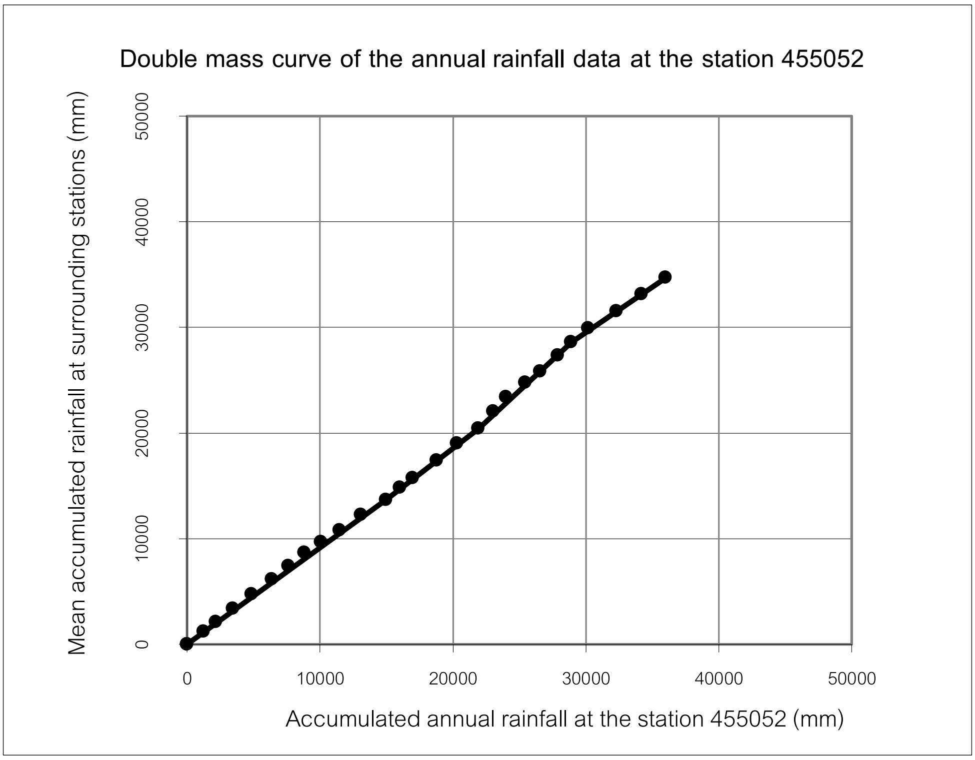


Figure A-11 Double mass curve of the annual rainfall data at the station 455052

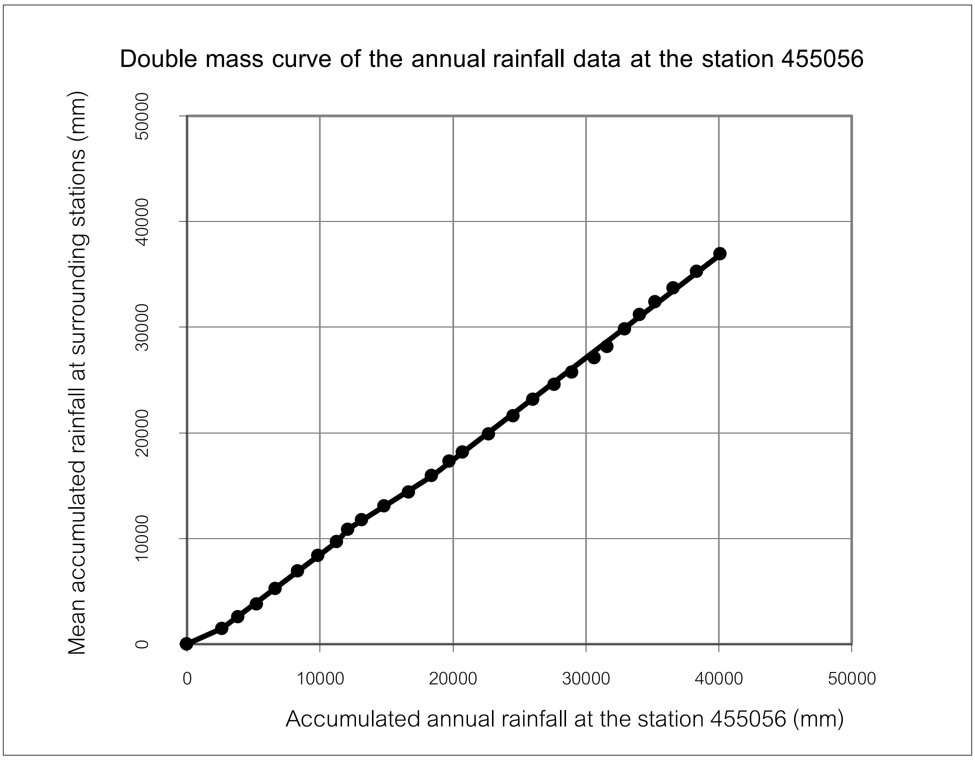


Figure A-12 Double mass curve of the annual rainfall data at the station 455056

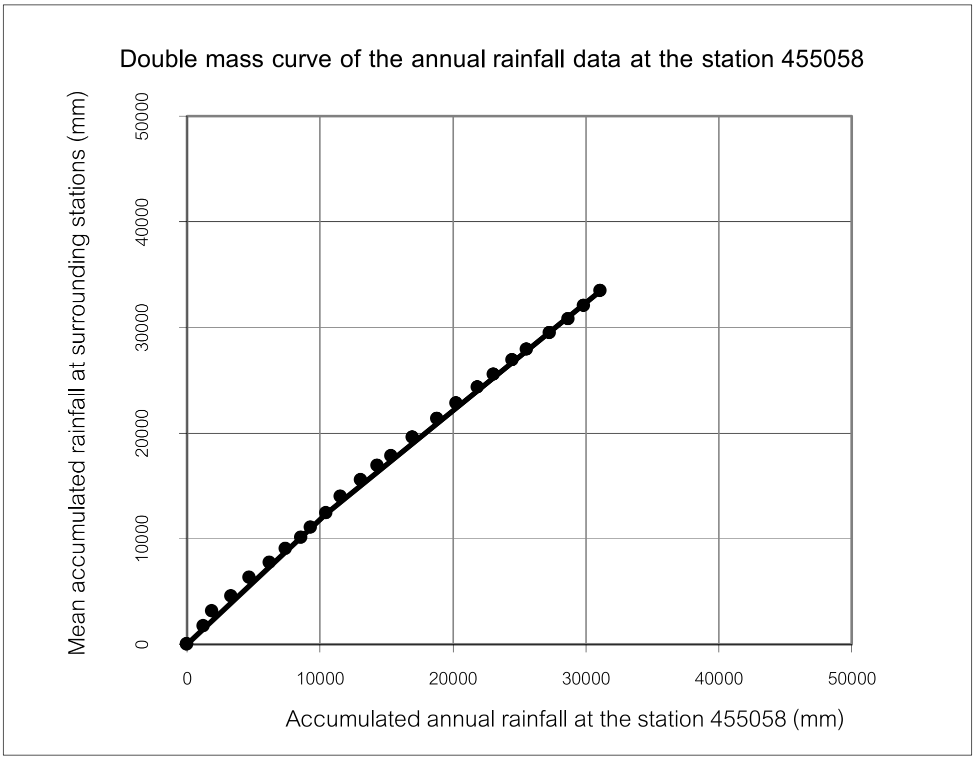


Figure A-13 Double mass curve of the annual rainfall data at the station 455058

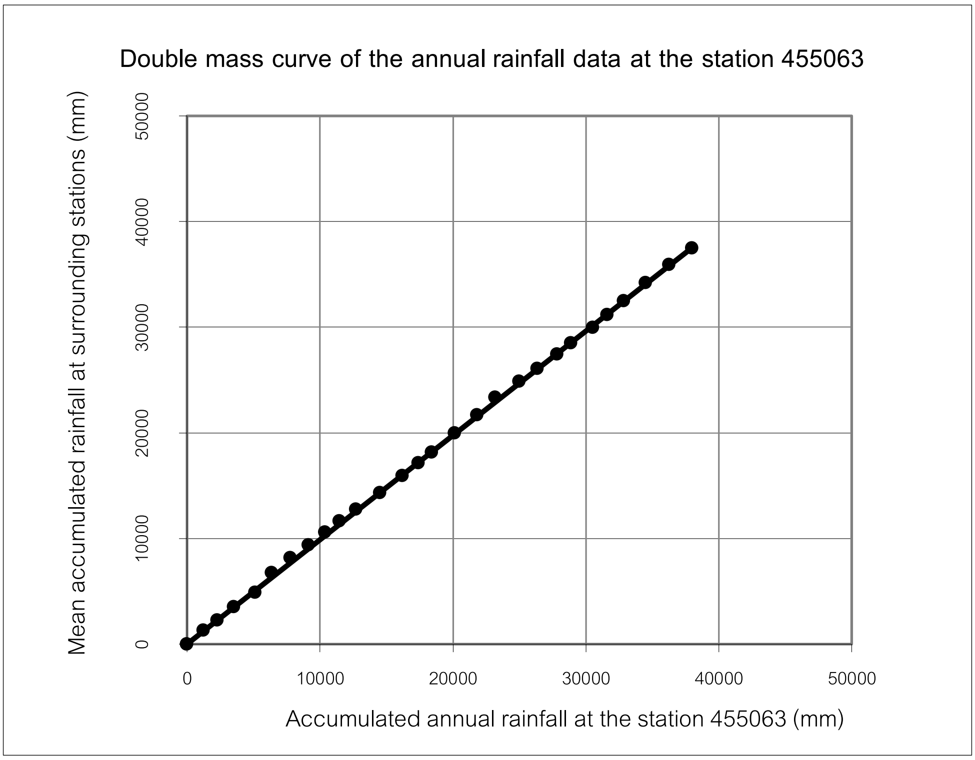


Figure A-14 Double mass curve of the annual rainfall data at the station 455063

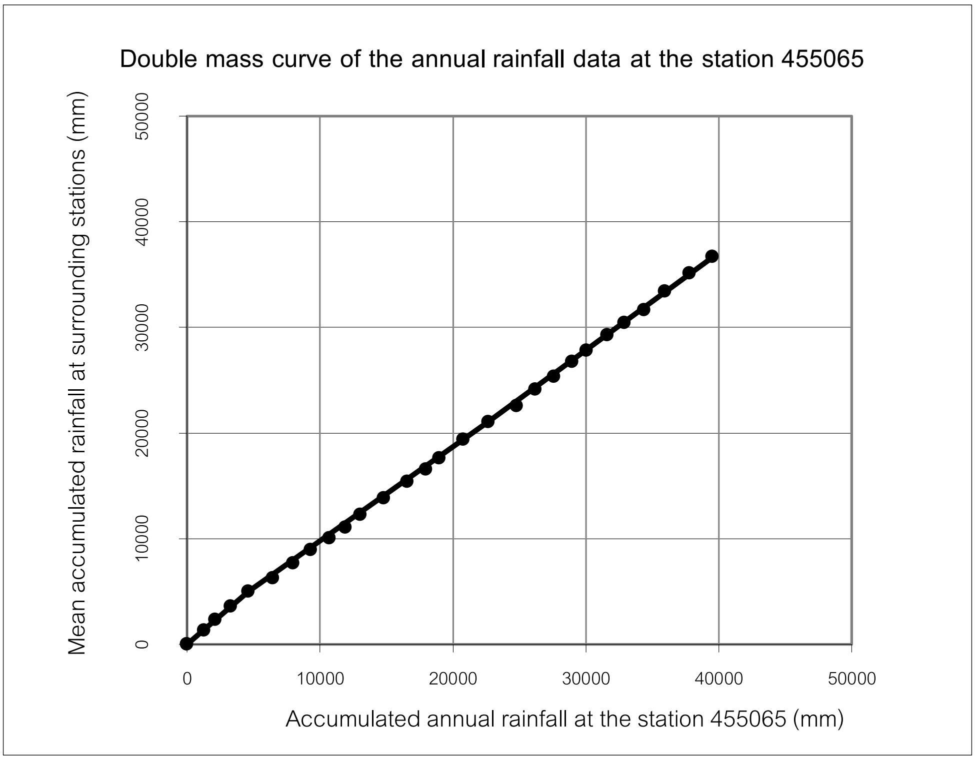


Figure A-15 Double mass curve of the annual rainfall data at the station 455065

APPENDIX B

ANNUAL RAINFALL TREND

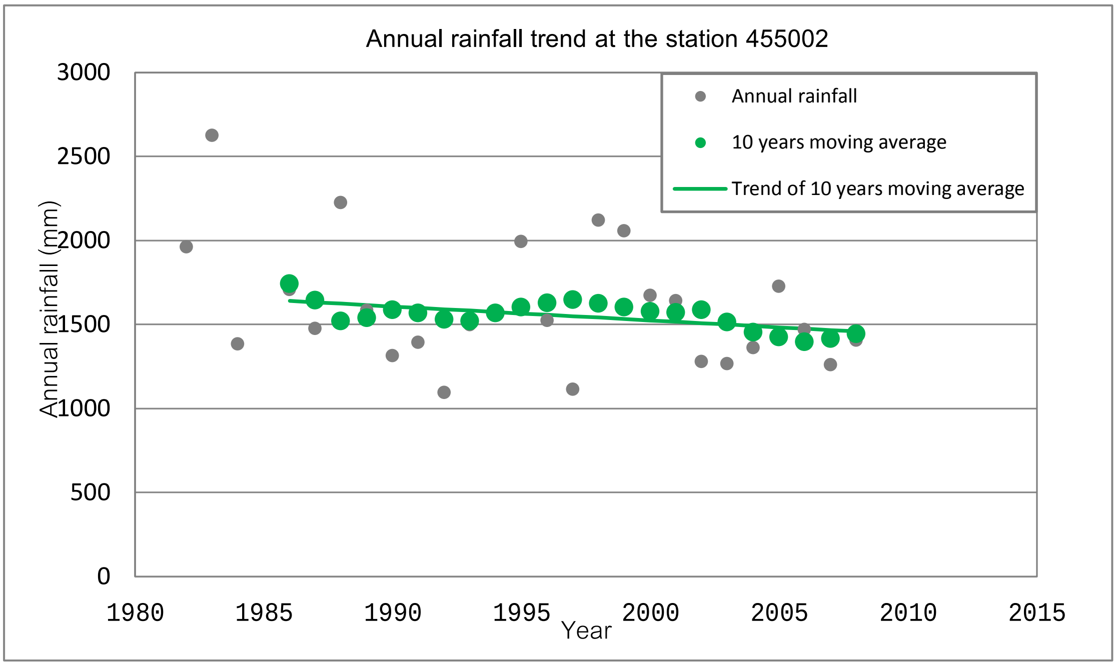


Figure B-1 Annual rainfall trend at the station 455002

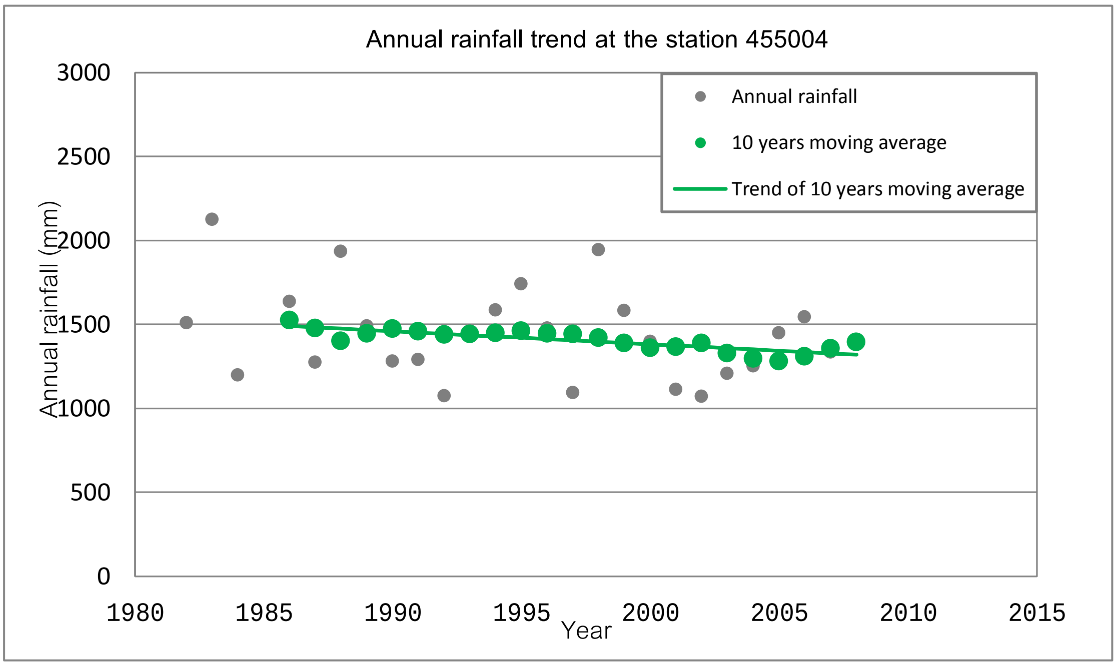


Figure B-2 Annual rainfall trend at the station 455004

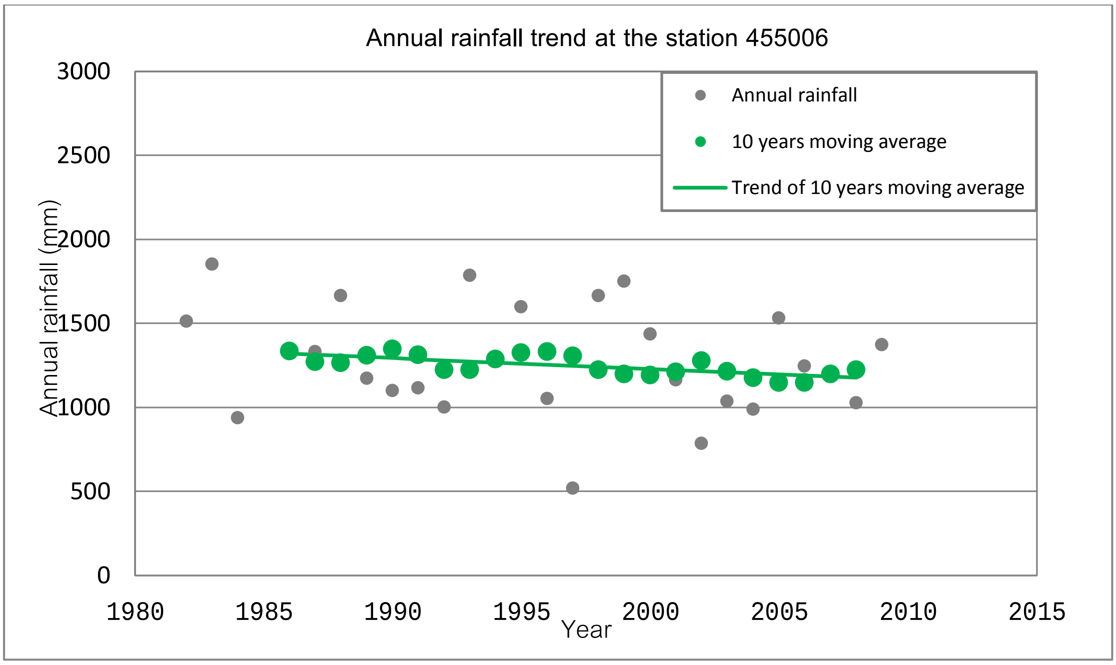


Figure B-3 Annual rainfall trend at the station 455006

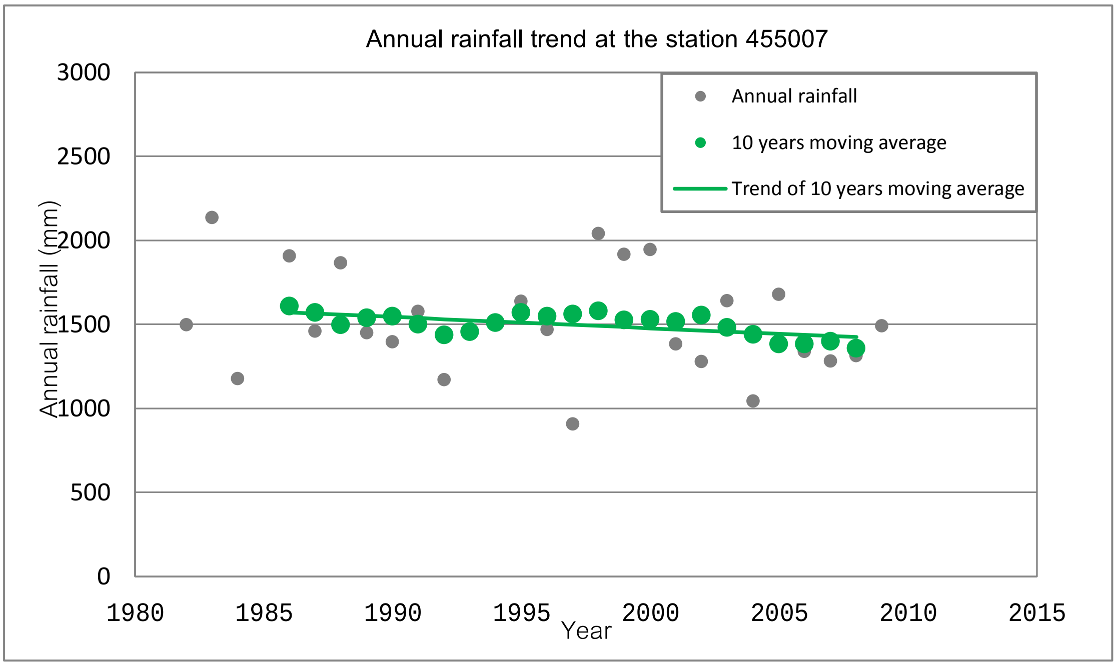


Figure B-4 Annual rainfall trend at the station 455007

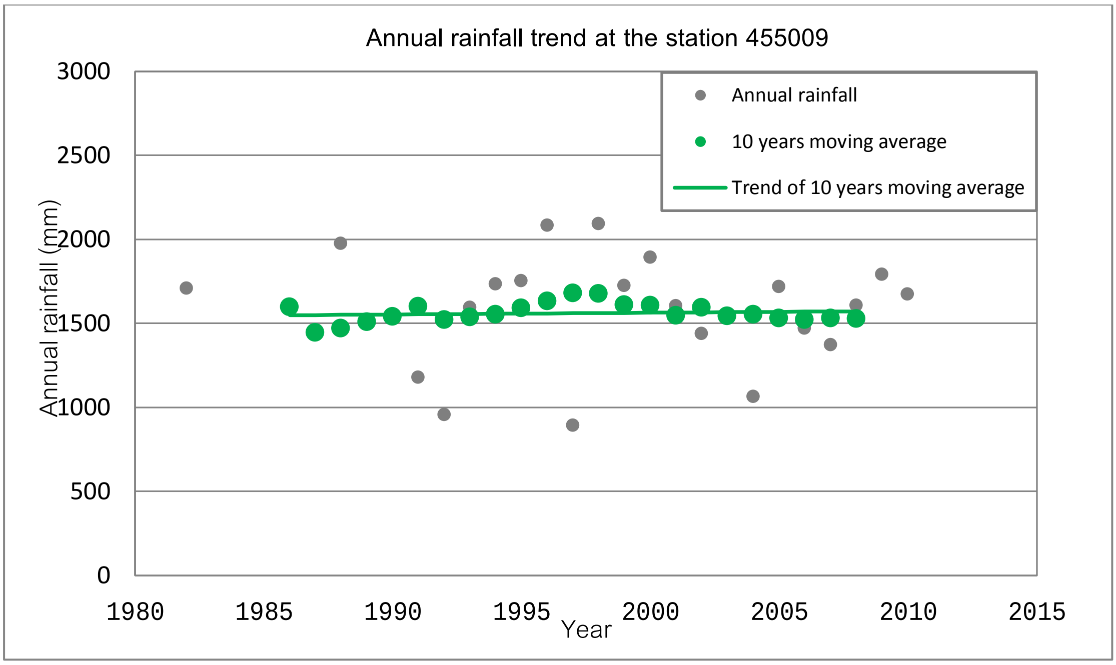


Figure B-5 Annual rainfall trend at the station 455009

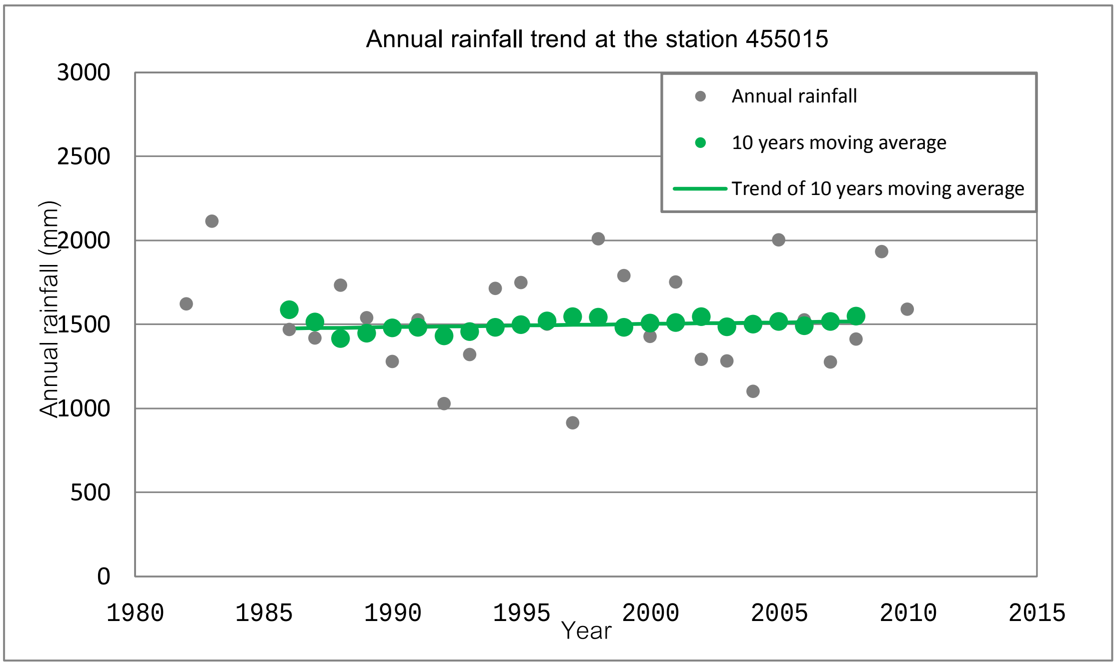


Figure B-6 Annual rainfall trend at the station 455015

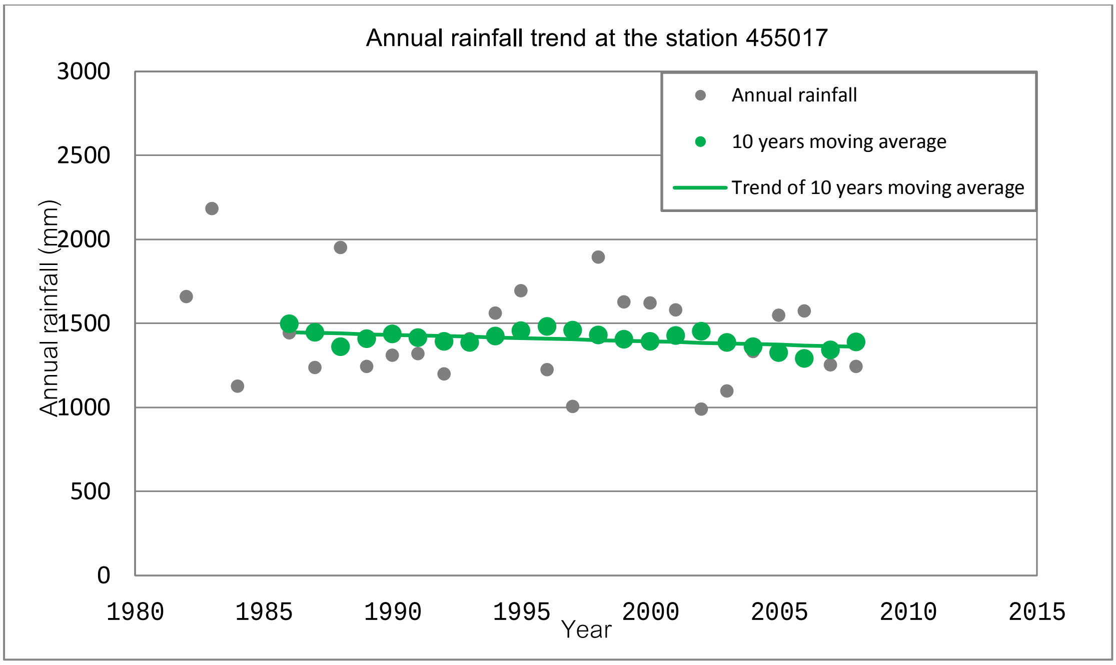


Figure B-7 Annual rainfall trend at the station 455017

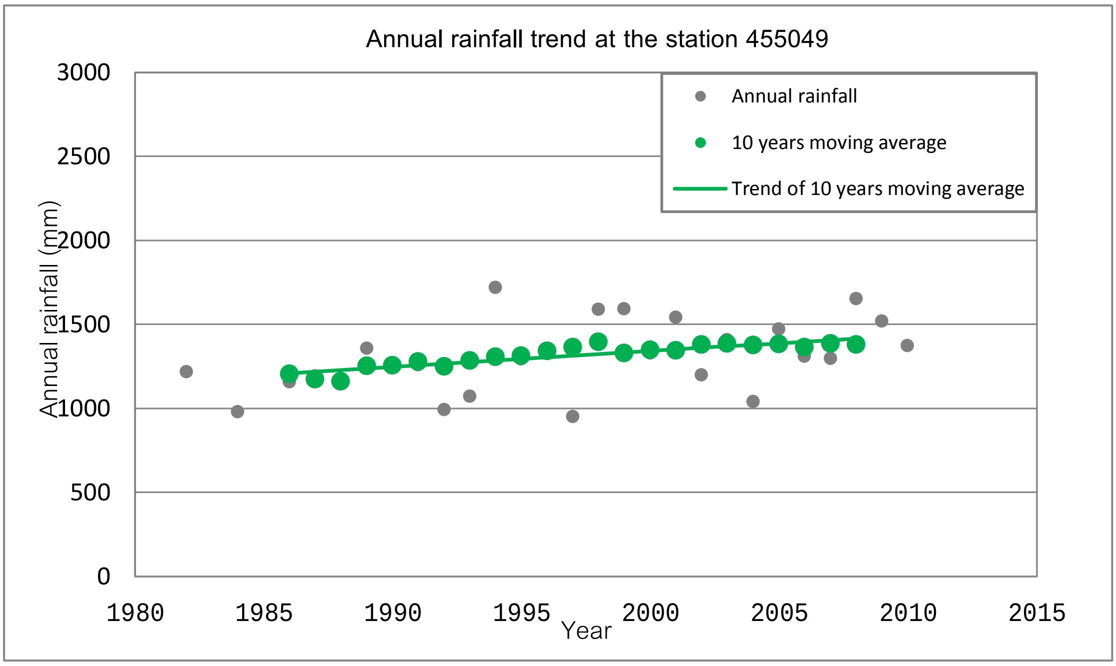


Figure B-8 Annual rainfall trend at the station 455049

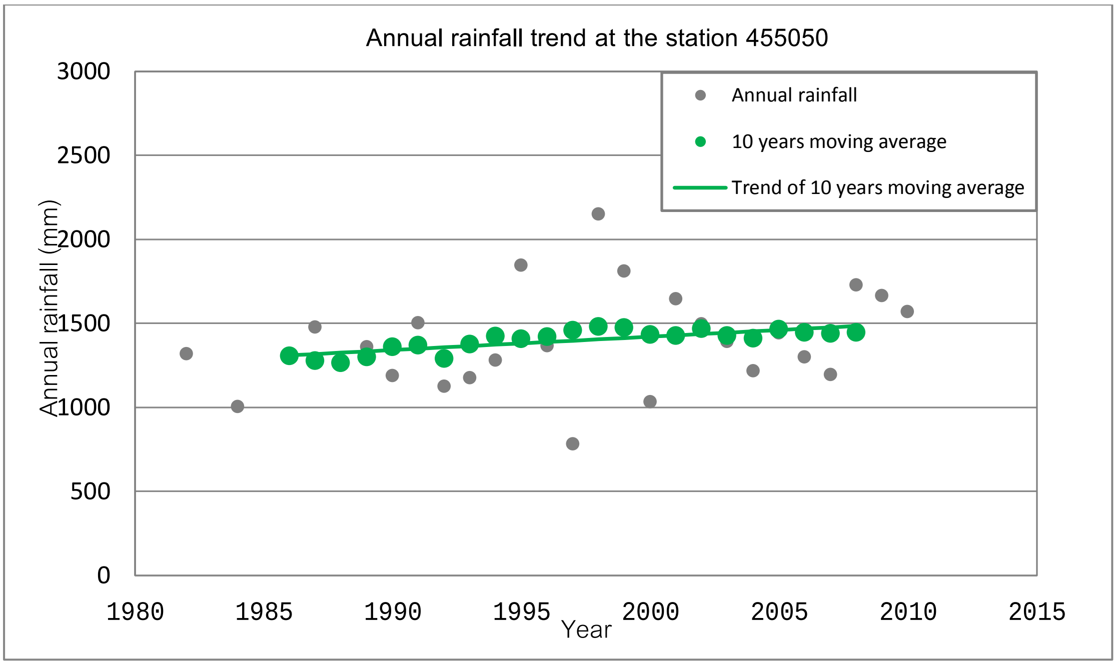


Figure B-9 Annual rainfall trend at the station 455050

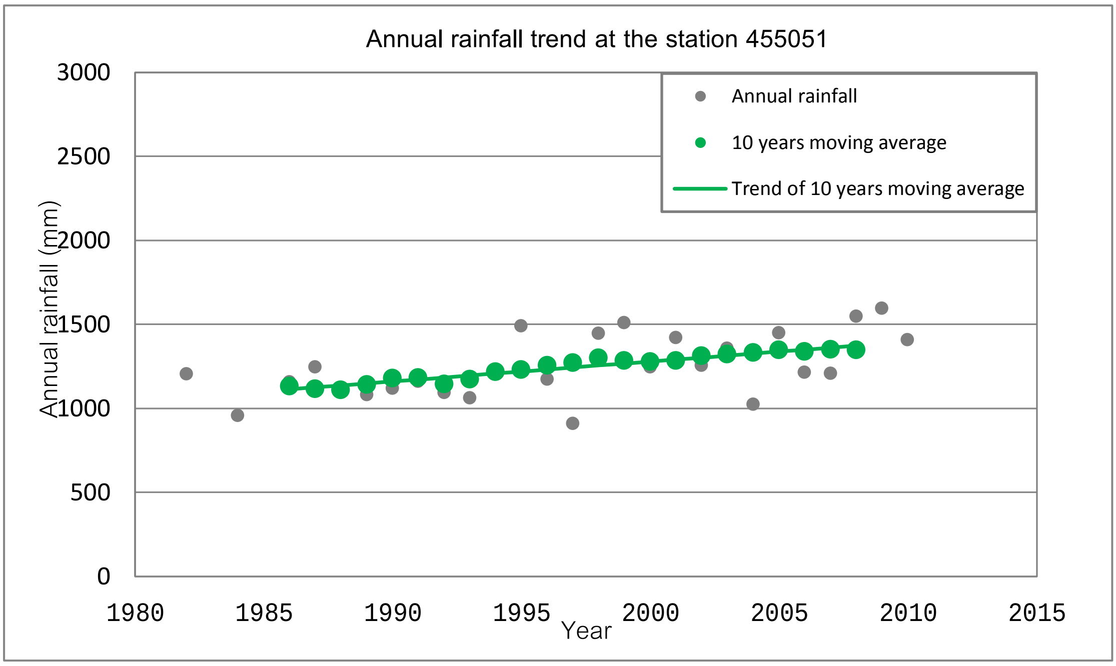


Figure B-10 Annual rainfall trend at the station 455051

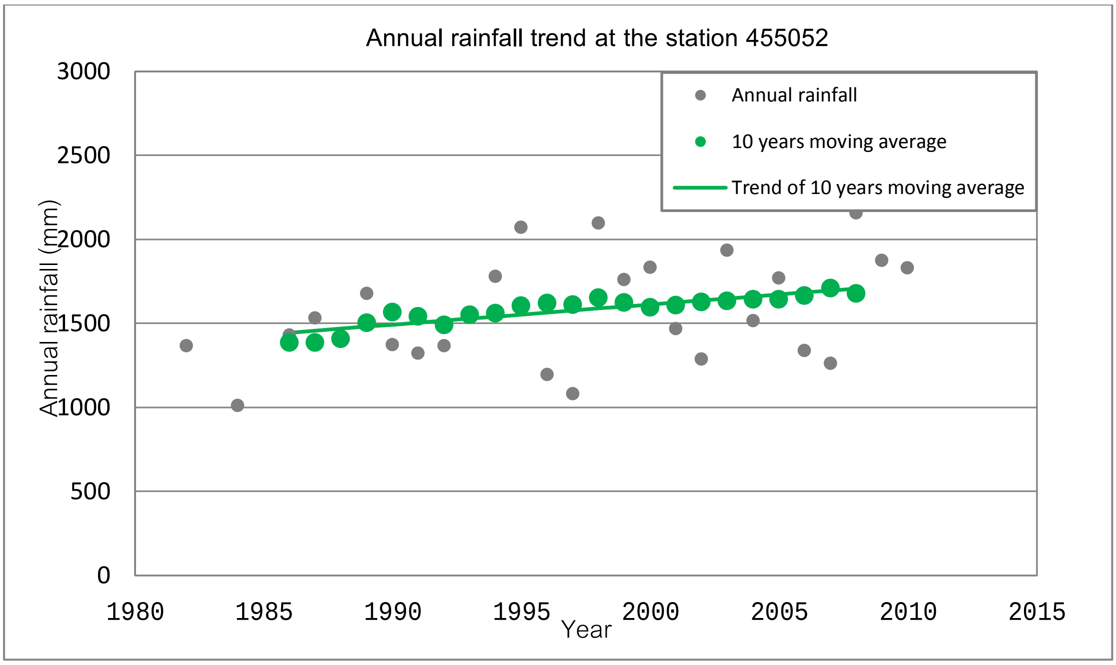


Figure B-11 Annual rainfall trend at the station 455052

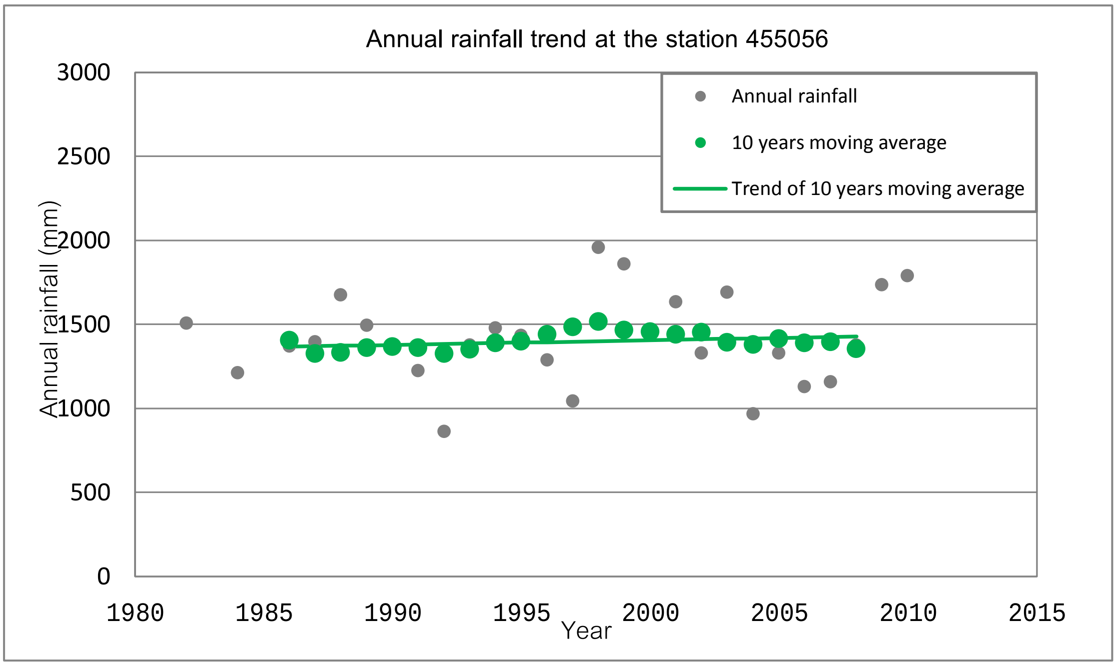


Figure B-12 Annual rainfall trend at the station 455056

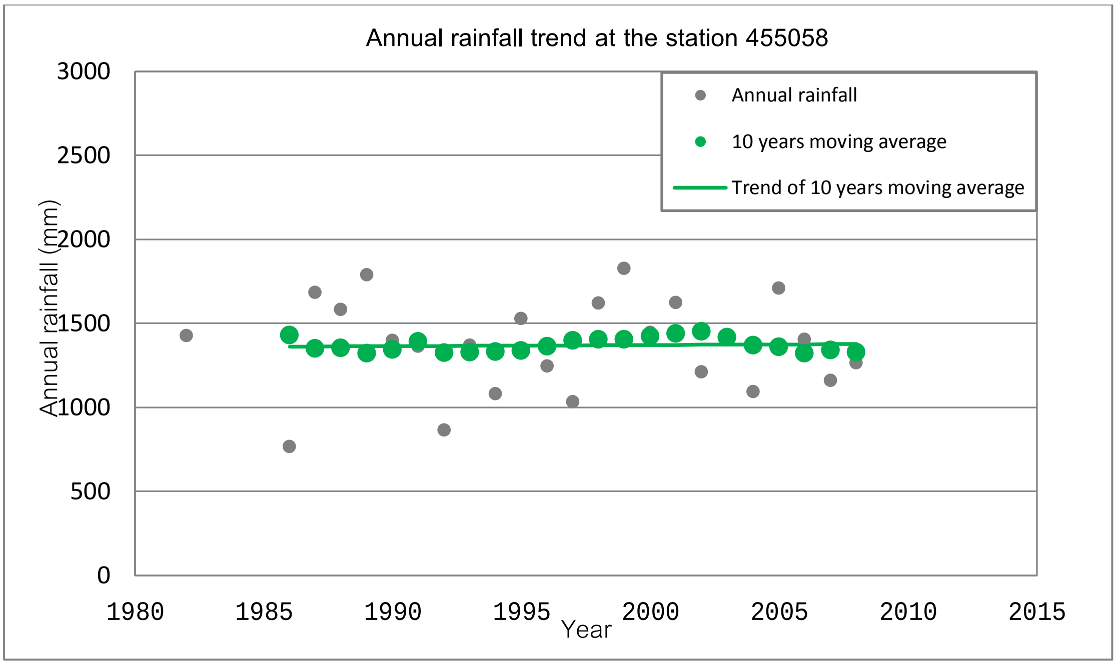


Figure B-13 Annual rainfall trend at the station 455058

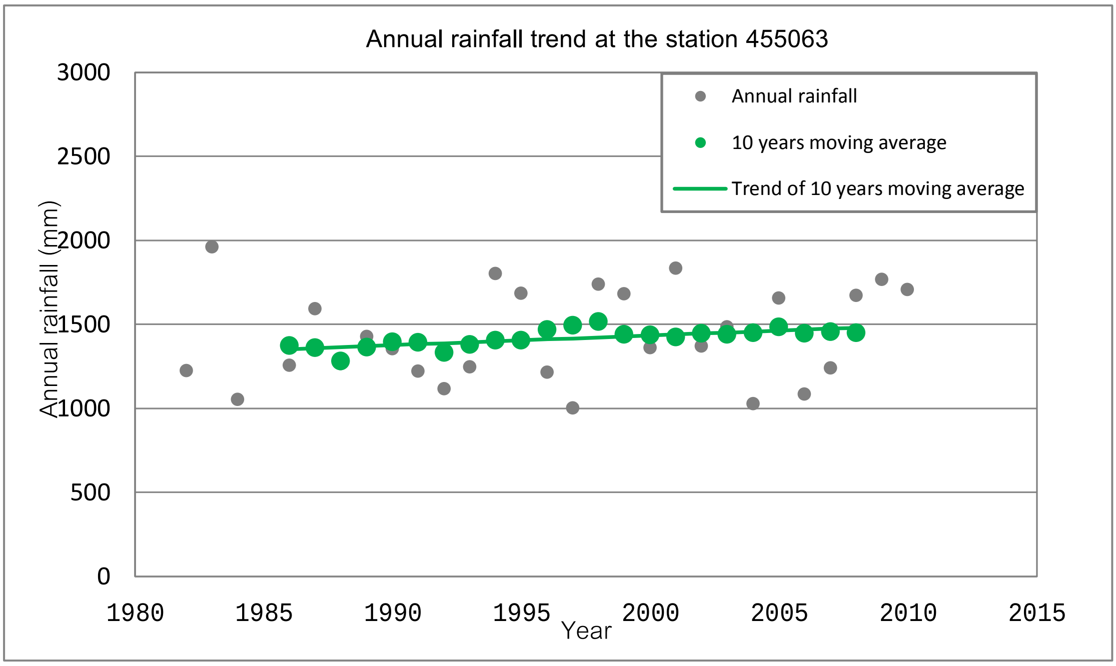


Figure B-14 Annual rainfall trend at the station 455063

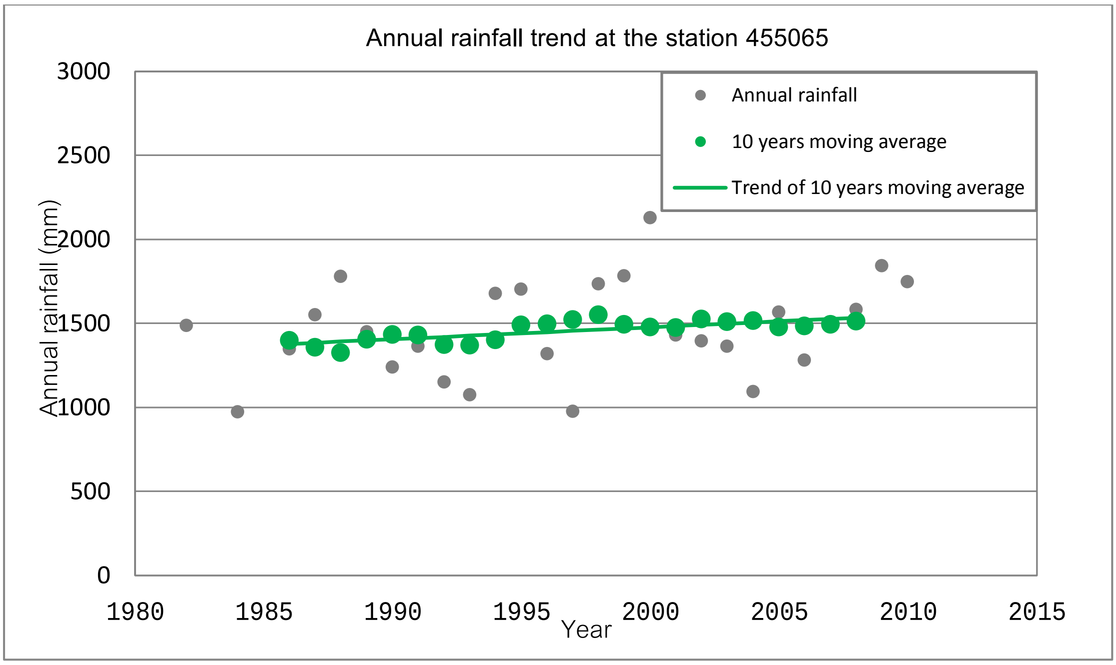


Figure B-15 Annual rainfall trend at the station 455065

APPENDIX C

MAXIMUM 1-DAY RAINFALL TREND

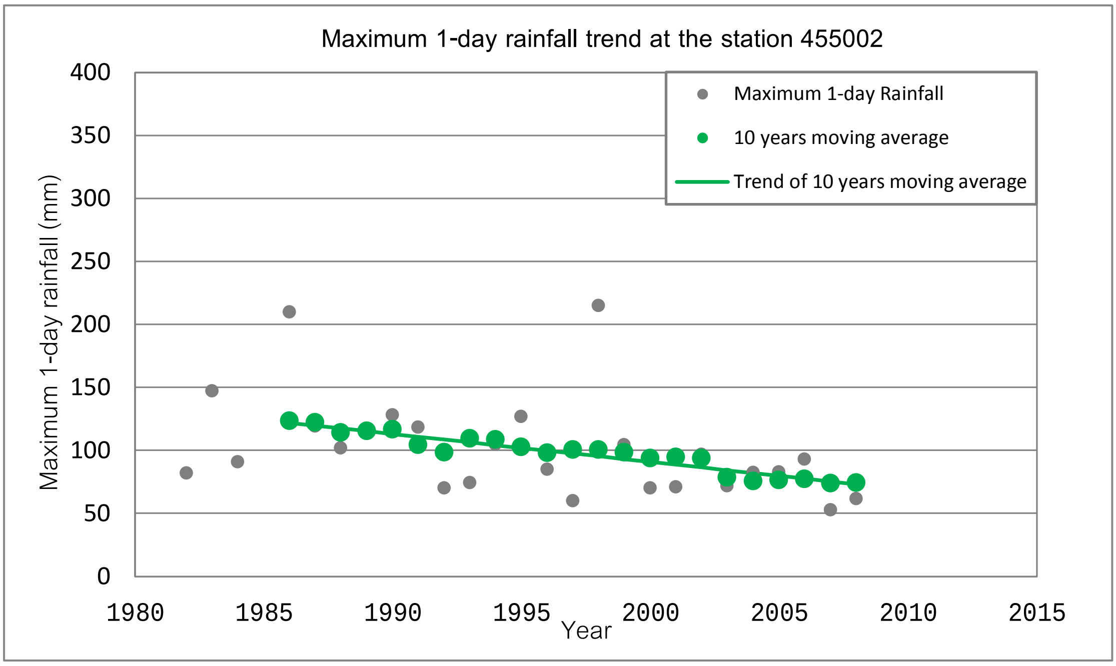


Figure C-1 Maximum 1-day rainfall trend at the station 455002

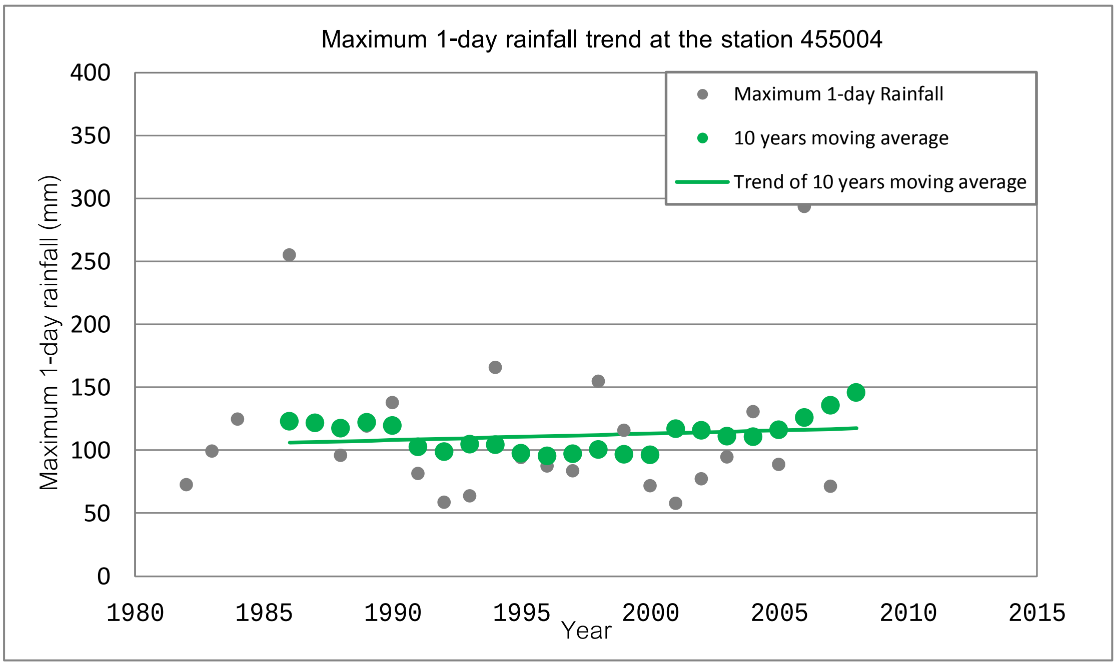


Figure C-2 Maximum 1-day rainfall trend at the station 455004

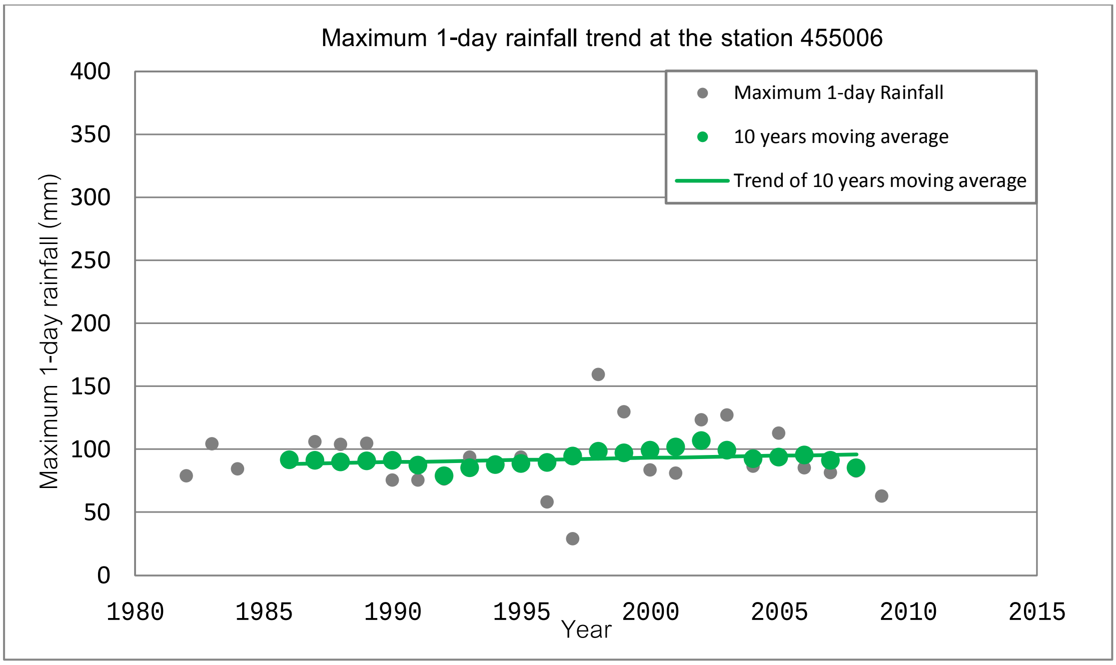


Figure C-3 Maximum 1-day rainfall trend at the station 455006

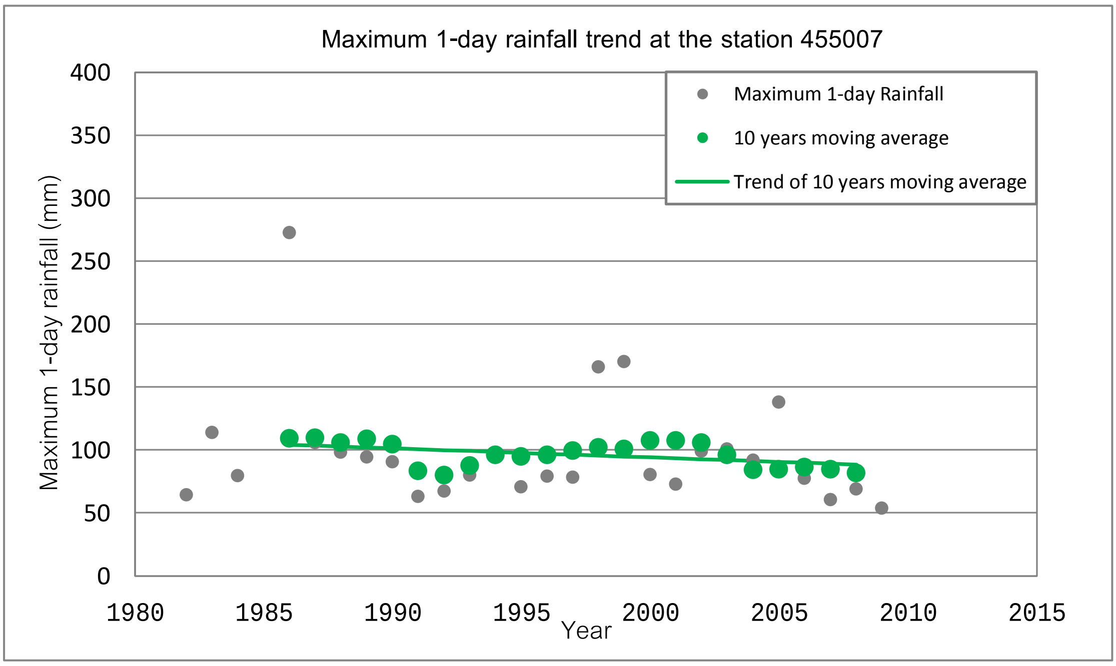


Figure C-4 Maximum 1-day rainfall trend at the station 455007

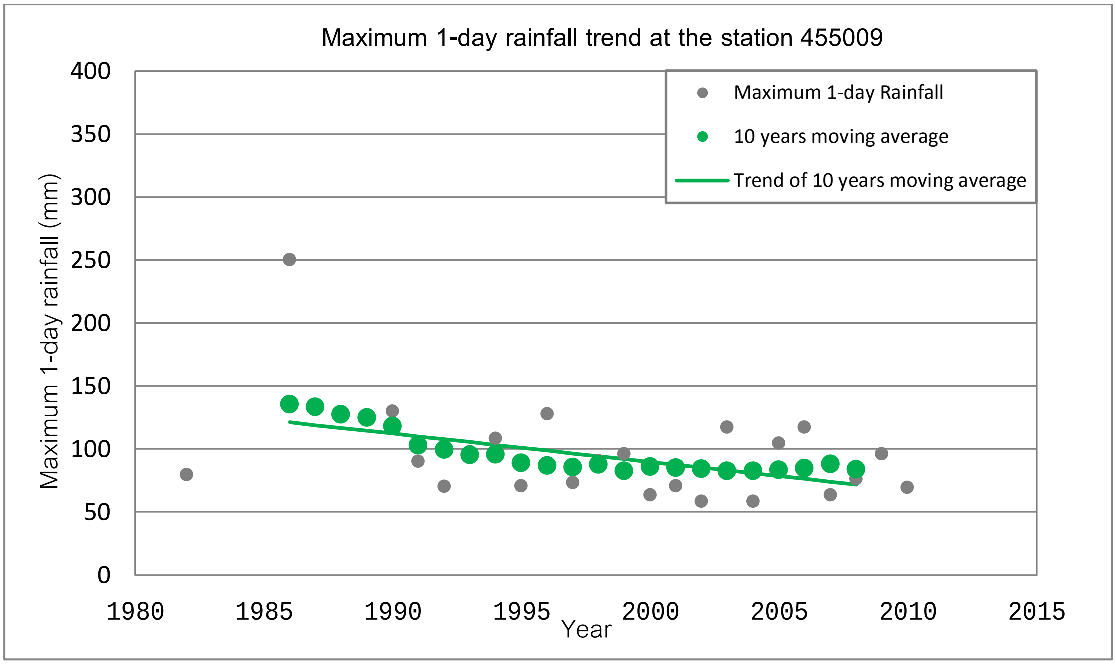


Figure C-5 Maximum 1-day rainfall trend at the station 455009

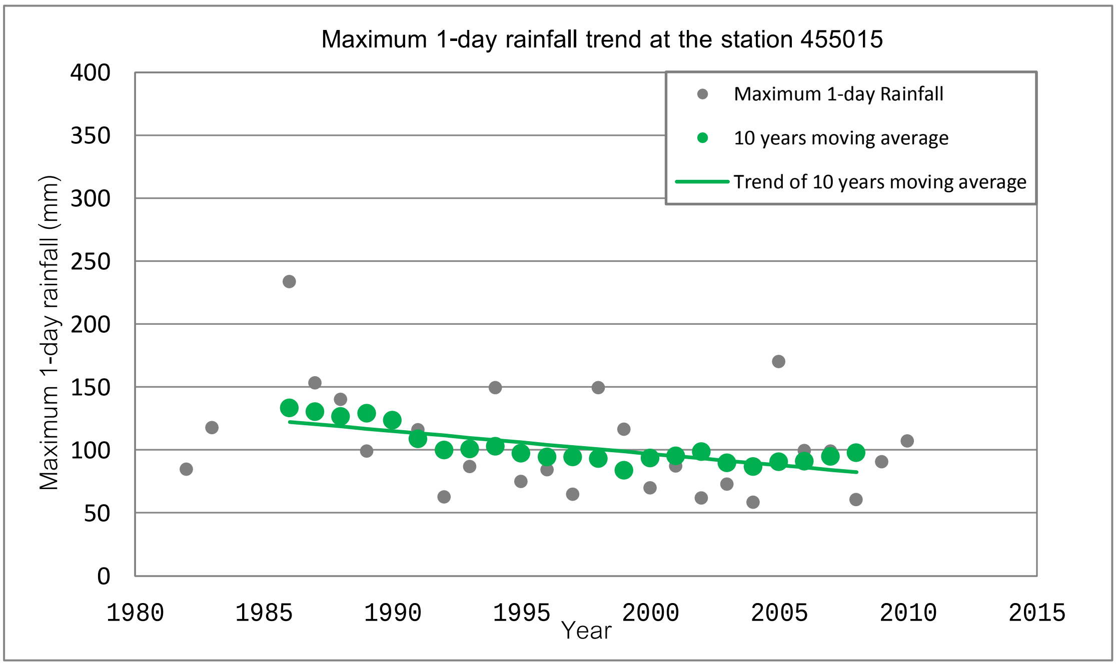


Figure C-6 Maximum 1-day rainfall trend at the station 455015

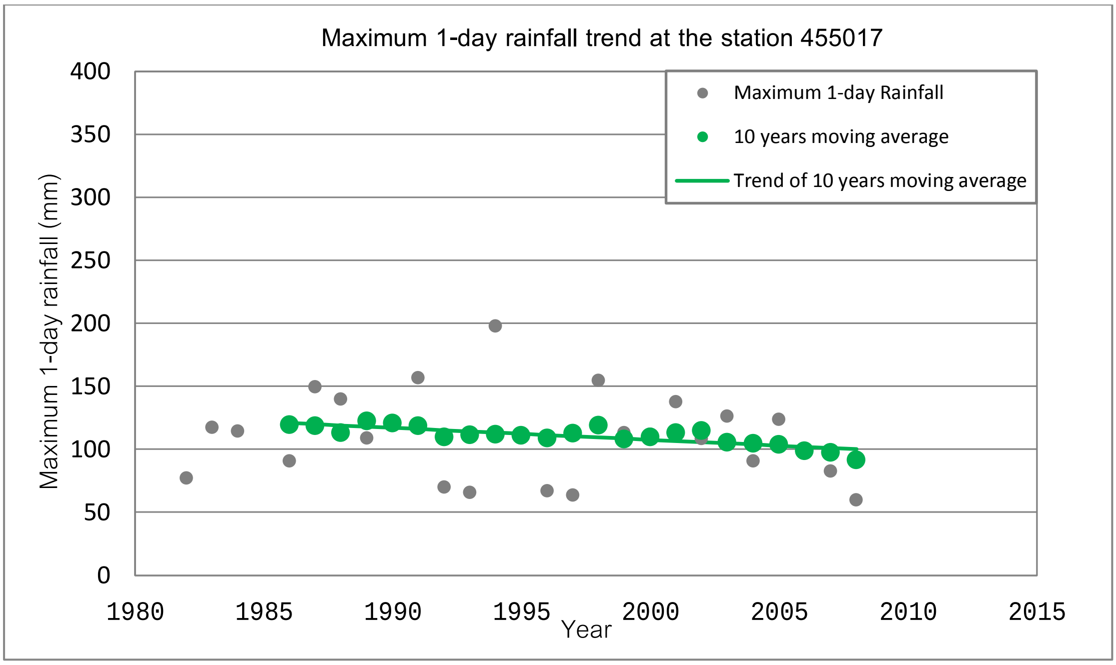


Figure C-7 Maximum 1-day rainfall trend at the station 455017

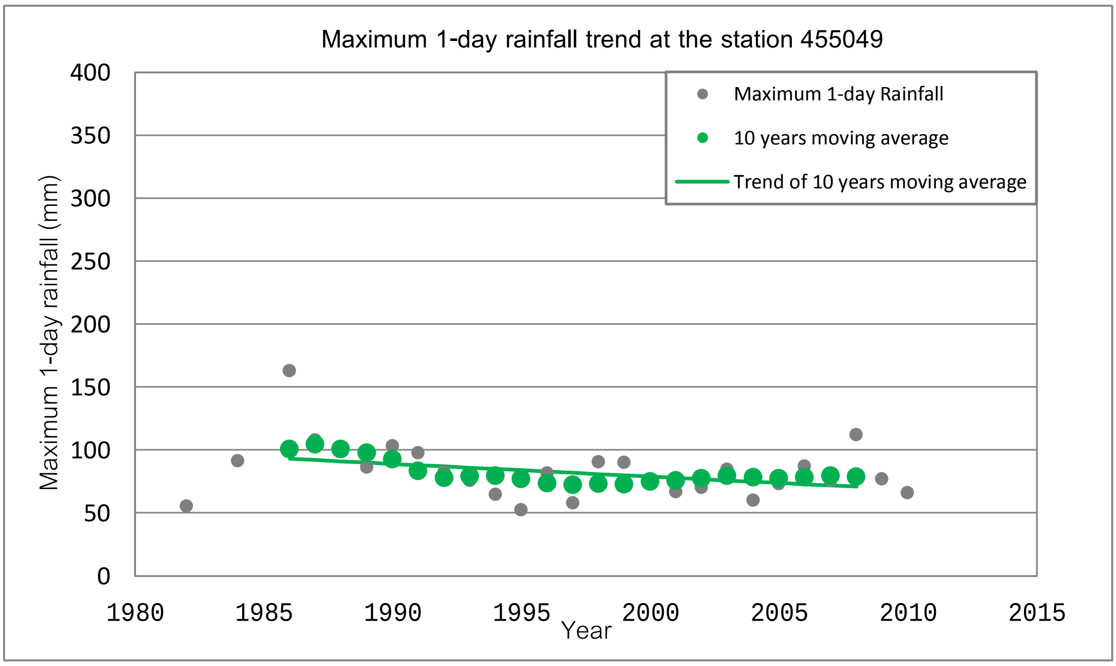


Figure C-8 Maximum 1-day rainfall trend at the station 455049

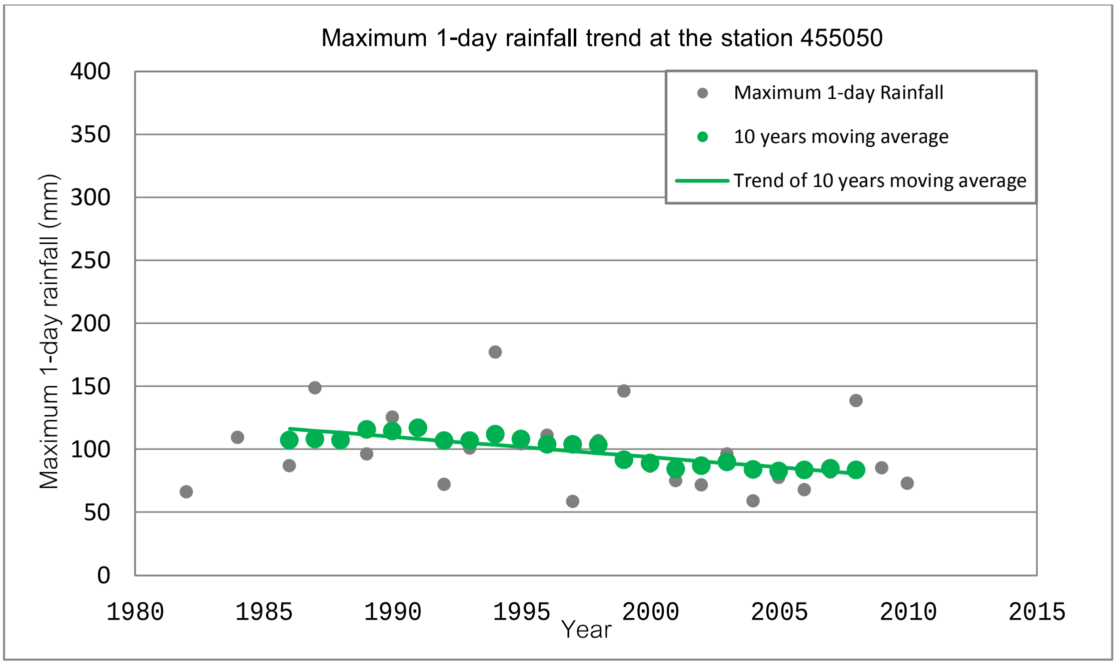


Figure C-9 Maximum 1-day rainfall trend at the station 455050

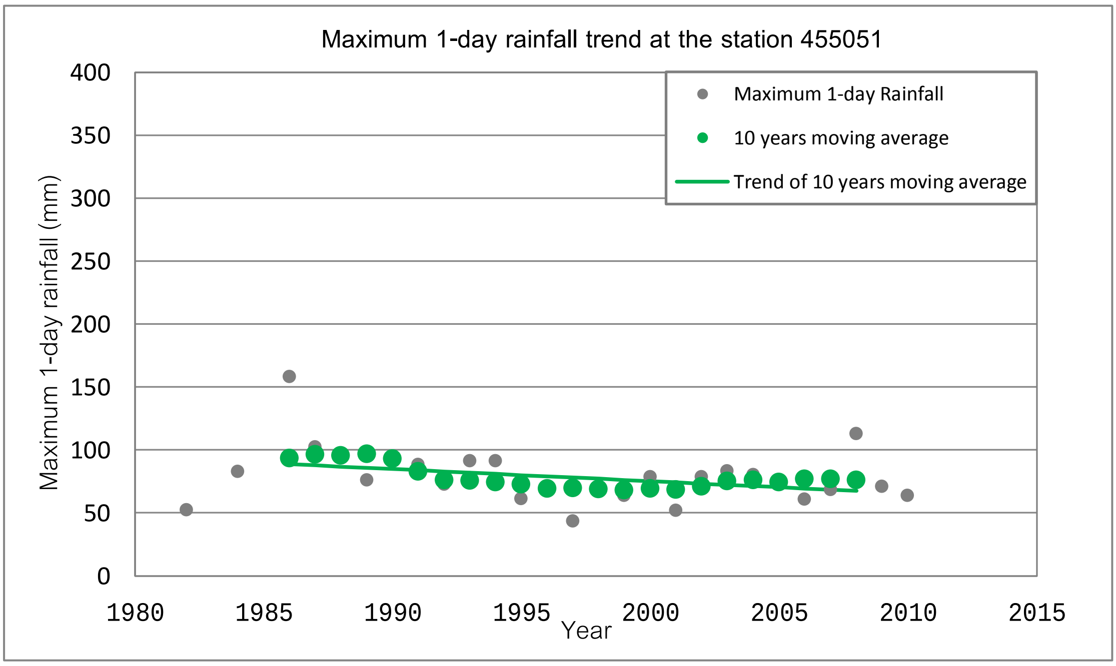


Figure C-10 Maximum 1-day rainfall trend at the station 455051

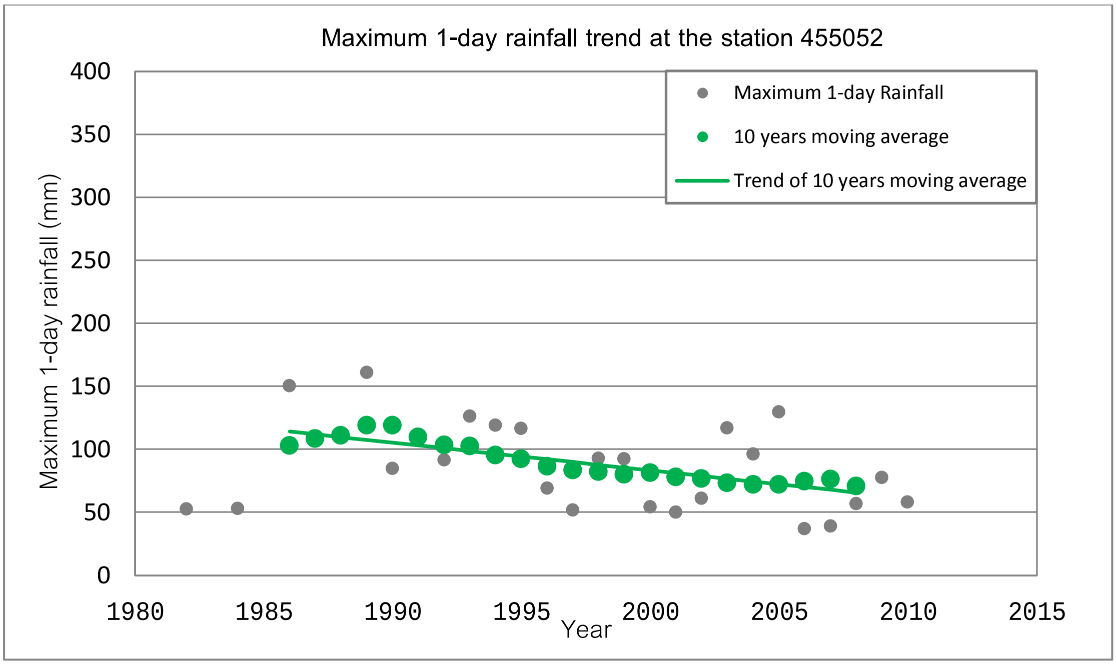


Figure C-11 Maximum 1-day rainfall trend at the station 455052

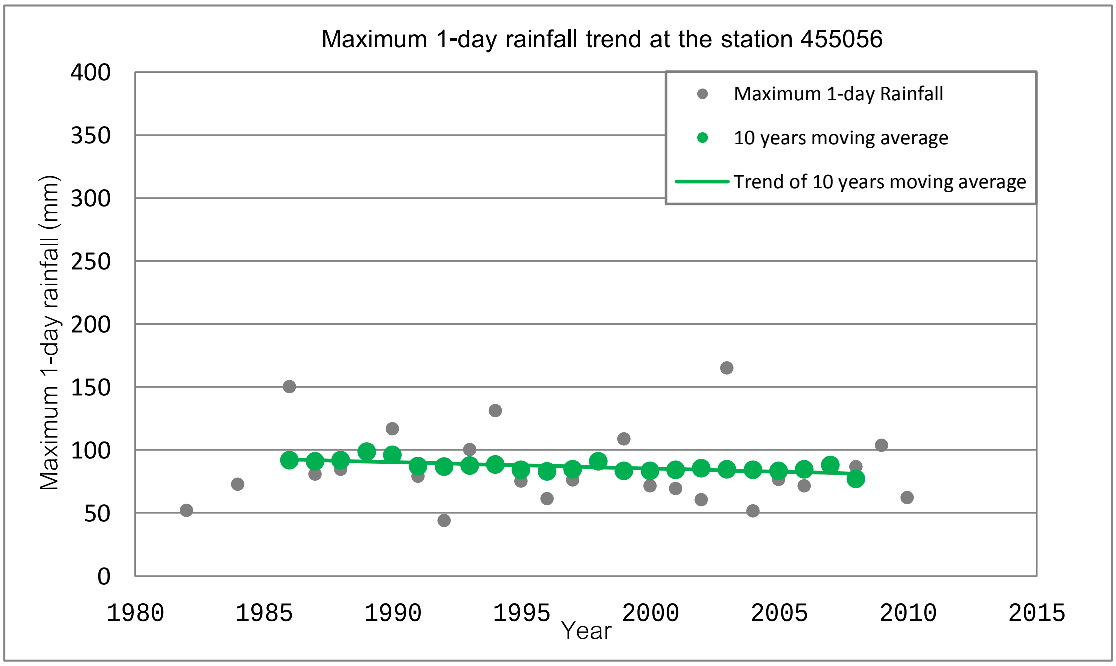


Figure C-12 Maximum 1-day rainfall trend at the station 455056

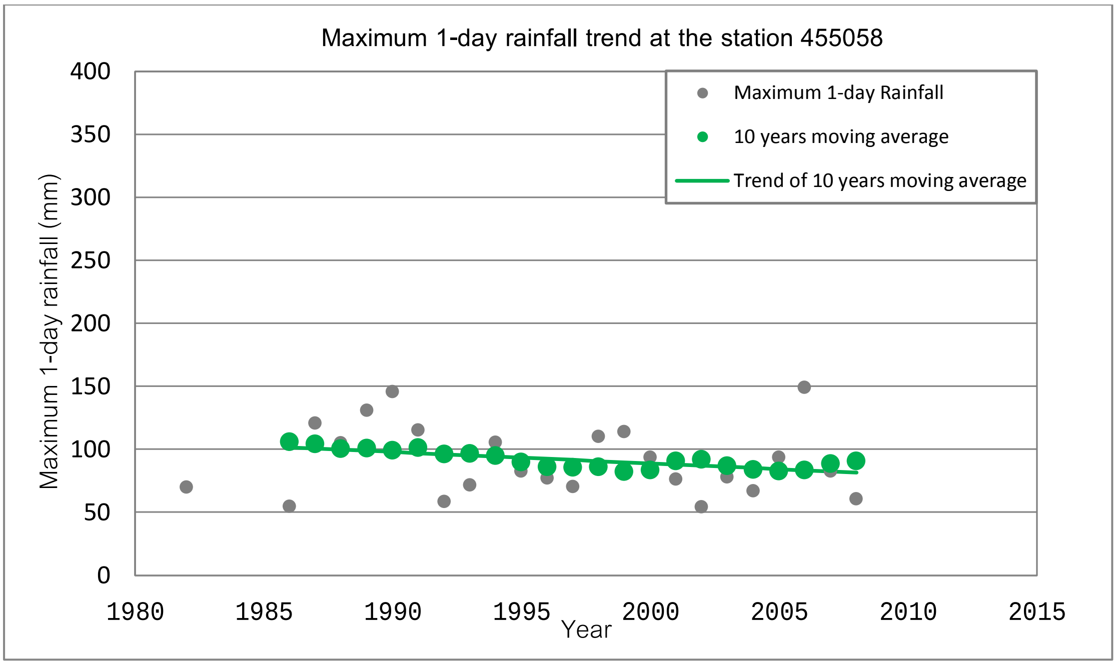


Figure C-13 Maximum 1-day rainfall trend at the station 455058

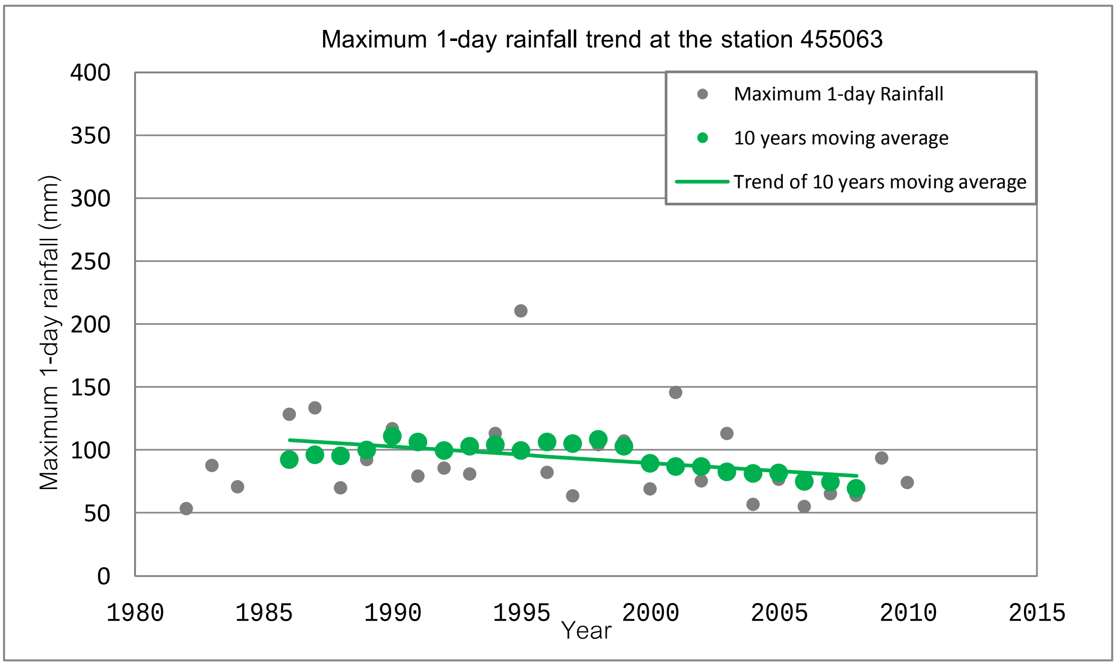


Figure C-14 Maximum 1-day rainfall trend at the station 455063

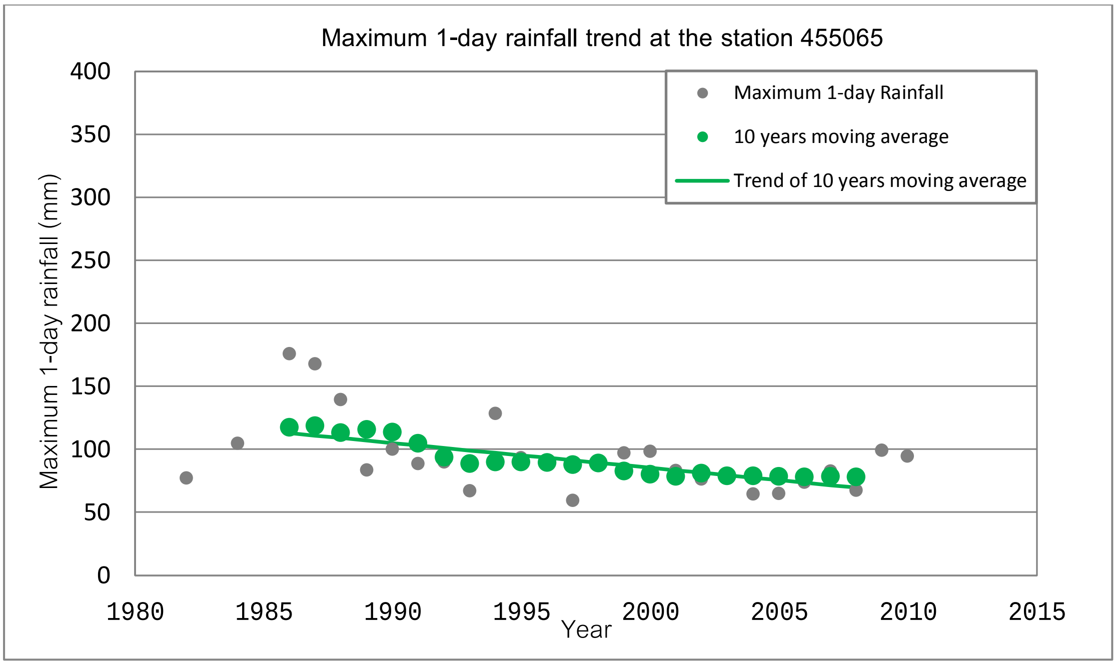


Figure C-15 Maximum 1-day rainfall trend at the station 455065

APPENDIX D

FREQUENCY CURVE

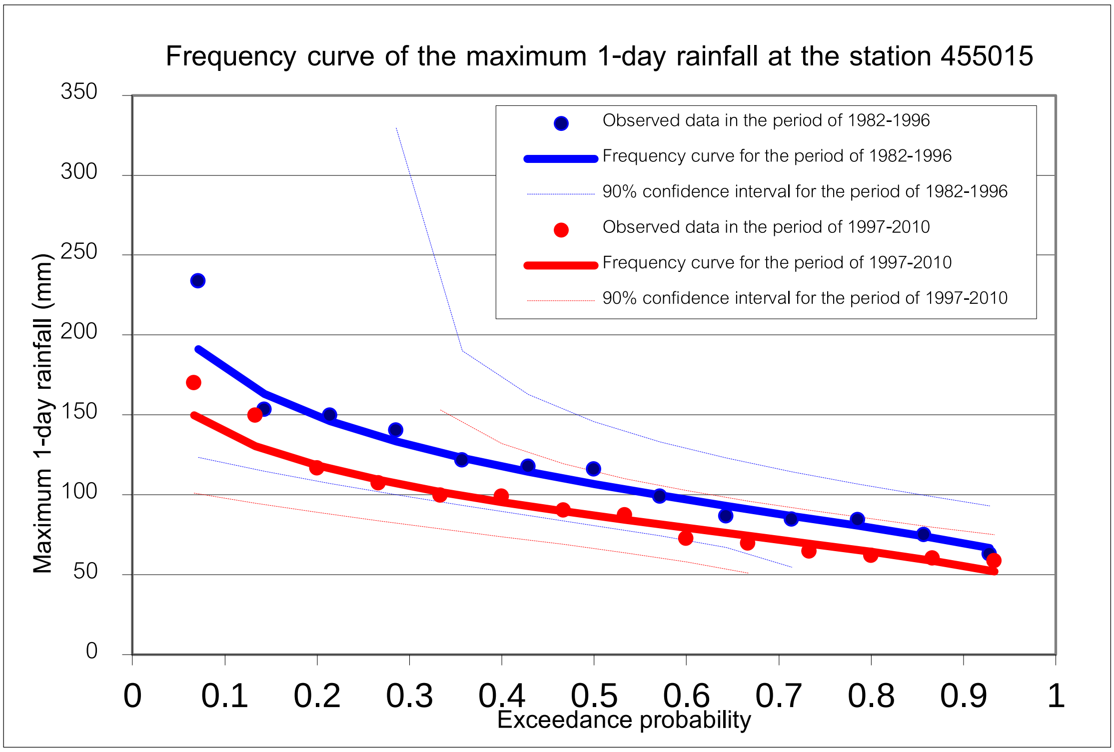


Figure D-1 Frequency curve of the maximum 1-day rainfall at the station 455015

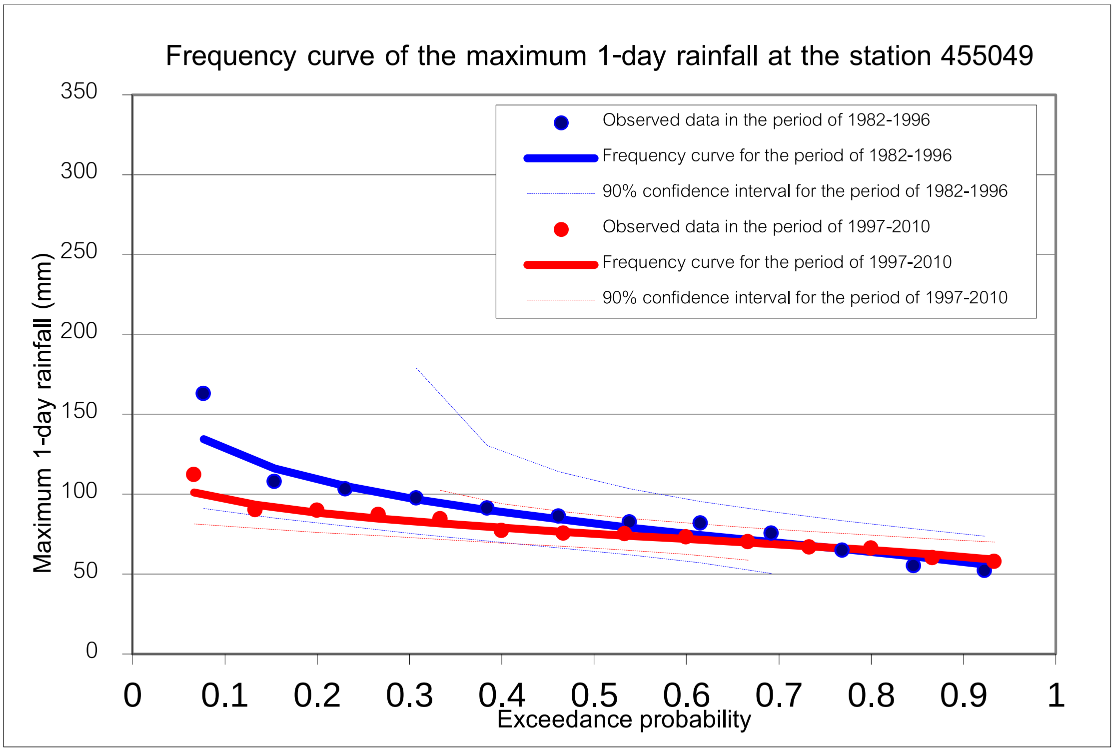


Figure D-2 Frequency curve of the maximum 1-day rainfall at the station 455049

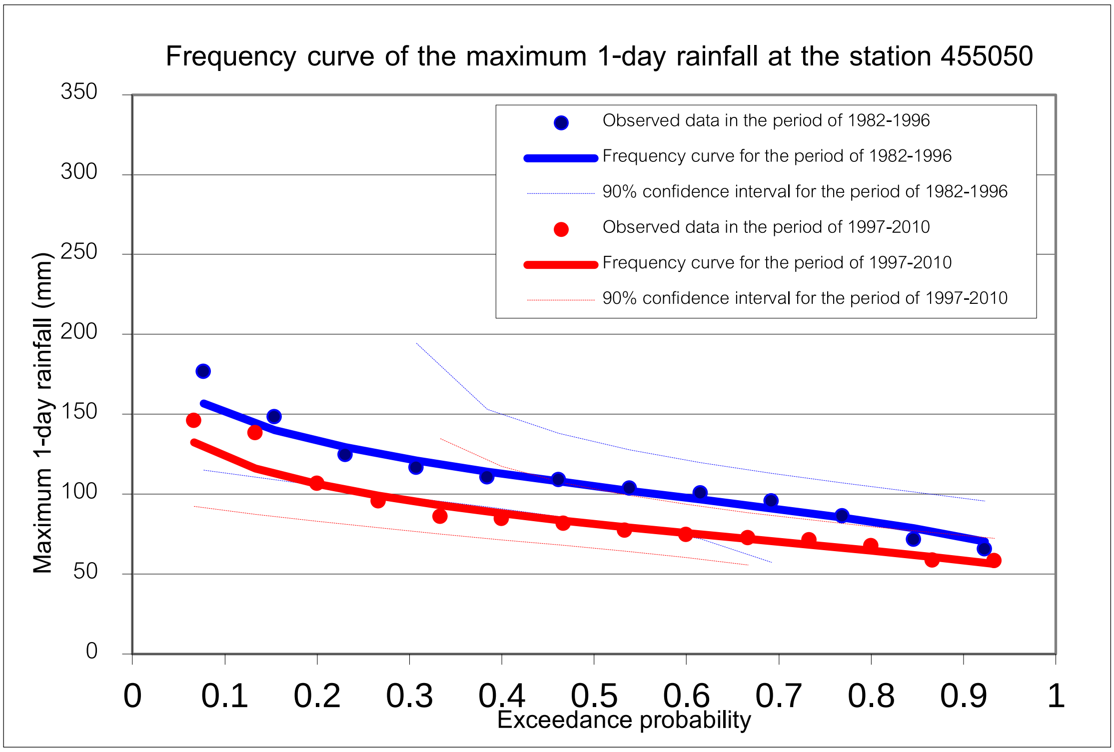


Figure D-3 Frequency curve of the maximum 1-day rainfall at the station 455050

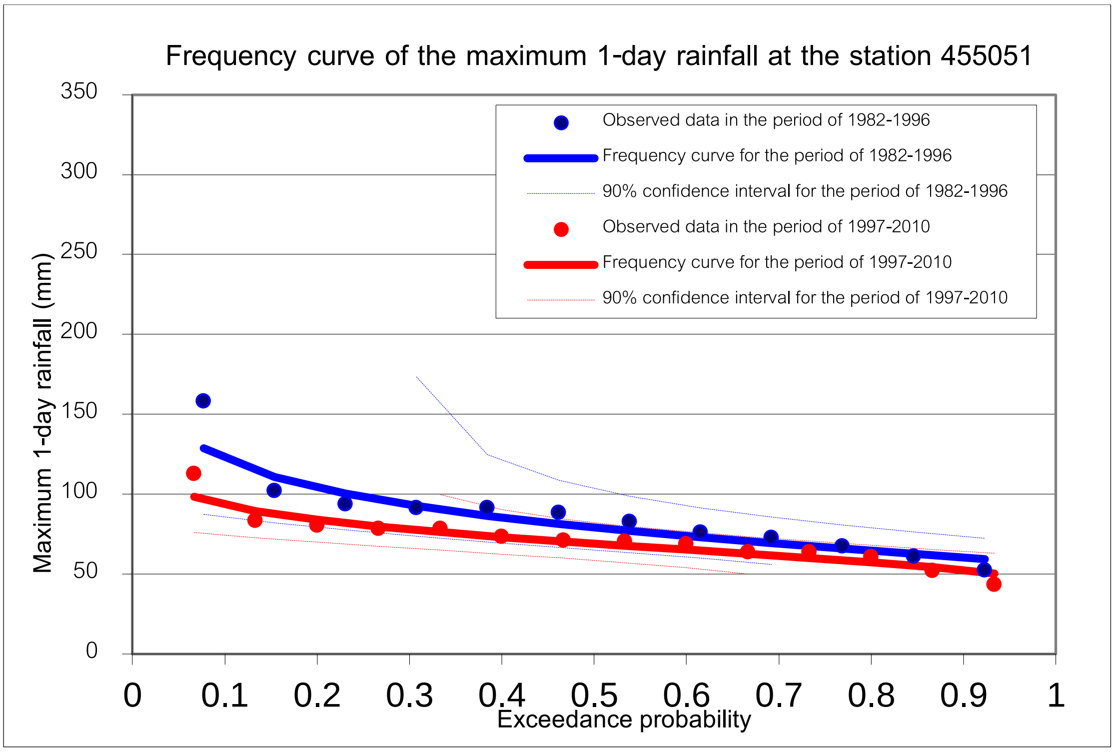


Figure D-4 Frequency curve of the maximum 1-day rainfall at the station 455051

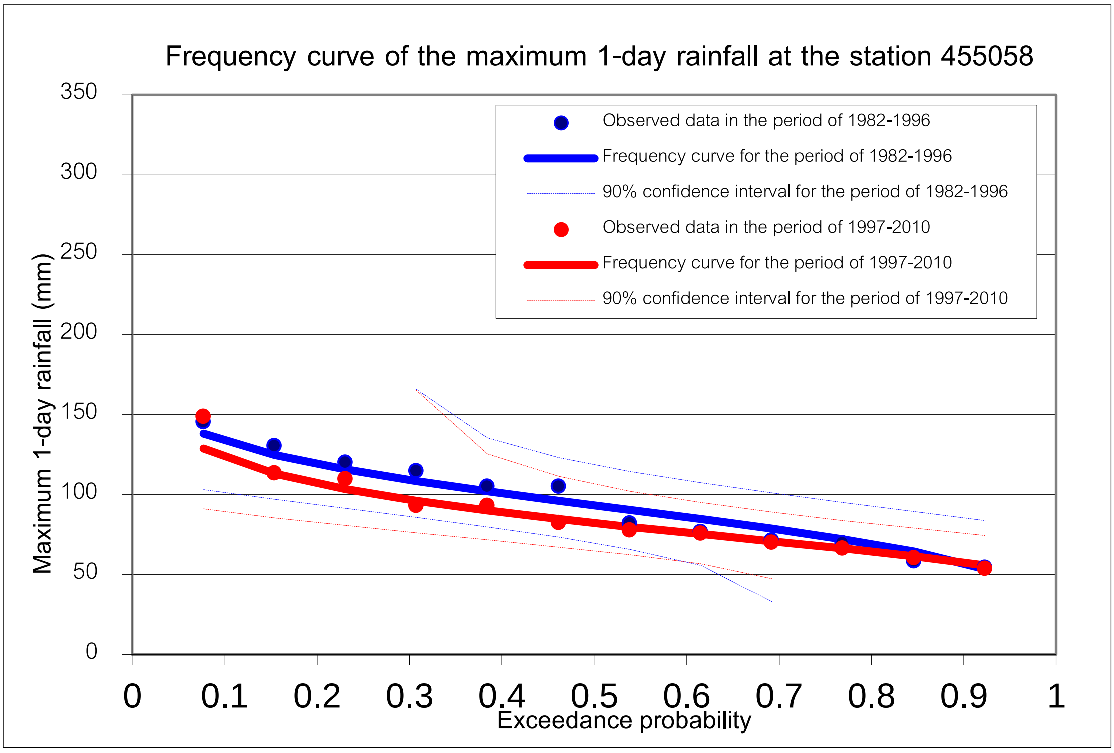


Figure D-5 Frequency curve of the maximum 1-day rainfall at the station 455058

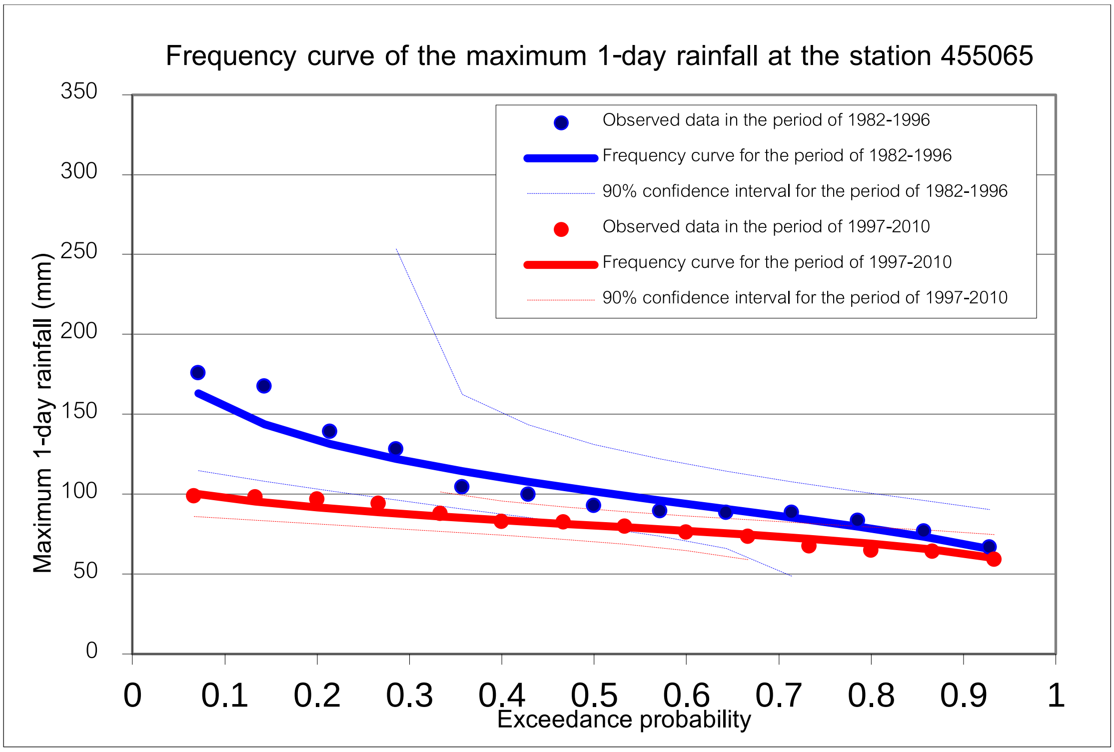
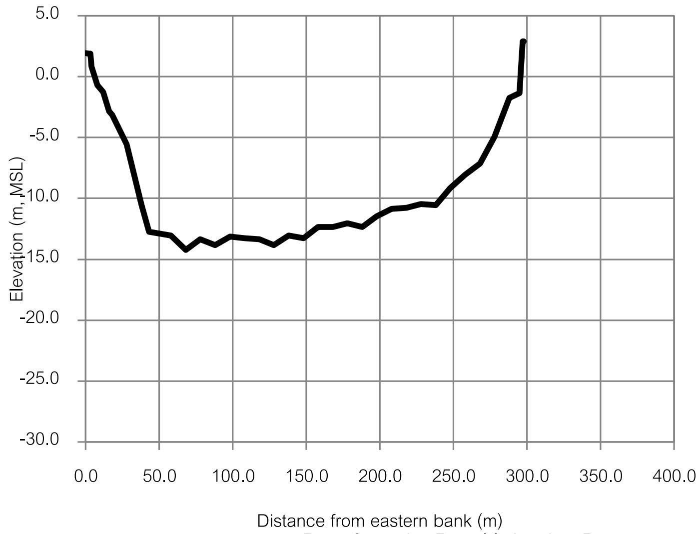


Figure D-6 Frequency curve of the maximum 1-day rainfall at the station 455065

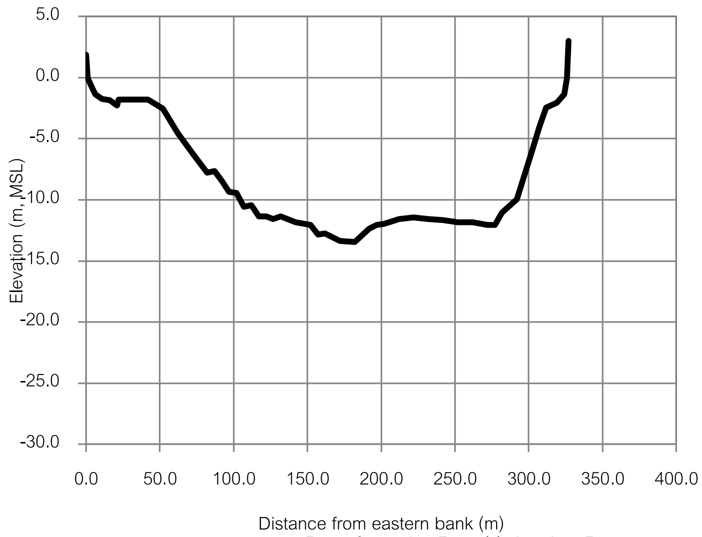
APPENDIX E

CROSS-SECTIONS OF THE CHAO PHRAYA RIVER



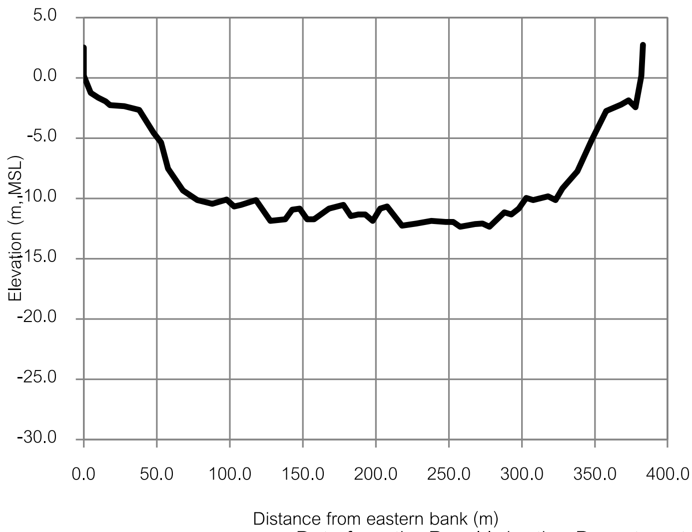
Data from the Royal Irrigation Department

Figure E-1 Cross-section of the Chao Phraya River at an easting of 663277 and a northing of 1525273



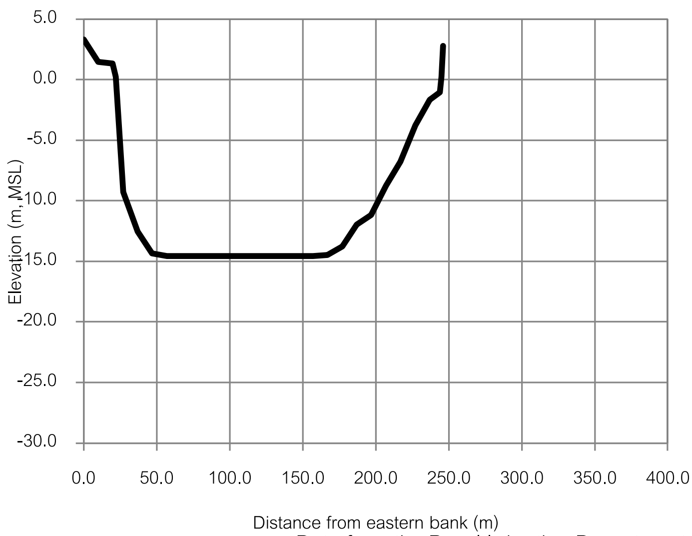
Data from the Royal Irrigation Department

Figure E-2 Cross-section of the Chao Phraya River at an easting of 662621 and a northing of 1524561



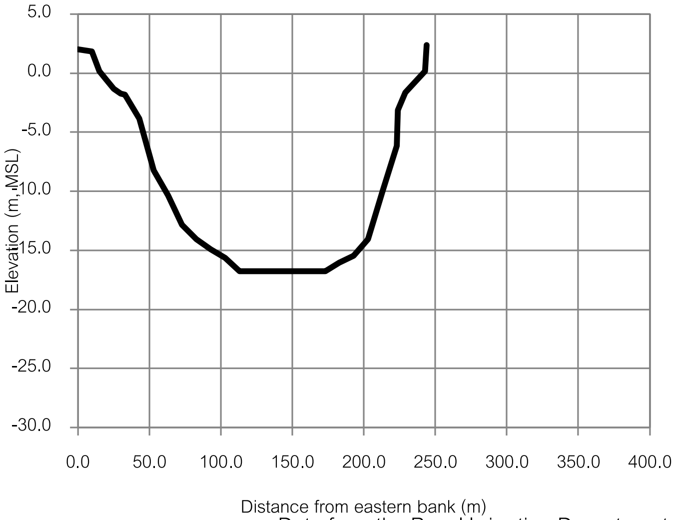
Data from the Royal Irrigation Department

Figure E-3 Cross-section of the Chao Phraya River at an easting of 662231 and a northing of 1523634



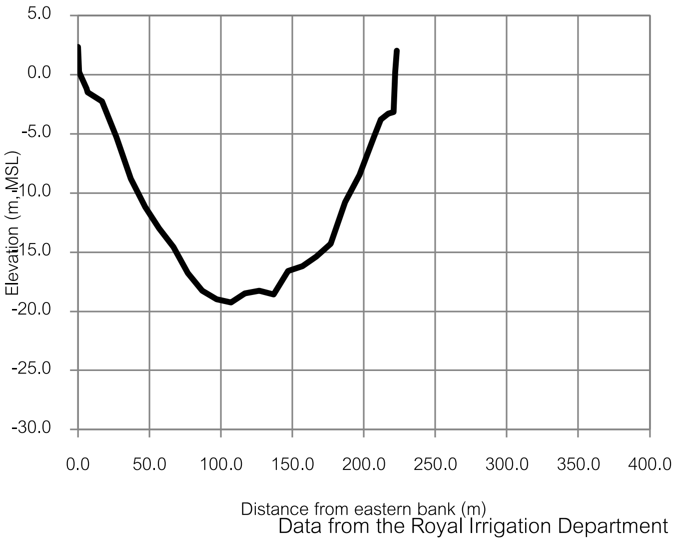
Data from the Royal Irrigation Department

Figure E-4 Cross-section of the Chao Phraya River at an easting of 661849 and a northing of 1522674



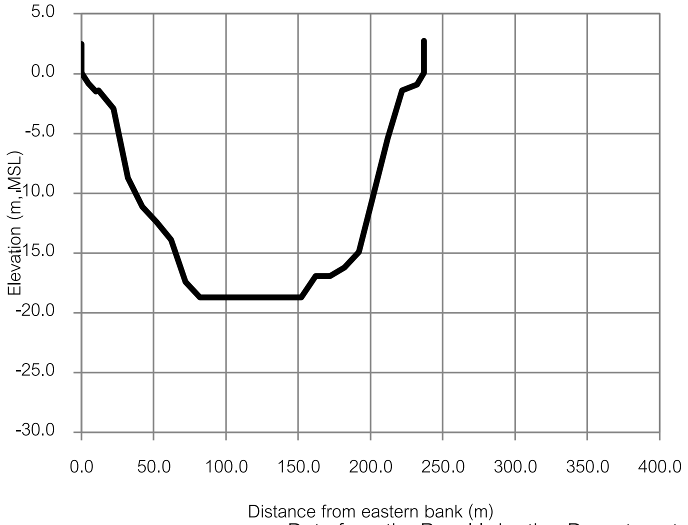
Data from the Royal Irrigation Department

Figure E-5 Cross-section of the Chao Phraya River at an easting of 661172 and a northing of 1521917



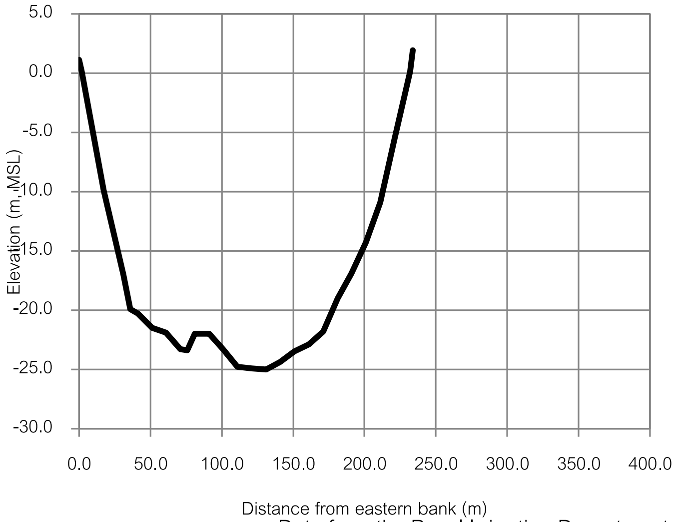
Data from the Royal Irrigation Department

Figure E-6 Cross-section of the Chao Phraya River at an easting of 660807 and a northing of 1521014



Data from the Royal Irrigation Department

Figure E-7 Cross-section of the Chao Phraya River at an easting of 660986 and a northing of 1520062



Data from the Royal Irrigation Department

Figure E-8 Cross-section of the Chao Phraya River at an easting of 661757 and a northing of 1519369

APPENDIX F

RAINFALL DIMENSIONLESS MASS CURVE

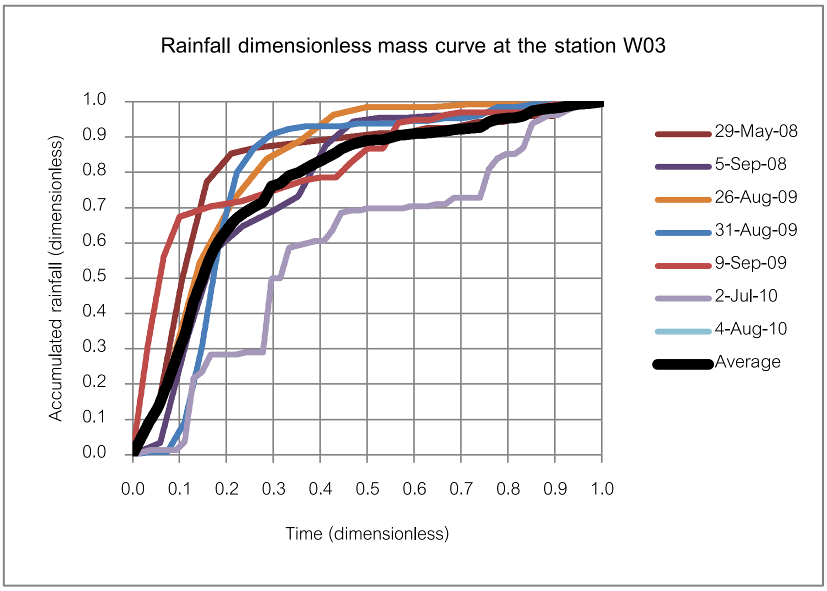


Figure F-1 Rainfall dimensionless mass curve at the station W03

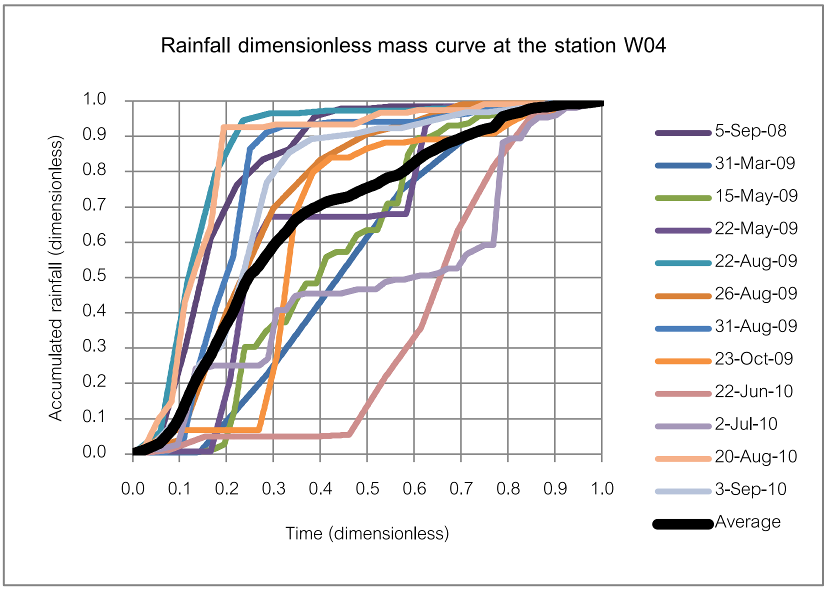


Figure F-2 Rainfall dimensionless mass curve at the station W04

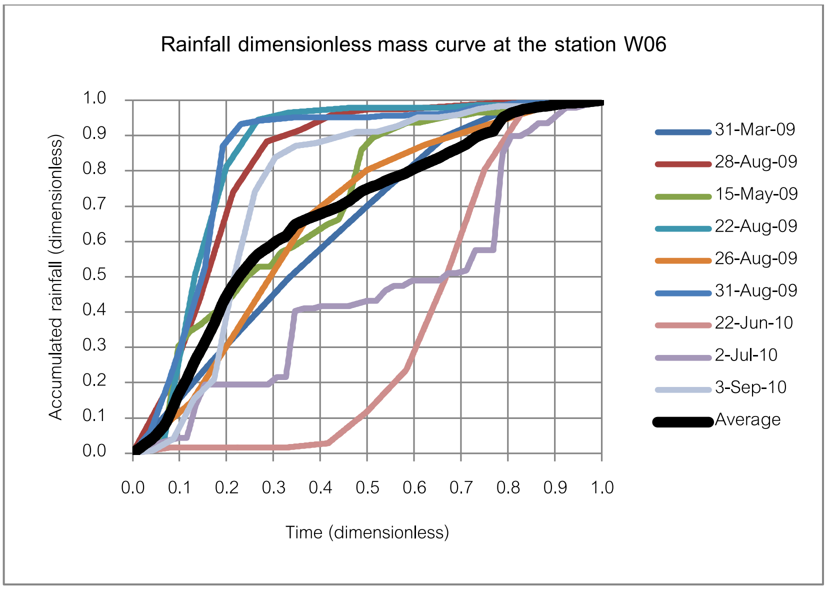


Figure F-3 Rainfall dimensionless mass curve at the station W06

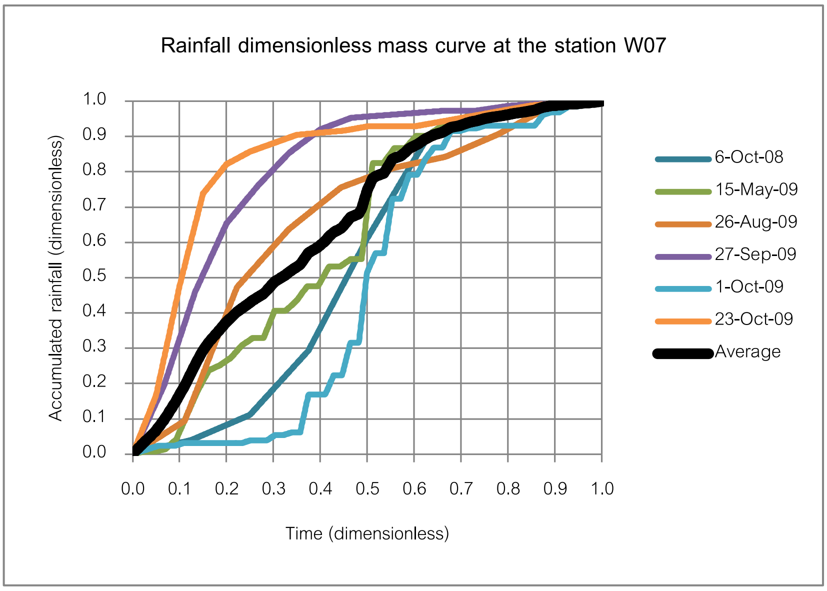


Figure F-4 Rainfall dimensionless mass curve at the station W07

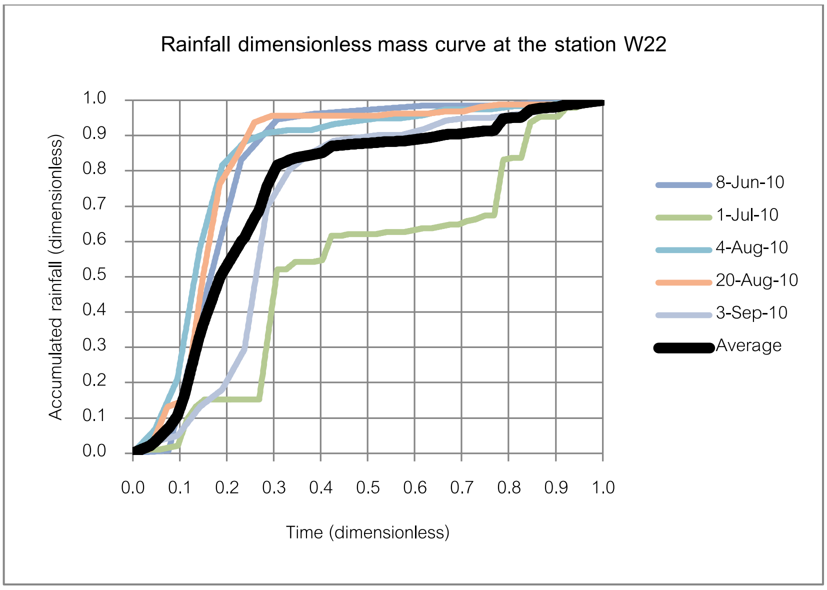


Figure F-5 Rainfall dimensionless mass curve at the station W22

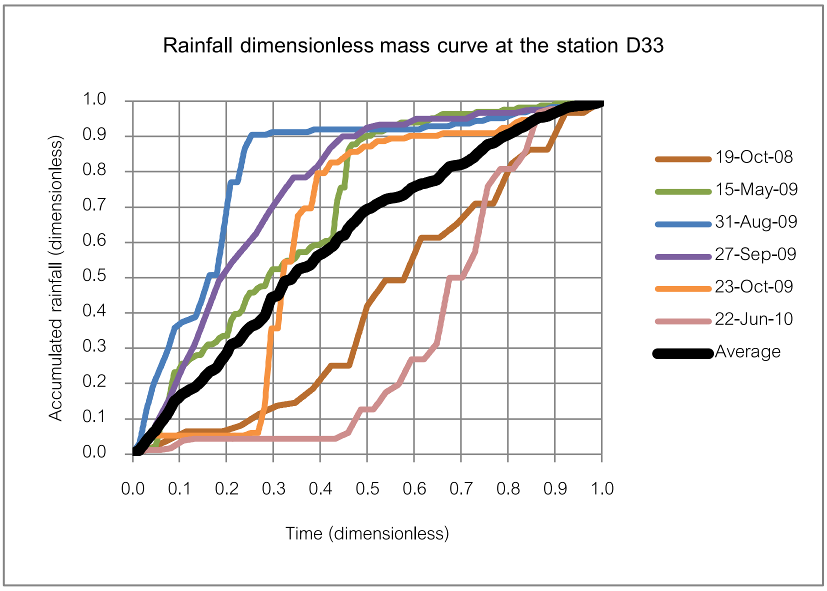


Figure F-6 Rainfall dimensionless mass curve at the station D33

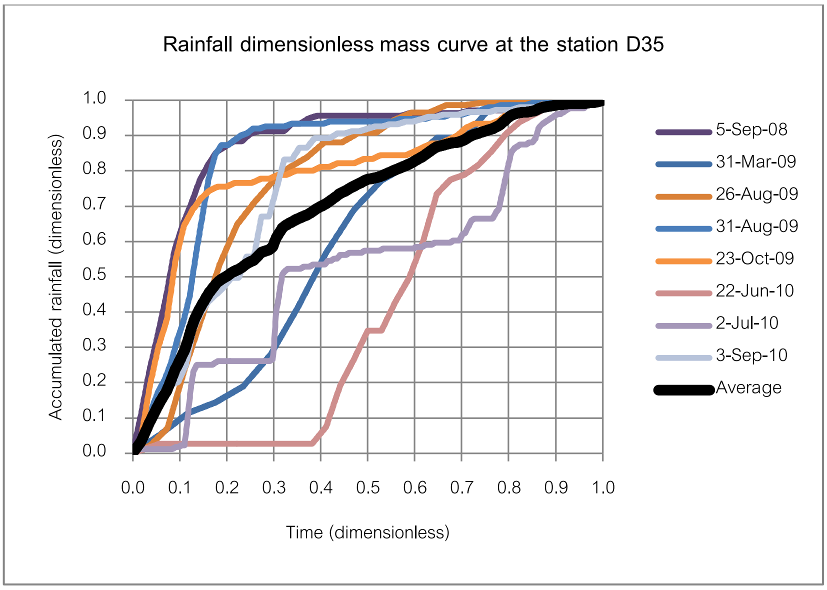


Figure F-7 Rainfall dimensionless mass curve at the station D35

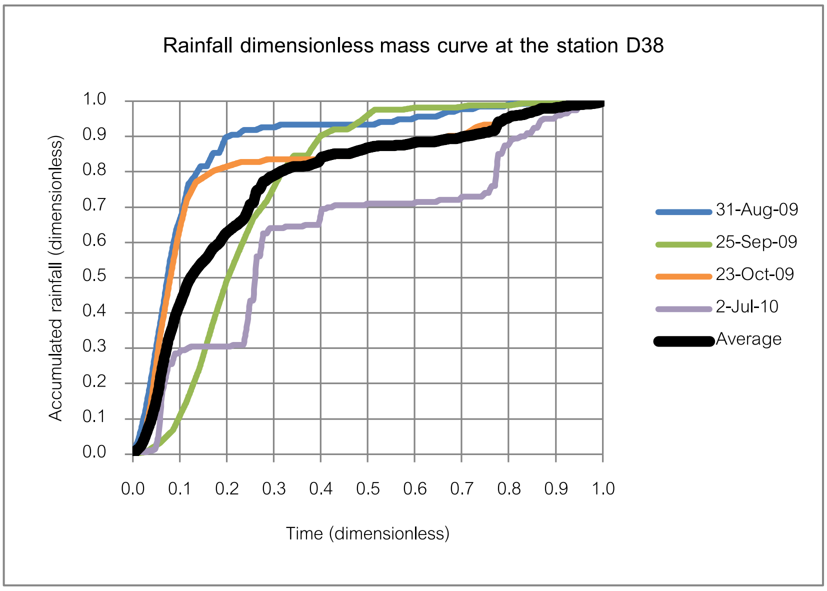


Figure F-8 Rainfall dimensionless mass curve at the station D38

APPENDIX G

WATER LEVEL DURING RAINFALL EVENT

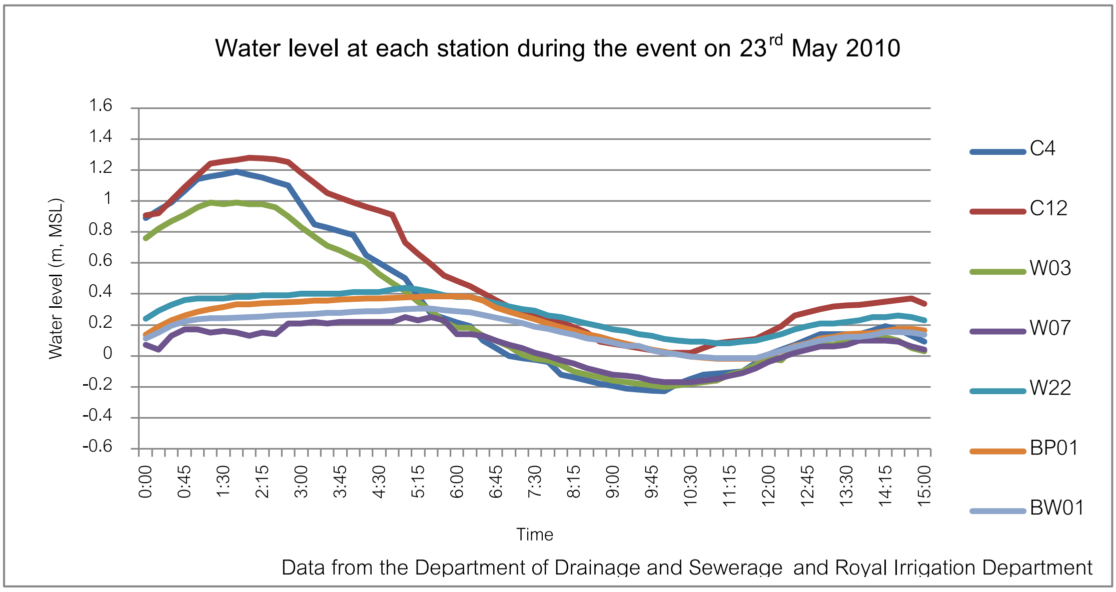


Figure G-1 Water level at each station during the event on 23rd May 2010

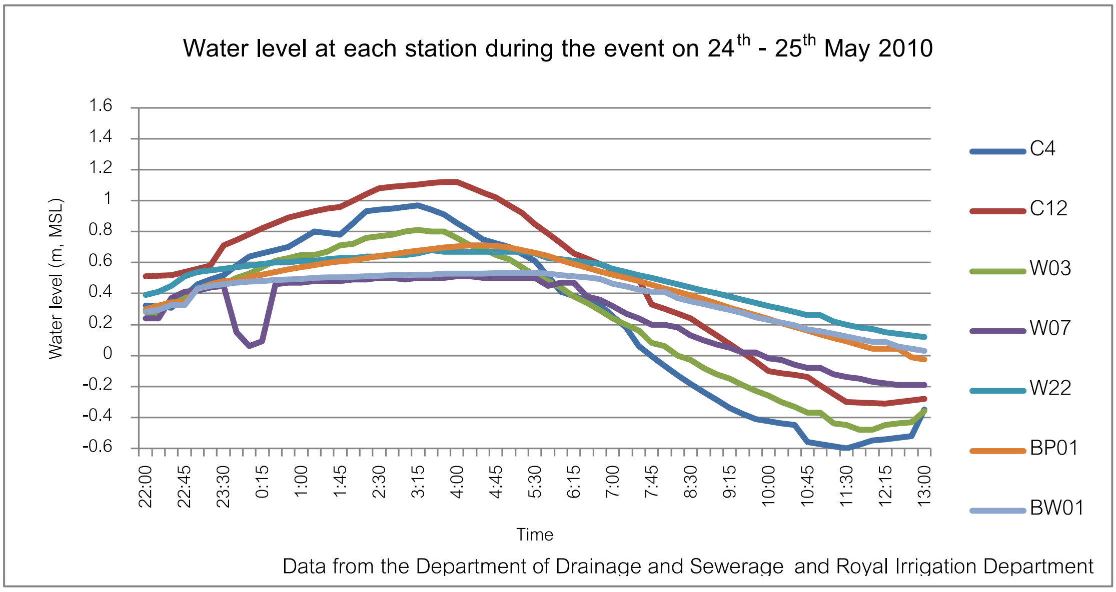


Figure G-2 Water level at each station during the event on 24th – 25th May 2010

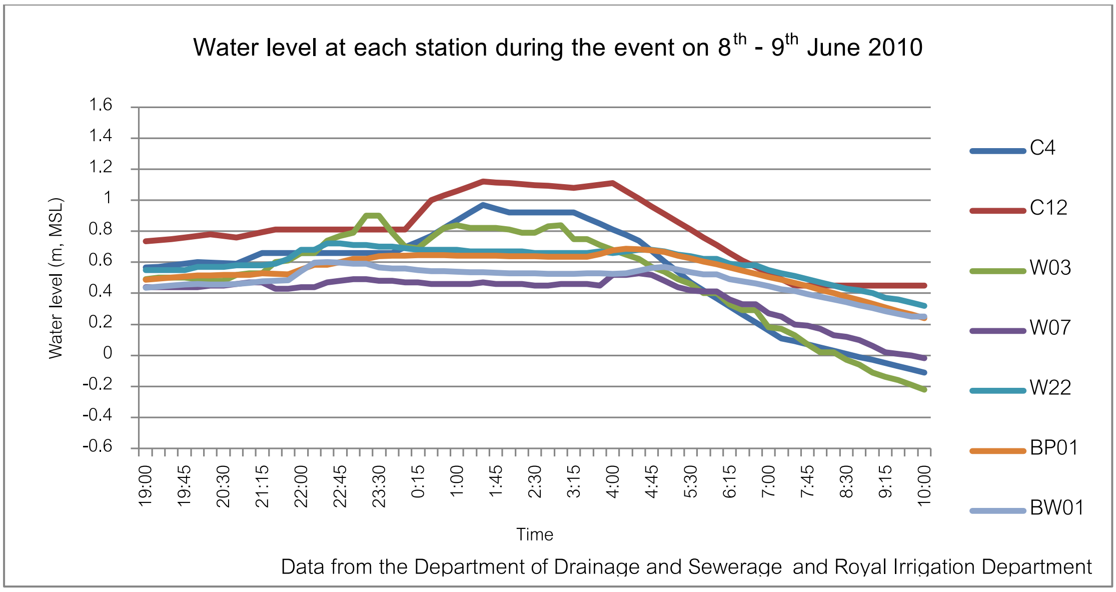


Figure G-3 Water level at each station during the event on 8^{th} – 9^{th} June 2010

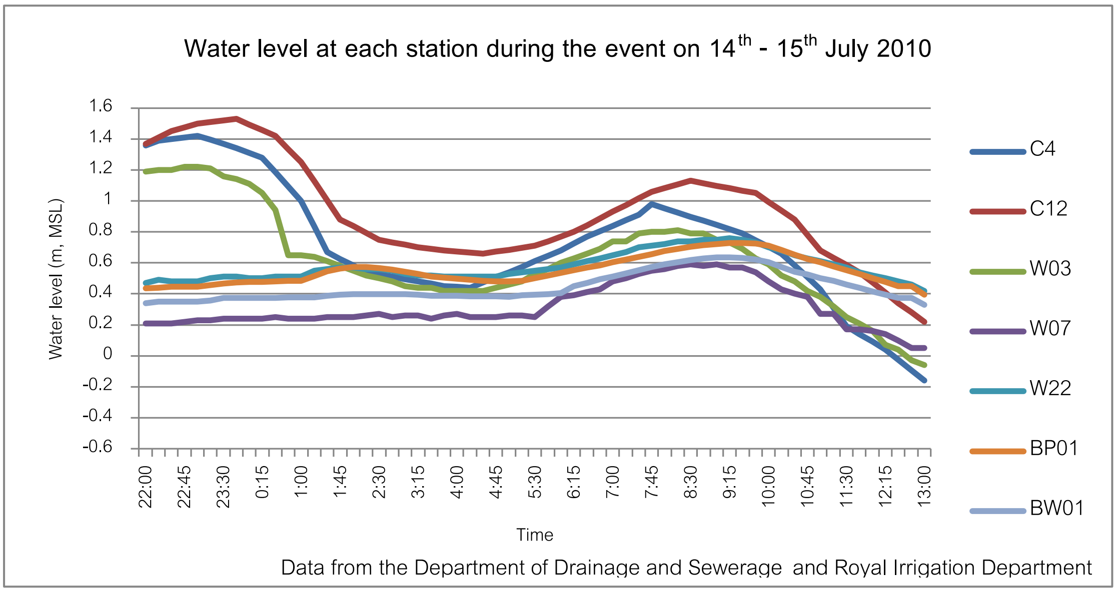


Figure G-4 Water level at each station during the event on 14^{th} – 15^{th} July 2010

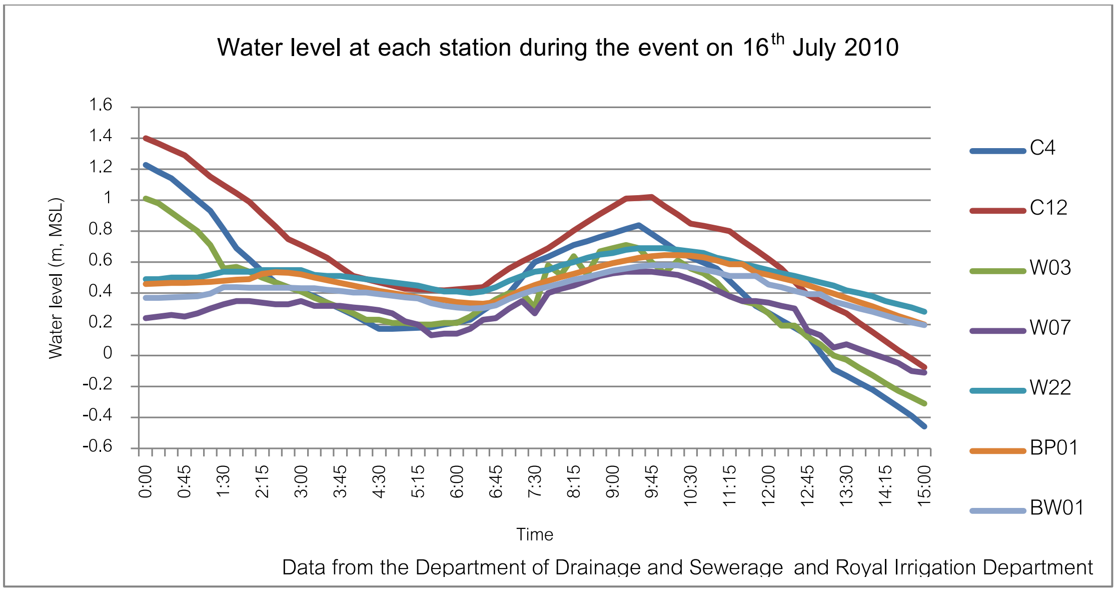


Figure G-5 Water level at each station during the event on 16th July 2010

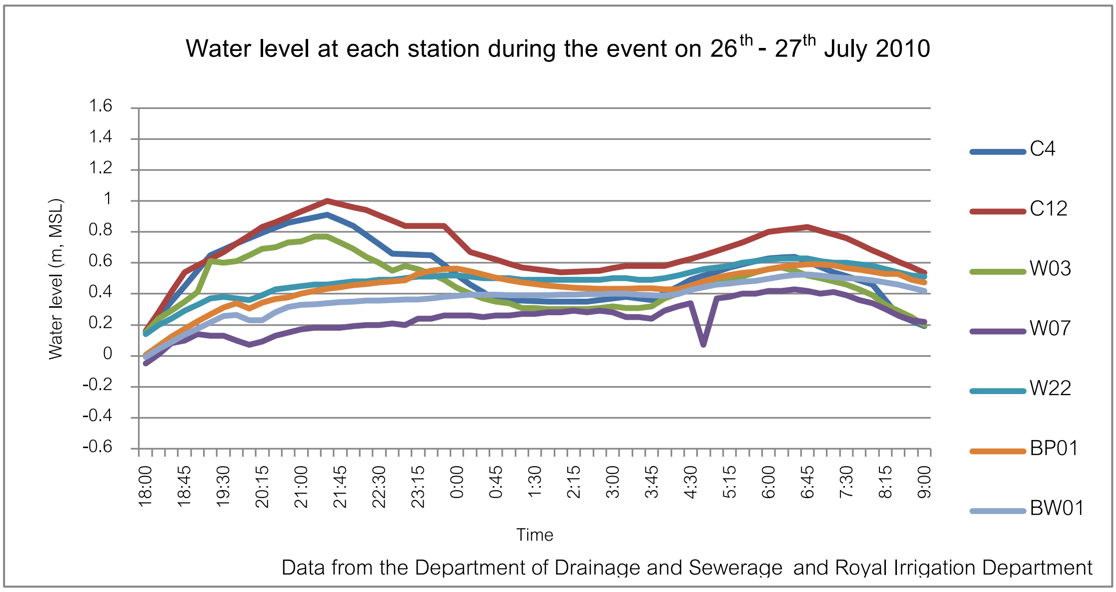


Figure G-6 Water level at each station during the event on 26th – 27th July 2010

APPENDIX H

RAINFALL DATA

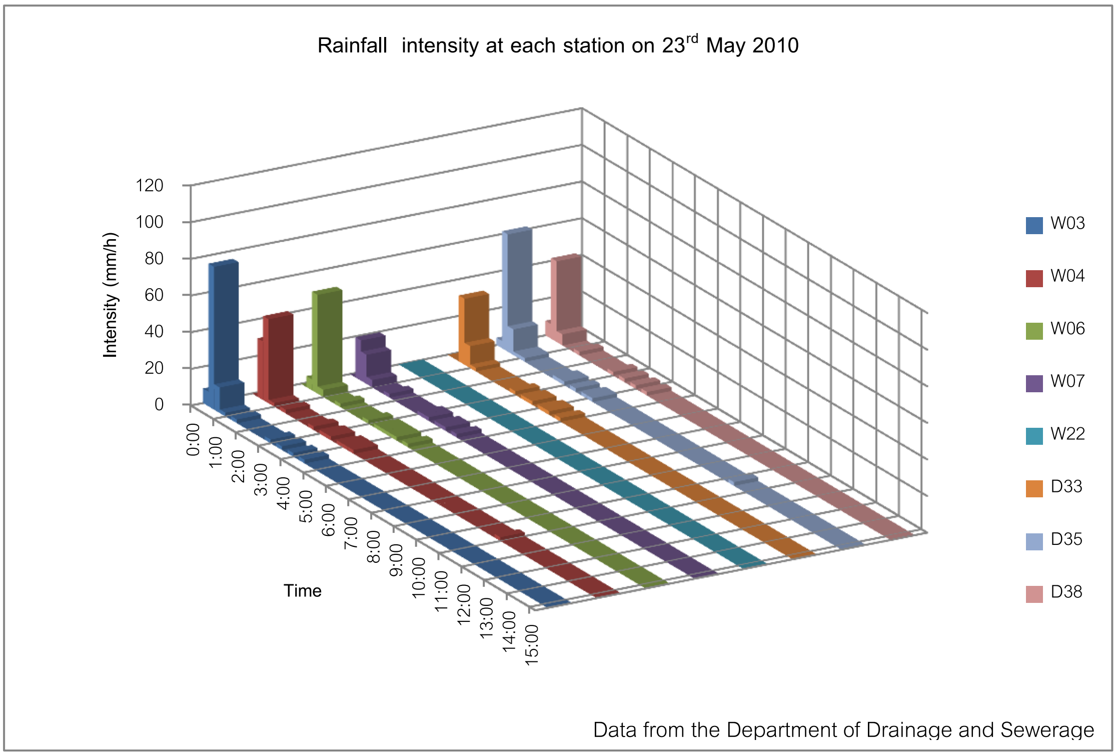


Figure H-1 Rainfall intensity at each station on 23rd May 2010

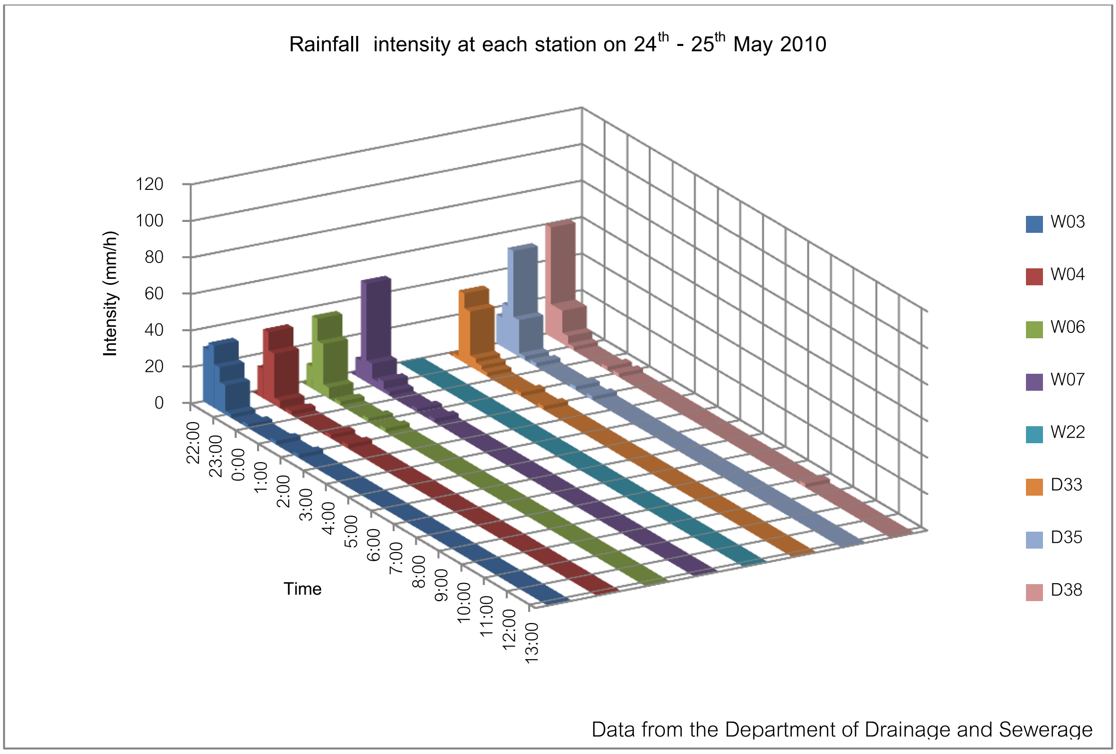


Figure H-2 Rainfall intensity at each station on 24^{th} – 25^{th} May 2010

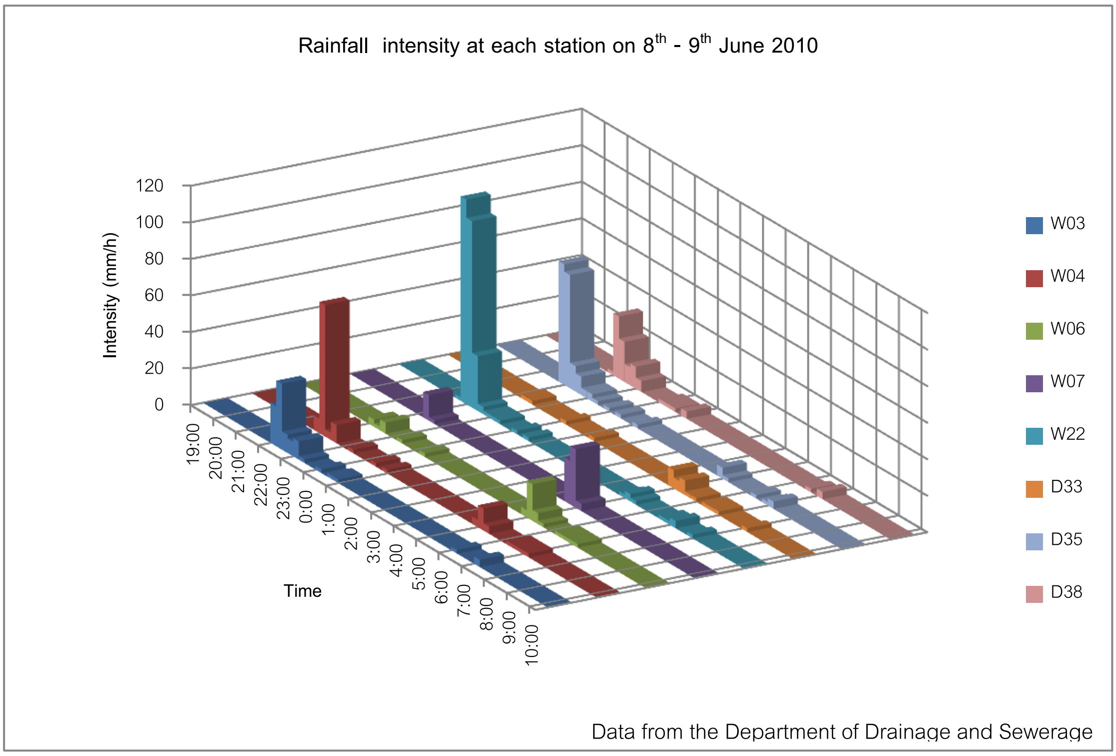


Figure H-3 Rainfall intensity at each station on 8^{th} – 9^{th} June 2010

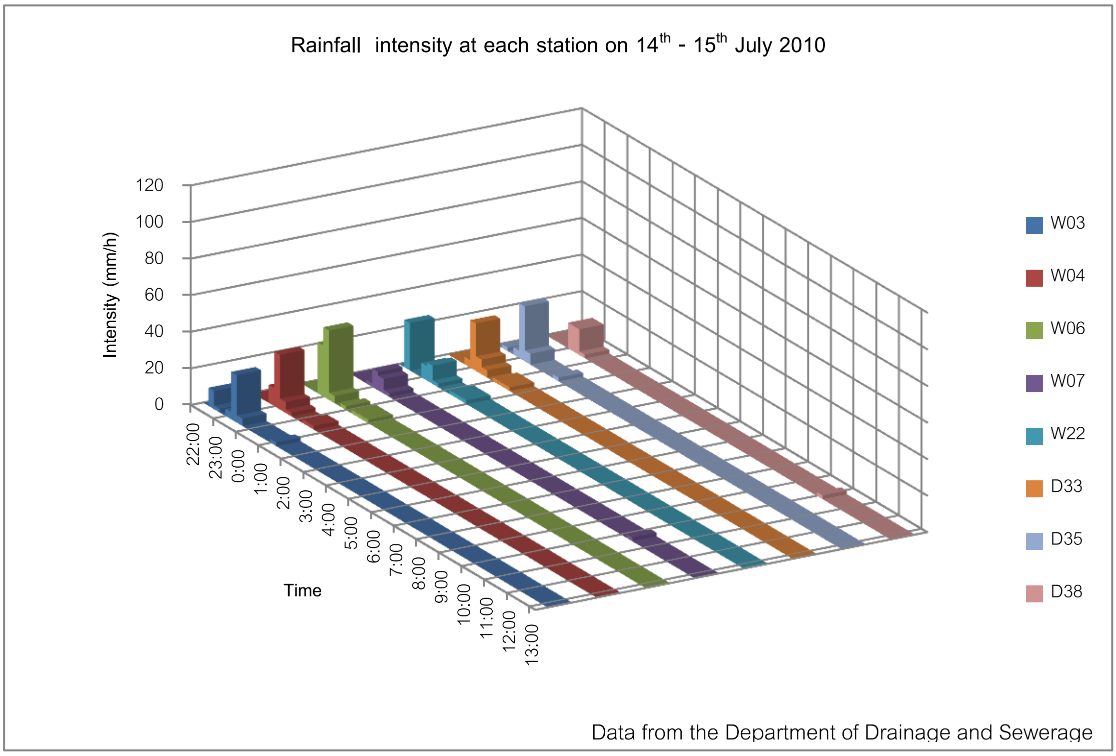


Figure H-4 Rainfall intensity at each station on 14th – 15th July 2010

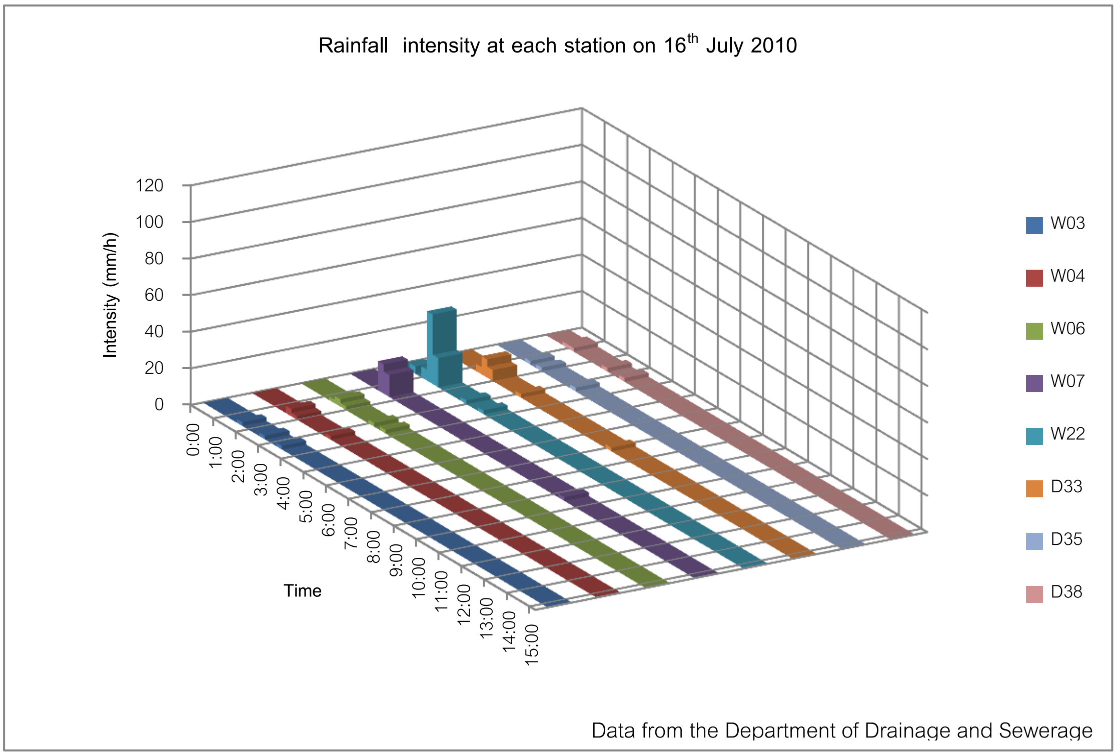


Figure H-5 Rainfall intensity at each station on 16th July 2010

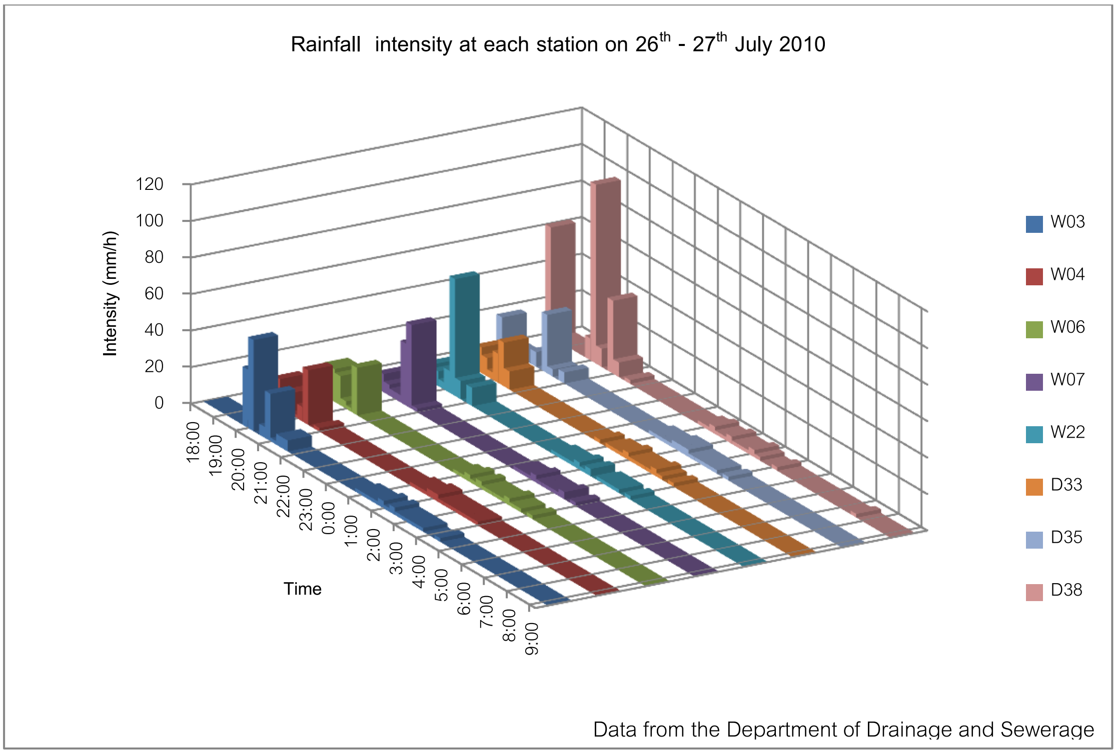


Figure H-6 Rainfall intensity at each station on 26th – 27th July 2010

APPENDIX I

FLOODGATE AND PUMP OPERATION

Table I-1 Floodgates operation and pumping (m^3/s) during the event on 23rd May 2010

Inner Channel	Chakphra		Mon		Bangkok Yai		Jaoarm		Pawana		Bangkhunnon		Jakthong		Wat Rakang		Wat Arun	
Outer Channel	Bangkok Noi		Chaopraya		Chaopraya		Bangkok Noi		Bangkok Noi		Bangkok Noi		Bangkok Noi		Chaopraya		Chaopraya	
Time	Gate	Pump	Gate	Pump	Gate	Pump	Gate	Pump	Gate	Pump	Gate	Pump	Gate	Pump	Gate	Pump	Gate	Pump
0:00	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Open	-	Closed	2	Closed	-	Closed	-
0:15	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Open	-	Closed	2	Closed	-	Closed	-
0:30	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Open	-	Closed	2	Closed	-	Closed	-
0:45	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Open	-	Closed	2	Closed	-	Closed	-
1:00	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Open	-	Closed	2	Closed	-	Closed	-
1:15	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Open	-	Closed	2	Closed	-	Closed	-
1:30	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Open	-	Closed	2	Closed	-	Closed	-
1:45	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Open	-	Closed	2	Closed	-	Closed	-
2:00	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Open	-	Closed	-	Closed	-	Closed	-
2:15	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Open	-	Closed	-	Closed	-	Closed	-
2:30	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Open	-	Closed	-	Closed	-	Closed	-
2:45	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Open	-	Closed	-	Closed	-	Closed	-
3:00	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Open	-	Closed	-	Closed	-	Closed	-
3:15	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Open	-	Closed	-	Closed	-	Closed	-
3:30	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Open	-	Closed	-	Closed	-	Closed	-
3:45	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Open	-	Closed	-	Closed	-	Closed	-
4:00	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Open	-	Closed	-	Closed	-	Closed	-
4:15	Closed	-	Closed	-	Closed	-	Closed	-	Open	-	Open	-	Open	-	Closed	-	Closed	-
4:30	Closed	-	Closed	-	Closed	-	Closed	-	Open	-	Open	-	Open	-	Closed	-	Closed	-
4:45	Closed	-	Open	-	Closed	-	Closed	-	Open	-	Open	-	Open	-	Closed	-	Open	-
5:00	Closed	-	Open	-	Closed	-	Closed	-	Open	-	Open	-	Open	-	Closed	-	Open	-
5:15	Closed	-	Open	-	Closed	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
5:30	Closed	-	Open	-	Closed	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
5:45	Closed	-	Open	-	Closed	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
6:00	Closed	-	Open	-	Closed	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
6:15	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
6:30	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
6:45	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
7:00	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
7:15	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
7:30	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
7:45	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
8:00	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
8:15	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
8:30	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
8:45	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
9:00	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
9:15	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
9:30	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
9:45	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
10:00	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
10:15	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
10:30	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
10:45	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
11:00	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
11:15	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
11:30	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
11:45	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
12:00	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
12:15	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
12:30	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
12:45	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
13:00	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
13:15	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
13:30	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
13:45	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
14:00	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
14:15	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
14:30	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
14:45	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
15:00	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-

Data from the Department of Drainage and Sewerage

Table I-2 Floodgates operation and pumping (m^3/s) during the event on 24th – 25th May 2010

Inner Channel	Chakphra		Mon		Bangkok Yai		Jaoarm		Pawana		Bangkhunnon		Jakthong		Wat Rakang		Wat Arun	
Outer Channel	Bangkok Noi		Chaopraya		Chaopraya		Bangkok Noi		Bangkok Noi		Bangkok Noi		Bangkok Noi		Chaopraya		Chaopraya	
Time	Gate	Pump	Gate	Pump	Gate	Pump	Gate	Pump	Gate	Pump	Gate	Pump	Gate	Pump	Gate	Pump	Gate	Pump
22:00	Open	-	Closed	-	Closed	-	Open	-	Open	-	Open	-	Open	-	Open	-	Closed	-
22:15	Open	-	Closed	-	Closed	-	Open	-	Open	-	Open	-	Open	-	Open	-	Closed	-
22:30	Open	-	Closed	-	Closed	-	Open	-	Open	-	Open	-	Open	-	Open	-	Closed	-
22:45	Open	-	Closed	-	Closed	-	Open	-	Open	-	Open	-	Open	-	Closed	-	Closed	-
23:00	Open	-	Closed	-	Closed	-	Open	-	Open	-	Open	-	Open	-	Closed	-	Closed	-
23:15	Open	-	Closed	-	Closed	-	Open	-	Open	-	Open	-	Open	-	Closed	-	Closed	-
23:30	Open	-	Closed	-	Closed	-	Open	-	Open	-	Open	-	Open	-	Closed	-	Closed	-
23:45	Open	-	Closed	-	Closed	-	Open	-	Open	-	Open	-	Open	-	Closed	-	Closed	-
0:00	Open	-	Closed	-	Closed	-	Open	-	Open	-	Open	-	Closed	-	Closed	-	Closed	-
0:15	Open	-	Closed	-	Closed	-	Open	-	Open	-	Open	-	Closed	-	Closed	-	Closed	-
0:30	Open	-	Closed	-	Closed	-	Open	-	Open	-	Open	-	Closed	-	Closed	-	Closed	-
0:45	Open	-	Closed	-	Closed	-	Open	-	Open	-	Open	-	Closed	-	Closed	-	Closed	-
1:00	Open	-	Closed	-	Closed	-	Closed	-	Closed	-	Open	-	Closed	-	Closed	-	Closed	-
1:15	Open	-	Closed	-	Closed	-	Closed	-	Closed	-	Open	-	Closed	-	Closed	-	Closed	-
1:30	Open	-	Closed	-	Closed	-	Closed	-	Closed	-	Open	-	Closed	-	Closed	-	Closed	-
1:45	Open	-	Closed	-	Closed	-	Closed	-	Closed	-	Open	-	Closed	-	Closed	-	Closed	-
2:00	Open	-	Closed	-	Closed	-	Closed	-	Closed	-	Open	-	Closed	-	Closed	-	Closed	-
2:15	Open	-	Closed	-	Closed	-	Closed	-	Closed	-	Open	-	Closed	-	Closed	-	Closed	-
2:30	Open	-	Closed	-	Closed	-	Closed	-	Closed	-	Open	-	Closed	-	Closed	-	Closed	-
2:45	Open	-	Closed	-	Closed	-	Closed	-	Closed	-	Open	-	Closed	-	Closed	-	Closed	-
3:00	Open	-	Closed	-	Closed	-	Closed	-	Closed	-	Open	-	Closed	-	Closed	-	Closed	-
3:15	Open	-	Closed	-	Closed	-	Closed	-	Closed	-	Open	-	Closed	-	Closed	-	Closed	-
3:30	Open	-	Closed	-	Closed	-	Closed	-	Closed	-	Open	-	Closed	-	Closed	-	Closed	-
3:45	Open	-	Closed	-	Closed	-	Closed	-	Closed	-	Open	-	Closed	-	Closed	-	Closed	-
4:00	Open	-	Closed	-	Closed	-	Closed	-	Closed	-	Open	-	Closed	-	Closed	-	Closed	-
4:15	Open	-	Closed	-	Closed	-	Closed	-	Closed	-	Open	-	Closed	-	Closed	-	Closed	-
4:30	Open	-	Closed	-	Closed	-	Closed	-	Closed	-	Open	-	Closed	-	Closed	-	Closed	-
4:45	Open	-	Closed	-	Closed	-	Closed	-	Closed	-	Open	-	Closed	-	Closed	-	Closed	-
5:00	Open	-	Closed	-	Closed	-	Closed	-	Closed	-	Open	-	Closed	-	Closed	-	Closed	-
5:15	Open	-	Closed	-	Closed	-	Open	-	Open	-	Open	-	Open	-	Closed	-	Closed	-
5:30	Open	-	Closed	-	Closed	-	Open	-	Open	-	Open	-	Open	-	Closed	-	Closed	-
5:45	Open	-	Open	-	Closed	-	Open	-	Open	-	Open	-	Open	-	Closed	-	Open	-
6:00	Open	-	Open	-	Closed	-	Open	-	Open	-	Open	-	Open	-	Closed	-	Open	-
6:15	Open	-	Open	-	Closed	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
6:30	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
6:45	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
7:00	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
7:15	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
7:30	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
7:45	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
8:00	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
8:15	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
8:30	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
8:45	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
9:00	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
9:15	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
9:30	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
9:45	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
10:00	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
10:15	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
10:30	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
10:45	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
11:00	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
11:15	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
11:30	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
11:45	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
12:00	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
12:15	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
12:30	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
12:45	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
13:00	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-

Data from the Department of Drainage and Sewerage

Table I-3 Floodgates operation and pumping (m^3/s) during the event on 8^{th} – 9^{th} June 2010

Inner Channel	Chakphra		Mon		Bangkok Yai		Jaoarm		Pawana		Bangkhunnon		Jakthong		Wat Rakang		Wat Arun	
Outer Channel	Bangkok Noi		Chaopraya		Chaopraya		Bangkok Noi		Bangkok Noi		Bangkok Noi		Bangkok Noi		Chaopraya		Chaopraya	
Time	Gate	Pump	Gate	Pump	Gate	Pump	Gate	Pump	Gate	Pump	Gate	Pump	Gate	Pump	Gate	Pump	Gate	Pump
19:00	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Closed	-	Open	-	Open	-
19:15	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Closed	-	Open	-	Open	-
19:30	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Closed	-	Open	-	Open	-
19:45	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Closed	-	Open	-	Open	-
20:00	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Closed	-	Open	-	Open	-
20:15	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Closed	-	Open	-	Open	-
20:30	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Closed	-	Open	-	Open	-
20:45	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Closed	-	Open	-	Open	-
21:00	Closed	-	Open	-	Open	-	Closed	-	Open	-	Open	-	Closed	-	Open	-	Closed	-
21:15	Closed	-	Open	-	Closed	-	Closed	-	Open	-	Open	-	Closed	-	Open	-	Closed	-
21:30	Closed	-	Closed	-	Closed	-	Closed	-	Open	-	Open	-	Closed	-	Open	-	Closed	-
21:45	Closed	-	Closed	-	Closed	-	Closed	-	Open	-	Open	-	Closed	-	Open	-	Closed	-
22:00	Closed	-	Closed	-	Closed	-	Closed	-	Open	-	Open	-	Closed	-	Closed	-	Closed	-
22:15	Closed	-	Closed	-	Closed	-	Closed	-	Open	-	Open	-	Closed	-	Closed	-	Closed	-
22:30	Closed	-	Closed	-	Closed	-	Closed	-	Open	-	Open	-	Closed	-	Closed	-	Closed	-
22:45	Closed	-	Closed	-	Closed	-	Closed	-	Open	-	Open	-	Closed	-	Closed	-	Closed	-
23:00	Closed	-	Closed	-	Closed	-	Closed	-	Open	-	Open	-	Closed	-	Closed	-	Closed	-
23:15	Closed	-	Closed	-	Closed	-	Closed	-	Open	-	Open	-	Closed	-	Closed	-	Closed	-
23:30	Closed	-	Closed	-	Closed	-	Closed	-	Open	-	Open	-	Closed	-	Closed	-	Closed	-
23:45	Closed	-	Closed	-	Closed	-	Closed	-	Open	-	Open	-	Closed	-	Closed	-	Closed	-
0:00	Closed	-	Closed	-	Closed	-	Closed	-	Open	-	Open	-	Closed	-	Closed	-	Closed	-
0:15	Closed	-	Closed	-	Closed	-	Closed	-	Open	-	Open	-	Closed	-	Closed	-	Closed	-
0:30	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Open	-	Closed	-	Closed	-	Closed	-
0:45	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Open	-	Closed	-	Closed	-	Closed	-
1:00	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Open	-	Closed	-	Closed	-	Closed	-
1:15	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Open	-	Closed	-	Closed	-	Closed	-
1:30	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Open	-	Closed	-	Closed	-	Closed	-
1:45	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Open	-	Closed	-	Closed	-	Closed	-
2:00	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Open	-	Closed	-	Closed	-	Closed	-
2:15	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Open	-	Closed	-	Closed	-	Closed	-
2:30	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Open	-	Closed	-	Closed	-	Closed	-
2:45	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Open	-	Closed	-	Closed	-	Closed	-
3:00	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Open	-	Closed	-	Closed	-	Closed	-
3:15	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Open	-	Open	-	Closed	-	Closed	-
3:30	Open	-	Closed	-	Closed	-	Closed	-	Closed	-	Open	-	Open	-	Closed	-	Closed	-
3:45	Open	-	Closed	-	Closed	-	Closed	-	Closed	-	Open	-	Open	-	Closed	-	Closed	-
4:00	Open	-	Closed	-	Closed	-	Closed	-	Closed	-	Open	-	Open	-	Closed	-	Closed	-
4:15	Open	-	Closed	-	Open	-	Closed	-	Closed	-	Open	-	Open	-	Closed	-	Open	-
4:30	Open	-	Closed	-	Open	-	Closed	-	Closed	-	Open	-	Open	-	Closed	-	Open	-
4:45	Open	-	Open	-	Open	-	Closed	-	Closed	-	Open	-	Open	-	Closed	-	Open	-
5:00	Open	-	Open	-	Open	-	Closed	-	Closed	-	Open	-	Open	-	Closed	-	Open	-
5:15	Open	-	Open	-	Open	-	Open	-	Closed	-	Open	-	Open	-	Open	-	Open	-
5:30	Open	-	Open	-	Open	-	Open	-	Closed	-	Open	-	Open	-	Open	-	Open	-
5:45	Open	-	Open	-	Open	-	Open	-	Closed	-	Open	-	Open	-	Open	-	Open	-
6:00	Open	-	Open	-	Open	-	Open	-	Closed	-	Open	-	Open	-	Open	-	Open	-
6:15	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
6:30	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
6:45	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
7:00	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
7:15	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
7:30	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
7:45	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
8:00	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
8:15	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
8:30	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
8:45	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
9:00	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
9:15	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
9:30	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
9:45	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
10:00	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-

Data from the Department of Drainage and Sewerage

Table I-4 Floodgates operation and pumping (m^3/s) during the event on 14th –15th July 2010

Inner Channel	Chakphra		Mon		Bangkok Yai		Jaoarm		Pawana		Bangkhunnon		Jakthong		Wat Rakang		Wat Arun	
Outer Channel	Bangkok Noi		Chaopraya		Chaopraya		Bangkok Noi		Bangkok Noi		Bangkok Noi		Bangkok Noi		Chaopraya		Chaopraya	
Time	Gate	Pump	Gate	Pump	Gate	Pump	Gate	Pump	Gate	Pump	Gate	Pump	Gate	Pump	Gate	Pump	Gate	Pump
22:00	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Closed	2	Closed	-	Closed	-
22:15	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Closed	2	Closed	-	Closed	-
22:30	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Closed	2	Closed	-	Closed	-
22:45	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Closed	2	Closed	-	Closed	-
23:00	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Closed	0.3	Closed	2	Closed	-	Closed	-
23:15	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Closed	0.3	Closed	2	Closed	-	Closed	-
23:30	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Closed	0.3	Closed	2	Closed	-	Closed	-
23:45	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Closed	0.3	Closed	2	Closed	-	Closed	-
0:00	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Closed	2	Closed	-	Closed	-
0:15	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Closed	2	Closed	-	Closed	-
0:30	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Closed	2	Closed	-	Closed	-
0:45	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Closed	2	Closed	-	Closed	-
1:00	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-
1:15	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-
1:30	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-
1:45	Open	-	Open	-	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Open	-
2:00	Open	-	Open	-	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Open	-
2:15	Open	-	Open	-	Closed	-	Open	-	Open	-	Open	-	Closed	-	Open	-	Open	-
2:30	Open	-	Open	-	Closed	-	Open	-	Open	-	Open	-	Closed	-	Open	-	Open	-
2:45	Open	-	Open	-	Closed	-	Open	-	Open	-	Open	-	Closed	-	Open	-	Open	-
3:00	Open	-	Open	-	Closed	-	Open	-	Open	-	Open	-	Closed	-	Open	-	Open	-
3:15	Open	-	Open	-	Closed	-	Open	-	Open	-	Open	-	Closed	-	Open	-	Open	-
3:30	Open	-	Open	-	Closed	-	Open	-	Open	-	Open	-	Closed	-	Open	-	Open	-
3:45	Open	-	Open	-	Closed	-	Open	-	Open	-	Open	-	Closed	-	Open	-	Open	-
4:00	Open	-	Open	-	Closed	-	Open	-	Open	-	Open	-	Closed	-	Open	-	Open	-
4:15	Open	-	Open	-	Closed	-	Open	-	Open	-	Open	-	Closed	-	Open	-	Open	-
4:30	Open	-	Open	-	Closed	-	Open	-	Open	-	Open	-	Closed	-	Open	-	Open	-
4:45	Open	-	Open	-	Closed	-	Open	-	Open	-	Open	-	Closed	-	Open	-	Open	-
5:00	Open	-	Open	-	Closed	-	Open	-	Open	-	Open	-	Closed	-	Open	-	Open	-
5:15	Open	-	Open	-	Closed	-	Open	-	Open	-	Open	-	Closed	-	Open	-	Open	-
5:30	Open	-	Open	-	Closed	-	Open	-	Open	-	Open	-	Closed	-	Open	-	Open	-
5:45	Open	-	Open	-	Closed	-	Open	-	Open	-	Open	-	Closed	-	Open	-	Open	-
6:00	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Closed	-	Open	-	Open	-
6:15	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Closed	-	Open	-	Open	-
6:30	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Closed	-	Open	-	Open	-
6:45	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Closed	-	Open	-	Open	-
7:00	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Closed	-	Open	-	Open	-
7:15	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Closed	-	Open	-	Open	-
7:30	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Closed	-	Open	-	Open	-
7:45	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Closed	-	Open	-	Open	-
8:00	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Closed	-	Open	-	Open	-
8:15	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Closed	-	Open	-	Open	-
8:30	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Closed	-	Open	-	Open	-
8:45	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Closed	-	Open	-	Open	-
9:00	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Closed	-	Open	-	Open	-
9:15	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Closed	-	Open	-	Open	-
9:30	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Closed	-	Open	-	Open	-
9:45	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Closed	-	Open	-	Open	-
10:00	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Closed	-	Open	-	Open	-
10:15	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
10:30	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
10:45	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
11:00	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
11:15	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
11:30	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
11:45	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
12:00	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
12:15	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
12:30	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
12:45	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
13:00	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-

Data from the Department of Drainage and Sewerage

Table I-5 Floodgates operation and pumping (m^3/s) during the event on 16th July 2010

Inner Channel	Chakphra		Mon		Bangkok Yai		Jaoarm		Pawana		Bangkhunnon		Jakthong		Wat Rakang		Wat Arun	
Outer Channel	Bangkok Noi		Chaopraya		Chaopraya		Bangkok Noi		Bangkok Noi		Bangkok Noi		Bangkok Noi		Chaopraya		Chaopraya	
Time	Gate	Pump	Gate	Pump	Gate	Pump	Gate	Pump	Gate	Pump	Gate	Pump	Gate	Pump	Gate	Pump	Gate	Pump
0:00	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-
0:15	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-
0:30	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-
0:45	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-
1:00	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Closed	2	Closed	-	Closed	-
1:15	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Closed	2	Closed	-	Closed	-
1:30	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Closed	2	Closed	-	Closed	-
1:45	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Closed	2	Closed	-	Closed	-
2:00	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-
2:15	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Open	-	Closed	-
2:30	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Open	-	Closed	-
2:45	Closed	-	Open	-	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Open	-	Closed	-
3:00	Closed	-	Open	-	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Open	-	Closed	-
3:15	Closed	-	Open	-	Closed	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
3:30	Open	-	Open	-	Closed	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
3:45	Open	-	Open	-	Closed	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
4:00	Open	-	Open	-	Closed	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
4:15	Open	-	Open	-	Closed	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
4:30	Open	-	Open	-	Closed	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
4:45	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
5:00	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
5:15	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
5:30	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
5:45	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
6:00	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Closed	-	Open	-	Open	-
6:15	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Closed	-	Open	-	Open	-
6:30	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Closed	-	Open	-	Open	-
6:45	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Closed	-	Open	-	Open	-
7:00	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Closed	-	Open	-	Open	-
7:15	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Closed	-	Open	-	Open	-
7:30	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Closed	-	Open	-	Open	-
7:45	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Closed	-	Open	-	Open	-
8:00	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Closed	-	Open	-	Open	-
8:15	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Closed	-	Open	-	Open	-
8:30	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Closed	-	Open	-	Open	-
8:45	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Closed	-	Open	-	Open	-
9:00	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Closed	-	Open	-	Open	-
9:15	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Closed	-	Open	-	Open	-
9:30	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Closed	-	Open	-	Open	-
9:45	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Closed	-	Open	-	Open	-
10:00	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Closed	-	Open	-	Open	-
10:15	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Closed	-	Open	-	Open	-
10:30	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Closed	-	Open	-	Open	-
10:45	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Closed	-	Open	-	Open	-
11:00	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Closed	-	Open	-	Open	-
11:15	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
11:30	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
11:45	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
12:00	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
12:15	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
12:30	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
12:45	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
13:00	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
13:15	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
13:30	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
13:45	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
14:00	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
14:15	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
14:30	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
14:45	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
15:00	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-

Data from the Department of Drainage and Sewerage

Table I-6 Floodgates operation and pumping (m^3/s) during the event on 26^{th} – 27^{th} July 2010

Inner Channel	Chakphra		Mon		Bangkok Yai		Jaoarm		Pawana		Bangkhunnon		Jakthong		Wat Rakang		Wat Arun	
Outer Channel	Bangkok Noi		Chaopraya		Chaopraya		Bangkok Noi		Bangkok Noi		Bangkok Noi		Bangkok Noi		Chaopraya		Chaopraya	
Time	Gate	Pump	Gate	Pump	Gate	Pump	Gate	Pump	Gate	Pump	Gate	Pump	Gate	Pump	Gate	Pump	Gate	Pump
18:00	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Open	-	Closed	2	Closed	-	Closed	-
18:15	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Open	-	Closed	2	Closed	-	Closed	-
18:30	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Open	-	Closed	2	Closed	-	Closed	-
18:45	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Open	-	Closed	2	Closed	-	Closed	-
19:00	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Open	-	Closed	2	Closed	-	Closed	-
19:15	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Open	-	Closed	2	Closed	-	Closed	-
19:30	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Open	-	Closed	2	Closed	-	Closed	-
19:45	Closed	-	Closed	-	Closed	-	Closed	-	Closed	-	Open	-	Closed	2	Closed	-	Closed	-
20:00	Closed	-	Closed	-	Closed	12	Closed	-	Closed	-	Open	-	Closed	2	Closed	-	Closed	-
20:15	Closed	-	Closed	-	Closed	12	Closed	-	Closed	-	Open	-	Closed	2	Closed	-	Closed	-
20:30	Closed	-	Closed	-	Closed	12	Closed	-	Closed	-	Open	-	Closed	2	Closed	-	Closed	-
20:45	Closed	-	Closed	-	Closed	12	Closed	-	Closed	-	Open	-	Closed	2	Closed	-	Closed	-
21:00	Closed	-	Closed	-	Closed	12	Closed	-	Closed	-	Open	-	Closed	2	Closed	-	Closed	-
21:15	Closed	-	Closed	-	Closed	12	Closed	-	Closed	-	Open	-	Closed	2	Closed	-	Closed	-
21:30	Closed	-	Closed	-	Closed	12	Closed	-	Closed	-	Open	-	Closed	2	Closed	-	Closed	-
21:45	Closed	-	Closed	-	Closed	12	Closed	-	Closed	-	Open	-	Closed	2	Closed	-	Closed	-
22:00	Closed	-	Closed	-	Closed	12	Closed	-	Closed	-	Open	-	Closed	2	Closed	-	Closed	-
22:15	Closed	-	Closed	-	Closed	12	Closed	-	Closed	-	Open	-	Closed	2	Closed	-	Closed	-
22:30	Closed	-	Closed	-	Closed	12	Closed	-	Closed	-	Open	-	Closed	2	Closed	-	Closed	-
22:45	Closed	-	Closed	-	Closed	12	Closed	-	Closed	-	Open	-	Closed	2	Closed	-	Closed	-
23:00	Closed	-	Closed	-	Closed	12	Closed	-	Closed	-	Open	-	Closed	-	Closed	-	Closed	-
23:15	Open	-	Closed	-	Closed	12	Closed	-	Closed	-	Open	-	Closed	-	Closed	-	Closed	-
23:30	Open	-	Closed	-	Closed	12	Closed	-	Closed	-	Open	-	Closed	-	Closed	-	Closed	-
23:45	Open	-	Closed	-	Closed	12	Closed	-	Closed	-	Open	-	Closed	-	Closed	-	Closed	-
0:00	Open	-	Open	-	Closed	-	Closed	-	Closed	-	Open	-	Closed	-	Closed	-	Closed	-
0:15	Open	-	Open	-	Closed	-	Open	-	Open	-	Open	-	Closed	-	Open	-	Open	-
0:30	Open	-	Open	-	Closed	-	Open	-	Open	-	Open	-	Closed	-	Open	-	Open	-
0:45	Open	-	Open	-	Closed	-	Open	-	Open	-	Open	-	Closed	-	Open	-	Open	-
1:00	Open	-	Open	-	Closed	-	Open	-	Open	-	Open	-	Closed	-	Open	-	Open	-
1:15	Open	-	Open	-	Closed	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
1:30	Open	-	Open	-	Closed	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
1:45	Open	-	Open	-	Closed	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
2:00	Open	-	Open	-	Closed	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
2:15	Open	-	Open	-	Closed	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
2:30	Open	-	Open	-	Closed	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
2:45	Open	-	Open	-	Closed	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
3:00	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
3:15	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
3:30	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
3:45	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
4:00	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
4:15	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
4:30	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
4:45	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
5:00	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
5:15	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
5:30	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
5:45	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
6:00	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
6:15	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
6:30	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
6:45	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
7:00	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
7:15	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
7:30	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
7:45	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
8:00	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
8:15	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
8:30	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
8:45	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-
9:00	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-	Open	-

Data from the Department of Drainage and Sewerage

APPENDIX J

WATER LEVEL WITHOUT PROPOSED STRUCTURES

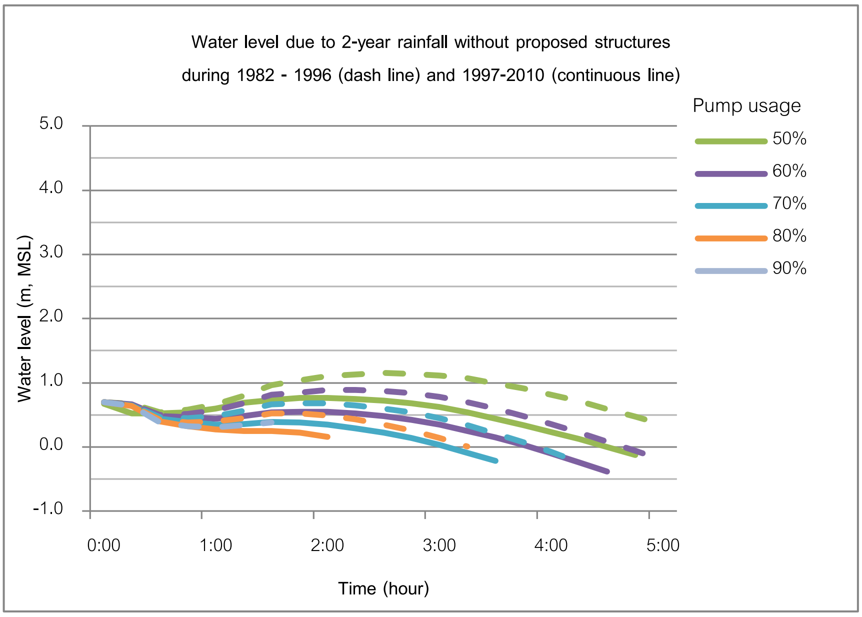


Figure J-1 Water level due to 2-year rainfall without proposed structures during 1982-1996 (dash line) and 1997-2010 (continuous line)

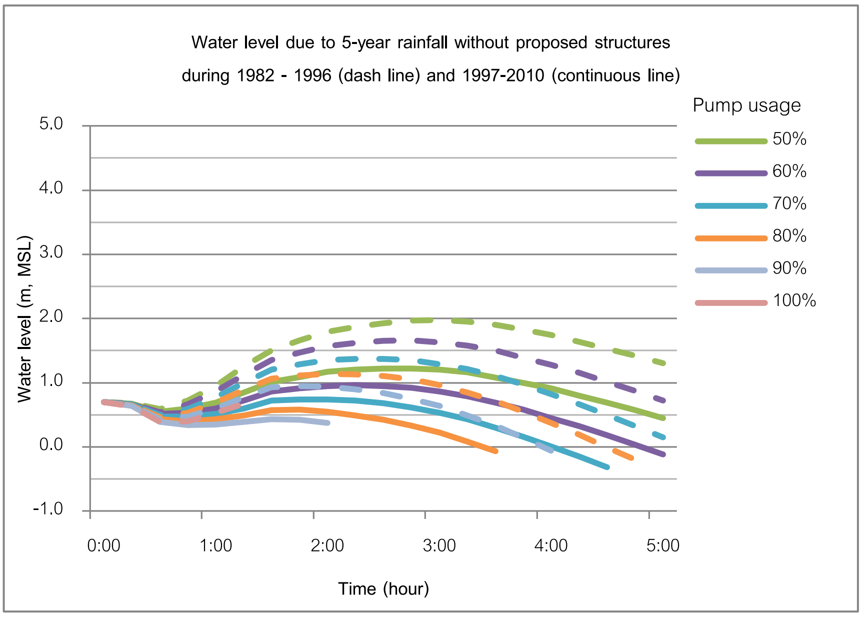


Figure J-2 Water level due to 5-year rainfall without proposed structures during 1982-1996 (dash line) and 1997-2010 (continuous line)

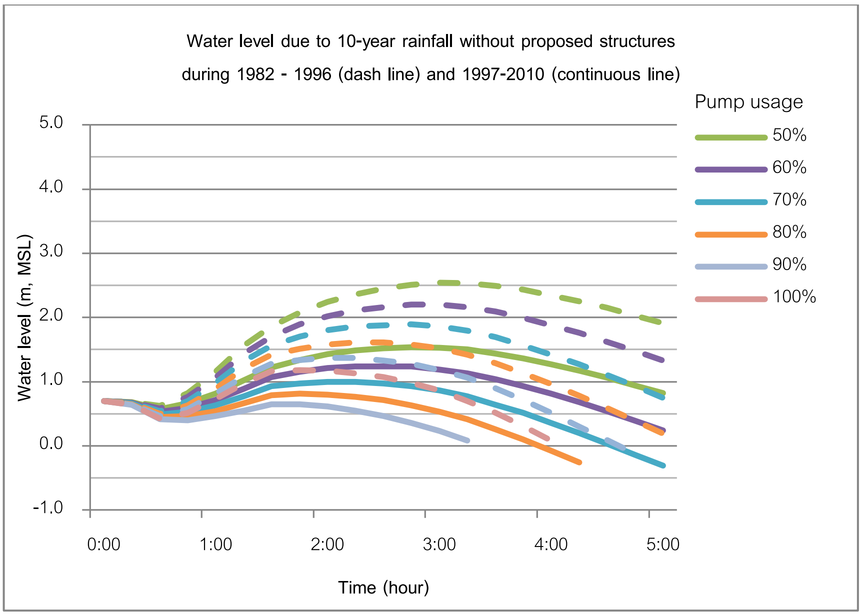


Figure J-3 Water level due to 10-year rainfall without proposed structures during 1982-1996 (dash line) and 1997-2010 (continuous line)

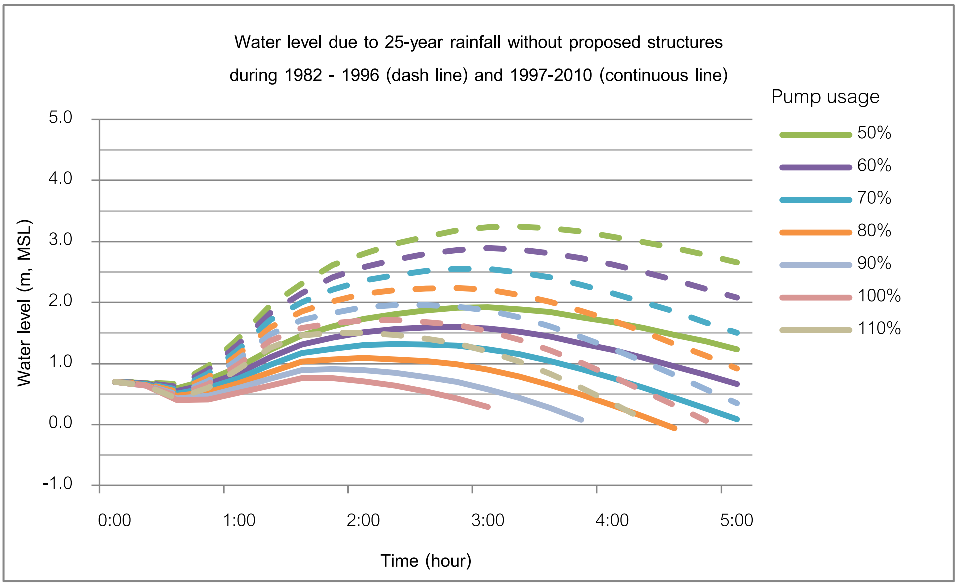


Figure J-4 Water level due to 25-year rainfall without proposed structures during 1982-1996 (dash line) and 1997-2010 (continuous line)

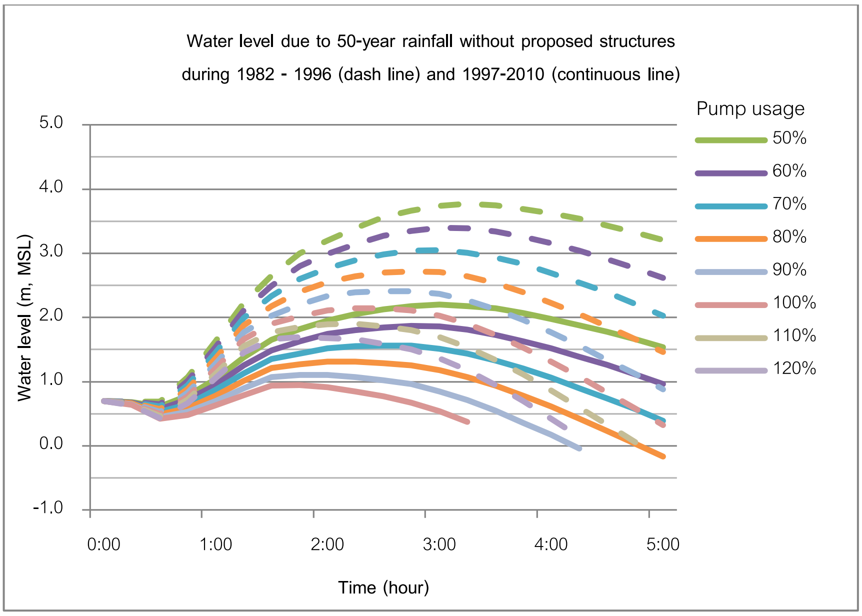


Figure J-5 Water level due to 50-year rainfall without proposed structures during 1982-1996 (dash line) and 1997-2010 (continuous line)

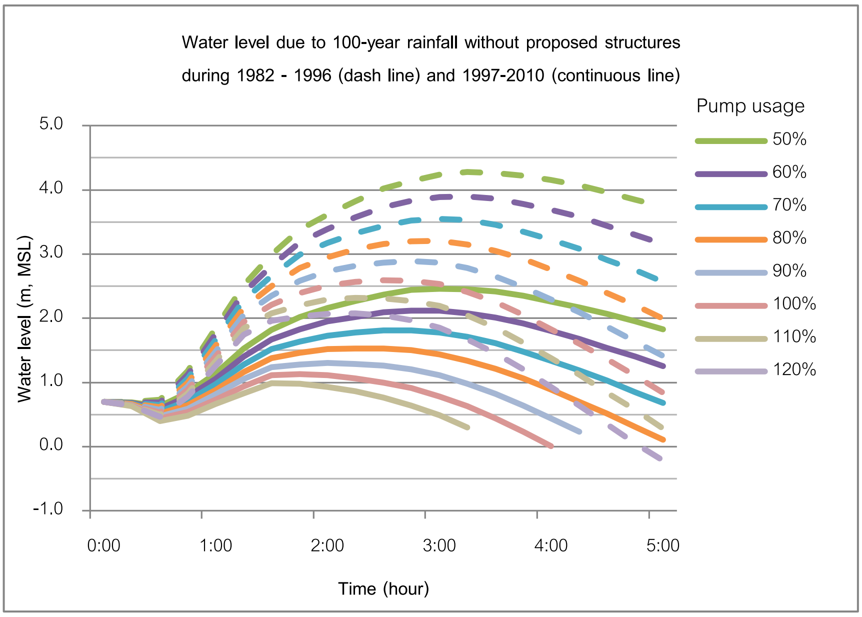


Figure J-6 Water level due to 100-year rainfall without proposed structures during 1982-1996 (dash line) and 1997-2010 (continuous line)

APPENDIX K

MAXIMUM FLOOD FROM SIMULATED RAINFALL

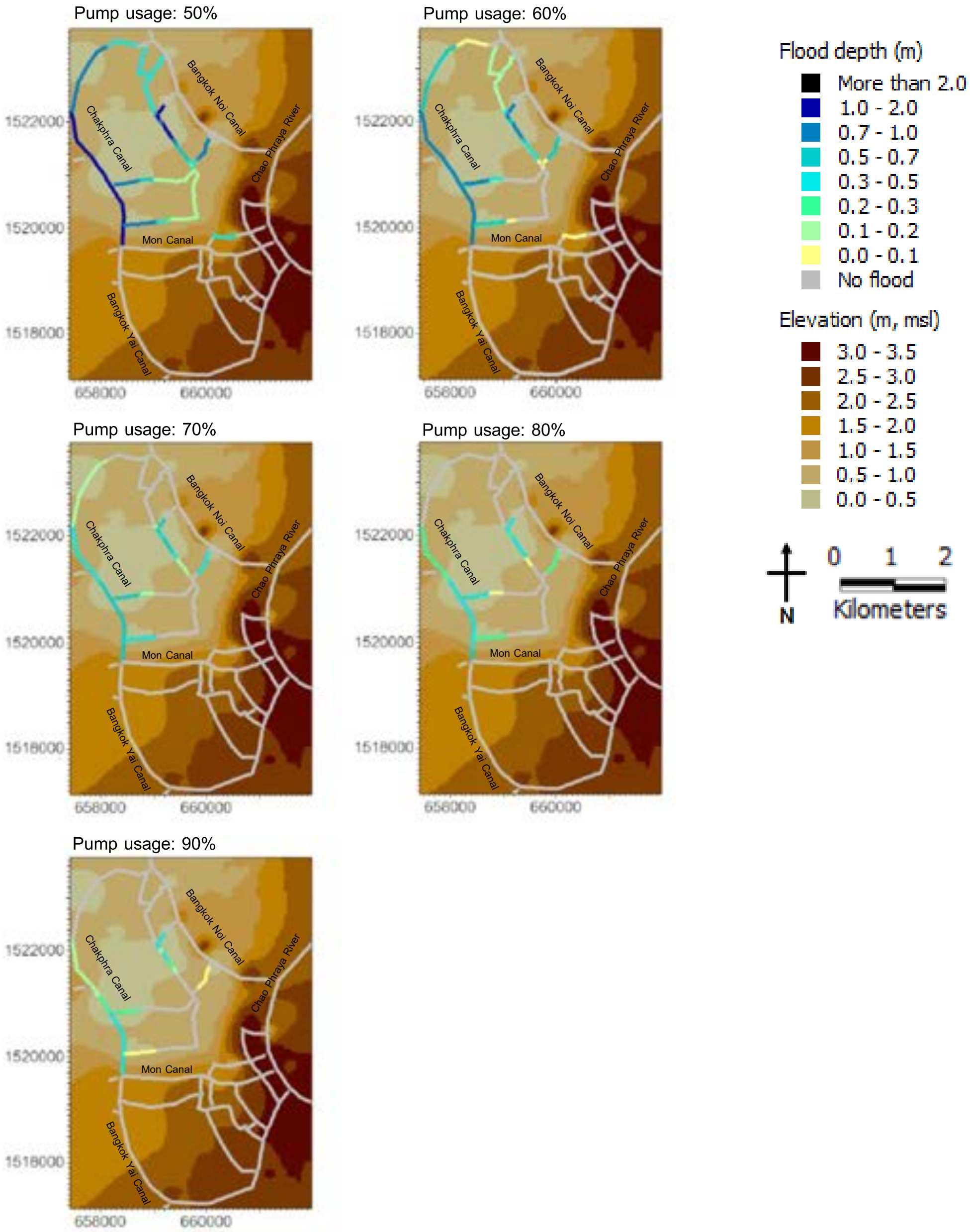


Figure K-1 Maximum flood due to 2-year rainfall in the period of 1982-1996

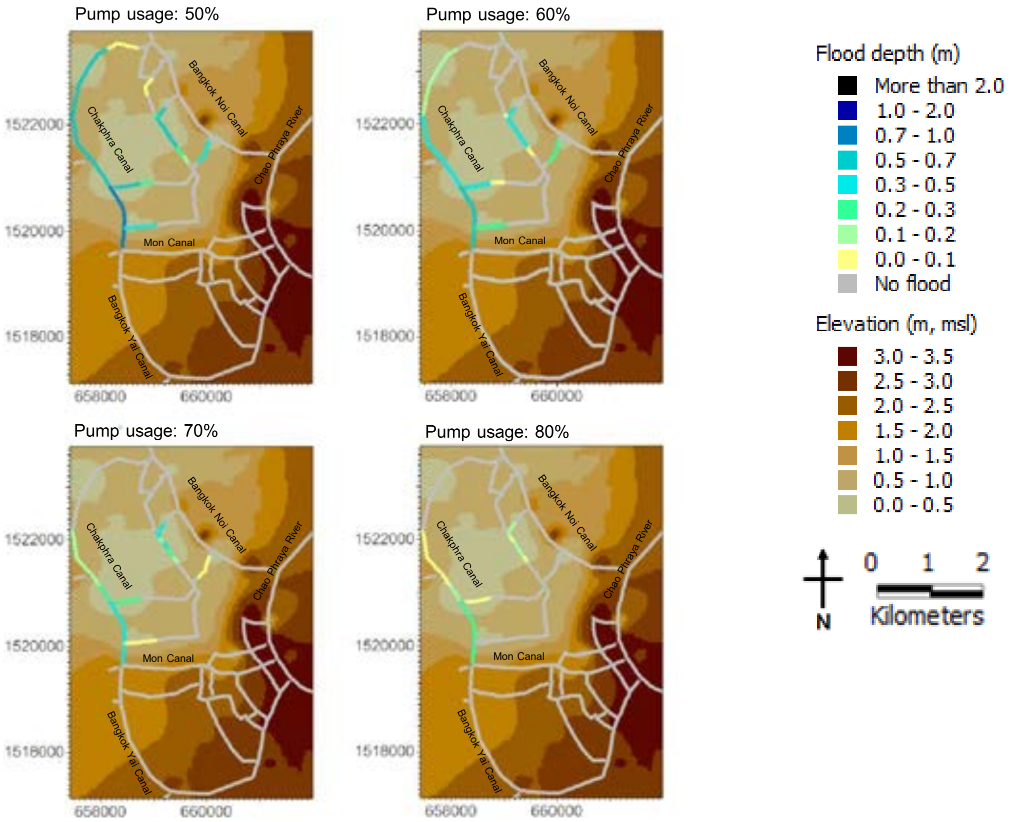


Figure K-2 Maximum flood due to 2-year rainfall in the period of 1997-2010

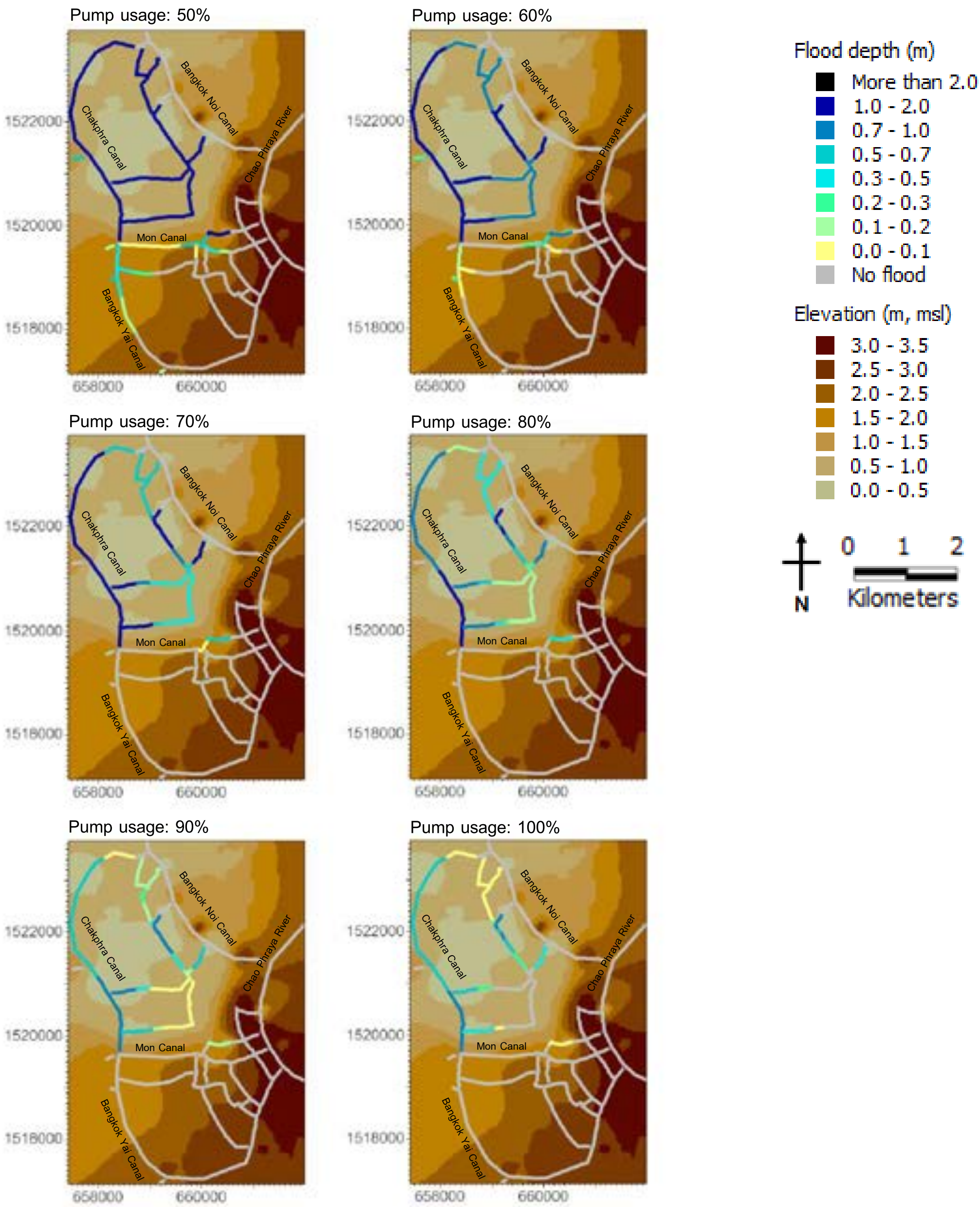


Figure K-3 Maximum flood due to 5-year rainfall in the period of 1982-1996

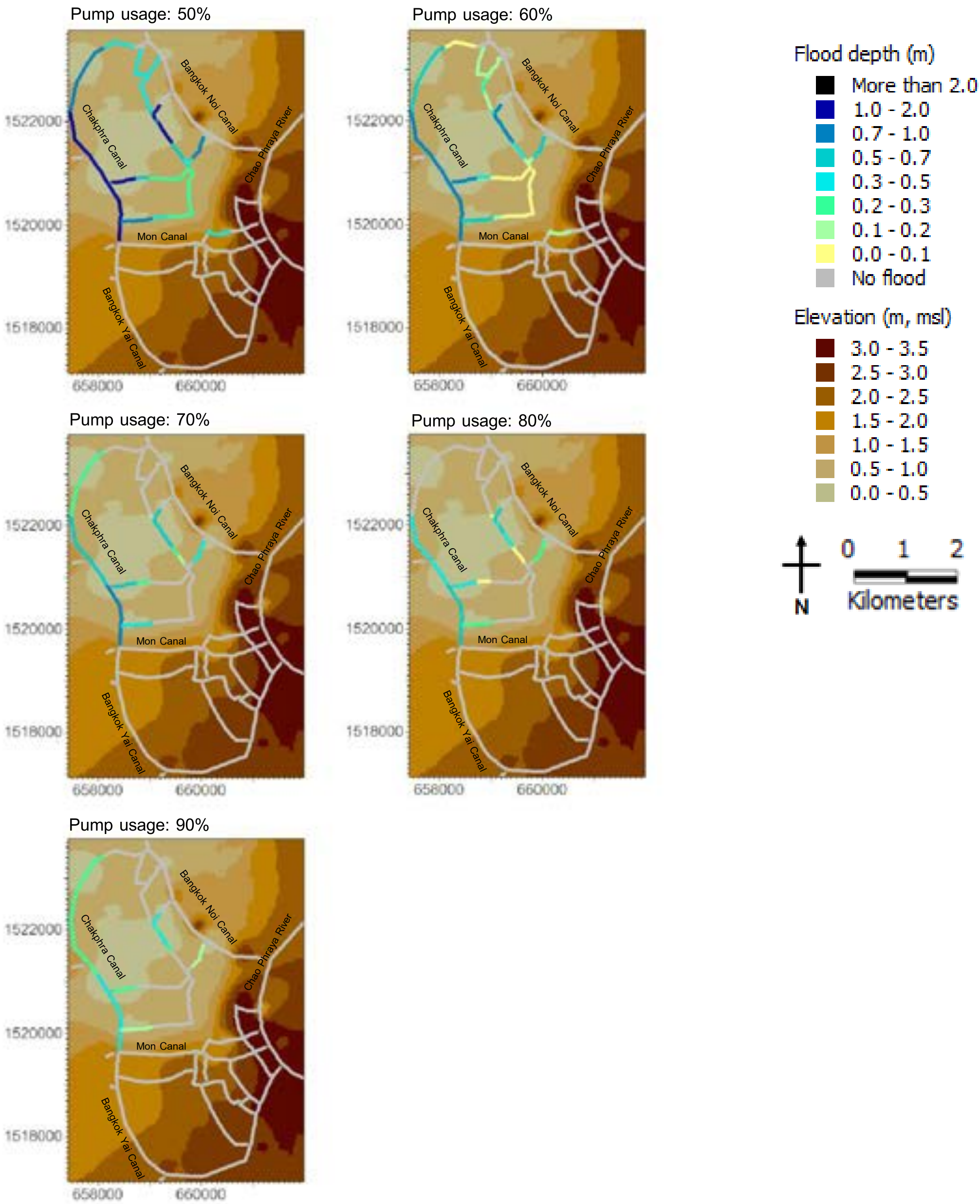


Figure K-4 Maximum flood due to 5-year rainfall in the period of 1997-2010

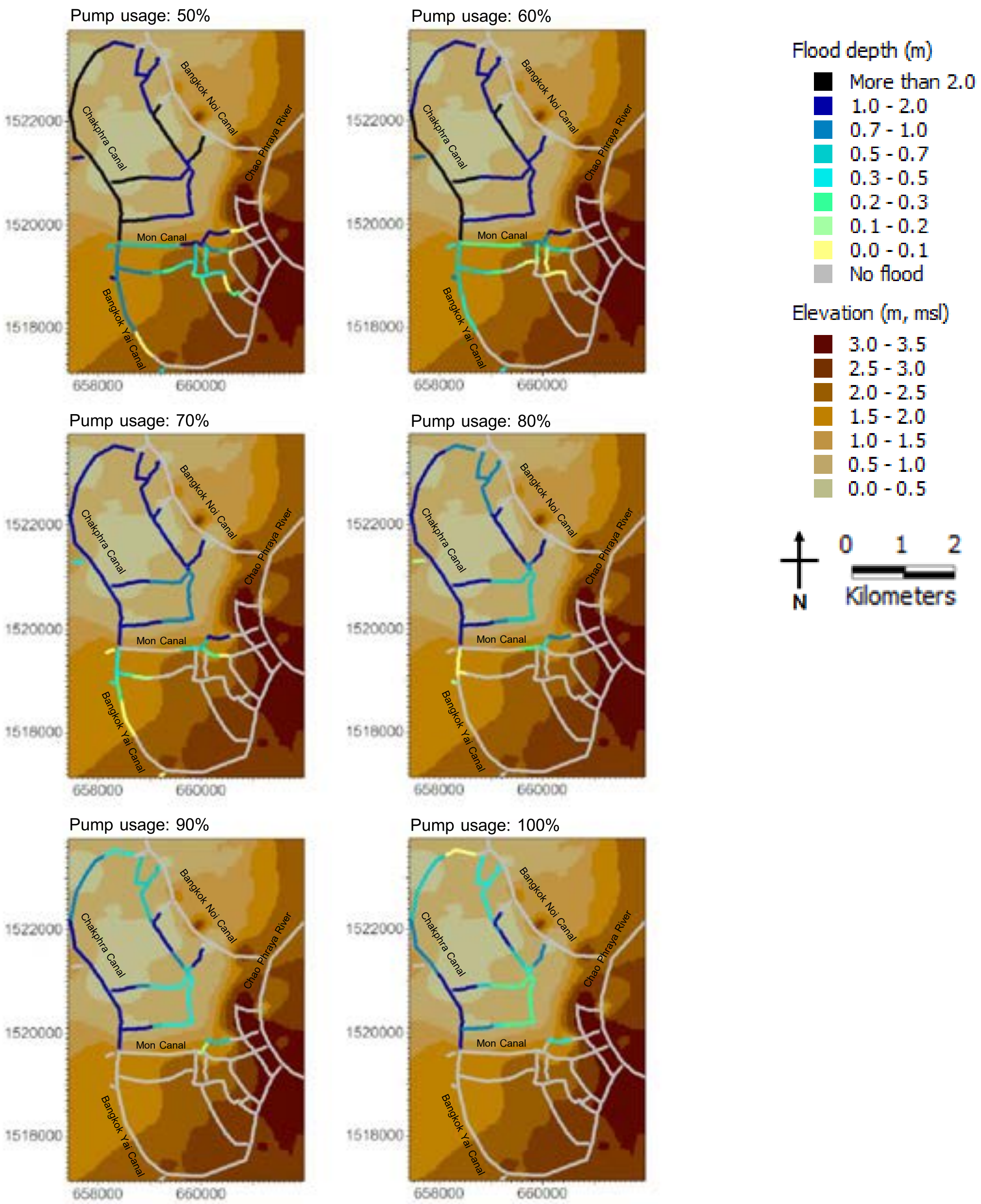


Figure K-5 Maximum flood due to 10-year rainfall in the period of 1982-1996

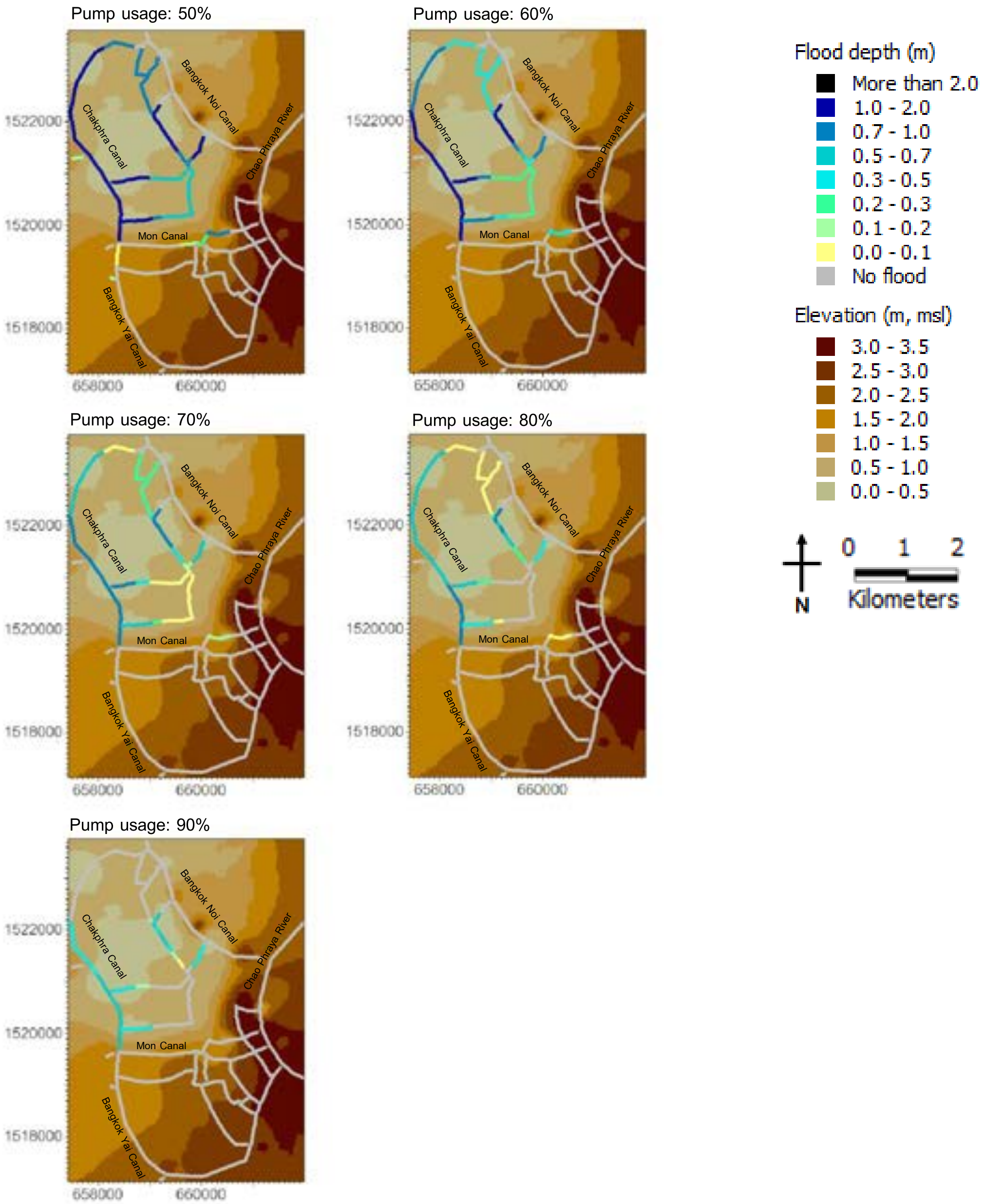


Figure K-6 Maximum flood due to 10-year rainfall in the period of 1997-2010

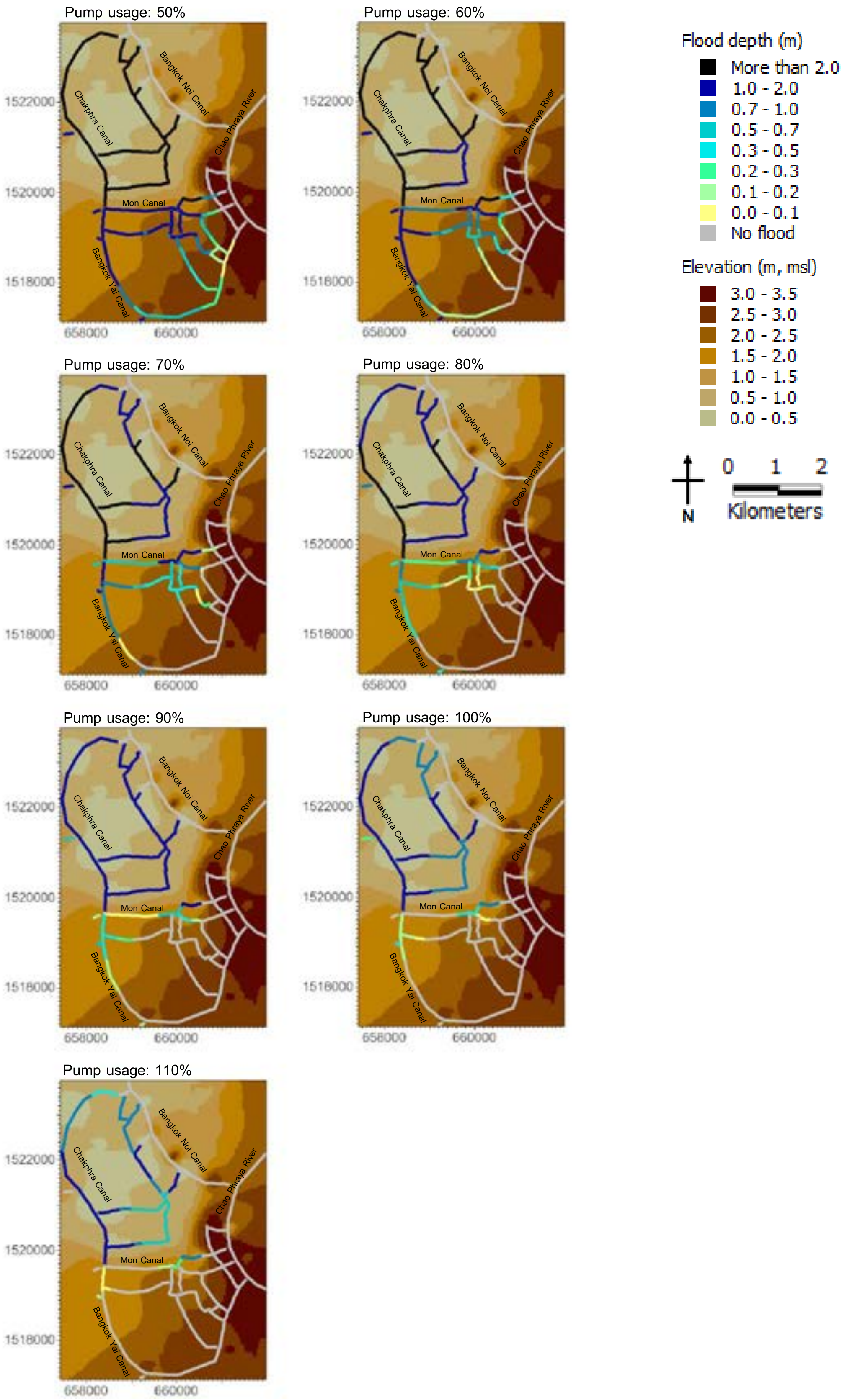


Figure K-7 Maximum flood due to 25-year rainfall in the period of 1982-1996

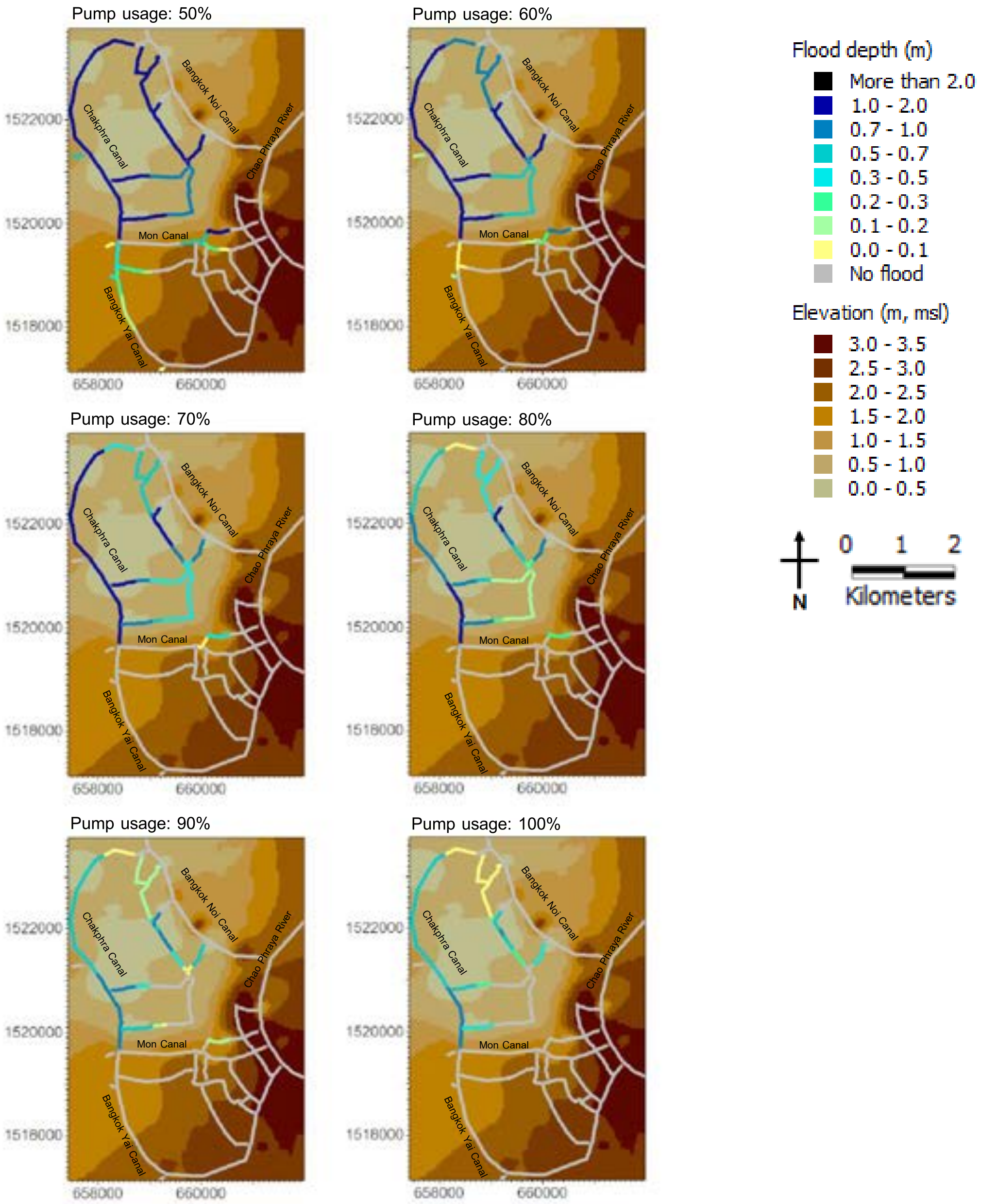


Figure K-8 Maximum flood due to 25-year rainfall in the period of 1997-2010

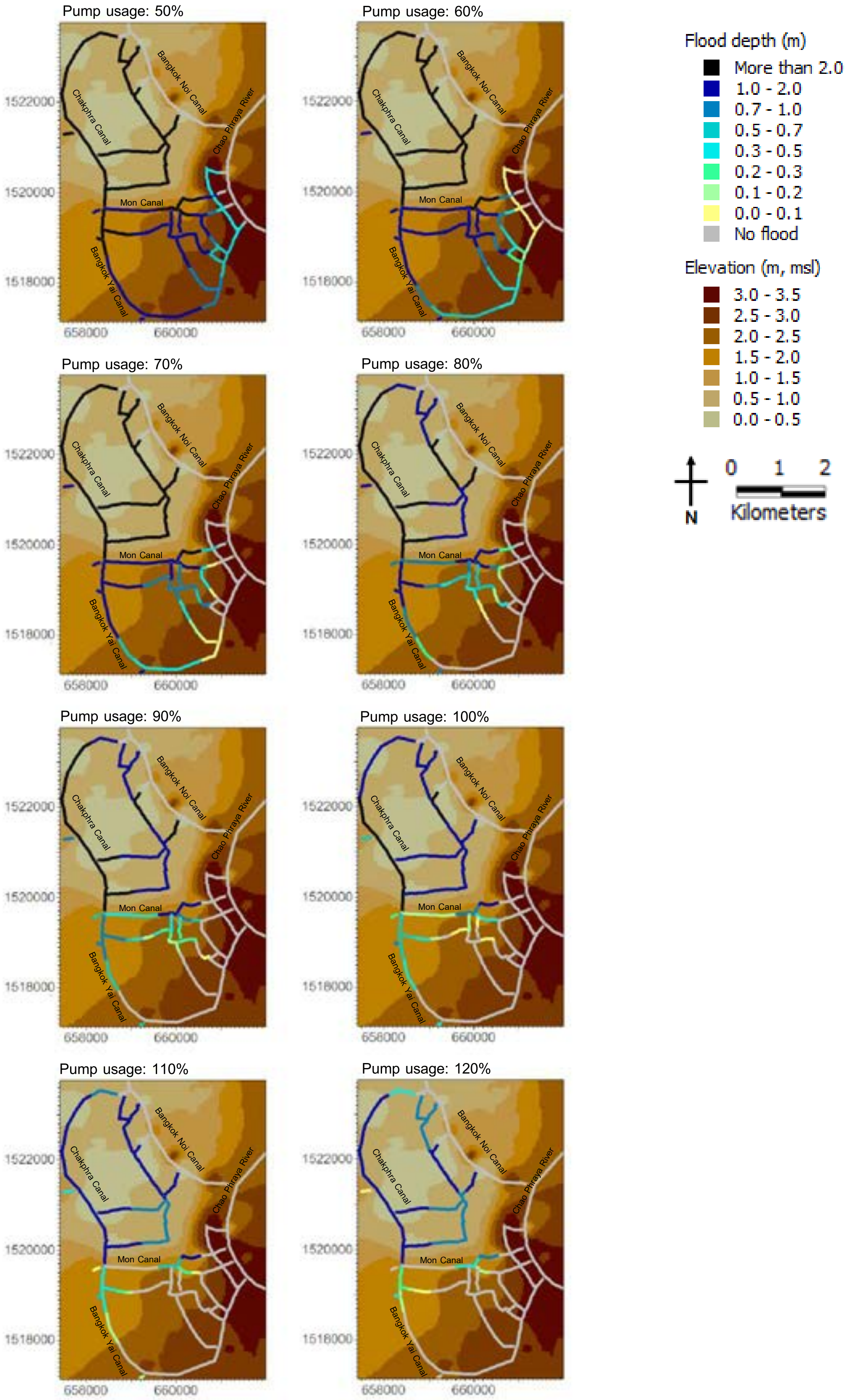


Figure K-9 Maximum flood due to 50-year rainfall in the period of 1982-1996

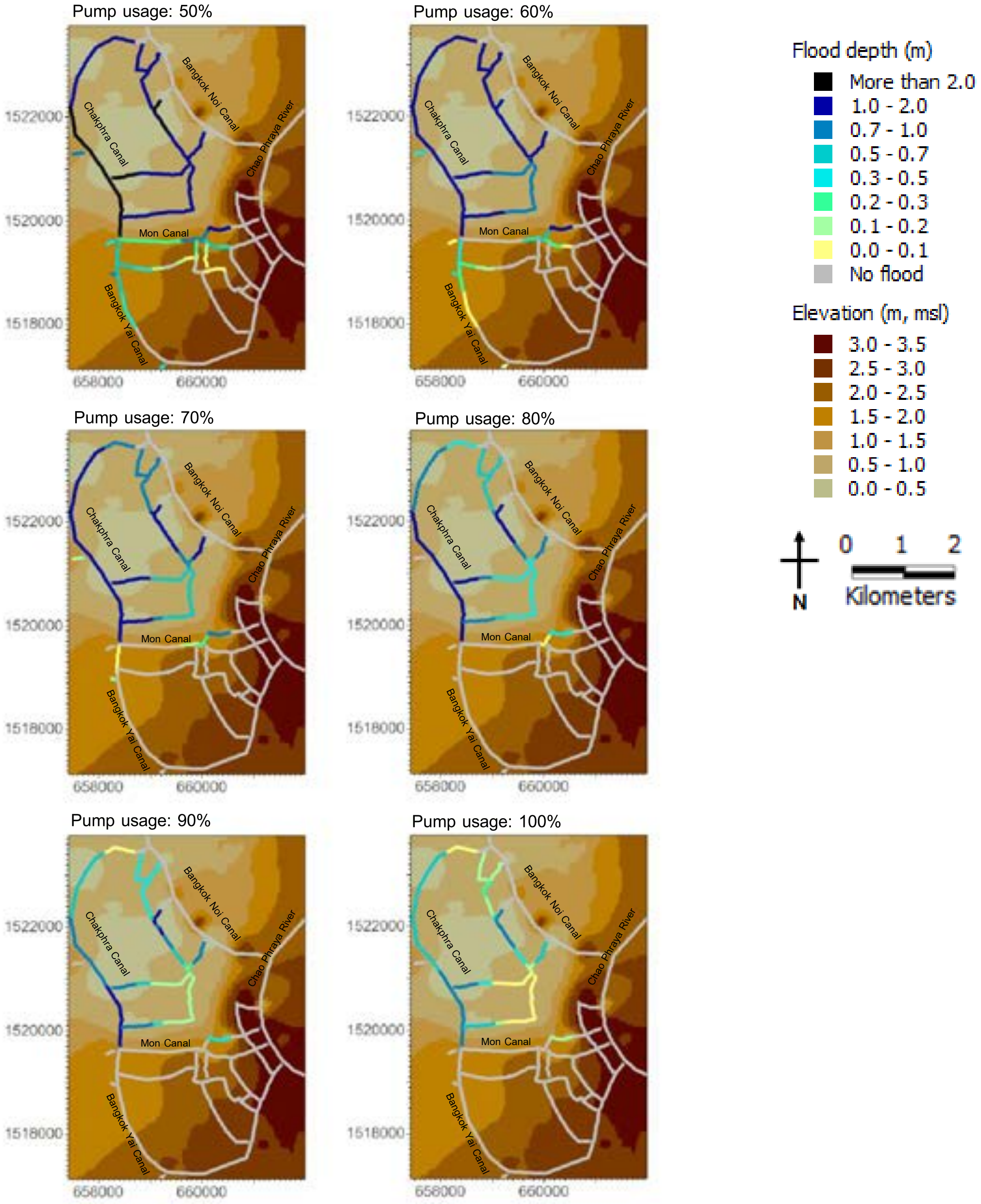


Figure K-10 Maximum flood due to 50-year rainfall in the period of 1997-2010

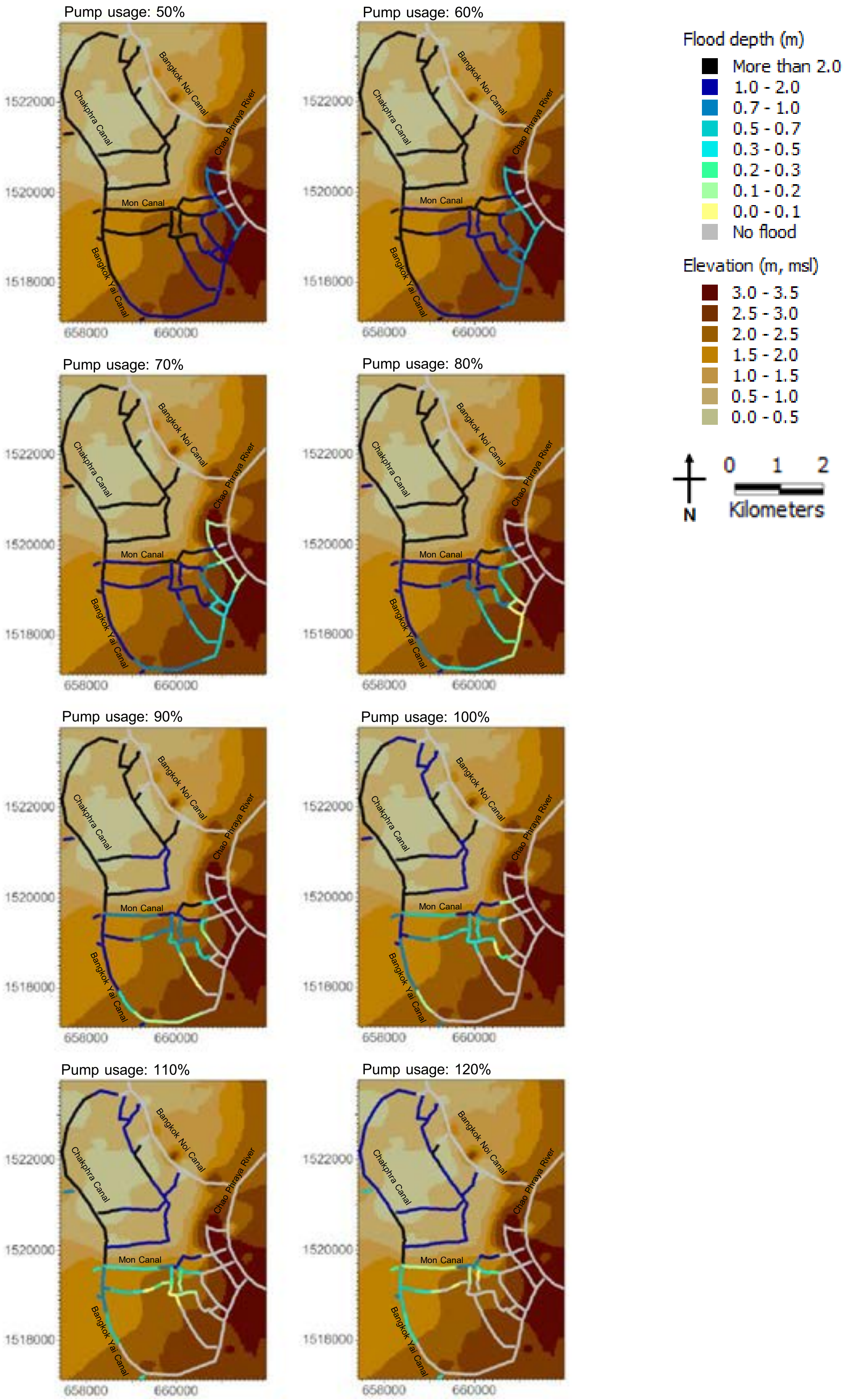


Figure K-11 Maximum flood due to 100-year rainfall in the period of 1982-1996

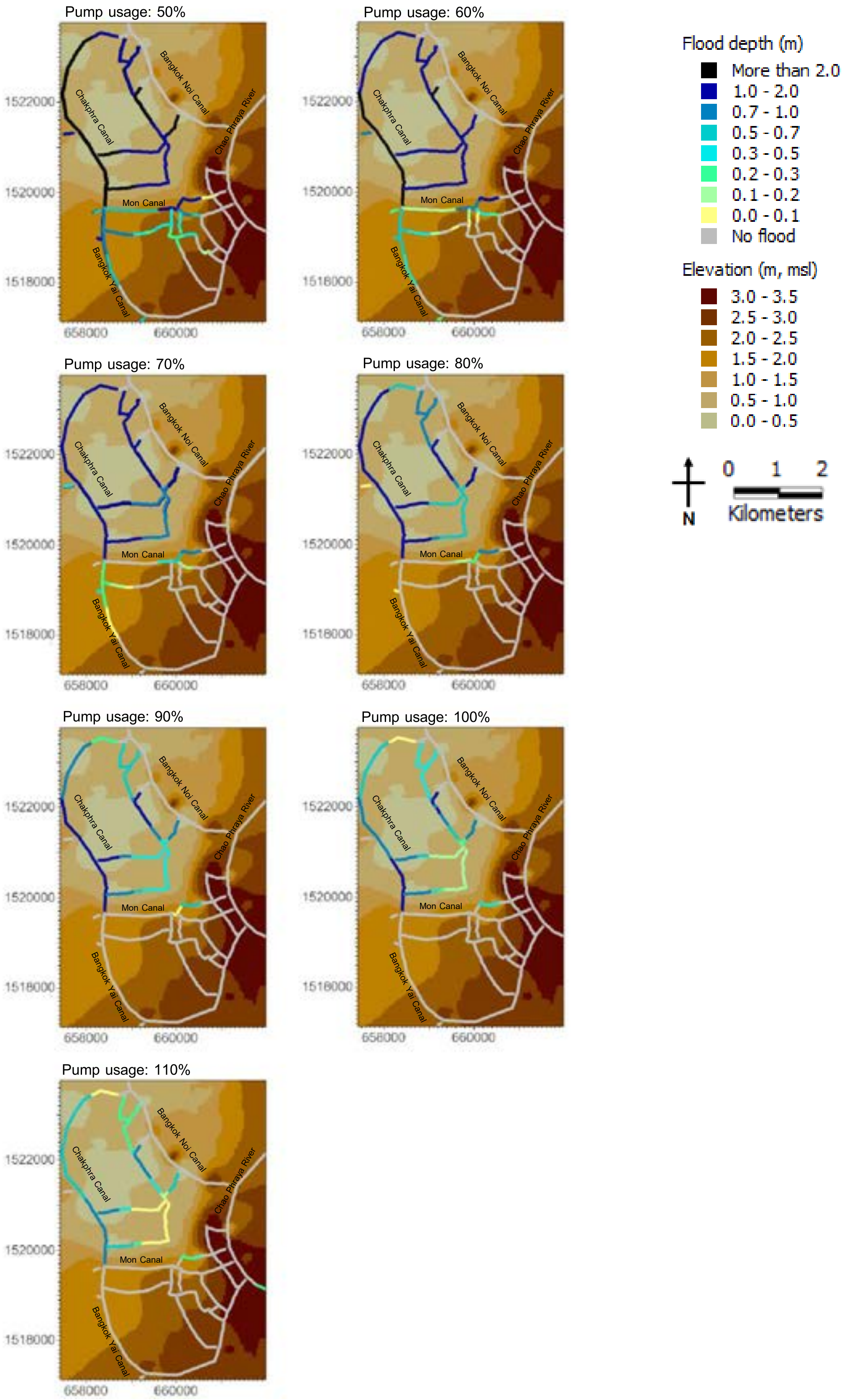


Figure K-12 Maximum flood due to 100-year rainfall in the period of 1997-2010

APPENDIX L

WATER LEVEL WITH PROPOSED STRUCTURES

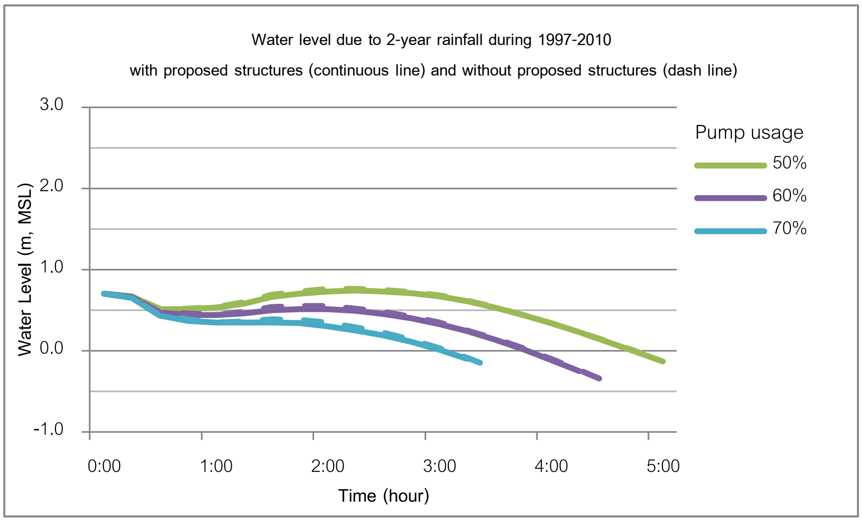


Figure L-1 Water level due to 2-year rainfall during 1997-2010 with proposed structures (continuous line) and without proposed structures (dash line)

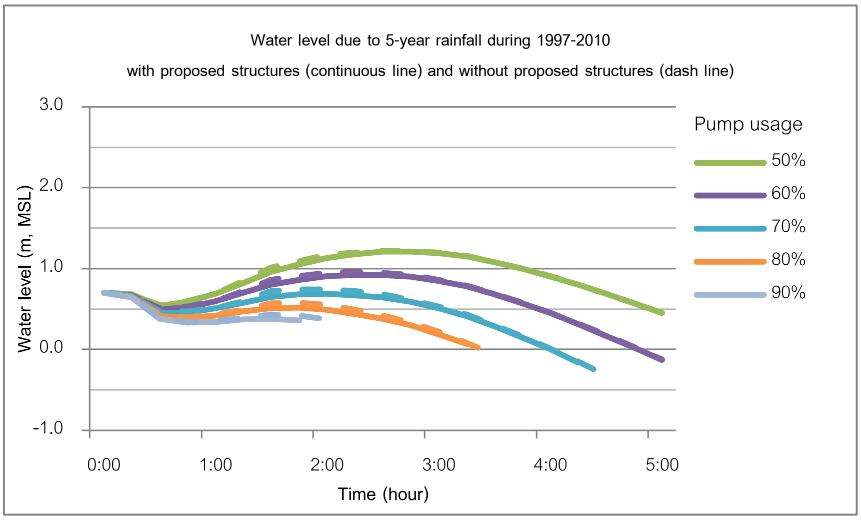


Figure L-2 Water level due to 5-year rainfall during 1997-2010 with proposed structures (continuous line) and without proposed structures (dash line)

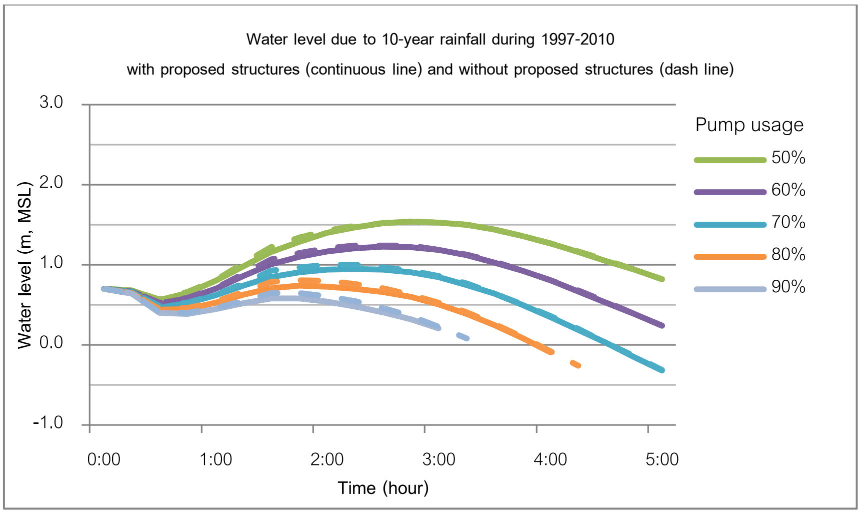


Figure L-3 Water level due to 10-year rainfall during 1997-2010 with proposed structures (continuous line) and without proposed structures (dash line)

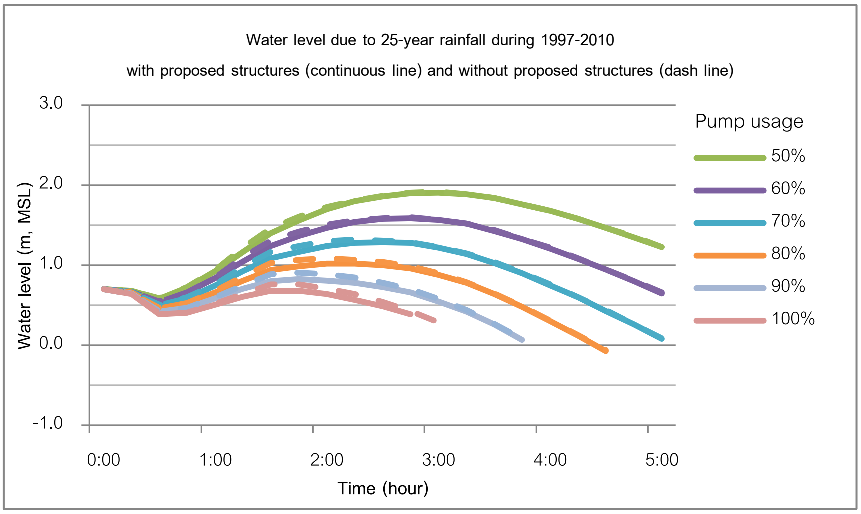


Figure L-4 Water level due to 25-year rainfall during 1997-2010 with proposed structures (continuous line) and without proposed structures (dash line)

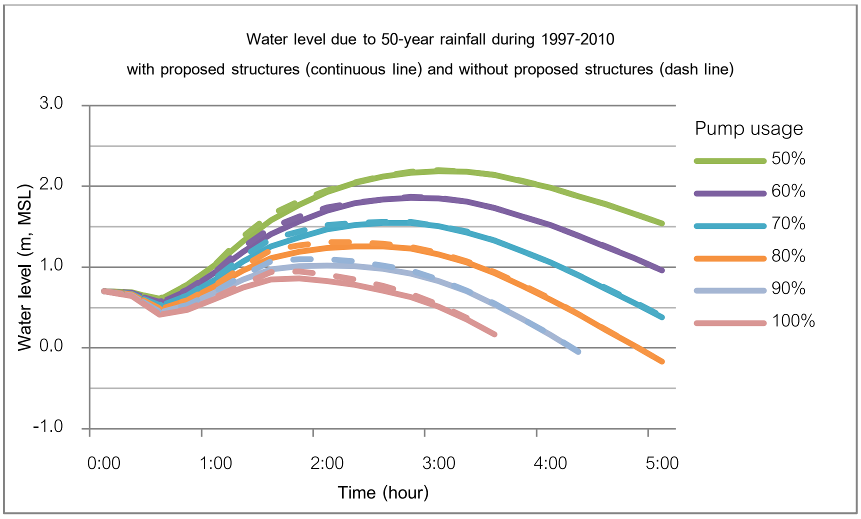


Figure L-5 Water level due to 50-year rainfall during 1997-2010 with proposed structures (continuous line) and without proposed structures (dash line)

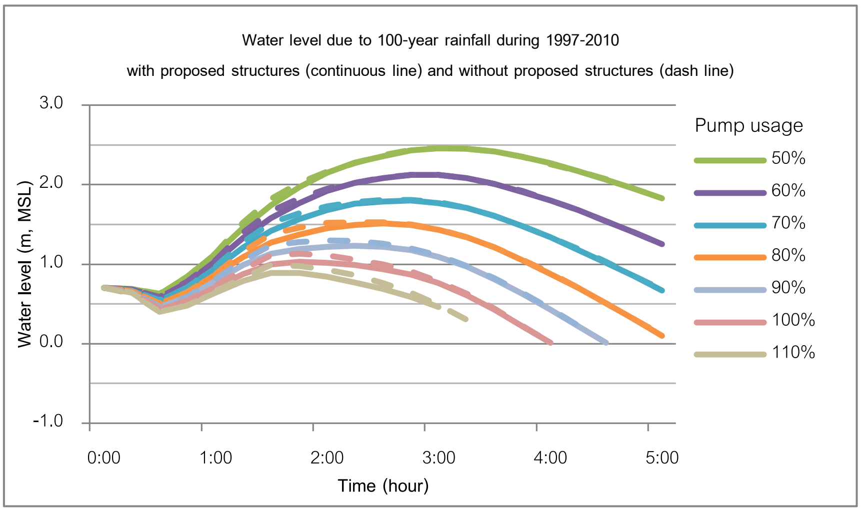


Figure L-6 Water level due to 100-year rainfall during 1997-2010 with proposed structures (continuous line) and without proposed structures (dash line)

BIOGRAPHY

Name: Mr.Pawee Klongvessa

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University Education:

Bachelor of Science in Mathematics with the first class honor from Chulalongkorn University, Bangkok, Thailand with the first class honor. Project Title: Developing Transportation Service Using Queuing Theory: Case Study of Chulalongkorn University Shuttle Bus at the Shuttle Bus Station in front of the Dormitory.

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