Design of Feasible Storm Water Drainage for Urban Development of The Waymouth Hills

Bachelor Thesis
Final Report

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Thesis Title: ***Design of a Feasible Storm Water Drainage for the Urban Development of The Waymouth Hills***

Location: *Philipsburg, St Maarten*
Date: *August 04th, 2018*

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Abstract

Due to lack of proper stormwater drainage in the Waymouth Hills, stormwater runoff have often caused undermining of the road infrastructure and disrupting traffic flow in the study area. Sediments are often deposited to the low-lying terrain from the runoff and this reduces the storm drains capacity downstream resulting the area prone to flooding. Future urban expansion of the Waymouth Hills and the effects of climate change are predicted to further increase this problem through increased urban stormwater runoff.

This report aims to analyze a sustainable stormwater solution through a multi-criteria analysis (MCA) for the urban development in the Waymouth Hills. This was done by establishing a set of criteria relevant to the research together with the desired outcome and guidelines established by the Client; Ministry of Public Housing, Spatial Planning, Environment and Infrastructure of Sint Maarten. Furthermore, a sensitivity analysis was carried out to evaluate the MCA results.

Manual calculation together with hydrodynamic modeling simulation (done through the Autodesk Storm and Sanitary Analysis) were carried out to analyze the stormwater runoff effect in the present storm drain infrastructure and the newly designed storm drains for the future urban development. These analyses include the use of both stationary and dynamic rainfall for a 10-year storm event. Furthermore, preventative measures for potential flooding were assessed through the use of a detention pond.

A technical requirement report was drawn for the realization of the storm drains and the overall infrastructure upgrade for the future urban development for the Waymouth Hills. Lastly, recommendations were given for the measures that can be undertaken to improve for future relative projects.

Acknowledgements

I acknowledge my deepest sense of gratitude to my supervisor Ir. Gerben Tommassen for his patient guidance, excellent advice, comment and criticism, and constant encouragement on each step from the beginning up to the end of this study.

I would also like to acknowledge Dr. Nikola Stanić for his support, constructive criticism, and patient guidance throughout the research period.

I would like to express my sincere pleasure to express my sincere gratitude to Ronal Daal, the president and principal at ICE for allowing me to carry out my research in his company.

My appreciation specially goes to all the staff members at ICE who helped me directly and indirectly throughout my research in the company and provided a friendly atmosphere.

I would like to express my sincere gratitude to my sisters and friends who have supported me along the way.

Finally, I would like to express my deep appreciation to my parents for their patience and moral support during my research in St Maarten.

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1. Introduction

1.1. Background

St Maarten is an island situated in the North Eastern Caribbean Sea and it is shared between the French Republic and the Kingdom of the Netherlands. The Dutch side of the island is called St. Maarten and encompasses an area of approximately 3380 hectares bounded by the French side of the island (St. Martin) on the North and the Caribbean Sea on the South. In the 1950s the island's main sources of income were farming and the exploitation of salt flats but today tourism had taken over as the main source of income for the population (de SEZE, 2014). The rapid development of St. Maarten over the past 10 years has led to both residential and commercial development. The total population has grown from 13,156 in 1980 to nearly 41,000 in year 2000 (Ediriweera , 2007). To sustain this rapid economic development in St Maarten there is an increasing demand for more infrastructural upgrade and affordable housing. Hence, the government is presently in the process of a major road enhancement project, which includes the construction of new roads, the repaving of existing roads and the implementation of roundabouts in the Dutch Cul de Sac area. This project is proposed to alleviate the traffic problem in the Dutch Cul de Sac area and enhance the ability for more affordable residential developments in the area.

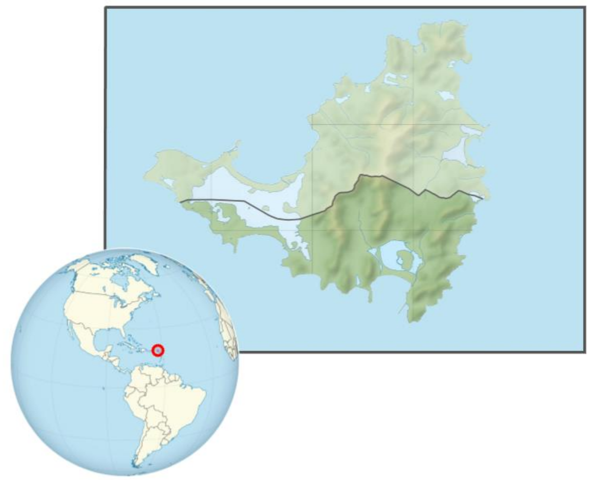

Figure 1: Location map of Sint Maarten (Gaba, 2010).

Moreover, in St. Maarten, residents, homes, business and public infrastructure from time to time are under the threat of flooding due to heavy rainfall. With the frequent presence of hurricanes, tropical storms, and an increase in the frequency of high-intensity storms due to climate change, flooding has become a growing and serious problem for the island territory of St Maarten. Apart from the natural causes, new developments will result in a greater influence on the island's existing flooding problem because the infrastructure has not kept pace with this continuous development and growth of St Maarten.

1.2. Study area

The studied area for this research is the Waymouth Hills situated on the North-East in the region of Dutch Cul de Sac on the Dutch side of the island Territory St Maarten. The Waymouth Hills encompasses an area approximately of 17 hectares partially developed with residential houses. Based on the report "St Maarten Storm Water Modelling study" (2006), it is estimated that 0-10 percent of the area is currently developed. Moreover, in the same study it is estimated that there will be an increase of 100 percent development in the area if the maximum development capability is utilized to a range of 10-20 percent for the future urbanization. The Dutch Cul de Sac is spread out over flatlands and steep slopes, with the Waymouth Hills having an elevation difference of 250 meters.

The main accessibility of the Waymouth Hills is by the Mildrum road which is joined by five subsidiary roads (Paradise Hill road, the Quil Road, Brimstone Hill Road, Mount Pele Hill Road, and Mount Souffriere Road). These roads are either unpaved or partially paved and are severely damaged by erosion of storm water runoff (Maarten, 2016). The result of this outcome is primarily due to the non-existence of any drainage structures in the Waymouth Hills where storm water runoff is solely confined to and conveyed by these roads (Figure 2). The converged flow from these roads courses flow downhill and enter into the main stream situated in the lower lying area of the Dutch Cul de Sac. The main stream is channelized to convey all captured storm water from the Waymouth Hills and all other areas along the L.B Scott road and stretches through the Coralita Road then lastly discharged the storm water into the receiving water body "the Fresh pond" (Figure 3).

I: EXAMPLE OF ERODED ROAD OF MILDRIUM RD   II: UNPAVED ROAD OF BRIMSTONE ROAD

Figure 2: The Waymouth Hills existing road network (above) and the condition of the existing road (bottom).

Figure 3: Map of Dutch Cul de Sac displaying the stormwater travel along the mainstream to the Fresh Pond.

1.3. Problem statement

The expansion of urban areas causes the change of landscape from natural landforms and vegetative covers towards unnatural and impervious areas. The change of landscape also leads to changes of the hydraulic systems within the basin. Regarding storm water, this has two major effects: on one hand, on the storm water runoff quantity, on the other hand, on storm water runoff quality (ZILLER, 2010). With urbanization the sealed surface area increases alongside the increase in impervious surfaces, which results in increased hydraulic efficiency in urban catchments (Putnam, 1972) and can cause substantially decreased capacity for a given landscape or region to infiltrate precipitation, with a concomitant increase in the production of runoff (Booth D. , 2000) and surface runoff velocities (Figure 4).

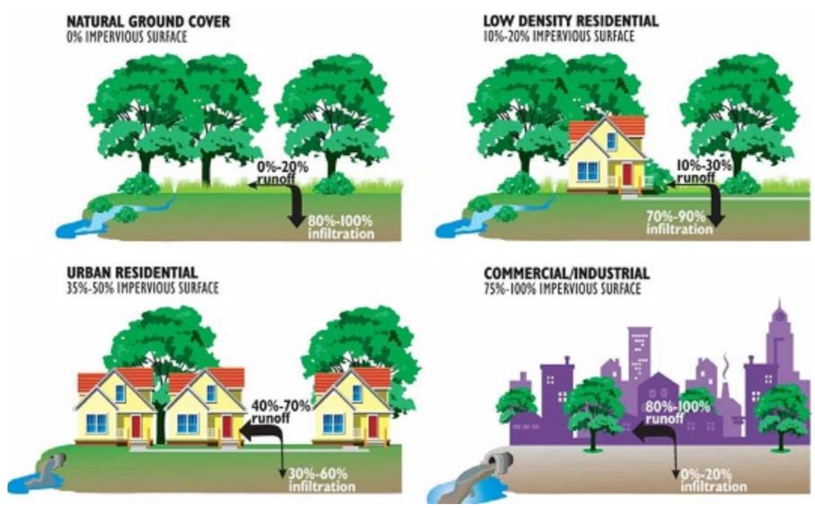

Figure 4: Changes in hydrology and runoff due to urbanization (Juneau Watershed Partnership, 2018).

The rapid development in St Maarten over that last decade resulted in the increased demand for more infrastructure and affordable housing developments. During storm events there are frequently many temporary disruptions to the overall transportation systems within the low-lying areas of the Dutch Cul de Sac. Moreover, during heavy storm events, large quantities of surface runoff is produced with high velocity due to the steep terrain, thereby causing erosion. This erosion results in the transport of silt and debris which then clogs the main channel in the flat terrains and generates flash urban flooding causing damage to the adjoining roads, properties and public areas.

As development in the Dutch Cul de Sac region continues, the increase in the volume of the surface run-off overwhelms the drainage system. The pressure on the drainage system with run-off water going beyond its capacity will be more frequent with increased urbanization. Combine these factors with the gradient of the landscape, the larger storms, and the fact that the storm water runoff from the entire area of the Dutch Cul de Sac is converged and conveyed by the Dutch Cul de Sac stream (main stream) (Figure 3), it is conceivable that this leads to increases in the frequency of flooding.

With the existing condition of the current infrastructure in the Waymouth hills, not only can the road not convey the storm water, but it also degrades and deteriorates the road rapidly during any storm events. Consequently, not only does this inhibit the area from further growth prospect, but it also impacts the flooding problem in the Dutch Cul de Sac.

In order to address these problems, the current state of the storm drainage network in the Dutch Cul Sac must keep pace with continuing development and future storm events; ultimately, this can be done at micro- instead of macro-level. The focus of this research is to ameliorate the roads and storm water drainage in the Waymouth Hills.

1.4. Research objective

The objective of this research is to analyse the changes in the catchment hydrology for future urbanization in the Waymouth Hills and design a feasible storm water drainage system to prevent the occurrence of uncontrolled flooding in the area and mitigate the existing flooding problem in the flood prone area within the Dutch Cul de Sac.

1.5. Research questions

To reflect the above stated problems and research objective, the main research question was formulated as:

> **What is the most optimal stormwater drainage solutions for the Waymouth Hills that can cope with the urban development and future climate change?**

To answer the main question defined for this research, it was important to break down the question into sub-questions that can serve as step-by-step guide to achieve the objective. The following sub-questions can be posed:

1. What is the current situation in the area?
    a. What is the current drainage system used for stormwater runoff?
    b. What is the effect of this runoff in the current drainage?
2. What is the program of requirements for the future stormwater drainage design?
3. How is it possible to determine the stormwater drainage solutions and use them for urban development?
    a. Which are the different stormwater drainage alternatives can be used?
    b. Which criteria will be used for the comparison of the alternatives?
    c. How will these criteria be evaluated?
4. How will the future urban development affect the stormwater drainage design?
    a. To what extent of the area will be urbanized?
    b. What is the hydrologic effect of this urbanization o the future storm water drainage?
    c. What is the stormwater runoff impact between stationary and dynamic rainfall have in the stormwater drainage?
5. How can the project area be best utilized to accommodate climate change and can mitigate the occurrence of flooding in flood prone area?

2. Theoretical Framework

In this chapter, a wide overview of the necessities of the project is looked into. Here, the theory behind the work that is required for this project is explained in order to give useful context to both the following chapters of Methodology and Results.

When looking at the situation on the Waymouth Hills and its interconnected pathway to further downstream systems, it becomes immediately apparent that as urbanization expansion occurs, the downstream systems also need to be updated and expanded. The expansion of the downstream systems is outside the scope of this project, but efforts will therefore be made within this project to lower the effect of additional flood water reaching the downstream systems.

When taking this all into consideration, it is therefore important to understand what the climate is like, since that is representative for the way in which rainfall will affect this area. Furthermore, it is important to understand the effect of urbanization on the increase in storm water runoff. Once this is understood, the next step is to look into mitigation methods for storm water runoff. For this it's possible to use the road itself, channels, gutters, ditches, culverts, retention ponds, detention ponds, weirs, and orifices. All this will be discussed in the sub-chapters below.

2.1. Climate characteristics

St Maarten is located 63.5 degrees West and 18.5 degrees North. The island has a tropical monsoon climate in the classification scheme of köppen (Curacao, 2015) which has a dry season dry season from January to April and a rainy season from August to December. Based on records (1981-2010) of Princess Julianna airport the driest month on record is March while the wettest is November. On average, there are about 142 rain days a year with April having the least (8 days) and November the most (15days) (Meteorological Department St. Maarten, 2017). Furthermore, the island experiences tropical temperatures with very little variation in temperature throughout the year with December to March being the cooler months at around 25ºC on average, and April to November being the warmer months on average with temperatures between 27ºC and 29ºC. The coldest temperatures recorded are around 18ºC; the hottest temperatures are around 33ºC (MacRa, Nisbet , & Blok, 2009).

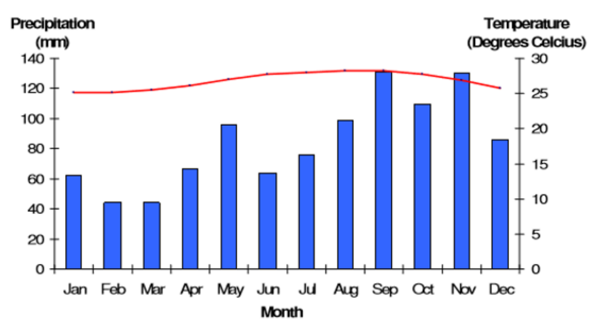

Figure 5: Average Precipitation and Temperatures for Princess Juliana International Airport St Marten, 1971 – 2002 (MacRa, Nisbet , & Blok, 2009).

According to the Meteorological Department Curacao (2015), the mentioned climatic conditions can mainly be attributed to the displacement of the Azores subtropical ridge during the year. This displacement of the ridges is due to sea-level pressure (SLP) difference between the Azores high and the Icelandic low (Université catholique de Louvain, 2008) and is characterized by the North Atlantic Oscillation (NAO). The NAO's strength and sign may be defined as the normalized sea-level pressure difference between the Azores and Iceland (Jones et al., 1998). When the NAO index is high, the sea-level pressure difference is stronger than average while being weaker than the mean when the NAO index is negative (Université catholique de Louvain, 2008). The correlation of the winter NAO index and the winter SLP (averaged over December, January, and February) in the Azores high and Iceland region is presented in Figure 6 below.

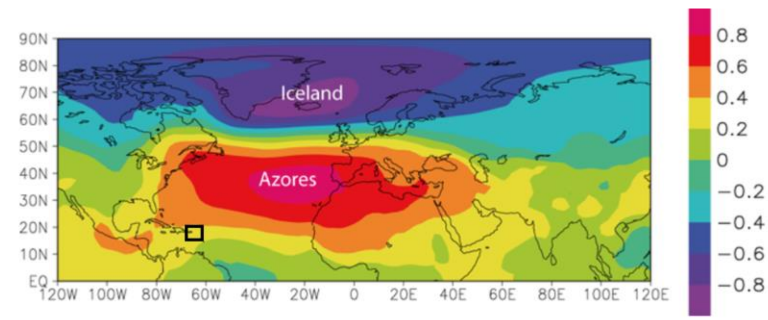

Figure 6: The geographic location of St. Maarten (within the black square) is situated within the Azores subtropical high enclosed with the correlation of the winter NAO index with the winter SLP (Université catholique de Louvain, 2008).

During the northern hemisphere summer, the Azores subtropical high is located more to the south over the central Atlantic and suppresses the formation of clouds that can produce rain. Showers are limited and of light intensity during these months. As the northern autumn approaches, the Azores subtropical high retracts to a more northern position, moving away from the island. Its influence on the atmosphere above the island diminishes and hence making significant cloud formation and rain possible. These showers are moderate to heavy and can often be accompanied by thunder (Curacao, 2015). The causes of such effect are governed by the circulation of wind and the sea-level pressure in the atmosphere throughout the year (Figure 7). The changes of the NAO index (from high to low or vice versa) reflecting the precipitation in the Caribbean during the annual seasons are presented in Figure 8 below. The result of the precipitation shown in Figure 8 involving the seasonal and NAO index changes can be correlated in Sint Maarten.

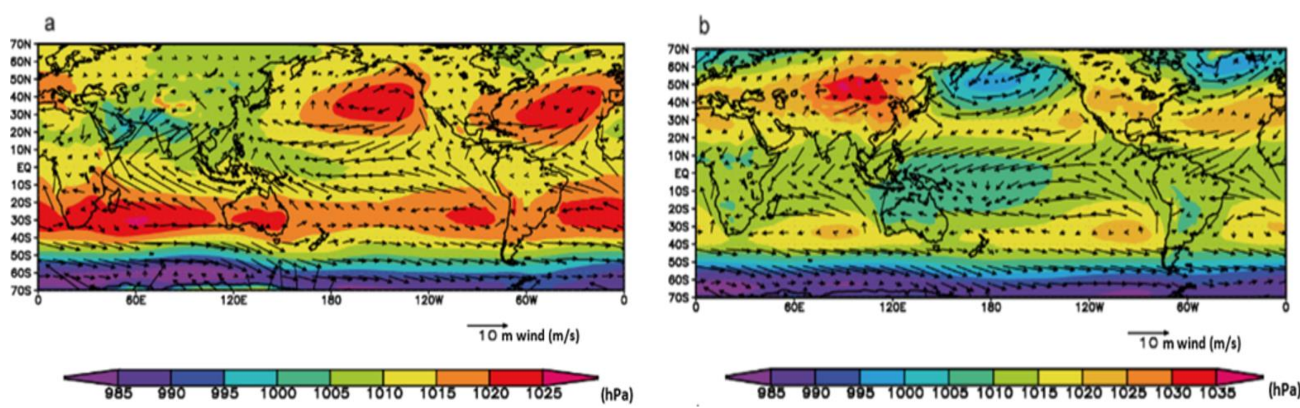

Figure 7: (a) The long-term average wind characteristic and sea-level pressure during the summer period (June, July, and August) and (b) the winter period (December, January, and February) (Kalnay et al., 1996).

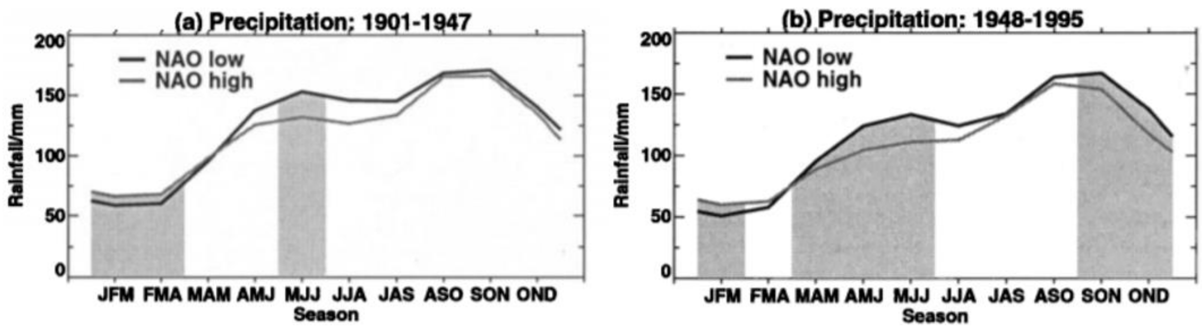

Figure 8: Three monthly seasonal rainfall totals in years with low and high preceding DJF NAO index values during (a) 1901- 1947 and (b) 1948-1995 for the Caribbean. Shading indicates the seasons where the difference in rainfall between the low and high NAO composite years is significant at the 90% level or above (George & Saunde, 2001).

St. Maarten is located in the Atlantic hurricane zone (Figure 10) and on average one tropical storm or hurricane passes at a distance of less than 200 km each year (MacRa, Nisbet , & Blok, 2009). Once every 4 or 5 years St. Maarten is hit by a hurricane (see Table 1 below) (MacRa, Nisbet , & Blok, 2009) (Curacao, 2015). The hurricane season runs from June 1st to November 30th, with a peaked season from August through October (Curacao, 2015).

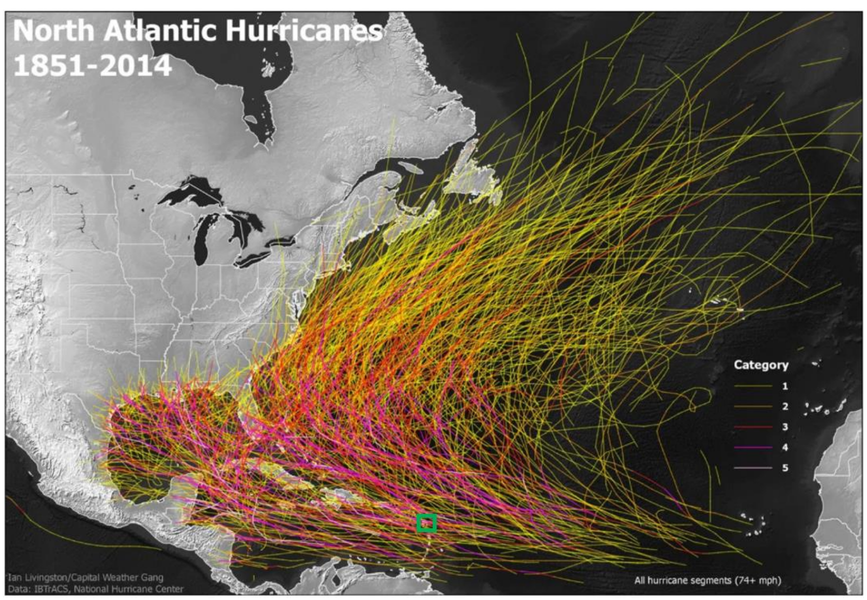

Figure 9: Hurricane segments of all storms in the Atlantic Ocean from 1851-2014. Colors are stacked from category 1 through category 5 here to highlight the most intense activity (Livingston, 2015). The location of St. Maarten (within the green square) is situated within the hurricane zone.

Table 1: Hurricanes and tropical storms to affect St. Maarten since 1960.

Date			Wind Speed (MPH)	Category	CPOA	Name
5	**Sep**	**1960**	**138**	**h4**	**13**	**Donna**
26	Aug	1966	92	h1	42	Faith
17	Jul	1979	46	ts	5	Claudette
3	Sep	1979	58	ts	13	Frederic
4	Sep	1981	40	ts	20	Floyd
6	Oct	1990	69	ts	50	Klaus
5	**Sep**	**1995**	**132**	**h4**	**24**	**Luis**
8	Jul	1996	81	h1	11	Bertha
21	**Sep**	**1998**	**115**	**h3**	**50**	**Georges**
21	Oct	1999	86	h1	16	Jose
18	**Nov**	**1999**	**144**	**h4**	**2**	**Lenny**
22	Aug	2000	75	h1	5	Debby
10	Dec	2007	40	ts	17	Olga
16	**Oct**	**2008**	**132**	**h4**	**40**	**Omar**
30	Aug	2010	121	h3	30	Earl

Categories: ts= Tropical storm, h1= minimal, h2= moderate, h3= extensive, h4= extreme, h5= catastrophic. CPOA= Closest Point of Approach (miles) (Caribbean Hurricane Network, 2011).

In recent years there have been several events that brought considerable damage to the island. In September 1995 St. Maarten was severely damaged by Luis, a category 4 hurricane. In 1996 Hurricane Bertha passed by. In 1998 Hurricane Georges damaged many properties and in 1999, the island was hit by Hurricanes Jose and Lenny causing mudslides, floods and considerable beach erosion (MacRa, Nisbet , & Blok, 2009).

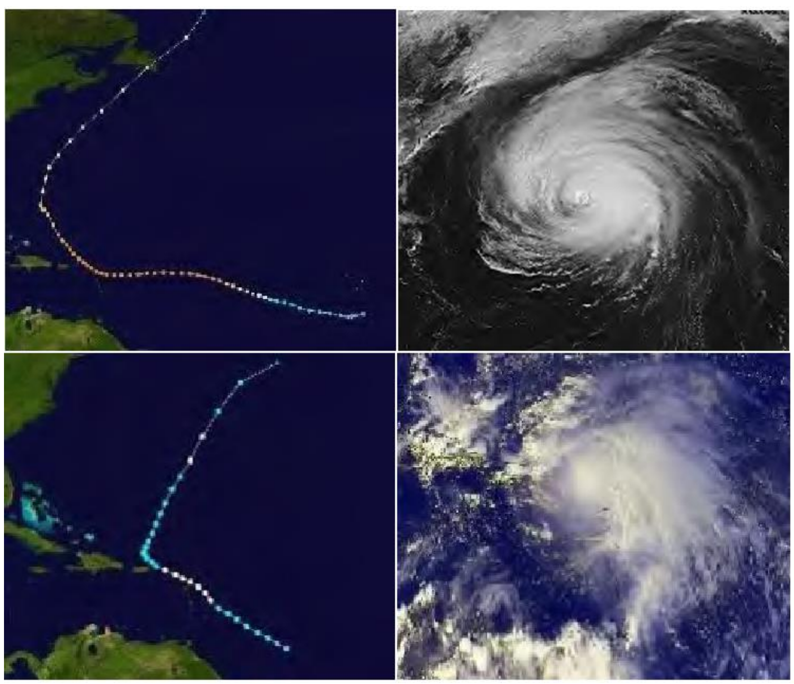

Figure 10: (Top) the path taken by Hurricane Luis in 1995, and an image of Luis passing over St. Maarten in 1995. (Bottom) track of Hurricane Jose and an aerial photograph of Jose passing St. Maarten in 1999 (en.wikipedia.org, 2018).

2.2. Expected local climate changes

In the report published by the Intergovernmental Panel on Climate Change (IPCC) in 2013/2014, four different scenarios (RCP2.6, RCP4.5, RCP6, and RCP8.5) are considered for the different rates and magnitudes of climate change. These scenarios are projected by considering the different amount of possible greenhouse gas concentration that are expected to be emitted in the years to come. The four RCP scenarios are named after a possible range of radiative forcing values in the year 2100 relative to pre-industrial values (+2.6, +4.5, +6.0, and +8.5 W/m2, respectively) (IPCC, 2009). The pre-industrial values determined the listing of the scenario with the smallest value of +2.6 as the 'best' and the highest value of +8.5 as the 'worst or most extreme' scenario with scenario RCP 4.5 considered the most realistic scenario (IPCC, 2013). Furthermore, RCP 4.5 scenario consists of different models, including a minimum, average and maximum projections.

These scenarios were only projected to 2100, hence the short-term projections are until 2050 and the long-term projections are from 2050 till 2100. Sometimes, the years 2035, 2065 and 2100 are used as an example in the report. Therefore, the year 2035 can be considered short–term, whereas 2065 and 2100 can be considered long-term.

In this following sub chapter, the projected results that are solely focused in the Caribbean can be used to correlate to the local climate change in Sint Maarten.

2.2.1. Near surface air temperature

According to the IPCC fifth assessment report (AR5), the annual near-surface air temperature in the Caribbean region is projected to increase in every scenario (Figure 11). This rise of the annual near-surface temperature prediction was further compared the changes between the summer period (June to August) and the winter period (December to February). The rise of temperature during the summer period is slightly higher than the winter period. But overall, there is not significant difference of the rise in temperature between the summer and winter period for all the scenarios (refer to Figure 132 and 13 below).

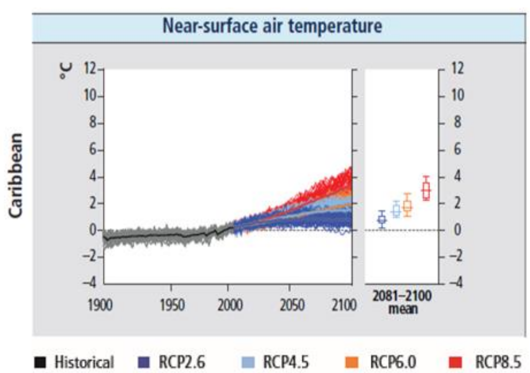

Figure 11: Time series of all RCP scenarios for the annual near-surface air temperature prediction changes relative to 1986–2005 for the Caribbean region (IPCC, 2013) .

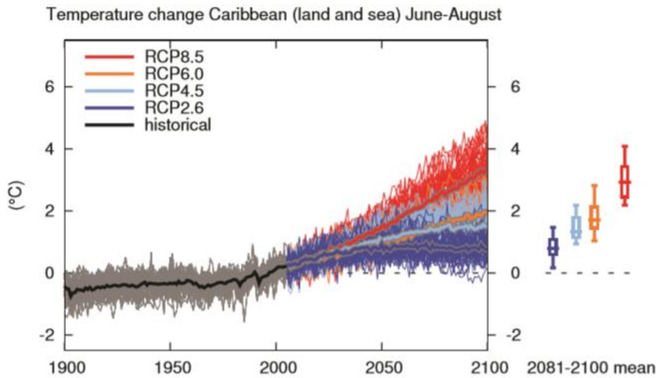

Figure 12: Time series of temperature change during the summer period relative to 1986–2005 for the Caribbean (IPCC, 2013).

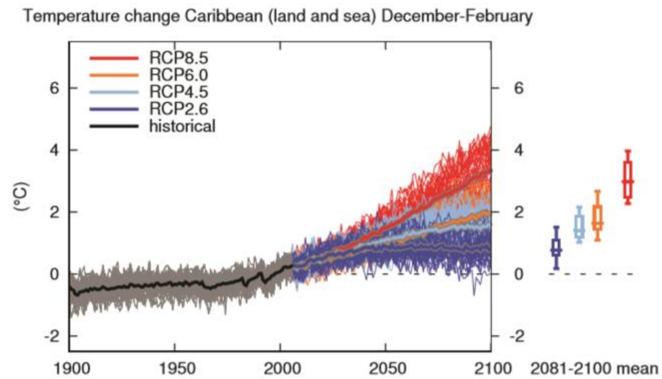

Figure 13: Time series of temperature change during the winter period relative to 1986–2005 for the Caribbean (IPCC, 2013).

For scenario RCP4.5 the short-term projection for near-surface air temperature will rise by a minimum of 0.3 ˚C and of a maximum of 1.1 ˚C by the year 2035 compared with the mean of the period 1986-2005 (IPCC, 2013). Furthermore, by 2065 the air temperature will rise by a minimum of 0.6 ˚C and by a maximum of 1.9 ˚C. Lastly, by 2100 the temperature will rise by a minimum of 0.7 ˚C and a maximum of 2.4 ˚C (IPCC, 2013). The summary of the rise in temperature for scenario RCP4.5 is presented in **Table 1**Table 2.

Table 2: Annual temperature predictions (scenario RCP4.5) for the Caribbean region compared to the mean of 1986-2005 (IPCC, 2013).

Annual Temperature (⁰C)			
Year	**Minimum**	**Average**	**Maximum**
2035	0.3	0.6	1.1
2065	0.6	1.1	1.9
2100	0.7	1.4	2.4

2.2.2. Precipitation

The Caribbean region is affected by several phenomena, this includes the annual cycle which results from air–sea interactions over the Western Hemisphere warm pool in the tropical eastern north Pacific and the Intra Americas Seas (Amador et al., 2006) (Wang et al., 2007). The Caribbean Low-Level Jet is a key element of the region's summer climate (Cook & Vizy, 2010) and is controlled by the size and intensity of the Western Hemisphere warm pool (Wang, 2008). El Niño–Southern Oscillation (ENSO) is the main driver of climate variability, with El Niño being associated with dry conditions and La Niña with wet conditions (Karmalkar , 2011).

According to the IPCC fifth assessment report (AR5), the annual precipitation in the Caribbean region is projected to decrease in every scenario (Figure 14). This reduction of the annual precipitation was further compared to the changes between the period April to September and the period October to March. The reduction in precipitation during the period April to September is slightly higher than the period during October to March in all projected scenarios (Figure 15 and 16 below).

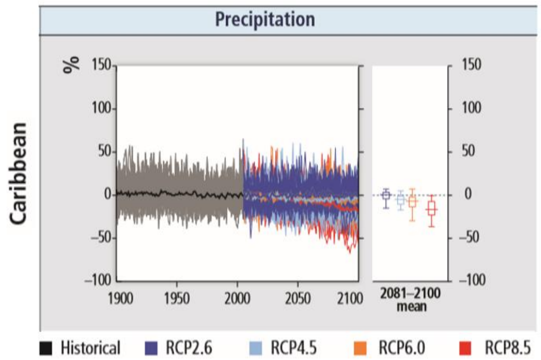

Figure 14: Time series of annual change in precipitation relative to 1986–2005 for the Caribbean (IPCC, 2013).

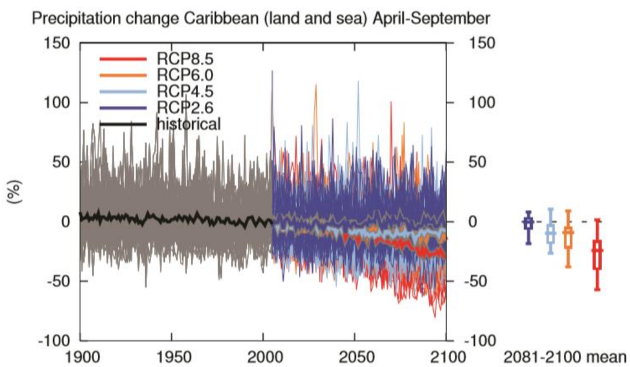

Figure 15: Time series of relative change relative to 1986–2005 in precipitation in April to September for the Caribbean (IPCC, 2013).

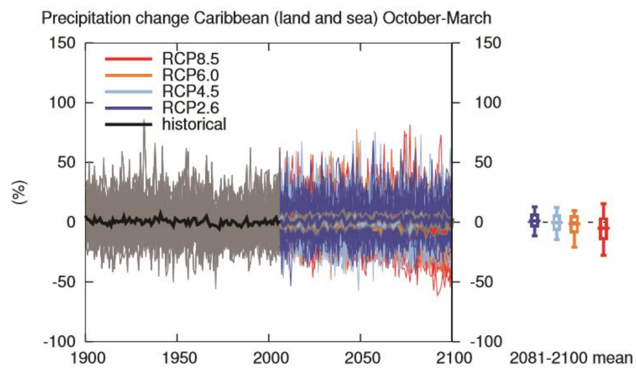

Figure 16: Time series of relative change relative to 1986-2005 in precipitation in October to March for the Caribbean (IPCC, 2013).

The projections from scenario RCP4.5 for both short-term and long- term between now and 2100 shows the minimum precipitation expected to decrease relative to the 1986–2005 mean. This is also the case for the average annual precipitation as well, but not as severe as the minimum precipitation projection. The projection for the maximum annual precipitation for both short-term and long-term expected a huge increased relative to the 1986–2005 mean. The described projection of the precipitation for both the short-term and long-term of scenario RCP4.5 for the Caribbean is presented in Table 3.

Table 3: Annual precipitation projections (scenario RCP4.5) for the Caribbean region compared to the mean of 1986-2005 (IPCC, 2013).

Annual Precipitation (%)			
Year	**Minimum**	**Average**	**Maximum**
2035	-12	-3	8
2065	-19	-5	17
2100	-29	-5	14

Pertaining to the long-term annual precipitation change for scenario RCP4.5 affecting the Caribbean region, the degree of this result (referring to Table 3) reflecting the dry and wet period are presented in Figure 17. Moreover, this seasonal change in precipitation can be related to and expected in the near future in Sint Maarten.

The overall projection in precipitation for Scenario RCP4.5 expects a reduction over much of the Caribbean region, future drying may also be related to strengthening of the Caribbean Low-Level Jet (Taylor et al., 2012) and subsidence over the Caribbean region associated with warmer sea-surface temperature (SSTs) (IPCC, 2013).

ENSO will continue to influence Caribbean climate, but changes in ENSO frequency or intensity remain uncertain. Projected drier conditions may also be related to decreased frequency of tropical cyclone, though the associated rainfall rate of these systems is higher in future projections (IPCC, 2013).

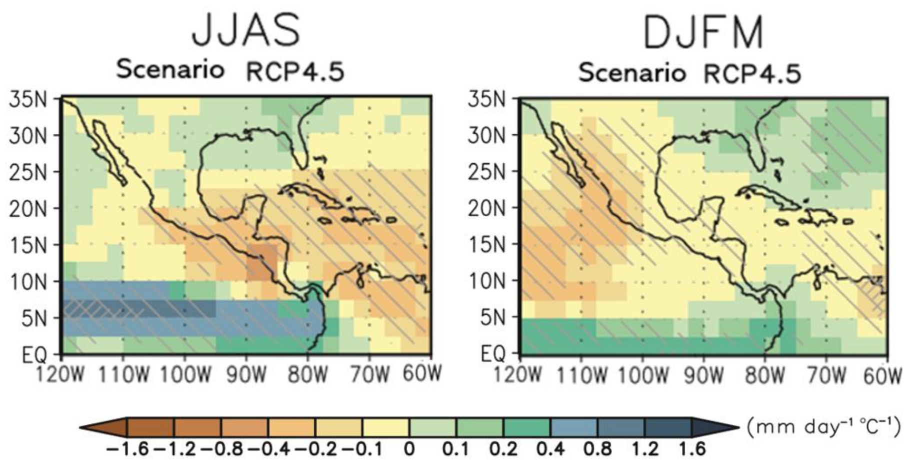

Figure 17: Map of precipitation changes for the Caribbean in 2080–2099 with respect to 1986–2005 in June to September (left) and December to March (right). Precipitation changes are normalized by the global annual mean surface air temperature changes in scenario RCP4.5 (IPCC, 2013).

2.3. Urban environment and stormwater runoff

The primary agent responsible for hydrologic changes associated with the urbanization process, is the increased proportional area under impervious surface (Shuster, Bonta, Thurston, & Warne, 2005) and the reduction in catchment storages as waterways become channelled and piped (Laurenson, Codnernd, & Mein, 1985) (Schuele, 1987b). Such development typically results in a radical and widespread disruption of existing runoff process and flow paths (Booth D. , 1990). As land is urbanized, it becomes covered by impervious surfaces such as paved roads, parking lots and buildings, which prevent rainfall from infiltrating into the ground (Kang, Park, & Singh, 1998). The net effect of these changes is that a higher proportion of rainfall is translated into runoff, this runoff occurs more quickly, and flood flows are therefore higher and 'more flash' than was the case in the catchment before urbanization (Hollis, 1975).

The volume of runoff is governed primarily by infiltration characteristics and is related to land slope and soil type as well as to the type of vegetative cover (Leopold, 1968). One factor stating the relation between the storm and the runoff is lag time (Leopold, 1968). Lag time defined by (Yu, Rose, Ciesiolka, & Cakur , 2000) is the time difference between peak runoff and the mass centre of rainfall. The lag time can validate the relationship between the hydrological lag time and runoff rate and use to quantify the storage effect on runoff rate. The larger the lag time, the greater the attenuation of the runoff rate. Vegetative cover not only increases the amount of infiltration but also reduces the flow velocity, lengthens the lag time, and increases the storage effect on runoff rate (Yu, Rose, Ciesiolka, & Cakur , 2000).

The effects of storm water runoff caused by urbanization is illustrated in Figure 18 and can be summarised in terms of changes in the characteristics of runoff hydrographs generated:

- increased peak discharges and runoff volume
- decreased response time
- increased frequency and severity of flooding
- change in characteristics of urban waterways from ephemeral to perennial systems.

(Wong , Breen, & Lloyd, 2000)

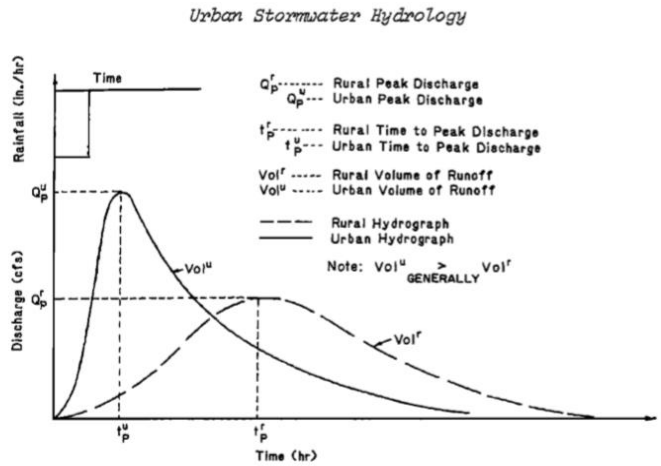

Figure 18: Effect of urbanization on storm water runoff characteristic (Delleur , 1982).

2.3.1. Effects of urbanization on streams

The direct effects on a stream due to urbanization include channel enlargement and flooding. A channel's depth and width can both be increased due to headwater urbanization (Booth D. , 1990). This occurs because of an increase in runoff discharge and frequency, which requires a larger channel in which to be carried. Increase in depth, also known as channel incision, is commonly caused by an "excess sediment-transporting capacity" in relation to the amount of bed material transported from upstream (Simon & Rinaldi, 2006). This excess in capacity is also due to the increased discharge during storm events because of urbanization. As erosion occurs in some areas of a stream, sediment is carried until deposition occurs downstream.

2.4. Drainage methods & mitigation downstream flooding

As a consequence of the urban-induced runoff changes that cause flooding, erosion, and habitat damage, engineered facilities can mitigate many of the hydrologic changes associated with development. As urbanization causes an increased amount of surface runoff, it is important to have drainage facilities that can quickly convey the water away from inhabited areas. This can be done using the roadway's surface itself, drainages next to the road way such as open channels or piped options, and culverts to divert the water collected to the downstream water drainage system. Additionally, with the increased urbanization, there is an increased volume of runoff that will be entering the downstream systems. As these systems are not enlarged alongside the urbanization, it is important to mitigate potential downstream flooding. The most common approach has been to reduce flows and increase retention times through the use of storm water storage facilities. The two most utilized storage methods are retention and detention ponds. Below, a few examples are shown alongside their function.

2.4.1. Road surface runoff

The first method of high volume drainage is the utilization of the roadway to either direct storm water flow to larger drainages. Utilizing the cross-sectional slope of the roadway, water can be dispersed to its outer edge where it can either be transferred to a larger catchment or allowed to run naturally over un-urbanized areas further below (in the case of a hillside roadway such as Waymouth hill road). Another method is to utilize the road itself as a conveyor of water along its transversal slope. This idea has been implemented for example in Denmark. In order to withstand cloud burst events, they have come up with a management plan called that "Cloudburst management plan 2012". It details the use of roadways to act as temporary rivers in order to convey large quantities of water to storage locations (Municipality of Copenhagen, 2012).

2.4.2. Side ditches

In the event that the road surface is not sufficient enough to act as a "river road" and water cannot be released into the surrounding lands, then side ditches next to the road are required to convey the water to a suitable location. Side ditches collect road water and lead it onward to outlet ditches, collection locations, or to further downstream systems (RoadEx Network, 2017).

Figure 19: An example of a side ditch. In this case the side ditch is a triangular profile. (RoadEx Network, 2017).

There are various shapes of side ditches. Their cross-sectional shapes help their function depending on the terrain they are put in. The various types are listed below:

1. Parabolic – This ditch is best in terms of long-term cost and efficiency. It has the same capacity as the trapezoidal side drain (to be explained later) with less erosion. The sides are easily vegetated, further reducing erosion. It is usually the most difficult one to build and it's expensive.
2. Trapezoidal – The flat bottom is easier to construct than the round bottom (parabolic ditch). Compared to the triangular shape (to be explained next), the wider flat bottom slows water and reduces erosion. It takes more time and it's expensive to construct, but does not require as much maintenance and it has a greater capacity.
3. Triangular – The v-shaped bottom ditch is the most easily constructed and requires the least roadside area. Of the three types, it requires the most maintenance, has the lowest water-carrying capacity, and is the most susceptible to erosion.

An option other than side ditches is to use underground pipes with several openings to the road surface along its length. Water would be allowed to pass through the grated openings, enter the pipe and be conveyed along the length of the road profile. While having a large carrying capacity, the cost is high, and the cost of maintenance is also high as the entire road must first be closed, then broken open, in order to repair or enlarge the system as further urbanization may occur (DiBiaso, 2000).

2.4.3. Culvert

If two roads intersect, conveyed rainwater will need to pass under the roads in order to continue on their path to the downstream systems. This task is normally carried out by culverts. A culvert is a pipe or box structure generally used as a cross drain for ditch relief and to pass water under a road. The shape of a culvert is usually a round pipe, but culverts can also be a pipe arch, structural arch or box. The shape depends on the site, the required area, the discharge volumes of water, the required carrying capacity (if cars drive over), and the allowable height of soil cover (RoadEx Network, 2017).

Pipe culverts are widely used culverts and are round in shape. In the event of choosing a single culvert, then a larger diameter will need to be used. If the width of the channel is great and the surrounding land is relatively low in relation to the bed of the channel, then multiple pipe culverts should be used.

Figure 20: An example of a multiple pipe culvert (The Constructor, 2017).

Pipe culverts are suitable for larger water flows but the flow should be stable. This is usually chosen for aesthetical purposes.

Figure 21: An example of a pipe arch culvert (The Constructor, 2017).

Box culverts are rectangular in shape and are generally constructed out of concrete. Reinforcement is also usually required in the construction of a box culvert as they are normally used to go under roadways, therefore they must be able to carry significant weight. Their main purpose is to dispose of rain water and are normally dry otherwise (The Constructor, 2017).

Figure 22: Example of a box culvert. To the right is a single box culvert and to the left is a multiple-box culvert. (The Constructor, 2017).

2.4.4. Storm-water storage

As a consequence of the increased urban-induced runoff, more flooding, erosion and habitat damage have occurred. With the use of engineered facilities, the project can mitigate many of the hydrologic changes associated with development. The most common approach has been to reduce flows through the use of storm water storage facilities. The two most utilized storage methods are retention and/or detention ponds.

Both retention facilities and detention facilities (such as ponds) are intended to capture and detain storm water runoff from developed areas (Booth, Hartley, & Jackson, 2002). The difference between the two are that retention facilities maintain a pool of water throughout the year and hold storm water runoff following storms, whereas detention facilities can be considered “dry” facilities for most of the year, where the exception is when there is a heavy rainfall event, when water may enter, in case the water level rises enough to allow water to enter the detention pond (Laramie County Conservation District, 2016). A classic example of a retention pond can be found in most parks in the Netherlands. These ponds contain water all-year-round with a water-level that is significantly lower than the surrounding land. This allows the pond to store excess storm water. If the levels rise too high, there is usually an outlet to surrounding drainages. The pond allows a relatively controlled release of water.

On the other hand, a classic example of a detention pond can be found in the planning of the “room for the river” projects within the Netherlands (Ministerie van Infrastructuur en Milieu, et. al, 2016). These are areas of sacrificial land and the following occurs: as the water level rises, water can spread out into those areas to reduce velocity and to retain large quantities of water. Detention ponds cannot be used as effectively in parks as they do not slow down the velocity of the water entering the system without the effects of erosion occurring. Retention ponds already contain water that immediately slows down the storm water entering into the pond, minimizing erosion of the bottom and slopes. Both serve the purpose of temporarily holding runoff so that flow rates of a stream do not increase above a desired level (McCoy, 2012).

As a consequence of the increased urban-induced runoff, more flooding, erosion, and habitat damage have occurred. With the use of engineered facilities, the project can mitigate many of the hydrologic changes associated with development. The most common approach has been to reduce flows through the use of storm water storage facilities. The two most utilized storage methods are retention and/or detention ponds.

2.4.5. Weir and orifice

Weirs are overflow structures that stretch across an open channel of water with the purpose of affecting the volumetric rate of water flow. They act like miniature dams, blocking the flow of water and causing it to pool up behind them until the water level rises enough to flow over the top of the weir. In conjunction with a retention pond, a weir will ensure that some water will stay behind in the retention pond.

Orifices are submerged openings with a closed perimeter through which water flows. Orifices are generally used as measuring and hydraulic control devices, but in combination with a retention pond and a weir, they can be utilized to optimize the function of the storage system. For example, the weir has the orifice. What this does, is that the weir traps water behind it, but the orifice allows the retention pond to slowly drain to a lower water level. What this does is that the retention pond now has extra volume storage capacity. As the water level rises again, it will be slowly and controllably spilling through the orifice into the downstream systems at a rate that the downstream system can manage. This will also delay the over-topping of the weir, thereby increasing the retention time. In summary, the combination of a weir and orifice structure to a retention pond will allow for a higher storage potential, less erosion, a controlled release rate of water, and higher retention times (Southern Sandoval County Arroyo Flood Control Authority, 2010).

3. Program of Requirements

The program of requirements states all the criteria that must be considered in order to develop a successful design within the specifications set out by the client; in this case the Ministry of Public Housing, Spatial Planning, Environment and Infrastructure of Sint Maarten (also refer to as VROMI). The final design of this project must meet certain functional and technical requirements. These requirements are determined by the client (VROMI), the guidelines set by VROMI, and the national and international standards (e.g. Eurocodes).

3.1. Functional requirements

<u>Overview of the Waymouth Hills' infrastructural design supporting the future development</u>

- The new stormwater drainage plan will have the function of:
  - Dealing with only stormwater runoff.
  - Having adequate capacity to support the urban development from the surface runoff for a 10-year storm event.
  - Reduce excessive runoff from the urban development overwhelming the Dutch Cul de Sac stream during storm events.
  - Reduce the sediment transport and deposition into the low-lying area, particularly in the Dutch Cul de Sac stream.
  - Where possible and feasible, reuse of effluent for irrigation should be considered, by infiltration points or something similar
- Drainage design should have the advantage in terms of cost, capacity, multiple use (e.g. recreation, wildlife habitat, etc.) and maintenance.
- For the choice of stormwater drainage design, the alternatives are to be evaluated and compared between each other, so as to select the system that is best suited to the project area and the project's goals.
- The drainage system has to be placed next to or underneath the projected road to make it passable even during heavy rainfall.
- The stormwater drainage structure is not necessary if the roadway is able to convey surface runoff but it is mandatory to be present for primary road (e.g. Mildrum Road).
- Changes of the original development plan must not affect the amount of living space planned and or decrease the land value within the development plan and /or limit the safety of road ways.
- The drainage system has to be constructed in such manner that it fits the road construction and time lifespan required.
- Drainage designs shall be reviewed to determine if some form of protective treatment will be required to prevent entry to facilities that present a hazard to children and to adults.
- The design and location of open channels shall comply with roadside safety and clear zone requirements.

3.2. Technical requirements

Storm drains:

- The construction of the structure shall be carried out using precast or cast in situ concrete.
- The structural lifespan of the drains required is 50 years.
- The design of the drains shall be based on the peak stormwater runoff traveling in each section of the drain accordingly.
- The maximum filling in the drain allowed is 75% of the proposed drain size.
- The velocity of the filling shall not exceed 6 meters per second in the drains.
- The longitudinal slope of the drains shall adapt to existing terrain as much as possible.
  - The maximum longitudinal slope of the drain shall not produce a velocity that exceeded the permitted.
  - The minimum longitudinal slope can be zero percent only if it does not exceed the maximum filling permitted.
- The channel width shall be designed to accommodate the hydraulic capacity of the cross-section, recognizing the limitations on velocity and depth. Width shall be adequate to allow necessary maintenance.
- The sides wall of the drains should be at least 0.5 meter away from any structural walls.
- The foundation of the drains shall be designed in a manner such as no significant differential settlements will occur and that it won't require significant repair works within 20 years after completion.

Drain inlet:

- Grated drain inlet is mandatory on primary road to converge runoff into the storm drains
  - Inlets shall be placed at the low points in the street grade.
  - Center to center (C.T.C) distance of 2.5 meter between drain inlet is required.
- Opening of road curb is to be used as drain inlet on side road to converge runoff into the storm drains.
  - Maximum C.T.C curb opening of 1 meter is required.

System outlet:

- The outlet of a drainage system must be placed at a location where the downstream area or receiving stream is capable of accepting the design flow.
- Downstream erosion, stream degradation and flooding impacts must be considered.

Land use:

- The construction of any structure shall remain within the parcel boundary of the state (government). If this is inevitable, avoid the use of easement as much as possible.
- Disrupting of the natural environment and habitat is to be avoided as much as possible.
- Minimize disruption of existing topography, erosion, and sedimentation problem by reducing and limiting cut and fill requirement.

Detention pond:

- Stormwater must be detained such that the peak flow rate released from the site does not exceed 0.05 cubic meter per second per ha (m3/s /ha). The following limitations apply to detention basin.
- No part of the bottom of a landscaped detention area may be flatter than 3% slope.

- Within 3 meter of the outlet, the slope of a landscaped basin bottom must not be flatter than 5% unless a concrete apron is constructed around the outlet to control erosion occurring in the receiving downstream channel.
- Storm drain channels are to continue through detention areas to allow low flows to proceed through the storm drainage system without having to come to the surface. These low flows must still pass through the outlet restriction that limits runoff rates.
- Basins are to be designed such that water does not run into them after they reach a maximum depth (unless a free-flowing overflow is provided).
- Outlet works selected for the detention pond shall include a principal spillway and an emergency overflow to convey flows larger than those which can be handled by the storage system or to divert water in case the system becomes inoperable for any reason.
- The max depth of 3 meter (bottom of the basin to the top of the ground surface) is allowed on the basins.
- The freeboard of the basin should have at least 0.2 meter from the bottom of the basin to the top of the ground surface.
- Side slopes shall not be steeper than 3-meter horizontal to 1-meter vertical (3:1).

Structures stability:

- For any structure constructed using concrete, the following Eurocode shall be used:
  - EN 1990 – Basics of Structural Design
  - EN 1991 – Actions on Structures
  - EN 1992 – Design of Concrete Structures
  - EN 1994 – Design of composite steel and concrete structures
- Structure shall resist earth quake load from earth quake zone 3

The program of requirement in this chapter has set up to fulfil the final design of the new stormwater drainage pertaining to this research report. Moreover, a separate technical requirement has been drawn up for the realization of the infrastructure upgrade in the Waymouth Hills: this includes the construction of new roads (resurfacing road pavement), the stormwater drainage and followed with relevant activities involving the total infrastructure upgrade (refer to Appendix F).

4. Methodology

The overall methodology of this research study is shown in Figure 23 and discussed in this chapter. In order to achieve the research objective, the relevant literature on the local climate, the effect of urbanization and the mitigation of the storm water runoff was reviewed to acquire the knowledge about, conventional and current methodology.

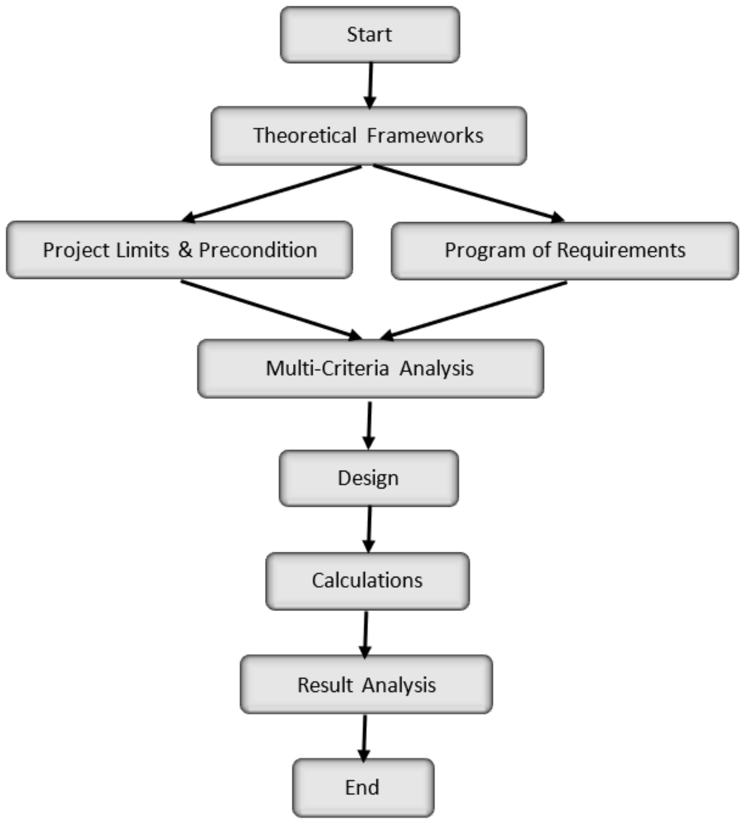

Figure 23: Methodology flow chart of this research study.

4.1. Multi-Criteria Analysis

Multi-criteria analysis (MCA) is a tool used to help decision-makers finding the best compromise (or solution) out of many alternative options to a complicated problem. For the purpose of this research, a MCA was conducted to identify the most feasible stormwater drainage system that can be used for the Waymouth Hills.

To do so, three viable alternatives have been set up to evaluate and compare the alternatives against each other. The comparison of the alternatives was based on criteria relevant to the established project specifications. Weights are assigned to each criteria in order to highlight their importance in the MCA. The evaluation of each alternative was scored (i.e. through rating) by assessing how well they perform with respect to each criterion and a pre-defined scale is used for this. The alternative that received the highest score is the alternative that (in overall) is associated with the most positive grading (i.e. the most proffered among the selected alternatives).

4.1.1. Identified criteria

Six criteria have been formulated for the comparison of the alternatives which would best reflect the important aspects which were required to achieve the goal and objectives of the study. The criteria that were chosen for the evaluation are described below in Table 4.

Table 4: Summary of the criteria used for the MCA.

Criteria	Parameter analyzed	Description
Investment cost	• Land cost • Material and construction cost	• Different alternatives may require different amount of land area for construction – estimation of the land costs • Estimation of construction- and material cost for building the stormwater alternative on site.
Maintenance cost	• Operation- and maintenance costs of the system	• Estimation of the frequency of maintenance required for each alternative and the cost for such activity.
Environmental impact	• Impact on urban soil quality and erosion potential • Impact on the ecological habitat	• Estimation of effect on soil quality on site, sediment retention and erosion potential. • Potential for change in biological diversity at site.
Design aspects	• Complexity of construction	• Different alternative may require specific type of activities, machinery and equipment
Structural reliability	• Lifespan and reliability of structure	• Estimation of lifespan of system and measure of strength
Implementation time	• Duration of implementation	• Estimation of the duration required for the implementation of each alternative.

4.1.2. Identified criteria's weight

A total score of 100% was used which was divided in accordance to their importance for this research into the following:

Investment Cost: The investment cost provides a large influence on the global decision making, since it reflects the spending from the client or any parties involved. Therefore, this criterion is given 25%.

Maintenance cost: The activities involved in the alternatives will reflect the overall long- term cost of the client's investment. Therefore, this criterion is given 20%

Environmental impact: The terrain in the study area are predominately with steep slope. Disrupting of the natural environment and habitat will increase potential erosion in the study area, clogging the drainage systems, and reflect increase sediments transport to lower lying terrain. Therefore, this criterion is given 20%.

Design aspects: This criterion relates the functions of the structure and their capability to perform in long-term. Therefore, this criterion is given 15%.

Structural reliability: The construction of the complex drainage system may require the use of specific machineries and equipment. Accessibility of these machineries to the study area may be limited. Therefore, this criterion is given 10%.

Implementation time: Construction activities can interfere the traffic flow in the study area. Since, the hurricane season is during the period from June to November (5 months period), construction within this period can be crucial. Work can be disrupted and delayed, which translate a higher investment cost. Therefore, this criterion is given 10%.

4.1.3. Scoring of the alternatives

The scoring of each criterion was carried out in terms of a rating. A scoring set of 5 options was used to rate the different alternatives with respect to the criteria in accordance to their performance presented in Table 5. The three alternatives were compared and scored separately from each other.

Table 5: Summary of the alternative rating for the MCA.

Criteria	Alternative Rating				
	1 - Poor	2 - Fair	3 - Good	4 - Very good	5 - Excellent
Investment Cost	Large easement required, construction and material are very expensive	Average easement required, construction and material are very expensive	Average easement required, construction and material are expensive	Average easement required, construction and material are average	No easement required, construction and material are cheap
Maintenance Cost	Very frequent maintenance required, and overall operation is very expensive	Frequent maintenance required, and overall operation is expensive	Few maintenance required, and overall operation is expensive	Few maintenance required, and overall operation is average	Few maintenance required, and overall operation is cheap
Environmental impact	Very high impact on soil quality, erosion potential and biodiversity changes	High impact on soil quality, erosion potential and biodiversity changes	Average impact on soil quality, erosion potential and low biodiversity changes	Low impact on soil quality, erosion potential and low biodiversity changes	No impact on soil quality, erosion potential and biodiversity changes
Structural reliability	Very low lifespan and delicate	Low lifespan and low sturdy	Average lifespan and average sturdy	Long life span and sturdy	Very long lifespan and very sturdy
Design Aspect	Very specific activities, machinery and equipment required	Specific activities, machinery and equipment required	General activity, machinery and equipment required	General activity and machineries required	General activity and equipment required
Execution time	Very long duration	Long duration	Average duration	Short duration	Very short duration

4.2. Rainfall runoff & stormdrain calculation

Calculations required for this research is the analysis for the storm water runoff, hydraulic structure required for the runoff and determine its hydraulic capacity. The storm water runoff analysis includes the current development scenario, the future urban development scenario and the necessary mitigation measure in case of flooding occuring within the system. The calculation for the hydraulic structure will be based on the peak rainfall runoff from the future urban development runoff analyses. To achieve the outcome, steps were set and discussed in this chapter.

4.2.1. Runoff analysis

In order to analyze the runoff potential for the Waymouth Hills catchment, firstly, the catchment was divided into sub catchment to more accurately analyze its characteristics. These sub catchments were determined using a contour map of the entire catchment by analyzing the direction of surface flow from the catchment. The storm water runoff from each sub catchment defined the stream network and the area with runoff converging to each and every stream was identified. The sub-catchments of concern were then identified using alphabet and/or followed by numbers. Furthermore, each stream was identified using numeric numbers.
The Rational method was used to give a simple overview of the runoff analysis pertaining to the future urban development scenario. This method of the runoff analysis carried out manually is defined in E.q (1).

$$Q = C.I.A \tag{1}$$

where:
Q= maximum rate of runoff (m^3/s), C = runoff coefficient (-); I= average rainfall intensity (mm/hr.);
A= sub catchment area (m^2)

The runoff coefficient is a dimensionless ratio intended to indicate the amount of runoff generated by a watershed given an average intensity of precipitation for a storm (Thompson, 2006). Hence, in order to determine the potential runoff from each such catchment, Table 6 was used which was based on the terrain slopes and land use.

Table 6: Runoff Coefficient for Rational Method (Basisrioleringsplan Sint Maarten, 1998)

Runoff Coefficients for Rational Formula	
Land use characteristics	**Runoff Coefficient, C***
Urban area: relatively flat lots of pavement, densely built with businesses, shops, and apartments	
Slope <1%	0.9
Residential area: detached, free standing houses, some stores, moderately infiltrating soil	
Slope <1%	0.3
Slope 1-7%	0.4
Slope >7%	0.6
Undeveloped area: overgrown with grass, shrubs and trees, moderately infiltrating soil	
Slope <1%	0.10
Slope 1-7%	0.25
Slope >7%	0.35

The parameter Rainfall intensity I, is defined (Critchley & Siegert, 1990) as the ratio of the total amount of rain (rainfall depth) falling during a given period to the duration of the period. It is true that the longer the return interval (hence, the shorter the exceedance frequency), the greater the precipitation intensity for a given storm duration. Furthermore, the longer the length of the storm, the lower the storm average rainfall intensity. These data are represented by the intensity-duration-frequency (IDF) curve. Hence, to determine the rainfall intensity for the study area, Table 7 (values derived from the IDF curve) was used.

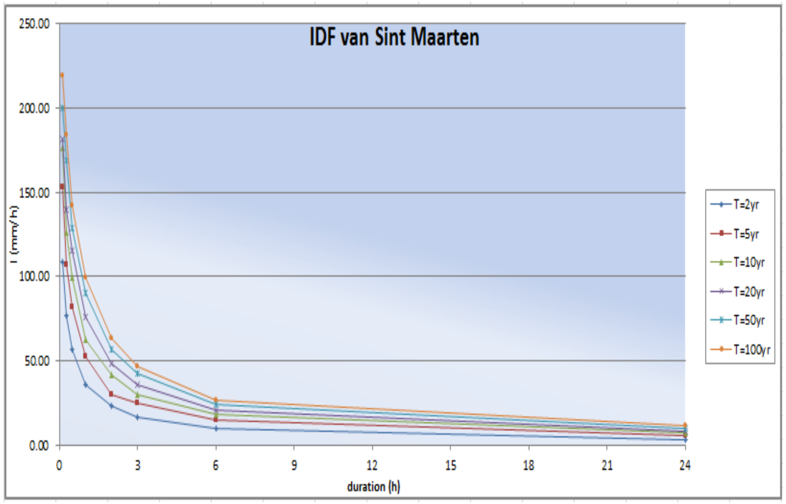

Figure 24: IDF curve of Sint Maarten (Vojinovic & Bonilo, 2006).

Table 7: Intensity values (mm/hr) for different duration and return period.

Duration	Return Period (years)					
(hour)	2	5	10	20	50	100
0.1	108.3	152.8	176.3	181.8	199.9	219.5
0.25	77.2	107.3	126.7	139.6	169.1	184.0
0.5	56.6	81.6	99.1	115.0	128.8	141.8
1	36.1	52.4	62.4	76.4	90.3	99.7
2	23.0	30.0	41.3	48.1	56.7	63.2
3	16.4	24.9	30.3	35.8	42.3	47.0
6	9.8	14.6	18.4	20.7	24.0	26.8
24	3.4	5.6	7.0	8.3	10.1	11.5

The returned interval was selected based on the program of requirement, and the duration of the storm is calculated for each sub catchment. Since, the time of concentration is equivalent to the duration of the storm when peak rainfall occurs. After obtaining the time of concentration from of each sub catchment, the values presented in Table 7

Duration	Return Period (years)					
(hour)	2	5	10	20	50	100

0.1	108.3	152.8	176.3	181.8	199.9	219.5
0.25	77.2	107.3	126.7	139.6	169.1	184.0
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1	36.1	52.4	62.4	76.4	90.3	99.7
2	23.0	30.0	41.3	48.1	56.7	63.2
3	16.4	24.9	30.3	35.8	42.3	47.0
6	9.8	14.6	18.4	20.7	24.0	26.8
24	3.4	5.6	7.0	8.3	10.1	11.5

Table 7 were graphed to determine the linear function of the rainfall intensity associated with each sub catchment.

The time of concentration (T_c) defined in NRCS (NRCS, 1986) is the time at which the entire watershed begins to contribute to runoff. *Tc* was calculated using the TR-55 method defined in E.q (2).

$$T_c = T_{ti} + T_s + T_v \qquad (2)$$

Where:

Tc=time of concentration (mins); Tti= sheet flow (min); Ts=shallow concentration (min); Tv= channel flow (min).

Since, the time of concentration is dependent on the type of flow in each sub catchment. The flow type occurred for each sub catchment was determined visually in Civil 3D and was calculated accordingly.

Sheet flow is which is defined as "flow over plane surfaces". Time of travel for sheet flow (less than 130 meters) was found using E.q (3).

$$T_{ti} = \frac{0.692}{(I)^{0.4}} * \left(\frac{n * L}{{S_p}^{0.5}}\right) \qquad (3)$$

Where:

*Tti= Sheet flow (min); n= Roughness coefficient (**Error! Reference source not found.**Table 8); L= Flow length (m); I= rainfall intensity (mm/hr); Sp= Land slope (m/m).*

Since *I* depended on *Tc* and *Tc* was not initially known, the computation of *Tc* was an iterative process. This was carried out first by using an initial estimate of *Tc* which is assumed and used to obtain *I* from the IDF curve for the locality. If they are not the same, the process was repeated until two successive *Tc* estimates are the same (Brown, Schall, Morris, & Dohert, 2013).

Table 8: Roughness coefficient (manning's n) for sheet flow (Brown, Schall, Morris, & Dohert, 2013).

Roughness coefficients (Manning's n) for sheet flow

Surface description	n 1/
Smooth surfaces (concrete, asphalt, gravel, or bare soil)	0.011
Fallow (no residue)	0.05
Cultivated soils:	
Residue cover ≤20%	0.06
Residue cover >20%	0.17
Grass:	
Short grass prairie	0.15
Dense grasses 2/	0.24
Bermudagrass	0.41
Range (natural)	0.13
Woods: 3/	
Light underbrush	0.40
Dense underbrush	0.80

For flow lengths with shallow concentrated flow, E.q. (5) was used to calculate the flow velocity. The flow velocity is influenced by the flow paths conditions and the characteristic of land coverage defined as the intercept coefficient given in Table 9. The time of travel for shallow concentrated flow was calculated using E.q. (4)(4 after the velocity was found.

$$T_s = {}^{\mathrm{L}}/_{v} \tag{4}$$

$$v = K_u \cdot \mathrm{K} \cdot {S_p}^{0.5} \tag{5}$$

Where:
v= *velocity (m/s); Ku =coefficient (10); K= Intercept coefficient (**Error! Reference source not found.**) (-) ; Sp= s lope per cent (m/m); L = Flow length*

Table 9: Intercept Coefficients for shallow concentrated flow using Velocity vs Slope Relationship (McCuen, Johnson, & Ragan, 2002).

Intercept Coefficients for Velocity vs. Slope Relationship	
Land cover/ Flow Regime	*k*
Forest with heavy ground litter; hay meadow (overland flow)	0.076
Trash fallow or minimum tillage cultivation; contour or strip cropped; woodland (overland flow)	0.152
Short grass pasture (overland flow)	0.213
Cultivated straight row (overland flow)	0.274
Nearly bare and untilled (overland flow); alluvial fans in western mountain regions	0.305
Grassed waterway (shallow concentrated flow)	0.457
Unpaved (shallow concentrated flow)	0.491
Paved area (shallow concentrated flow): small upland gullies	0.619

Channel flow is the flow located in an open channel. The time of travel can be found with E.q. (4), with the velocity being determined with Manning's Formula, given in E.q. (6).

$$V = \frac{K_u}{n} \cdot R^{2/3} \cdot S^{\frac{1}{2}} \tag{6}$$

Where:
*V = velocity (m/s); n = manning's roughness co-efficient (**Error! Reference source not found.**) (-); S=Slope (m/m); K_u=co-efficient (-); L= flow/ channel length; R=hydraulic radius (m) which is the ratio of cross-sectional area to wetted perimeter of the channel.*

Table 10: Manning's roughness coefficient (n) for channel and pipe (McCuen, Johnson, & Ragan, 2002).

Table 3-4. Typical Range of Manning's Coefficient (n) for Channels and Pipes.	
Conduit Material	Manning's n*
Closed Conduits	
Concrete pipe	0.010 - 0.015
CMP	0.011 - 0.037
Plastic pipe (smooth)	0.009 - 0.015
Plastic pipe (corrugated)	0.018 - 0.025
Pavement/gutter sections	0.012 - 0.016
Small Open Channels	
Concrete	0.011 - 0.015
Rubble or riprap	0.020 - 0.035
Vegetation	0.020 - 0.150
Bare Soil	0.016 - 0.025
Rock Cut	0.025 - 0.045
Natural channels (minor streams, top width at flood stage <30 m (100 ft))	
Fairly regular section	0.025 - 0.050
Irregular section with pools	0.040 - 0.150
*Lower values are usually for well-constructed and maintained (smoother) pipes and channels	

4.2.2. Hydraulic analysis

Once the peak discharge in the individual stream has been found, the capacity of the drainage structures can then be determined. Road pavement are designed to not only facilitate safe traffic movement but also collect and convey the concentrated storm water runoff in a storm event (Guo, 2000). Hence, the conveyance capacity of the proposed road geometry was first analyzed by using the revised Manning equation (Izzard & Hicks, 1946) defined in E.q 7.

$$Q = \frac{K}{n} Sx^{1.67} \cdot T^{2.67} \cdot \sqrt{S_o} \tag{7}$$

Where:

Q= Street hydraulic conveyance capacity (m^3/s); K= Coefficient (1) (-); S_x= Street transverse slope (m/m); S_o= Street longitudinal slope (m/m); T= water spread width on the street (m); n = Manning roughness ($s/m^{1/3}$).

In the streams network, where the proposed road geometry was not sufficient to convey the peak discharge, a storm drainage structure was used. The dimension of the storm drain needed for each reach was calculated by using the Manning Equation defined in E.q. (8).

$$Q = (A * R^{0.667} * S_o{}^{0.5})/n \tag{8}$$

Where:

Q= Discharge capacity (m^3/s); A=wetted area (m^2); R= hydraulic radius (m); S_o= longitudinal slope (m/m); n=manning's roughness factor ($s/m^{1/3}$) (Table 10).

E.q. (8) required the wetted area and the hydraulic radius of the drain, hence, to determine this information, a desired size drain was assigned to each reach and initially assumed with a 75 % filling. The results of the hydraulic capacity for each drain structures were then compared to the peak runoff entering to it. If the hydraulic capacity of the initial drain size was not sufficient, a larger drain size was assigned until the peak runoff was satisfied. When the capacity of the drains versus the peak runoff is determined sufficient, the actual filling of the peak runoff in each drain is calculated, since the travel time in channel flow affects the time of concentration, and overall the rainfall intensity and peak runoff. This process of calculating the actual filling was repeated until two successive Tc are the same.

4.2.3. Hydrodynamic modelling

In order to achieve better understanding and effect of surface runoff caused by future urban development and the hydraulics in the designed drainage network structures of the Waymouth Hills, Autodesk Storm and Sanitary Analysis (SSA) was used for the hydrodynamic modeling. SSA is an advanced, powerful, and comprehensive modelling package for analyzing and designing urban drainage systems, storm water sewers, and sanitary sewers. The software can simultaneously model complex hydrology, hydraulic and water quality. Typical application includes design and sizing of drainage system components and detention facilities for flood control, as well as, floodplain mapping of natural channel systems.

Three models were created in SSA for the analysis of the hydrologic effects of the future urban development and the hydraulically effect in the drainage network structures. The first model was created to compare and verify the conveyance capacity of the proposed structures from the manual calculation. To do so, the characteristics of the sub-catchment and the stream network (defined in chapter 4.2.1) were incorporated into this model, together with results of the proposed drainage structures obtained from the manual calculation. This model was simulated using the Rational method and incorporated the stationary rainfall of the 10-year storm event data from the IDF curve defined Table 7.

The second model was created to illustrate the hydrologic effect of the current development scenario and the hydraulic capacity of the existing drainage network (existing road network). The hydrologic effect of the current development scenario was simulated using the EPA SWMM (United States Environmental Protection Agency Storm Water Management Model) and was incorporated with dynamic rainfall event of the 10-year storm. This type of simulation required the use of the Soil Conservative Service (SCS) method to determine the stormwater runoff.

SCS is a statistical method for peak flow determination based on rainfall, soil type, and land use (McCuen R. , 2005). This method uses a variable known as Curve Number (CN) that represents the specific hydrologic soil group (HSG), land cover, antecedent moisture condition, and hydrologic condition of an area (NRCS, 1986). The value of CN varies between for 0 to 100, with 0 resulting in no runoff and 100 representing a completely impervious area which generates an excess rain equal to the rainfall. For natural catchments CN it is normally between 50 and 100.

The main hypothesis of the SCS method is that the ratio between the additional water retained in catchment area after the start of the runoff process and the potential maximum retention is equal to the ratio between the excess precipitation and the potential runoff:

$$\frac{Fa}{S} = \frac{Pe}{P - Ia} \qquad (9)$$

Where:

Ia = initial abstraction (Losses occurred before runoff begins)
Fa = additional depth of water retained in the sub-catchment after the start of the runoff process
Pe = excess precipitation contributing to runoff

P = rainfall (equal to Pe+ Ia + Fa)
S = Potential maximum retention after runoff begins.

The potential maximum retention, in turn, is directly related to the initial abstraction, *Ia*, as displayed in E.q. (10).

$$I_a = 0.2 \text{ x } S \tag{10}$$

Considering Ia=0.2*S and arranging the equation, the depth of excess rainfall from a storm is defined in E.q. (11).

$$Pe = \frac{(P - I_a)^2}{(P - I_a) + S} = \frac{(P - 0.2 \text{ x } S)^2}{(P + 0.8 \text{ x } S)} \tag{11}$$

Based on the soil type and the land use and the land use an equivalent curve number can be defined for each sub-catchment. The value of S (in mm) and the curve number, CN, are define in E.q. (12).

$$S = \frac{25400}{CN} - 254 \tag{12}$$

The curve number applied in this research was obtained from the report (St Maarten Stormwater Modelling Study, 2006), the CN was identified based on the land use and the type of soil. As the slope has an influence on the sub-catchment runoff. The CN values obtained from the report is presented in Table 11.

Table 11: CN for each land use and soil slope (Vojinovic & Bonilo, 2006).

Land Use	Slope	CN
Ponds		100
Building and paved surfaces		95
Non-developed	>40°	71
	30°- 40°	68
	20°- 30°	65
	10°- 20°	61
	0°- 10°	58

Since the land use changes overtime, the value of the CN was calculated differently to represent the present and future urban development scenarios. The CN corresponding to each sub-catchment is calculated weighting the CN value in Table 11 by the percentage of the sub-catchment with each land use and slope range for the modelling. Moreover, the properties of the existing network's structures incorporated for this simulation were obtained from available field survey data.

Furthermore, the third model was created to compare the hydrologic effect of the future urban development scenario with the current development scenario. The corresponding CN value of the sub-catchment pertaining to the future urban development scenario was incorporated in the model and was simulated using the EPA SWMM method. This model was also simulated with the dynamic rainfall of the 10-year storm event to analyze the hydraulic capacity of the proposed drainage structure and compare the effect of stationary rainfall event (from Model 1) versus the dynamic rainfall effect having on the future urban development drainage system.

The current and future urban development scenarios models (Model 2 and 3 respectively) using the EPA SWMM method were simulated with different CN values for each sub-catchment. These values were calculated by taking into account the degree of development and different slopes terrain. Refer to Appendix C to see the method used to calculate the CN values used for the present development and future urban development scenario.

Lastly, the Dutch Cul de Sac stream was incorporated into both the present development and future urban development models to analyze the hydraulics in the stream from the both scenarios. Then, a detention pond was introduced in the upstream of the Dutch Cul de Sac to the future urban development scenario (the third model), to assess the benefits of the pond use to mitigate potential flooding from the excessive stormwater runoff.

5. Results and Discussion

5.1. Multicriterial analysis results

5.1.1. Identified alternatives

Three alternatives were drawn up and incorporated for the MCA. These alternatives were proposed to reflect and solve the defined problem of this research. The determined alternatives used for the MCA are an open ditch, a concrete U-Gutter and an underground drain.

Alternative 1

The first alternative suggested using an open ditch along the road (Figure 25). An open ditch would then be constructed on the lining of the road where storm water runoff could be collected and conveyed from the surface runoff from sub-catchment areas and from the road pavement. Open ditch could be designed as V-shaped, U-shaped and or trapezium shaped channel sections. The shape or the slope in which the open ditch could be used is significantly dependent on the soil properties as the banks of the open ditch could collapse when over-saturated. The construction for this type of drainage can be relatively easy and cheap to construct comparing to other stormwater drainage systems. Due to the necessity of having slopes for its stability, it requires more space. In areas where the terrain profile is steep, scouring or undercutting on the bottom and sides of the channel may occur. This result in transporting debris and may block the drainage downstream. This type of storm drainage not only requires more frequent maintenance of the bank but also the downstream drainage where sediments is deposited or settled.

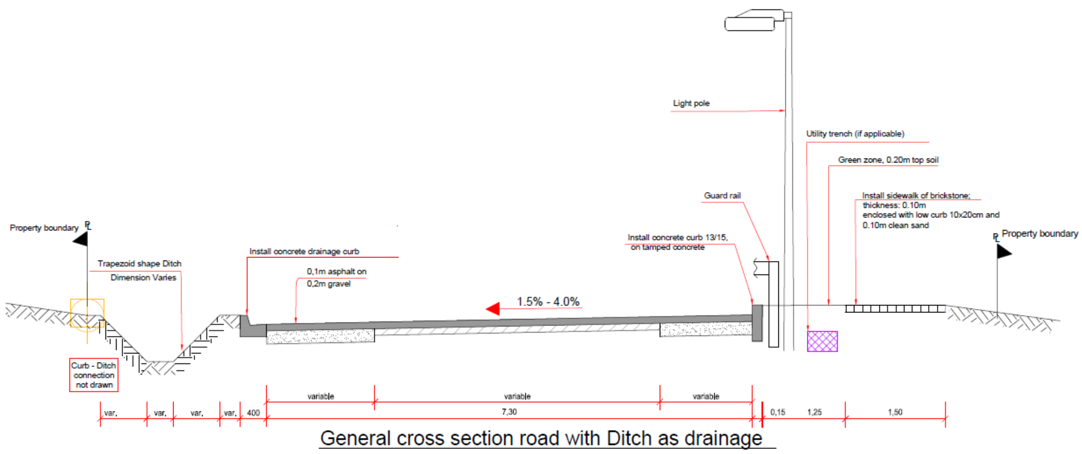


Figure 25: Alternative 1 (Open ditch).

Alternative 2

The second alternative was to use a concrete gutter (Figure 25). This type of storm water drainage works similarly to an open ditch and is also constructed on the lining of the road. Concrete gutter could have any desired shape and does not require side slopes for its structural stability (depending on the shape of the channel used). Square and rectangular concrete gutters are mostly used in urban areas as they take up less space ensuring that space for the road infrastructures are not limited by these drainage systems. This type of drainage can be constructed in situ or precast concrete and owing to its durability, erosion of the channel is significantly lower in comparison to open ditch.

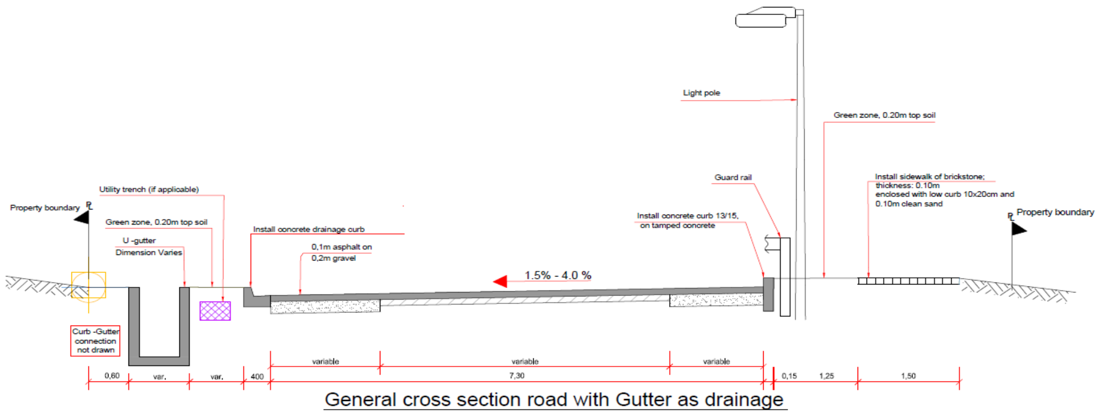


Figure 26: Alternative 2 (Concrete U-Gutter).

Alternative 3

The third alternative suggested the use of underground drains. This type of storm water drainage is mainly used in the form of precast concrete or plastic elements. It is built beneath the center or on the sides of the street. This type of drain is mainly used in urban areas and city centers, where very limited space is available. Having this type of drain utilizes the road space more efficiently. This type of drain can be used for stormwater, waste water purposes and or combined. For stormwater drainage purpose, stormwater is collected or fed by the side inlets from street curb and grated inlet.

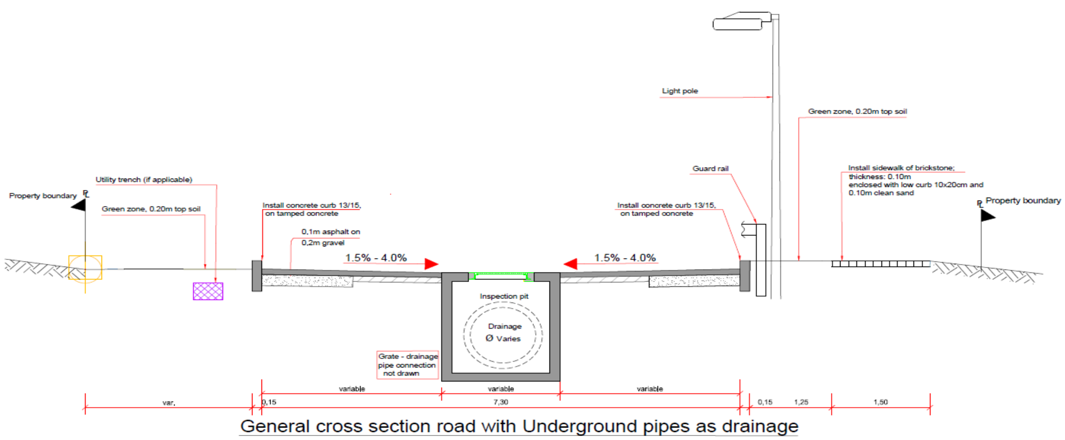


Figure 27: Alternative 3 (Underground Drain).

5.1.2. Final result of the MCA

The final result (also called weighted score) of the MCA is calculated by multiplying the scores obtained from each criterion (Table 12) by their importance level (weight value, refer to 4.1.2). The alternative with highest score is the most feasible alternative. See Table 13 for the summary of the final scores for the alternatives.

Table 12: Rated scores from MCA.

Criteria	Alternative 1	Alternative 2	Alternative 3
Investment Cost	4	3	2
Maintenance Cost	2	4	3
Environmental impact	1	3	3
Structural reliability	2	5	4
Design Aspect	4	2	1
Execution time	4	2	1
Sum:	17	19	14

Table 13: Weighted Scores from MCA.

Criteria	Alternative 1	Alternative 2	Alternative 3
Investment Cost	1.00	0.75	0.50
Maintenance Cost	0.40	0.80	0.60
Environmental impact	0.20	0.60	0.60
Structural reliability	0.30	0.75	0.60
Design Aspect	0.40	0.20	0.10
Execution time	0.40	0.20	0.10
Sum:	2.70	3.30	2.50

To sum up the MCA results, Alternative 2 secured the highest score (for both the rated scoring and weighted scoring) followed by Alternative 1 and then Alternative 3. The area that Alternative 2 scored the most in is structural reliability, due to the fact that with this alternative the structural lifespan of concrete can be up to 50 years. Moreover, this alternative does not need to withstand traffic load whereas Alternative 3 must, since it is constructed beneath the street profile. This additional load acting on Alternative 3 might expect larger settlement, and with preventative measures the cost of investment will be increased as well, hence the scoring of this criterion for Alternative 2 was more favorable than Alternative 3. Moreover, Alternative 1 is constructed by lining of natural soil or vegetative and hence its lifespan was expected to be the shortest in comparison to both Alternative 2 and 3. Additionally, it scored the least among the alternatives.

The area that Alternative 2 scored least in was both the design aspect and the execution time; due to the fact that the construction is in concrete, it requires involving more complex activities and it also results in a longer execution time overall. Alternative 3 is in same situation, but its construction requires much more complex activities in comparison to Alternative 2, hence it scored less compared to Alternative 2. On the other hand, the activities involved in the execution of Alternative 1 are minimum and the least complex comparing to both Alternative 2 and 3, hence resulting in a more favorable score. Refer to Table 3A to 5A (Appendix A) for the description and motivation of the scoring result from each alternative.

5.1.3. Sensitivity analysis

A sensitivity analysis was performed for MCA to further evaluate the results and see how much the results (i.e. weighted scores) are affected if the weights given to each of the six criteria are changed but using the same rated score presented in Table 12. An analysis where these parameters were changed was preformed and two examples are shown in Table 15 and 16 below. The two different scenarios are summarized in Table 14.

Table 14: Description of 2 scenarios that were tested.

Scenario Nr.	Criteria						Description
	Investment cost	Maintenance cost	Environmental impact	Structural reliability	Design aspects	Implementation time	
1	16.67	16.67	16.67	16.67	16.67	16.67	All criteria of equal importance (i.e. the perfect world scenario)
2	10	10	10	20	25	25	The design aspect, its structural reliability and execution time is the most important criteria (i.e. if the company want to focus more on the structural aspects)

TABLE 15: Scenario 1- All criteria with equal weight distribution.

Criteria	Alternative 1	Alternative 2	Alternative 3
Investment Cost	0.67	0.30	0.33
Maintenance Cost	0.33	0.40	0.50
Environmental impact	0.17	0.30	0.50
Structural reliability	0.33	1.00	0.67
Design Aspect	0.67	0.50	0.17
Execution time	0.67	0.50	0.17
Sum:	2.83	3.00	2.33

When the scenario (nr 1) was compared to the original MCA, the weighted score for Alternative 1 were higher compared to the original MCA, whereas both alternative 2 and 3 resulted less than the original MCA weighted score. The conclusion is that changing the original weight distribution (investment cost 25%, maintenance cost 20%, environmental impact 20 %, structural reliability 15%, design aspect 10% and execution time 10 %) to 16.67 % for all criteria did not change the main result (i.e. Alternative 2 was still determined to be the best alternative).

TABLE 16: Scenario 2- Higher weight distribution for design aspect, structural reliability and execution time criteria.

Criteria	Alternative 1	Alternative 2	Alternative 3
Investment Cost	0.40	0.30	0.20
Maintenance Cost	0.20	0.40	0.30
Environmental impact	0.10	0.30	0.30
Structural reliability	0.40	1.00	0.80
Design Aspect	1.00	0.50	0.25
Execution time	1.00	0.50	0.25
Sum:	3.10	3.00	2.10

When the scenario (nr 2) is compared to the original MCA, the weighted score for alternative were higher comparing to the original MCA, whereas both alternative 2 and 3 resulted less than the original MCA weighted score. The conclusion is that changing the original weight distribution (investment cost 25%, maintenance cost 20%, environmental impact 20 %, structural reliability 15%, design aspect 10% and execution time 10 %) to 10 % for investment cost, maintenance cost, environmental impact, 20 % for structural reliability, and 25 % for both design aspect and execution time criteria changes the main results (i.e Alternative 1 is now the best alternative followed by Alternative 2 and 3).

In both scenarios (nr 1 and 2) of Alternative 3, the overall weighted score did not increase but instead decreased. This can be explained by the rated score it received, where the majority of the criteria scored were relatively low in comparison to both Alternative 1 and 2, hence the weighted score (final result) in both scenarios was not able to match both the Alternative 1 and 2.

To sum up the MCA results, Alternative 2 was found to be the best alternative from the MCA together with scenario 1 and fell only slightly in scenario 2. Hence Alternative 2 can be considered as relatively solid (when analyzed with the chosen criteria that were selected on the basis of this research) and was selected to be used as the stormwater drainage for the new urban development in Waymouth Hills.

5.2. Rainfall runoff and stormdrain calculation

5.2.1. Rainfall analysis

The delineation of the Waymouth Hills catchment was performed in Civil 3D with Google Earth imagery of the location, a total of 26 sub-catchments were obtained and identified (Figure 28).

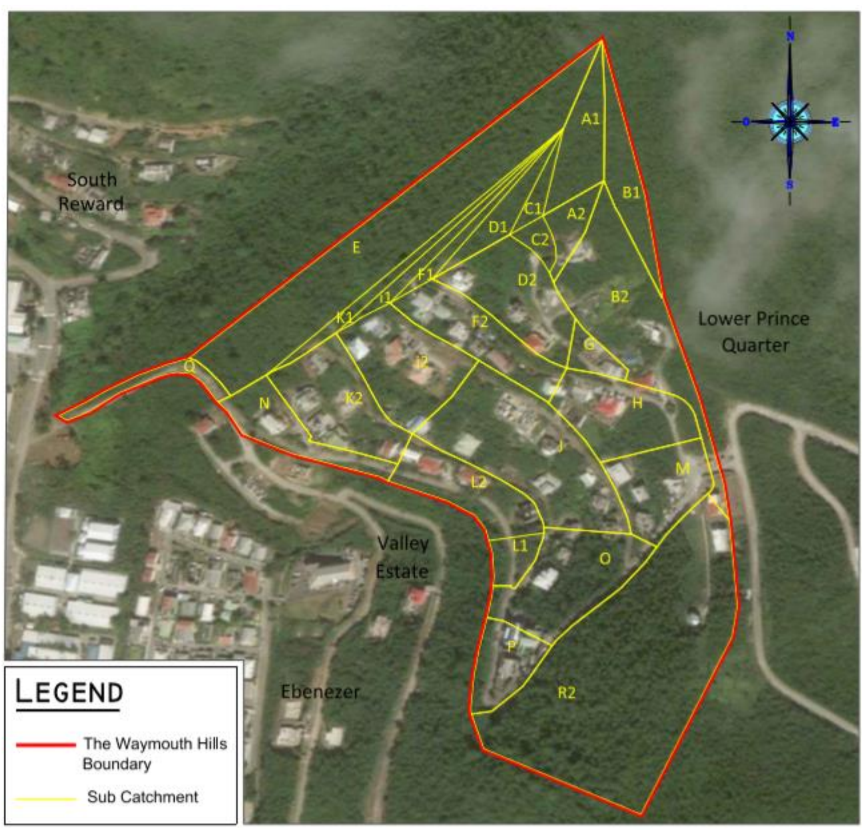


Figure 28: Aerial view of the Waymouth Hills catchment with delineated sub catchments.

A total of 21 streams (also referred to as reaches) in the study area carries the storm water runoff from the sub-catchments and then eventually discharges into the main drainage network of the study. Theses streams were numbered, and the direction of runoff is indicated in Figure 29. Moreover, 4 outlets were identified in the drainage network. The identified outlet's location within the drainage network are as follows: exit point in Reach 3, 10, 15 and 18 and shown Figure 29 below.

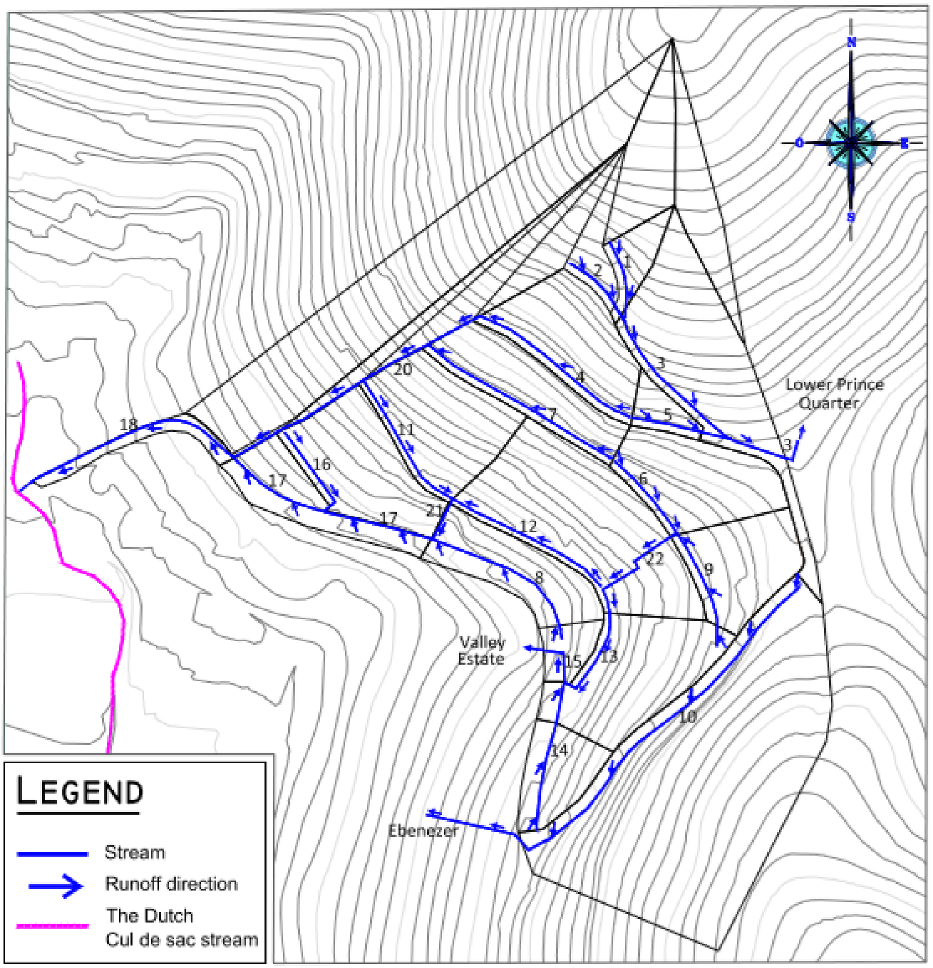


Figure 29: Contour and stream path map of the Waymouth Hills Catchment.

The manual calculation for the rainfall analysis used the Rational Method pertaining to the future urban development scenario. Table 6 was used to define the runoff coefficient for each sub-catchment which was based on the terrain slopes and land use. The assigned value for each sub-catchment is presented in Table 1B (Appendix B). The result of the analysis for the intensity and the peak discharge for the 10 years storm return period occurring in each reach is presented in Table 17. The calculated linear function of the 10-year storm event that was used to calculate the rainfall intensity is presented in Appendix B.

Table 17: Peak discharge versus the time on concentration and rainfall intensity for 10 year- storm event in each Reach.

Concentration Time							Rainfall Runoff	
Reach	Runoff Area A*R	Reach path	Cumm. Runoff Area	Time of entry	Time of flow	Conc. Time	Rain intensity	Peak Discharge Qcum
	[m²]		[m²]	[min]	[min]	[min]	mm/hr	[m3/s]
1	2,561	1-3	2,561	14.91	0.23	15.15	129.8	0.09
2	916	2-3	916	14.59	0.70	15.29	129.4	0.03
3	8,444	3-Lower P. Quarter	11,921	15.32	0.52	15.83	127.7	0.42
4	5,020	4-20	5,020	10.14	1.77	11.92	141.5	0.20
5	842	5-4	842	5.08	1.37	6.44	171.4	0.04
6	4,169	6-20	4,169	15.32	0.35	15.67	128.2	0.15
7	3,565	7-20	3,565	5.14	1.29	6.43	171.6	0.17
8	3,354	8-17	4,298	10.84	0.52	11.36	143.8	0.17
9	3,435	9-20	4,703	17.96	0.51	18.47	120.2	0.16
10	19,466	10-Ebenezer	19,466	15.12	0.80	15.92	127.4	0.69
11	4,621	11-22	4,621	10.84	0.41	11.25	144.3	0.19
12	8,291	12-21	15,895	14.24	0.43	14.67	131.4	0.58
13	5,072	13-15	9,217	18.69	0.35	19.04	118.7	0.30
14	2,180	14-15	7,252	8.45	0.33	8.78	156.4	0.32
15	944	15-Valley Estate	14,857	19.04	0.14	19.18	118.3	0.49
16	4,380	16-17	4,380	11.02	0.35	11.37	143.8	0.17
17	2,576	17-18.	24,166	14.74	0.46	15.20	129.6	0.87
18	957	18-Dutch Cul de Sac	42,397	15.20	0.40	15.60	128.4	1.51
20	-	20-18	17,274	14.79	0.45	15.24	129.5	0.62
21	-	21-17	12,911	14.67	0.08	14.74	131.1	0.47
22	-	22-12	7,604	18.47	0.22	18.69	119.6	0.25

The analysis of the peak discharge shows that the longer the duration of the storm, defined as the time of concentration, resulted in a lower intensity, whereas, the shorter the time of concentration (*Tc*) the higher the intensity. However, having a larger intensity does not directly correlate to a larger discharge into the Reach. The peak discharge generated into the Reaches is also dependent on the runoff from the sum of the contributing sub-catchment areas. As a result of this, it can be observed that the larger the contributing areas were, the larger the discharge into the Reach was. Taking Reach 5 and 18 as an example, the peak discharge 0.7m^3/s runoffs to Reach 5 from a total contributing area of 842m^2 with an intensity of 171mm/hr. Whereas, the peak discharge runoff into Reach 18 is 1.52m^3/s resulting from an intensity of 128.4mm/hr to a total contributing area of 42,397m^2. Due to the overall ratio of area:intensity, it can be observed that Reach 18 has a higher peak discharge.

5.2.2. Hydraulic analysis

The geometry of the road used for the street conveyance capacity analysis is the proposed design for the road infrastructure upgrade in the Waymouth Hills. Whereas the longitudinal slope is obtained from the existing terrain map generated from Civil 3D. The longitudinal slope used for the manual hydraulic analysis, is calculated by taken the elevation difference between the two points (inlet and outlet) of each Reach by dividing the total length between the two points. The result of the conveyance capacity of the road within the defined Reach to drain the 10 year-storm peak discharge runoff calculation is presented in Table 18.

Table 18: Street conveyance capacity and peak runoff for10 year-storm event.

Reach	Qcum (m3/s)	Road width (m)	Transverse Slope	Longitudinal Slope	Qmax (m3/s)	Velocity (m/s)	Status	Runoff direction
1	0.09	4.5	2.0%	27%	0.24	4.21	SUFFICIENT	Lower P. Quarter
2	0.03	4.5	2.0%	3%	0.08	1.40	SUFFICIENT	Lower P. Quarter
3	0.42	4.5	2.0%	16%	0.19	3.24	NOT SUFFICIENT	Lower P. Quarter
4	0.20	4.5	2.0%	1%	0.05	0.81	NOT SUFFICIENT	Reach 20
5	0.04	4.5	2.0%	1%	0.05	0.81	SUFFICIENT	Lower P. Quarter
6	0.15	3.0	2.0%	9%	0.06	2.59	NOT SUFFICIENT	Reach 22
7	0.17	4.5	2.0%	2%	0.07	1.15	NOT SUFFICIENT	Reach 20
8	0.17	7.5	2.0%	12%	0.47	3.07	SUFFICIENT	Reach 18
9	0.16	4.5	2.5%	11%	0.22	2.99	SUFFICIENT	Reach 22
10	0.69	7.5	2.0%	12%	0.47	3.07	NOT SUFFICIENT	Ebenezer
11	0.19	3.5	2.0%	14%	0.10	2.89	NOT SUFFICIENT	Reach 21
12	0.58	4.0	2.0%	9%	0.11	2.36	NOT SUFFICIENT	Reach 21
13	0.30	4.0	2.0%	9%	0.11	2.36	NOT SUFFICIENT	Reach 15
14	0.32	7.5	2.0%	16%	0.54	3.55	SUFFICIENT	Reach 18
15	0.49	7.5	2.0%	12%	0.47	3.07	NOT SUFFICIENT	Valley Estate
16	0.17	4.0	2.5%	14%	0.19	3.24	SUFFICIENT	Reach 18
17	0.87	7.5	2.0%	10%	0.43	2.80	NOT SUFFICIENT	Reach 18
18	1.51	7.5	2.0%	12%	0.47	3.07	NOT SUFFICIENT	Dutch Cul de Sac

The result of the street conveyance capacity illustrated in Table 18 is based on the maximum filling allowance of 75 percent. The geometry and the profile of the road varies in each Reach, alternately resulting different conveyance capacities. It can be observed that the street geometry (such as the road width, the transverse slope) and the road profile (longitudinal slope) influence the conveyance capacity to drain storm water runoff. The influence of these parameters contributing to the conveyance capacity differences can be analysed by comparing the street characteristic in Reach 1, 2, 6, 12, and 16 given that the value of the Manning's roughness coefficient and the side and gutter size is the same in all cases. As shown in Table 18, given that the width and the transverse slope of the street in Reach 1 and 2 are constant, with the increase of steepness in the road profile the conveyance capacity of the street also increases with the flow velocity. Moreover, comparing the road geometry between Reach 6 and 12, the steepness remained the same. It is noticed that the conveyance capacity of the street also increases with a wider road section, however, the flow velocity decreases. Finally, with an increasing transverse slope, both the conveyance capacity of the street and the flow velocity increases. This can be observed in Table 19 taking Reach 16 as an example while both the road profile and the road width are constant for the comparison.

Table 19: Street conveyance capacity of Reach 16 with different transverse slope.

Reach 16						
Qcum (m3/s)	Road width (m)	Transverse Slope	Longitudinal Slope	Qmax (m3/s)	Velocity (m/s)	Status
0.17	4.0	2.0	14%	0.14	2.94	NOT SUFFICIENT
0.17	4.0	2.5	14%	0.19	3.24	SUFFICIENT

For the Reaches that the street conveyance capacity were determined to be not sufficient (to convey the peak runoff from the 10 year-storm event), U-gutters are used. Moreover, U-gutters are mandatory on primary roads, hence the design of U-gutter was introduced for Reach 8 and 14. The result of the U-gutter dimensions and its hydraulics used for stormwater discharge is presented in Table 20.

Table 20: U-gutter drainage capacity and peak runoff for 10 years storm event.

Reach	Name of Storm drain	Sizes HxB (m)	Qcum (m3/s)	Longitudinal Slope	Qmax (m3/s)	75% filling flow velocity (m/s)	Actual flow velocity (m/s)	Actual Flow depth (m)	Status
3	U-Gutter 3	0.30 x 0.40	0.42	16%	0.50	5.59	5.45	0.21	SUFFICIENT
4	U-Gutter 4	0.45 x0.40	0.20	1%	0.24	1.79	1.72	0.29	SUFFICIENT
6	U-Gutter 6	0.30 x 0.30	0.17	9%	0.29	4.30	3.67	0.13	SUFFICIENT
7	U-Gutter 7	0.50 x 0.40	0.17	2%	0.26	2.28	2.12	0.27	SUFFICIENT
8	U-Gutter 8	0.30 x 0.30	0.17	12%	0.34	4.23	3.07	0.13	SUFFICIENT
10	U-Gutter 10	0.75 x 0.30	0.69	12%	1.01	5.96	5.66	0.41	SUFFICIENT
11	U-Gutter 11	0.25 x 0.25	0.19	14%	0.22	4.75	4.60	0.17	SUFFICIENT
12	U-Gutter 12	0.50 x 0.50	0.58	9%	1.13	6.05	5.17	0.23	SUFFICIENT
13	U-Gutter 13	0.30 x 0.35	0.30	9%	0.36	4.56	4.39	0.20	SUFFICIENT
14	U-Gutter 14	0.35 x 0.35	0.32	16%	0.58	6.35	3.55	0.16	SUFFICIENT
15	U-Gutter 15	0.35 x 0.35	0.49	12%	0.51	5.50	5.46	0.26	SUFFICIENT
17	U-Gutter 17	0.60 x 0.60	0.87	10%	1.94	7.20	5.93	0.24	SUFFICIENT
18	U-Gutter 18	0.95 x 0.60	1.51	12%	3.75	8.77	7.30	0.34	SUFFICIENT
20	U-Gutter 20	0.30 x 1.10	0.62	49%	3.64	14.71	7.91	0.071	SUFFICIENT
21	U-Gutter 21	0.30 x 0.70	0.47	44%	1.99	12.62	7.93	0.085	SUFFICIENT
22	U-Gutter 22	0.30 x 1.00	0.25	44%	3.09	13.72	5.63	0.041	SUFFICIENT

The U-gutters were designed in such a way that the discharge capacity (Q-max) of each gutter would result in a 75 percent filling within the structure at a peak storm discharge (Q-cum) flowing into it. The results of the Qmax from the 75 percent filling from the U-gutters is larger than the Qcum needed whereas flow velocity generated from this discharge capacity exceeded the allowable of 6m/s (defined in the program of requirements).

Given that Qmax in the U-gutters is based on a 75 percent filling capacity rather than the actual peak discharge from the runoff, the travel velocity from this flow affects the actual time of concentration of the storm event (hence, this influences the overall rainfall intensity and peak discharge runoff). As a result of this, the actual travel velocity in each U-gutters was calculated by determining the depth of the flow in the U-gutter from the Qcum instead. This calculation process for the flow depth, velocity and the peak discharge runoff was iterated to define a more precise peak discharge runoff and the hydraulics occurring in the U-gutters.

The result defined in Table 20, illustrates that the actual flow velocity in Reach 18, 20 and 21 exceeded the allowable flow velocity, regardless of the calculation of the time of concentration in these Reaches. To satisfy the flow velocity in these channels, alteration of the influencing parameters (such as channel slope, size and surface roughness) have been undertaken in the hydrodynamic modelling. The result of the hydraulic analysis for each Reach is presented in Table 3D (Appendix D).

5.2.3. Hydrodynamic analysis

Model 1: Future urban development scenario using Rational Method for rainfall analysis

To validate the hydrodynamic analysis of manual calculations, the Rational method was used for the rainfall analysis, and the results for proposed hydraulic structure for the drainage network carried out in the manual calculation were incorporated in this SSA model. In additional, chainage was incorporated to the Reaches, this gives the detail representation of the actual characteristics (i.e. longitudinal slopes) of the drainage network to define the hydraulics. The properties of the Reaches' chainage used in the model is included in Table 1D

(Appendix D) and the SSA model of the drainage network is shown in Figure 30 below. The profile plots for all the model Reaches with the maximum discharge and the depth are presented in Appendix D.

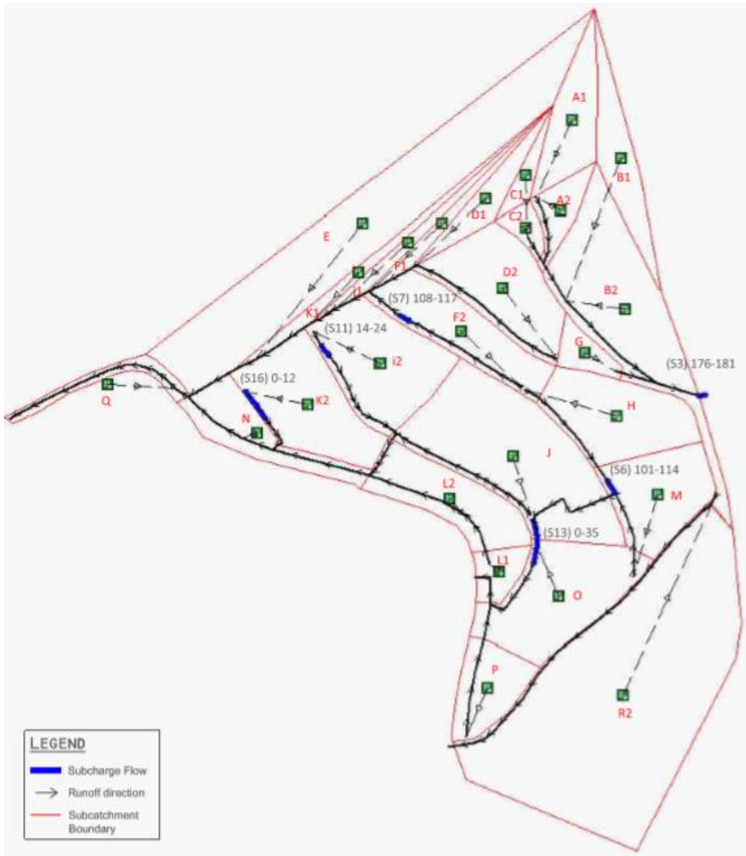


Figure 30: Drainage network of the Waymouth Hills.

The results from the hydrodynamic analysis of the model illustrated in Figure 30, surcharge flows (indicated in blue link) occurred in the drainage network during the simulation of the for 10-year storm event. In order to discuss the cause of the surcharge flow occurring in these Reaches, the critical duration of the rainfall event must have defined; since it is the duration necessary to produce the maximum peak flow occurring in the Reaches.

The critical duration of the storm event in the drainage system varied, since it is dependent on the rainfall profile and the system characteristic. To illustrated this, the hydrograph of the outlets (or exit point of Reaches 3, 10, 15 and 18) in drainage network was used and was generated from the model's simulation (Figure 31).

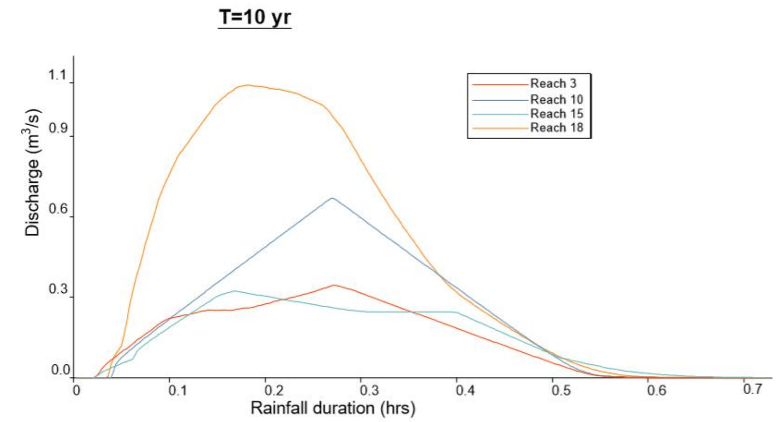


Figure 31: Hydrograph displaying the peak discharge versus rainfall duration of the system's outlets from model 1 simulation.

The result from the hydrograph (Figure 31) showed the maximum peak flow is produced at 0.17-hour (10.2 mins) for at Reach 18 and 15, and the critical duration for Reach 3 and 10 was found to be at 0.27-hour (16.2 mins). The flow velocity occurred in the drainage network at the both critical duration of the rainfall is presented in Figure 32.

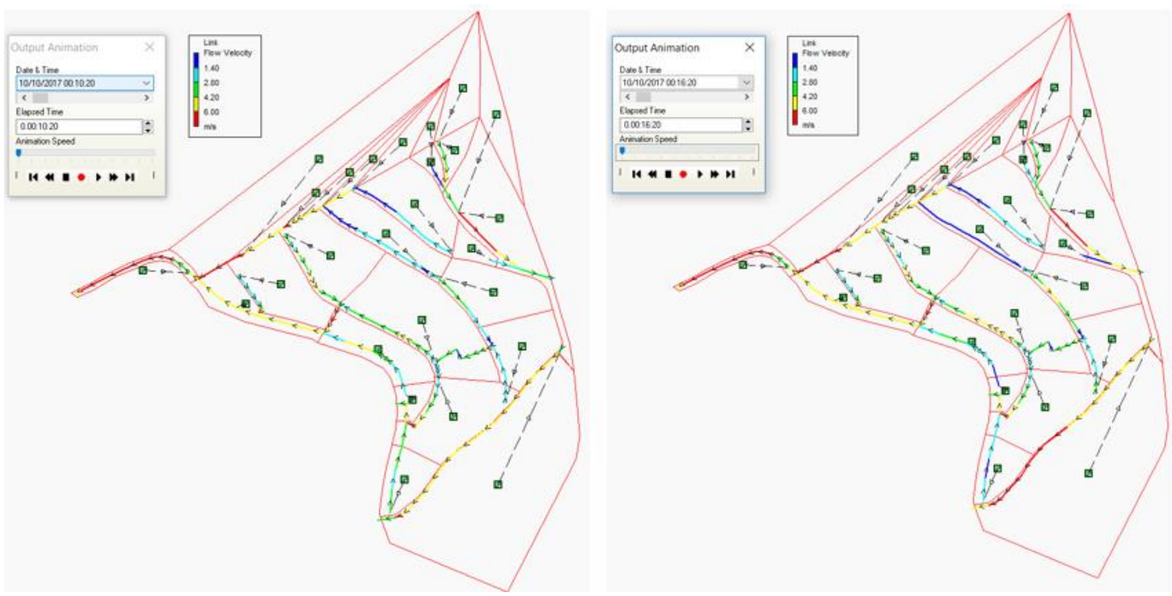
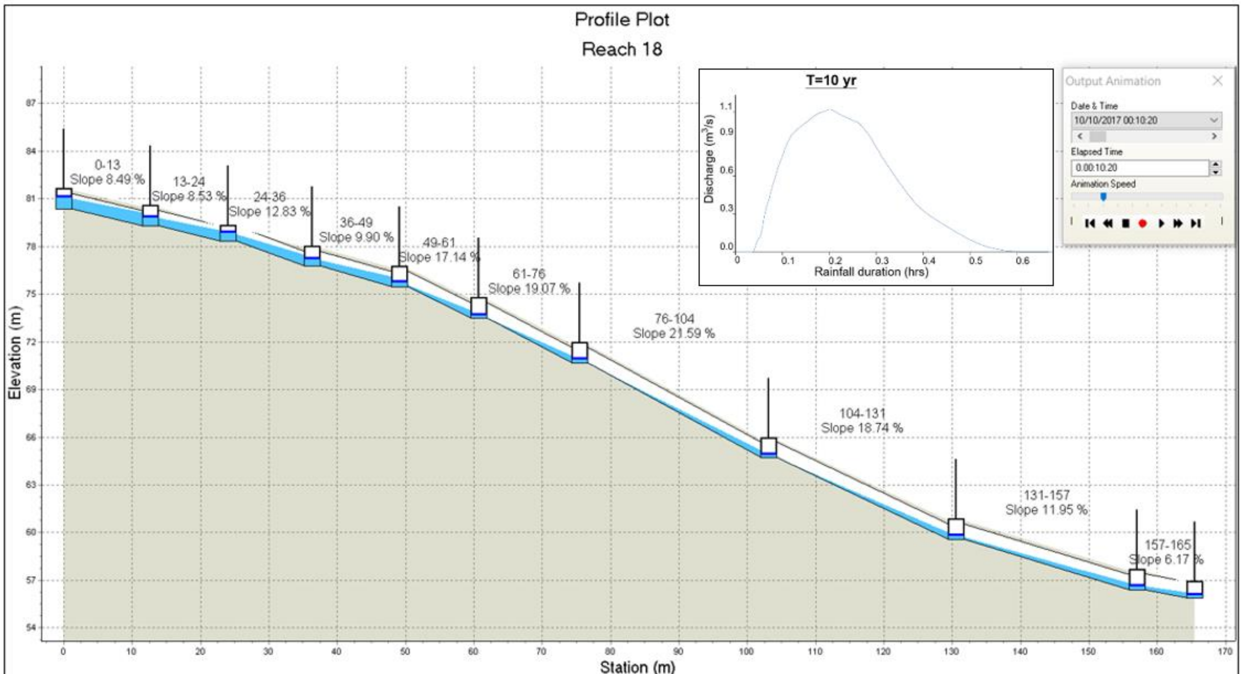


Figure 32: Overview of the modelled channels and their related flow velocity at rainfall duration of 10.2 mins (left) and 16.2 mins (right) for 10- year storm event from model 1 simulation.

The result of the hydraulics overview in the drainage network are presented in Figure 30 and 32. In the drainage network during the storm event's peak runoff not only did a surcharge flow (indicated in blue links referring to Figure 30) occur but also undesirable travel velocities as well (indicated by the red links in Figure 32). To discuss the alternatives that has been undertaken for both of these hydraulic problems, the profile plot of Reach 18 is taken to analyse the velocity profile changes with the surface roughness (using larger value for manning's coefficient) and or channel enlargement, whereas, profile plot of Reach 7 is taken to analyse the flow profile changes with channel enlargement and or increase channel slope.

The result of the initial simulation in Reach 18 (Figure 33 below) illustrated that the travel velocity increases with larger longitudinal slope of the channel. In addition, it was observed that the flow depth in these channels were lower but with higher travel velocity. However, when the Reach was simulated using the same channel properties but using a larger channel width, both the flow depth and flow velocity decreases in the channels (Table 21). Despite the travel velocity decreasing with a larger channel width, the reduction the flow velocity by the enlargement of the structure enlargement. Since, it required a significant amount of space. Moreover, greater flow velocity reduction in the channel flow can be observed with rougher channel surface (by using a larger Manning's coefficient value) comparing to solely structure enlargement (Figure 34).

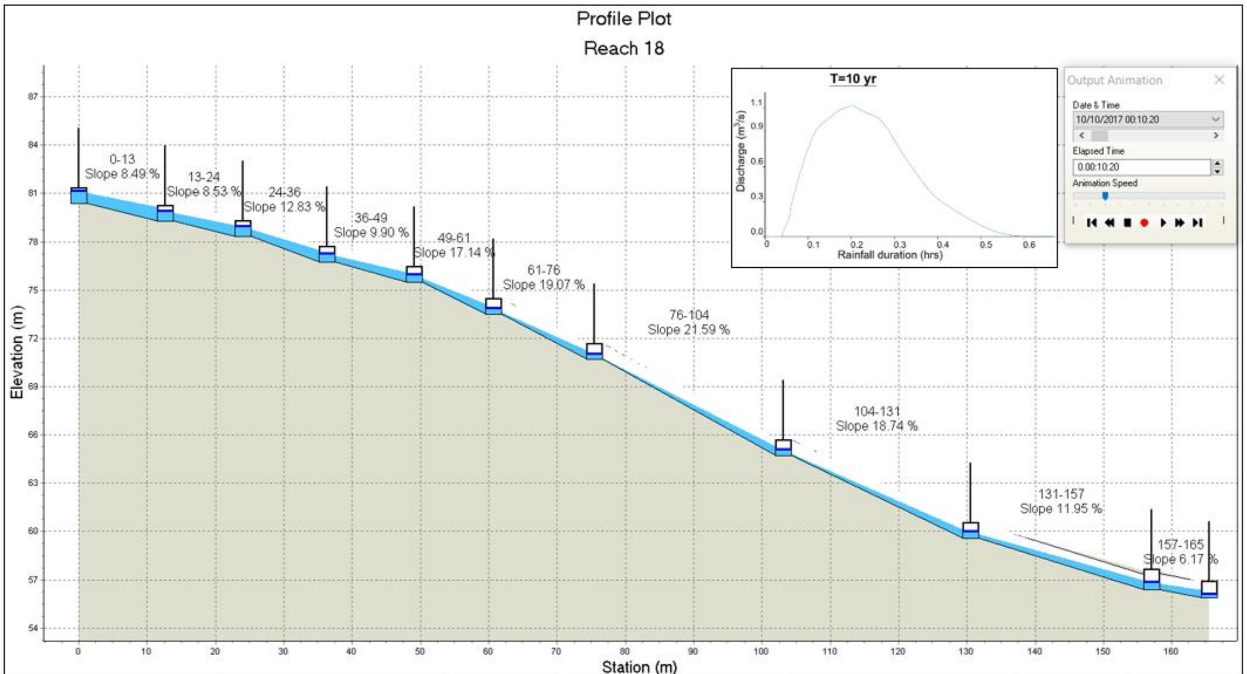


Chainage (-):	0-13	13-24	24-36	36-49	49-61	61-76	76-104	104-131	131-157
Max Vel (m/s):	5.92	5.93	6.91	6.27	7.68	7.98	8.35	7.93	6.73
Max Depth (m):	0.31	0.31	0.29	0.29	0.24	0.23	0.22	0.23	0.27

Figure 33: Profile plot of Reach 18 from model 1 simulation with proposed channels from manual calculation.

Table 21: Hydraulic results of Reach 18 from model 1 simulation with channel's width enlargement (from initial 0.6m to 1.2m).

Chainage (-):	0-13	13-24	24-36	36-49	49-61	61-76	76-104	104-131	131-157
Length (m):	12.67	11.31	12.34	12.80	11.63	14.71	27.72	27.33	26.54
Size (HxB):	0.95x1.20	0.95x1.20	0.95x1.20	0.95x1.20	0.95x1.20	0.95x1.20	0.95x1.20	0.95x1.20	0.95x1.20
Slope (%):	8.49	8.53	12.83	9.90	17.14	19.07	21.59	18.74	11.95
Up Invert (m):	80.37	79.30	78.33	76.75	75.48	73.49	70.68	64.70	59.58
Dn Invert (m):	79.30	78.33	76.75	75.48	73.49	70.68	64.70	59.58	56.40
Surface roughness (-):	0.014	0.014	0.014	0.014	0.014	0.014	0.014	0.014	0.014
Max Vel (m/s):	5.39	5.40	6.17	5.67	6.78	7.01	7.30	6.97	6.03
Max Depth (m):	0.17	0.17	0.15	0.16	0.13	0.13	0.12	0.13	0.15



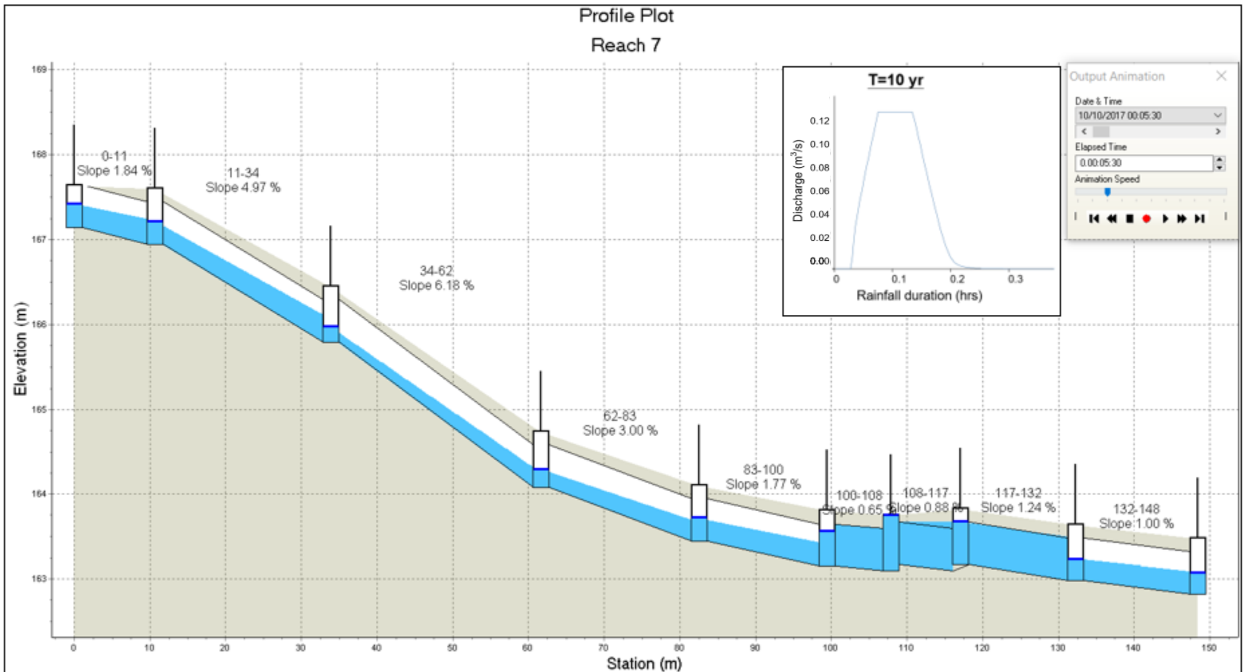
Chainage (-):	0-13	13-24	24-36	36-49	49-61	61-76	76-104	104-131	131-157
Max Vel (m/s):	3.91	3.91	4.59	4.15	5.14	5.35	5.61	5.31	4.47
Max Depth (m):	0.47	0.46	0.40	0.44	0.35	0.34	0.32	0.34	0.41

Figure 34: Profile plot of Reach 18 from model 1 simulation with increased channel's surface roughness (increase Manning's coefficient from 0.014 to 0.024).

The result of the initial simulation in Reach 7 (Figure 35) illustrated that a surcharge flow occurred only in flatter terrain of Reach 7 (chainage 108-117m). Due to the low channel longitudinal slope in chainage 108-117m, the discharge capacity in this channel was lower comparing to the rest of the channels within the same Reach. Under those circumstances, surcharge flow occurred in chainage 108-117m because the peak inflow was greater than the channel's discharge capacity. Moreover, the surcharge flow induced back water in the upper chainage, this was observed by the high flow depth in chainage 100-108m.

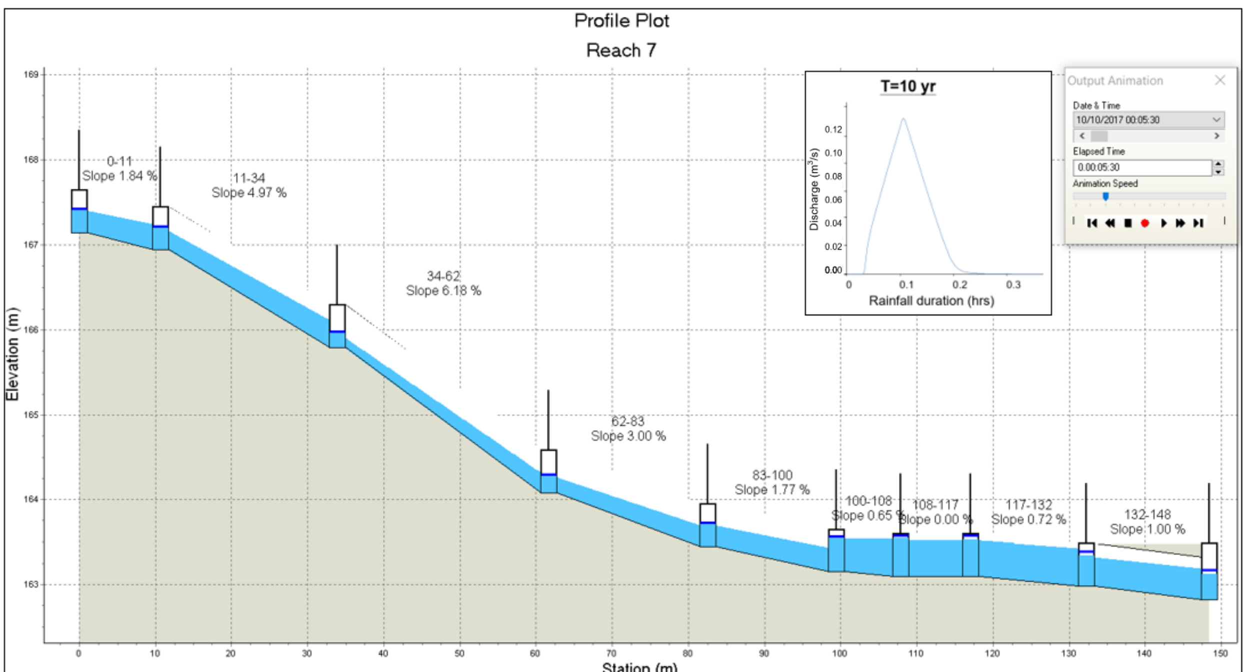
Knowing that surcharge flow occurred in chainage 108-117m due to its low discharge capacity, a larger channel was incorporated to chainage 108-117m to analyse the effectiveness of increasing the flow capacity. It was noticed with slight channel width enlargement (from 0.3m to 0.4m) in chainage 108-117m, the surcharge flow earlier occurred in the channel was mitigated (Figure 36 below), however, high flow depth still retained in the channel. Moreover, with increasing enlargement of the channel, the flow depth can decrease more.

In addition, having enlargement in the channels upstream, there was no increase in efficiency of reducing the flow depth in the channels downstream. Thus, it is rather more efficient to solely have enlargement in the channels where an increase flow capacity is needed to alleviate surcharge flow.



Chainage (-):	0-11	11-34	34-62	62-83	83-100	100-108	108-117	117-132	132-148
Max Vel (m/s):	2.05	3.04	3.30	2.50	2.02	1.34	0.77	1.31	1.49
Max Depth (m):	0.28	0.18	0.17	0.22	0.28	0.42	0.50	0.29	0.26

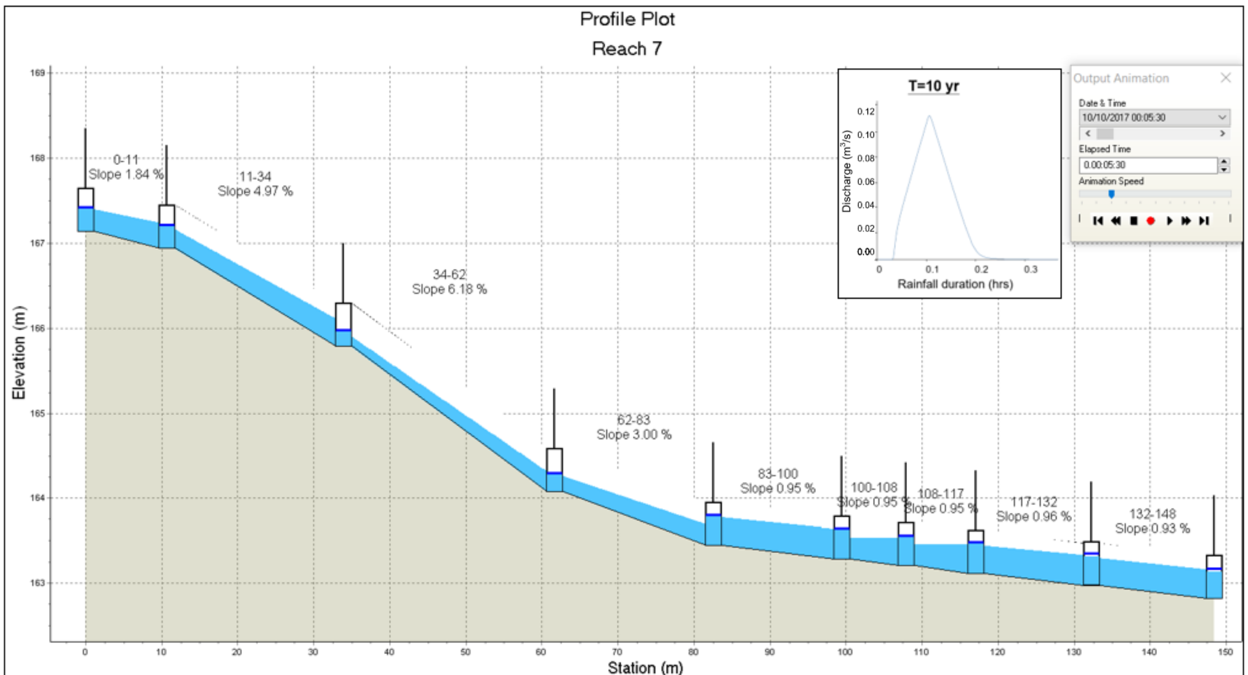
Figure 35: Profile plot of Reach 7 from model 1 simulation with proposed channels from manual calculation.



Chainage (-):	0-11	11-34	34-62	62-83	83-100	100-108	108-117	117-132	132-148
Max Vel (m/s):	2.05	3.04	3.30	2.50	2.02	1.34	0.87	1.39	1.59
Max Depth (m):	0.28	0.18	0.17	0.22	0.28	0.42	0.48	0.40	0.35

Figure 36: Profile plot of Reach 7 from model 1 simulation incorporated with channel width enlargement at chainage 108-117m.

To illustrate the effect of channel slope on the discharge capacity in the channels, the model was simulated using the initial proposed channel size but with an increase channel slope in chainage 107-118m by altering the inlet and outlet's invert elevations. By doing so, the channels' slope further downstream can reflect a decrease depending on the transition of channel the invert elevation modification. The changes made on the channel downstream (chainage 108- 148m) in Reach 7 and the result of the simulation is presented in Figure 37 below. The result showed an increase in discharge capacity in chainage 108-117m, and the surcharge flow was alleviated. In addition, the flow capacity of the downstream channels was more efficient in discharging the storm water runoff comparing with having structure enlargement. This can be observed by the flow depth in the channels (refer to Figure 36 and Figure 37 for the comparison).



Chainage (-):	0-11	11-34	34-62	62-83	83-100	100-108	108-117	117-132	132-148
Max Vel (m/s):	2.05	3.04	3.30	2.49	1.57	1.56	1.56	1.57	1.55
Max Depth (m):	0.28	0.18	0.17	0.22	0.36	0.35	0.36	0.35	0.36

Figure 37: Profile plot of Reach 7 from model 1 simulation with channel slope improvement.

Since alteration of the channels' characteristics was undertaken to rectify the hydraulic problems such as the occurrence of surcharge flow and high flow velocity, this changes the hydraulics of the network system as a whole. The improvement of the channel network reflected time delay of the critical duration during the storm event; due to the reduced of flow velocity in the channels. Since widespread of the channels' longitudinal profile are steep, only a slight time delay was noticed within the network system. Furthermore, an increase of peak flow was observed releasing out into the network system, this occurred because the occurred surcharge flow is alleviated. In the event when surcharge flow occurred in the network's channels, storm water runoff was stored temporarily in the channels during the storm event's peak runoff and this water was not released out of the network during the critical duration, but it's rather released at a slower rate and a longer period during the storm event. For an illustration of the defined hydraulic changes in the network system, refer to Figure 38.

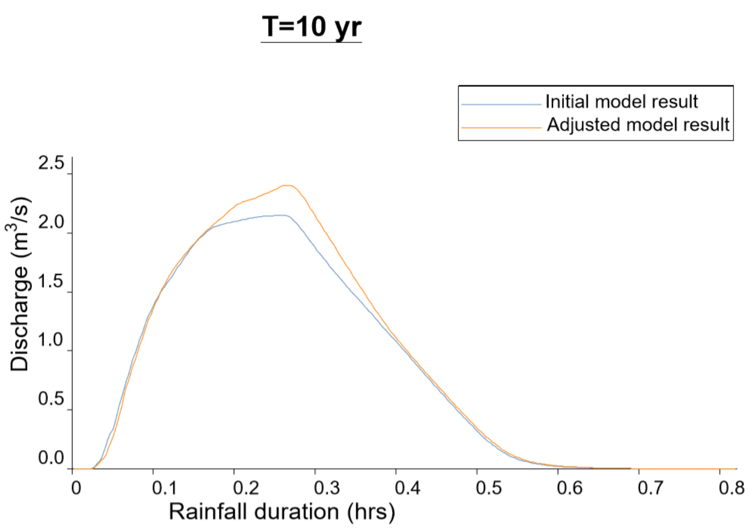


Figure 38: Hydrograph of the system's network for the initial model 1 simulation verses the adjusted model 1 simulations (because of flooding and undesirable velocity) for the 10-year

<u>**Comparison of Model 1 results with manual rainfall runoff and storm drain calculation results**</u>

Using the results presented earlier in this chapter, the comparison of both the rainfall and hydraulic analysis from the manual and hydrodynamic model of the drainage network can be compared between each other. To do so, reach 7 and reach 18 is taken for the comparison, and using their related hydrograph simulated from model 1 for the comparison (Figure 39 below)

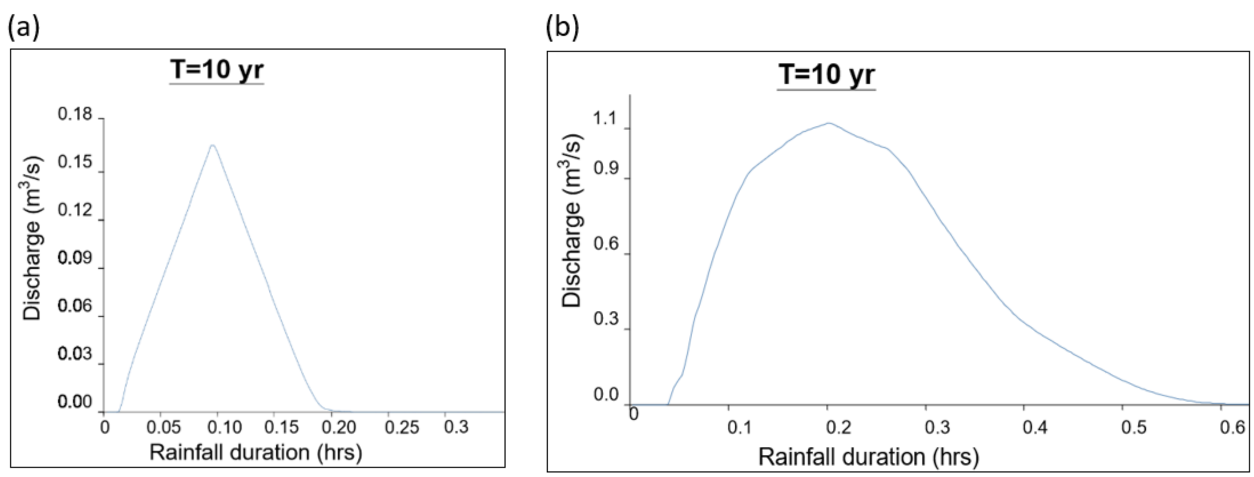


Figure 39: (a) Hydrograph at Reach 7 and (b) hydrograph at Reach 18 from the adjusted model displaying the peak runoff versus the rainfall duration for 10-year storm event.

The critical duration of the 10-year storm event in the hydrodynamic modeling for Reach 18 was 0.27 hours (16.2mins) and the peak runoff occur during this duration was 1.12m^3/s (refer to Figure 39 above). Whereas, in the manual calculation the critical duration (can also refer to as the time of concentration *Tc*,) was 15.6 minutes and the peak runoff at this duration was 1.52m^3/s (refer to Table 17). The reason behind a larger peak runoff resulted in the manual calculation for Reach 18 can be explained by the shorter critical duration comparing to the hydrodynamic modelling result.

As mentioned earlier in chapter 4.2.1 and 5.2.1, the shorter the duration for a given storm event, gives a greater average rainfall intensity, whereas, the longer the length of the storm, the lower the storm average rainfall intensity is found to be. A shorter critical duration obtained from the manual calculations resulted in a larger rainfall intensity which in turn increases the peak runoff during the storm event. This can be seen in Table 22, by using the mentioned critical duration obtained from the hydrodynamic modelling and using the carried-out method for the manual calculation.

Table 22: Comparison of critical duration (or *Tc*) influencing the rainfall intensity and peak runoff between the manual calculation and the hydrodynamic modeling for Reach 18 using the manual calculation method.

Rainfall Runoff				
	Cum. runoff area (m^2)	Tc (min)	Rainfall intensity (mm/hr)	Peak runoff (m^3/s)
Initial manual calculation result	42,397	15.60	128.4	1.51
Result of the manual calculation using *Tc* from Model 1	42,397	16.2	126.5	1.49

It can be observed that the peak runoff decreased when the *Tc* from the hydrodynamic model was used. Despite the critical duration of the storm event had an influence on both the rainfall intensity and the peak runoff. The result of the peak runoff between the manual calculation and the hydrodynamic modelling for Reach 18 was still different. The reason for this can be explained by the runoff characteristic from the contributing sub-catchment area that is discharged into the Reach.

In the manual calculation, the peak runoff was resulted from overland flow from all contributing area discharging into the Reach from the most distant point (define by the *Tc*) at the same time. Whereas, in the hydrodynamic modelling the overland flow from the contributing area discharges in the Reach accordingly to their travel distance to the Reach during the rainfall event. Meaning, overland flow from contributing area that are closer to the Reach will start discharging into the Reach when the soil is saturated. Furthermore, the overland flow from the contributing area that are further will takes a longer time and will enter the Reach at a longer time. Therefore, the peak runoff in the hydrodynamic modeling resulted less comparing to the manual calculation because the runoff from the contributing area may not necessarily enter the Reach at the same time but when they do some of the stormwater has already left the Reach.

The illustrated mentioned point, reach 7 is used to demonstrate the effect of peak runoff between the manual calculation and hydrodynamic modelling without overland runoff from multiple sub-catchment contributing to the Reach. This Reach solely convey overland runoff from sub-catchment F2 and the results from both the manual calculation and the hydrodynamic modelling for the peak runoff in this Reach was 0.17m3/s (refer to Figure 39a for the hydrodynamic result and Table 17 for the manual calculation result). The contributing overland flow contributing to Reach 18 and Reach 7 is shown in Figure 40 below.

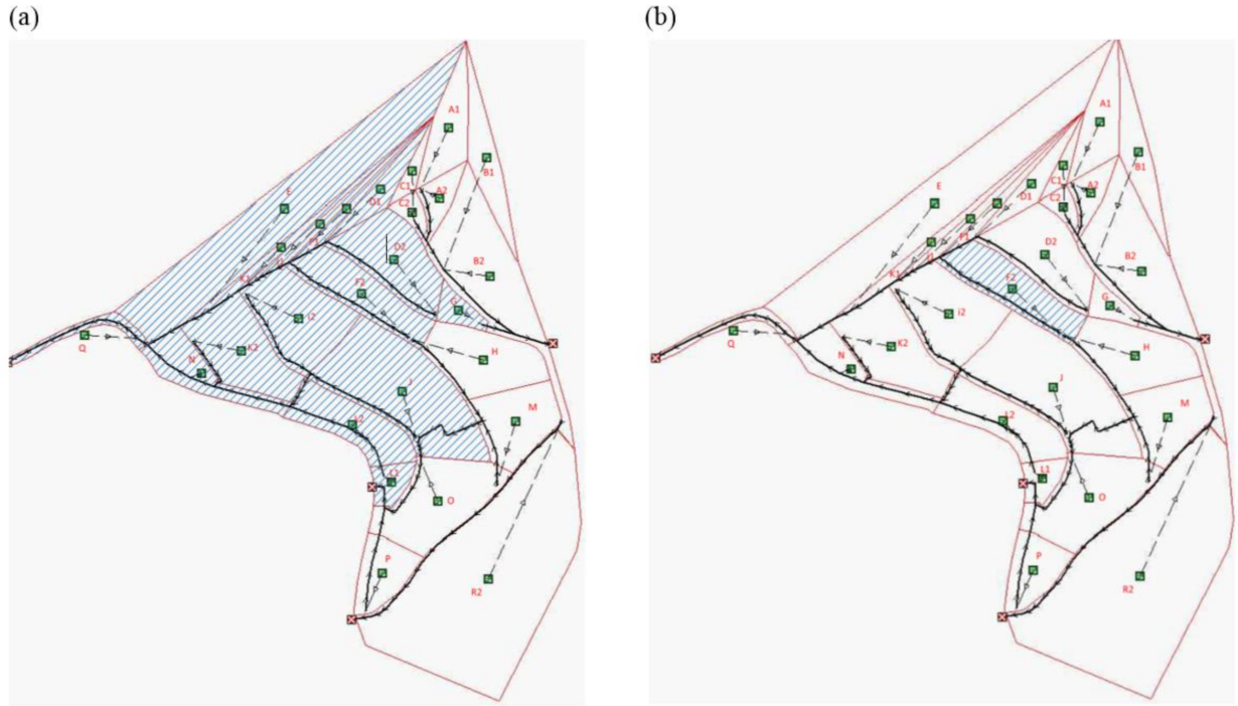


Figure 40: Contributing overland runoff entering in Reach 18 (a) and Reach 7 (b) used for the manual calculation and hydrodynamic modelling.

As mentioned earlier in chapter 4.2.2 , the longitudinal slope for a reach was determined the elevation difference between only 2 points i.e., the inlet and outlet of the reach. Whereas in the hydrodynamic modelling the reaches used chainages to represent the changes in longitudinal slope accordingly to the terrain. Hence, to compare the flow depth in the drainage structure pertaining to the Reach, a particular chainage within the reach from the hydrodynamic modelling must have the same or similar longitudinal slope used in the manual calculation.

From the manual calculation the flow depth in the drainage structure of Reach 7 was observed to be 0.27meter (refer to Table 17), whereas in the hydrodynamic modelling the flow depth in chainage 0-8m of Reach 7 was 0.28meter (refer to Figure 35). The reason for the flow depth in the drainage structure of the hydrodynamic model was higher is because of small difference in the longitudinal slope used comparing to the manual calculation. In the manual calculation the longitudinal slope used for Reach 7 was 2% whereas in the hydrodynamic modelling the longitudinal slope for Reach 7 in chainage 0-8m was 1.86%. As explained earlier, with a larger longitudinal slope in a drainage structure a higher discharge capacity is observed (due to potential increase in travel velocity) comparing to a drainage structure with a smaller longitudinal slope. The influence of the drainage structure's discharge capacity on the flow depth can be explained by the continuity equation for fluid mechanics expressed in terms of discharge (or flow rate) E.q. (13).

$$Q = A * V \tag{13}$$

Where:
Q= discharge capacity (m^3/s); A= wetted area (m^2); V; flow velocity (m/s)

To explain how E.q. (13) influences the flow depth in the channel, the flow in the structure is assumed to be steady uniform (flow at a given section remains close to constant).

E.q. (13) defines the discharge of the drainage structure which is the product of its velocity by the wetted area (depth of the water by the width of the structure) and by assuming the flow is steady uniform in the structure, the discharge remained constant even when the structures cross sections, longitudinal changes and surface roughness changes. It only reflects different flow velocity and wetted area in the drainage structure.

The Manning equation defined in E.q.(8) was used in this research for the structure's hydraulic capacity calculation as the Manning's equation considered the mentioned paraments (structures cross sections, longitudinal changes and surface roughness). Since both the manual calculation and the hydrodynamic modeling for the peak runoff result was 0.17m^3/s and used the same drainage structure section and surface roughness (defined by Manning's coefficient) but different longitudinal slope. Because the longitudinal slope used in the manual calculation was greater than in the hydrodynamic modelling for Reach 7, this reflected a larger flow velocity and a lower wetted area (flow depth because the structure width is constant) in the structure.

For the comparison of the manual calculation with the hydrodynamic modelling result in the structure's flow depth regarding in Reach 18, chainage 131-157m in Reach 18 was taken from the hydrodynamic modelling for the comparison. The structure's flow depth in Reach 18 from the manual calculation was 0.34m (refer to Table 20) and the flow depth in Reach 18 chainage 131-157m from the hydrodynamic modelling was 0.27m (refer to Figure 33). The reason the structure's flow depth obtained from the manual calculation was higher than the result obtained from the hydrodynamic modeling, due to the peak runoff difference. With larger peak runoff, both the velocity and wetted area increase when the structure's characteristics (i.e. Cross-section, longitudinal slope and surface roughness) remained the same. This was the case, because chainage 131-157m in Reach 18 used for the comparison had the same structure's characteristic used in the manual calculation. However, the result of the flow velocity in both the manual calculation and the initial model from the hydrodynamic in Reach 18 had exceeded the permitted value (of 6m/s) defined in the program of requirement. Because of this, alternative such as increasing the surface roughness had been undertaken to reduce the flow velocity in the Reach. With flow velocity decreased in the structure the flow depth increased and this can be seen in the 'adjusted model' for Reach 18 (refer Figure 34) having a higher flow depth in comparison to both the initial model and the manual calculation.

Model 2: Current development scenario using SCS Method for the dynamic rainfall analysis

As mentioned in the previous chapter, presently stormwater runoff is solely conveyed by the roads in the current situation for Waymouth Hills. Hence, to simulate the hydrodynamic model of the existing system, transversal and longitudinal profile of the roads were generated from field survey data. However, roads where field surveyed data was not available, such as Mount Pele Hill Road and Mount Souffiere Road, a contour map was used to generate those data.

Moreover, in the present situation Reach 20,21 and 22 are overland flow. Thus, theses Reaches were included in the model since runoff from contributing Reaches are converged to and travel in these directions. These Reaches (20, 21, and 22) was modelled as natural channel using a Manning coefficient of 0.14 and the width of channel was 2 meters by a 0.2 meters channel depth. The result of the simulation for the existing system are presented in Figure 41 and 42. The sectional and longitudinal properties of the road section used for the simulation are given in Appendix D.

The simulation of this model was carried by using the CN for each catchment by determining the degree of built area. The CN values incorporated in this model for the runoff simulation are presented in Table 4C (Appendix C).

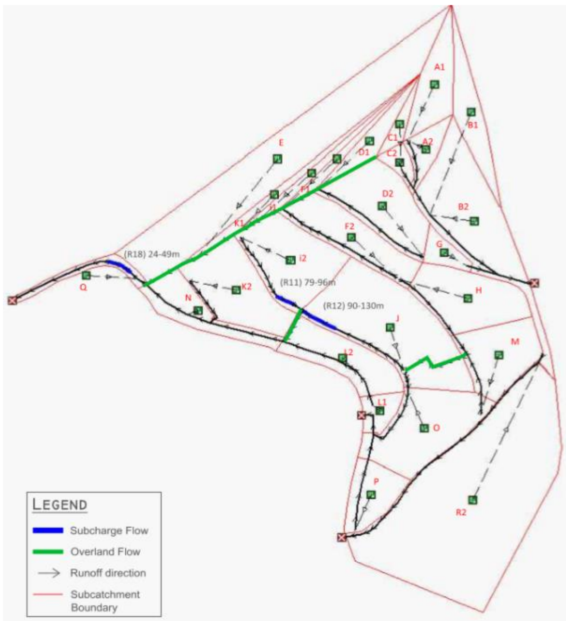


Figure 41: Overview of the modelled existing system's channels and their related capacity (the green link indicates the location of the Reaches modelled as natural streams) from model 2 simulation.

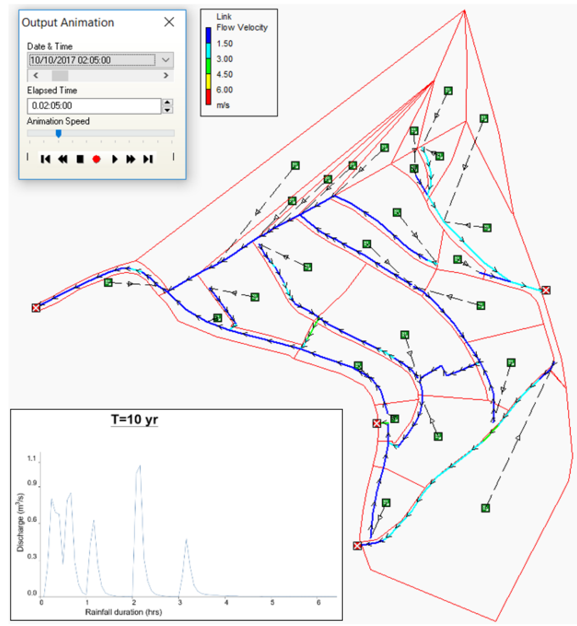


Figure 42: Overview of the modelled existing system's channels and their related flow velocity at critical duration of 2:05 hr (shown in the hydrograph; bottom left) during 10- year storm event from model 2 simulation.

The occurrence of the surcharge flow in the system indicated in Figure 41 can be explained by the transversal section road profile presented in Figure 43 below. Road that consist of small undermined section, storm water is converged in, these sections are inadequate to convey the storm water and overfilling to the top of the road surface. Overtime, with consistent storm water overflowing on the road, the undermined section becomes wider and deeper. However, road sections that has transversal slope outwards, storm water runs off the road and flows out to the hill side.

Since, the existing roads tend to have wider and irregular shape and being partial paved and unpaved, storm water flowing in them has a larger flow boundary and rougher surface in which it travels through. As a result, it can be observed that the flow velocity in the system does not exceed 3m/s even in Reaches with steep longitudinal slope (refer to Figure 42 for the flow velocity in the drainage system). The typical flow hydraulics of the existing system can be observed with the profile plot of Reach 18 presented in Figure 43 below, while all the modelled Reaches for the current development scenario are presented in Appendix D.

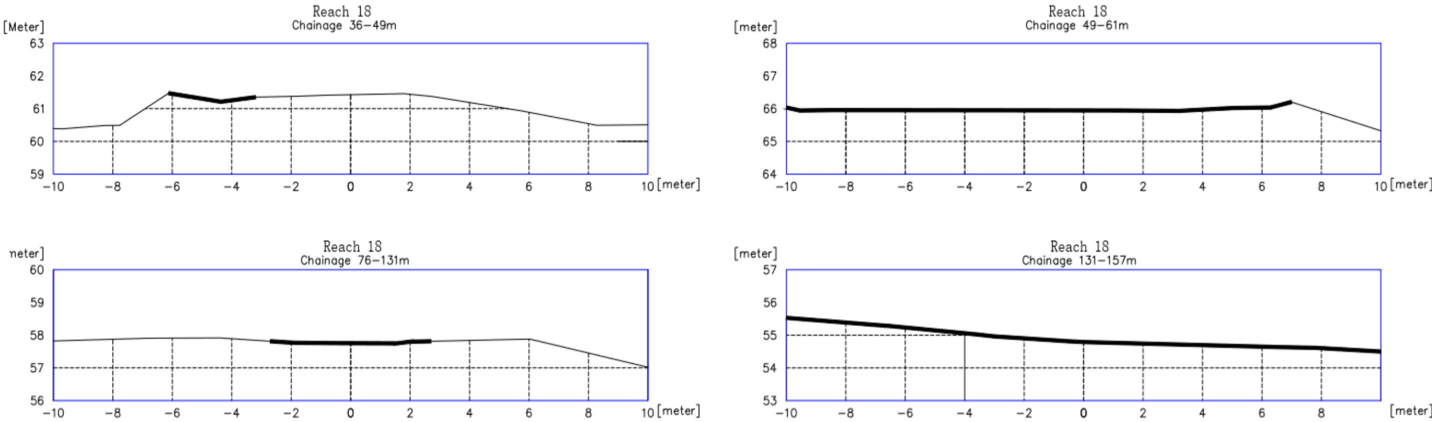
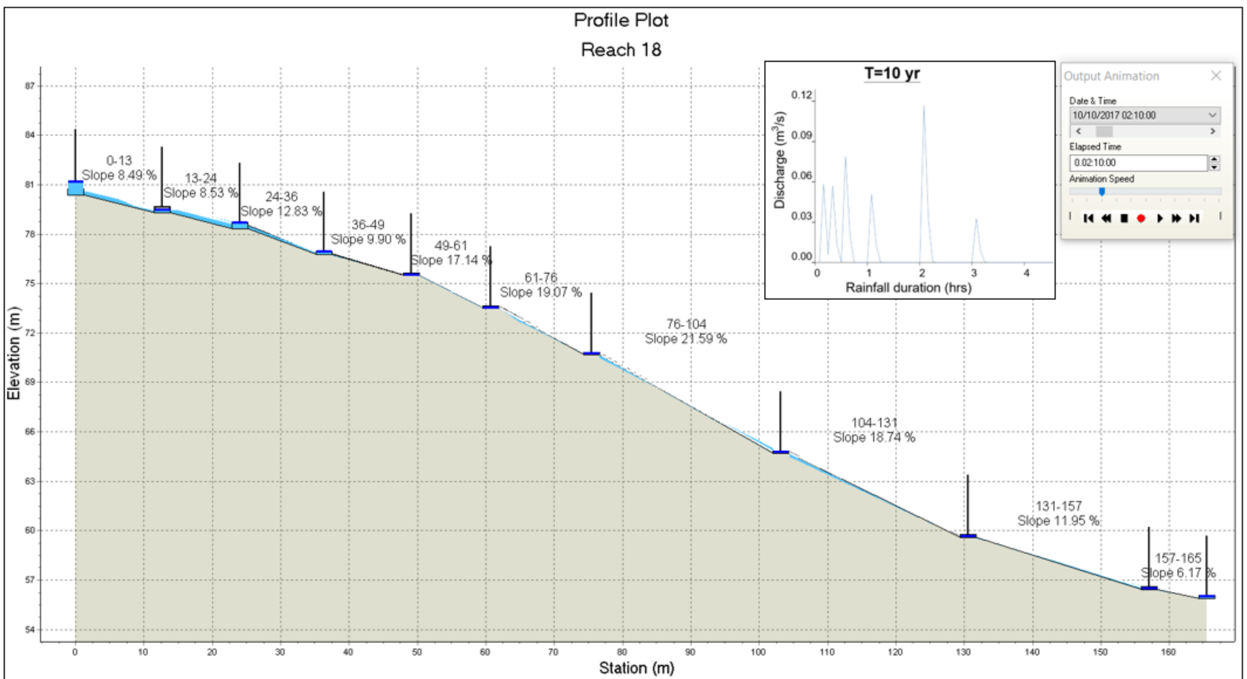


Figure 43: Cross-sections used to model the drainage structure in Reach 18 for the current development scenario.



Chainage (-):	0-13	13-24	24-36	36-49	49-61	61-76	76-104	104-131	131-157
Max Vel (m/s):	1.78	1.79	1.37	1.92	0.87	1.24	1.24	1.18	1.53
Max Depth (m):	0.13	0.13	0.18	0.02	0.03	0.05	0.05	0.34	0.08

Figure 44: Profile plot of Reach 18 from model 2 simulation with existing road sections as drainage structure

<u>Model 3: Future urban development scenario using SCS Method for the dynamic rainfall analysis</u>

This model of the future urban development scenario was simulated with the adjusted critical channels (with reference to Figure 30 and 32) from model 1. The result of the simulation displaying the systems' network is illustrated in Figure 46 and 46. The designed channels satisfied both the discharge capacity and the flow velocity respectively for the 10-year storm event.

With resurfacing of the road pavement and channelization of storm water runoff into drains, it can be observed that the channel's flow velocity is greater in the future urban development's drainage network (with reference to Figure 42 and 47). Whereas, storm water runoff in the existing system was conveyed by the roads and acted like natural streams. The increase of the channel's flow velocity can also be seen by the longitudinal profile plot of Reach 18 presented in Figure 49 below for the future development scenario simulation (refer to Figure 44 for the profile plot of Reach 18 of the existing development scenario for the comparison). All the modelled Reaches for the future development scenario with dynamic rainfall simulation (Model 3) are presented in appendix D.

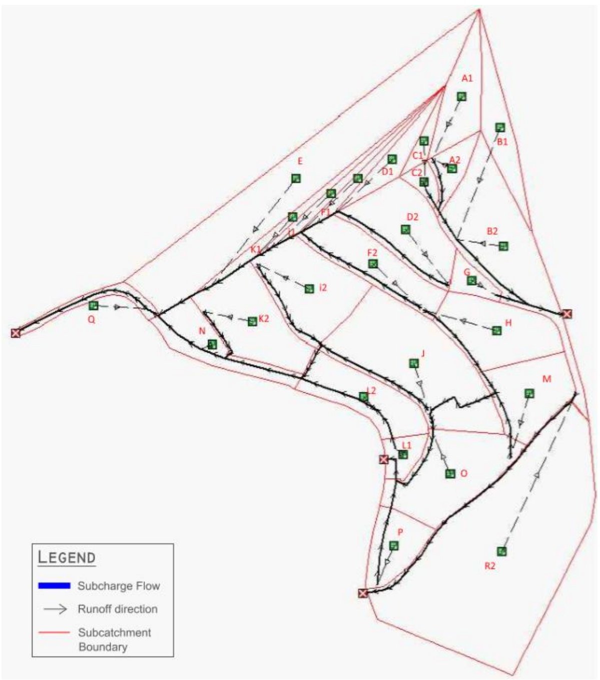


Figure 46: Overview of the modelled future development system's channels and their related capacity from Model 3 simulation.

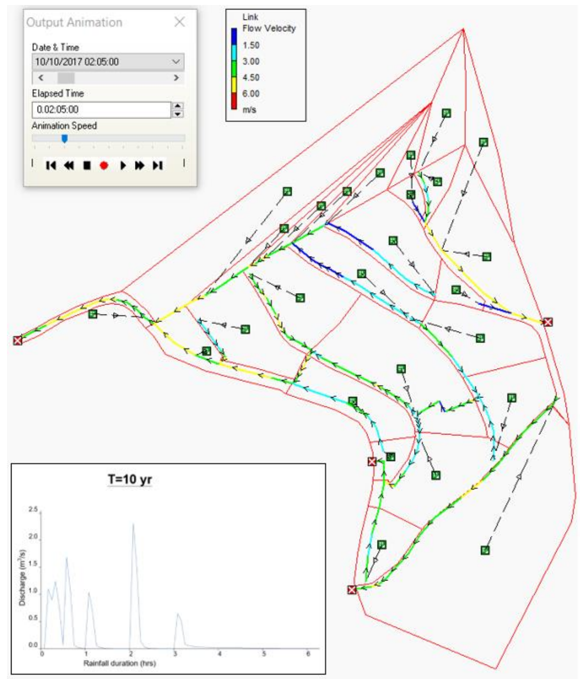
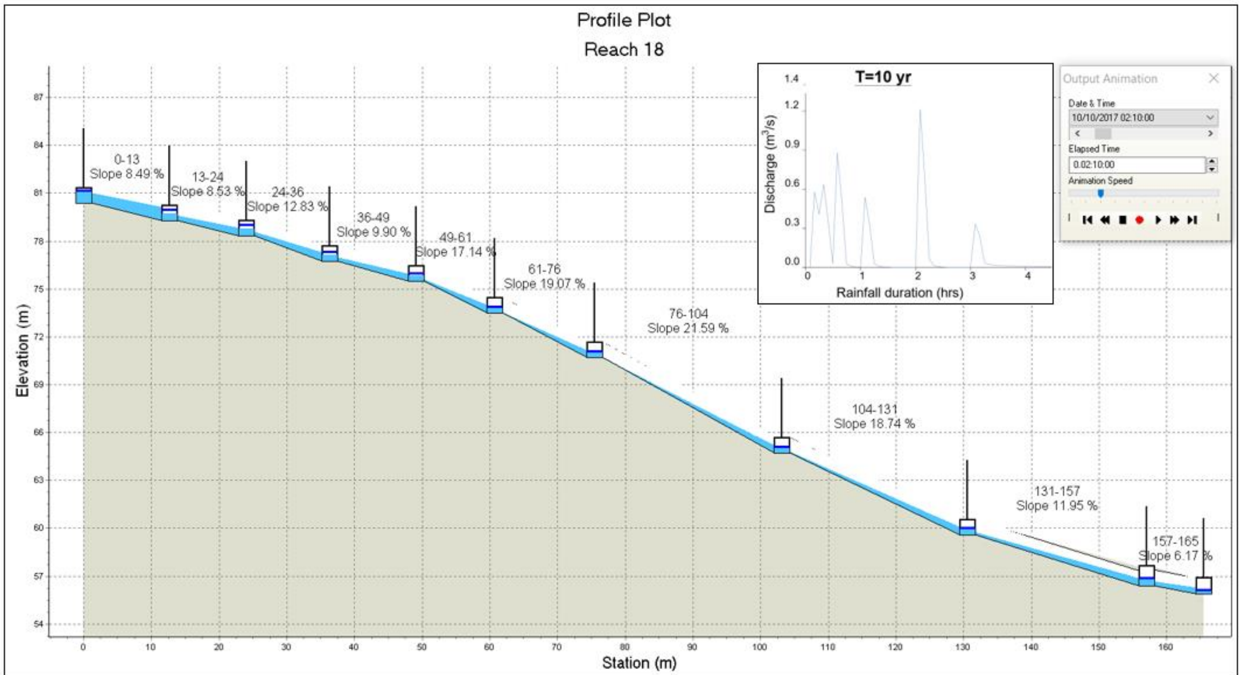


Figure 45: Overview of the modelled future development system's channels and their related flow velocity at critical duration of 2:05 hr. (shown in the hydrograph; bottom left) during 10- year storm event from Model 3 simulation.



Chainage (-):	0-13	13-24	24-36	36-49	49-61	61-76	76-104	104-131	131-157
Max Vel (m/s):	4.15	3.29	3.90	4.44	5.53	5.77	5.85	5.74	4.80
Max Depth (m):	0.51	0.65	0.55	0.48	0.38	0.37	0.34	0.36	0.42

Figure 47: Profile plot of Reach 18 from model 3 simulation with adjusted channels used from model 1.

<u>**Comparison of the effect between stationary with the dynamic rainfall on the future urban development scenario**</u>

The comparison of the stationary rainfall event (SRE) and the dynamic rainfall event (DRE) pertaining to the future urban development scenario is carried out by using hydrograph at Reach 18 simulated from Model 1 and Model 3 respectively.

As shown in Figure 48 below, it can be noticed that the critical duration of the 10-year storm occurred in Reach 18 from the SRE simulation was shorter at 0.17hrs (16.2min) comparing to DRE simulation which occurred at 2:05 hrs. However, the peak runoff at that duration between the SRE and the DRE had little to no significant difference (1.12m^3/s for SRE and 1.13m^3/s for DRE).

Due to the lack of spatially continuous and accurate long-term precipitation dataset over Sint Maarten. A more precise dynamic rain dataset could not be incorporated for the DRE simulation. For this reason, the IDF curve values of Sint Maarten was incorporated into the DRE simulation in Model 3 which was also used for the SRE simulation for Model 1. Hence, this explains the little to no changes of the peak runoff between both the SRE and DRE model simulation.

Because the peak runoff between the SRE and the DRE simulation was relatively similar, this did not reflect any hydraulic problem such surcharge flow and or undesirable velocity in the storm drains for the future urban development from the Model 3 (refer to Figure 46 and 46). However, with the longer storm duration from DRE this might have an impact in downstream channel (such as the Dutch Cul de Sac stream) particularly in the lower lying channels because the volume of storm water runoff will be larger (comparing to the SRE simulation) and discharged in it.

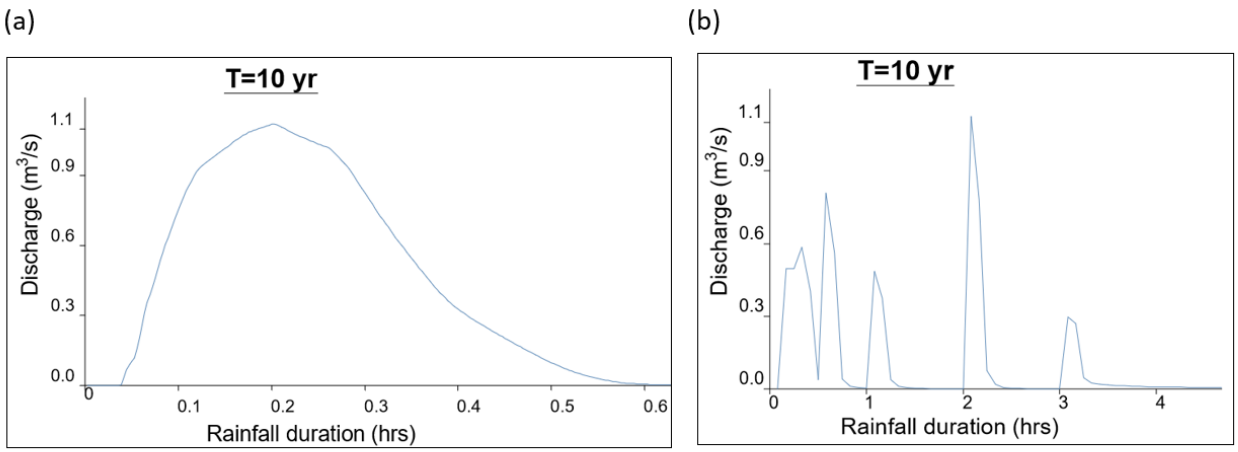


Figure 48: Hydrograph at Reach 18 displaying the peak runoff versus the rainfall duration of the SRE (a) and the DRE (b) for the for 10-year storm simulation from Model 1 and 3 respectively.

Comparison of the hydrologic effect between the current development scenario with the future urban development scenario

With increasing development expected in the Waymouth hills, large proportion of the surfaces becomes impervious due to the development (such as of paved roads, parking lots and building). As a result, storm water is prevented from infiltrating into the ground and the storage capacity of the soil is reduced (Table 23).

Table 23: Infiltration & runoff volume for the current development scenario vs future urban development scenario

Total	Current development	Future Urban development
Infiltration (m^3)	10,717	10,231
Runoff (m^3)	4,228	4,419

The values presented in Table 23 shown above were generated from Model 2 and Model 3. Both of these models used their respective calculated CN value representing the degree of development in the sub-catchment for both the scenarios. In the current development scenario, it was measured and found that 20,238m^2 (13.7% of the total catchment area) is impervious occupied by housings and existing road infrastructure in the current development. Whereas in the future urban development scenario it was calculated and found that 31,550 m2 (21.3% of the total catchment area) will be impervious occupied by housings and future road infrastructure after if the full potential of the Waymouth Hill is built. This increase in development for the future development scenario showed a larger CN value when comparing to the current development scenario and yield higher runoff as shown Table 23. Refer to Table 7C and 8C (Appendix C) for the hydrologic effect (i.e. infiltration and runoff volume) from each sub-catchment for the current and future urban development scenario from the simulation of their respectively CN value for the 10-year storm event. Under the circumstances of increased impervious area and higher runoff volume from the future urban development this leads to higher peak flow rates in the network system (Figure 49 below).

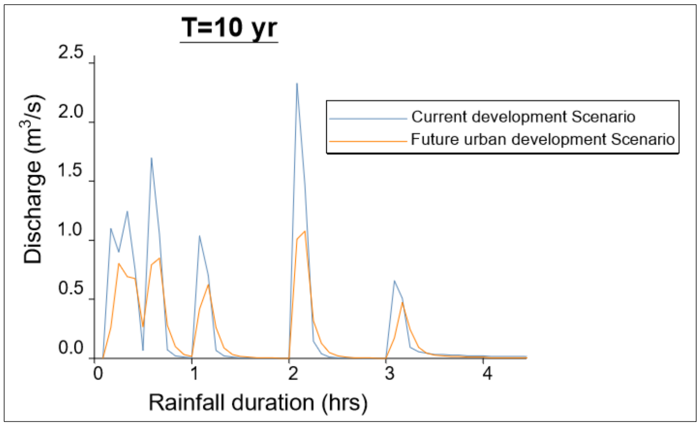


Figure 49: Hydrograph of the system's network outflow rate for the current development scenario versus the future urban development scenario for the 10-year storm event.

Comparison of the hydraulic effect of the stormwater runoff between the current development scenario with the future urban development scenario in the Dutch Cul de Sac Stream

In the report St Maarten Stormwater Modelling Study (Vojinovic & Bonilo, 2006), it described the Dutch Cul de Sac stream consisted of lined channels and natural waterways which stretches 3275 meters until it discharges into the Fresh Pond, moreover this stream conveys stormwater runoff from a total area of 617 ha. Figure 50 below illustrating the stormwater flow network into the stream.

To illustrate the mentioned runoff effect from both the development scenarios in the Dutch Cul de Sac stream, only the relevant sections of the stream was incorporated in the modelling. Since modelling the full extent of this stream is out of the project scope. The sectional and longitudinal property of this stream incorporated in the modelling are presented in Table 24 below obtained from the report 'St Maarten Stormwater Modelling Study' (Vojinovic & Bonilo, 2006).

Table 24: Cross section and longitudinal section properties of the Dutch Cul de Sac channels incorporated in the modeling (Vojinovic & Bonilo, 2006).

		Cross section (m)			Longitudinal section (m)			Characteristic
	Chainage	Top	Bottom	Depth	Up Invert	Dn Invert	Length	
a5	-	2.00	2.00	1.25	56.40	54.54	50	Lined channel
a0	300-680	6.00	3.50	1.20	54.54	39.54	380	Natural waterway
a0	680-942	4.60	1.00	1.20	39.54	33.00	263	Natural waterway
a0	942-1250	4.80	1.10	1.30	33.00	28.00	307	Natural waterway
a0	1250-1458	4.00	2.70	1.20	28.00	19.00	208	Natural waterway
a0	1458-1781	8.00	1.00	2.25	19.00	17.00	304	Natural waterway

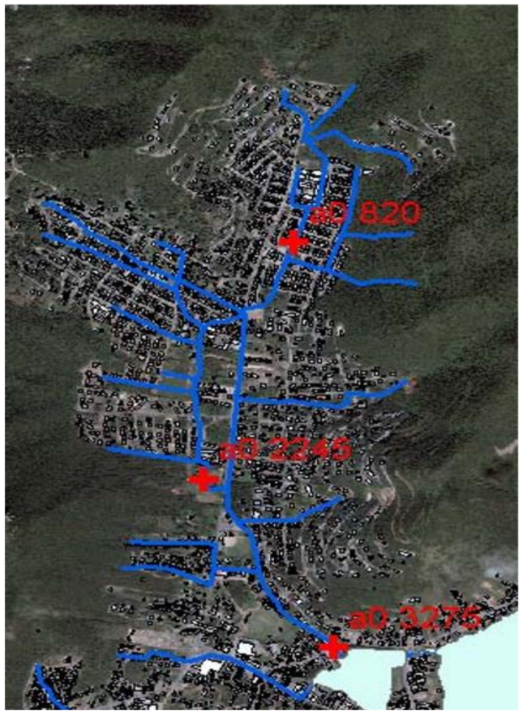
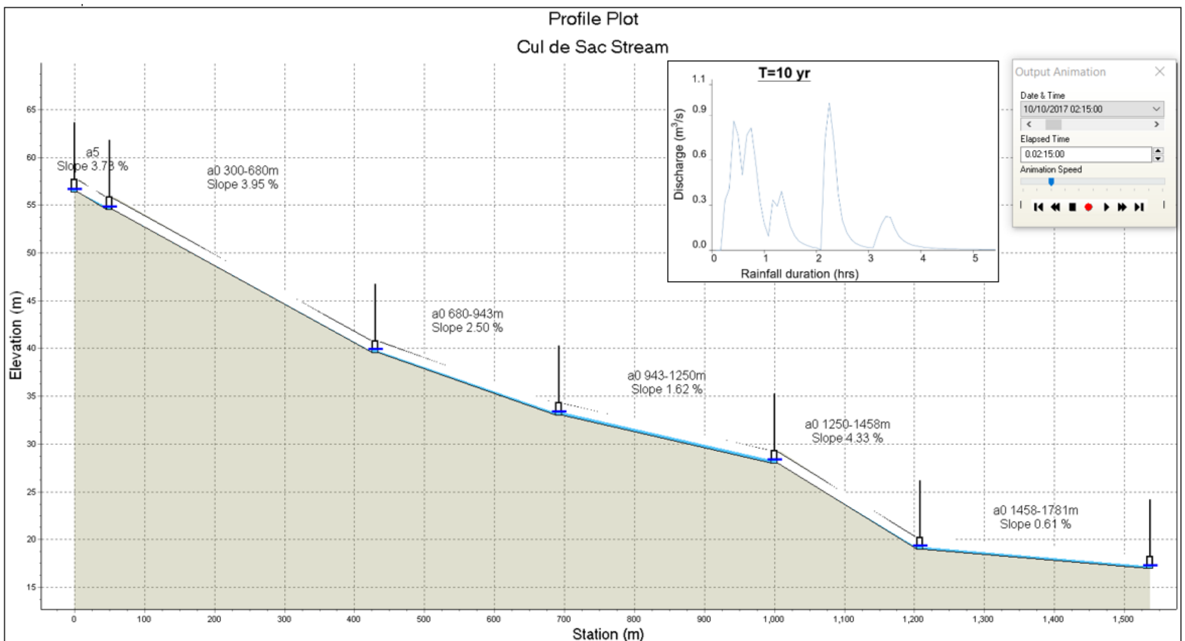


Figure 50: Map of the Dutch Cul de Sac displaying the stormwater flow network into the Dutch Cul de Sac stream (Vojinovic & Bonilo, 2006).

As mentioned earlier in this chapter, there were 4 outlets in which stormwater discharges out from the Waymouth Hill's drainage network. Reach 18 can be considered as the main outlet since it conveys 50% of the study area's storm water runoff and contributing to the headwater (or upstream) of the Dutch Cul de Sac stream. Whereas, Reach 3 conveys 16% and Reach 15 conveys 20% of the total runoff from the Waymouth Hills catchment. The storm water runoff from Reach 3 and Reach 15 are also discharged into the Dutch Cul de Sac stream but further downstream at a0 chainage 942m. Finally, reach 10 conveys 14% of the total runoff from Waymouth Hills and does not contribute into the Dutch Cul de Sac stream this Reach discharges into the Lower Prince Quarter catchment. Figure 51 and 52 below illustrate the effect of the stormwater runoff in the Dutch Cul de Sac stream for both the current development and future urban development scenario simulation.

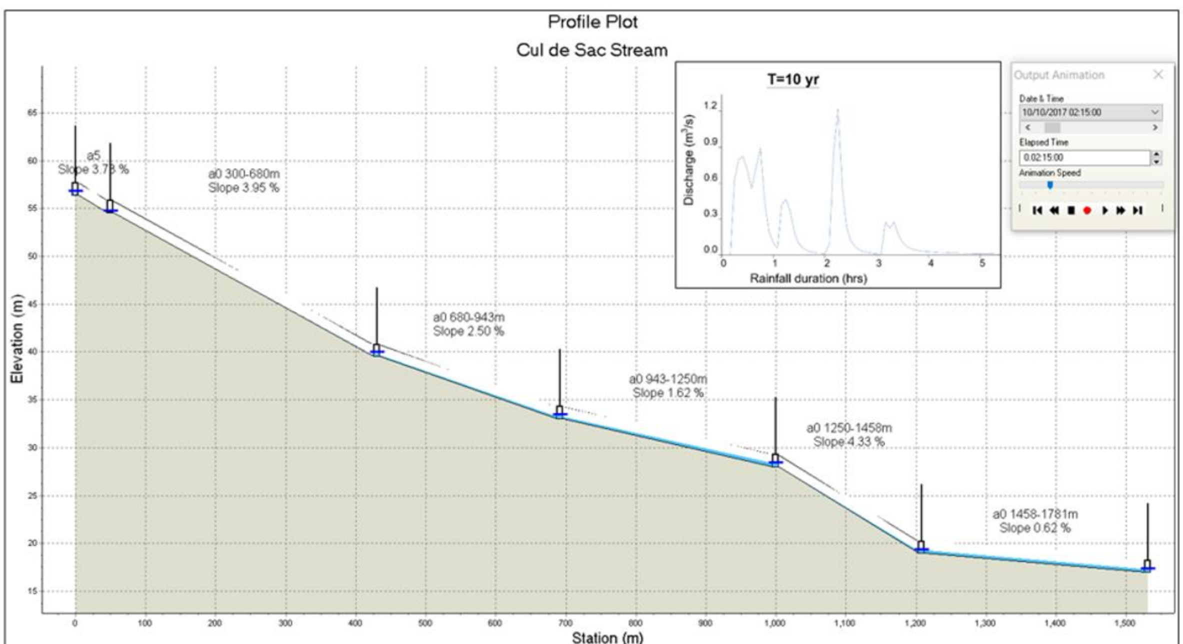
By comparing the both results illustrated in Figure 52 and 53 above, it can be observed that the excessive runoff from the future urban development the flow depth and flow velocity in the Dutch Cul de Sac stream increased. Moreover, a larger increased in flow depth was observed in the flatter terrain channel (at chainage 1458-1782m) of the stream.

Despite an increase of both flow depth and flow velocity is seen in the Dutch de Sac stream, it can be noted that this excessive runoff did not have a significant impact in the stream. This can be explained by its relatively large channel sections comparing to the ones used for the study area. This stream was designed to convey stormwater runoff from a total area of 617ha and whereas the runoff from the Waymouth Hills encompass of 16.4 ha, which is only 3% of the total runoff it is conveying.



Chainage (-):	a5	a0 300-680	A0 680-943	A0 943-1250	A0 1250-1458	a0 1458-1782
Max Vel m/s):	2.06	1.85	1.92	1.62	1.99	1.03
Max Depth(m):	0.22	0.15	0.28	0.33	0.18	0.26

Figure 51: Profile plot of the Dutch Cul de Sac stream with present development stormwater runoff from 10-year storm event (with hydrograph at chainage a0 1458-1782m of Dutch Cul de Sac stream).



Chainage (-):	a5	a0 300-680	a0 680-943	a0 943-1250	a0 1250-1458	a0 1458-1782
Max Vel m/s):	4.05	2.18	2.17	1.86	2.19	1.12
Max Depth(m):	0.15	0.16	0.39	0.34	0.20	0.30

Figure 52: Profile plot of the Dutch Cul de Sac stream with future development stormwater runoff from 10-year storm event (with hydrograph at chainage a0 1458-1782m of Dutch Cul de Sac stream).

Assessing the potential benefit of having a detention pond

In order to assess the benefit of a potential pond for the excessive stormwater runoff in the Dutch Cul de Sac stream, the hydrodynamic model for the future urban development scenario (Model3) has been altered to incorporate such structure. The pond was introduced along the upstream of the Dutch Cul de Sac stream chainage a0 300-680m (Figure 53 below). The pond was modelled with a storage area about 980 m2 with an invert level at 51m with the top of the pond at an elevation of 53.5m. The inflow to the pond was modelled through chainage a5 (i.e. runoff from Reach 18 flow into), and the outflow of the pond was discharge into chainage a0 350m. In order to simulate the pond outflow structure an orifice and a weir has been introduced at the end of the pond close to chainage a0 350m. A rectangular orifice of 0.4m width by 0.40m height was modelled, with the invert level at 51m and the crest level 53m. Moreover, a rectangular weir with a width of 1m, the crest level was set at elevation 53 m, which is 1.5m higher that the invert level of the pond.

Figure 53: Location of the modelled detention pond

With the detention pond placing at the headwater of the Dutch Cul de Sac stream it reflected a decrease peak flow rate downstream of the Dutch Cul de Sac stream. The overall reflected decreased peak flow discharge downstream at chainage a0 1458-1782m from 1.10m^3/s to 0.80m^3/s for the future urban development scenario (Figure 55a below). Moreover, a larger decreased of peak discharge rate can be seen in chainage a300-1250m from 1.27m^3/s to 0.37m^3/s (Figure 54b below). The reason for this can be explained by the stormwater from reach 18 is directly discharged into the pond and the peak flow discharge entering chainage a0 300-600 is controlled by the orifice and weir, whereas in chainage a0 1478m the peak flow discharge also includes the runoff from Reach 3 and 10 from the Waymouth Hills which was discharged into the Dutch Cul de Sac stream at chainage a0 1250m (further downstream which was not detained in the pond). Refer to Figure 55 below to see the profile plot of the Dutch Cul de Sac stream for the describe hydraulics.

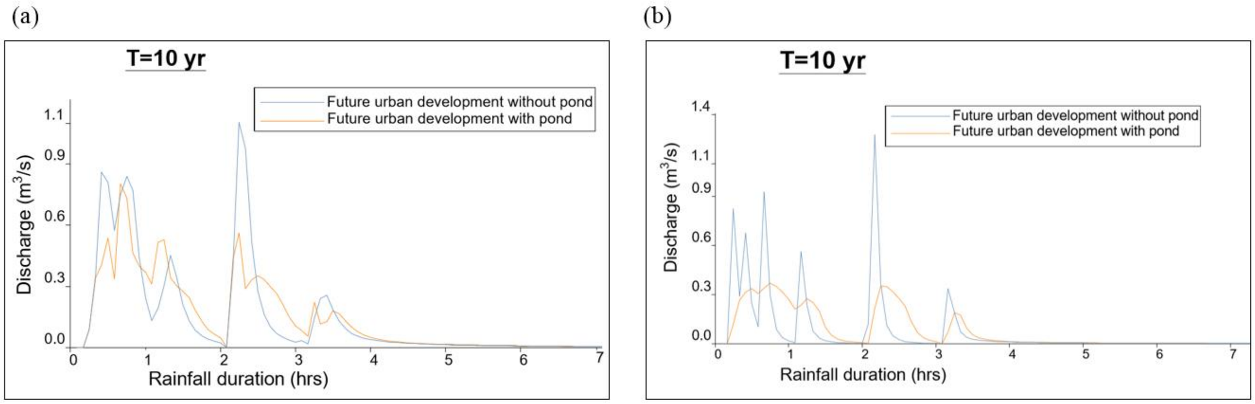
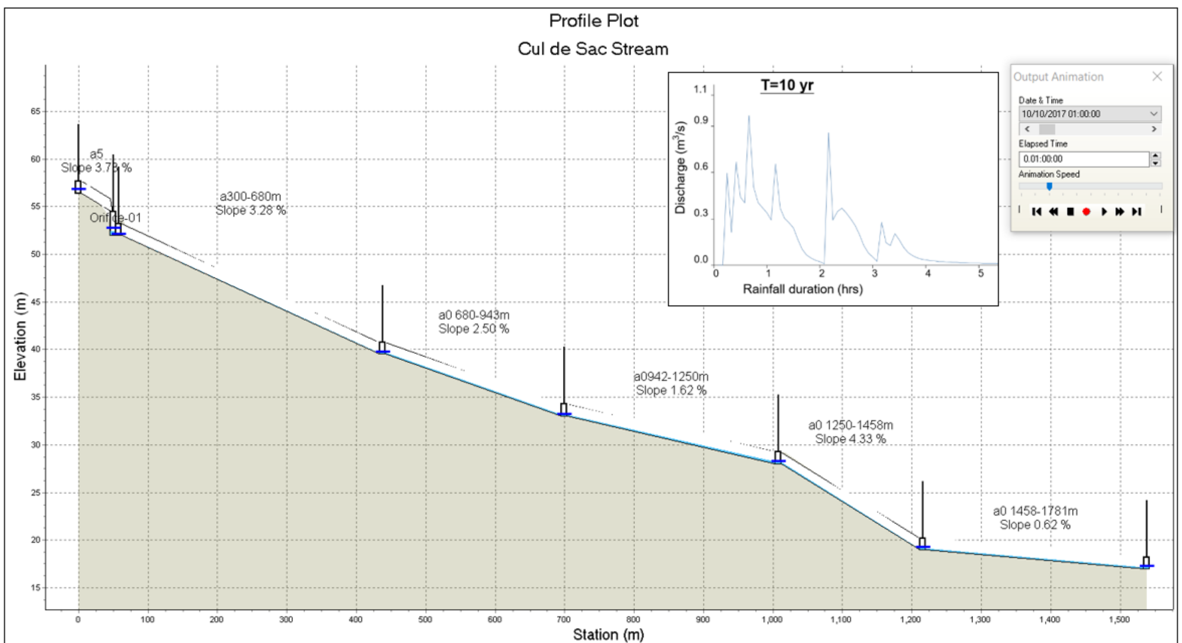


Figure 54: (a) Hydrograph at chainage a0 1458-1782m and (b) hydrograph at chainage a0300-680m of the Dutch Cul de Sac stream for the future urban development with pond versus the future urban development without pond for the 10-year storm event.



Chainage (-):	a5	a0 300-680	a0 680-943	a0 943-1250	a0 1250-1458	a0 1458-1782
Max Vel m/s):	4.05	1.13	1.41	1.21	2.00	1.00
Max Depth(m):	0.15	0.09	0.20	0.22	0.17	0.25

Figure 55: Profile plot of the Dutch Cul de Sac stream with the detention pond (with hydrograph at chainage a0 1458-1782m of Dutch Cul de Sac stream).

The peak outflow regulated by the outlet control structure in the pond can be said to be from the orifice since the max surface water level in the pond obtained was 51.7m. Moreover, the outflow of orifice starts to discharge as soon as storm water enters and detained in the pond during the stormevent, this outflow increases with increasing water depth in the pond. Moreover, the peak outflow discharge from the orifice (0.37m3/s) occurred when the surface water level in the pond raised to an elevation of 51.7m. This can be seen in Figure 56 below.

(a) (b)

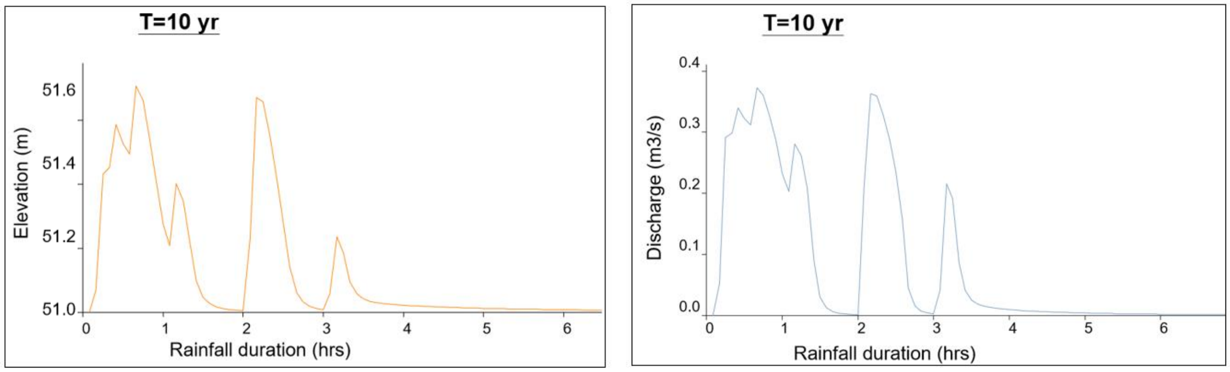


Figure 56: (a) Hydrograph of the pond displaying the water depth in respect to its elevation versus the rainfall duration and (b) hydrograph of the orifice displaying the peak discharge versus rainfall duration.

In order define the limit of the pond and illustrate the benefits of the weir and addition inflow is incorporated in the model. The inflow is taken from the runoff of the South Rewards, since the runoff from this area also discharges into the headwater of the Dutch Cul de Sac and convey in this stream. The characteristic of this catchment area was simulated with an area of 38 ha and using the same dynamic rainfall event. Two simulations were carried out, one was to assess the effect downstream at chainage a0 1458-1782m of the Dutch Cul de Sac from addition runoff and another was to assess the benefit and the limit of the pond with addition runoff detained in the pond.

With the additional runoff from the South Cul de Sac, max rise of the pond water level was at an elevation of 53.3m. This rise of the pond water level was cause by the large inflow rate of the addition runoff detained in the pond greater than the outflow of the orifice. In the event of this, overflow over the weir occurred at 53m at which the crest level was set, the peak outflow discharged into the Dutch Cul de Sac stream was greater during this peak water level rise can be observed. This prevented overtopping occurring in the detention pond (Figure 57).

(a) (b)

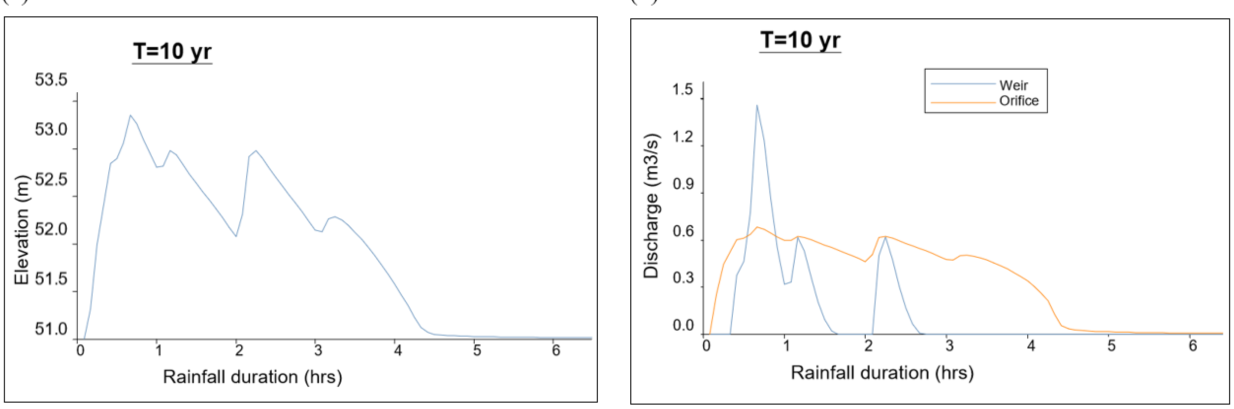


Figure 57: (a) Hydrograph of the pond displaying the water depth in respect to its elevation versus the rainfall duration and (b) hydrograph of the orifice displaying the peak discharge versus rainfall duration with addition runoff from South reward catchment detained in the pond.

By channeling additional runoff into the pond, the peak flowrate in the downstream at chainage 10-1458-1782m of the Dutch Cul de Sac stream reflected a decreased from 3.6m^3/s to 2.1m^3/s (Figure 58). The hydraulic of the additional runoff in the Dutch Cul de Sac stream are illustrated in Figure 59 and 60 below.

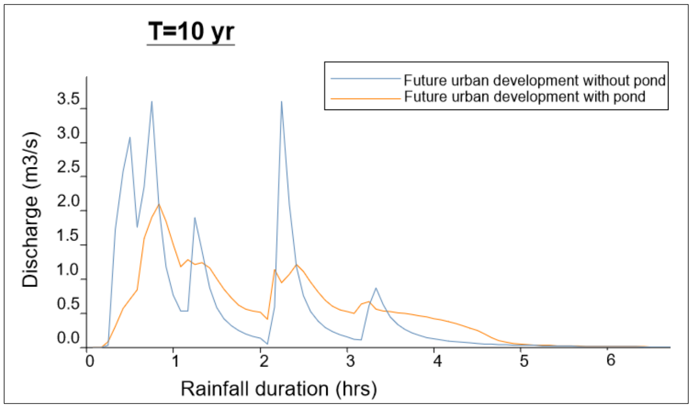
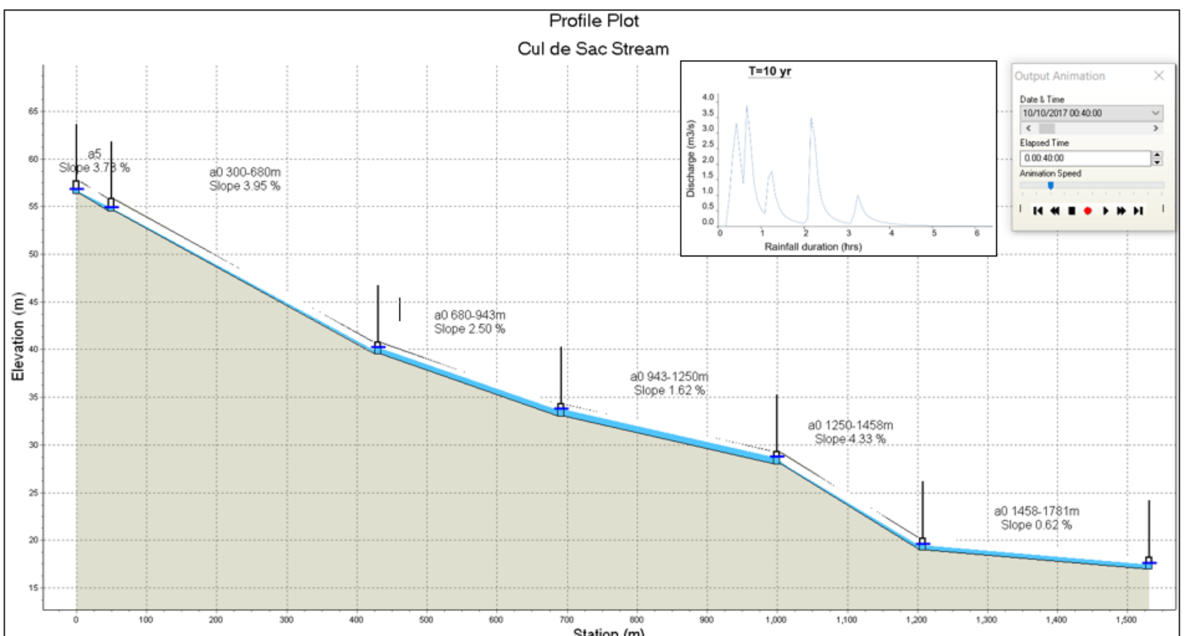
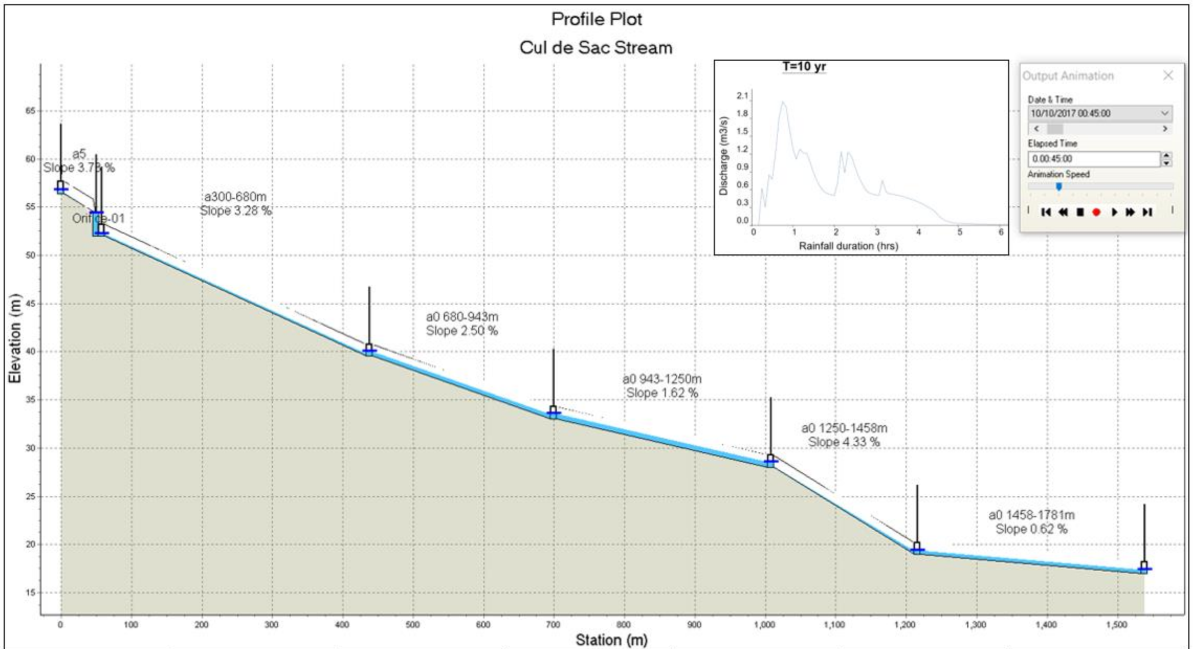


Figure 58: Hydrograph at chainage a0 300-680m of the Dutch Cul de Sac stream for the future urban development with pond versus the future urban development without pond for the 10-year storm event with addition runoff from the South Rewards catchment.



Chainage (-):	a5	a0 300-680	a0 680-943	a0 943-1250	a0 1250-1458	a0 1458-1782
Max Vel (m/s):	5.82	2.99	2.82	2.44	3.25	1.58
Max Depth (m):	0.33	0.31	0.67	0.74	0.43	0.54

Figure 59: Profile plot of the Dutch Cul de Sac stream with the detention pond (with hydrograph at chainage a0 1458-1782m of Dutch Cul de Sac stream).



Chainage (-):	a5	a0 300-680	a0 680-943	a0 943-1250	a0 1250-1458	a0 1458-1782
Max Vel (m/s):	5.82	2.17	2.33	2.00	2.57	1.24
Max Depth (m):	0.33	0.26	0.51	0.56	0.29	0.42

Figure 60: Profile plot of the Dutch Cul de Sac stream with the detention pond (with hydrograph at chainage a0 1458-1782m of Dutch Cul de Sac stream).

The overview analysis of the upstream and the intermediate downstream of the Dutch Cul de Sac from the simulations presented in this chapter that the Dutch Cul de Sac stream was sufficient of conveying the excessive rainfall runoff from the future urban development of the Waymouth Hills. Moreover, this include the addition runoff from the South Rewards catchment. This analysis can also be compared with the channel capacity of the Dutch Cul de Sac from the report 'St Maarten Stormwater Modelling Study' (2006) presented in Figure 61 below.

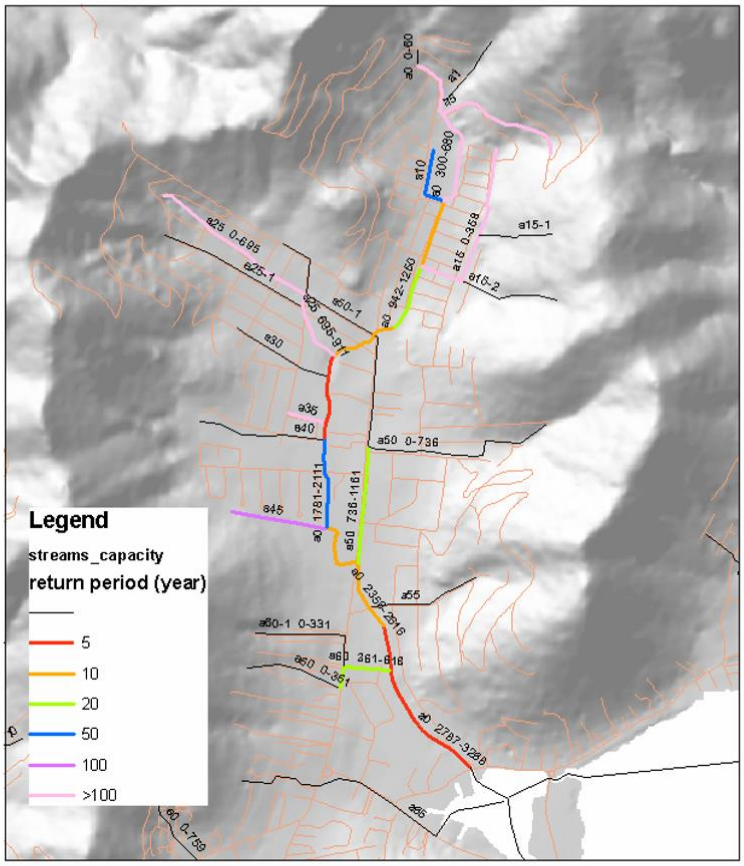


Figure 61: Map of the Dutch Cul de Sac illustrated the system network in respect to its channel capacity for different storm return period (Vojinovic & Bonilo, 2006).

Figure 61 illustrated that the channel capacity in the Waymouth Hills has a potential to convey stormwater runoff from a 100 year-storm event this, capacity can be explained by the steepness of the terrain in the Waymouth Hill. From the results presented earlier in this chapter (with reference to Table 20), it can be noted in the channel such as Reach 18, 20, 21, and 22 the flow depth occurred in these channels were relatively low and comparing to their respective channels. With the extra free board (distance between the normal water level in the structure to the top of the structure) and steep channel slope characteristics in these channels not doubt that they can convey additional excessive runoff from heavier rainfall than the design 10-year storm event. On the other hand, in the event of a larger storm event occurs significant increase in the flow velocity in these channels can be expected and this will heavily affect the downstream channels in the Dutch Cul de Sac stream.

In the report 'St Maarten Stormwater Modelling Study' (2006), DEM (digital elevation modelling) were used to map the potential flood hazard in the Dutch Cul de Sac area presented in Figure 62 below. During the 10-year storm event, severe flooding can be expected in the downstream of the Dutch Cul de Sac in the future when the entire region of the Dutch Cul de Sac region is developed.

The result of this flooding can be concluded as result from excessive runoff due to future development. Additionally, insufficient capacity of the downstream channels which are only limited to convey stormwater runoff for 5-year stormevent.

For this reason, by inducing the detention pond at the headwater of the Dutch Cul de Sac stream the peak flow rate entering into the downstream can be controlled. This reduces overwhelming of the channels with lower conveyance capacity during heavy storm events.

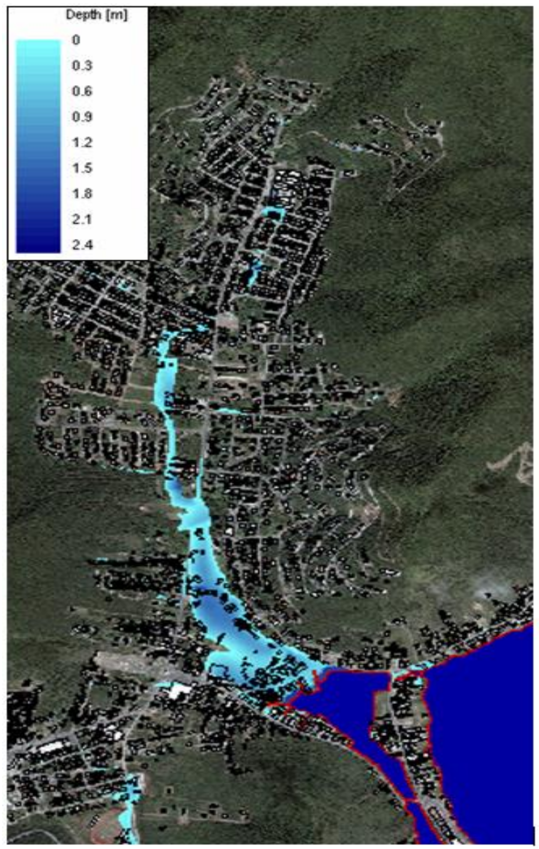


Figure 62: Flood hazard Map for 10-year storm event for future development condition (Vojinovic & Bonilo, 2006). .

6. Conclusion and Recommendations

6.1. Conclusions

The purpose of this research study was to determine the most optimal stormwater drainage solution that can best be used for the future urban development in the Waymouth Hills. The analysis of the current situation in the Waymouth Hills concluded that stormwater runoff is solely conveyed by its current road infrastructures. These roads are partially paved and unpaved, and it is inadequate to convey the storm water runoff. Furthermore, these roads are constantly undermined from the runoff.

The requirement for the new stormwater drainage system was to convey stormwater runoff for a 10-year storm event with a structural life span of 50 years. The 10-year stormwater runoff entering in to the drains are limited to a maximum filling of 75% in them. Furthermore, the maximum flow velocity in the drains is restricted to 6m/s.

A multi-criteria analysis (MCA) was performed to derive the most feasible stormwater drainage that can be used for the future urban development for the study area. Three selected alternatives (i.e. open ditch, concrete U-gutter, and underground drains) were evaluated by using formulated criteria based on the project specifications and limitations. The alternative concrete U-gutter scored the highest with 3.3 points, followed by open ditch and underground drains with 2.7 and 2.5 points respectively. Furthermore, the sensitivity analysis of the MCA showed that the results of concrete U-gutter can be considered to be relatively robust. Hence, this concludes that the concrete U-gutter is the best choice for the study area.

The development in the current situation occupied by 20,238m^2 (13.2%) of impervious surface whereas in the future urban development the impervious surface cover 31,550 m^2 (21.3%) of the total study area. This increase in impervious area reflected a larger stormwater runoff volume from 4241m^3 to 4463m^3 for the 10-year stormevent. Furthermore, the peak flow rate in the storm drain also increased from this runoff. This peak flow rate was used to calculate the stormdrain's size for the respective location in which it was discharged into.

The analysis between the stationary and dynamic rainfall for the 10-year storm event simulation illustrated that the critical duration occurred at 0.17 hr. where as in the dynamic rainfall it occurred at 2.02hr in the study area. On the other hand, the peak runoff from both rainfall events showed no significant differences in the storm drains (such as in Reach 18 the peak runoff was 1.12m3/s and 1.13 m3/s respectively). As a result of this, both flooding and undesirable velocity did not occur in the new determined storm drains.

The assessment of the flood prone area in the Dutch Cul de Sac area resulted from low channel capacity in the downstream channels of the Dutch Cul de Sac which are only limited to convey stormwater runoff for a 5-year storm event. Additionally, with increasing development expected in the Dutch Cul de Sac region in the future, excessive rainfall runoff can reflect a more frequent and severe flooding. Having a detention pond at the head-water of the stream can decrease the severity of the flooding, since it can decrease the peak flow rate discharging into the downstream systems.

The full extent of the climate change impact on the stormwater runoff and in the storm drains in the Waymouth Hills could not be determined in this research, however it was noted that warmer temperature, less rainfall but heavier intensity storm events are expected.

6.2. Recommendations

Multiple storm return event should be simulated in a hydrodynamic modelling program so that the impact on the drainage system can be analyzed and preventative measures can be proposed for such events that occur in the study area and area that are considered to be flood prone.

Stricter building regulations and inspections should be carried out in the building sector as many building structures were situated beyond their parcel boundary. This can restrict the potential of new road infrastructures and drainage infrastructure.

On-site-detention on each lot could be used. In certain parts of the world (e.g., Australia) such systems have proved to be capable of proving temporary storage of stormwater runoff from new developments and resisting discharge from property to a rate that existing channel are capable of accommodating. This would include the process of constructing the reservoir/ detention facilities within the properties area, to prevent excessive discharge from the new development.

It is recommended to have the detention pond at the head water of the Dutch Cul de Sac stream since it does not only store excessive stormwater runoff but also regulates the peak outflow to the Dutch Cul de Sac stream. This is already an area that is vulnerable to flooding particularly in the low-lying area. Such a pond could also serve as a recreational facility, by landscaping it to be visually attractive and sympathetic with the environment. It could also serve as a playground and picnic place with designated parking lot. Only in the case of heavy storm the lower part would be flooded and then take over the function of a detention facility.

In addition to any of the above measures, maintenance and cleanup of all waterways or channels and culverts should be done on a regular basis. This is particularly necessary after each storm event so that the amount of erosion material and sediments is maintained at a minimum level.

7. References

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Appendix A. Multi-criteria Analysis

Table of Contents

1. Introduction

The current infrastructure in the Waymouth Hills is predominantly with unpaved and partially paved roads. Stormwater runoff are confined to and is conveyed by these roads. Moreover, these roads are often heavy eroded from the runoff, due to its inadequate hydraulic capacity and the non-existence of any storm water drainage structure in the Waymouth Hills. For the purpose of this research, a MCA was conducted to identify the most feasible stormwater drainage system, that can be used for the Waymouth Hills. To do so, three viable alternatives have been set up, to evaluate and compare the alternatives against each other. The comparison of the alternatives is based on criteria in relevance to the established project specifications. Weights are assigned to each criterion in order to highlight their importance in the MCA. The evaluation of each alternative was scored (i.e. through rating) by assessing how well they perform with respect to each criterion and a pre-defined scale is used for this. The alternative that received the highest score is the alternative that (in overall) is associated with the most positive grading (i.e. the most proffered among the selected alternatives).

2. Identified alternatives

Three alternatives been drawn up and incorporated for the MCA. These alternatives were proposed to reflect the defined problem of this research. The determined alternatives used for MCA is as follows: (Open ditch, Concrete U-Gutter, and Underground drain)

Alternative 1

The first alternative is by using open ditch along the road. Open ditch is constructed on the lining of the road where storm water runoff is collected and conveyed from surface runoff from subcatchment areas and from road pavement. Open ditch can have a V-shaped ,U-shaped and or trapezium shaped channel section. The shape or the slope in which the open ditch can be use is heavily dependent on the soil properties be such that the banks do not collapse when over-saturated. The construction for this type of drainage can be relatively easier and cheaper to construct comparing to other stormwater drainage. Due to the necessity of having slopes for its stability, it requires more space. In areas where the terrain profile is steep scouring or undercutting on the bottom and sides of the channel may occur. This result of transporting debris and may block the drainage downstream. This type of storm drainage not only require more frequent maintenance of the bank but also the downstream drainage.

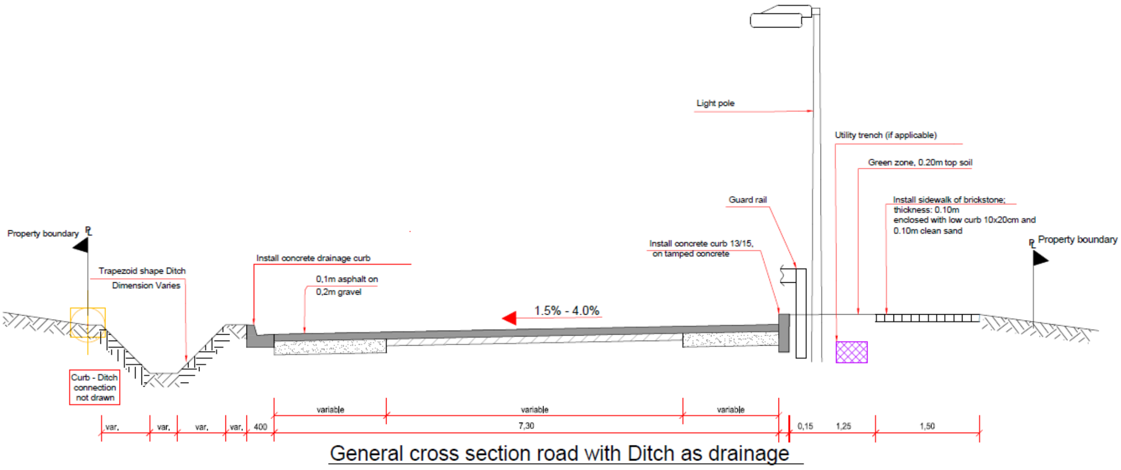

Figure 1A: Alternative 1 (Open ditch)

Alternative 2

The second alternative is by using concrete gutter. This type of storm water drainage works similarly to the open ditch and it is constructed on the lining of the road. Concrete gutter can have any desire shapes and does not require side slopes depending on the shape of the channel used. Square and rectangular concrete gutters are mostly used in urban areas because it utilizes the road space more efficiently. This type of drainage can be constructed in cast in situ or precast concrete and due to its durability, erosion of the channel is significantly lower comparing to open ditch.

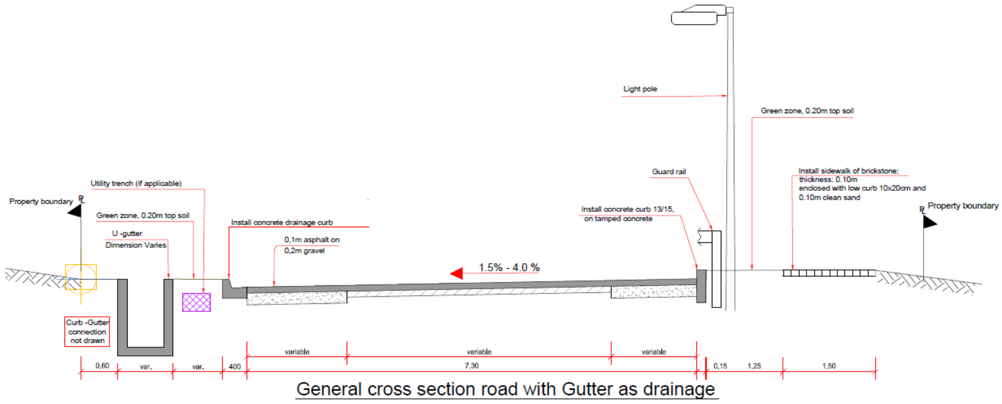

Figure 2A: Alternative 2 (Concrete U-Gutter)

Alternative 3

The third alternative is by using underground drains. This type of storm water drainage is mainly used from precast concrete or plastic elements. It is built beneath the centre or on the sides of the street. This type of drain is mainly use in urban areas and city centre, where very limited space is available. Having this type of drain utilizes the space road more efficiently. This type of drain can used for storm water, waste water purposes and or combined. For storm water drainage purpose, storm water is collected or fed by side inlets from the street curb and grated inlet.

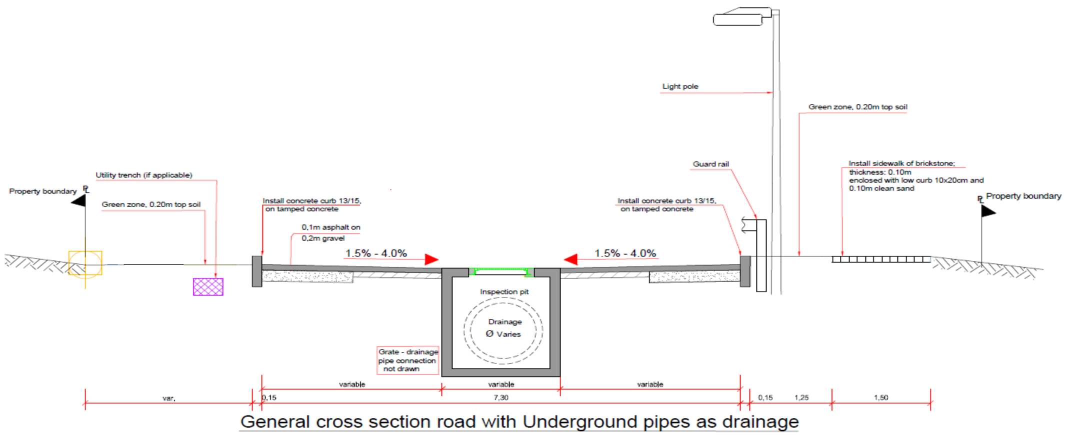

Figure 3A: Alternative 3 (Underground drain)

3. Identified alternatives

Six criteria have been formulated for the comparison of the alternatives which would best reflect the important aspects that was required to achieve the goal and objectives of the study. The criteria that were chosen for the evaluation is describe below Table 1A.

Table 1A: Summary of the criteria used for the MCA

Criteria	Parameter analyzed	Description
Investment cost	• Land cost • Material and construction cost	• Different alternatives may require different amount of land area for construction – estimation of the land costs • Estimation of construction- and material cost for building the stormwater alternative on site.
Maintenance cost	• Operation- and maintenance costs of the system	• Estimation of the frequency of maintenance required for each alternative and the cost for such activity.
Environmental impact	• Impact on urban soil quality and erosion potential • Impact on the ecological habitat	• Estimation of effect on soil quality on site, sediment retention and erosion potential. • Potential for change in biological diversity at site.
Design aspects	• Complexity of construction	• Different alternative may require specific type of activities, machinery and equipment
Structural reliability	• Lifespan and reliability of structure	• Estimation of lifespan of system and measure of strength
Implementation time	• Duration of implementation	• Estimation of the duration required for the implementation of each alternative.

4. Identified criteria's weight

The weight of the criteria reflects the of importance of the criteria relating any research or project. A total score of 100% was used that were divided in accordance to their importance of this research into the following:

A total score of 100% was used that were divided in accordance to their importance of this research into the following:

Investment Cost: The investment cost provides a large influence on the global decision making, since it reflects the spending from the client or any parties involved. Therefore, this criterion is given 25%.

Maintenance cost: The activities involved in the alternatives will reflect the overall cost in a long term from the client's investment. Therefore, this criterion is given 20%.

Environmental impact: The terrain in the study area are predominately with steep slope. Disrupting of the natural environment and habitat will increase potential erosion in the study area, clogging the drainage systems, and reflect sediments transport to lower lying terrain. Therefore, this criterion is given 20%.

Design aspects: This criterion relates the functions of the structure and their capability to perform in long-term. Therefore, this criterion is given 15%.

Structural reliability: For the construction of the drainage system, complex construction may require the use of specific machineries and equipment. Accessibility of these machineries to the study area may be limited. Therefore, this criterion is given 10%.

Implementation time: Construction activities can interfere the traffic flow in the study area. Since, the hurricane season is within June to November (5 months period), construction within this period can be crucial. Work can be disrupted and delayed, hence reflects a higher investment cost. Therefore, this criterion is given 10%.

5. Scoring of the alternatives

The scoring of each criterion was carried out in terms of a rating. A scoring set of 5 option was used to rate the different alternatives with respect to the criteria in accordance to their performance presented in Table . The three alternatives (Open Ditch, Concrete U-Gutter, and Underground drain) was compared and scored separately from each other.

Table 2A: Summary of the alternative rating for the MCA

Criteria	Alternative Rating				
	1 - Poor	2 - Fair	3 - Good	4 - Very good	5 - Excellent
Investment Cost	Large easement required, construction and material are very expensive	Average easement required, construction and material are very expensive	Average easement required, construction and material are expensive	Average easement required, construction and material are average	No easement required, construction and material are cheap
Maintenance Cost	Very frequent maintenance required, and overall operation is very expensive	Frequent maintenance required, and overall operation is expensive	Few maintenances required, and overall operation is expensive	Few maintenances required, and overall operation is average	Few maintenances required, and overall operation is cheap
Environmental impact	Very high impact on soil quality, erosion potential and biodiversity changes	High impact on soil quality, erosion potential and biodiversity changes	Average impact on soil quality, erosion potential and low biodiversity changes	Low impact on soil quality, erosion potential and low biodiversity changes	No impact on soil quality, erosion potential and biodiversity changes
Structural reliability	Very low lifespan and delicate	Low lifespan and low sturdy	Average lifespan and average sturdy	Long life span and sturdy	Very long lifespan and very sturdy
Design Aspect	Very specific activities, machinery and equipment required	Specific activities, machinery and equipment required	General activity, machinery and equipment required	General activity and machineries required	General activity and equipment required
Execution time	Very long duration	Long duration	Average duration	Short duration	Very short duration

6. Description and motivation of MCA scoring results

This part of the chapter presents the scoring results of each alternative pertaining to the criteria and the motivation supporting its scores value.

Table 3A: Scoring results of alternative 1.

Alternative 1			
Criteria	**Score**	**Rating Description**	**Motivation**
Investment Cost	4	Average easement required, construction and material are average	Since this alternative is constructed on the lining of the road and require more spaces for its structure stability, it is expected that the construction of this alternative requires additional space that maybe outside of the parcel boundary from the state (government).
Maintenance Cost	2	Frequent maintenance required, and overall operation is expensive	Due to the steep slope of the terrain, surface runoff is expected to produced high flow velocity and undermine the system and in flatter terrain, sedimented is expected to deposit in after every heavy storm event. Hence this alternative required frequent maintenance. However, due to drainage structure are open surface (or channel), the maintenance operation for this alternative is simple and it is less expensive comparing to alternative 3.
Environmental impact	1	Very high impact on soil quality, erosion potential and biodiversity changes	Due to the steep slope of the terrain, surface runoff is expected to produced high flow velocity and undermine the system constantly. Sediments are expected to deposit in flatter terrain after every heavy storm event. Biodiversity constantly changes due to the maintenance operation and the changes of land use for the drainage structure.
Structural reliability	2	Low lifespan and low sturdy	Due to the possibility of constant of erosion in the system, the morphology of the channel is heavily affected, hence the lifespan of this alternative is expected to be low. Whereas in flat terrain, stormwater is expected to retain in channel for a moderate duration due to low hydraulic capacity and saturate the boundary of the channel then collapse.
Design Aspect	4	General activity and machineries required	Very limited complex activity is required for this alternative, and it mainly rely on excavator for the digging and shaping of the ditches.
Execution time	4	Short duration	Due to the simplicity of the activities required for the construction, and it is constructed on the lining of the road interference of traffic flow is expected not to be strongly influence

Table 4A: Scoring results of alternative 2.

Alternative 2			
Criteria	**Score**	**Rating Description**	**Motivation**
Investment Cost	3	Average easement required, construction and material are expensive	Since this alternative is constructed on the lining of the road, it is expected that the construction of this alternative requires additional space that maybe outside of the parcel boundary from the state (government). Also, the drainage structure expected to be constructed using cast-in place concrete, the labour and material of such is expected to be relatively more expensive comparing to alternative 1.
Maintenance Cost	4	Few maintenances required, and overall operation is average	With the use of concrete as the drainage structure, erosion of the channel is less severe to nil comparing to alternative 1. However, from time to time, maintenance is expected to be carried out in low lying area where sediment may deposit. Moreover, due to the drainage structure are open surface (or channel), the maintenance operation for this alternative is simple and it is less expensive comparing to alternative 3.
Environmental impact	3	Low impact on soil quality, erosion potential and low biodiversity changes	Due to the construction of the drainage structure, such as excavation, large amount of the earth is expected to disturb existing environment. Biodiversity is expected to have slight changes due the changes of land use for the drainage structure.
Structural reliability	5	Very long lifespan and very sturdy	Since the construction is carried using concrete structure, it is expected to have a very long lifespan (up to 50 years), the concrete structures included reinforcement for the stability etc. and no traffic load, the structure is expected to be very study
Design Aspect	2	Specific activities, machinery and equipment required	Complex activity is required for this alternative, this type of structure involves excavation, installing formwork and steel reinforcement for the cast-in-place storm drains. Heavy machinery such as excavator and concrete trucks was expected
Execution time	2	Long duration	Due to the complexity of the activities required for the construction, the duration of the implementation is expected a long time. Also, the storm drains are constructed on the lining of the road interference of traffic flow is expected not to be strongly influence

Table 5A: Scoring results of alternative 3.

Alternative 3			
Criteria	**Score**	**Rating Description**	**Motivation**
Investment Cost	2	Average easement required, construction and material are expensive	Since this alternative is constructed on beneath the road, it is expected that the construction of this alternative doesn't requires additional space since it is within the parcel boundary of the state (government). However, the alternative is expected to be implemented using precast structure, the cost for implementing the drainage structure and the structure itself is expected to be relatively more expensive comparing to alternative 1 and 2.
Maintenance Cost	3	Few maintenances required, and overall operation is average	With the use of precast-concrete as the drainage structure, erosion of the channel is less severe to nil comparing to alternative 1. However, from time to time, maintenance is expected to be carried out in low lying terrain where sediment may deposit. Due to the drainage structure are underground drains, the maintenance operation for this alternative is more complex, resulting the cost of maintenance operation to be more expensive comparing to alternative 1 and 2.
Environmental impact	3	Average impact on soil quality, erosion potential and low biodiversity changes	Due to the construction of the drainage structure, such as excavation, large amount of the earth is expected to disturb existing environment. Since, the drainage structure is constructed underground, the biodiversity of within the surrounding environment might slightly affected only during the construction.
Structural reliability	4	long lifespan and sturdy	Since the construction is carried using precast-concrete structure, it is expected to have a very long lifespan (up to 50 years). However, the drainage structures are constructed beneath the road, it is expected some settlement overtime due to external and traffic load, hence it is considered less sturdy comparing to alternative 2.
Design Aspect	1	Very specific activities, machinery and equipment required	Very limited complex activity is required for this alternative, and it mainly rely on excavator for the digging and shaping of the ditches.
Execution time	1	Very long duration	Due to the complexity of the activities required for the construction, and it is constructed beneath the centre of the road. The interference of traffic flow is expected to be strongly influence. Therefore, the score of this criterion is less favourable comparing to alternative 2 and 1.

7. Final result of the MCA

The final result (also called weighted score) of the MCA is calculated by multiplying the scores obtained from each criterion by their importance level (weight value). The alternative with highest score is the most feasible alternative see Table 7A below for the summary of the weighted scores for the alternatives.

Table 6A: Rated scores from MCA.

Criteria	Alternative 1	Alternative 2	Alternative 3
Investment Cost	4	3	2
Maintenance Cost	2	4	3
Environmental impact	1	3	3
Structural reliability	2	5	4
Design Aspect	4	2	1
Execution time	4	2	1
Sum:	17	19	14

Table 7A: Summary of weighted scores from MCA

Criteria	Alternative 1	Alternative 2	Alternative 3
Investment Cost	1.00	0.75	0.50
Maintenance Cost	0.40	0.80	0.60
Environmental impact	0.20	0.60	0.60
Structural reliability	0.30	0.75	0.60
Design Aspect	0.40	0.20	0.10
Execution time	0.40	0.20	0.10
Sum:	2.70	3.30	2.50

To sum up the MCA results, Alternative 2 has the highest score (for both the rated scoring and weighted scoring) followed by Alternative 1 and then Alternative 3. The area that Alternative 2 scored the most in is structural reliability, due to the fact that with this alternative the structural lifespan of concrete can be up to 50 years. Moreover, this alternative does not need to withstand traffic load whereas in Alternative 3 must, since it is constructed beneath the street profile. This additional load acting on Alternative 3 might expect larger settlement, and with preventative measures the cost of investment will be increased as well, hence the scoring of this criterion for Alternative 2 was more favourable than Alternative 3. Moreover, Alternative 1 is constructed by lining of natural soil or vegetative hence its lifespan was expected the shortest comparing to both Alternative 2 and 3 and scored the least among the alternatives.

The area that Alternative 2 scored less in was both design aspect and execution time, due to the fact that the construction is in concrete, this would require more complex activities involved and also resulting a longer execution time overall. The same case also reflects Alternative 3, but the construction for this requires much more activities and more complex one comparing to Alternative 2, hence scored less comparing to Alternative 2. On the other hand, the activities involved in Alternative 1 are less and the least complex comparing to both Alternative 2 and 3, hence resulting a more favourable score.

8. Sensitivity analysis

A sensitivity analysis was performed for MCA to further evaluate the results and see how much the result (i.e. weighted scores) are affected if the weights given to each of the six criteria are changed but using the same rated score presented in Table 6A. An analysis where these parameters were changed was preformed and two examples are shown in Table 9A and 10A below. The two different scenarios are summarized in **Error! Reference source not found.**.

Table 8A: Description of 2 scenarios that were tested

Scenario Nr.	Criteria						Description
	Investment cost	Maintenance cost	Environmental impact	Structural reliability	Design aspects	Implementation time	
1	16.67	16.67	16.67	16.67	16.67	16.67	All criteria of equal importance (i.e. the perfect world scenario)
2	10	10	10	20	25	25	The design aspect, its structural reliability and execution time is the most important criteria (i.e. if the company want to focus more on the structural aspects)

Table 9A: Scenario 1- All criteria with equal weight distribution.

Criteria	Alternative 1	Alternative 2	Alternative 3
Investment Cost	0.67	0.30	0.33
Maintenance Cost	0.33	0.40	0.50
Environmental impact	0.17	0.30	0.50
Structural reliability	0.33	1.00	0.67
Design Aspect	0.67	0.50	0.17
Execution time	0.67	0.50	0.17
Sum:	2.83	3.00	2.33

When the scenario (nr 1) was compared to the original MCA, the weighted score for Alternative 1 were higher comparing to the original MCA, whereas both alternative 2 and 3 resulted less than the original MCA weighted score. The conclusion is that changing the original weight distribution (investment cost 25%, maintenance cost 20%, environmental impact 20 %, structural reliability 15%, design aspect 10% and execution time 10 %) to 16.67 % for all criteria did not change the main result (i.e. Alternative 2 is still the best alternative).

Table 10A: Scenario 2- Higher weight distribution for design aspect, structural reliability and execution time criteria.

Criteria	Alternative 1	Alternative 2	Alternative 3
Investment Cost	0.40	0.30	0.20
Maintenance Cost	0.20	0.40	0.30
Environmental impact	0.10	0.30	0.30
Structural reliability	0.40	1.00	0.80
Design Aspect	1.00	0.50	0.25
Execution time	1.00	0.50	0.25
Sum:	3.10	3.00	2.10

When the scenario (nr 2) is compared to the original MCA, the weighted score for alternative were higher comparing to the original MCA, whereas both alternative 2 and 3 resulted less than the original MCA weighted score. The conclusion is that changing the original weight distribution (investment cost 25%, maintenance cost 20%, environmental impact 20 %, structural reliability 15%, design aspect 10% and execution time 10 %) to 10 % for investment cost, maintenance cost, environmental impact, 20 % for structural reliability, and 25 % for both design aspect and execution time criteria changes the main results (i.e Alternative 1 is now the best alternative followed by Alternative 2 and 3).

In both scenario (nr 1 and 2) of Alternative 3, the overall weighted score did not increase but instead decreased. This can be explained by the rated score it received, where majority of the criteria scored were relatively low comparing to both Alternative 1 and 2, hence the weighted score (final result) in both scenarios it was not able to match both the Alternative 1 and 2.

To sum up the MCA results, Alternative 2 is indicated to be the best alternative from the MCA together with scenario 1 and fell only slightly in scenario 2. Hence Alternative 2 can be considered as relatively solid (when analysed with the chosen criteria that were selected on the basis of this research) and was selected to use as the stormwater drainage for the Waymouth Hills.

Appendix B. Manual Calculation of Rainfall Runoff & Storm Drains

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1. Rainfall runoff calculation

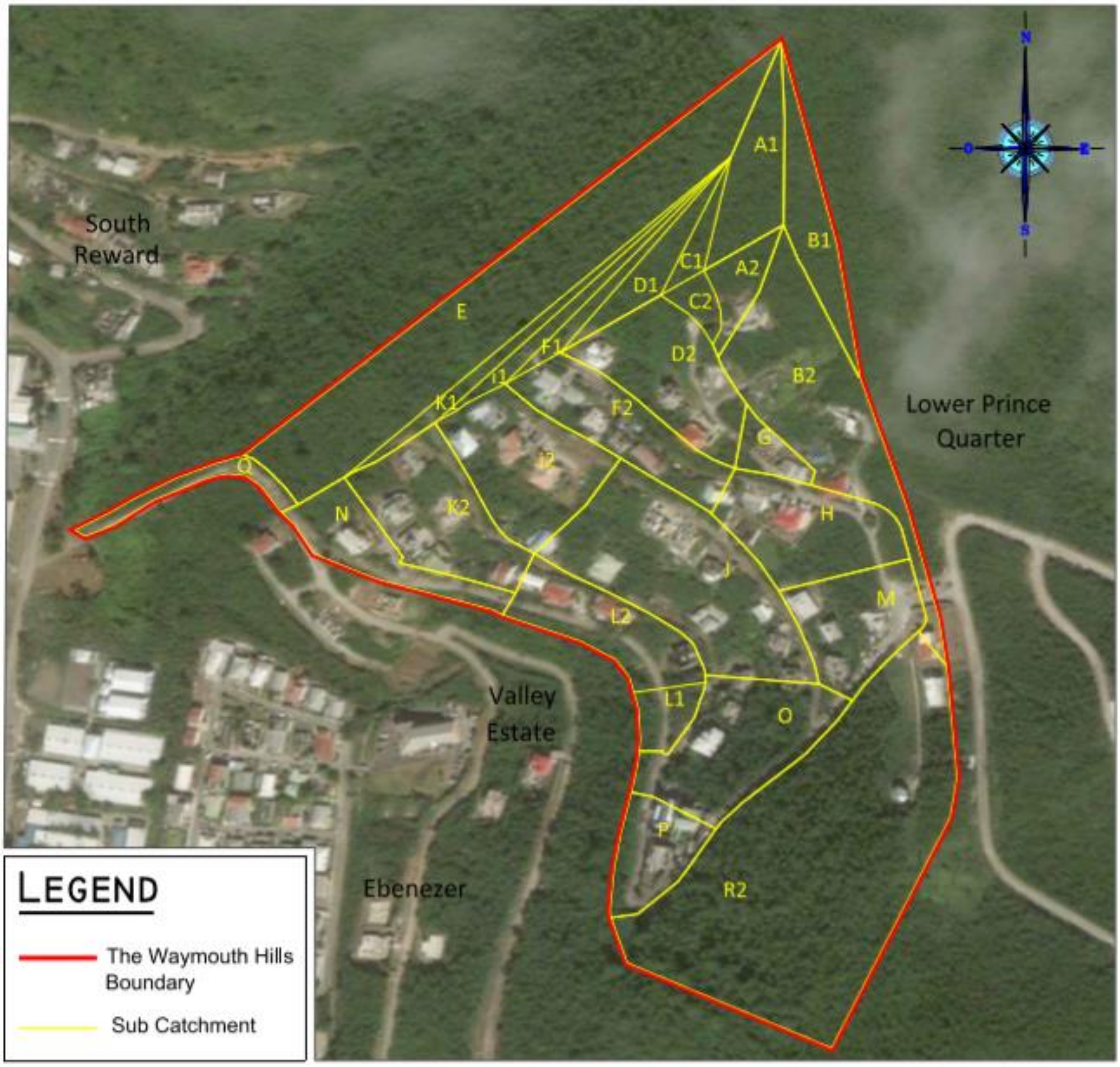

Figure 1B: Aerial view of the Waymouth Hills catchment with delineated sub catchments.

Table 1B: Summary of the subcatchments with runoff coefficient assigned based on their individual land use and terrain characteristic.

Sub catchment	Land use	R (-)	A (m2)	A*R (m2)
A1	Rural/ protected	0.35	4,159	1,456
A2	Residential	0.6	1,842	1,105
B1	Rural/protected	0.35	3,871	1,355
B2	Residential	0.6	11,815	7,089
C1	Rural/protected	0.35	991	347
C2	Residential	0.6	949	569
D1	Rural/protected	0.35	2,306	807
D2	Residential	0.6	7,022	4,213
E	Rural/ protected	0.35	19,987	6,995
F1	Rural/protected	0.35	1,272	445
F2	Residential	0.6	5,941	3,565
G	Residential	0.6	1,404	842
H	Residential	0.6	6,949	4,169
i1	Rural/ protected	0.35	1,629	570
i2	Residential	0.6	7,701	4,621
J	Residential	0.6	13,818	8,291
K1	Rural/ protected	0.35	1,938	678
K2	Residential	0.6	7,300	4,380
L1	Residential	0.6	1,573	944
L2	Residential	0.6	5,590	3,354
M	Residential	0.6	5,725	3,435
N	Residential	0.6	4,294	2,576
O	Residential	0.6	8,453	5,072
P	Residential	0.6	3,634	2,180
Q	Residential	0.6	1,595	957
R2	Residential	0.6	32,444	19,466
Total			**164,202**	**89,483**

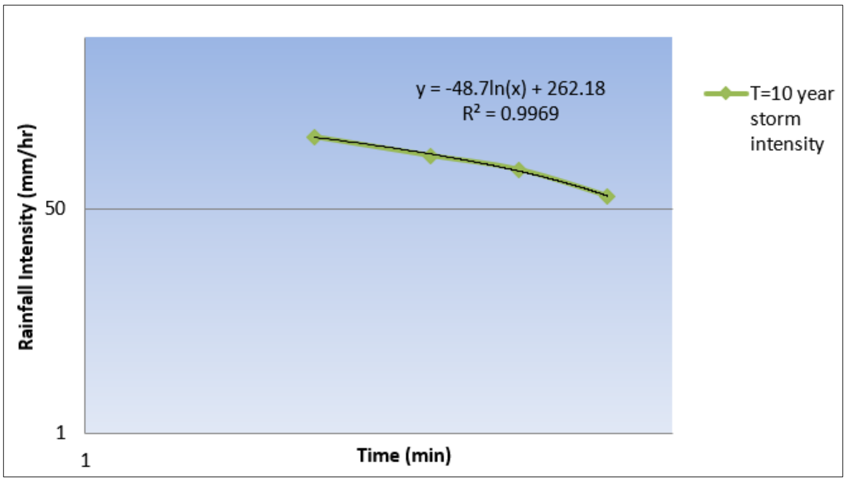

Figure 2B: Linear function of the 10-year storm intensity derived from the IDF curve of St Maarten.

Table 2B: Rainfall runoff discharge into Reach 1.

Runoff Characteristics			
Length	L=	161.3	m
Slope	Sp=	50.0	%

Formula for Sheet Flow:	$T_{ti} = 0.007 (n * L)^{0.8} / (P_2^{0.5} * Sp^{0.4})$		
Sheet Flow			
Roughness Factor	n=	0.400	$s/m^{1/3}$
Length	L=	95.0	m/m
2 Years, 24hrs rainfall	P_2=	82.00	mm
Travel time	Tti=	14.59	min

Formula shallow concentration flow	$T_s = (K_u * k * S_p^{0.5}) / v$		
Shallow concentrated flow			
Length	L=	66.3	m
Intercept coefficient	k=	0.491	-
Velocity	v=	3.47	m/s
Coefficient	K_u=	10.0	-
Travel time	T_s=	0.32	min

Formula channel flow	$T_v = L_s/v$		
Channel flow			
Length of stream	L=	58.9	m
Velocity	v=	4.21	m/s
Travel time	T_v=	0.23	min

Time of concentration	$T_c = T_{ti} + T_s + T_v$		
Reach path		1-3	
Cumulative Runoff area	A*R=	2560.9	m^2
Sheet flow	Tti=	14.6	min
Shallow concentrated flow	Ts=	0.32	
Cumulative channel flow	Tv=	0.23	min
Concentration time	Tc=	15.15	min
Rainfall intensity	i=	129.83	mm/hr
Discharge (Q10-year)	Qmax=	0.09	m^3/s

Table 3B: Rainfall runoff discharge into Reach 2.

Runoff Characteristics			
Length	L=	95.0	m
Slope	Sp=	50.0	%

Formula for Sheet Flow:	$T_{ti} = 0.007\ (n * L)^{0.8} / (P_2^{0.5} * Sp^{0.4})$		
Sheet Flow			
Roughness Factor	n=	0.400	$s/m^{1/3}$
Length	L=	95.0	m/m
2 Years, 24hrs rainfall	P_2=	82.0	mm
Travel time	Tti=	14.59	min

Formula shallow concentration flow	$T_s = (K_u * k * S_p^{0.5}) / v$		
Shallow concentrated flow			
Length	L=	0.0	m
Intercept coefficient	k=	0.491	-
Velocity	v=	3.5	m/s
Coefficient	K_u=	10.0	-
Travel time	T_s=	0.00	min

Formula channel flow	$T_v = L_s/v$		
Channel flow			
Length of stream	L=	58.9	m
Velocity	v=	1.40	m/s
Travel time	T_v=	0.70	min

Time of concentration	$T_c = T_{ti} + T_s + T_v$		
Reach path		2-3	
Cumulative Runoff area	A*R=	916.3	m^2
Sheet flow	Tti=	14.6	min
Shallow concentrated flow	Ts=	0.0	
Cumulative channel flow	Tv=	0.7	min
Concentration time	Tc=	15.29	min
Rainfall intensity	i=	129.35	mm/hr
Discharge (Q10-year)	Qmax=	0.03	m^3/s

Table 4B: Rainfall runoff discharge into Reach 3.

Runoff Characteristics			
Length	L=	245.3	m
Slope	Sp=	50.0	%

Formula for Sheet Flow:	$T_{ti} = 0.007 (n * L)^{0.8} / (P_2^{0.5} * Sp^{0.4})$		
Sheet Flow			
Roughness Factor	n=	0.400	$s/m^{1/3}$
Length	L=	95.0	m/m
2 Years, 24hrs rainfall	P_2=	82.0	mm
Travel time	Tti=	14.59	min

Formula shallow concentration flow	$T_s = (K_u * k * S_p^{0.5}) / v$		
Shallow concentrated flow			
Length	L=	150.3	m
Intercept coefficient	k=	0.491	-
Velocity	v=	3.47	m/s
Coefficient	K_u=	10.0	-
Travel time	T_s=	0.72	min

Formula channel flow	$T_v = L_s/v$		
Channel flow			
Length of stream	L=	169.0	m
Velocity	v=	5.45	m/s
Travel time	T_v=	0.52	min

Time of concentration	$T_c = T_{ti} + T_s + T_v$		
Reach path	3-Lower P. Quarter		
Cumulative Runoff area	A*R=	11921.0	m^2
Sheet flow	Tti=	14.6	min
Shallow concentrated flow	Ts=	0.7	
Cumulative channel flow	Tv=	0.52	min
Concentration time	Tc=	15.83	min
Rainfall intensity	i=	127.67	mm/hr
Discharge (Q10-year)	Qmax=	0.42	m^3/s

Table 5B: Rainfall runoff discharge into Reach 4.

Runoff Characteristics			
Length	L=	58.0	m
Slope	Sp=	46.3	%

Formula for Sheet Flow:	$T_{ti} = 0.007\ (n * L)^{0.8} / (P_2^{0.5} * Sp^{0.4})$		
Sheet Flow			
Roughness Factor	n=	0.400	$s/m^{1/3}$
Length	L=	58.0	m/m
2 Years, 24hrs rainfall	P_2=	82.0	mm
Travel time	Tti=	10.14	min

Formula shallow concentration flow	$T_s = (K_u * k * S_p^{0.5}) / v$		
Shallow concentrated flow			
Length	L=	0.0	m
Intercept coefficient	k=	0.491	-
Velocity	v=	3.34	m/s
Coefficient	K_u=	10.0	-
Travel time	T_s=	0.00	min

Formula channel flow	$T_v = L_s/v$		
Channel flow			
Length of stream	L=	183.0	m
Velocity	v=	1.72	m/s
Travel time	T_v=	1.77	min

Time of concentration	$T_c = T_{ti} + T_s + T_v$		
Reach path		4-20	
Cumulative Runoff area	A*R=	5020.3	m^2
Sheet flow	Tti=	10.1	min
Shallow concentrated flow	Ts=	0.0	
Cumulative channel flow	Tv=	1.77	min
Concentration time	Tc=	11.92	min
Rainfall intensity	i=	141.51	mm/hr
Discharge (Q10-year)	Qmax=	0.20	m^3/s

Table 6B: Rainfall runoff discharge into Reach 5.

Runoff Characteristics			
Length	L=	27.8	m
Slope	Sp=	60.0	%

Formula for Sheet Flow:	$T_{ti} = 0.007\ (n * L)^{0.8} / (P_2^{0.5} * Sp^{0.4})$		
Sheet Flow			
Roughness Factor	n=	0.400	$s/m^{1/3}$
Length	L=	27.8	m/m
2 Years, 24hrs rainfall	P_2=	82.0	mm
Travel time	Tti=	5.08	min

Formula shallow concentration flow	$T_s = (K_u * k * S_p^{0.5}) / v$		
Shallow concentrated flow			
Length	L=	0.0	m
Intercept coefficient	k=	0.491	-
Velocity	v=	0.0	m/s
Coefficient	K_u=	0.0	-
Travel time	T_s=	0.00	min

Formula channel flow	$T_v = L_s/v$		
Channel flow			
Length of stream	L=	74.0	m
Velocity	v=	0.90	m/s
Travel time	T_v=	1.37	min

Time of concentration	$T_c = T_{ti} + T_s + T_v$		
Reach path		5-4	
Cumulative Runoff area	A*R=	842.4	m^2
Sheet flow	Tti=	5.1	min
Shallow concentrated flow	Ts=	0.00	
Cumulative channel flow	Tv=	1.37	min
Concentration time	Tc=	6.44	min
Rainfall intensity	i=	171.44	mm/hr
Discharge (Q10-year)	Qmax=	0.04	m^3/s

Table 7B: Rainfall runoff discharge into Reach 6.

Runoff Characteristics			
Length	L=	183.0	m
Slope	Sp=	47.6	%

Formula for Sheet Flow:	$T_{ti} = 0.007\ (n * L)^{0.8} / (P_2^{0.5} * Sp^{0.4})$		
Sheet Flow			
Roughness Factor	n=	0.400	$s/m^{1/3}$
Length	L=	95.0	m/m
2 Years, 24hrs rainfall	P_2=	82.0	mm
Travel time	Tti=	14.88	min

Formula shallow concentration flow	$T_s = (K_u * k * Sp^{0.5}) / v$		
Shallow concentrated flow			
Length	L=	88.0	m
Intercept coefficient	k=	0.491	-
Velocity	v=	3.39	m/s
Coefficient	K_u=	10.0	-
Travel time	T_s=	0.43	min

Formula channel flow	$T_v = L_s/v$		
Channel flow			
Length of stream	L=	78.0	m
Velocity	v=	3.67	m/s
Travel time	T_v=	0.35	min

Time of concentration	$T_c = T_{ti} + T_s + T_v$		
Reach path		6-20	
Cumulative Runoff area	A*R=	4169.4	m^2
Sheet flow	Tti=	14.9	min
Shallow concentrated flow	Ts=	0.4	
Cumulative channel flow	Tv=	0.35	min
Concentration time	Tc=	15.67	min
Rainfall intensity	i=	128.16	mm/hr
Discharge (Q10-year)	Qmax=	0.15	m^3/s

Table 8B: Rainfall runoff discharge into Reach 7.

Runoff Characteristics			
Length	L=	27.8	m
Slope	Sp=	58.3	%

Formula for Sheet Flow:	$T_{ti} = 0.007 (n * L)^{0.8} / (P_2^{0.5} * Sp^{0.4})$		
Sheet Flow			
Roughness Factor	n=	0.400	$s/m^{1/3}$
Length	L=	27.8	m/m
2 Years, 24hrs rainfall	P_2=	82.0	mm
Travel time	Tti=	5.14	min

Formula shallow concentration flow	$T_s = (K_u * k * Sp^{0.5}) / v$		
Shallow concentrated flow			
Length	L=	0.0	m
Intercept coefficient	k=	0.491	-
Velocity	v=	3.7	m/s
Coefficient	K_u=	10.0	-
Travel time	T_s=	0.00	min

Formula channel flow	$T_v = L_s/v$		
Channel flow			
Length of stream	L=	164.0	m
Velocity	v=	2.12	m/s
Travel time	T_v=	1.29	min

Time of concentration	$T_c = T_{ti} + T_s + T_v$		
Reach path		7-20	
Cumulative Runoff area	A*R=	3564.6	m^2
Sheet flow	Tti=	5.1	min
Shallow concentrated flow	Ts=	0.0	
Cumulative channel flow	Tv=	1.29	min
Concentration time	Tc=	6.43	min
Rainfall intensity	i=	171.57	mm/hr
Discharge (Q10-year)	Qmax=	0.17	m^3/s

Table 9B: Rainfall runoff discharge into Reach 8.

Runoff Characteristics			
Length	L=	70.9	m
Slope	Sp=	58.6	%

Formula for Sheet Flow:	$T_{ti} = 0.007\ (n * L)^{0.8} / (P_2^{0.5} * Sp^{0.4})$		
Sheet Flow			
Roughness Factor	n=	0.400	$s/m^{1/3}$
Length	L=	70.9	m/m
2 Years, 24hrs rainfall	P_2=	82.0	mm
Travel time	Tti=	10.84	min

Formula shallow concentration flow	$T_s = (K_u * k * Sp^{0.5}) / v$		
Shallow concentrated flow			
Length	L=	0.0	m
Intercept coefficient	k=	0.491	-
Velocity	v=	3.76	m/s
Coefficient	K_u=	10.0	-
Travel time	T_s=	0.00	min

Formula channel flow	$T_v = L_s/v$		
Channel flow			
Length of stream	L=	132.0	m
Velocity	v=	4.23	m/s
Travel time	T_v=	0.52	min

Time of concentration	$T_c = T_{ti} + T_s + T_v$		
Reach path		8-17	
Cumulative Runoff area	A*R=	4297.8	m^2
Sheet flow	Tti=	10.8	min
Shallow concentrated flow	Ts=	0.0	
Cumulative channel flow	Tv=	0.52	min
Concentration time	Tc=	11.36	min
Rainfall intensity	i=	143.84	mm/hr
Discharge (Q10-year)	Qmax=	0.17	m^3/s

Table 10B: Rainfall runoff discharge into Reach 9.

Runoff Characteristics			
Length	L=	104.0	m
Slope	Sp=	30.0	%

Formula for Sheet Flow:	$T_{ti} = 0.007 (n * L)^{0.8} / (P_2^{0.5} * Sp^{0.4})$		
Sheet Flow			
Roughness Factor	n=	0.400	$s/m^{1/3}$
Length	L=	95.0	m/m
2 Years, 24hrs rainfall	P_2=	82.0	mm
Travel time	Tti=	17.90	min

Formula shallow concentration flow	$T_s = (K_u * k * Sp^{0.5}) / v$		
Shallow concentrated flow			
Length	L=	9.0	m
Intercept coefficient	k=	0.491	-
Velocity	v=	2.69	m/s
Coefficient	K_u=	10.0	-
Travel time	T_s=	0.06	min

Formula channel flow	$T_v = L_s/v$		
Channel flow			
Length of stream	L=	92.0	m
Velocity	v=	2.99	m/s
Travel time	T_v=	0.51	min

Time of concentration	$T_c = T_{ti} + T_s + T_v$		
Reach path		9-20	
Cumulative Runoff area	A*R=	4703.0	m^2
Sheet flow	Tti=	17.9	min
Shallow concentrated flow	Ts=	0.1	
Cumulative channel flow	Tv=	0.51	min
Concentration time	Tc=	18.47	min
Rainfall intensity	i=	120.16	mm/hr
Discharge (Q10-year)	Qmax=	0.16	m^3/s

Table 11B: Rainfall runoff discharge into Reach 10.

Runoff Characteristics			
Length	L=	127.3	m
Slope	Sp=	47.0	%

Formula for Sheet Flow:	$T_{ti} = 0.007 (n * L)^{0.8} / (P_2^{0.5} * Sp^{0.4})$		
Sheet Flow			
Roughness Factor	n=	0.400	$s/m^{1/3}$
Length	L=	95.0	m/m
2 Years, 24hrs rainfall	P_2=	82.0	mm
Travel time	Tti=	14.96	min

Formula shallow concentration flow	$T_s = (K_u * k * Sp^{0.5}) / v$		
Shallow concentrated flow			
Length	L=	32.3	m
Intercept coefficient	k=	0.491	-
Velocity	v=	3.37	m/s
Coefficient	K_u=	10.0	-
Travel time	T_s=	0.16	min

Formula channel flow	$T_v = L_s/v$		
Channel flow			
Length of stream	L=	273.0	m
Velocity	v=	5.66	m/s
Travel time	T_v=	0.80	min

Time of concentration	$T_c = T_{ti} + T_s + T_v$		
Reach path		10-Ebenezer	
Cumulative Runoff area	A*R=	19466.4	m^2
Sheet flow	Tti=	15.0	min
Shallow concentrated flow	Ts=	0.2	
Cumulative channel flow	Tv=	0.80	min
Concentration time	Tc=	15.92	min
Rainfall intensity	i=	127.39	mm/hr
Discharge (Q10-year)	Qmax=	0.69	m^3/s

Table 12B: Rainfall runoff discharge into Reach 11.

Runoff Characteristics			
Length	L=	70.9	m
Slope	Sp=	58.6	%

Formula for Sheet Flow:	$T_{ti} = 0.007\ (n * L)^{0.8} / (P_2^{0.5} * Sp^{0.4})$		
Sheet Flow			
Roughness Factor	n=	0.400	$s/m^{1/3}$
Length	L=	70.9	m/m
2 Years, 24hrs rainfall	P_2=	82.0	mm
Travel time	Tti=	10.84	min

Formula shallow concentration flow	$T_s = (K_u * k * S_p^{0.5}) / v$		
Shallow concentrated flow			
Length	L=	0.0	m
Intercept coefficient	k=	0.491	-
Velocity	v=	3.76	m/s
Coefficient	K_u=	10.0	-
Travel time	T_s=	0.00	min

Formula channel flow	$T_v = L_s/v$		
Channel flow			
Length of stream	L=	114.0	m
Velocity	v=	4.60	m/s
Travel time	T_v=	0.41	min

Time of concentration	$T_c = T_{ti} + T_s + T_v$		
Reach path		11-22	
Cumulative Runoff area	A*R=	4620.6	m^2
Sheet flow	Tti=	10.8	min
Shallow concentrated flow	Ts=	0.0	
Cumulative channel flow	Tv=	0.41	min
Concentration time	Tc=	11.25	min
Rainfall intensity	i=	144.30	mm/hr
Discharge (Q10-year)	Qmax=	0.19	m^3/s

Table 13B: Rainfall runoff discharge into Reach 12.

Runoff Characteristics			
Length	L=	95.0	m
Slope	Sp=	53.2	%

Formula for Sheet Flow:	$T_{ti} = 0.007\ (n * L)^{0.8} / (P_2^{0.5} * Sp^{0.4})$		
Sheet Flow			
Roughness Factor	n=	0.400	$s/m^{1/3}$
Length	L=	95.0	m/m
2 Years, 24hrs rainfall	P_2=	82.0	mm
Travel time	Tti=	14.24	min

Formula shallow concentration flow	$T_s = (K_u * k * S_p^{0.5}) / v$		
Shallow concentrated flow			
Length	L=	0.0	m
Intercept coefficient	k=	0.491	-
Velocity	v=	3.6	m/s
Coefficient	K_u=	10.0	-
Travel time	T_s=	0.00	min

Formula channel flow	$T_v = L_s/v$		
Channel flow			
Length of stream	L=	134.0	m
Velocity	v=	5.17	m/s
Travel time	T_v=	0.43	min

Time of concentration	$T_c = T_{ti} + T_s + T_v$		
Reach path		12-21	
Cumulative Runoff area	A*R=	15895.2	m^2
Sheet flow	Tti=	14.2	min
Shallow concentrated flow	Ts=	0.0	
Cumulative channel flow	Tv=	0.43	min
Concentration time	Tc=	14.67	min
Rainfall intensity	i=	131.38	mm/hr
Discharge (Q10-year)	Qmax=	0.58	m^3/s

Table 14B: Rainfall runoff discharge into Reach 14.

Runoff Characteristics			
Length	L=	99.0	m
Slope	Sp=	47.8	%

Formula for Sheet Flow:	$T_{ti} = 0.007\ (n * L)^{0.8} / (P_2^{0.5} * Sp^{0.4})$		
Sheet Flow			
Roughness Factor	n=	0.400	$s/m^{1/3}$
Length	L=	95.0	m/m
2 Years, 24hrs rainfall	P_2=	82.0	mm
Travel time	Tti=	14.86	min

Formula shallow concentration flow	$T_s = (K_u * k * Sp^{0.5}) / v$		
Shallow concentrated flow			
Length	L=	4.0	m
Intercept coefficient	k=	0.491	-
Velocity	v=	3.39	m/s
Coefficient	K_u=	10.0	-
Travel time	T_s=	0.02	min

Formula channel flow	$T_v = L_s/v$		
Channel flow			
Length of stream	L=	92.0	m
Velocity	v=	4.39	m/s
Travel time	T_v=	0.35	min

Time of concentration	$T_c = T_{ti} + T_s + T_v$		
Reach path		13-15	
Cumulative Runoff area	A*R=	9217.2	m^2
Sheet flow	Tti=	-	min
Shallow concentrated flow	Ts=	-	
Cumulative channel flow	Tv=	19.04	min
Concentration time	Tc=	19.04	min
Rainfall intensity	i=	118.69	mm/hr
Discharge (Q10-year)	Qmax=	0.30	m^3/s

Table 15B: Rainfall runoff discharge into Reach 14.

Runoff Characteristics			
Length	L=	54.0	m
Slope	Sp=	63.4	%

Formula for Sheet Flow:	$T_{ti} = 0.007\ (n\ {*}L)^{0.8} / (P_2^{0.5} * Sp^{0.4})$		
Sheet Flow			
Roughness Factor	n=	`	$s/m^{1/3}$
Length	L=	54.0	m/m
2 Years, 24hrs rainfall	P_2=	82.0	mm
Travel time	Tti=	8.45	min

Formula shallow concentration flow	$T_s = (K_u * k * Sp^{0.5}) / v$		
Shallow concentrated flow			
Length	L=	0.0	m
Intercept coefficient	k=	0.491	-
Velocity	v=	3.9	m/s
Coefficient	K_u=	10.0	-
Travel time	T_s=	0.00	min

Formula channel flow	$T_v = L_s/v$		
Channel flow			
Length of stream	L=	110.0	m
Velocity	v=	5.5	m/s
Travel time	T_v=	0.33	min

Time of concentration	$T_c = T_{ti} + T_s + T_v$		
Reach path		14-15	
Cumulative Runoff area	A*R=	7252.2	m^2
Sheet flow	Tti=	8.4	min
Shallow concentrated flow	Ts=	0.0	
Cumulative channel flow	Tv=	0.33	min
Concentration time	Tc=	8.78	min
Rainfall intensity	i=	156.39	mm/hr
Discharge (Q10-year)	Qmax=	0.32	m^3/s

Table 16B: Rainfall runoff discharge into Reach 15.

Runoff Characteristics			
Length	L=	45.4	m
Slope	Sp=	46.7	%

Formula for Sheet Flow:	$T_{ti} = 0.007\ (n * L)^{0.8} / (P_2^{0.5} * Sp^{0.4})$		
Sheet Flow			
Roughness Factor	n=	0.400	$s/m^{1/3}$
Length	L=	45.4	m/m
2 Years, 24hrs rainfall	P_2=	82.0	mm
Travel time	Tti=	8.31	min

Formula shallow concentration flow	$T_s = (K_u * k * S_p^{0.5}) / v$		
Shallow concentrated flow			
Length	L=	0.0	m
Intercept coefficient	k=	0.491	-
Velocity	v=	3.36	m/s
Coefficient	K_u=	10.0	-
Travel time	T_s=	0.00	min

Formula channel flow	$T_v = L_s/v$		
Channel flow			
Length of stream	L=	47.0	m
Velocity	v=	5.46	m/s
Travel time	T_v=	0.14	min

Time of concentration	$T_c = T_{ti} + T_s + T_v$		
Reach path	15-Valley Estate		
Cumulative Runoff area	A*R=	14856.6	m^2
Sheet flow	Tti=	-	min
Shallow concentrated flow	Ts=	-	
Cumulative channel flow	Tv=	19.18	min
Concentration time	Tc=	19.18	min
Rainfall intensity	i=	118.32	mm/hr
Discharge (Q10-year)	Qmax=	0.49	m^3/s

Table 17B: Rainfall runoff discharge into Reach 16.

Runoff Characteristics			
Length	L=	66.9	m
Slope	Sp=	50.0	%

Formula for Sheet Flow:	$T_{ti} = 0.007\ (n * L)^{0.8} / (P_2^{0.5} * Sp^{0.4})$		
Sheet Flow			
Roughness Factor	n=	0.400	$s/m^{1/3}$
Length	L=	66.9	m/m
2 Years, 24hrs rainfall	P_2=	82.0	mm
Travel time	Tti=	11.02	min

Formula shallow concentration flow	$T_s = (K_u * k * S_p^{0.5}) / v$		
Shallow concentrated flow			
Length	L=	0.0	m
Intercept coefficient	k=	0.491	-
Velocity	v=	3.5	m/s
Coefficient	K_u=	10.0	-
Travel time	T_s=	0.00	min

Formula channel flow	$T_v = L_s/v$		
Channel flow			
Length of stream	L=	67.0	m
Velocity	v=	3.24	m/s
Travel time	T_v=	0.35	min

Time of concentration	$T_c = T_{ti} + T_s + T_v$		
Reach path		16-17	
Cumulative Runoff area	A*R=	4380.0	m^2
Sheet flow	Tti=	11.0	min
Shallow concentrated flow	Ts=	0.0	
Cumulative channel flow	Tv=	0.35	min
Concentration time	Tc=	11.37	min
Rainfall intensity	i=	143.79	mm/hr
Discharge (Q10-year)	Qmax=	0.17	m^3/s

Table 18B: Rainfall runoff discharge into Reach 17.

Runoff Characteristics			
Length	L=	30.5	m
Slope	Sp=	50.0	%

Formula for Sheet Flow:	$T_{ti} = 0.007\ (n * L)^{0.8} / (P_2^{0.5} * Sp^{0.4})$		
Sheet Flow			
Roughness Factor	n=	0.400	$s/m^{1/3}$
Length	L=	30.5	m/m
2 Years, 24hrs rainfall	P_2=	82.0	mm
Travel time	Tti=	5.88	min

Formula shallow concentration flow	$T_s = (K_u * k * S_p^{0.5}) / v$		
Shallow concentrated flow			
Length	L=	0.0	m
Intercept coefficient	k=	0.491	-
Velocity	v=	3.5	m/s
Coefficient	K_u=	10.0	-
Travel time	T_s=	0.00	min

Formula channel flow	$T_v = L_s/v$		
Channel flow			
Length of stream	L=	163.0	m
Velocity	v=	5.9	m/s
Travel time	T_v=	0.46	min

Time of concentration	$T_c = T_{ti} + T_s + T_v$		
Reach path		17-18.	
Cumulative Runoff area	A*R=	24165.6	m^2
Sheet flow	Tti=	-	min
Shallow concentrated flow	Ts=	-	
Cumulative channel flow	Tv=	15.20	min
Concentration time	Tc=	15.20	min
Rainfall intensity	i=	129.64	mm/hr
Discharge (Q10-year)	Qmax=	0.87	m^3/s

Table 19B: Rainfall runoff discharge into Reach 18.

Runoff Characteristics			
Length	L=	10.9	m
Slope	Sp=	9.7	%

Formula for Sheet Flow:	$T_{ti} = 0.007 (n * L)^{0.8} / (P_2^{0.5} * Sp^{0.4})$		
Sheet Flow			
Roughness Factor	n=	0.400	$s/m^{1/3}$
Length	L=	10.9	m/m
2 Years, 24hrs rainfall	P_2=	82.0	mm
Travel time	Tti=	4.98	min

Formula shallow concentration flow	$T_s = (K_u * k * S_p^{0.5}) / v$		
Shallow concentrated flow			
Length	L=	0.0	m
Intercept coefficient	k=	0.491	-
Velocity	v=	1.53	m/s
Coefficient	K_u=	10.0	-
Travel time	T_s=	0.00	min

Formula channel flow	$T_v = L_s/v$		
Channel flow			
Length of stream	L=	170.0	m
Velocity	v=	7.30	m/s
Travel time	T_v=	0.39	min

Time of concentration	$T_c = T_{ti} + T_s + T_v$		
Reach path	18-Cul de Sac stream		
Cumulative Runoff area	A*R=	42396.6	m^2
Sheet flow	Tti=	-	min
Shallow concentrated flow	Ts=	-	
Cumulative channel flow	Tv=	15.59	min
Concentration time	Tc=	15.59	min
Rainfall intensity	i=	128.42	mm/hr
Discharge (Q10-year)	Qmax=	1.51	m^3/s

Table 20B: Rainfall runoff discharge into Reach 20.

Runoff Characteristics			
Length	L=	256.3	m
Slope	Sp=	52.3	%

Formula for Sheet Flow:	$T_{ti} = 0.007 (n *L)^{0.8} / (P_2^{0.5} * Sp^{0.4})$		
Sheet Flow			
Roughness Factor	n=	0.400	$s/m^{1/3}$
Length	L=	95.0	m/m
2 Years, 24hrs rainfall	P_2=	82	mm
Travel time	Tti=	14.33	min

Formula shallow concentration flow	$T_s = (K_u * k * S_p^{0.5}) / v$		
Shallow concentrated flow			
Length	L=	130.0	m
Intercept coefficient	k=	0.491	-
Velocity	v=	3.55	m/s
Coefficient	K_u=	10.0	-
Travel time	T_s=	0.61	min

Formula channel flow	$T_v = L_s/v$		
Channel flow			
Length of stream	L=	214.0	m
Velocity	v=	7.9	m/s
Travel time	T_v=	0.45	min

Time of concentration	$T_c = T_{ti} + T_s + T_v$		
Reach path		20-18	
Cumulative Runoff area	A*R=	17274.0	m^2
Sheet flow	Tti=	14.3	min
Shallow concentrated flow	Ts=	0.6	
Cumulative channel flow	Tv=	0.45	min
Concentration time	Tc=	15.40	min
Rainfall intensity	i=	129.03	mm/hr
Discharge (Q10-year)	Qmax=	0.62	m^3/s

Table 21B: Rainfall runoff discharge into Reach 21.

Runoff Characteristics			
Formula channel flow	$T_v = L_s/v$		
Channel flow			
Length of stream	L=	36.0	m
Velocity	v=	7.93	m/s
Travel time	T_v=	0.08	min

Time of concentration	$T_c = T_{ti} + T_s + T_v$		
Reach path		21-17	
Cumulative Runoff area	A*R=	12911.4	m^2
Sheet flow	Tti=	-	min
Shallow concentrated flow	Ts=	-	
Cumulative channel flow	Tv=	14.74	min
Concentration time	Tc=	14.74	min
Rainfall intensity	i=	131.13	mm/hr
Discharge (Q10-year)	Qmax=	0.47	m^3/s

Table 22B: Rainfall runoff discharge into Reach 22.

Runoff Characteristics			
Formula channel flow	$T_v = L_s/v$		
Channel flow			
Length of stream	L=	74.0	m
Velocity	v=	5.64	m/s
Travel time	T_v=	0.22	min

Time of concentration	$T_c = T_{ti} + T_s + T_v$		
Reach path		22-12	
Cumulative Runoff area	A*R=	7604.4	m^2
Sheet flow	Tti=	-	min
Shallow concentrated flow	Ts=	-	
Cumulative channel flow	Tv=	18.69	min
Concentration time	Tc=	18.69	min
Rainfall intensity	i=	119.59	mm/hr
Discharge (Q10-year)	Qmax=	0.25	m^3/s

Table 23B: Summary of the rainfall runoff discharge into the Reaches in the drainage network.

Concentration Time							Rainfall Runoff	
Reach	Runoff Area A*R	Reach path	Cumulative runoff area	Time of entry	Time of flow	Conc. Time	Rain intensity	Peak Discharge
	[m2]		[m2]	[min]	[min]	[min]	mm/hr	[m3/s]
1	2,561	1-3	2,561	14.91	0.23	15.15	129.8	0.09
2	916	2-3	916	14.59	0.70	15.29	129.4	0.03
3	8,444	3-Lower P. Quarter	11,921	15.32	0.52	15.83	127.7	0.42
4	5,020	4-20	5,020	10.14	1.77	11.92	141.5	0.20
5	842	5-4	842	5.08	1.37	6.44	171.4	0.04
6	4,169	6-20	4,169	15.32	0.35	15.67	128.2	0.15
7	3,565	7-20	3,565	5.14	1.29	6.43	171.6	0.17
8	3,354	8-17	4,298	10.84	0.52	11.36	143.8	0.17
9	3,435	9-20	4,703	17.96	0.51	18.47	120.2	0.16
10	19,466	10- Ebenezer	19,466	15.12	0.80	15.92	127.4	0.69
11	4,621	11-22	4,621	10.84	0.41	11.25	144.3	0.19
12	8,291	12-21	15,895	14.24	0.43	14.67	131.4	0.58
13	5,072	13-15	9,217	18.69	0.35	19.04	118.7	0.30
14	2,180	14-15	7,252	8.45	0.33	8.78	156.4	0.32
15	944	15-Valley Estate	14,857	19.04	0.14	19.18	118.3	0.49
16	4,380	16-17	4,380	11.02	0.35	11.37	143.8	0.17
17	2,576	17-18.	24,166	14.74	0.46	15.20	129.6	0.87
18	957	18-Cul de Sac stream	42,397	15.20	0.39	15.59	128.4	1.51
20	-	20-18	17,274	14.79	0.45	15.24	129.5	0.62
21	-	21-17	12,911	14.67	0.08	14.74	131.1	0.47
22	-	22-12	7,604	18.47	0.22	18.69	119.6	0.25

2. Storm drain calculation

Table 24B: Storm drain calculation for Reach 1.

Storm drain Characteristics				
Longitudinal Slope	So =	27.0%	m/m	
Formula for hydraulic conveyance capacity of street:	$Q = K / n * Sx^{1.67} * T^{2.67} * \sqrt{So}$			
Road + side gutter				
Total Width Road (incl. gutter)	Wd =	4.50	m	
Total Width Gutter	Wg =	0.30	m	
Side flow width (Max)	Tx =	2.10	m	
Gutter flow width	W =	0.30	m	
Total water spread width on street	T =	2.40	m	
Roughness Factor	n =	0.014	s/m1/3	
Street transverse slope	Sx =	2.0%	m/m	
Extra capacity due depth standard gutter	Qextra	0.03	m3/s	
Drain Capacity	Q =	0.24	m3/s	SUFFICIENT
Velocity	v =	4.21	m/s	SUFFICIENT

Table 25B: Storm drain calculation for Reach 1.

Storm drain Characteristics				
Longitudinal Slope	So =	3.0%	m/m	
Formula for hydraulic conveyance capacity of street:	$Q = K / n * Sx^{1.67} * T^{2.67} * \sqrt{So}$			
Road + side gutter				
Total Width Road (incl. gutter)	Wd =	4.50	m	
Total Width Gutter	Wg =	0.30	m	
Side flow width (Max)	Tx =	2.10	m	
Gutter flow width	W =	0.30	m	
Total water spread width on street	T =	2.40	m	
Roughness Factor	n =	0.014	s/m1/3	
Street transverse slope	Sx =	2.0%	m/m	
Extra capacity due depth standard gutter	Qextra	0.01	m3/s	
Drain Capacity	Q =	0.08	m3/s	SUFFICIENT
Velocity	v =	1.40	m/s	SUFFICIENT

Table 26B: Storm drain calculation for Reach 3

Storm drain Characteristics				
Longitudinal Slope	So =	16.0%	m/m	
Formula for hydraulic conveyance capacity of street:	$Q = K / n * Sx^{1.67} * T^{2.67} * \sqrt{So}$			
Road + side gutter				
Total Width Road (incl. gutter)	Wd =	4.50	m	
Total Width Gutter	Wg =	0.30	m	
Side flow width (Max)	Tx =	2.10	m	
Gutter flow width	W =	0.30	m	
Total water spread width on street	T =	2.40	m	
Roughness Factor	n =	0.014	s/m1/3	
Street transverse slope	Sx =	2.0%	m/m	
Extra capacity due depth standard gutter	Qextra	0.02	m3/s	
Drain Capacity	Q =	0.19	m3/s	NOT SUFFICIENT
Velocity	v =	3.24	m/s	SUFFICIENT

Formula for Manning:	$Q = (A * R^{2/3} * \sqrt{So}) / n$			
U-gutter				
height	H=	0.30	m	
width	B=	0.40	m	
slope	So=	16.0%	m/m	
Roughness Factor	n =	0.016	s/m1/3	
Area	A =	0.12	m2	
Actual flow				
Discharge (Q10-year)	Q =	0.42	m3/s	
Flow height	Ha =	0.21	m	
69 % Flow Area	Aa =	0.083	m2	
69 % Flow Outline	Oa =	0.81	m	
Hydraulic Beam	R =	0.10	m	
Actual Velocity	v =	5.45	m/s	SUFFICIENT
Maximum Allowed flow				
75% Area	A75% =	0.09	m^2	
75% Outline	O75% =	0.85	m	
Hydraulic Beam	R =	0.11	m	
Drain Capacity	Q =	0.50	m^3/s	SUFFICIENT
Velocity	v =	5.59	m/s	

Table 27B: Storm drain calculation for Reach 4.

Storm drain Characteristics				
Longitudinal Slope	So =	1.0%	m/m	

Formula for hydraulic conveyance capacity of street:	$Q = K / n * Sx^{1.67} * T^{2.67} * \sqrt{So}$			
Road + side gutter				
Total Width Road (incl. gutter)	Wd =	4.50	m	
Total Width Gutter	Wg =	0.30	m	
Side flow width (Max)	Tx =	2.10	m	
Gutter flow width	W =	0.30	m	
Total water spread width on street	T =	2.40	m	
Roughness Factor	n =	0.014	s/m1/3	
Street transverse slope	Sx =	2.0%	m/m	
Extra capacity due depth standard gutter	Qextra	0.01	m3/s	
Drain Capacity	Q =	0.05	m3/s	**NOT SUFFICIENT**
Velocity	v =	0.81	m/s	**SUFFICIENT**

Formula for Manning:	$Q = (A * R^{2/3} * \sqrt{So}) / n$			
U-gutter				
height	H=	0.45	m	
width	B=	0.40	m	
slope	So=	1.0%	m/m	
Roughness Factor	n =	0.014	s/m1/3	
Area	A =	0.18	m2	
Actual flow				
Discharge (Q10-year)	Q =	0.20	m3/s	
Flow height	Ha =	0.29	m	
64 % Flow Area	Aa =	0.115	m2	
64 % Flow Outline	Oa =	0.98	m	
Hydraulic Beam	R =	0.12	m	
Actual Velocity	v =	1.72	m/s	**SUFFICIENT**
Maximum Allowed flow				
75% Area	A75% =	0.14	m^2	
75% Outline	O75% =	1.08	m	
Hydraulic Beam	R =	0.13	m	
Drain Capacity	Q =	0.24	m^3/s	**SUFFICIENT**
Velocity	v =	1.79	m/s	

Table 28B: Storm drain calculation for Reach 5.

Storm drain Characteristics				
Longitudinal Slope	So =	1.0%	m/m	
Formula for hydraulic conveyance capacity of street:	$Q = K / n * Sx^{1.67} * T^{2.67} * \sqrt{So}$			
Road + side gutter				
Total Width Road (incl. gutter)	Wd =	4.50	m	
Total Width Gutter	Wg =	0.30	m	
Side flow width (Max)	Tx =	2.10	m	
Gutter flow width	W =	0.30	m	
Total water spread width on street	T =	2.40	m	
Roughness Factor	n =	0.014	s/m1/3	
Street transverse slope	Sx =	2.5%	m/m	
Extra capacity due depth standard gutter	Qextra	0.01	m3/s	
Drain Capacity	Q =	0.06	m3/s	**SUFFICIENT**
Velocity	v =	0.90	m/s	**SUFFICIENT**

Table 29B: Storm drain calculation for Reach 6.

Storm drain Characteristics				
Longitudinal Slope	So =	9.0%	m/m	
Formula for hydraulic conveyance capacity of street:	$Q = K / n * Sx^{1.67} * T^{2.67} * \sqrt{So}$			
Road + side gutter				
Total Width Road (incl. gutter)	Wd =	3.00	m	
Total Width Gutter	Wg =	0.30	m	
Side flow width (Max)	Tx =	1.35	m	
Gutter flow width	W =	0.30	m	
Total water spread width on street	T =	1.65	m	
Roughness Factor	n =	0.014	s/m1/3	
Street transverse slope	Sx =	2.0%	m/m	
Extra capacity due depth standard gutter	Qextra	0.01	m3/s	
Drain Capacity	Q =	0.06	m3/s	NOT SUFFICIENT
Velocity	v =	2.32	m/s	SUFFICIENT
Formula for Manning:	$Q = (A * R^{2/3} * \sqrt{So})$			
U-gutter				
height	H=	0.30	m	
width	B=	0.30	m	
slope	So=	9.0%	m/m	
Roughness Factor	n =	0.014	s/m1/3	
Area	A =	0.09	m2	
Actual flow				
Discharge (Q10-year)	Q =	0.15	m3/s	
Flow height	Ha =	0.134	m	
44.7 % Flow Area	Aa =	0.040	m2	
44.7 % Flow Outline	Oa =	0.57	m	
Hydraulic Beam	R =	0.07	m	
Actual Velocity	v =	3.67	m/s	SUFFICIENT
Maximum Allowed flow				
75% Area	A75% =	0.07	m^2	
75% Outline	O75% =	0.75	m	
Hydraulic Beam	R =	0.09	m	
Drain Capacity	Q =	0.29	m^3/s	SUFFICIENT
Velocity	v =	4.30	m/s	

Table 30B: Storm drain calculation for Reach 7.

Storm drain Characteristics				
Longitudinal Slope	So =	2.0%	m/m	
Formula for hydraulic conveyance capacity of street:	$Q = K / n * Sx^{1.67} * T^{2.67} * \sqrt{So}$			
Road + side gutter				
Total Width Road (incl. gutter)	Wd =	4.50	m	
Total Width Gutter	Wg =	0.30	m	
Side flow width (Max)	Tx =	2.10	m	
Gutter flow width	W =	0.30	m	
Total water spread width on street	T =	2.40	m	
Roughness Factor	n =	0.014	s/m1/3	
Street transverse slope	Sx =	2.0%	m/m	
Extra capacity due depth standard gutter	Qextra	0.01	m3/s	
Drain Capacity	Q =	0.07	m3/s	NOT SUFFICIENT
Velocity	v =	1.15	m/s	SUFFICIENT
Formula for Manning:	$Q = (A * R^{2/3} * \sqrt{So})$			
U-gutter				
height	H=	0.50	m	
width	B=	0.30	m	
slope	So=	2.0%	m/m	
Roughness Factor	n =	0.014	s/m1/3	
Area	A =	0.15	m2	
Actual flow				
Discharge (Q10-year)	Q =	0.17	m3/s	
Flow height	Ha =	0.266	m	
53.2 % Flow Area	Aa =	0.080	m2	
53.2 % Flow Outline	Oa =	0.83	m	
Hydraulic Beam	R =	0.10	m	
Actual Velocity	v =	2.12	m/s	SUFFICIENT
Maximum Allowed flow				
75% Area	A75% =	0.11	m^2	
75% Outline	O75% =	1.05	m	
Hydraulic Beam	R =	0.11	m	
Drain Capacity	Q =	0.26	m^3/s	SUFFICIENT
Velocity	v =	2.28	m/s	

Table 31B: Storm drain calculation for Reach 8.

Storm drain Characteristics				
Longitudinal Slope	So =	12.0%	m/m	

Formula for hydraulic conveyance capacity of street:	$Q = K / n * Sx^{1.67} * T^{2.67} * \sqrt{So}$			
Road + side gutter				
Total Width Road (incl. gutter)	Wd =	7.50	m	
Total Width Gutter	Wg =	0.30	m	
Side flow width (Max)	Tx =	3.60	m	
Gutter flow width	W =	0.30	m	
Total water spread width on street	T =	3.90	m	
Roughness Factor	n =	0.016	s/m1/3	
Street transverse slope	Sx =	2.0%	m/m	
Extra capacity due depth standard gutter	Qextra	0.02	m3/s	
Drain Capacity	Q =	0.47	m3/s	SUFFICIENT
Velocity	v =	3.07	m/s	SUFFICIENT

Formula for Manning:	$Q = (A * R^{2/3} * \sqrt{So})$			
U-gutter				
height	H=	0.30	m	
width	B=	0.30	m	
slope	So=	12.0%	m/m	
Roughness Factor	n =	0.014	s/m1/3	
Area	A =	0.09	m2	
Actual flow				
Discharge (Q10-year)	Q =	0.17	m3/s	
Flow height	Ha =	0.134	m	
44.7 % Flow Area	Aa =	0.040	m2	
44.7 % Flow Outline	Oa =	0.57	m	
Hydraulic Beam	R =	0.07	m	
Actual Velocity	v =	4.23	m/s	SUFFICIENT
Maximum Allowed flow				
75% Area	A75% =	0.07	m^2	
75% Outline	O75% =	0.75	m	
Hydraulic Beam	R =	0.09	m	
Drain Capacity	Q =	0.34	m^3/s	SUFFICIENT
Velocity	v =	4.97	m/s	

Table 32B: Storm drain calculation for Reach 9.

Storm drain Characteristics				
Longitudinal Slope	So =	11.0%	m/m	
Formula for hydraulic conveyance capacity of street:	$Q = K / n * Sx^{1.67} * T^{2.67} * \sqrt{So}$			
Road + side gutter				
Total Width Road (incl. gutter)	Wd =	4.50	m	
Total Width Gutter	Wg =	0.30	m	
Side flow width (Max)	Tx =	2.10	m	
Gutter flow width	W =	0.30	m	
Total water spread width on street	T =	2.40	m	
Roughness Factor	n =	0.014	s/m1/3	
Street transverse slope	Sx =	2.5%	m/m	
Extra capacity due depth standard gutter	Qextra	0.02	m3/s	
Drain Capacity	Q =	0.22	m3/s	**SUFFICIENT**
Velocity	v =	2.99	m/s	**SUFFICIENT**

Table 33B: Storm drain calculation for Reach 10.

Storm drain Characteristics				
Longitudinal Slope	So =	12.0%	m/m	

Formula for hydraulic conveyance capacity of street:	$Q = K / n * Sx^{1.67} * T^{2.67} * \sqrt{So}$			
Road + side gutter				
Total Width Road (incl. gutter)	Wd =	7.50	m	
Total Width Gutter	Wg =	0.30	m	
Side flow width (Max)	Tx =	3.60	m	
Gutter flow width	W =	0.30	m	
Total water spread width on street	T =	3.90	m	
Roughness Factor	n =	0.016	s/m1/3	
Street transverse slope	Sx =	2.0%	m/m	
Extra capacity due depth standard gutter	Qextra	0.02	m3/s	
Drain Capacity	Q =	0.47	m3/s	NOT SUFFICIENT
Velocity	v =	3.07	m/s	SUFFICIENT

Formula for Manning:	$Q = (A * R^{2/3} * \sqrt{So})$			
U-gutter				
height	H=	0.75	m	
width	B=	0.30	m	
slope	So=	12.0%	m/m	
Roughness Factor	n =	0.014	s/m1/3	
Area	A =	0.23	m2	
Actual flow				
Discharge (Q10-year)	Q =	0.69	m3/s	
Flow height	Ha =	0.41	m	
54.0 % Flow Area	Aa =	0.12	m2	
54.0 % Flow Outline	Oa =	1.11	m	
Hydraulic Beam	R =	0.11	m	
Actual Velocity	v =	5.66	m/s	SUFFICIENT
Maximum Allowed flow				
75% Area	A75% =	0.17	m^2	
75% Outline	O75% =	1.43	m	
Hydraulic Beam	R =	0.12	m	
Drain Capacity	Q =	1.01	m^3/s	SUFFICIENT
Velocity	v =	5.96	m/s	

Table 34B: Storm drain calculation for Reach 11.

Storm drain Characteristics				
Longitudinal Slope	So =	14.0%	m/m	

Formula for hydraulic conveyance capacity of street:	$Q = K / n * Sx^{1.67} * T^{2.67} * \sqrt{So}$			
Road + side gutter				
Total Width Road (incl. gutter)	Wd =	3.50	m	
Total Width Gutter	Wg =	0.30	m	
Side flow width (Max)	Tx =	1.60	m	
Gutter flow width	W =	0.30	m	
Total water spread width on street	T =	1.90	m	
Roughness Factor	n =	0.014	s/m1/3	
Street transverse slope	Sx =	2.0%	m/m	
Extra capacity due depth standard gutter	Qextra	0.02	m3/s	
Drain Capacity	Q =	0.10	m3/s	NOT SUFFICIENT
Velocity	v =	2.89	m/s	SUFFICIENT

Formula for Manning:	$Q = (A * R^{2/3} * \sqrt{So})$			
U-gutter				
height	H=	0.75	m	
width	B=	0.30	m	
slope	So=	12.0%	m/m	
Roughness Factor	n =	0.014	s/m1/3	
Area	A =	0.23	m2	
Actual flow				
Discharge (Q10-year)	Q =	0.19	m3/s	
Flow height	Ha =	0.17	m	
66.8 % Flow Area	Aa =	0.04	m2	
66.8 % Flow Outline	Oa =	0.58	m	
Hydraulic Beam	R =	0.07	m	
Actual Velocity	v =	4.60	m/s	SUFFICIENT
Maximum Allowed flow				
75% Area	A75% =	0.05	m^2	
75% Outline	O75% =	0.63	m	
Hydraulic Beam	R =	0.08	m	
Drain Capacity	Q =	0.22	m^3/s	SUFFICIENT
Velocity	v =	4.75	m/s	

Table 35B: Storm drain calculation for Reach 12.

Storm drain Characteristics				
Longitudinal Slope	So =	9.0%	m/m	

Formula for hydraulic conveyance capacity of street:	$Q = K / n * Sx^{1.67} * T^{2.67} * \sqrt{So}$			
Road + side gutter				
Total Width Road (incl. gutter)	Wd =	4.00	m	
Total Width Gutter	Wg =	0.30	m	
Side flow width (Max)	Tx =	1.85	m	
Gutter flow width	W =	0.30	m	
Total water spread width on street	T =	2.15	m	
Roughness Factor	n =	0.014	s/m1/3	
Street transverse slope	Sx =	2.0%	m/m	
Extra capacity due depth standard gutter	Qextra	0.02	m3/s	
Drain Capacity	Q =	0.11	m3/s	NOT SUFFICIENT
Velocity	v =	2.36	m/s	SUFFICIENT

Formula for Manning:	$Q = (A * R^{2/3} * \sqrt{So})$			
U-gutter				
height	H=	0.50	m	
width	B=	0.50	m	
slope	So=	9.0%	m/m	
Roughness Factor	n =	0.014	s/m1/3	
Area	A =	0.25	m2	
Actual flow				
Discharge (Q10-year)	Q =	0.58	m3/s	
Flow height	Ha =	0.225	m	
45.0 % Flow Area	Aa =	0.113	m2	
45.0 % Flow Outline	Oa =	0.95	m	
Hydraulic Beam	R =	0.12	m	
Actual Velocity	v =	5.17	m/s	SUFFICIENT
Maximum Allowed flow				
75% Area	A75% =	0.19	m^2	
75% Outline	O75% =	1.25	m	
Hydraulic Beam	R =	0.15	m	
Drain Capacity	Q =	1.13	m^3/s	SUFFICIENT
Velocity	v =	6.05	m/s	

Table 36B: Storm drain calculation for Reach 13.

Storm drain Characteristics				
Longitudinal Slope	So =	9.0%	m/m	

Formula for hydraulic conveyance capacity of street:	$Q = K / n * Sx^{1.67} * T^{2.67} * \sqrt{So}$			
Road + side gutter				
Total Width Road (incl. gutter)	Wd =	4.00	m	
Total Width Gutter	Wg =	0.30	m	
Side flow width (Max)	Tx =	1.85	m	
Gutter flow width	W =	0.30	m	
Total water spread width on street	T =	2.15	m	
Roughness Factor	n =	0.014	s/m1/3	
Street transverse slope	Sx =	2.0%	m/m	
Extra capacity due depth standard gutter	Qextra	0.02	m3/s	
Drain Capacity	Q =	0.11	m3/s	NOT SUFFICIENT
Velocity	v =	2.36	m/s	SUFFICIENT

Formula for Manning:	$Q = (A * R^{2/3} * \sqrt{So})$			
U-gutter				
height	H=	0.30	m	
width	B=	0.35	m	
slope	So=	9.0%	m/m	
Roughness Factor	n =	0.014	s/m1/3	
Area	A =	0.11	m2	
Actual flow				
Discharge (Q10-year)	Q =	0.30	m3/s	
Flow height	Ha =	0.198	m	
66.0 % Flow Area	Aa =	0.069	m2	
66.0 % Flow Outline	Oa =	0.75	m	
Hydraulic Beam	R =	0.09	m	
Actual Velocity	v =	4.39	m/s	SUFFICIENT
Maximum Allowed flow				
75% Area	A75% =	0.08	m^2	
75% Outline	O75% =	0.80	m	
Hydraulic Beam	R =	0.10	m	
Drain Capacity	Q =	0.36	m^3/s	SUFFICIENT
Velocity	v =	4.56	m/s	

Table 37B: Storm drain calculation for Reach 14.

Storm drain Characteristics				
Longitudinal Slope	So =	16.0%	m/m	

Formula for hydraulic conveyance capacity of street:	$Q = K / n * Sx^{1.67} * T^{2.67} * \sqrt{So}$			
Road + side gutter				
Total Width Road (incl. gutter)	Wd =	7.50	m	
Total Width Gutter	Wg =	0.30	m	
Side flow width (Max)	Tx =	3.60	m	
Gutter flow width	W =	0.30	m	
Total water spread width on street	T =	3.90	m	
Roughness Factor	n =	0.016	s/m1/3	
Street transverse slope	Sx =	2.0%	m/m	
Extra capacity due depth standard gutter	Qextra	0.02	m3/s	
Drain Capacity	Q =	0.54	m3/s	NOT SUFFICIENT
Velocity	v =	3.55	m/s	SUFFICIENT

Formula for Manning:	$Q = (A * R^{2/3} * \sqrt{So})$			
U-gutter				
height	H=	0.35	m	
width	B=	0.35	m	
slope	So=	16.0%	m/m	
Roughness Factor	n =	0.014	s/m1/3	
Area	A =	0.12	m2	
Actual flow				
Discharge (Q10-year)	Q =	0.32	m3/s	
Flow height	Ha =	0.164	m	
47.0 % Flow Area	Aa =	0.058	m2	
47.0 % Flow Outline	Oa =	0.68	m	
Hydraulic Beam	R =	0.08	m	
Actual Velocity	v =	5.51	m/s	SUFFICIENT
Maximum Allowed flow				
75% Area	A75% =	0.09	m^2	
75% Outline	O75% =	0.88	m	
Hydraulic Beam	R =	0.11	m	
Drain Capacity	Q =	0.58	m^3/s	SUFFICIENT
Velocity	v =	6.35	m/s	

Table 38B: Storm drain calculation for Reach 15.

Storm drain Characteristics				
Longitudinal Slope	So =	12.0%	m/m	

Formula for hydraulic conveyance capacity of street:	$Q = K / n * Sx^{1.67} * T^{2.67} * \sqrt{So}$			
Road + side gutter				
Total Width Road (incl. gutter)	Wd =	7.50	m	
Total Width Gutter	Wg =	0.30	m	
Side flow width (Max)	Tx =	3.60	m	
Gutter flow width	W =	0.30	m	
Total water spread width on street	T =	3.90	m	
Roughness Factor	n =	0.016	s/m1/3	
Street transverse slope	Sx =	2.0%	m/m	
Extra capacity due depth standard gutter	Qextra	0.02	m3/s	
Drain Capacity	Q =	0.47	m3/s	NOT SUFFICIENT
Velocity	v =	3.07	m/s	SUFFICIENT

Formula for Manning:	$Q = (A * R^{2/3} * \sqrt{So})$			
U-gutter				
height	H=	0.35	m	
width	B=	0.35	m	
slope	So=	12.0%	m/m	
Roughness Factor	n =	0.014	s/m1/3	
Area	A =	0.12	m2	
Actual flow				
Discharge (Q10-year)	Q =	0.49	m3/s	
Flow height	Ha =	0.255	m	
73.2 % Flow Area	Aa =	0.089	m2	
73.2 % Flow Outline	Oa =	0.86	m	
Hydraulic Beam	R =	0.10	m	
Actual Velocity	v =	5.46	m/s	SUFFICIENT
Maximum Allowed flow				
75% Area	A75% =	0.09	m^2	
75% Outline	O75% =	0.88	m	
Hydraulic Beam	R =	0.11	m	
Drain Capacity	Q =	0.51	m^3/s	SUFFICIENT
Velocity	v =	5.50	m/s	

Table 39B: Storm drain calculation for Reach 16.

Storm drain Characteristics				
Longitudinal Slope	So =	14.0%	m/m	
Formula for hydraulic conveyance capacity of street:	$Q = K / n * Sx^{1.67} * T^{2.67} * \sqrt{So}$			
Road + side gutter				
Total Width Road (incl. gutter)	Wd =	4.00	m	
Total Width Gutter	Wg =	0.30	m	
Side flow width (Max)	Tx =	1.85	m	
Gutter flow width	W =	0.30	m	
Total water spread width on street	T =	2.15	m	
Roughness Factor	n =	0.014	s/m1/3	
Street transverse slope	Sx =	2.5%	m/m	
Extra capacity due depth standard gutter	Qextra	0.02	m3/s	
Drain Capacity	Q =	0.19	m3/s	SUFFICIENT
Velocity	v =	3.24	m/s	SUFFICIENT

Table 40B: Storm drain calculation for Reach 17.

Storm drain Characteristics				
Longitudinal Slope	So =	10.0%	m/m	
Formula for hydraulic conveyance capacity of street:	$Q = K / n * Sx^{1.67} * T^{2.67} * \sqrt{So}$			
Road + side gutter				
Total Width Road (incl. gutter)	Wd =	7.50	m	
Total Width Gutter	Wg =	0.30	m	
Side flow width (Max)	Tx =	3.60	m	
Gutter flow width	W =	0.30	m	
Total water spread width on street	T =	3.90	m	
Roughness Factor	n =	0.016	s/m1/3	
Street transverse slope	Sx =	2.0%	m/m	
Extra capacity due depth standard gutter	Qextra	0.02	m3/s	
Drain Capacity	Q =	0.43	m3/s	NOT SUFFICIENT
Velocity	v =	2.80	m/s	SUFFICIENT

Formula for Manning:	$Q = (A * R^{2/3} * \sqrt{So})$			
U-gutter				
height	H=	0.60	m	
width	B=	0.60	m	
slope	So=	10.0%	m/m	
Roughness Factor	n =	0.014	s/m1/3	
Area	A =	0.36	m2	
Actual flow				
Discharge (Q10-year)	Q =	0.87	m3/s	
Flow height	Ha =	0.244	m	
38.1 % Flow Area	Aa =	0.146	m2	
38.1 % Flow Outline	Oa =	1.09	m	
Hydraulic Beam	R =	0.13	m	
Actual Velocity	v =	5.93	m/s	SUFFICIENT
Maximum Allowed flow				
75% Area	A75% =	0.27	m^2	
75% Outline	O75% =	1.50	m	
Hydraulic Beam	R =	0.18	m	
Drain Capacity	Q =	1.94	m^3/s	SUFFICIENT
Velocity	v =	7.20	m/s	

Table 41B: Storm drain calculation for Reach 18.

Storm drain Characteristics				
Longitudinal Slope	So =	12.0%	m/m	

Formula for hydraulic conveyance capacity of street:	$Q = K / n * Sx^{1.67} * T^{2.67} * \sqrt{So}$			
Road + side gutter				
Total Width Road (incl. gutter)	Wd =	7.50	m	
Total Width Gutter	Wg =	0.30	m	
Side flow width (Max)	Tx =	3.60	m	
Gutter flow width	W =	0.30	m	
Total water spread width on street	T =	3.90	m	
Roughness Factor	n =	0.016	s/m1/3	
Street transverse slope	Sx =	2.0%	m/m	
Extra capacity due depth standard gutter	Qextra	0.02	m3/s	
Drain Capacity	Q =	0.47	m3/s	NOT SUFFICIENT
Velocity	v =	3.07	m/s	SUFFICIENT

Formula for Manning:	$Q = (A * R^{2/3} * \sqrt{So})$			
U-gutter				
height	H=	0.95	m	
width	B=	0.60	m	
slope	So=	12.0%	m/m	
Roughness Factor	n =	0.014	s/m1/3	
Area	A =	0.57	m2	
Actual flow				
Discharge (Q10-year)	Q =	1.51	m3/s	
Flow height	Ha =	0.344	m	
36.2 % Flow Area	Aa =	0.206	m2	
36.2 % Flow Outline	Oa =	1.29	m	
Hydraulic Beam	R =	0.16	m	
Actual Velocity	v =	7.30	m/s	SUFFICIENT
Maximum Allowed flow				
75% Area	A75% =	0.43	m^2	
75% Outline	O75% =	2.03	m	
Hydraulic Beam	R =	0.21	m	
Drain Capacity	Q =	3.75	m^3/s	SUFFICIENT
Velocity	v =	8.77	m/s	

Table 42B: Storm drain calculation for Reach 20.

Storm drain Characteristics				
Longitudinal Slope	So =	49.0%	m/m	

Formula for Manning:	$Q = (A * R^{2/3} * \sqrt{So})$			
U-gutter				
height	H=	0.30	m	
width	B=	1.10	m	
slope	So=	49.0%	m/m	
Roughness Factor	n =	0.014	s/m1/3	
Area	A =	0.33	m2	
Actual flow				
Discharge (Q10-year)	Q =	0.62	m3/s	
Flow height	Ha =	0.071	m	
23.7 % Flow Area	Aa =	0.078	m2	
23.7 % Flow Outline	Oa =	1.24	m	
Hydraulic Beam	R =	0.06	m	
Actual Velocity	v =	7.91	m/s	NOT SUFFICIENT
Maximum Allowed flow				
75% Area	A75% =	0.25	m^2	
75% Outline	O75% =	1.55	m	
Hydraulic Beam	R =	0.16	m	
Drain Capacity	Q =	3.64	m^3/s	SUFFICIENT
Velocity	v =	14.71	m/s	

Table 43B: Storm drain calculation for Reach 21.

Storm drain Characteristics				
Longitudinal Slope	So =	44.3%	m/m	
Formula for Manning:	$Q = (A * R^{2/3} * \sqrt{So})$			
U-gutter				
height	H=	0.30	m	
width	B=	0.70	m	
slope	So=	44.3%	m/m	
Roughness Factor	n =	0.014	s/m1/3	
Area	A =	0.21	m2	
Actual flow				
Discharge (Q10-year)	Q =	0.47	m3/s	
Flow height	Ha =	0.085	m	
28.2 % Flow Area	Aa =	0.059	m2	
28.2 % Flow Outline	Oa =	0.87	m	
Hydraulic Beam	R =	0.07	m	
Actual Velocity	v =	7.93	m/s	**NOT SUFFICIENT**
Maximum Allowed flow				
75% Area	A75% =	0.16	m^2	
75% Outline	O75% =	1.15	m	
Hydraulic Beam	R =	0.14	m	
Drain Capacity	Q =	1.99	m^3/s	**SUFFICIENT**
Velocity	v =	12.62	m/s	

Table 44B: Storm drain calculation for Reach 22.

Storm drain Characteristics				
Longitudinal Slope	So =	44.3%	m/m	

Formula for Manning:	$Q = (A * R^{2/3} * \sqrt{So})$			
U-gutter				
height	H=	0.30	m	
width	B=	1.00	m	
slope	So=	44.3%	m/m	
Roughness Factor	n =	0.014	s/m1/3	
Area	A =	0.30	m2	
Actual flow				
Discharge (Q10-year)	Q =	0.25	m3/s	
Flow height	Ha =	0.041	m	
14.5 % Flow Area	Aa =	0.041	m2	
14.5 % Flow Outline	Oa =	1.08	m	
Hydraulic Beam	R =	0.04	m	
Actual Velocity	v =	5.64	m/s	SUFFICIENT
Maximum Allowed flow				
75% Area	A75% =	0.23	m^2	
75% Outline	O75% =	1.45	m	
Hydraulic Beam	R =	0.16	m	
Drain Capacity	Q =	3.09	m^3/s	SUFFICIENT
Velocity	v =	13.72	m/s	

Table 45B: Summary of the streets in their relative reach used as storm drains for the 10-year storm event.

Reach	Qcum (m3/s)	Road width (m)	Transverse Slope (%)	Longitudinal slope (%)	Qmax(m3/s)	Flow Velocity (m/s)	Status	Runoff direction
1	0.09	4.5	2.0	27%	0.24	27%	SUFFICIENT	Lower P. Quarter
2	0.03	4.5	2.0	3%	0.08	3%	SUFFICIENT	Lower P. Quarter
3	0.42	4.5	2.0	16%	0.19	16%	NOT SUFFICIENT	Lower P. Quarter
4	0.20	4.5	2.0	1%	0.05	1%	NOT SUFFICIENT	Reach 20
5	0.04	4.5	2.0	1%	0.06	1%	SUFFICIENT	Lower P. Quarter
6	0.15	3.0	2.0	9%	0.06	9%	NOT SUFFICIENT	Reach 22
7	0.17	4.5	2.0	2%	0.07	2%	NOT SUFFICIENT	Reach 20
8	0.17	7.5	2.0	12%	0.47	12%	SUFFICIENT	Reach 18
9	0.16	4.5	2.5	11%	0.22	11%	SUFFICIENT	Reach 22
10	0.69	7.5	2.0	12%	0.47	12%	NOT SUFFICIENT	Ebenezer
11	0.19	3.5	2.0	14%	0.10	14%	NOT SUFFICIENT	Reach 21
12	0.58	4.0	2.0	9%	0.11	9%	NOT SUFFICIENT	Reach 21
13	0.30	4.0	2.0	9%	0.11	9%	NOT SUFFICIENT	Reach 15
14	0.32	7.5	2.0	16%	0.54	16%	SUFFICIENT	Reach 18
15	0.49	7.5	2.0	12%	0.47	12%	NOT SUFFICIENT	Valley estate
16	0.17	4.0	2.5	14%	0.19	14%	SUFFICIENT	Reach 18
17	0.87	7.5	2.0	10%	0.43	10%	NOT SUFFICIENT	Reach 18
18	1.51	7.5	2.0	12%	0.47	12%	NOT SUFFICIENT	Dutch Cul de Sac

Table 46B: Summary of the U-gutters discharge capacity in their relative reach for the 10-year storm event.

Reach	Name of Storm drain	Sizes HxB (m)	Qcum (m3/s)	Longitudinal Slope (%)	Qmax (m3/s	75% filling flow velocity (m/s)	Actual flow velocity (m/s)	Actual flow depth (m)	Status
3	U-Gutter 3	0.30 x 0.40	0.42	16%	0.50	5.59	5.45	0.21	SUFFICIENT
4	U-Gutter 4	0.45 x0.40	0.20	1%	0.24	1.79	1.72	0.29	SUFFICIENT
6	U-Gutter 6	0.30 x 0.30	0.17	9%	0.29	4.30	3.67	0.13	SUFFICIENT
7	U-Gutter 7	0.50 x 0.40	0.17	2%	0.26	2.28	2.12	0.27	SUFFICIENT
8	U-Gutter 8	0.30 x 0.30	0.17	12%	0.34	4.23	3.07	0.13	SUFFICIENT
10	U-Gutter 10	0.75 x 0.30	0.69	12%	1.01	5.96	5.66	0.41	SUFFICIENT
11	U-Gutter 11	0.25 x 0.25	0.19	14%	0.22	4.75	4.60	0.17	SUFFICIENT
12	U-Gutter 12	0.50 x 0.50	0.58	9%	1.13	6.05	5.17	0.23	SUFFICIENT
13	U-Gutter 13	0.30 x 0.35	0.30	9%	0.36	4.56	4.39	0.20	SUFFICIENT
14	U-Gutter 14	0.35 x 0.35	0.32	16%	0.58	6.35	3.55	0.16	SUFFICIENT
15	U-Gutter 15	0.35 x 0.35	0.49	12%	0.51	5.50	5.46	0.26	SUFFICIENT
17	U-Gutter 17	0.60 x 0.60	0.87	10%	1.94	7.20	5.93	0.24	SUFFICIENT
18	U-Gutter 18	0.95 x 0.60	1.51	12%	3.75	8.77	7.30	0.34	SUFFICIENT
20	U-Gutter 20	0.30 x 1.10	0.62	49%	3.64	14.71	7.91	0.071	SUFFICIENT
21	U-Gutter 21	0.30 x 0.70	0.47	44%	1.99	12.62	7.93	0.085	SUFFICIENT
22	U-Gutter 22	0.30 x 1.00	0.25	44%	3.09	13.72	5.63	0.041	SUFFICIENT

Appendix C. SCS Method

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1. SCS Calculation Method

Soil Conservation Service (SCS) is a statistical method for peak flow determination based on rainfall, soil type, and land use (McCuen R. , 2005). This method uses a variable known as Curve Number (CN) that represents the specific hydrologic soil group (HSG), land cover, antecedent moisture condition, and hydrologic condition of an area (NRCS, 1986). The value of CN various between for 0 to 100, with 0 resulting in no runoff and 100 representing a completely impervious area which generates an excess rain equal to the rainfall. For natural catchments CN is normally between 50 and 100.

The main hypothesis of the SCS method is that the ratio between the additional water retained in catchment area after the start of the runoff process and the potential maximum retention is equal to the ratio between the excess precipitation and the potential runoff:

$$\frac{Fa}{S} = \frac{Pe}{P - Ia} \tag{1}$$

Where:

Ia = initial abstraction (Losses occurred before runoff begins)

Fa = additional depth of water retained in the subcatchment after the start of the runoff process

Pe = excess precipitation contributing to runoff

P = rainfall (equal to Pe+ Ia + Fa)

S = Potential maximum retention after runoff begins.

The potential maximum retention, in turn, is directly related to the initial abstraction, Ia, as displayed in E.q. (2).

$$I_a = 0.2 \text{ x } S \tag{2}$$

Considering Ia=0.2*S and arranging the equation, the depth of excess rainfall from a storm is defined in E.q. (3).

$$Pe = \frac{(P - I_a)^2}{(P - I_a) + S} = \frac{(P - 0.2 \text{ x } S)^2}{(P + 0.8 \text{ x } S)} \tag{3}$$

Based on the soil type and the land use and the land use an equivalent curve number can be defined for each subcatchment. The value of S (in mm) and the curve number, CN, are define in E.q. (4).

$$S = \frac{25400}{CN} - 254 \tag{4}$$

The curve number applied in this research was obtained from the report (St Maarten Stormwater Modelling Study, 2006), the CN was identified based on the land use and the type of soil. As the slope has an influence on the subcatchment runoff. The calculated CN values obtained from the report is presented in Table 1C.

Since the land use changes overtime, the value of the curve number was calculated differently to represent present and future land development scenarios. The CN corresponding to each of the subcatchment is calculated weighting the CN value in Table 1C by the percentage of the subcatchment with each land use and slope range.

Table 1C: CN for each land use and soil slope (Vojinovic & Bonilo, 2006)

Land Use	Slope	CN
Ponds		100
Building and paved surfaces		95
Non developed	>40°	71
	30°- 40°	68
	20°- 30°	65
	10°- 20°	61
	0°- 10°	58

1.1. Current development scenario

For the calculation of the CN of each sub catchment the degree of urbanization has been determined by measuring the area of the existing houses and using a ratio to calculate the area occupied by paved (or impervious) infrastructure. The ratio between the infrastructure area and the building area depends on the density of buildings (or development density). Usually a low density of buildings is associated with a higher value of the ratio Infrastructure/Building.

The surface occupied by the buildings and infrastructure has been measured for the study area (Waymouth Hills). The Waymouth Hills catchment encompasses of roughly 16.4 ha, the total surface occupied by buildings was 1.09 ha, and the infrastructure was 0.93 ha. From such measurements, the ratio Infrastructure/building has been determined, as shown in Table 2C.

Table 2C: Measured percentage of buildings and infrastructure

Name of Area	Building %	Infrastructure %	Ratio Infra./ Build.	Impervious area %
The Waymouth Hills	7.4	6.3	0.85	13.7

For the calculation of the CN in the present scenario the area occupied by buildings has been measured for each subcatchment (e.g. for subcatchment F2, covering an area of 5945m^2, it was found that buildings occupy an area of 911m^2).

Impervious area was calculated as the summation of buildings and surrounding infrastructure (roads and footpaths). The area occupied by the infrastructure has been estimated by multiplying the area occupied by the buildings (i.e., the ratio between infrastructure and buildings). The portions of subcatchment areas are divided according to the slope and development areas and as such they are used in the calculation of the equivalent CN of each subcatchment.

Explanation: for example, for subcatchment F2 the ratio infrastructure/building is 0.85, so the area occupied by infrastructure will be 0.85 x 911=774m^2. The impervious surface will be 911 +744= 1655m^2, that us the 27.8% of the total area of the subcatchment. So the 72.2% remained undeveloped.

The portions of the subcatchment areas are divided according to the slope and the development areas and as such they are used in the calculation of the equivalent CN of each subcatchment. The following example describe the calculation process applied in this research:

For example: in subcatchment F2, 0% of the area has slope in the range 0-10^{O}, 0% in the range10-20^{O}, 0% in the range 20-30^{O}, 24% in the range 30-40^{O}and 76% in the range >40^{O}.

The values of CN for undeveloped land considering slopes are found from Table 1B: 58 (0-10^{O}), 61 (10-20^{O}), 65 (20-30^{O}), 68 (30-40^{O}) and 71 (>40^{O}). The curve number for the developed land is 95, so the equivalent curve number for the subcatchment is:

$$CN= 0.278 \times 95 + 0.722 \times (0 \times 58 + 0 \times 61 + 0 \times 65 + 0.24 \times 68 + 0.76 \times 71) = 77$$

The CN value of each subcatchment for the present development are presented Table 4C.

1.2. Future urban development scenario

The future urban development scenario represents the complete development of the studied area, with the exception of protected area within the subcatchment. The area allowed for future development has been calculated by subtracting the protected area (i.e., areas where development is not allowed, plus the pond area) from the total area of the subcatchment. The figures used to represent the future development scenario are calculated according to “General Guideline for building in Hillside Areas”. These guidelines are based on the following:

Table 3C: Maximum lot size and percentage of lot built (Ministerie van Infrastructuur en Milieu, et. al, 1986)

	Terrain Slope (degrees)			
	0^{o}-10^{o}	10^{o}-20^{o}	20^{o}-30^{o}	30^{o}-40^{o}
Lot size (m2)	400	800	1200	2000
Max % of lot built	35	30	25	15

The average lot size and percentage of lot built in each subcatchment are calculated from the lot size and the maximum percentage of lot build from Table 3C with the percentage of the surface of the subcatchment within the slope range. However, due to lack of government regulation, the lot size defined Table 3C is not fully complied within the slope range. Hence, the maximum percentage of lot built is based on the percentage of area within the different slope range by the maximum percentage allowed defined in consist within the subcatchment. Since, the maximum percentage of lot built for area that are steeper that 40^{o} was not included in the guideline defined in Table 3C, to be conservative for the calculation this value was assumed to be 15%. The following example illustrates the calculation process:

Example: For subcatchment R2 encompass a total area of 16691 m^2, 0% of its area has slope in the range 0-10^O, 0% in the range10-20^O, 32% in the range 20-30^O, 17% in the range 30-40^O and 51% in the range >40^O. The average lot size and percentage of the built is calculated as follows:

Av. Lot size = (0 x 0.35 + 0 x 0.30 + 0.32 x 0.25 + 0.17 x 0.15 + 0.51 x 0.15) x 16691m^2 = 3037.8 m^2

Percentage of Lot Built (PLB) = 16691/16691 = 18.2%

The area subjected to development in the future scenario is calculated by subtracting the protected area from the total area of the subcatchment. The potential developed area, **PD**, will be occupied by lot, **L,** lots with buildings **LB** and the infrastructure**, I.** One portion pf the total lots area will be occupied by buildings being the remaining lot area (garden, access, etc.). Figure 1B illustrate the flow chart of calculating the area of maximum potential development.

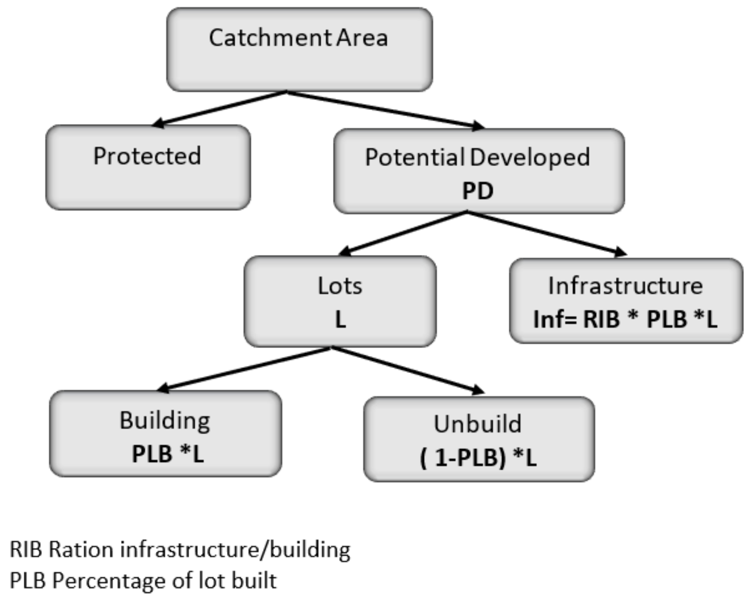

Figure 1C: Flow chart for maximum potential development calculation (Vojinovic & Bonilo, 2006)

The lots area, L, is calculated according to the following equation:

$$L = \left(\frac{PD}{1 + RIB \text{ x } PLB}\right) \quad (5)$$

And the impervious area is derived from the summation of the building area and infrastructure area:

$$PLB \text{ x } L \text{ x } (1 + RIB) \quad (6)$$

Considering the average percentage of lot build, the surface of the potential area subjected to development and a value for the ratio infrastructure/ building equal to one, the percentage of paved surface (building + infrastructure) in each sub catchment has been calculated as follows:

Example: For the subcatchment R2 the total area is 16,691m^2, 0 m^2 of the area is protected, so the potentially developed area, PD, is 16,691-0 =16,691m^2. The PLB is equal to 18.2% and the RIB is 1, so the area occupied by lots and the impervious area is derived from:

$$L = \left(\frac{PD}{1 + RIB * PLB}\right) = \left(\frac{16{,}691}{1 + 1 * 0.182}\right) = 14120m^2$$

The impervious area is calculated as:

$$PLB \text{ x } L \text{ x } (1 + RIB) = 0.182 \text{ x } 14120 \text{ x } (1 + 1) = 5139.7m^2$$

And that is the 30.7% of the total area of the subcatchment. So, 69.3% of the total area will be pervious.

The values of the equivalent Curve number for each of the sub catchments for the future urban development are calculated from the percentage of paved surface, undeveloped (classified according to the slopes) and ponds. Such calculations can be illustrated by the following example:

Example: For subcatchment R2, 0% of its area has slope in the range 0-10^O, 0% in the range10-20^O, 32% in the range 20-30^O, 17% in the range 30-40^{O}and 51% in the range >40^O. The impervious land covers (paved land) the 30.7% of the total area and the pervious land is the 69.1%. The CN values for the unpaved land with such slopes given in Table 1 are: 58 (0-10^O), 61 (10-20^O), 65 (20-30^O), 68 (30-40^O) and 71 (>40^O). The curve number for the paved land is 95, so the equivalent curve number for the subcatchment is calculated as:

CN= 0.307 x 95 + 0.691 x (0 x 58 + 0 x 61 + 0 .32 x 65 + 0.17 x 68 + 0.51 x 71) = 77

The CN value of each subcatchment for the future urban development are presented 5C. These values are higher than in the current development scenarios because of the new developments (the higher CN values will yield higher runoff).

2. Results

2.1. Land characteristic

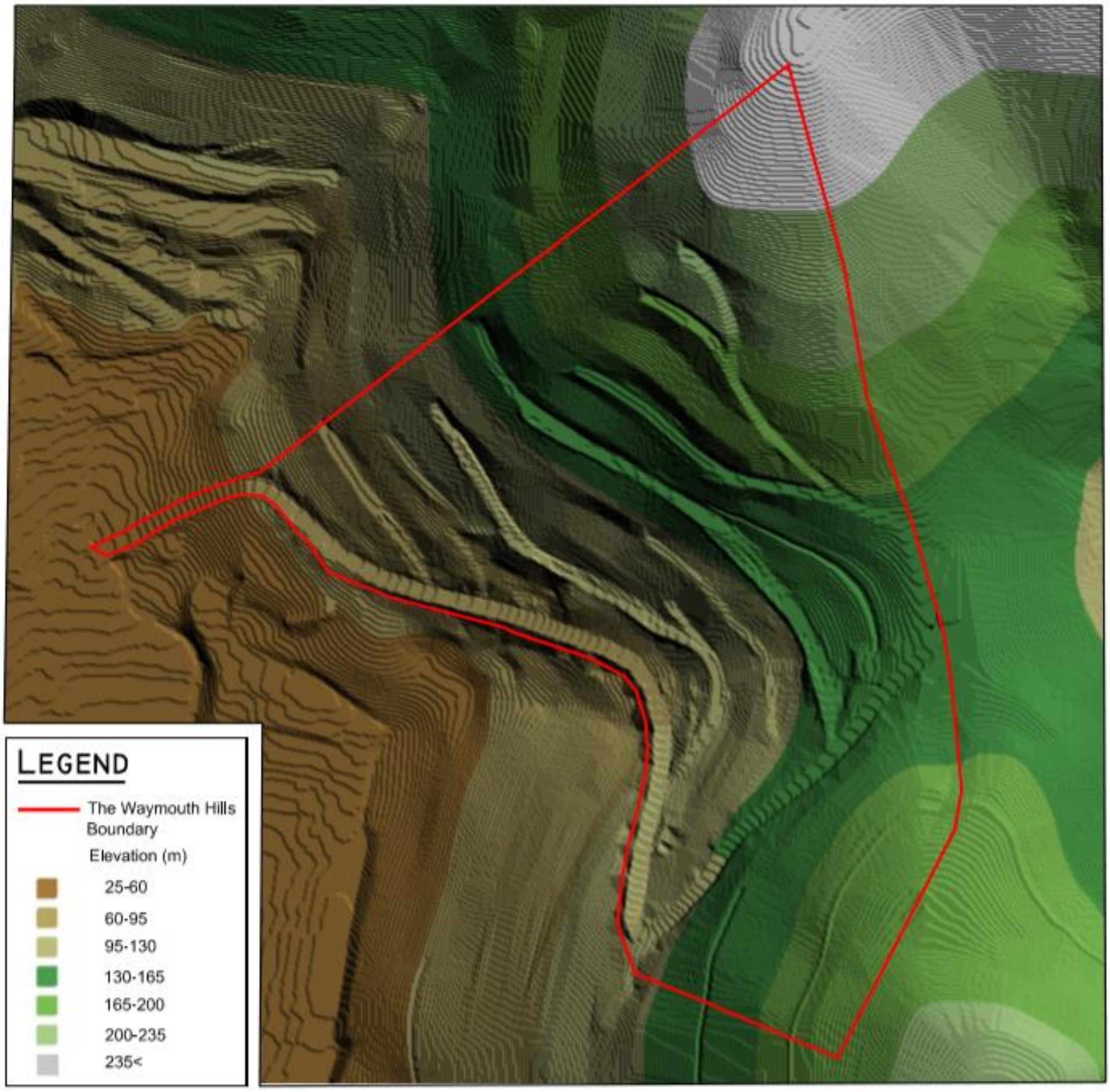

Figure 2C: Elevation map of the Waymouth Hills.

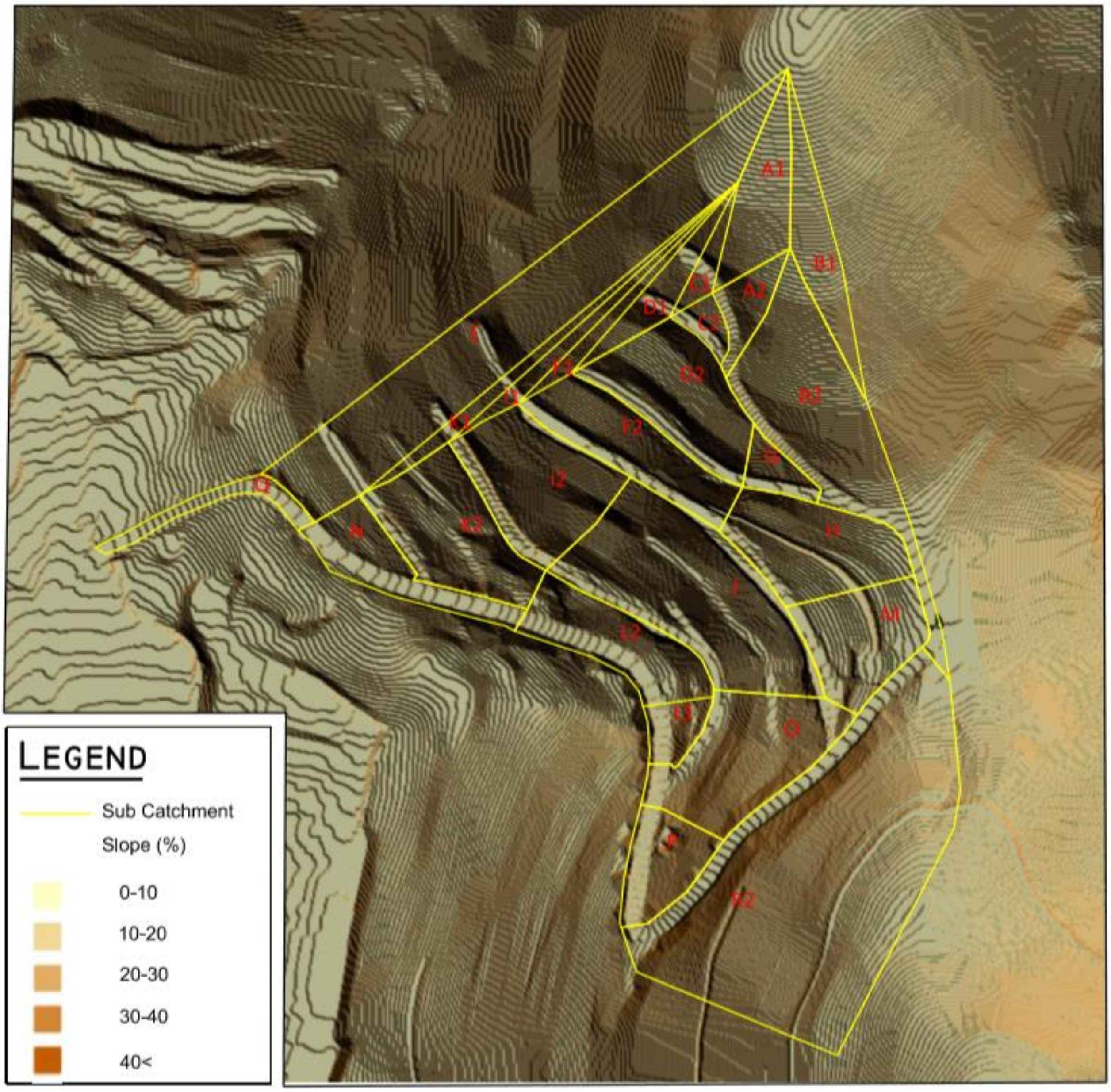

Figure 3C: Map of the Waymouth Hills displaying the slope range consist within the subcatchments.

Figure 4C: Map of the Waymouth Hills displaying the location of the protected areas.

2.2. Land uses and CN values of current and future urban development

Table 4C: Percentage of surface with different land uses and value of the Curve Number for the present (i.e., existing) land development scenario.

							Current Development Scenario							
Sub-catchment	Total area	% of area with slope in range					Built Surface		Infra/Build	B. + Inf. (Impervious)	Non- dev. (Pervious)		Pond	CN
	(m2)	0^0-10^0	10^0-	20^0-30^0	30^0-40^0	>40^0	(m2)	(%)	(-)	(%)	(m2)	(%)	(%)	
A1	4,165	0	0	0	0	100	0	0.0	0.85	0.0	4,165	100.0		71
A2	1,836	0	0	0	28	72	0	0.0	0.85	0.0	1,836	100.0		70
B1	3,869	0	0	0	0	100	0	0.0	0.85	0.0	3,869	100.0		71
B2	11,820	0	20	0	44	36	413	3.5	0.85	6.5	11,056	93.5		69
C1	1,002	0	0	0	0	100	0	0.0	0.85	0.0	1,002	100.0		71
C2	949	0	0	0	80	10	0	0.0	0.85	0.0	949	100.0		62
D1	3,254	0	0	0	11	89	0	0.0	0.85	0.0	3,254	100.0		71
D2	6,076	0	0	0	47	53	719	11.8	0.85	21.9	4,746	78.1		75
E	19,827	0	0	13	0	77	0	0.0	0.85	0.0	19,827	100.0		63
F1	1,163	0	0	0	0	100	0	0.0	0.85	0.0	1,163	100.0		71
F2	5,945	0	0	0	24	76	911	15.3	0.85	28.3	4,260	71.7		77
G	1,399	0	0	0	0	100	475	34.0	0.85	62.8	520	37.2		86
H	6,947	0	0	0	53	47	626	9.0	0.85	16.7	5,789	83.3		74
i1	1,445	0	0	0	0	100	0	0.0	0.85	0.0	1,445	100.0		71
i2	7,680	0	0	0	0	100	1097	14.3	0.85	26.4	5,651	73.6		77
J	13,827	0	0	0	29	71	1991	14.4	0.85	26.6	10,144	73.4		77
K1	1,630	0	0	0	0	100	0	0.0	0.85	0.0	1,630	100.0		71
K2	7,369	0	0	0	40	60	1186	16.1	0.85	29.8	5,175	70.2		77
L1	1,573	0	0	0	73	27	0	0.0	0.85	0.0	1,573	100.0		69
L2	5,590	0	0	0	0	100	909	16.3	0.85	30.1	3,908	69.9		78
M	5,948	0	0	100	0	0	905	15.2	0.85	28.1	4,274	71.9		73
N	4,284	0	0	0	93	7	185	4.3	0.85	8.0	3,942	92.0		70

							Current Development Scenario							
Sub-catchment	Total area	% of area with slope in range					Built Surface		Infra/Build	B. + Inf. (Impervious)	Non- dev. (Pervious		Pond	CN
	(m2)	0^0-10^0	10^0-	20^0-30^0	30^0-40^0	>40^0	(m2)	(%)	(-)	(%)	(m2)	(%)	(%)	
O	8,448	0	0	0	16	84	519	6.1	0.85	11.4	7,488	88.6		73
P	3,635	0	0	0	0	100	523	14.4	0.85	26.6	2,667	73.4		77
Q	1,595	0	100	0	0	0	0	0.0	0.85	0.0	1,595	100.0		61
R2	32,444	0	0	32	17	51	465	2.8	0.85	5.2	31,584	97.8		69
Total							**10924**							

Table 5C: Percentage of surface with different land use for the future urban development scenario and the Curve Number value.

									Future Urban Development Scenario							
Sub-catchment	Total Area	% of area with slopes in range					Protected area		Non -protected	Lot size	Lot built	Infra/ Build	Build. + Inf.	Non-develop.	Pond	CN
	(m2)	0^0-10^0	10^0-20^0	20^0-30^0	30^0-40^0	>40^0	(m2)	%	(%)	(m2)	(%)	(-)	(%)	(%)	(%)	
A1	4,165	0	0	0	0	100	4,165	100.0	0	0	0	1	0	100		71
A2	1,836	0	0	0	28	72	0	0.0	100	275	15	1	26	74		77
B1	3,869	0	0	0	0	100	3,869	100.0	0	0	0	1	0	100		71
B2	11,820	0	20	0	44	36	0	0.0	100	2,128	18	1	31	69		76
C1	1,002	0	0	0	0	100	1,002	100.0	0	0	0	1	0	100		71
C2	949	0	0	0	80	10	0	0.0	100	128	13.5	1	24	76		69
D1	3,254	0	0	0	11	89	3,254	100.0	0	0	0	1	0	100		71
D2	6,076	0	0	0	47	53	0	0.0	100	911	15	1	26	74		76
E	19,827	0	0	13	0	77	19,827	100.0	0	0	0	1	0	100		63
F1	1,163	0	0	0	0	100	1,163	100.0	0	0	0	1	0	100		71
F2	5,945	0	0	0	24	76	0	0.0	100	892	15	1	26	74		77

									Future Urban Development Scenario							
Sub-catchment	Total Area	% of area with slopes in range					Protected area		Non -protected	Lot size	Lot built	Infra/ Build	Build. + Inf.	Non-develop.	Pond	CN
	(m2)	0^0-10^0	10^0-20^0	20^0-30^0	30^0-40^0	>40^0	(m2)	(%)	(%)	(m2)	(%)	-	(%)	(%)	(%)	
G	1,399	0	0	0	0	100	0	0.0	100	210	15	1	26	74		77
H	6,947	0	0	0	53	47	0	0.0	100	1,042	15	1	26	74		76
i1	1,445	0	0	0	0	100	1,445	100.0	0	0	0	1	0	100		71
i2	7,680	0	0	0	0	100	0	0.0	100	1,152	15	1	26	74		77
J	13,827	0	0	0	29	71	0	0.0	100	2,074	15	1	26	74		77
K1	1,630	0	0	0	0	100	1,630	100.0	0	0	0	1	0	100		71
K2	7,369	0	0	0	40	60	0	0.0	100	1,105	15	1	26	74		76
L1	1,573	0	0	0	73	27	0	0.0	100	236	15	1	26	74		76
L2	5,590	0	0	0	0	100	0	0.0	100	839	15	1	26	74		77
M	5,948	0	0	100	0	0	0	0.0	100	1,487	25	1	40	60		77
N	4,284	0	0	0	93	7	0	0.0	100	643	15	1	26	74		75
O	8,448	0	0	0	16	84	0	0.0	100	1267	15	1	26	74		77
P	3,635	0	0	0	0	100	0	0.0	100	545	15	1	26	74		77
Q	1,595	0	100	0	0	0	0	0.0	100	479	30	1	46	54		77
R2	32,444	0	0	32	17	51	0	0.0	100	5,905	18.2	1	31	69		77
Total										**21,317**						

Table 6C: CN values for present and future scenarios

Sub catchment	CN	
	Present	Future
A1	71	71
A2	70	77
B1	71	71
B2	69	76
C1	71	71
C2	62	69
D1	71	71
D2	75	76
E	63	63
F1	71	71
F2	77	77
G	86	77
H	74	76
i1	71	71
i2	77	77
J	77	77
K1	71	71
K2	77	76
L1	69	76
L2	78	77
M	73	77
N	70	75
O	73	77
P	77	77
Q	61	77
R2	69	77

The subcatchmnets that are highlighted in the green are protected area, no changes in development in these areas occurs the CN value are the same in both development scenario. On the other hand, the subcatchments that are highlighted in grey illustrate the CN value in the current scenario is larger than the future development CN value. The built area in these subcatchment for the current development is larger than the permitted allowed defined in Table 3C 'General Guideline for building in Hillside Areas'(1986).

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2.3. Hydrologic effect of the current and future development

Table 7C: Infiltration and runoff effect in the present development.

Sub-catchment	Area	Weighted CN	Total Precipitation	Total Infiltration	Total Runoff	Peak Runoff	Time of concentration	Total	
								Infiltration	Runoff
I.D	(m2)	(-)	(mm)	(mm)	(mm)	(m3/s)	(mm:ss)	(m3)	(m3)
A1	4,165	71	91	66.2	25.2	0.09	15:09	275.6	104.7
A2	1,836	70	91	65.7	27.0	0.05	9:17	120.6	49.5
B1	3,869	71	91	66.2	25.3	0.08	14:30	256.0	97.7
B2	11,820	72	91	67.3	24.5	0.2	28:20	795.8	289.2
C1	1,002	71	91	64.3	30.1	0.03	6:24	64.4	30.2
C2	949	62	91	67.0	26.4	0.03	6:14	63.6	25.1
D1	3,254	71	91	65.9	26.3	0.06	10:38	214.5	85.6
D2	6,076	75	91	64.4	25.5	0.14	20:44	391.1	155.0
E	19,827	63	91	68.2	24.3	0.31	14:51	1351.3	480.8
F1	1,163	71	91	65.0	28.3	0.04	7:26	75.6	32.9
F2	5,945	72	91	66.8	24.9	0.12	18:45	396.9	147.7
G	1,399	86	91	37.0	53.5	0.07	7:53	51.8	74.8
H	6,947	74	91	66.0	25.2	0.14	20:37	458.8	174.8
I1	1,445	71	91	65.4	27.4	0.05	8:38	94.5	39.5
I2	7,680	72	91	67.0	24.6	0.14	21:55	514.6	189.2
J	13,827	77	91	60.8	27.5	0.24	31:08	841.0	380.5
K1	1,630	71	91	65.7	26.8	0.05	31:08	107.0	43.7
K2	7,369	77	91	60.7	27.5	0.15	21:14	447.6	202.4
L1	1,573	69	91	64.8	27.0	0.04	8:26	101.9	42.5
L2	5,590	78	91	56.5	31.5	0.12	18:05	315.6	176.0
M	5,948	73	91	66.3	25.1	0.12	18:21	394.5	149.2
N	4,284	70	91	66.8	25.1	0.09	18:21	286.2	107.3
O	8,448	73	91	66.7	24.7	0.16	15:26	563.7	208.8
P	3,635	77	91	60.6	28.2	0.09	23:11	220.1	102.3
Q	1,595	61	91	67.5	24.9	0.04	8:30	107.7	39.7

Sub-catchment I.D	Area (m2)	Weighted (-)	Total Precipitation (mm)	Total (mm)	Total (mm)	Peak (m3/s)	Time of concentration (mm:ss)	Total Infiltration (m3)	Total Runoff (m3)
R2	32,444	69	91	68.0	24.6	0.50	51:57	2206.8	799.4
							Total	**10717.1**	**4228.6**

Table 8C: Infiltration and runoff effect in the future urban development.

Sub-catchment I.D	Area (m2)	Weighted CN (-)	Total Precipitation (mm)	Total Infiltration (mm)	Total Runoff (mm)	Peak Runoff (m3/s)	Time of concentration (mm:ss)	Total Infiltration (m3)	Total Runoff (m3)
A1	4,165	71	91	66.2	25.15	0.09	15:09	275.6	104.7
A2	1,836	77	91	65.1	27.30	0.05	9:17	119.6	50.1
B1	3,869	71	91	66.2	25.26	0.08	14:30	256.0	97.7
B2	11,820	76	91	61.7	26.58	0.21	28:20	728.7	314.2
C1	1,002	71	91	64.3	30.13	0.03	6:24	64.4	30.2
C2	949	69	91	65.2	28.84	0.03	6:14	61.8	27.4
D1	3,254	71	91	65.9	26.31	0.06	10:38	214.5	85.6
D2	6,076	76	91	61.6	26.68	0.14	20:44	374.2	162.1
E	19,827	63	91	68.2	24.25	0.31	14:51	1351.3	480.8
F1	1,163	71	91	65.0	28.32	0.04	7:26	75.6	32.9
F2	5,945	77	91	60.7	27.54	0.13	18:45	360.9	163.7
G	1,399	77	91	58.5	31.85	0.05	7:53	81.8	44.6
H	6,947	76	91	61.6	26.68	0.14	20:37	427.9	185.3
I1	1,445	71	91	65.4	27.37	0.05	8:38	94.5	39.5
I2	7,680	77	91	60.7	27.44	0.15	21:55	466.5	210.7
J	13,827	77	91	60.8	27.52	0.24	31:08	841.0	380.5
K1	1,630	71	91	65.7	26.80	0.05	31:08	107.0	43.7
K2	7,369	76	91	61.6	26.67	0.14	21:14	453.9	196.5
L1	1,573	76	91	60.8	30.55	0.05	8:26	95.7	48.1
L2	5,590	77	91	60.7	27.74	0.12	18:05	339.2	155.1

Sub-catchment I.D	Area (m2)	Weighted (-)	Total Precipitation (mm)	Total (mm)	Total (mm)	Peak (m3/s)	Time of concentration (mm:ss)	Total Infiltration (m3)	Total Runoff (m3)
M	5,948	77	91	60.7	27.72	0.12	18:21	361.0	164.9
N	4,284	75	91	64.8	26.42	0.10	18:21	277.5	113.2
O	8,448	77	91	60.8	27.43	0.16	15:26	513.3	231.7
P	3,635	77	91	60.6	28.15	0.09	23:11	220.1	102.3
Q	1,595	77	91	58.8	31.23	0.05	8:30	93.7	49.8
R2	32,444	77	91	60.9	27.86	0.51	51:57	1975.0	903.9
							Total	**10231**	**4419.4**

Appendix D. Hydrodynamic Modelling Data and Simulation Results

Table of Contents

1. Current development scenario (Model 2)

1.1. Drainage network characteristics

Table 1D: Properties of drainage network used for the current development scenario (model 2).

Reach	Chainage	Length	Inlet elevation	Outlet elevation	Average slope	Manning's roughness Coefficient
	(m)	(m)	(m)	(m)	%	(-)
1	0-14	14.5	238.9	235.2	25.8	0.032
	14-33	18.7	235.2	230.6	24.6	0.032
	33-44	11.2	230.6	226.9	32.8	0.032
	44-56	12.2	226.9	222.9	33.0	0.032
2	0-25	25.6	224.5	222.9	6.1	0.032
3	0-35	34.8	222.6	213.2	26.9	0.032
	35-100	64.9	213.2	194.7	28.6	0.032
	100-137	36.6	194.7	188.9	15.9	0.032
	137-176	35.9	188.9	186.0	8.1	0.032
	176-181	5.4	186.0	185.0	17.7	0.032
4	0-12	12.5	187.5	187.1	3.0	0.032
	12-36	24.3	187.1	186.9	1.0	0.032
	36-82	46.1	186.9	186.4	1.0	0.032
	82-118	36.3	186.4	186.3	0.5	0.032
	118-139	20.7	186.3	186.2	0.4	0.032
5	0-37	36.8	189.7	189.0	1.9	0.032
6	0-13	13.2	167.0	166.7	2.3	0.032
	13-32	19.1	166.7	164.9	9.3	0.032
	32-62	30.2	164.9	160.1	15.9	0.032
	62-101	39.4	160.1	157.8	5.8	0.032
	101-114	12.7	157.8	157.6	2.0	0.032
7	0-11	10.7	167.1	166.9	1.8	0.032
	11-34	23.2	166.9	165.8	5.0	0.032
	34-62	27.8	165.8	164.1	6.2	0.032
	62-83	20.9	164.1	163.5	3.0	0.032
	83-100	16.8	163.5	163.3	1.0	0.032
	100-108	8.5	163.3	163.2	1.0	0.032
	108-117	9.1	163.2	163.1	1.0	0.032
	117-132	15.2	163.1	163.0	1.0	0.032
	132-148	16.2	163.0	162.8	0.9	0.032
8	0-27	27.1	113.1	109.8	12.0	0.032
	27-42	15.0	109.8	109.3	3.9	0.032
	42-50	7.9	109.3	108.6	8.4	0.032
	50-69	18.9	108.6	106.8	9.5	0.032
	69-103	34.4	106.8	101.3	16.1	0.032
	103-130	27.3	101.3	96.5	17.4	0.032
9	0-11	11.2	166.2	164.3	17.2	0.032
	11-20	9.2	164.3	162.8	15.7	0.032
	20-34	13.7	162.8	161.0	13.4	0.032

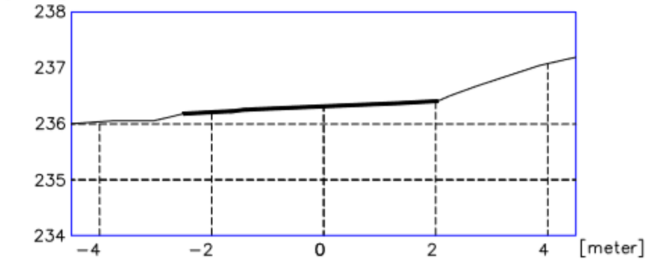
Reach	Chainage	Length	Inlet elevation	Outlet elevation	Average slope	Manning's roughness coefficient
	(m)	(m)	(m)	(m)	(%)	(-)
9	34-51	17.1	161.0	159.2	10.7	0.032
	51-71	20.4	159.2	157.7	7.0	0.032
10	0-11	10.7	178.4	177.8	6.4	0.032
	11-27	16.4	177.8	175.8	12.2	0.032
	27-52	24.9	175.8	172.6	12.7	0.032
	52-75	23.3	172.6	170.0	11.1	0.032
	75-97	21.6	170.0	167.6	11.4	0.032
	97-121	24.0	167.6	165.8	7.3	0.032
	121-150	29.5	165.8	162.7	10.6	0.032
	150-185	35.2	162.7	157.5	14.7	0.032
	185-208	23.5	157.5	153.7	16.1	0.032
	208-233	25.1	153.7	149.9	15.3	0.032
	233-250	16.6	149.9	147.0	17.4	0.032
	250-269	19.3	147.0	142.7	22.1	0.032
	269-279	9.8	142.7	140.6	21.6	0.032
	279-287	8.1	140.6	138.1	30.8	0.032
	287-309	21.7	138.1	136.6	7.0	0.032
11	0-14	13.9	129.0	127.8	8.0	0.032
	14-24	10.1	127.8	127.1	7.4	0.032
	24-32	7.8	127.1	125.7	18.5	0.032
	32-53	21.3	125.7	124.3	6.4	0.032
	53-65	11.8	124.3	122.2	18.3	0.032
	65-72	7.1	122.2	120.6	21.4	0.032
	72-79	7.5	120.6	119.6	13.7	0.032
	79-96	17.4	119.6	118.1	8.8	0.032
	96-110	13.8	118.1	116.6	11.0	0.032
12	0-11	11.5	127.3	127.1	2.3	0.032
	11-22	11.3	127.1	126.5	5.1	0.032
	22-35	13.3	126.5	125.1	10.6	0.032
	35-46	10.8	125.1	123.7	12.5	0.032
	46-54	7.9	123.7	122.3	17.7	0.032
	54-62	7.6	122.3	121.6	10.0	0.032
	62-90	28.4	121.6	118.3	11.5	0.032
	90-118	28.2	118.3	116.5	6.4	0.032
	118-130	11.7	116.5	116.3	1.5	0.032
13	0-5	4.9	127.9	127.7	3.1	0.032
	5-9	4.0	127.7	127.6	3.1	0.032
	9-14	5.0	127.6	127.4	3.1	0.032
	14-22	7.6	127.4	127.2	3.1	0.032
	22-35	13.5	127.2	126.8	3.1	0.032
	35-46	10.9	126.8	126.5	3.1	0.032
	46-61	15.4	126.5	126.0	3.1	0.032
	61-82	21.1	126.0	123.0	14.3	0.032
	82-94	11.7	122.8	118.4	38.3	0.032
14	0-29	29.3	136.0	130.5	18.7	0.032

Reach	Chainage	Length	Inlet elevation	Outlet elevation	Average slope	Manning's roughness coefficient
	(m)	(m)	(m)	(m)	(%)	(-)
14	29-53	23.7	130.5	126.7	16.0	0.032
	53-82	29.2	126.7	122.4	14.9	0.032
	82-110	27.6	122.4	118.4	14.5	0.032
15	0-22	22.4	118.4	114.7	16.4	0.032
	22-35	13.1	114.6	113.7	7.0	0.032
16	0-12	12.1	98.7	97.7	8.6	0.032
	12-26	14.2	97.7	96.7	6.9	0.032
	26-41	15.2	96.7	94.3	15.7	0.032
	41-50	8.7	94.3	92.2	24.0	0.032
	50-61	10.5	91.8	88.0	36.1	0.032
17	0-23	23.5	96.4	93.1	14.3	0.032
	23-51	27.5	93.1	90.6	8.8	0.032
	51-82	30.8	90.6	87.9	9.0	0.032
	82-108	26.2	87.8	84.6	12.1	0.032
	108-135	27.6	84.6	82.5	7.6	0.032
	135-164	28.6	82.5	80.7	6.4	0.032
18	0-13	12.7	80.4	79.3	8.5	0.032
	13-24	11.3	79.3	78.3	8.5	0.032
	24-36	12.3	78.3	76.8	12.8	0.032
	36-49	12.8	76.8	75.5	9.9	0.032
	49-61	11.6	75.5	73.5	17.1	0.032
	61-76	14.7	73.5	70.7	19.1	0.032
	76-104	27.7	70.7	64.7	21.6	0.032
	104-131	27.3	64.7	59.6	18.7	0.032
	131-157	26.5	59.6	56.4	12.0	0.032
	157-165	8.4	56.4	55.9	6.2	0.032
20	0-6	6.3	186.2	185.2	15.4	0.032
	6-44	38.0	185.2	162.8	58.9	0.032
	44-74	30.2	162.8	148.4	48.0	0.032
	74-80	6.3	148.4	145.3	48.5	0.032
	80-89	8.8	145.3	140.9	49.4	0.032
	89-97	8.3	140.9	134.7	74.6	0.032
	97-138	41.4	134.7	114.2	49.6	0.032
	138-152	13.8	114.2	106.9	52.7	0.032
	152-168	16.4	106.9	98.7	50.3	0.032
	168-210	41.5	98.7	83.3	37.0	0.032
	210-215	4.7	83.3	81.1	48.1	0.032
21	0-6	5.6	116.4	116.0	7.0	0.032
	6-20	13.9	116.0	103.4	90.1	0.032
	20-28	8.0	103.4	99.7	46.6	0.032
	28-36	8.0	99.7	97.9	22.1	0.032
	36-42	5.8	97.9	96.7	21.1	0.032
22	0-4	4.4	157.6	157.4	3.0	0.032
	4-10	5.5	157.4	155.3	39.4	0.032
	10-19	8.9	155.3	149.4	65.5	0.032

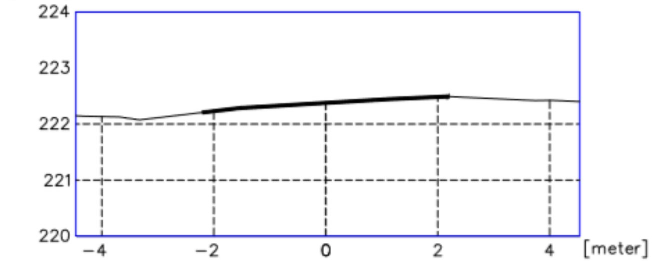
Reach	Chainage	Length	Inlet elevation	Outlet elevation	Average slope	Manning's roughness Coefficient
	(m)	(m)	(m)	(m)	(%)	(-)
22	19-30	11.6	149.4	144.2	44.5	0.032
	30-40	10.0	144.2	141.3	29.7	0.032
	40-53	13.0	141.3	141.1	1.0	0.032
	53-78	25.5	141.1	128.4	49.8	0.032
	78-84	5.6	128.4	127.9	8.7	0.032

1.2. Cross-sections of storm drains

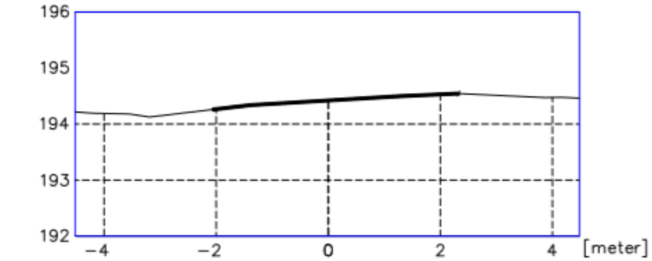
Reach 1
chainage 0-56m



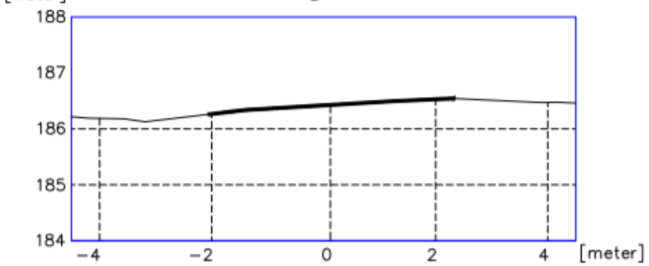
Reach 2
chainage 0-25m



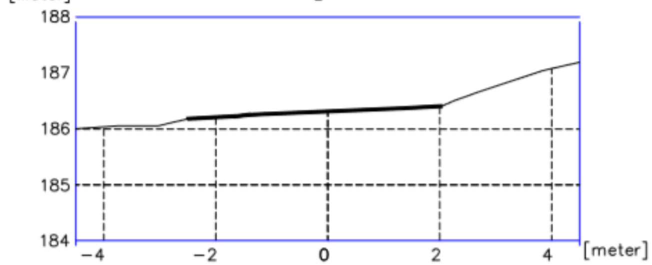
Reach 3
chainage 0-181m



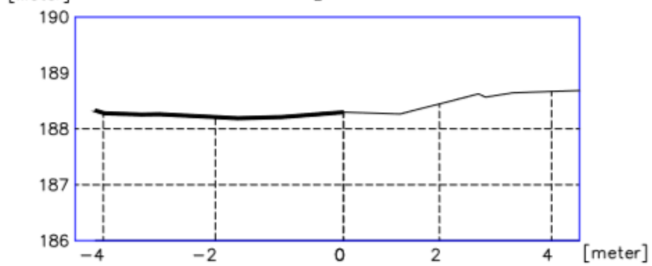
Reach 4
chainage 0-82m



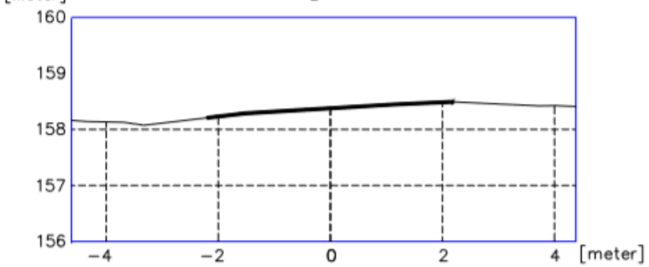
Reach 4
chainage 82-139m



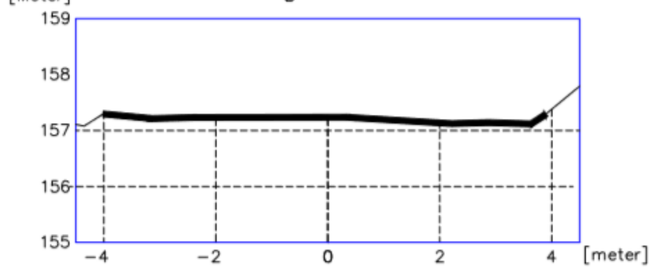
Reach 5
chainage 0-37m



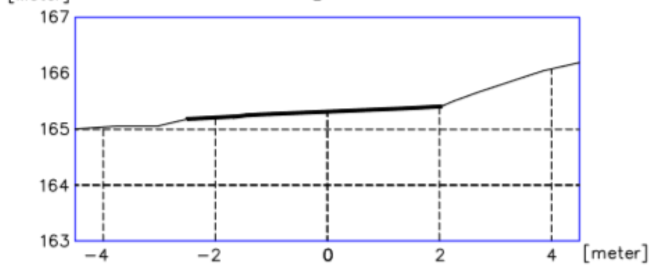
Reach 6
chainage 0-101m



Reach 6
chainage 101-114m

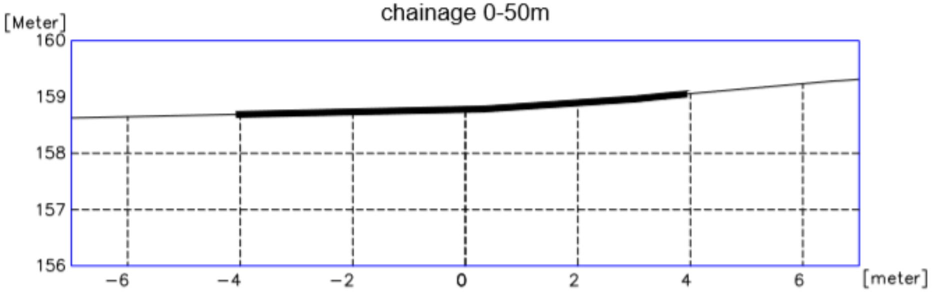


Reach 7
chainage 0-148m



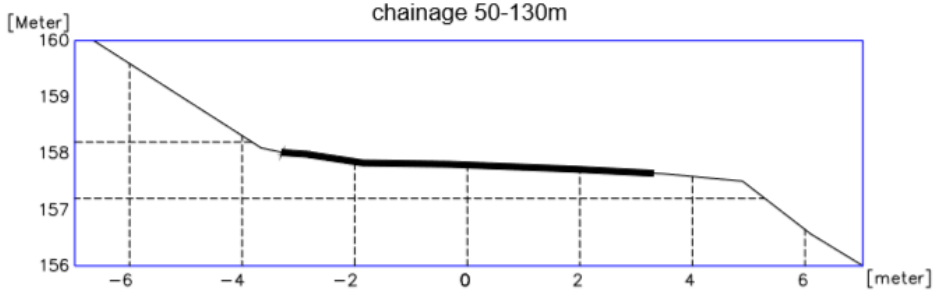
Reach 8

chainage 0-50m



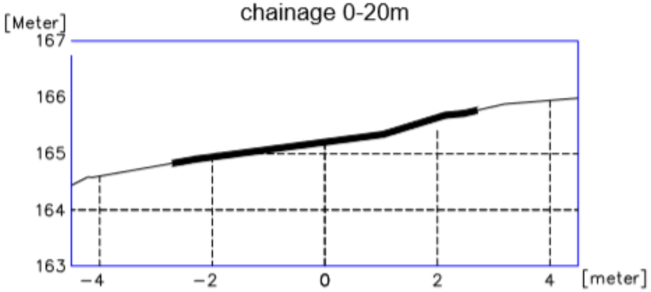
Reach 8

chainage 50-130m



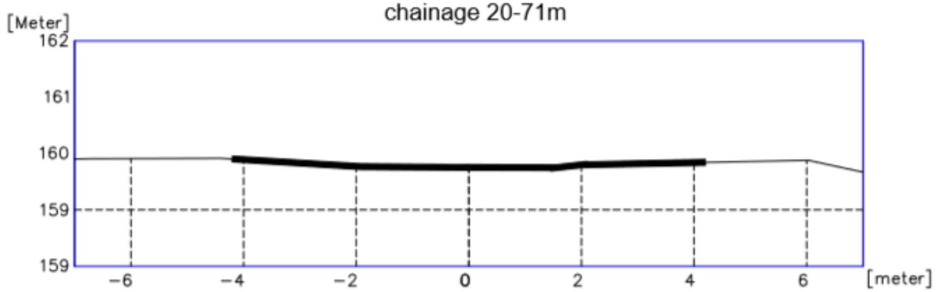
Reach 9

chainage 0-20m



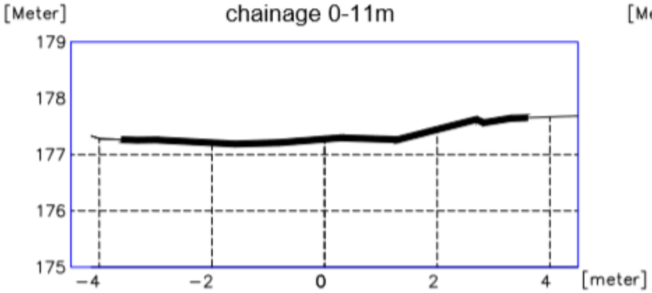
Reach 9

chainage 20-71m



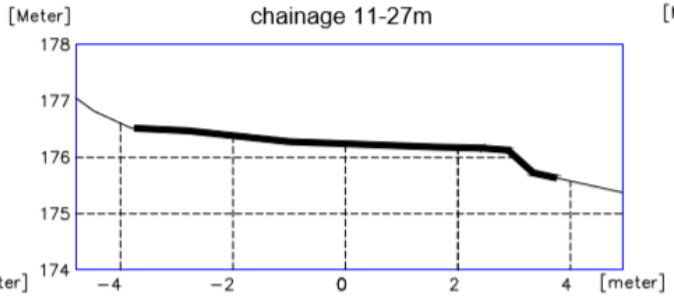
Reach 10

chainage 0-11m



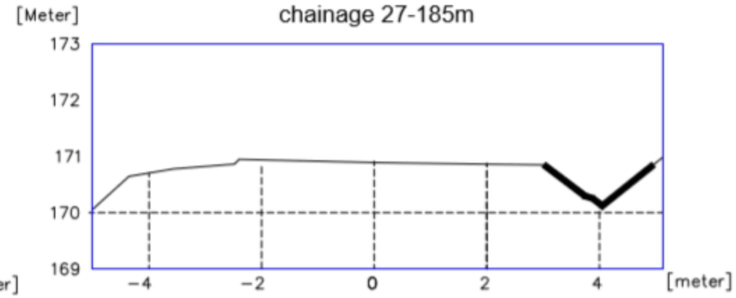
Reach 10

chainage 11-27m



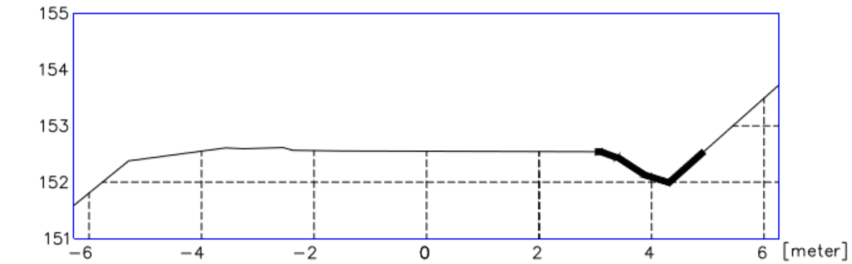
Reach 10

chainage 27-185m



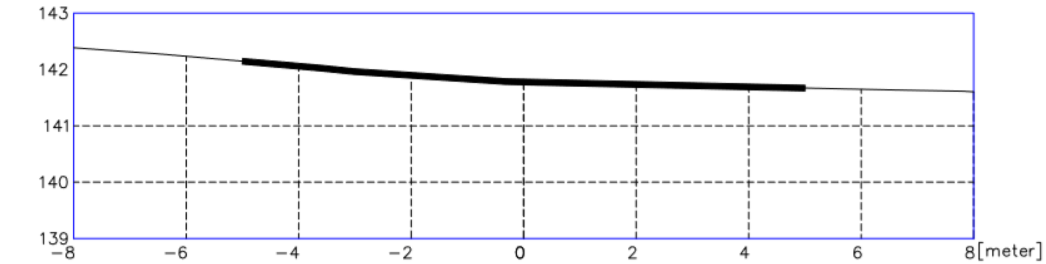
Reach 10

chainage 185-250m



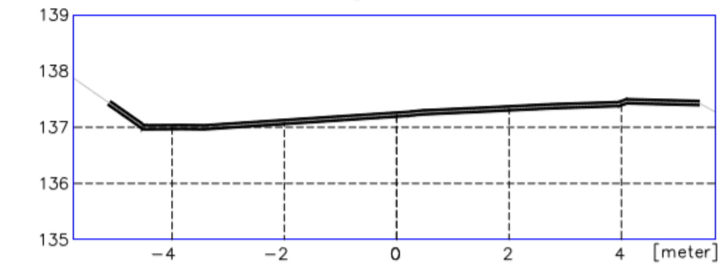
Reach 10

chainage 250-279m



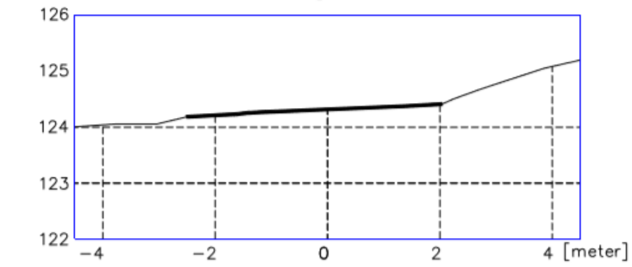
Reach 10

chainage 279-309m



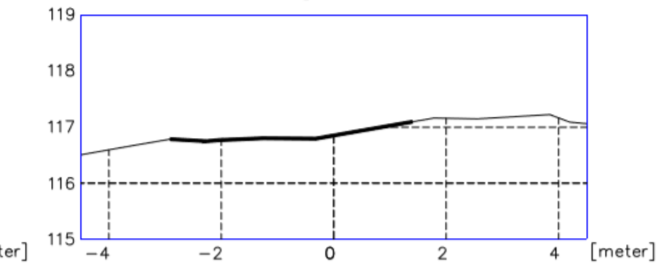
Reach 11

chainage 0-79m



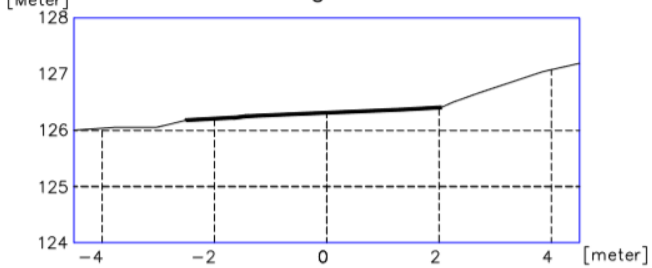
Reach 11

chainage 79-110m



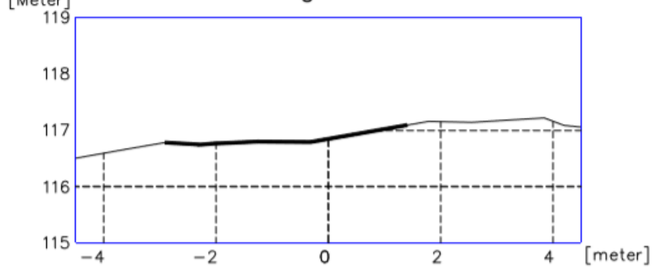
Reach 12

chainage 0-62m



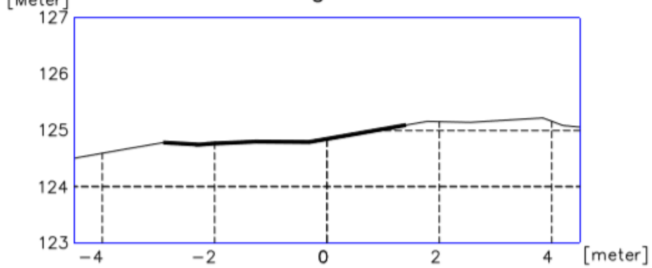
Reach 12

chainage 62-130m

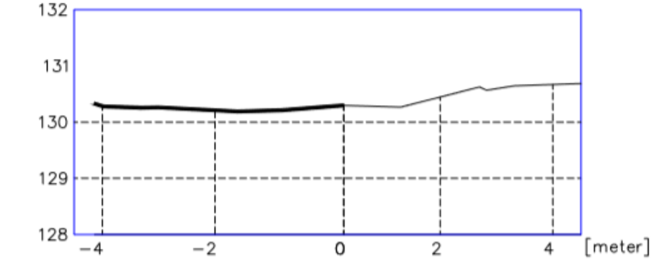


Reach 13

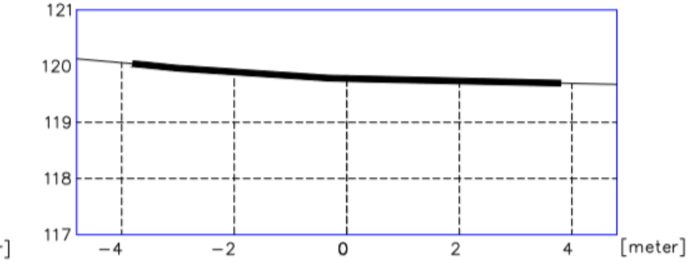
chainage 0-94m



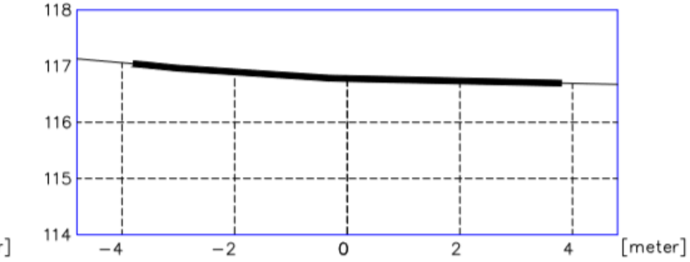
Reach 14
chainage 0-82m



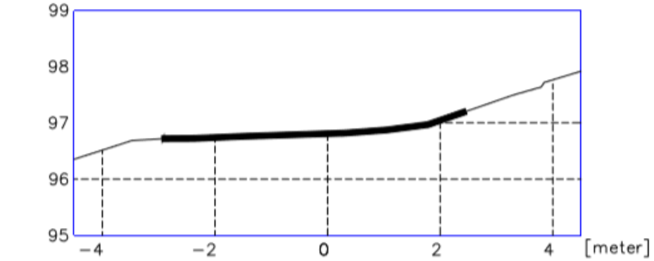
Reach 14
chainage 82-110m



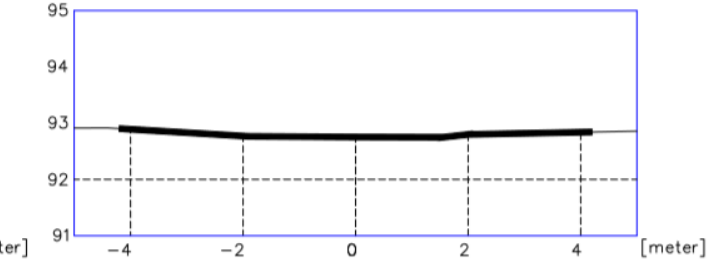
Reach 15
chainage 0-35m



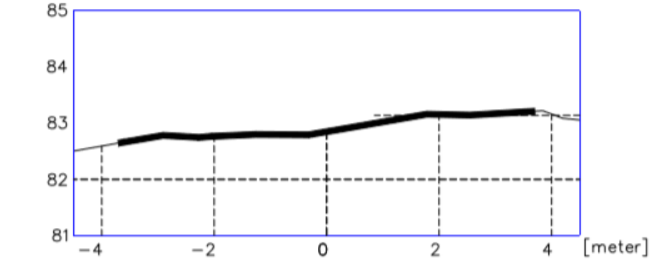
Reach 16
chainage 0-61m



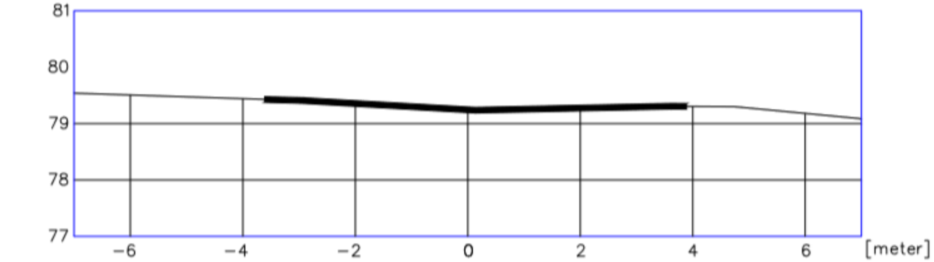
Reach 17
chainage 0-82m



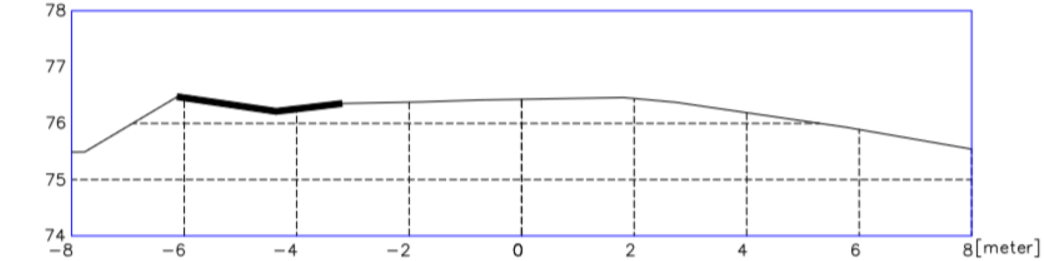
Reach 17
chainage 82-164m



Reach 18
chainage 0-36m

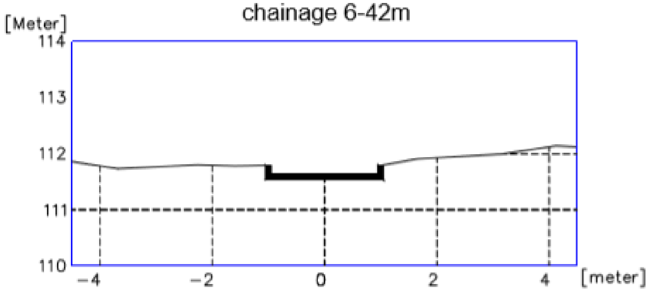


Reach 18
chainage 36-49m



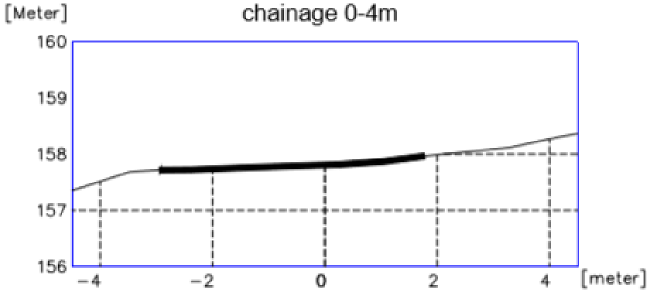
Reach 21

chainage 6-42m



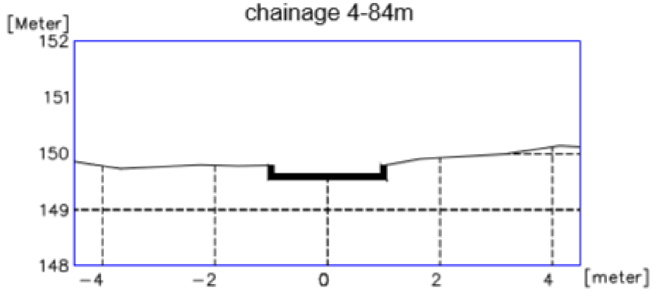
Reach 22

chainage 0-4m



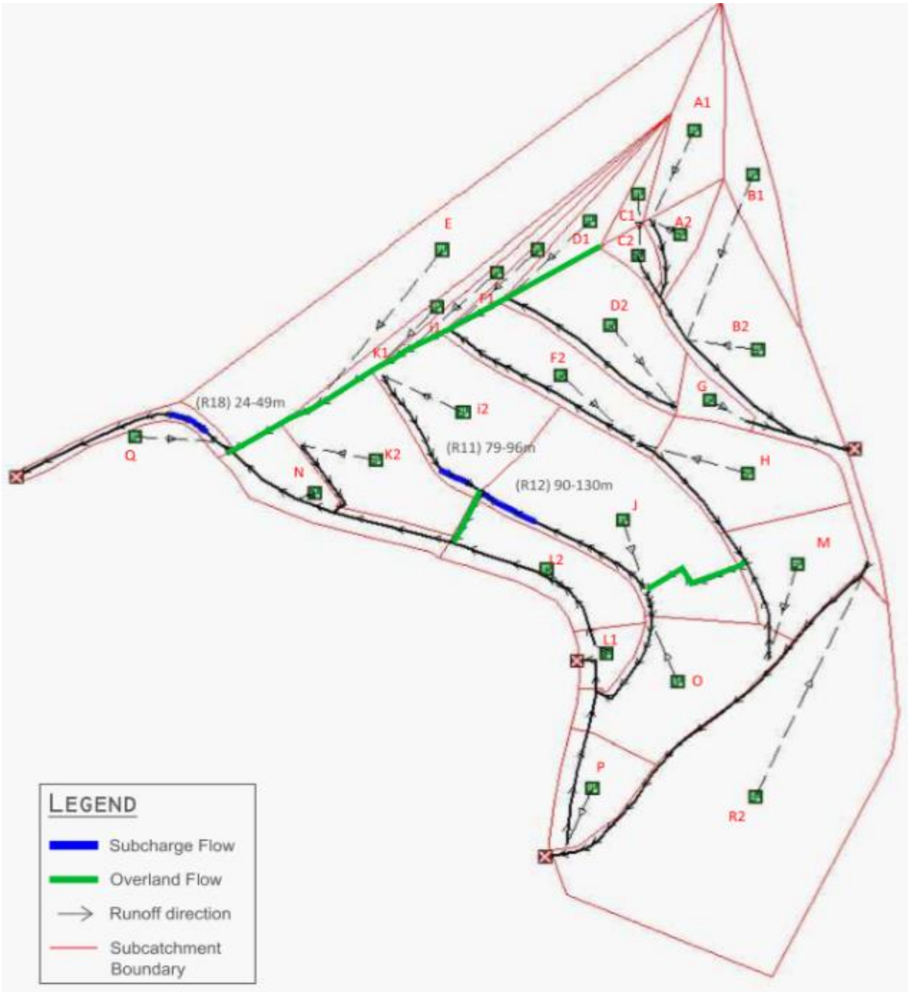
Reach 22

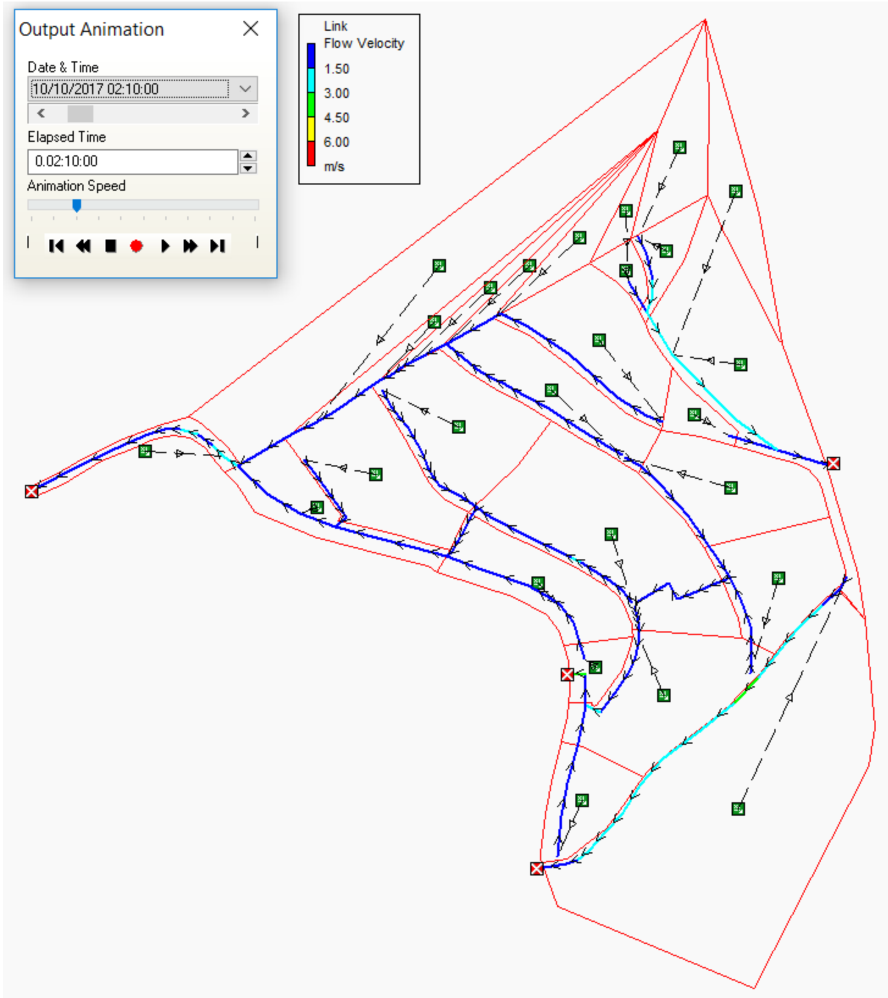
chainage 4-84m



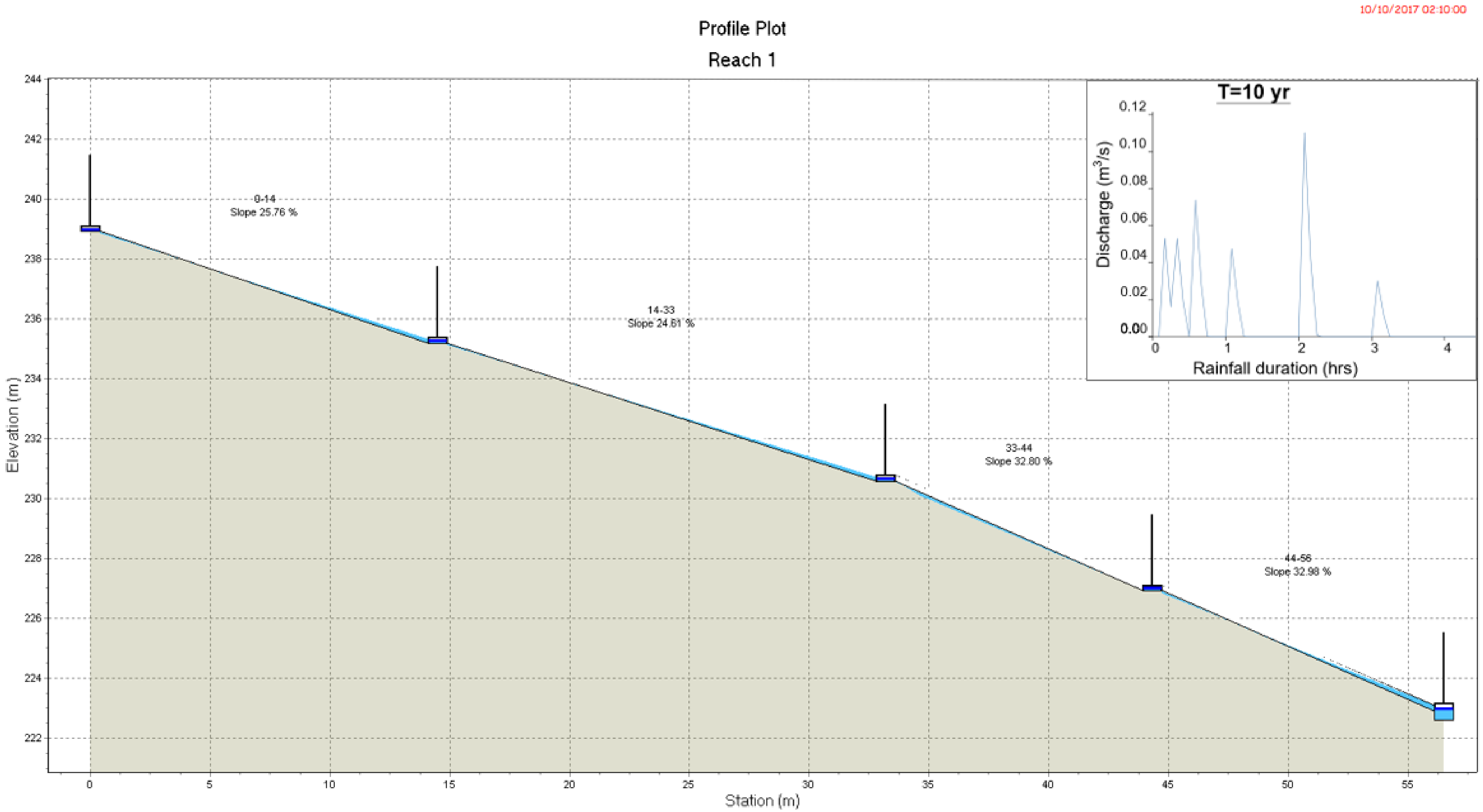
1.3. Simulation results

1.3.1. Overview of the drainage network

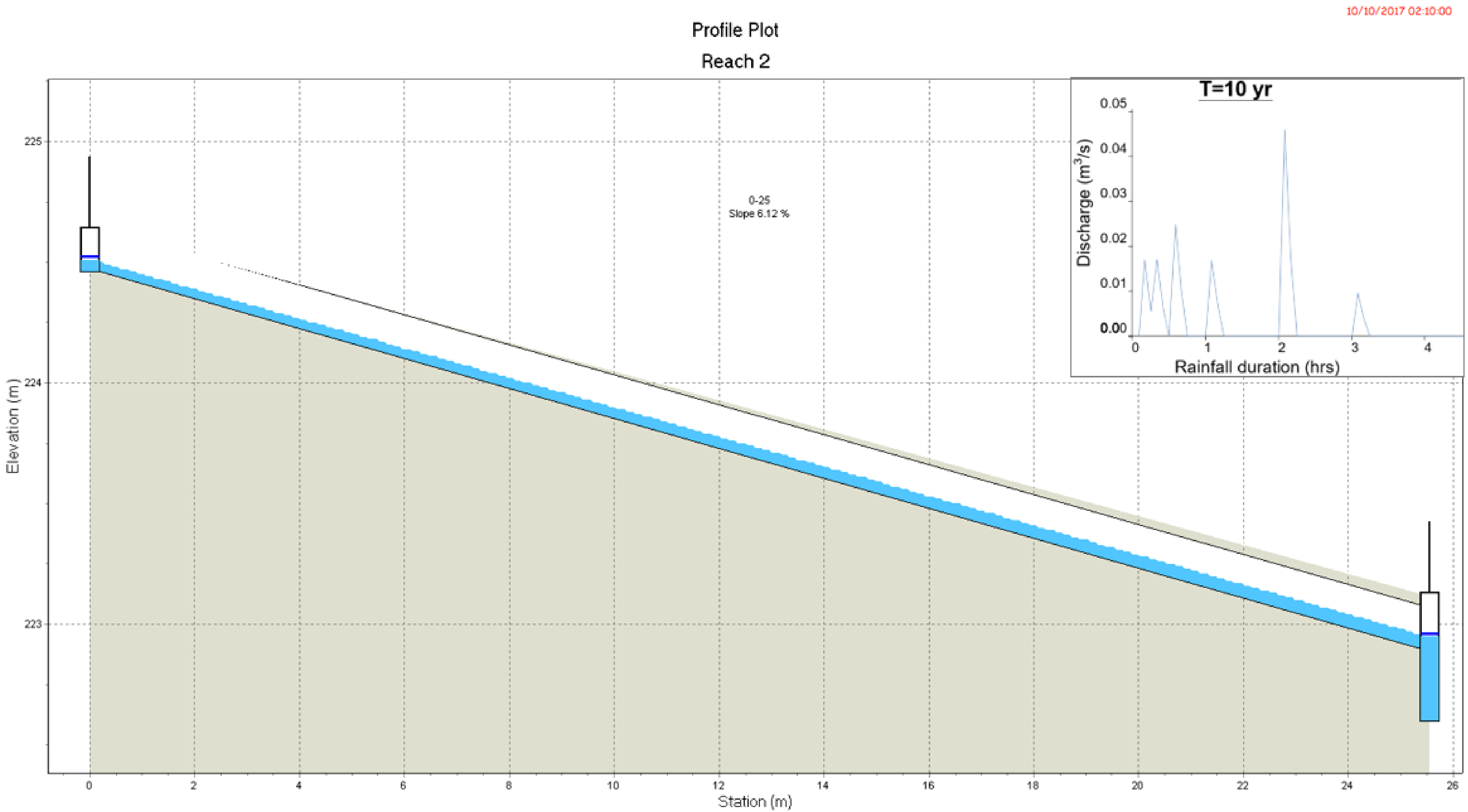




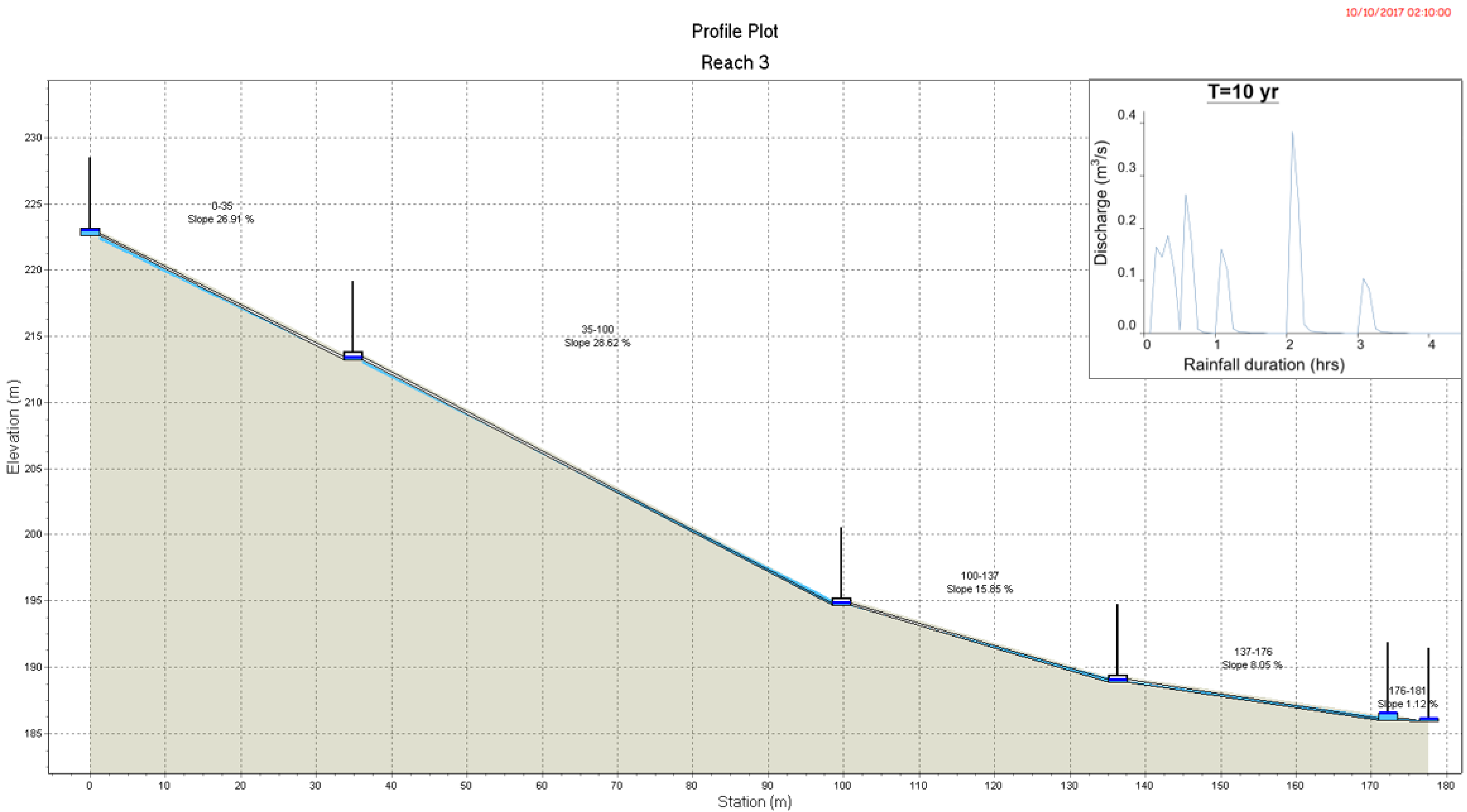
1.3.2. Longitudinal profile plots



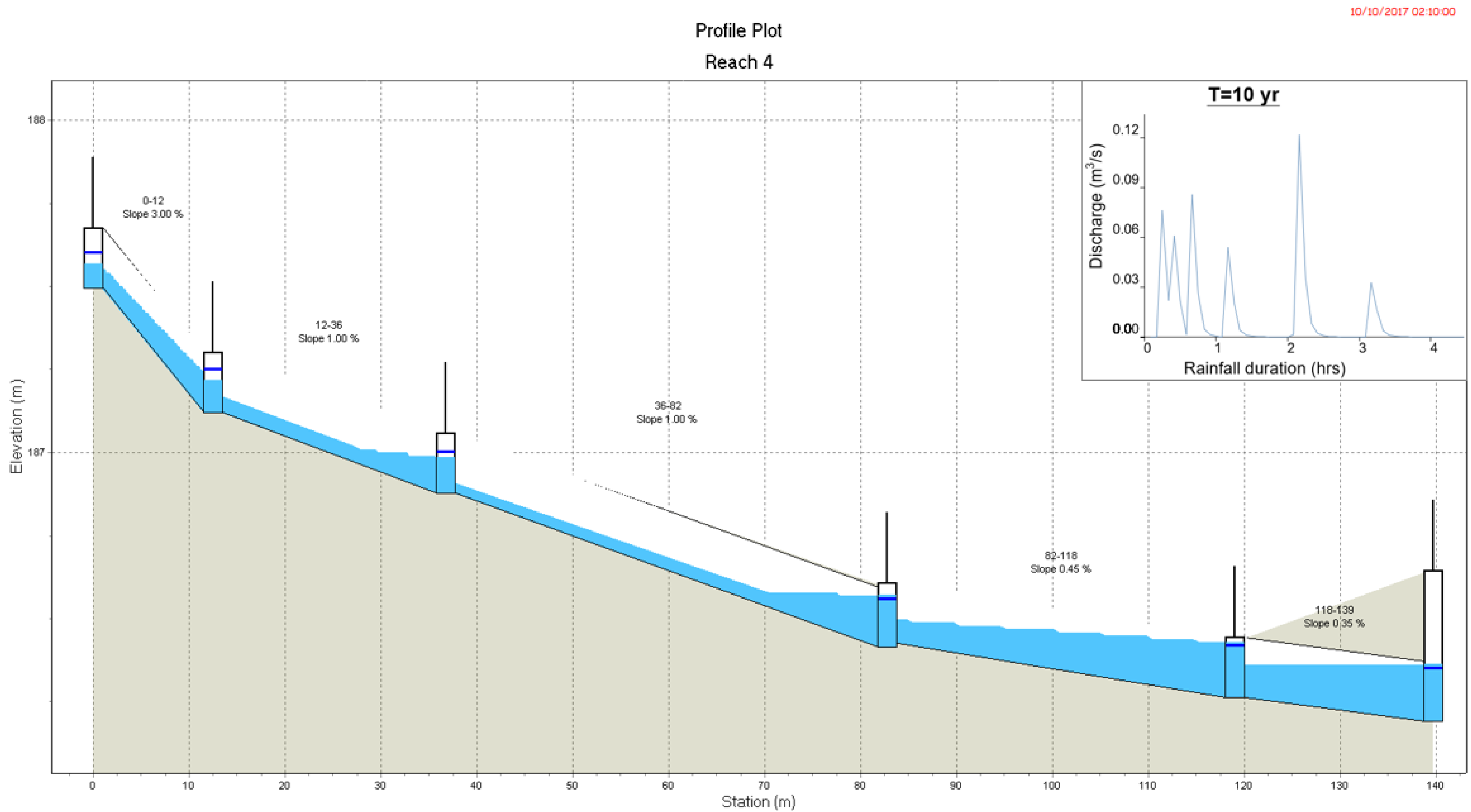
Chainage: (-)	0-14	14-33	33-44	44-56
Max vel (m/s)	1.88	1.85	2.05	2.05
Max depth (m)	0.07	0.07	0.07	0.07



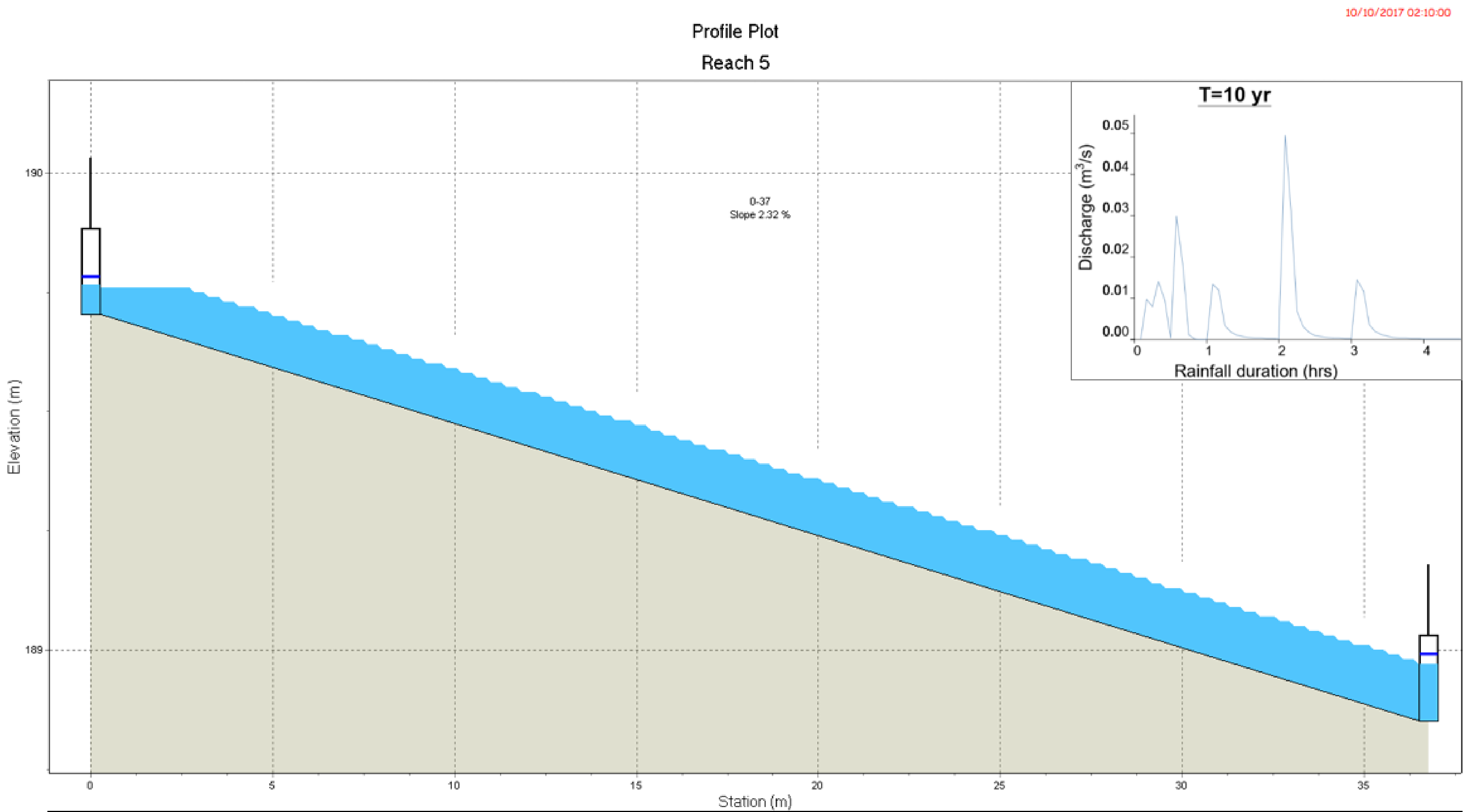
Chainage: (-)	0-25
Max vel (m/s)	1.71
Max depth (m)	0.07



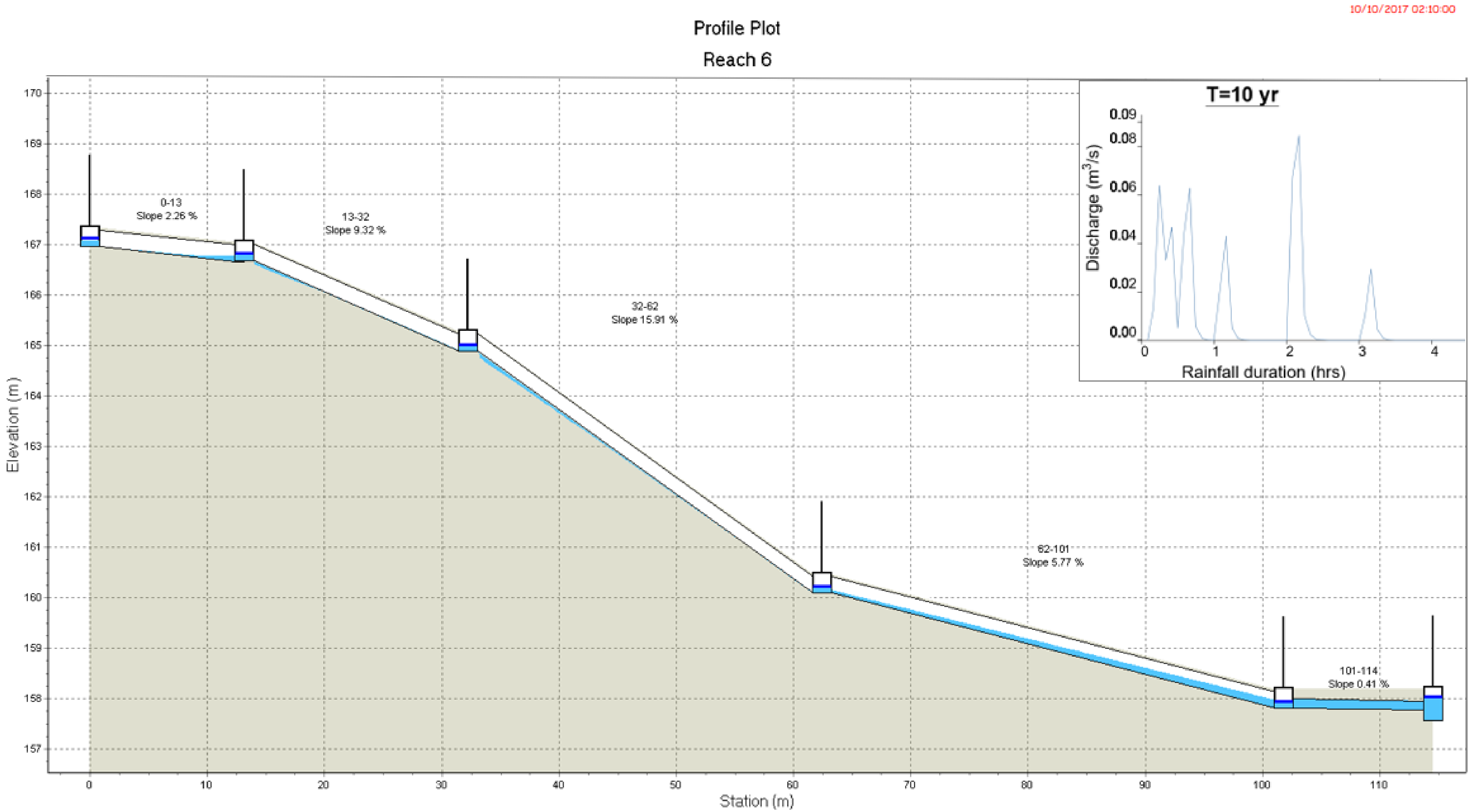
Chainage: (-)	0-35	35-100	100-137	137-176	176-181
Max vel (m/s)	2.10	2.69	2.14	1.71	0.69
Max depth (m)	0.08	0.11	0.12	0.14	0.18



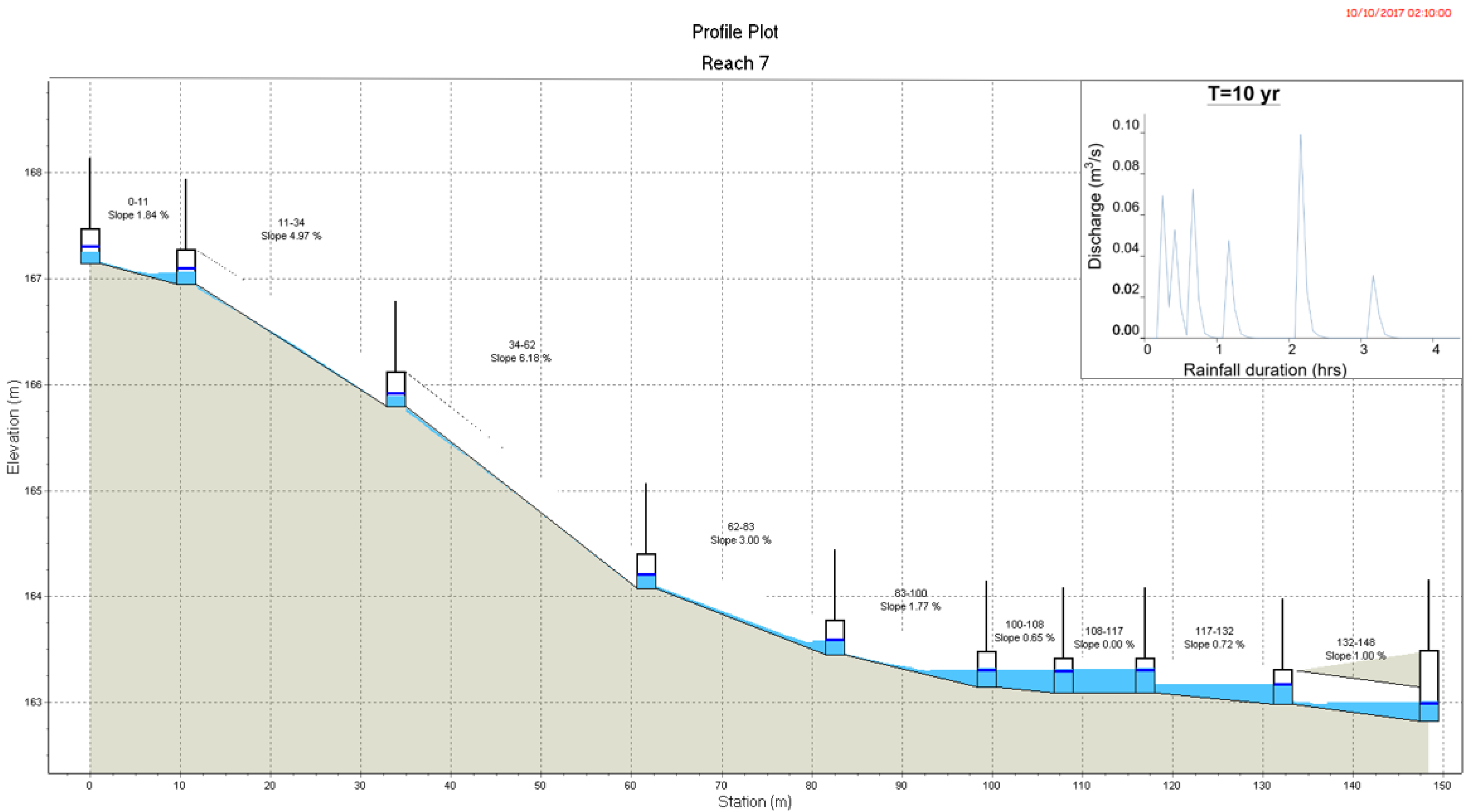
Chainage: (-)	0-12	12-36	36-86	82-118	118-139
Max vel (m/s)	0.86	0.62	0.80	0.80	0.38
Max depth (m)	0.11	0.13	0.12	0.15	0.16



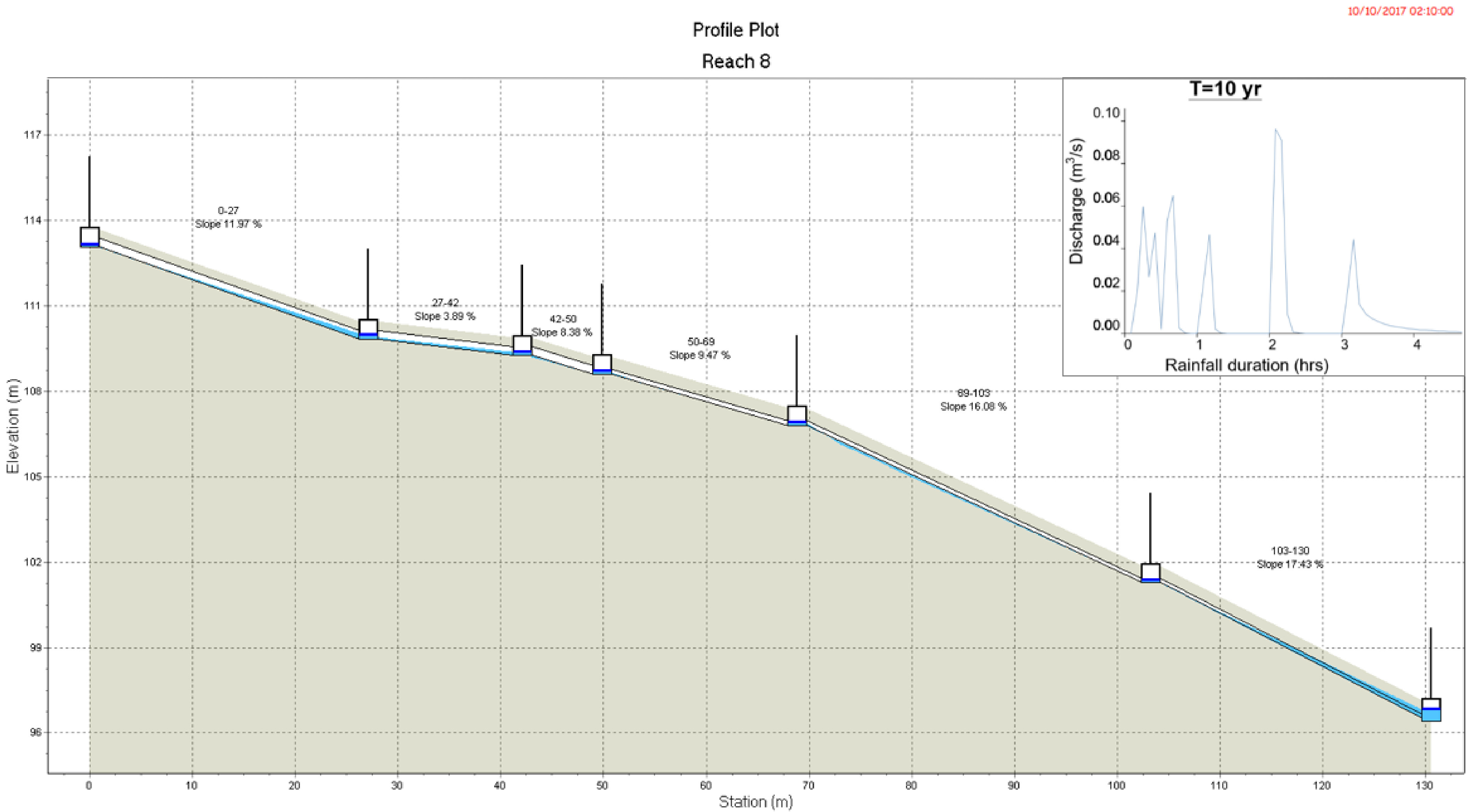
Chainage: (-)	0-37
Max vel (m/s)	1.12
Max depth (m)	0.09



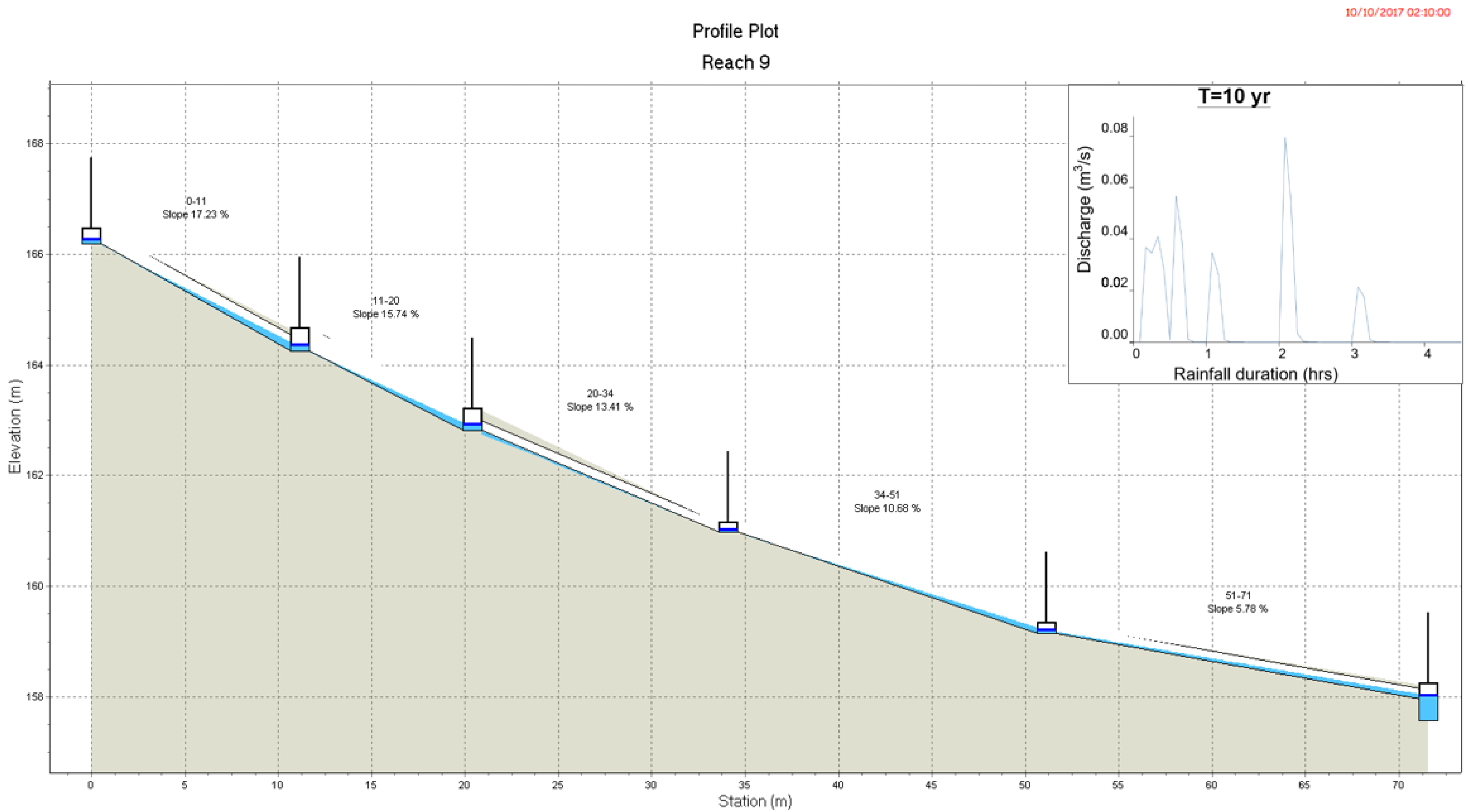
Chainage: (-)	0-13	13-32	32-62	62-101	101-114
Max vel (m/s)	0.69	1.16	1.42	0.99	0.27
Max depth (m)	0.16	0.12	0.11	0.12	0.10



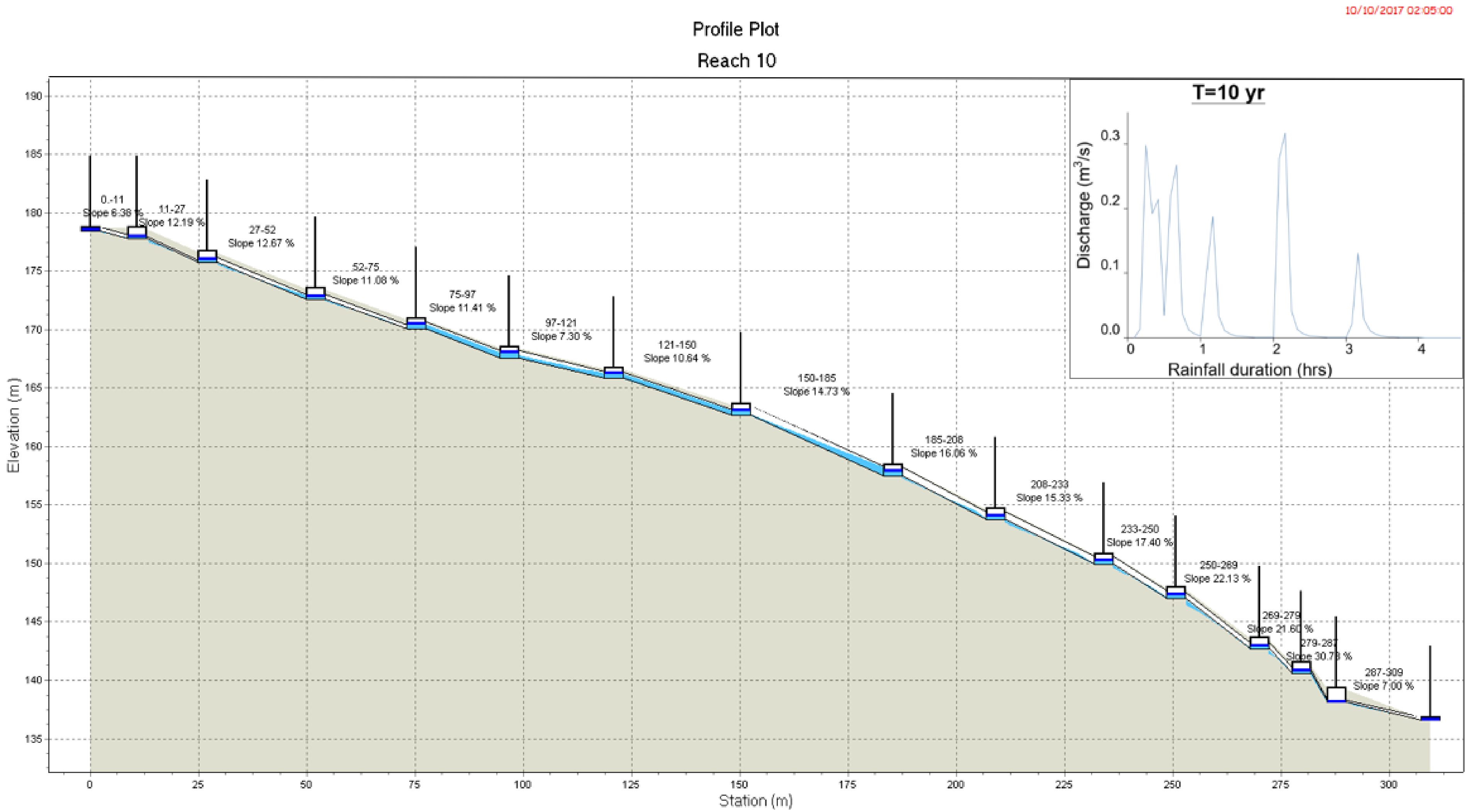
Chainage: (-)	0-11	11-34	34-62	62-83	83-100	100-108	108-117	117-132	132-148
Max vel (m/s)	0.62	0.89	0.96	0.73	0.60	0.41	0.26	0.43	0.48
Max depth (m)	0.16	0.13	0.12	0.13	0.14	0.17	0.21	0.18	0.17



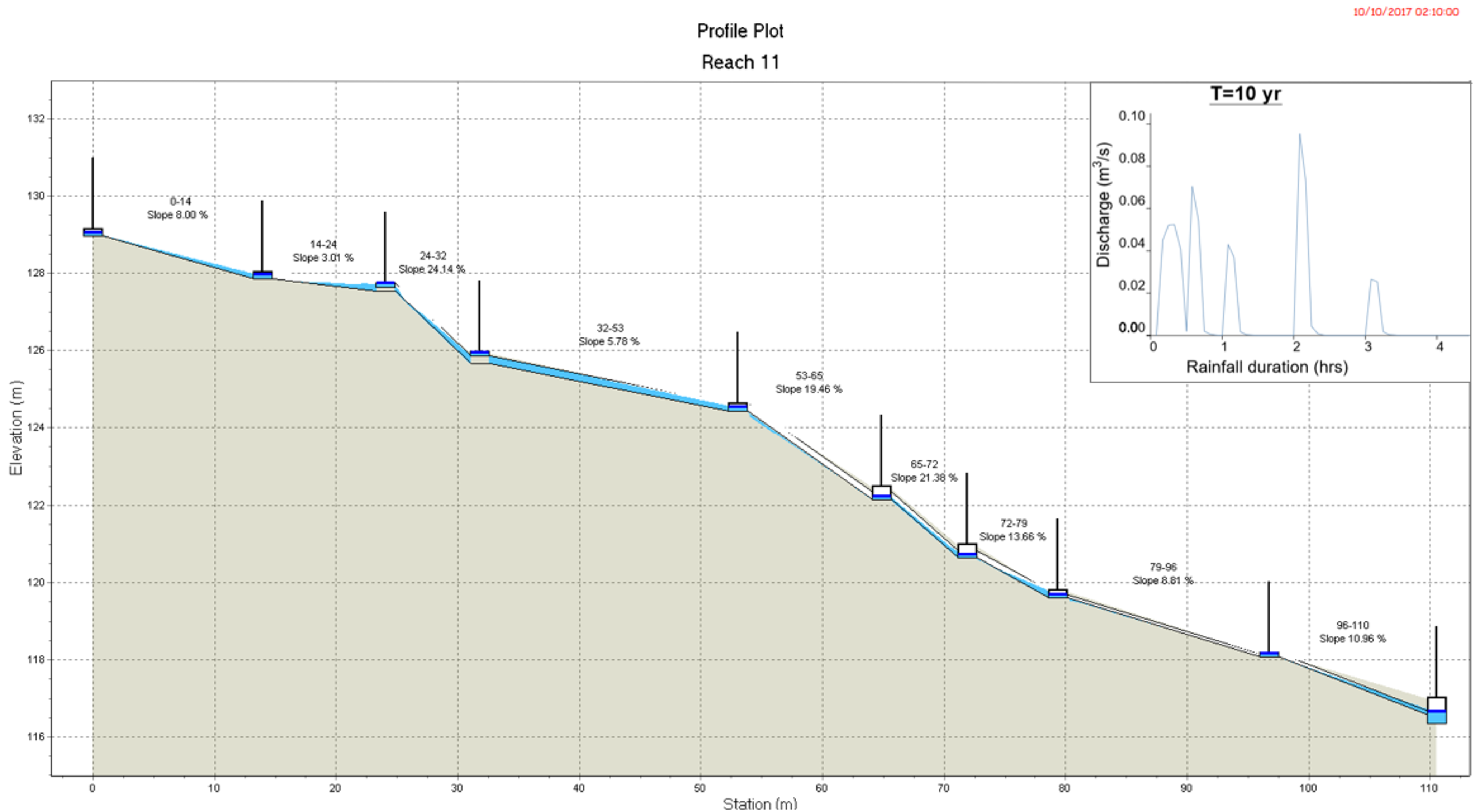
Chainage: (-)	0-27	27-42	42-50	50-69	69-103	103-130
Max vel (m/s)	1.60	0.78	1.04	0.92	1.18	1.23
Max depth (m)	0.06	0.12	0.10	0.08	0.08	0.08



Chainage: (-)	0-11	11-20	20-34	34-51	51-71
Max vel (m/s)	1.31	1.43	0.88	0.81	0.65
Max depth (m)	0.09	0.11	0.06	0.06	0.06

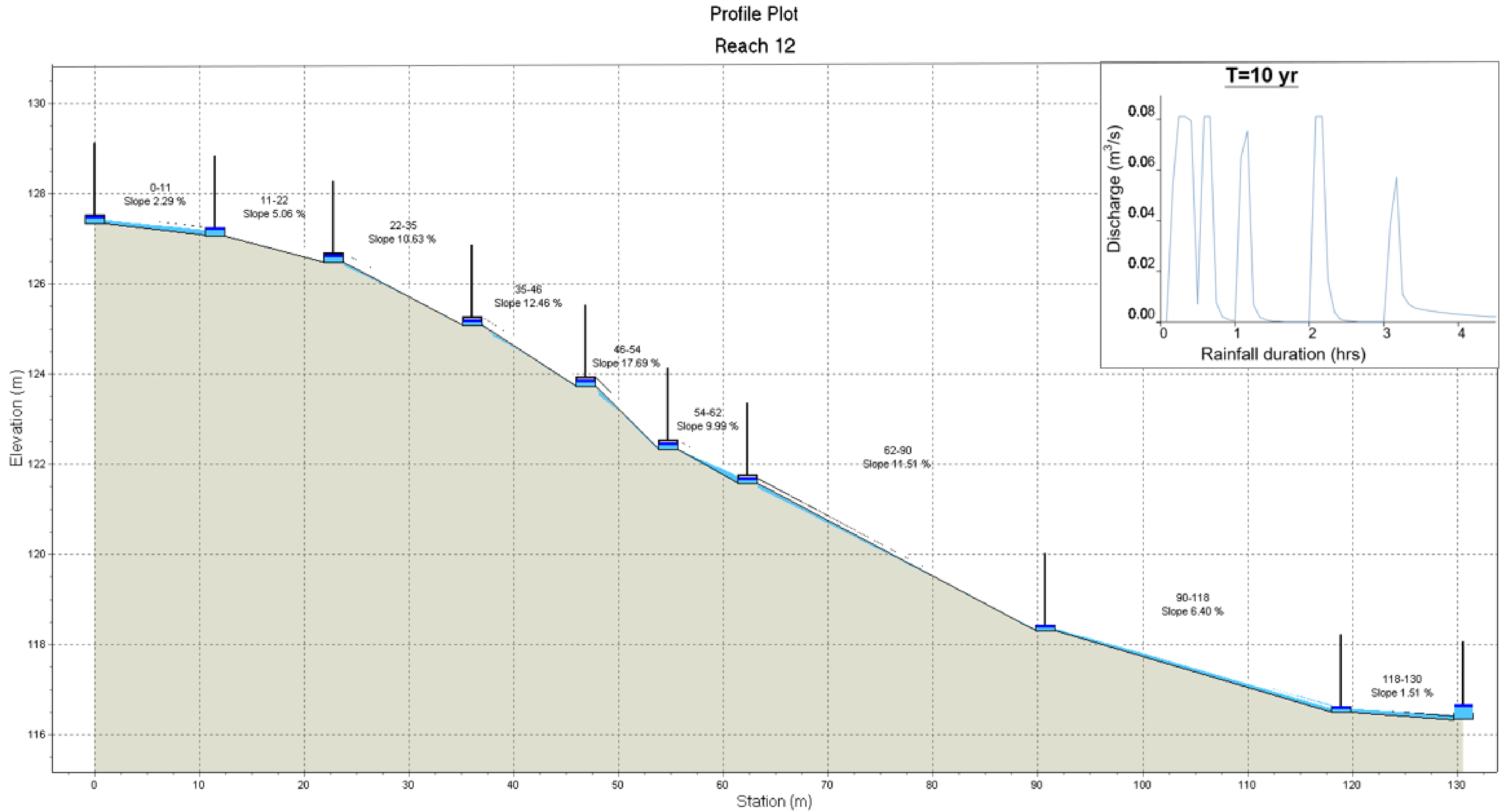


Chainage: (-)	0-11	11-27	27-52	52-75	75-97	97-121	121-150	150-185	185-208	208-253	233-250	250-269	269-279	279-287	287-309
Max vel (m/s)	1.24	1.43	2.87	2.73	2.09	4.59	2.02	2.29	2.78	2.73	2.86	2.00	1.98	1.43	0.91
Max depth (m)	0.10	0.17	0.23	0.23	0.46	0.31	0.40	0.38	0.34	0.33	0.32	0.20	0.20	0.03	0.04



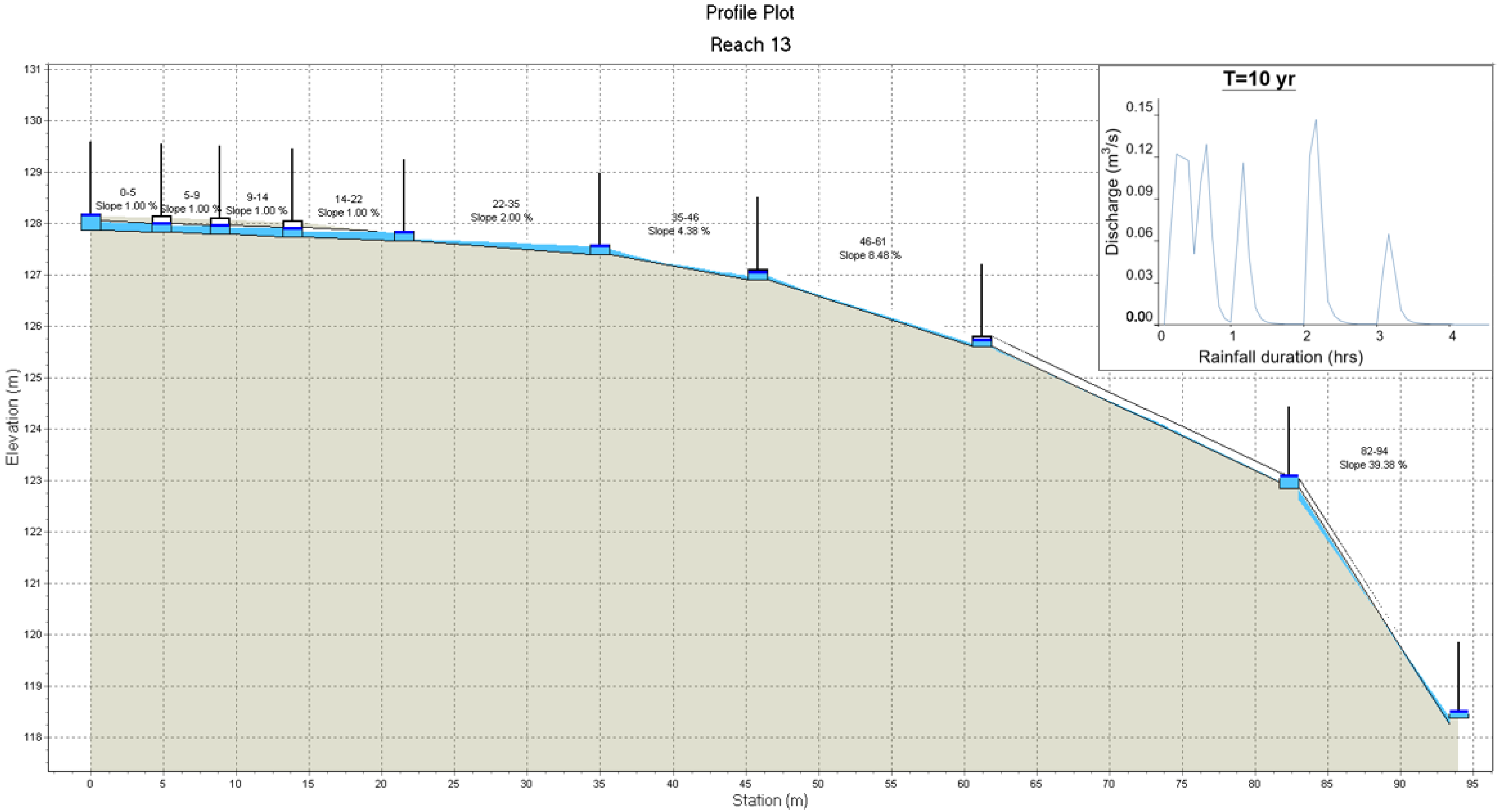
Chainage: (-)	0-14	14-24	24-32	32-53	53-65	65-72	72-79	79-96	96-110
Max vel (m/s)	1.23	0.74	1.82	1.15	1.70	1.76	1.49	0.80	0.97
Max depth (m)	0.10	0.12	0.08	0.10	0.08	0.07	0.08	0.07	0.07

10/10/2017 02:10:00

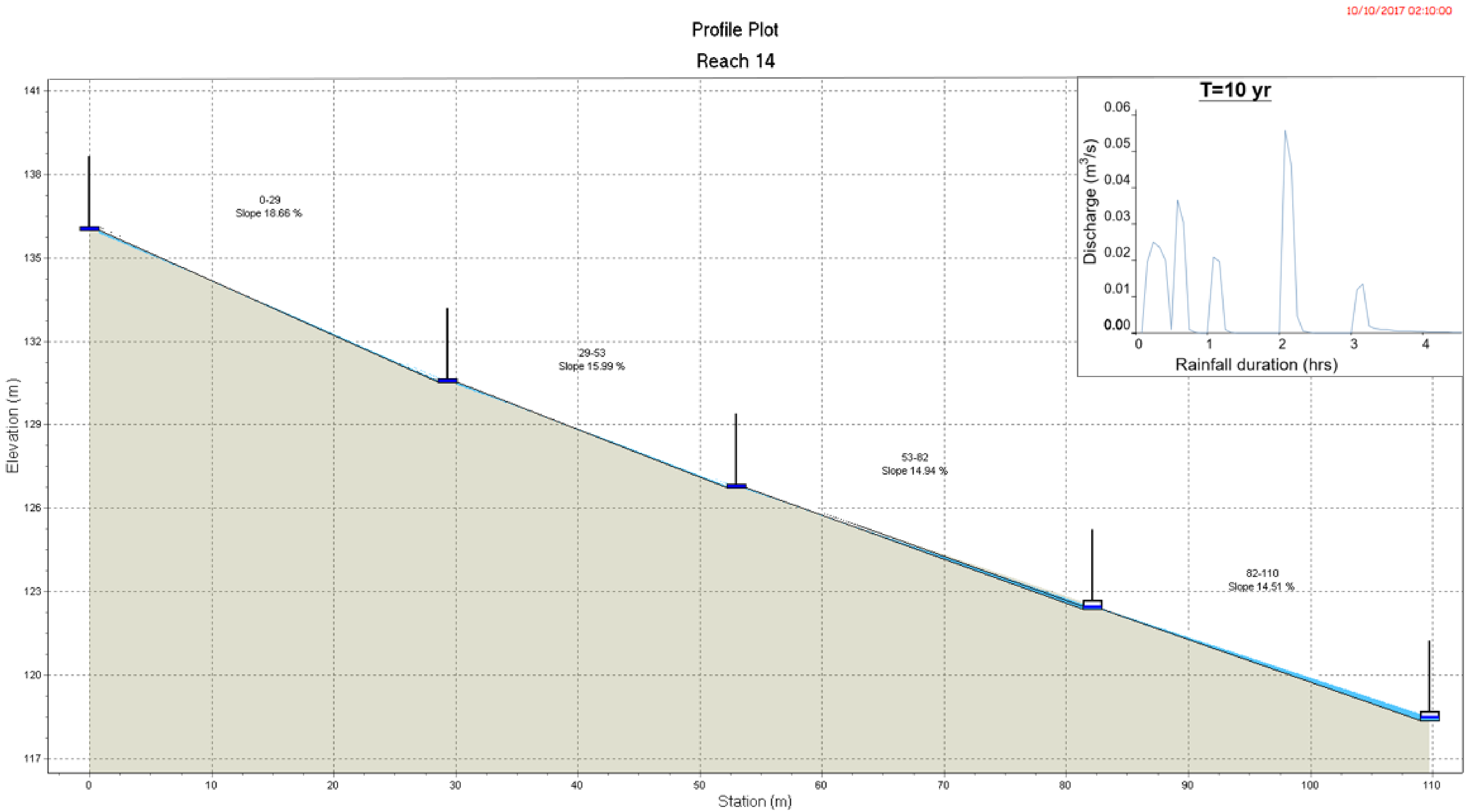


Chainage: (-)	0-11	11-22	22-35	35-46	46-54	54-62	62-90	90-118	118-130
Max vel (m/s)	0.88	1.18	1.56	1.65	1.88	1.52	1.17	0.89	0.45
Max depth (m)	0.15	0.13	0.11	0.10	0.10	0.10	0.09	0.10	0.10

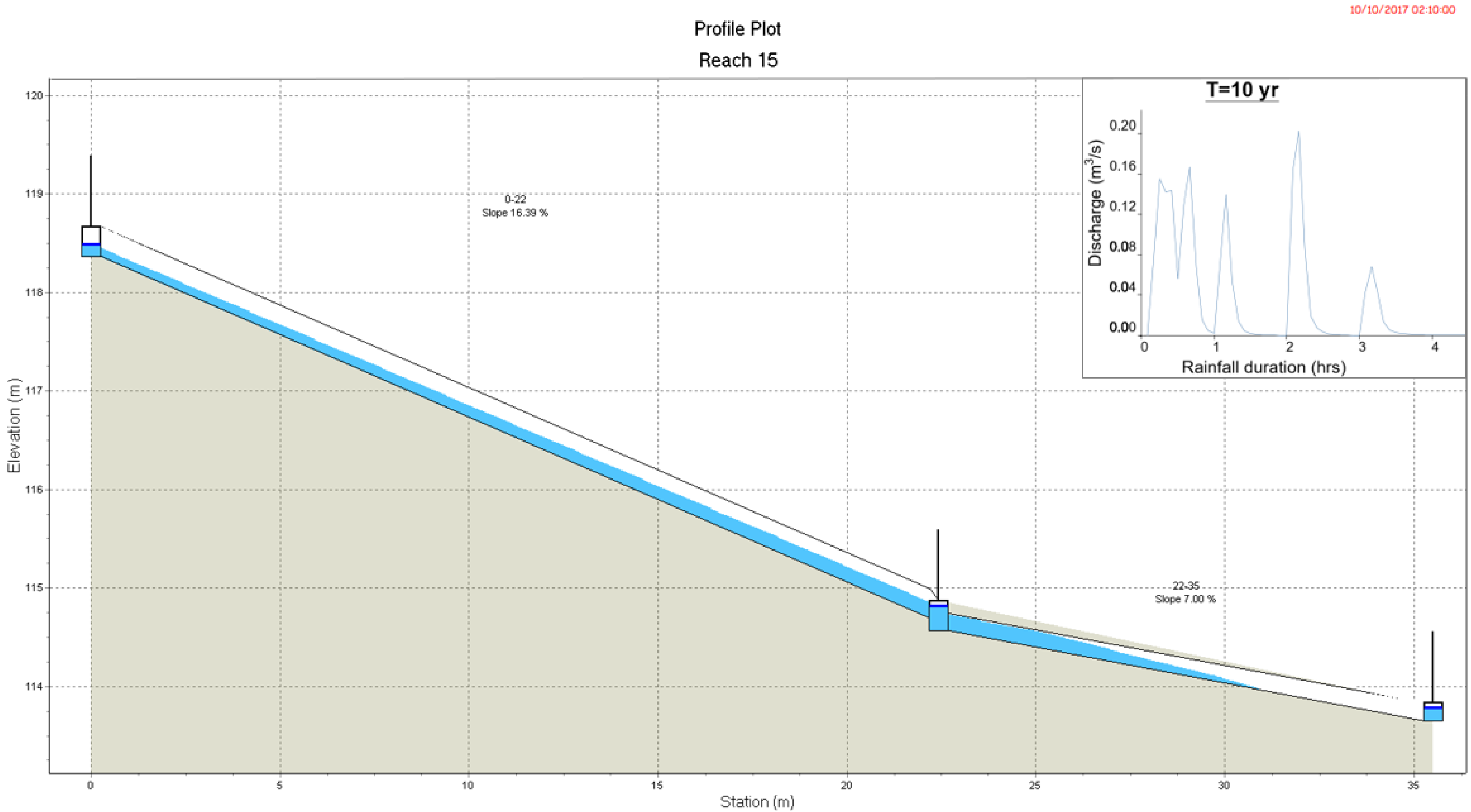
10/10/2017 02:10:00



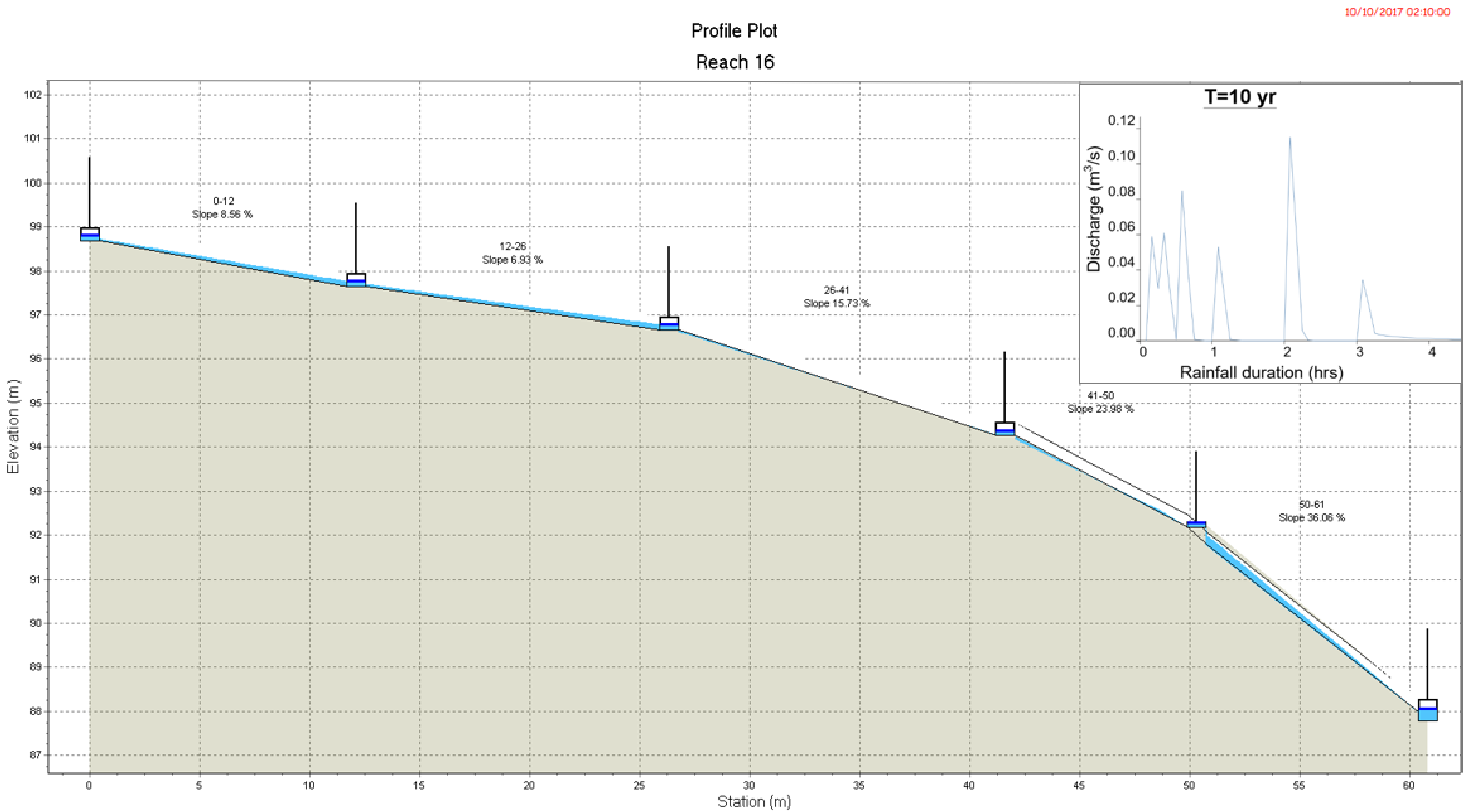
Chainage: (-)	0-5	5-9	9-14	14-22	22-35	35-46	46-61	61-82	82-94
Max vel (m/s)	0.64	0.64	0.64	0.64	0.89	1.19	1.53	1.77	2.68
Max depth (m)	0.16	0.16	0.17	0.17	0.16	0.14	0.12	0.12	0.09



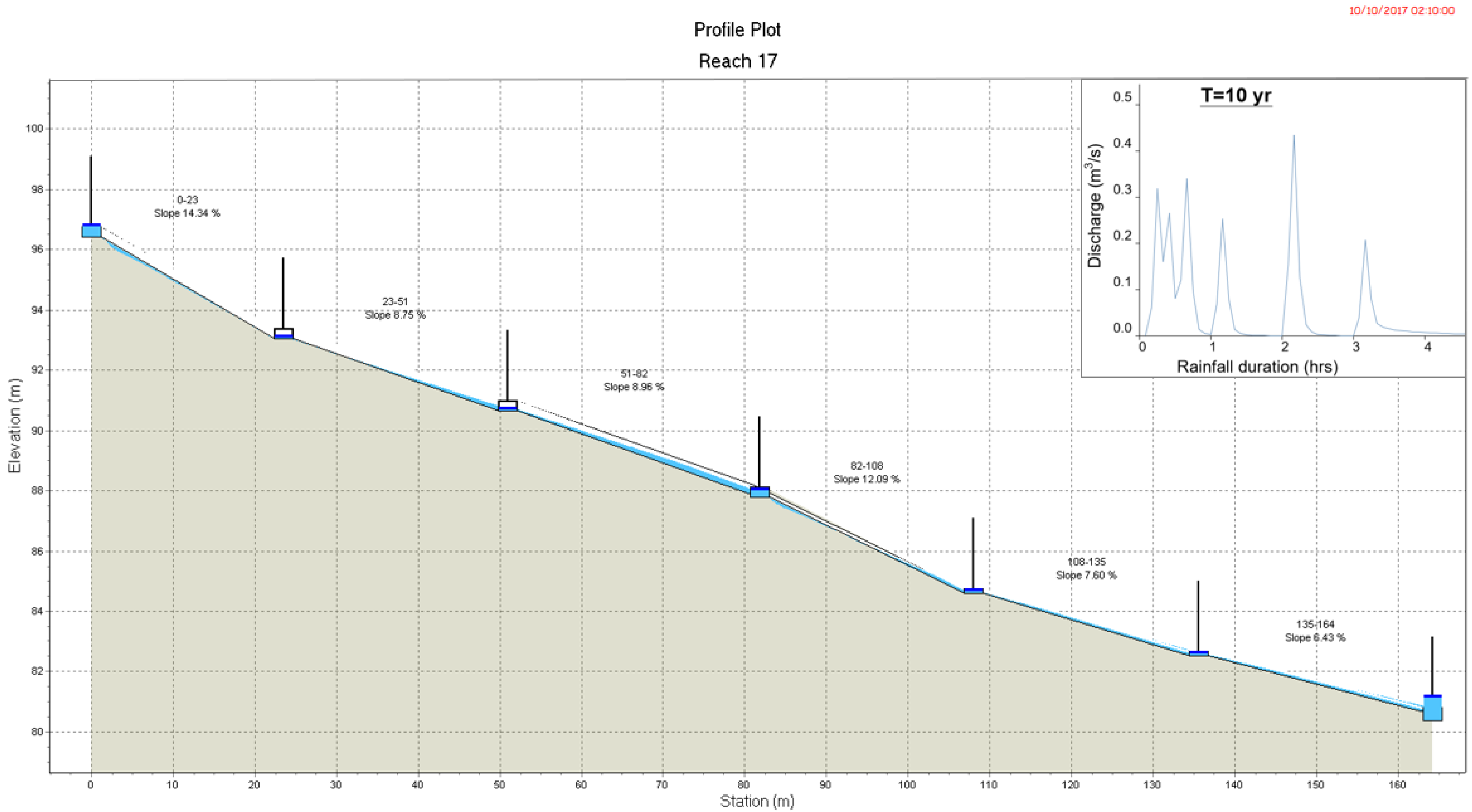
Chainage: (-)	0-29	29-53	53-82	82-110
Max vel (m/s)	1.56	1.15	1.12	1.10
Max depth (m)	0.05	0.05	0.05	0.07



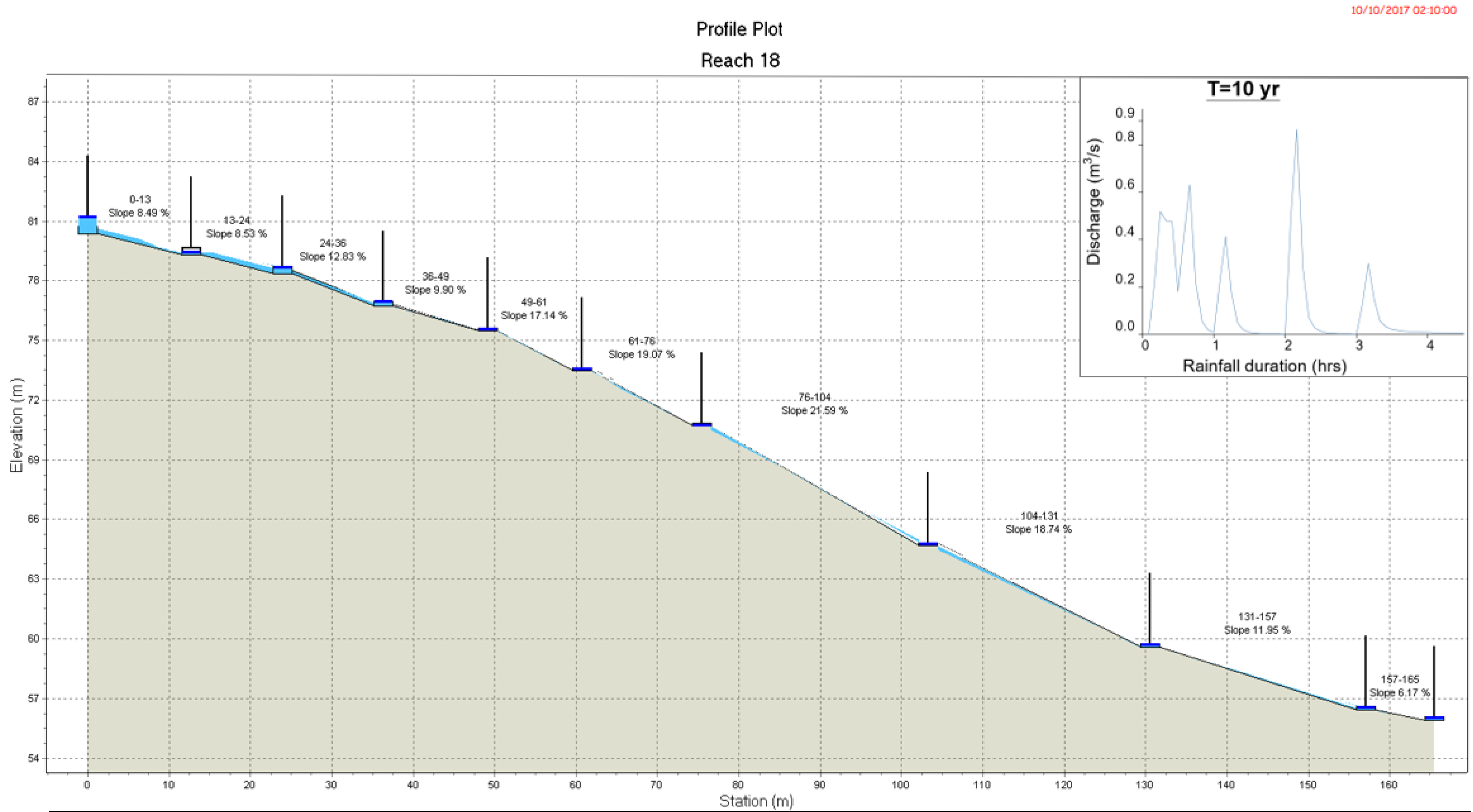
Chainage: (-)	0-22	22-35
Max vel (m/s)	1.65	1.49
Max depth (m)	0.13	0.14



Chainage: (-)	0-12	12-26	26-41	41-50	50-61
Max vel (m/s)	1.07	0.99	1.34	1.57	1.90
Max depth (m)	0.11	0.12	0.10	0.09	0.08

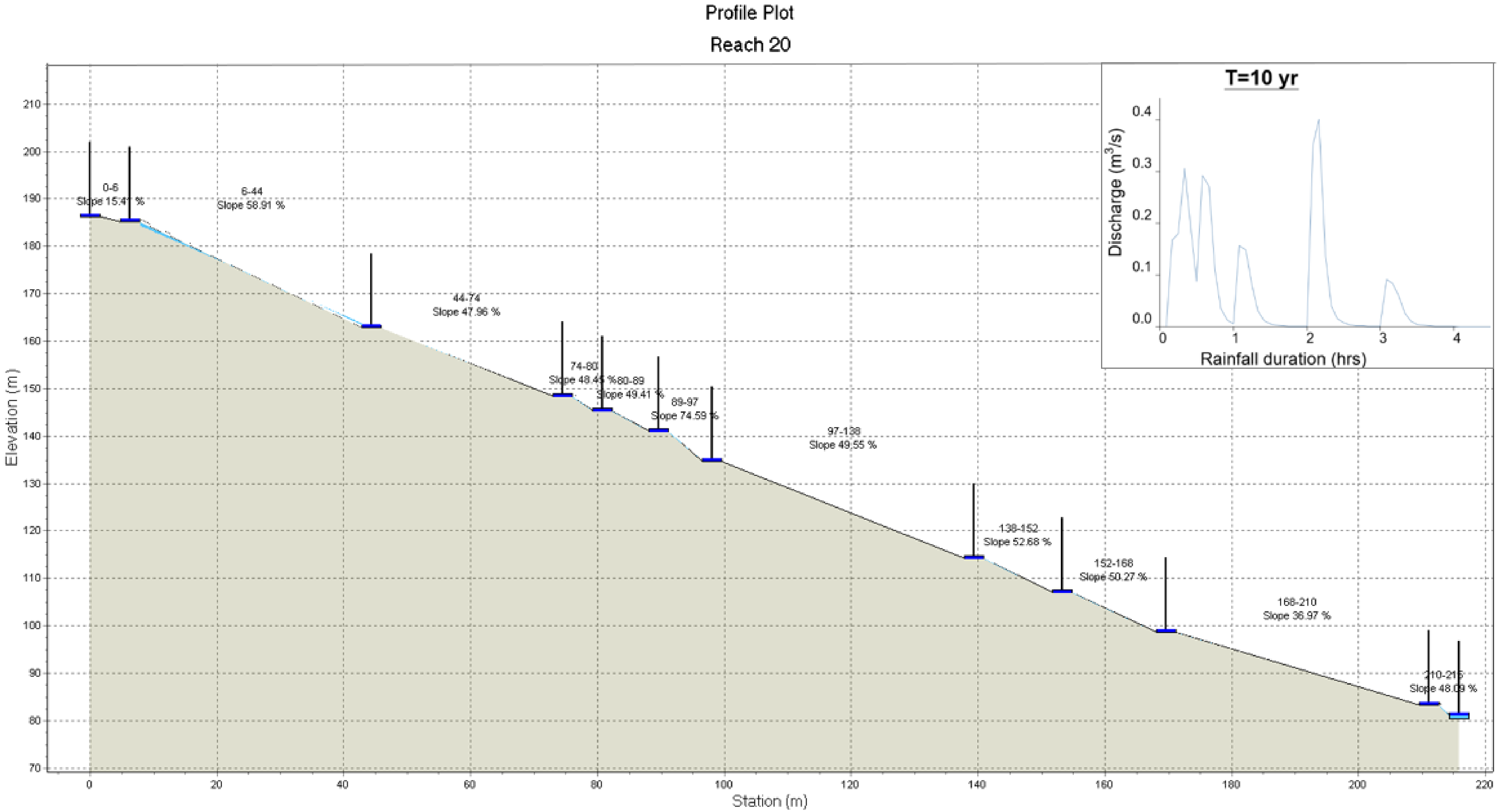


Chainage: (-)	0-23	23-51	51-82	82-108	108-135	135-164
Max vel (m/s)	1.45	1.23	1.24	1.57	1.37	1.28
Max depth (m)	0.06	0.07	0.07	0.09	0.10	0.11

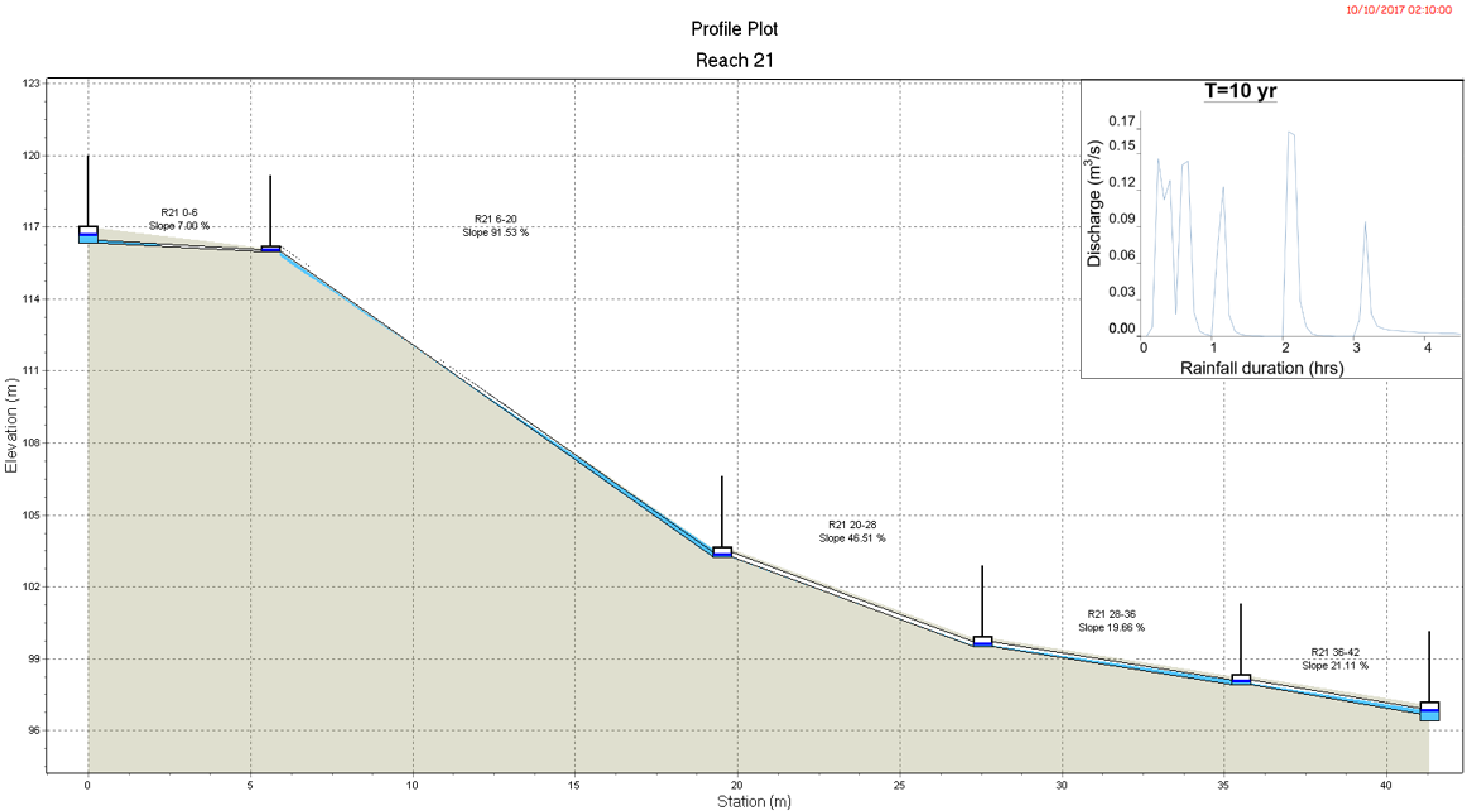


Chainage: (-)	0-13	13-24	24-66	36-49	49-61	61-76	76-104	104-131	131-157	157-165
Max vel (m/s)	1.71	1.71	1.37	1.93	0.87	1.11	1.24	1.18	1.57	1.09
Max depth (m)	0.13	0.13	0.18	0.05	0.02	0.03	0.04	0.05	0.08	0.10

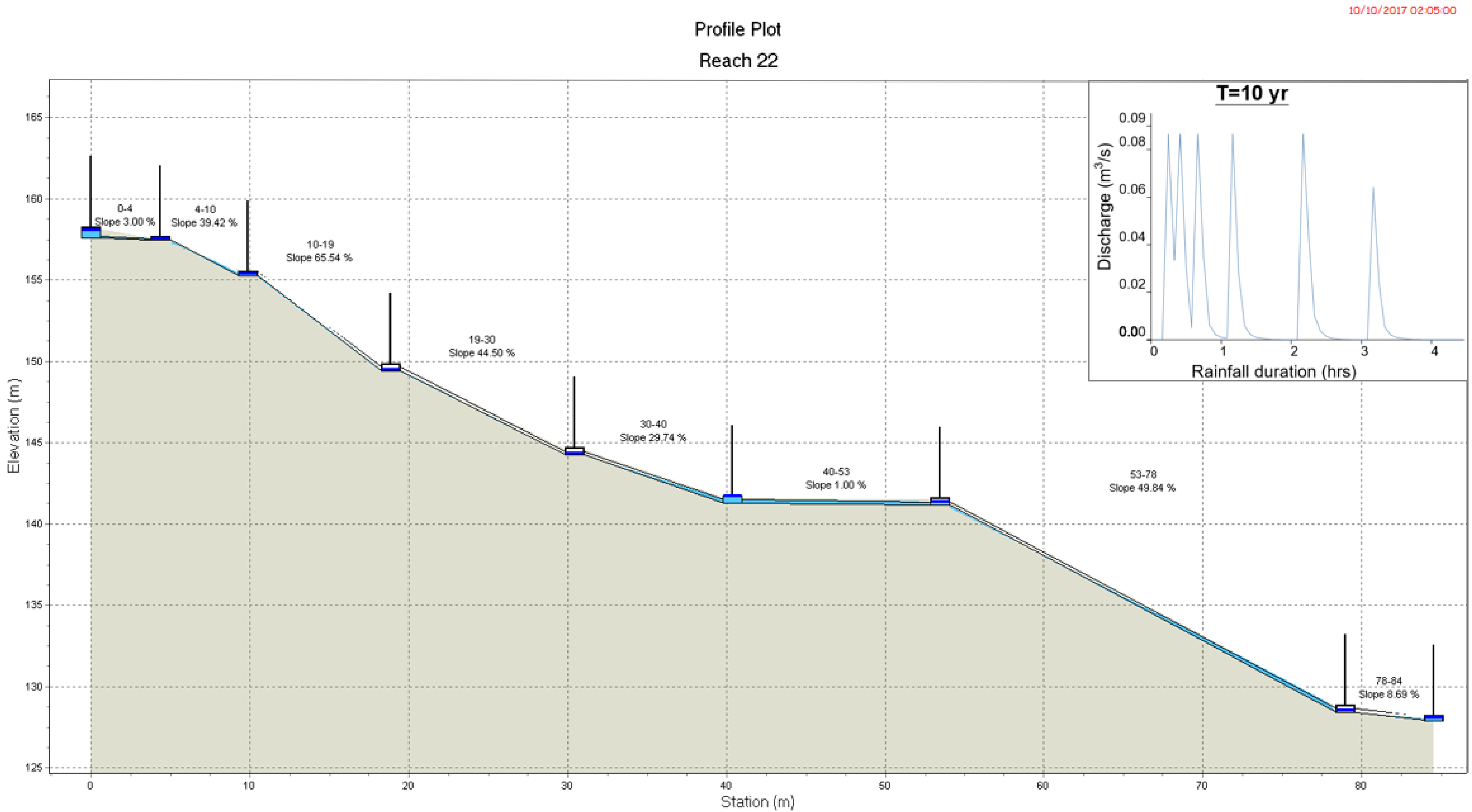
10/10/2017 02:05:00



Chainage: (-)	0-6	6-44	44-74	74-80	80-89	89-97	97-138	138-152	152-168	168-210	210-215
Max vel (m/s)	0.63	0.95	1.12	1.14	1.15	1.31	1.18	1.20	1.39	1.26	1.36
Max depth (m)	0.11	0.08	0.12	0.12	0.12	0.11	0.12	0.12	0.15	0.16	0.15



Chainage: (-)	0-6	6-20	20-28	28-36	36-42
Max vel (m/s)	1.07	1.23	1.00	0.84	0.86
Max depth (m)	0.04	0.07	0.09	0.11	0.11



Chainage: (-)	0-4	4-10	10-19	19-30	30-40	40-53	53-78	78-84
Max vel (m/s)	0.67	1.00	1.18	1.04	0.92	0.23	0.74	0.43
Max depth (m)	0.03	0.09	0.08	0.09	0.10	0.20	0.06	0.10

2. Future urban development scenario (Model 1)

2.1. Drainage network characteristics

Table 2D: Properties of drainage network used for the current development scenario (Model 1).

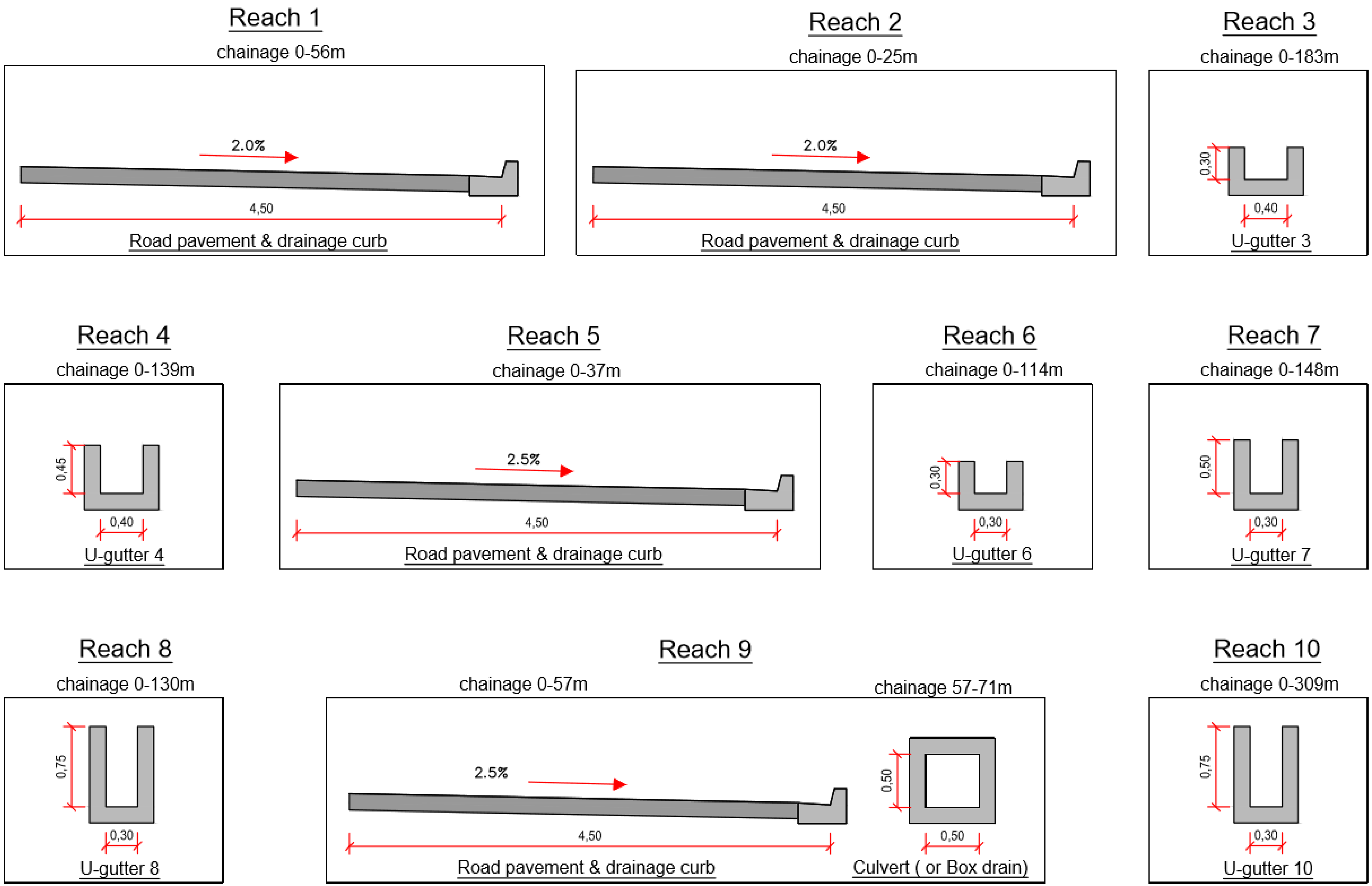
Reach	Chainage	Length	Inlet elevation	Outlet elevation	Average slope	Manning's roughness coefficient
	(m)	(m)	(m)	(m)	(%)	(-)
1	0-14	14.5	238.9	235.2	25.8	0.014
	14-33	18.7	235.2	230.6	24.6	0.014
	33-44	11.2	230.6	226.9	32.8	0.014
	44-56	12.2	226.9	222.9	33.0	0.014
2	0-25	25.6	224.5	222.9	6.1	0.014
3	0-35	34.8	222.6	213.2	26.9	0.014
	35-100	64.9	213.2	194.7	28.6	0.014
	100-137	36.6	194.7	188.9	15.9	0.014
	137-176	35.9	188.9	186.0	8.1	0.014
	176-181	5.4	186.0	185.9	1.1	0.014
4	0-12	12.5	187.5	187.1	3.0	0.014
	12-36	24.3	187.1	186.9	1.0	0.014
	36-82	46.1	186.9	186.4	1.0	0.014
	82-118	36.3	186.4	186.3	0.5	0.014
	118-139	20.7	186.3	186.2	0.4	0.014
5	0-37	36.8	189.7	189.2	1.4	0.014
6	0-13	13.2	167.0	166.7	2.3	0.014
	13-32	19.1	166.7	164.9	9.3	0.014
	32-62	30.2	164.9	160.1	15.9	0.014
	62-101	39.4	160.1	157.8	5.8	0.014
	101-114	12.7	157.8	157.6	2.0	0.014
7	0-11	10.7	167.1	166.9	1.8	0.014
	11-34	23.2	166.9	165.8	5.0	0.014
	34-62	27.8	165.8	164.1	6.2	0.014
	62-83	20.9	164.1	163.5	3.0	0.014
	83-100	16.8	163.5	163.3	1.0	0.014
	100-108	8.5	163.2	163.1	1.2	0.014
	108-117	9.1	163.1	163.1	0.0	0.014
	117-132	15.2	163.1	163.0	0.7	0.014
	132-148	16.2	163.0	162.8	1.1	0.014
8	0-27	27.1	113.1	109.8	12.0	0.014
	27-42	15.0	109.8	109.3	3.9	0.014
	42-50	7.9	109.3	108.6	8.4	0.014
	50-69	18.9	108.6	106.8	9.5	0.014
	69-103	34.4	106.8	101.3	16.1	0.014
	103-130	27.3	101.3	96.5	17.4	0.014
9	0-11	11.2	166.2	164.3	17.2	0.014
	11-20	9.2	164.3	162.8	15.7	0.014
	20-34	13.7	162.8	161.0	13.4	0.014

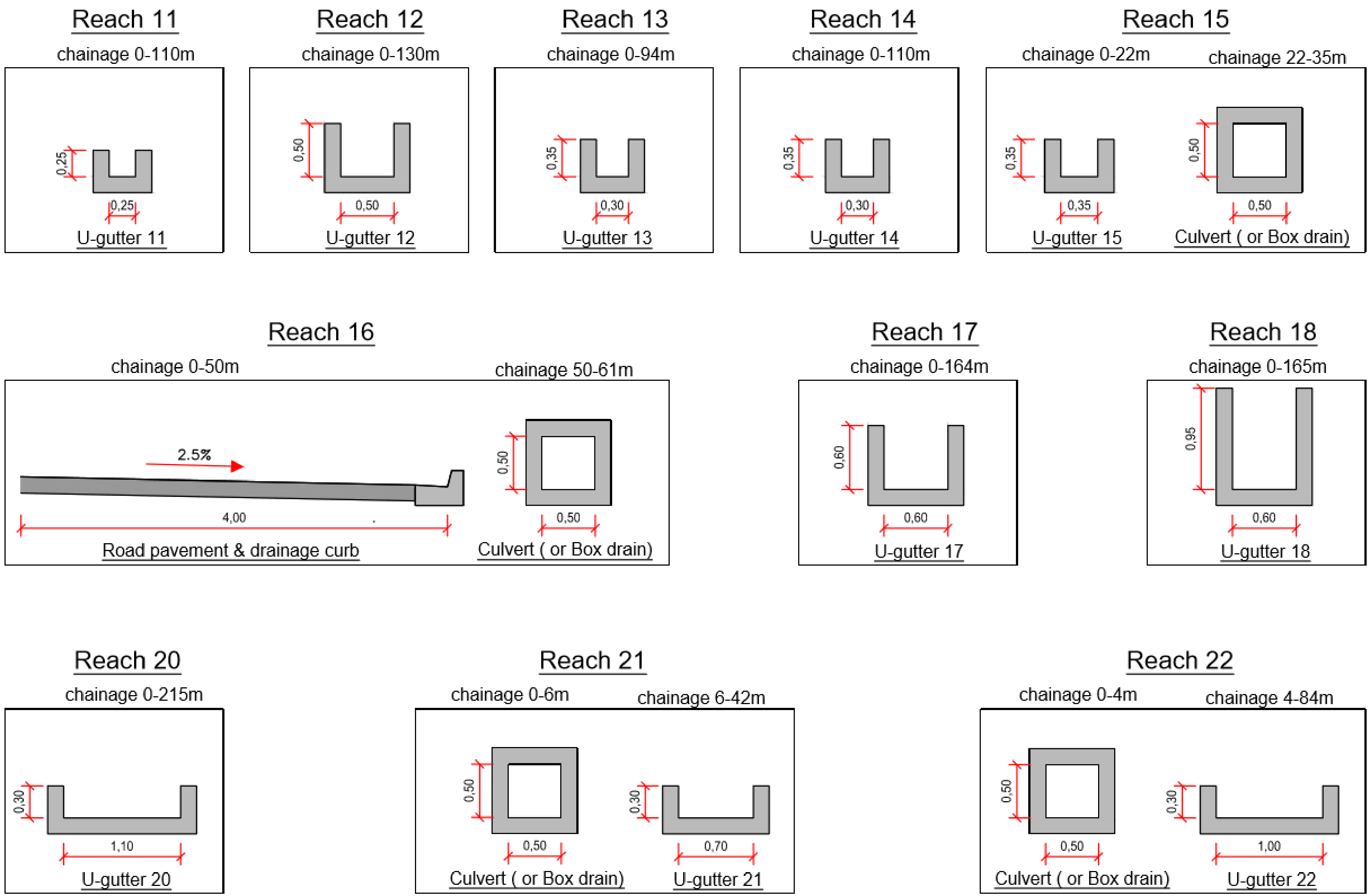
Reach	Chainage	Length	Inlet elevation	Outlet elevation	Average slope	Manning's roughness Coefficient
	(m)	(m)	(m)	(m)	(%)	(-)
9	34-51	17.1	161.0	159.2	10.7	0.014
	51-71	20.4	159.2	157.7	6.8	0.014
10	0-11	10.7	178.4	177.8	6.4	0.014
	11-27	16.4	177.8	175.8	12.2	0.014
	27-52	24.9	175.8	172.6	12.7	0.014
	52-75	23.3	172.6	170.0	11.1	0.014
	75-97	21.6	170.0	167.6	11.4	0.014
	97-121	24.0	167.6	165.8	7.3	0.014
	121-150	29.5	165.8	162.7	10.6	0.014
	150-185	35.2	162.7	157.5	14.7	0.014
	185-208	23.5	157.5	153.7	16.1	0.014
	208-233	25.1	153.7	149.9	15.3	0.014
	233-250	16.6	149.9	147.0	17.4	0.014
	250-269	19.3	147.0	142.7	22.1	0.014
	269-279	9.8	142.7	140.6	21.6	0.014
	279-287	8.1	140.6	138.1	30.8	0.014
	287-309	21.7	138.1	136.6	7.0	0.014
11	0-14	13.9	129.0	127.8	8.0	0.014
	14-24	10.1	127.8	127.6	3.0	0.014
	24-32	7.8	127.6	125.7	24.1	0.014
	32-53	21.3	125.7	124.4	5.8	0.014
	53-65	11.8	124.4	122.2	19.5	0.014
	65-72	7.1	122.2	120.6	21.4	0.014
	72-79	7.5	120.6	119.6	13.7	0.014
	79-96	17.4	119.6	118.1	8.8	0.014
	96-110	13.8	118.1	116.6	11.0	0.014
12	0-11	11.5	127.3	127.1	2.3	0.014
	11-22	11.3	127.1	126.5	5.1	0.014
	22-35	13.3	126.5	125.1	10.6	0.014
	35-46	10.8	125.1	123.7	12.5	0.014
	46-54	7.9	123.7	122.3	17.7	0.014
	54-62	7.6	122.3	121.6	10.0	0.014
	62-90	28.4	121.6	118.3	11.5	0.014
	90-118	28.2	118.3	116.5	6.4	0.014
	118-130	11.7	116.5	116.3	1.5	0.014
13	0-5	4.9	127.9	127.8	1.0	0.014
	5-9	4.0	127.8	127.8	1.0	0.014
	9-14	5.0	127.8	127.7	1.0	0.014
	14-22	7.6	127.7	127.6	1.0	0.014
	22-35	13.5	127.6	127.4	2.0	0.014
	35-46	10.9	127.4	127.0	3.7	0.014
	46-61	15.4	127.0	125.6	8.5	0.014
	61-82	21.1	125.6	123.0	12.5	0.014
	82-94	11.7	122.8	118.4	38.4	0.014
14	0-29	29.3	136.0	130.5	18.7	0.014

Reach	Chainage	Length	Inlet elevation	Outlet elevation	Average slope	Manning's roughness coefficient
	(m)	(m)	(m)	(m)	(%)	(-)
14	29-53	23.7	130.5	126.7	16.0	0.014
	53-82	29.2	126.7	122.4	14.9	0.014
	82-110	27.6	122.4	118.4	14.5	0.014
15	0-22	22.4	118.4	114.7	16.4	0.014
	22-35	13.1	114.6	113.7	7.0	0.014
16	0-12	12.1	98.7	97.7	8.6	0.014
	12-26	14.2	97.7	96.7	6.9	0.014
	26-41	15.2	96.7	94.3	15.7	0.014
	41-50	8.7	94.3	92.2	24.0	0.014
	50-61	10.5	91.8	88.0	36.1	0.014
17	0-23	23.5	96.4	93.1	14.3	0.014
	23-51	27.5	93.1	90.6	8.8	0.014
	51-82	30.8	90.6	87.9	9.0	0.014
	82-108	26.2	87.8	84.6	12.1	0.014
	108-135	27.6	84.6	82.5	7.6	0.014
	135-164	28.6	82.5	80.7	6.4	0.014
18	0-13	12.7	80.4	79.3	8.5	0.014
	13-24	11.3	79.3	78.3	8.5	0.014
	24-36	12.3	78.3	76.8	12.8	0.014
	36-49	12.8	76.8	75.5	9.9	0.014
	49-61	11.6	75.5	73.5	17.1	0.014
	61-76	14.7	73.5	70.7	19.1	0.014
	76-104	27.7	70.7	64.7	21.6	0.014
	104-131	27.3	64.7	59.6	18.7	0.014
	131-157	26.5	59.6	56.4	12.0	0.014
	157-165	8.4	56.4	55.9	6.2	0.014
20	0-6	6.3	186.2	185.2	15.4	0.014
	6-44	38.0	185.2	162.8	58.9	0.014
	44-74	30.2	162.8	148.4	48.0	0.014
	74-80	6.3	148.4	145.3	48.5	0.014
	80-89	8.8	145.3	140.9	49.4	0.014
	89-97	8.3	140.9	134.7	74.6	0.014
	97-138	41.4	134.7	114.2	49.6	0.014
	138-152	13.8	114.2	106.9	52.7	0.014
	152-168	16.4	106.9	98.7	50.3	0.014
	168-210	41.5	98.7	83.3	37.0	0.014
	210-215	4.7	83.3	81.1	48.1	0.014
21	0-6	5.6	116.4	116.0	7.0	0.014
	6-20	13.9	116.0	103.4	90.1	0.014
	20-28	8.0	103.4	99.7	46.6	0.014
	28-36	8.0	99.7	97.9	22.1	0.014
	36-42	5.8	97.9	96.7	21.1	0.014
22	0-4	4.4	157.6	157.4	3.0	0.014
	4-10	5.5	157.4	155.3	39.4	0.014
	10-19	8.9	155.3	149.4	65.5	0.014

Reach	Chainage	Length	Inlet elevation	Outlet elevation	Average slope	Manning's roughness Coefficient
	(m)	**(m)**	**(m)**	**(m)**	**(%)**	**(-)**
22	19-30	11.6	149.4	144.2	44.5	0.014
	30-40	10.0	144.2	141.3	29.7	0.014
	40-53	13.0	141.3	141.1	1.0	0.014
	53-78	25.5	141.1	128.4	49.8	0.014
	78-84	5.6	128.4	127.9	8.7	0.014

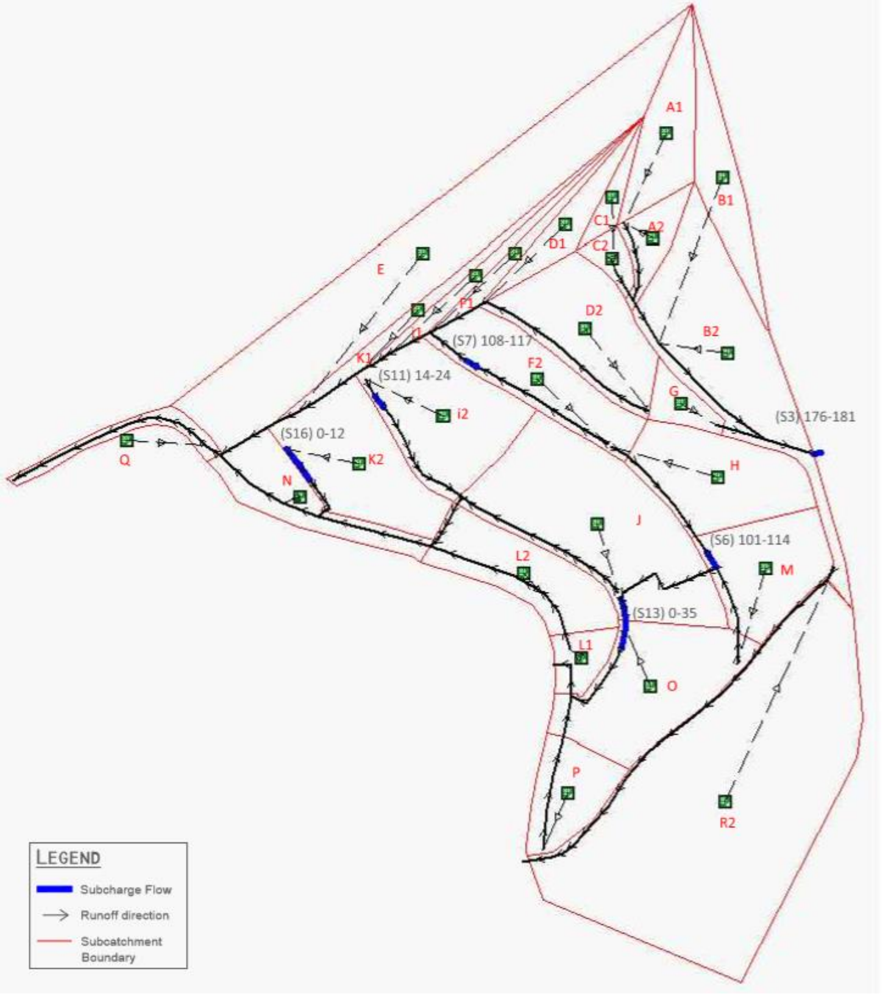
2.2. Cross-sections of storm drains

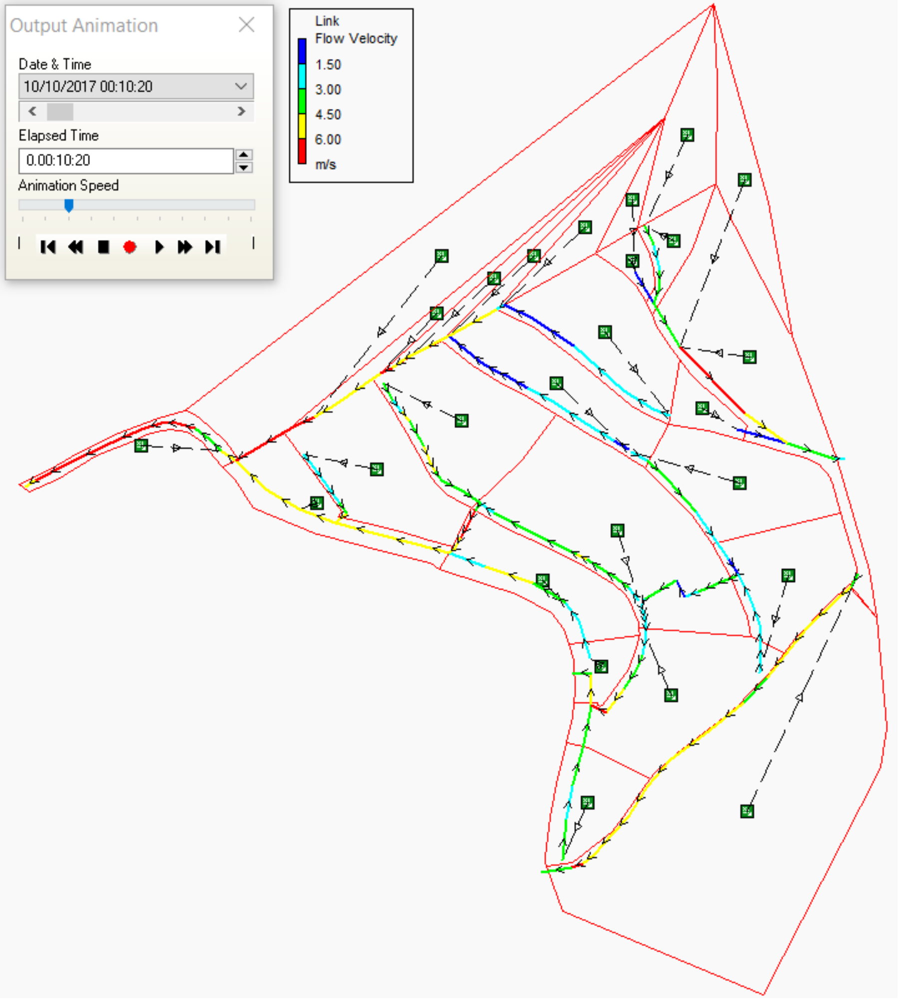



Reach 11
chainage 0-110m
0,25
0,25
U-gutter 11
Reach 12
chainage 0-130m
0,50
0,50
U-gutter 12
Reach 13
chainage 0-94m
0,35
0,30
U-gutter 13
Reach 14
chainage 0-110m
0,35
0,30
U-gutter 14
Reach 15
chainage 0-22m
chainage 22-35m
0,35
0,35
U-gutter 15
0,50
0,50
Culvert (or Box drain)
Reach 16
chainage 0-50m
chainage 50-61m
2.5%
4,00
Road pavement & drainage curb
0,50
0,50
Culvert (or Box drain)
Reach 17
chainage 0-164m
0,60
0,60
U-gutter 17
Reach 18
chainage 0-165m
0,95
0,60
U-gutter 18
Reach 20
chainage 0-215m
0,30
1,10
U-gutter 20
Reach 21
chainage 0-6m
chainage 6-42m
0,50
0,50
Culvert (or Box drain)
0,30
0,70
U-gutter 21
Reach 22
chainage 0-4m
chainage 4-84m
0,50
0,50
Culvert (or Box drain)
0,30
1,00
U-gutter 22

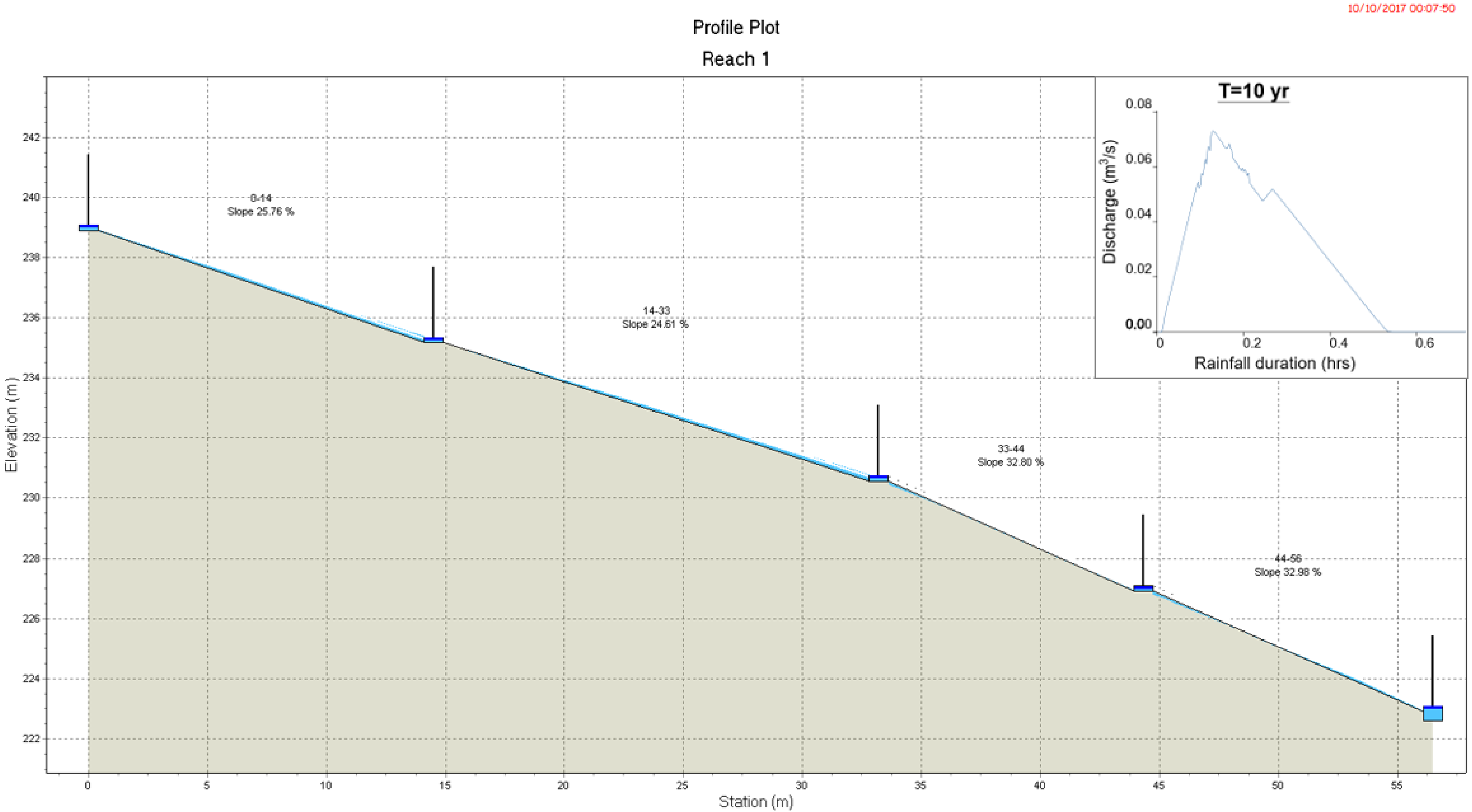
2.3. Simulation results

2.3.1. Overview of the drainage network

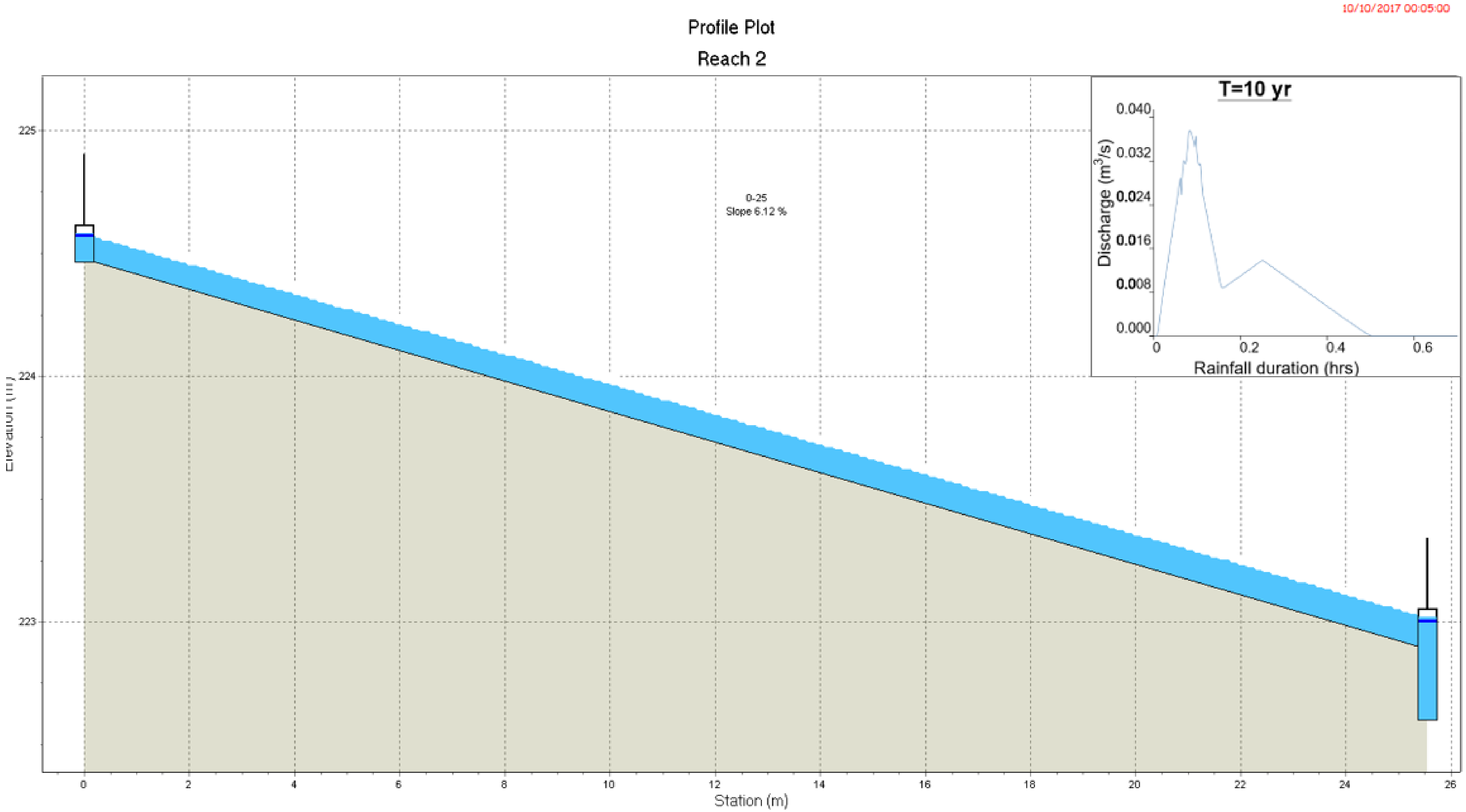




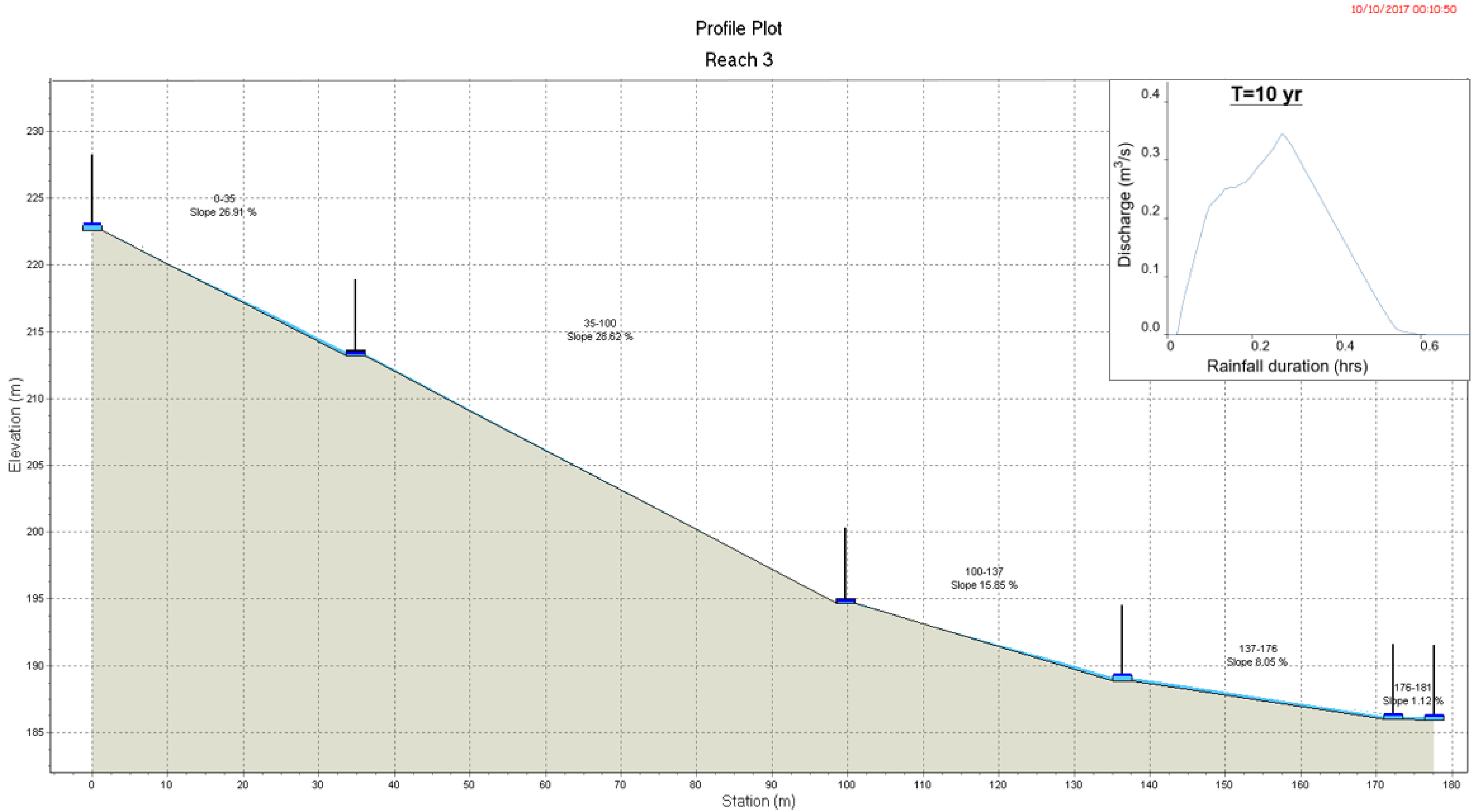
2.3.2. Longitudinal profile plots



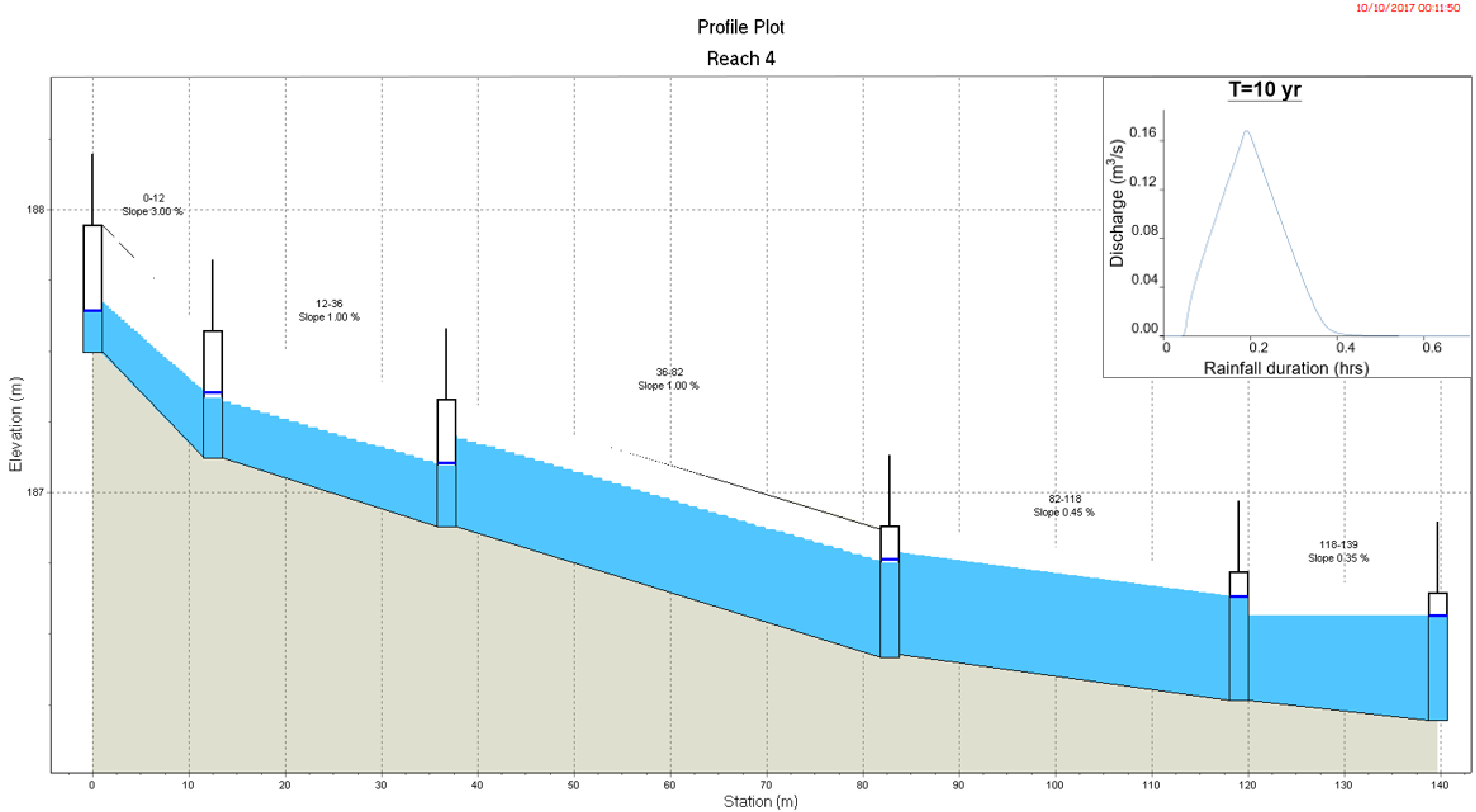
Chainage: (-)	0-14	14-33	33-44	44-56
Max vel (m/s)	4.68	3.92	4.50	4.63
Max depth (m)	0.11	0.12	0.11	0.11



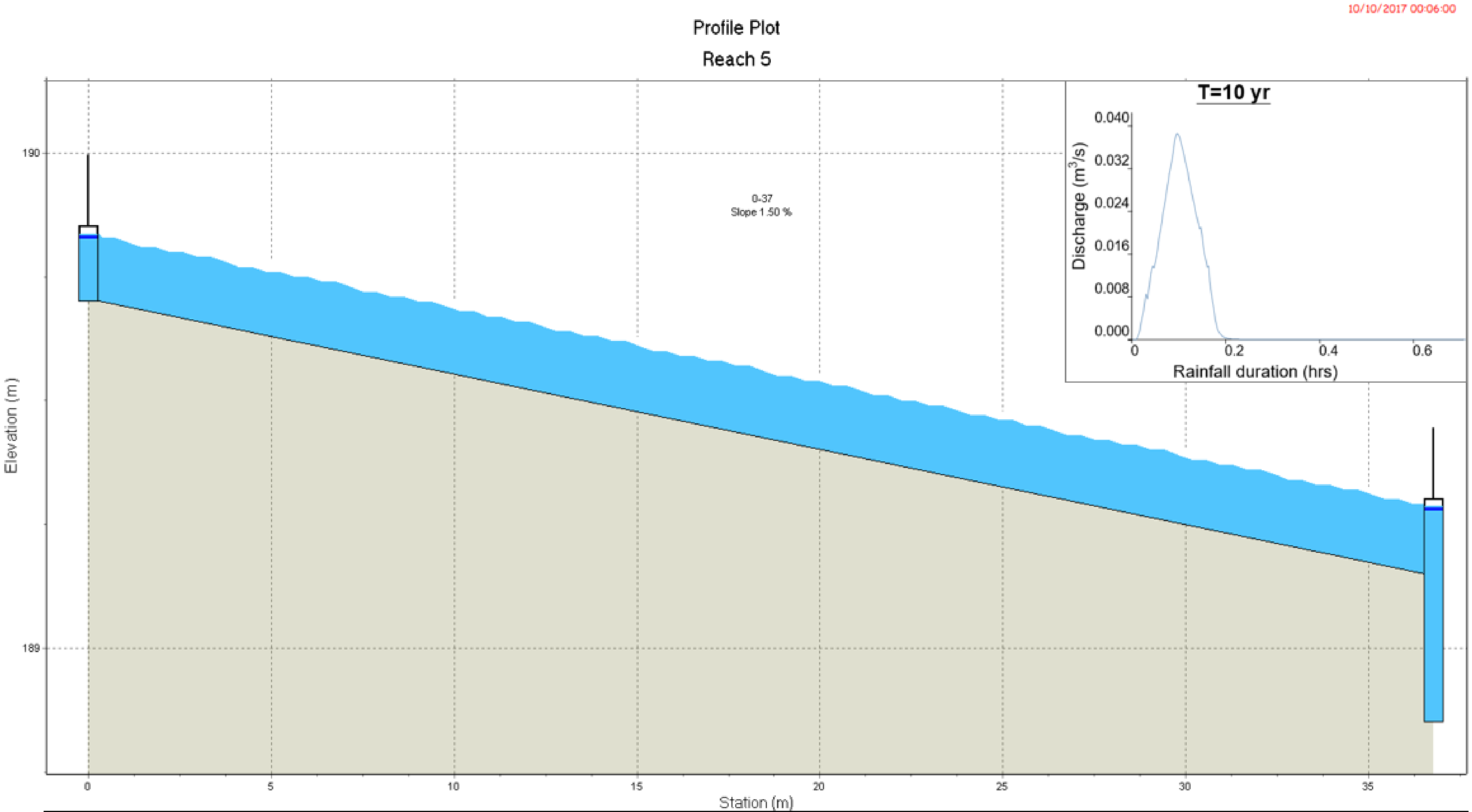
Chainage: (-)	0-25
Max vel (m/s)	2.66
Max depth (m)	0.11



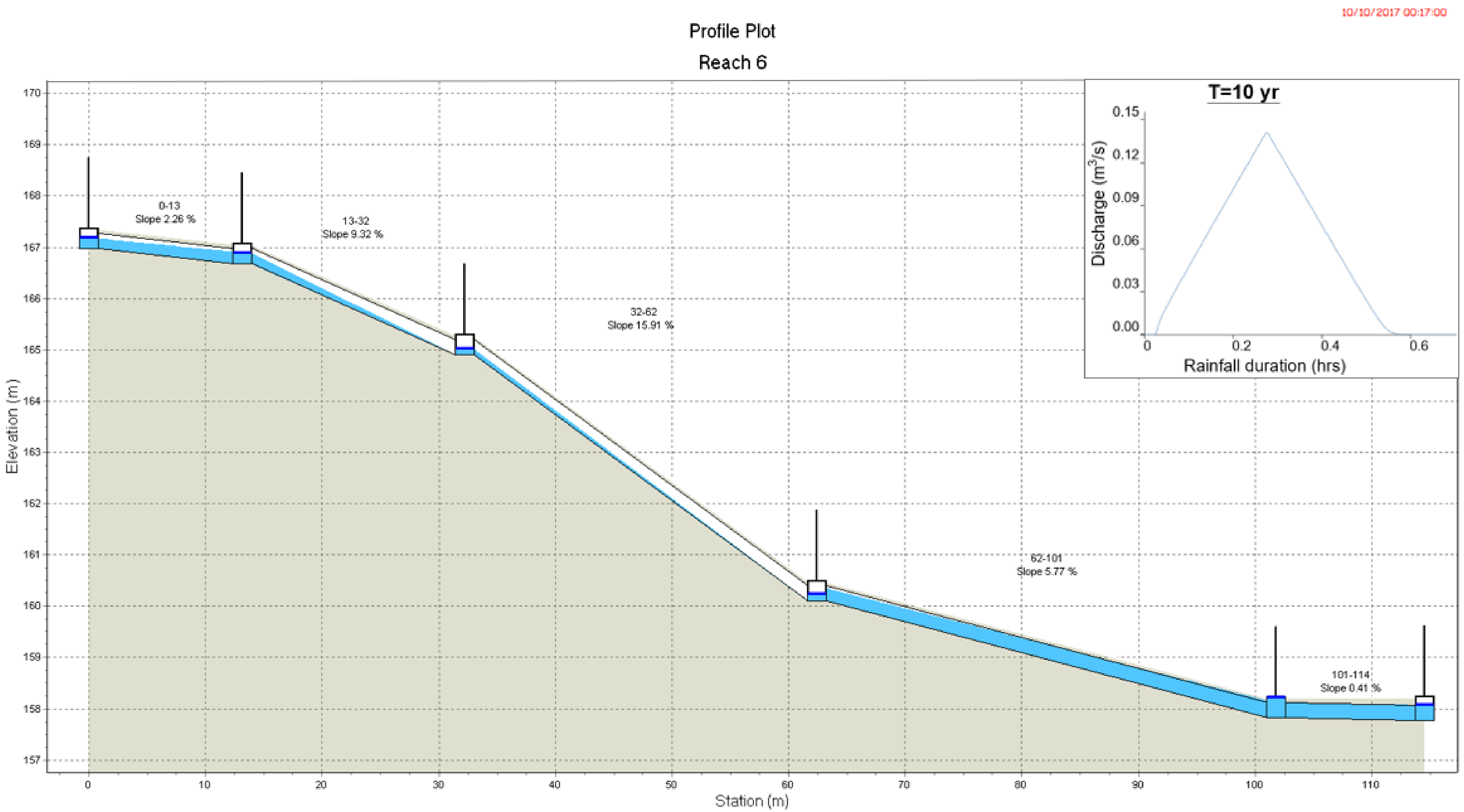
Chainage: (-)	0-35	35-100	100-137	137-176	176-181
Max vel (m/s)	4.49	6.92	5.59	4.36	1.84
Max depth (m)	0.05	0.12	0.15	0.20	0.30



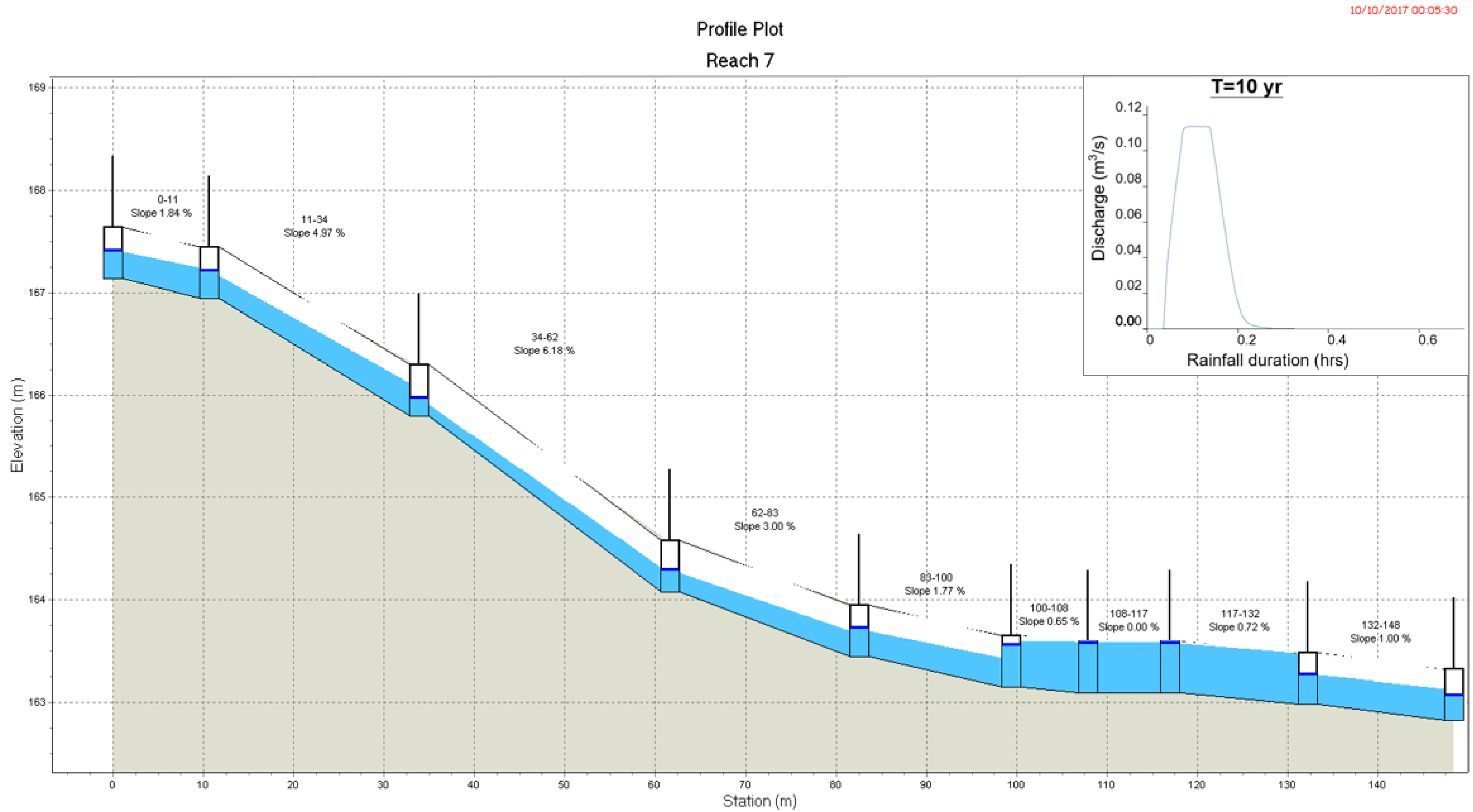
Chainage: (-)	0-12	12-36	36-86	82-118	118-139
Max vel (m/s)	2.64	1.75	1.76	1.21	1.09
Max depth (m)	0.16	0.23	0.23	0.34	0.37



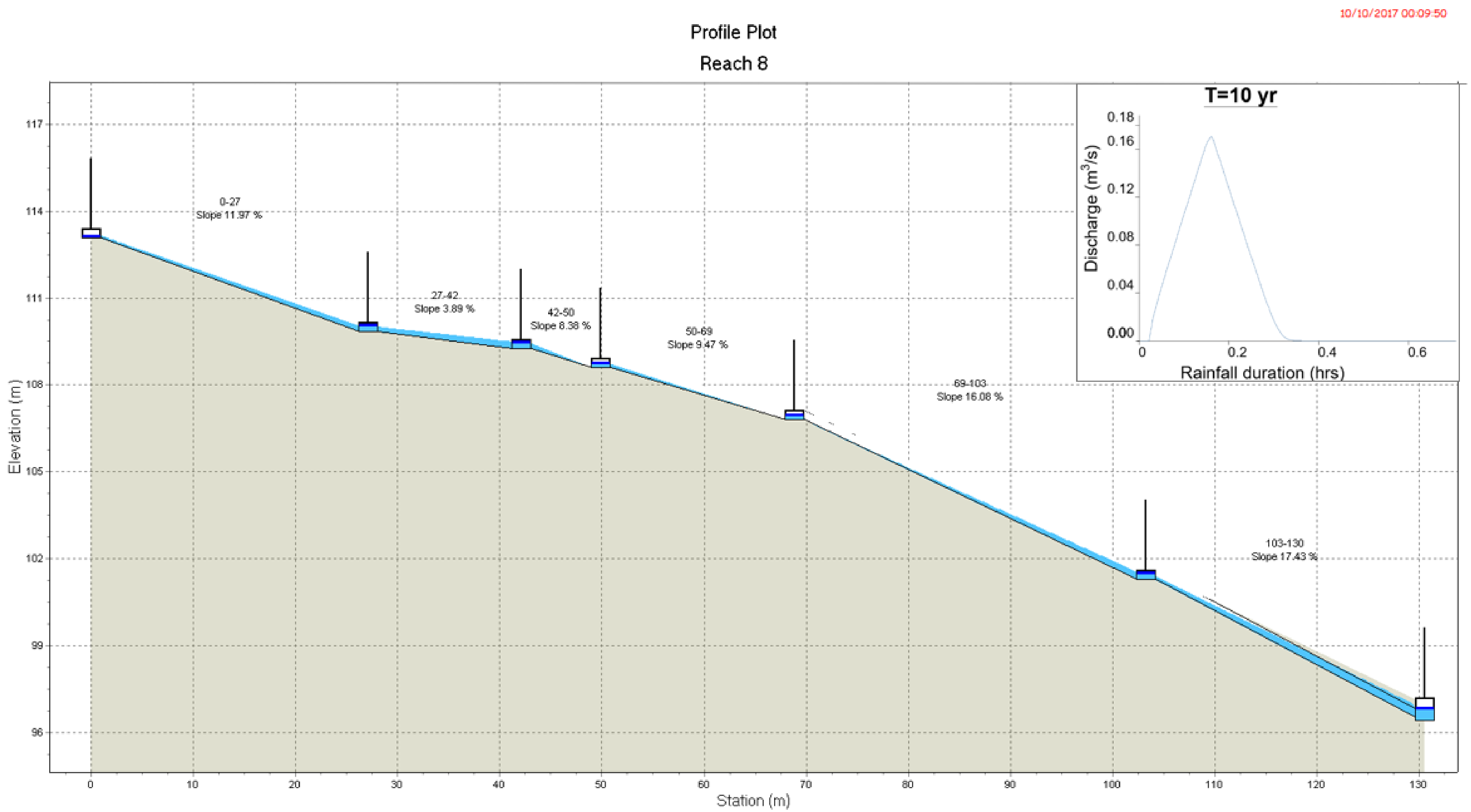
Chainage: (-)	0-37
Max vel (m/s)	3.07
Max depth (m)	0.13



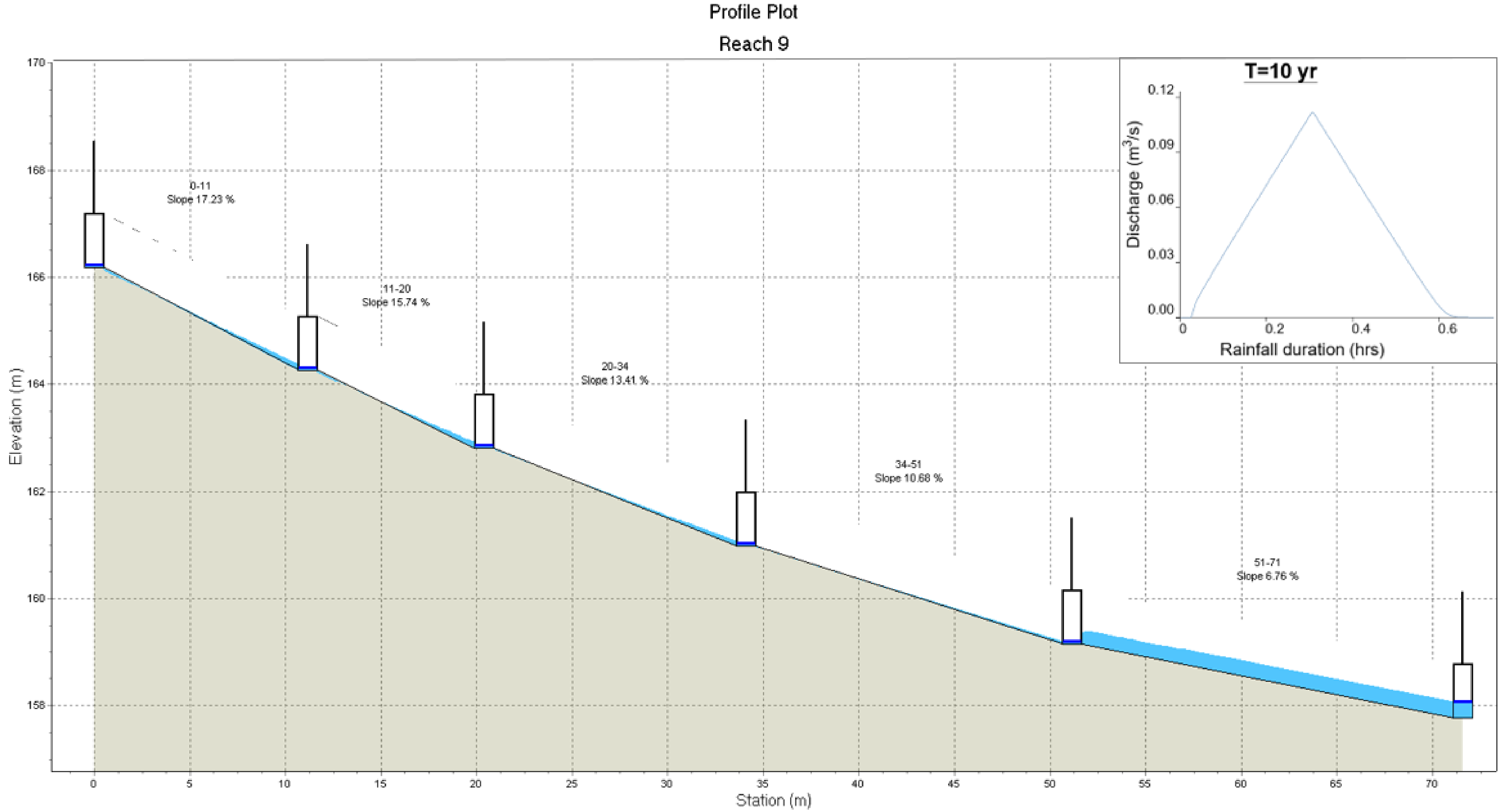
Chainage: (-)	0-13	13-32	32-62	62-101	101-114
Max vel (m/s)	2.14	3.68	4.47	3.08	0.99
Max depth (m)	0.22	0.13	0.11	0.15	0.30



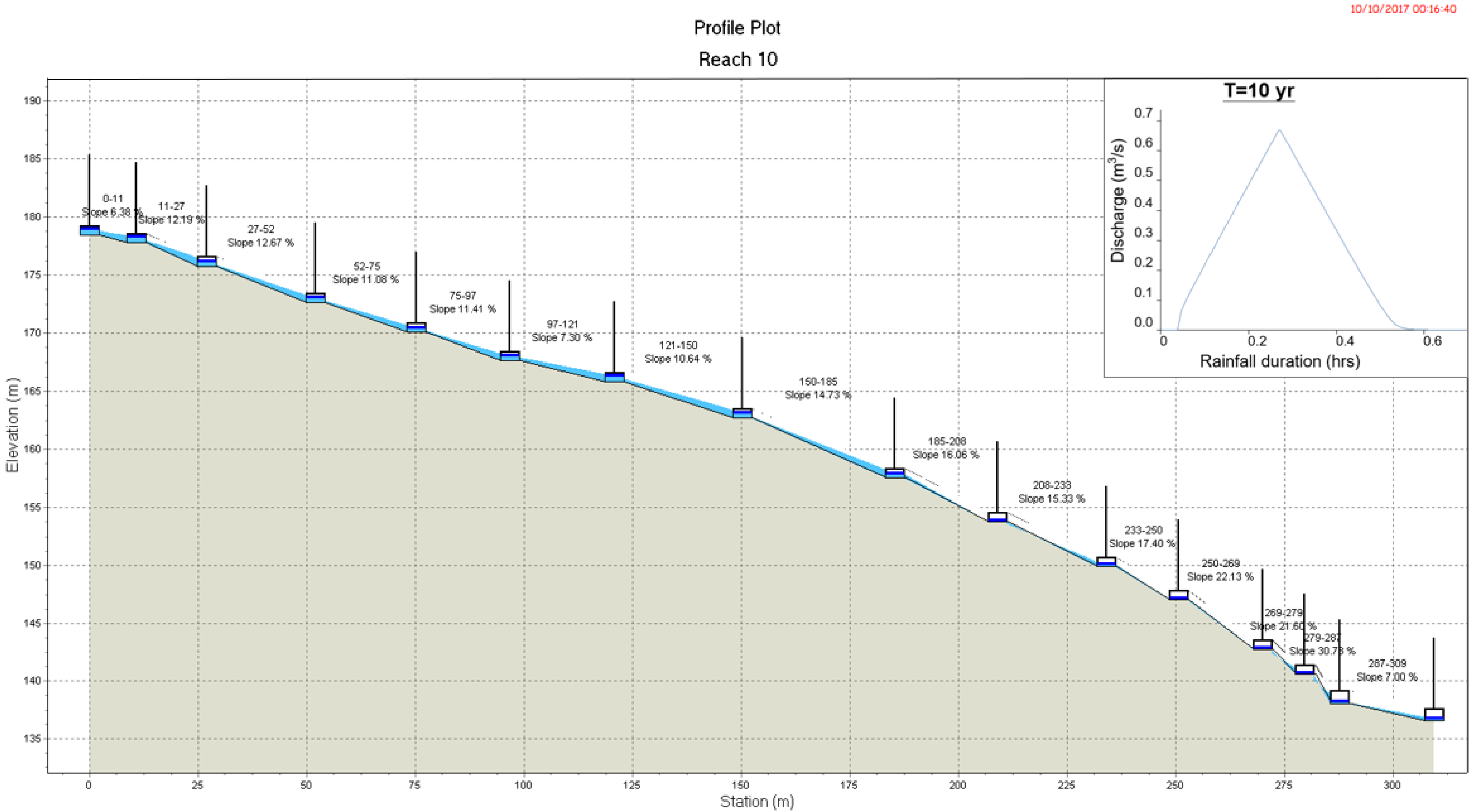
Chainage: (-)	0-11	11-34	34-62	62-83	83-100	100-108	108-117	117-132	132-148
Max vel (m/s)	2.05	3.04	3.30	2.50	2.02	1.34	0.77	1.31	1.49
Max depth (m)	0.28	0.18	0.17	0.22	0.28	0.42	0.50	0.29	0.26



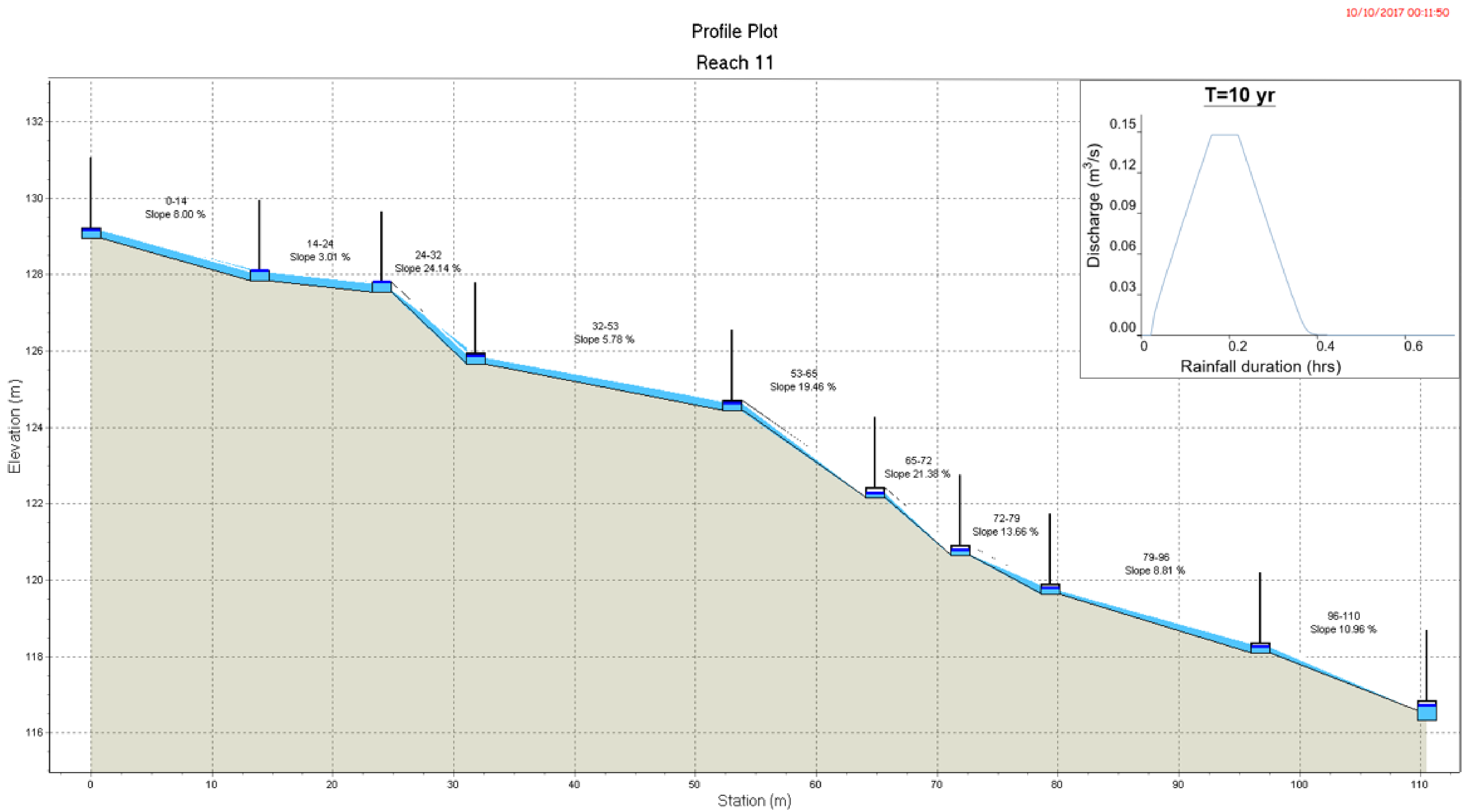
Chainage: (-)	0-27	27-42	42-50	50-69	69-103	103-130
Max vel (m/s)	2.72	2.77	3.71	3.89	4.73	2.61
Max depth (m)	0.05	0.21	0.15	0.15	0.12	0.22



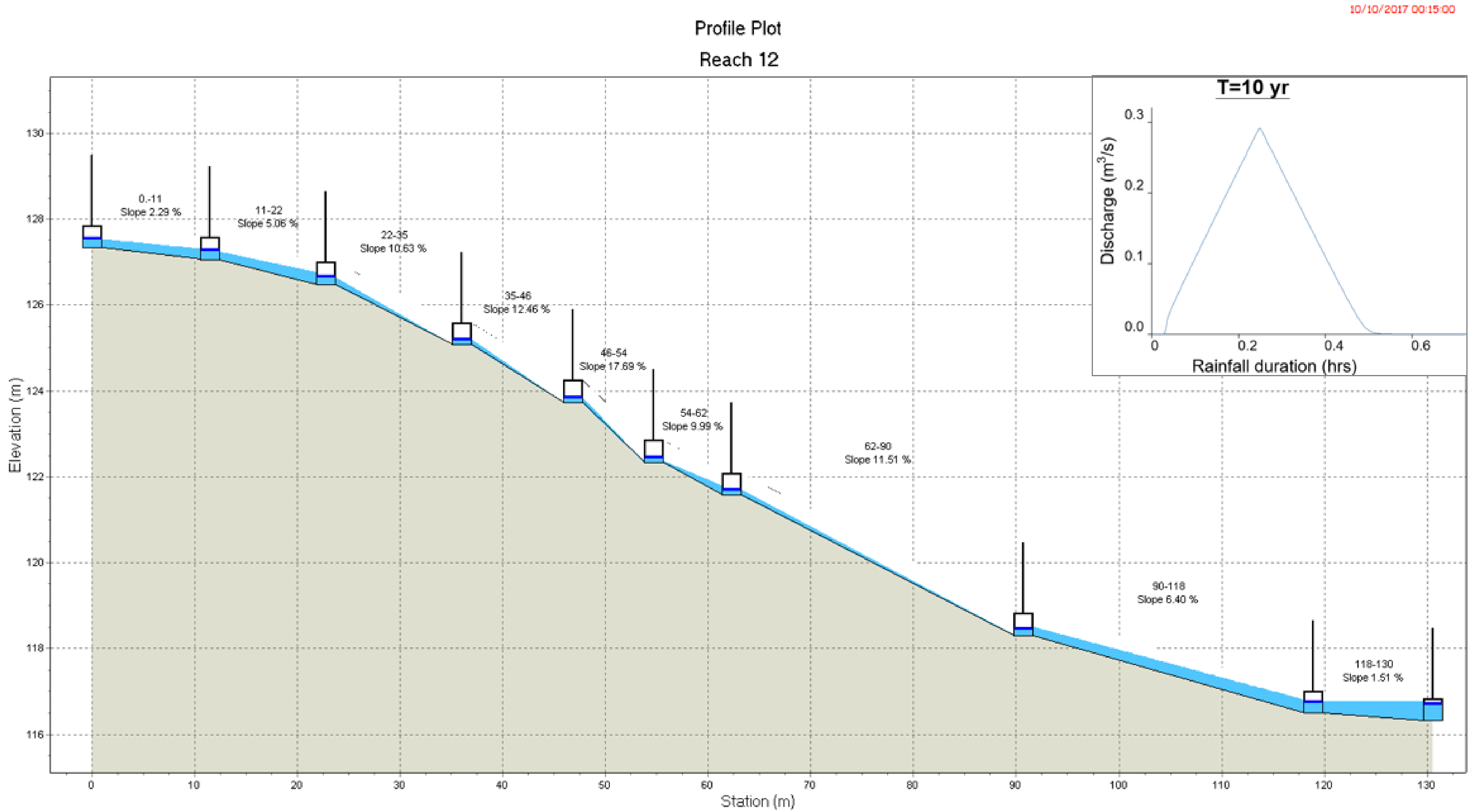
Chainage: (-)	0-11	11-20	20-34	34-51	51-71
Max vel (m/s)	3.10	3.01	2.87	2.67	2.32
Max depth (m)	0.04	0.04	0.04	0.04	0.05



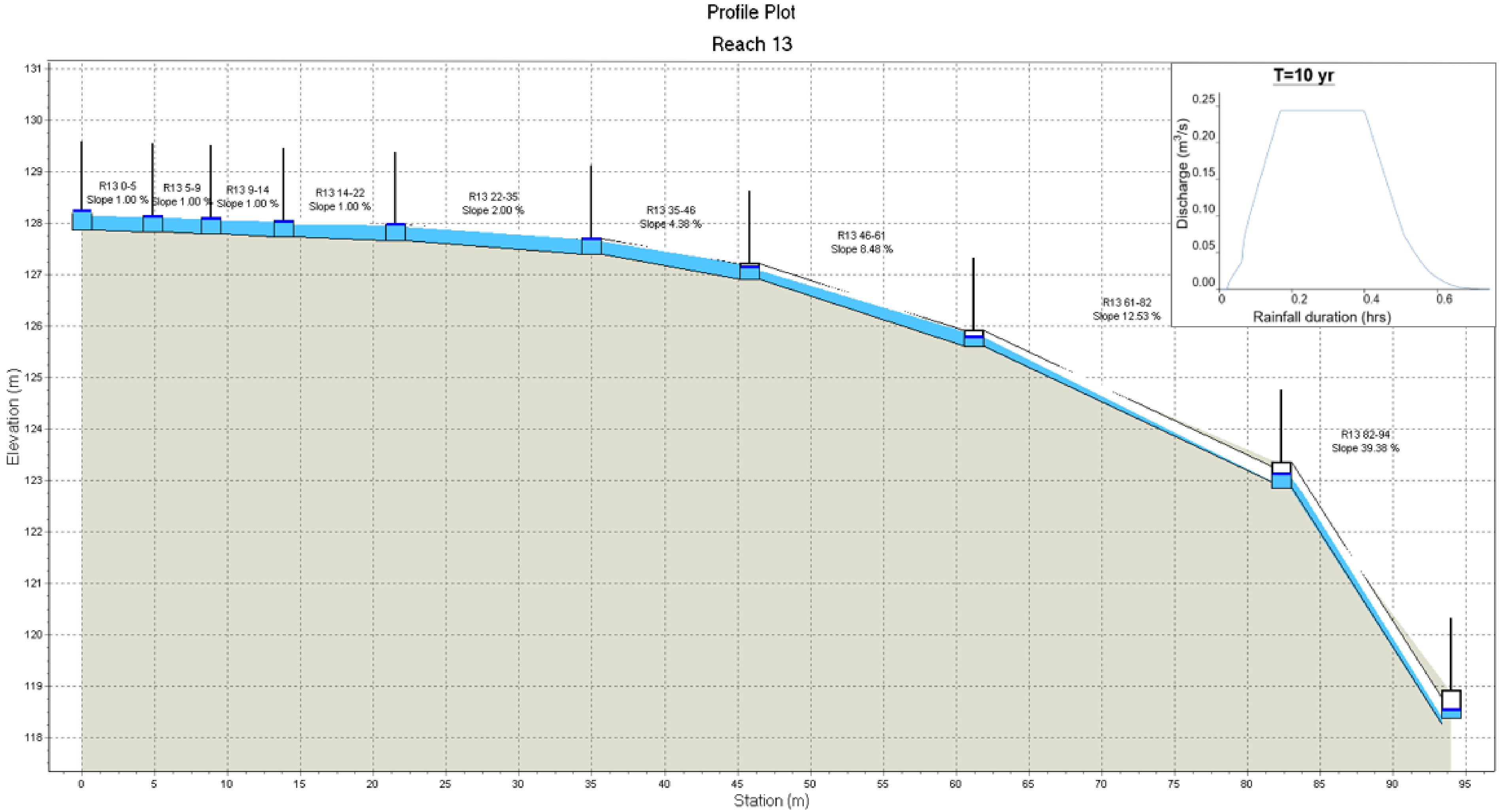
Chainage: (-)	0-11	11-27	27-52	52-75	75-97	97-121	121-150	150-185	185-208	208-253	233-250	250-269	269-279	279-287	287-309
Max vel (m/s)	4.31	5.69	5.78	5.46	5.53	4.57	5.37	6.16	6.29	6.19	6.46	7.01	6.95	7.82	4.64
Max depth (m)	0.52	0.39	0.41	0.40	0.49	0.42	0.36	0.14	0.14	0.14	0.14	0.13	0.13	0.11	0.17



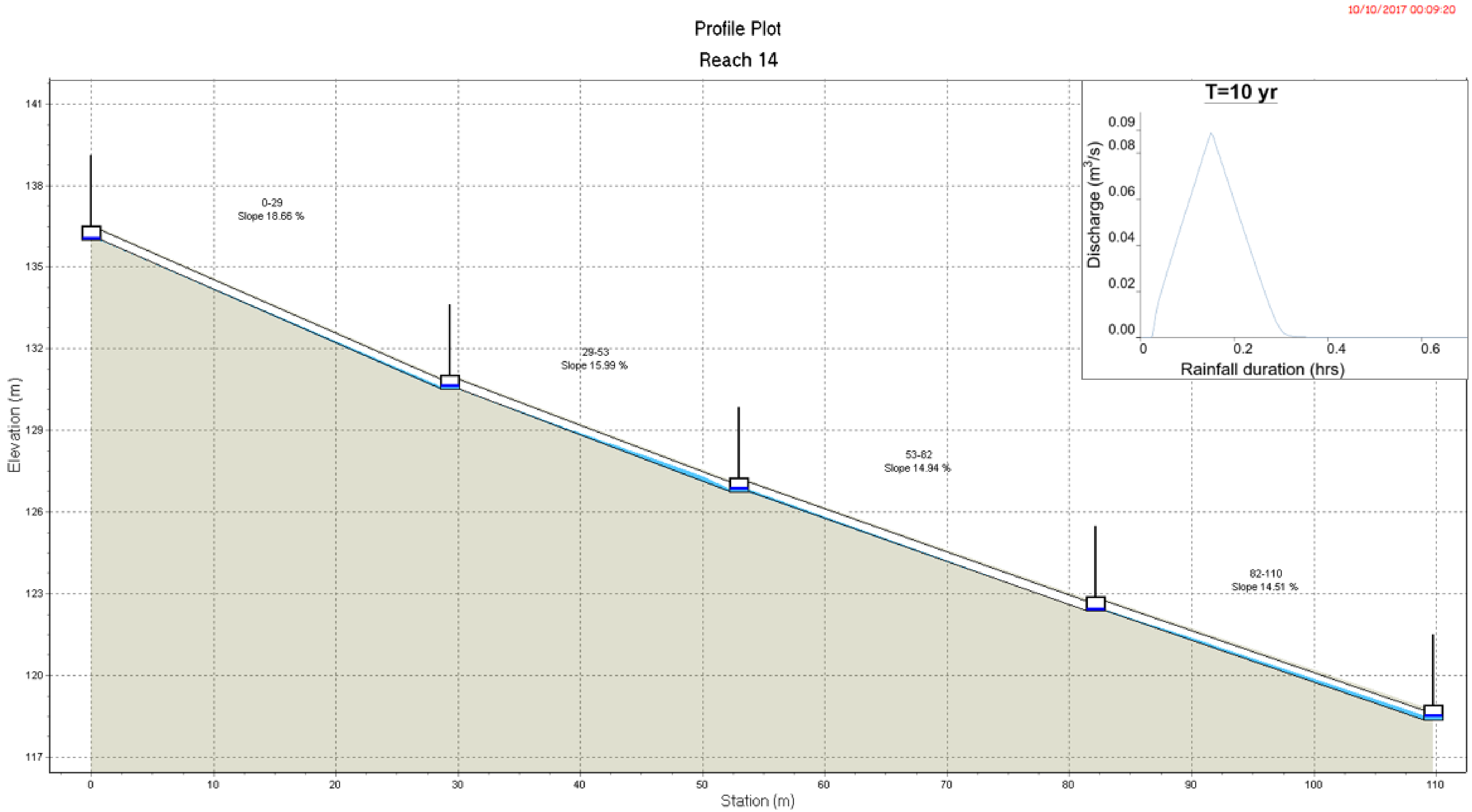
Chainage: (-)	0-14	14-24	24-32	32-53	53-65	65-72	72-79	79-96	96-110
Max vel (m/s)	3.64	2.37	5.31	3.08	4.91	5.08	4.29	3.63	3.95
Max depth (m)	0.20	0.25	0.11	0.19	0.12	0.12	0.14	0.16	0.15



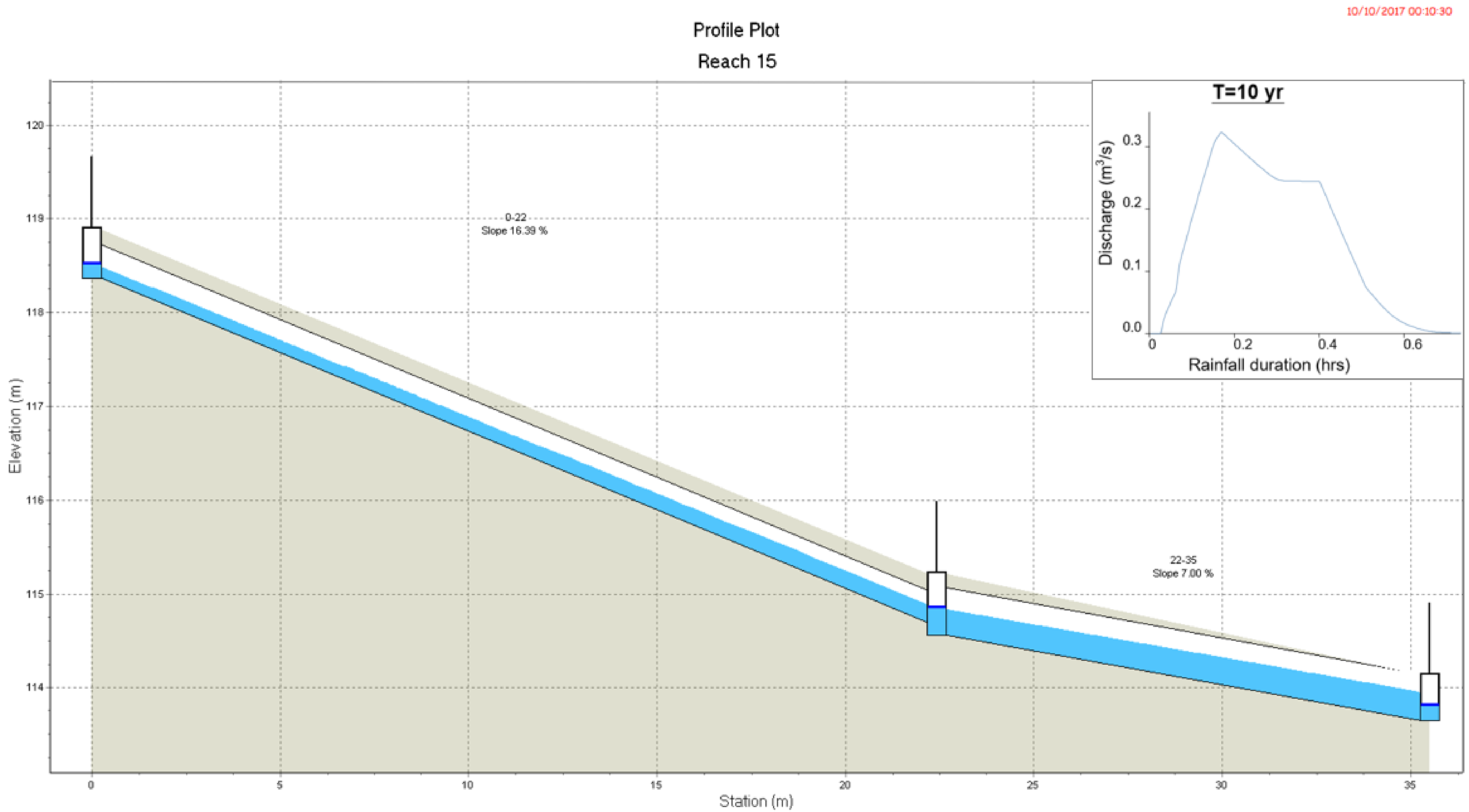
Chainage: (-)	0-11	11-22	22-35	35-46	46-54	54-62	62-90	90-118	118-130
Max vel (m/s)	2.61	3.48	4.52	4.77	5.38	4.42	4.65	3.78	2.23
Max depth (m)	0.22	0.17	0.13	0.12	0.11	0.13	0.13	0.15	0.26



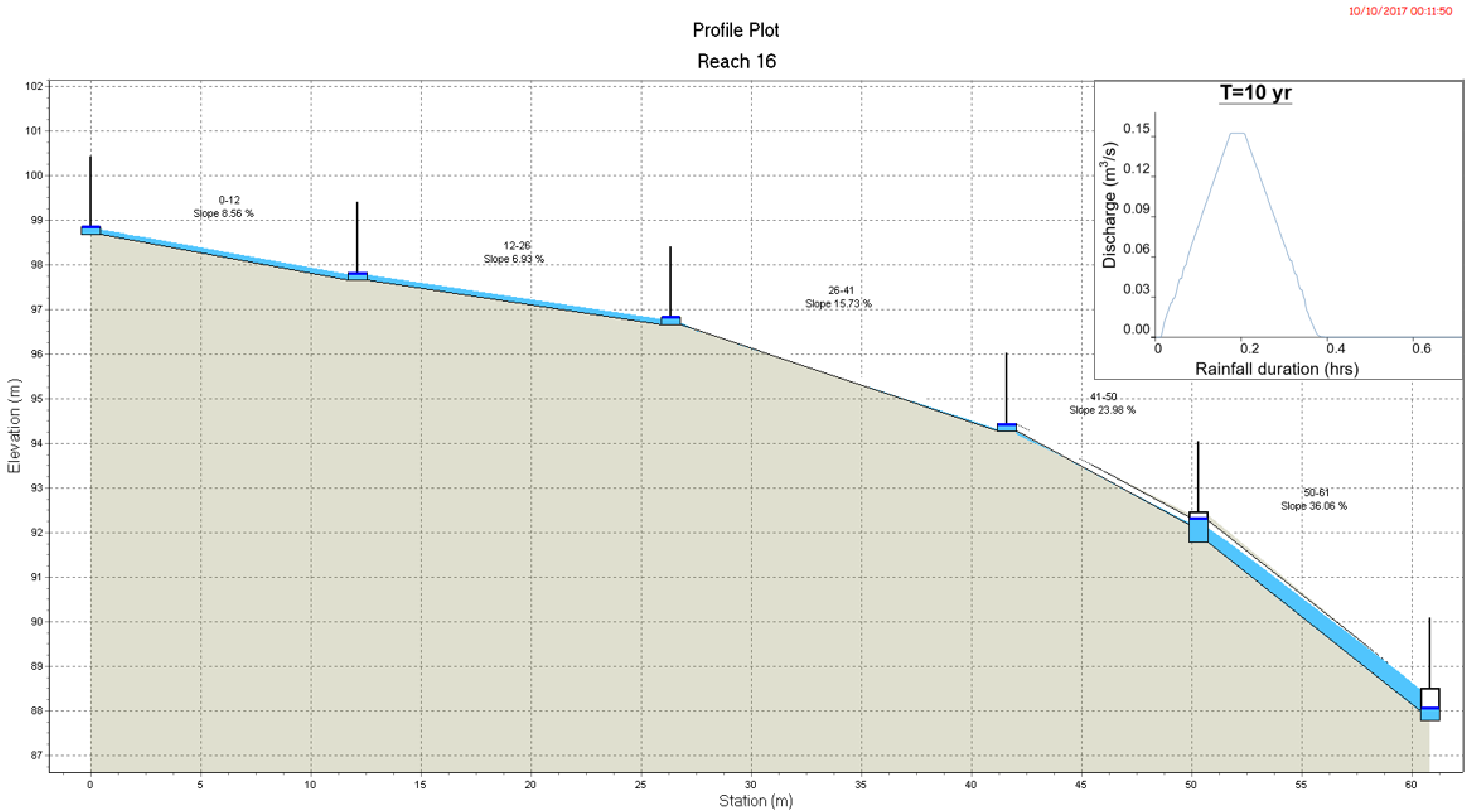
Chainage: (-)	0-5	5-9	9-14	14-22	22-35	35-46	46-61	61-82	82-94
Max vel (m/s)	1.65	1.64	1.64	1.65	2.33	3.17	4.07	4.71	6.28
Max depth (m)	0.30	0.30	0.30	0.30	0.30	0.22	0.17	0.15	0.08



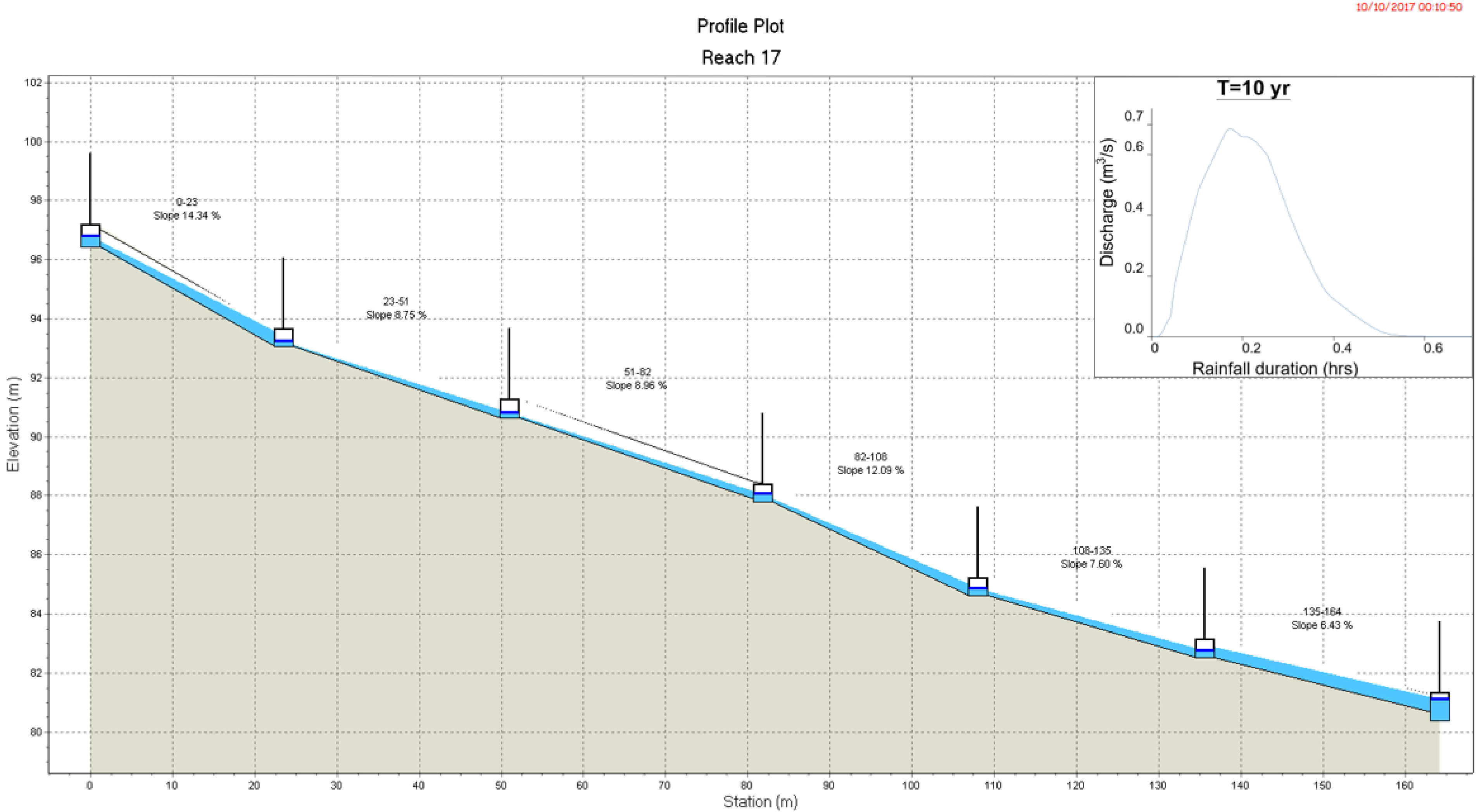
Chainage: (-)	0-29	29-53	53-82	82-110
Max vel (m/s)	4.02	2.15	3.72	3.68
Max depth (m)	0.06	0.12	0.07	0.07



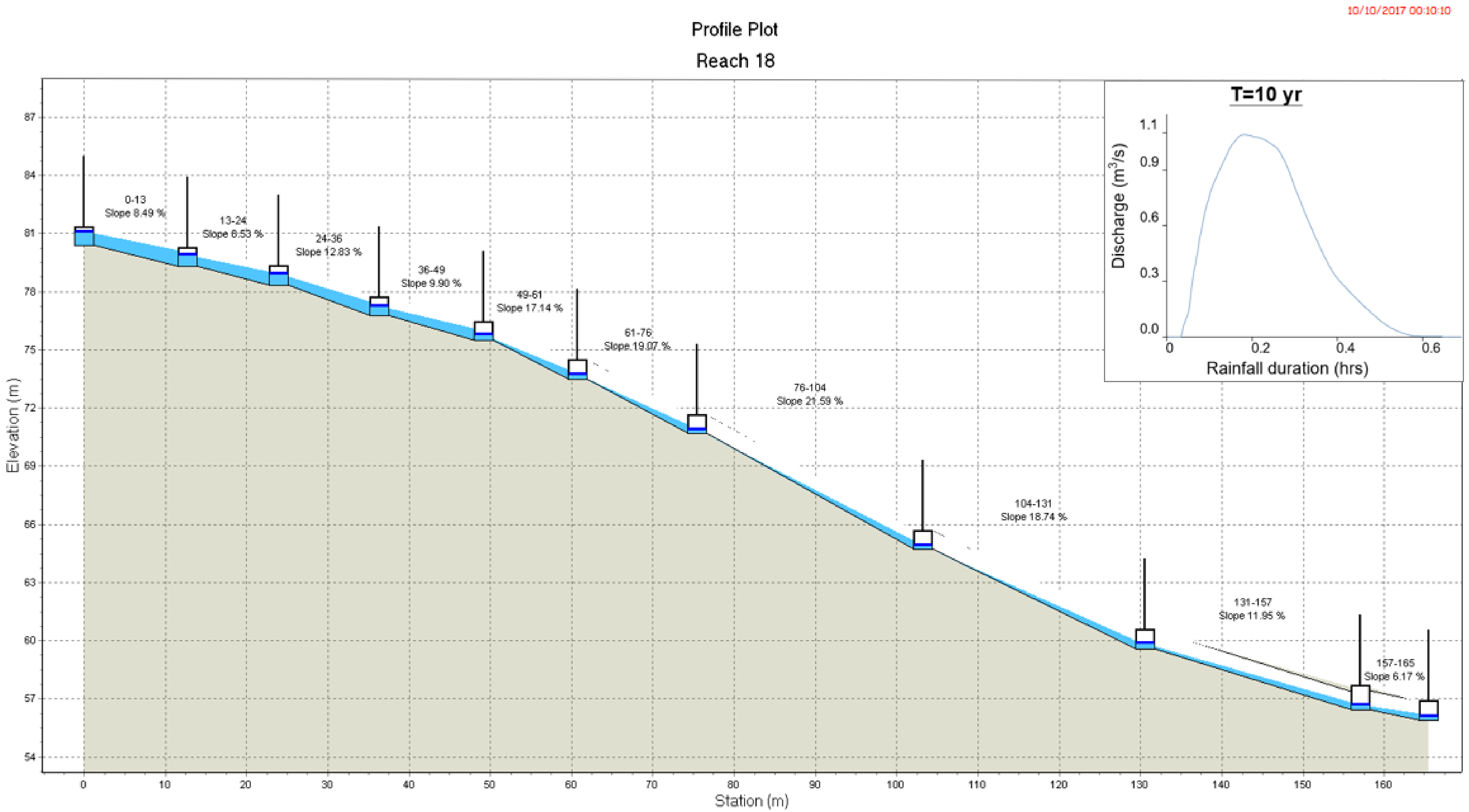
Chainage: (-)	0-22	22-35
Max vel (m/s)	5.58	3.82
Max depth (m)	0.17	0.17



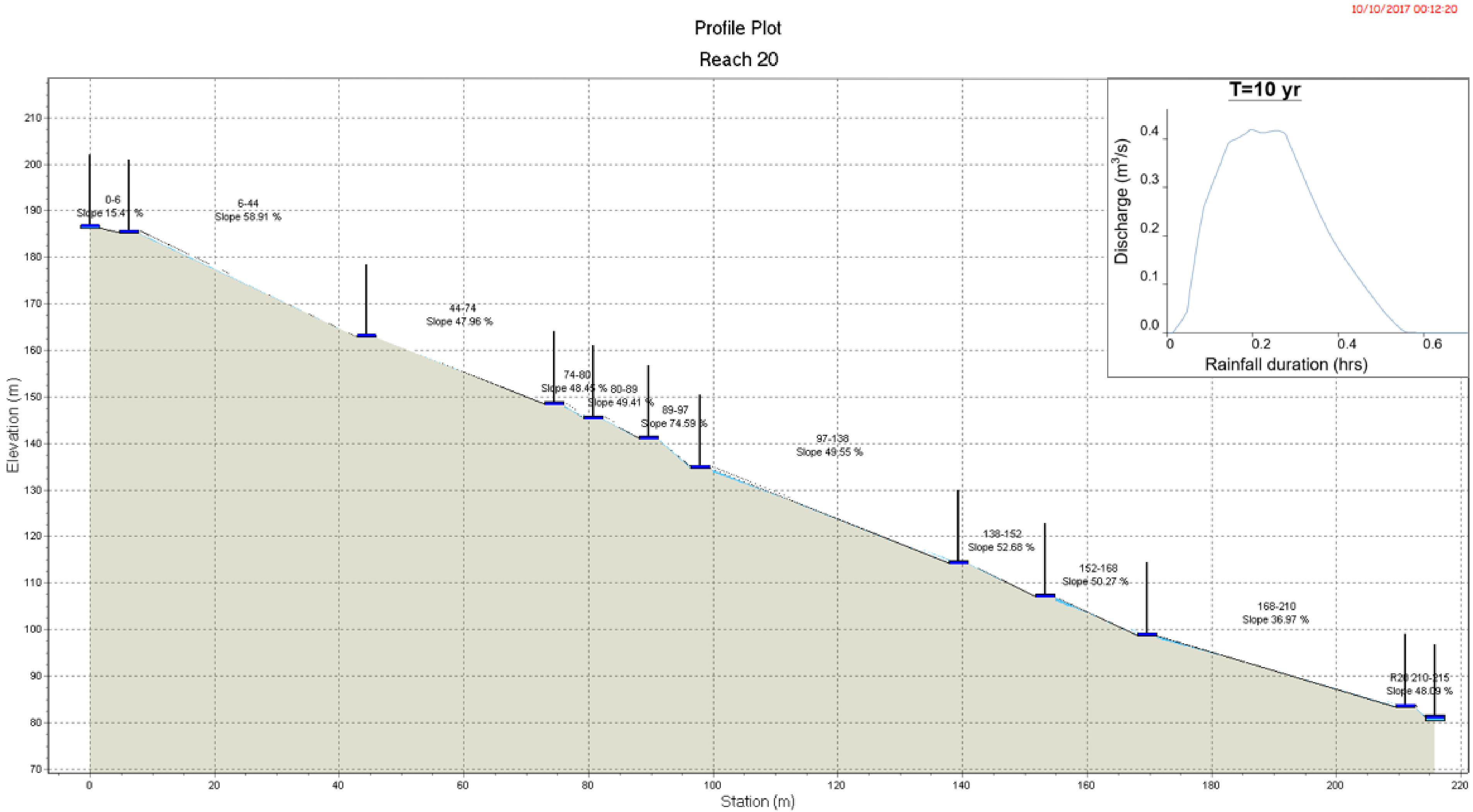
Chainage: (-)	0-12	12-26	26-41	41-50	50-61
Max vel (m/s)	4.61	2.19	3.19	3.86	5.48
Max depth (m)	0.15	0.15	0.14	0.13	0.06



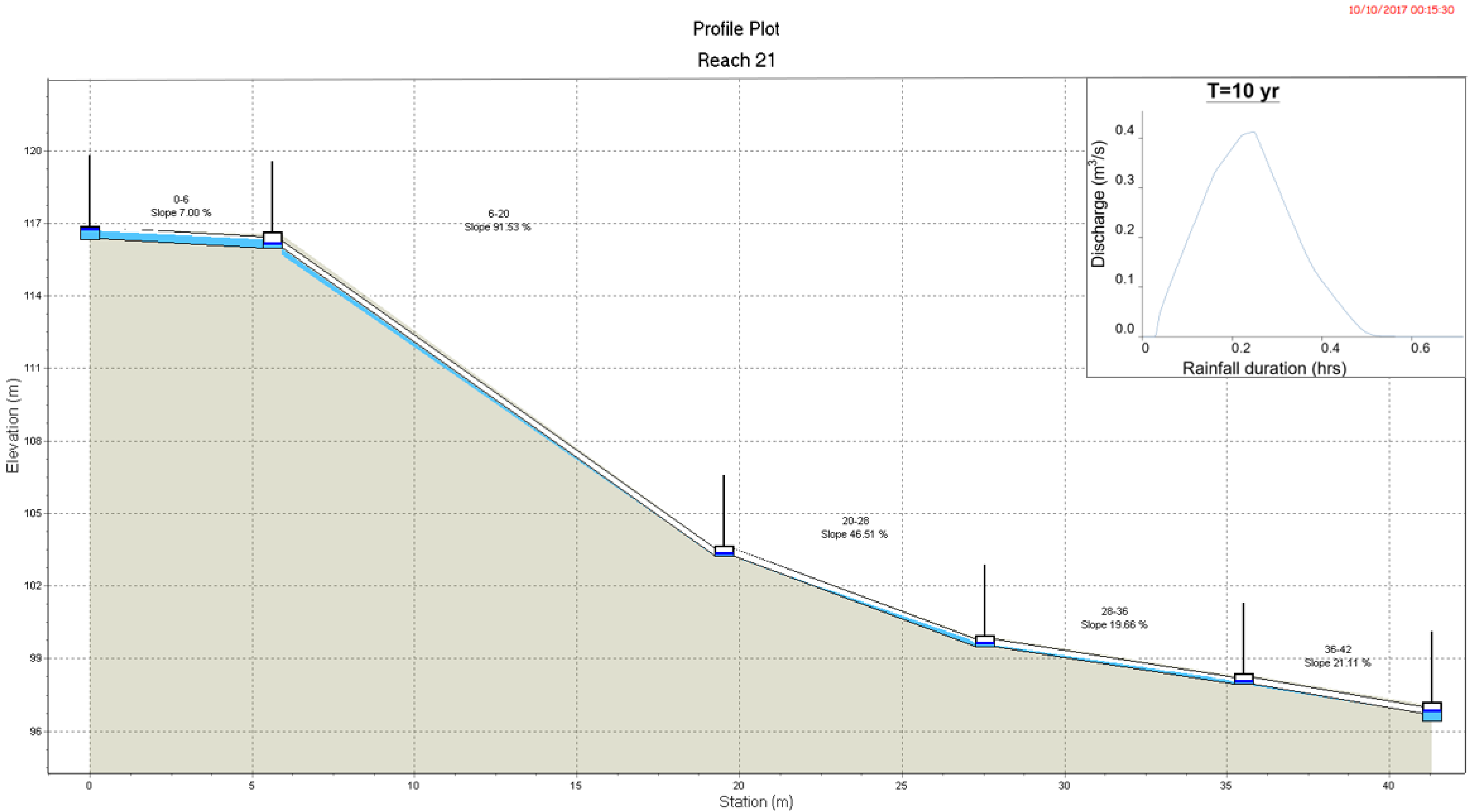
Chainage: (-)	0-23	23-51	51-82	82-108	108-135	135-164
Max vel (m/s)	5.77	4.86	4.90	5.88	5.03	4.74
Max depth (m)	0.15	0.17	0.17	0.19	0.23	0.24



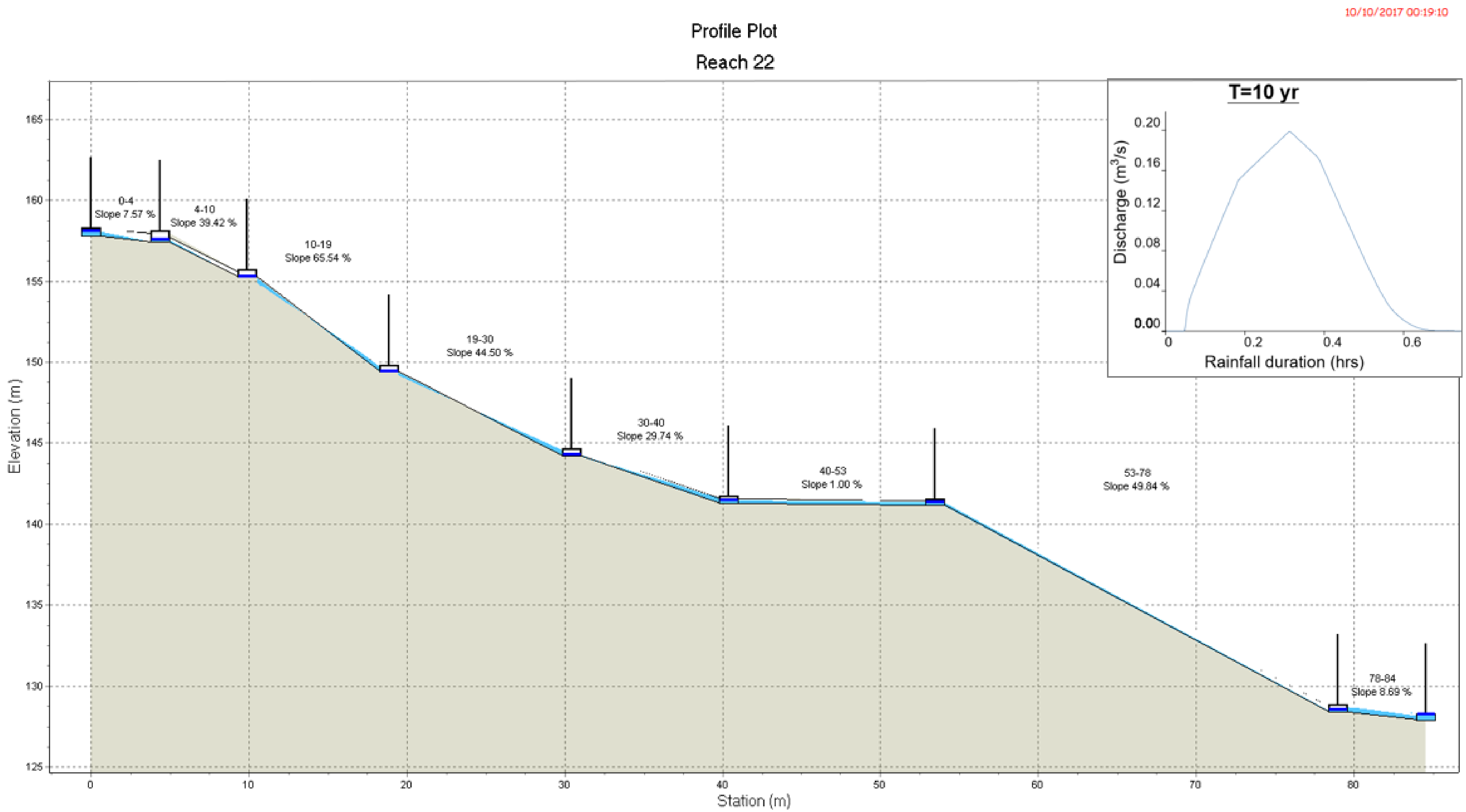
Chainage: (-)	0-13	13-24	24-66	36-49	49-61	61-76	76-104	104-131	131-157	157-165
Max vel (m/s)	5.92	3.10	3.66	6.27	7.68	7.98	8.35	7.93	6.73	5.03
Max depth (m)	0.31	0.59	0.50	0.29	0.24	0.23	0.22	0.23	0.27	0.22



Chainage: (-)	0-6	6-44	44-74	74-80	80-89	89-97	97-138	138-152	152-168	168-210	210-215
Max vel (m/s)	2.08	5.27	5.54	5.66	5.68	6.46	5.81	5.92	6.89	6.26	6.80
Max depth (m)	0.08	0.03	0.04	0.04	0.04	0.04	0.04	0.04	0.06	0.06	0.06



Chainage: (-)	0-6	6-20	20-28	28-36	36-42
Max vel (m/s)	4.30	9.58	7.70	5.82	5.95
Max depth (m)	0.19	0.06	0.08	0.10	0.10



Chainage: (-)	0-4	4-10	10-19	19-30	30-40	40-53	53-78	78-84
Max vel (m/s)	3.57	5.00	4.19	3.71	3.27	1.09	3.85	2.22
Max depth (m)	0.11	0.04	0.05	0.05	0.06	0.18	0.05	0.09

3. Future urban development scenario (Model 3)

3.1. Drainage network characteristics

Table 3D: Properties of drainage network used for the future urban development scenario (model 3).

Reach	Chainage	Length	Inlet elevation	Outlet elevation	Average slope	Manning's roughness Coefficient
	(m)	(m)	(m)	(m)	(%)	(-)
1	0-14	14.5	238.9	235.2	25.8	0.014
	14-33	18.7	235.2	230.6	24.6	0.014
	33-44	11.2	230.6	226.9	32.8	0.014
	44-56	12.2	226.9	222.9	33.0	0.014
2	0-25	25.6	224.5	222.9	6.1	0.014
3	0-35	34.8	222.6	213.2	26.9	0.014
	35-100	64.9	213.2	194.7	28.6	0.024
	100-137	36.6	194.7	188.9	15.9	0.014
	137-176	35.9	188.9	186.0	8.1	0.014
	176-181	5.4	186.0	185.0	17.7	0.014
4	0-12	12.5	187.5	187.1	3.0	0.014
	12-36	24.3	187.1	186.9	1.0	0.014
	36-82	46.1	186.9	186.4	1.0	0.014
	82-118	36.3	186.4	186.3	0.5	0.014
	118-139	20.7	186.3	186.2	0.4	0.014
5	0-37	36.8	189.7	189.0	1.9	0.032
6	0-13	13.2	167.0	166.7	2.3	0.014
	13-32	19.1	166.7	164.9	9.3	0.014
	32-62	30.2	164.9	160.1	15.9	0.014
	62-101	39.4	160.1	157.8	5.8	0.014
	101-114	12.7	157.8	157.6	2.0	0.014
7	0-11	10.7	167.1	166.9	1.8	0.014
	11-34	23.2	166.9	165.8	5.0	0.014
	34-62	27.8	165.8	164.1	6.2	0.014
	62-83	20.9	164.1	163.5	3.0	0.014
	83-100	16.8	163.5	163.3	1.0	0.014
	100-108	8.5	163.3	163.2	1.0	0.014
	108-117	9.1	163.2	163.1	1.0	0.014
	117-132	15.2	163.1	163.0	1.0	0.014
	132-148	16.2	163.0	162.8	0.9	0.014
8	0-27	27.1	113.1	109.8	12.0	0.014
	27-42	15.0	109.8	109.3	3.9	0.014
	42-50	7.9	109.3	108.6	8.4	0.014
	50-69	18.9	108.6	106.8	9.5	0.014
	69-103	34.4	106.8	101.3	16.1	0.024
	103-130	27.3	101.3	96.5	17.4	0.032
9	0-11	11.2	166.2	164.3	17.2	0.014
	11-20	9.2	164.3	162.8	15.7	0.014
	20-34	13.7	162.8	161.0	13.4	0.014

Reach	Chainage	Length	Inlet elevation	Outlet elevation	Average slope	Manning's roughness Coefficient
	(m)	(m)	(m)	(m)	(%)	(-)
9	34-51	17.1	161.0	159.2	10.7	0.014
	51-71	20.4	159.2	157.7	7.0	0.014
10	0-11	10.7	178.4	177.8	6.4	0.014
	11-27	16.4	177.8	175.8	12.2	0.024
	27-52	24.9	175.8	172.6	12.7	0.024
	52-75	23.3	172.6	170.0	11.1	0.024
	75-97	21.6	170.0	167.6	11.4	0.024
	97-121	24.0	167.6	165.8	7.3	0.014
	121-150	29.5	165.8	162.7	10.6	0.014
	150-185	35.2	162.7	157.5	14.7	0.024
	185-208	23.5	157.5	153.7	16.1	0.024
	208-233	25.1	153.7	149.9	15.3	0.024
	233-250	16.6	149.9	147.0	17.4	0.024
	250-269	19.3	147.0	142.7	22.1	0.024
	269-279	9.8	142.7	140.6	21.6	0.024
	279-287	8.1	140.6	138.1	30.8	0.024
	287-309	21.7	138.1	136.6	7.0	0.014
11	0-14	13.9	129.0	127.8	8.0	0.014
	14-24	10.1	127.8	127.1	7.4	0.014
	24-32	7.8	127.1	125.7	18.5	0.014
	32-53	21.3	125.7	124.3	6.4	0.014
	53-65	11.8	124.3	122.2	18.3	0.014
	65-72	7.1	122.2	120.6	21.4	0.014
	72-79	7.5	120.6	119.6	13.7	0.014
	79-96	17.4	119.6	118.1	8.8	0.014
	96-110	13.8	118.1	116.6	11.0	0.014
12	0-11	11.5	127.3	127.1	2.3	0.014
	11-22	11.3	127.1	126.5	5.1	0.014
	22-35	13.3	126.5	125.1	10.6	0.014
	35-46	10.8	125.1	123.7	12.5	0.014
	46-54	7.9	123.7	122.3	17.7	0.014
	54-62	7.6	122.3	121.6	10.0	0.014
	62-90	28.4	121.6	118.3	11.5	0.014
	90-118	28.2	118.3	116.5	6.4	0.014
	118-130	11.7	116.5	116.3	1.5	0.014
13	0-5	4.9	127.9	127.7	3.1	0.014
	5-9	4.0	127.7	127.6	3.1	0.014
	9-14	5.0	127.6	127.4	3.1	0.014
	14-22	7.6	127.4	127.2	3.1	0.014
	22-35	13.5	127.2	126.8	3.1	0.014
	35-46	10.9	126.8	126.5	3.1	0.014
	46-61	15.4	126.5	126.0	3.1	0.014
	61-82	21.1	126.0	123.0	14.3	0.024
	82-94	11.7	122.8	118.4	38.3	0.024
14	0-29	29.3	136.0	130.5	18.7	0.014

Reach	Chainage	Length	Inlet elevation	Outlet elevation	Average slope	Manning's roughness Coefficient
	(m)	(m)	(m)	(m)	(%)	(-)
14	29-53	23.7	130.5	126.7	16.0	0.032
	53-82	29.2	126.7	122.4	14.9	0.014
	82-110	27.6	122.4	118.4	14.5	0.014
15	0-22	22.4	118.4	114.7	16.4	0.024
	22-35	13.1	114.6	113.7	7.0	0.014
16	0-12	12.1	98.7	97.7	8.6	0.014
	12-26	14.2	97.7	96.7	6.9	0.014
	26-41	15.2	96.7	94.3	15.7	0.014
	41-50	8.7	94.3	92.2	24.0	0.014
	50-61	10.5	91.8	88.0	36.1	0.014
17	0-23	23.5	96.4	93.1	14.3	0.024
	23-51	27.5	93.1	90.6	8.8	0.014
	51-82	30.8	90.6	87.9	9.0	0.014
	82-108	26.2	87.8	84.6	12.1	0.024
	108-135	27.6	84.6	82.5	7.6	0.014
	135-164	28.6	82.5	80.7	6.4	0.014
18	0-13	12.7	80.4	79.3	8.5	0.024
	13-24	11.3	79.3	78.3	8.5	0.032
	24-36	12.3	78.3	76.8	12.8	0.032
	36-49	12.8	76.8	75.5	9.9	0.024
	49-61	11.6	75.5	73.5	17.1	0.024
	61-76	14.7	73.5	70.7	19.1	0.024
	76-104	27.7	70.7	64.7	21.6	0.024
	104-131	27.3	64.7	59.6	18.7	0.024
	131-157	26.5	59.6	56.4	12.0	0.024
	157-165	8.4	56.4	55.9	6.2	0.014
20	0-6	6.3	186.2	185.2	15.4	0.032
	6-44	38.0	185.2	162.8	58.9	0.024
	44-74	30.2	162.8	148.4	48.0	0.024
	74-80	6.3	148.4	145.3	48.5	0.014
	80-89	8.8	145.3	140.9	49.4	0.024
	89-97	8.3	140.9	134.7	74.6	0.024
	97-138	41.4	134.7	114.2	49.6	0.014
	138-152	13.8	114.2	106.9	52.7	0.024
	152-168	16.4	106.9	98.7	50.3	0.024
	168-210	41.5	98.7	83.3	37.0	0.024
	210-215	4.7	83.3	81.1	48.1	0.024
21	0-6	5.6	116.4	116.0	7.0	0.014
	6-20	13.9	116.0	103.4	90.1	0.032
	20-28	8.0	103.4	99.7	46.6	0.024
	28-36	8.0	99.7	97.9	22.1	0.024
	36-42	5.8	97.9	96.7	21.1	0.024
22	0-4	4.4	157.6	157.4	3.0	0.014
	4-10	5.5	157.4	155.3	39.4	0.014
	10-19	8.9	155.3	149.4	65.5	0.024

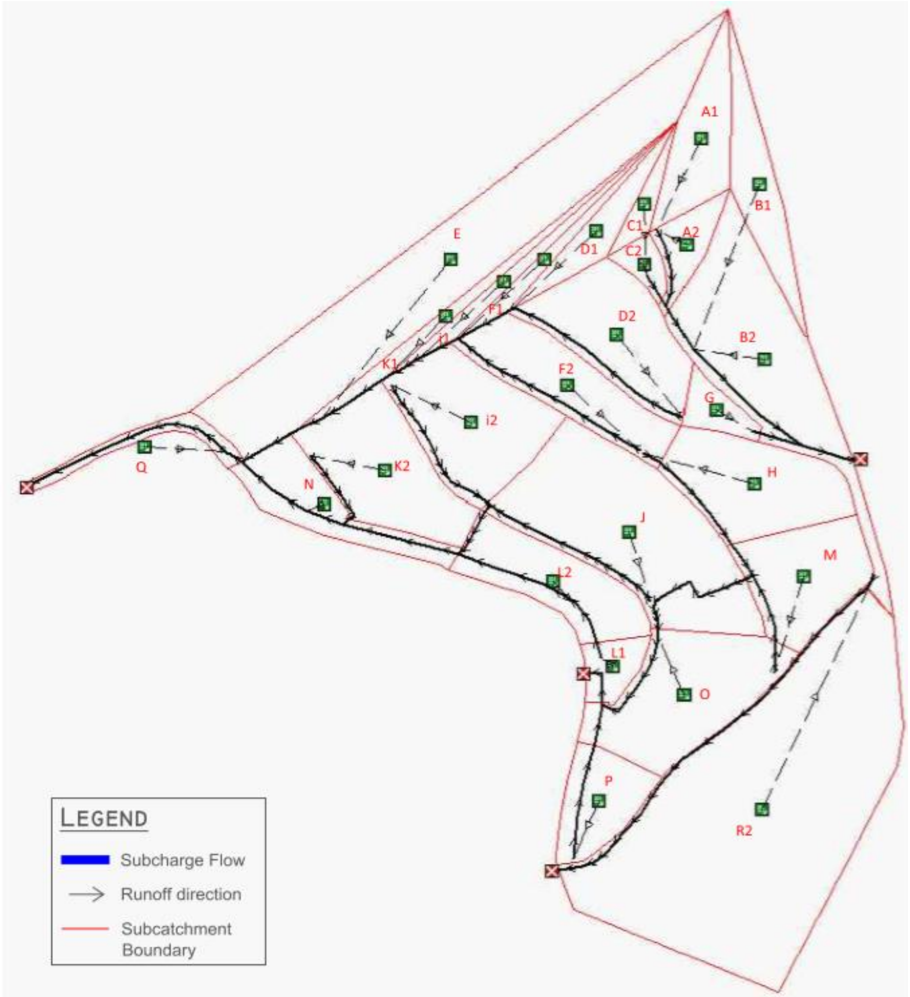
Reach	Chainage	Length	Inlet elevation	Outlet elevation	Average slope	Manning's roughness Coefficient
	(m)	(m)	(m)	(m)	(%)	(-)
22	19-30	11.6	149.4	144.2	44.5	0.024
	30-40	10.0	144.2	141.3	29.7	0.024
	40-53	13.0	141.3	141.1	1.0	0.024
	53-78	25.5	141.1	128.4	49.8	0.024
	78-84	5.6	128.4	127.9	8.7	0.024

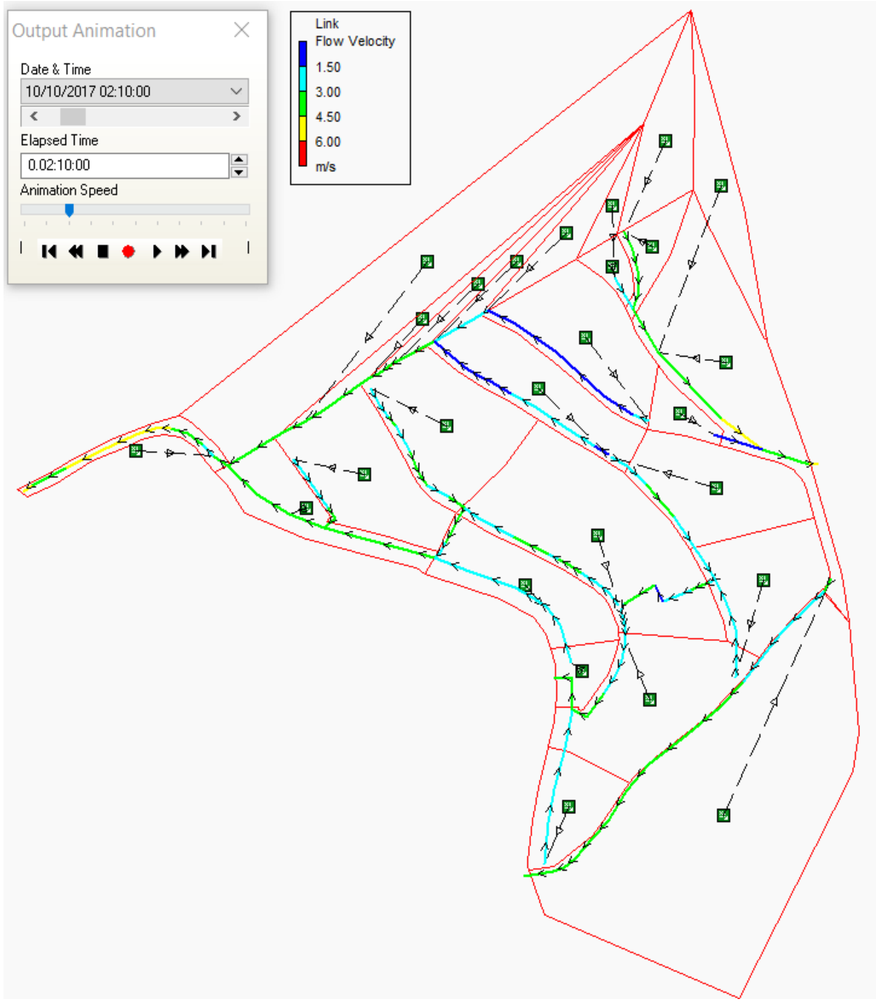
3.2. Cross-sections of storm drains

Refer to Chapter 2.2 for the storm drains’ cross-section used for this model simulation.

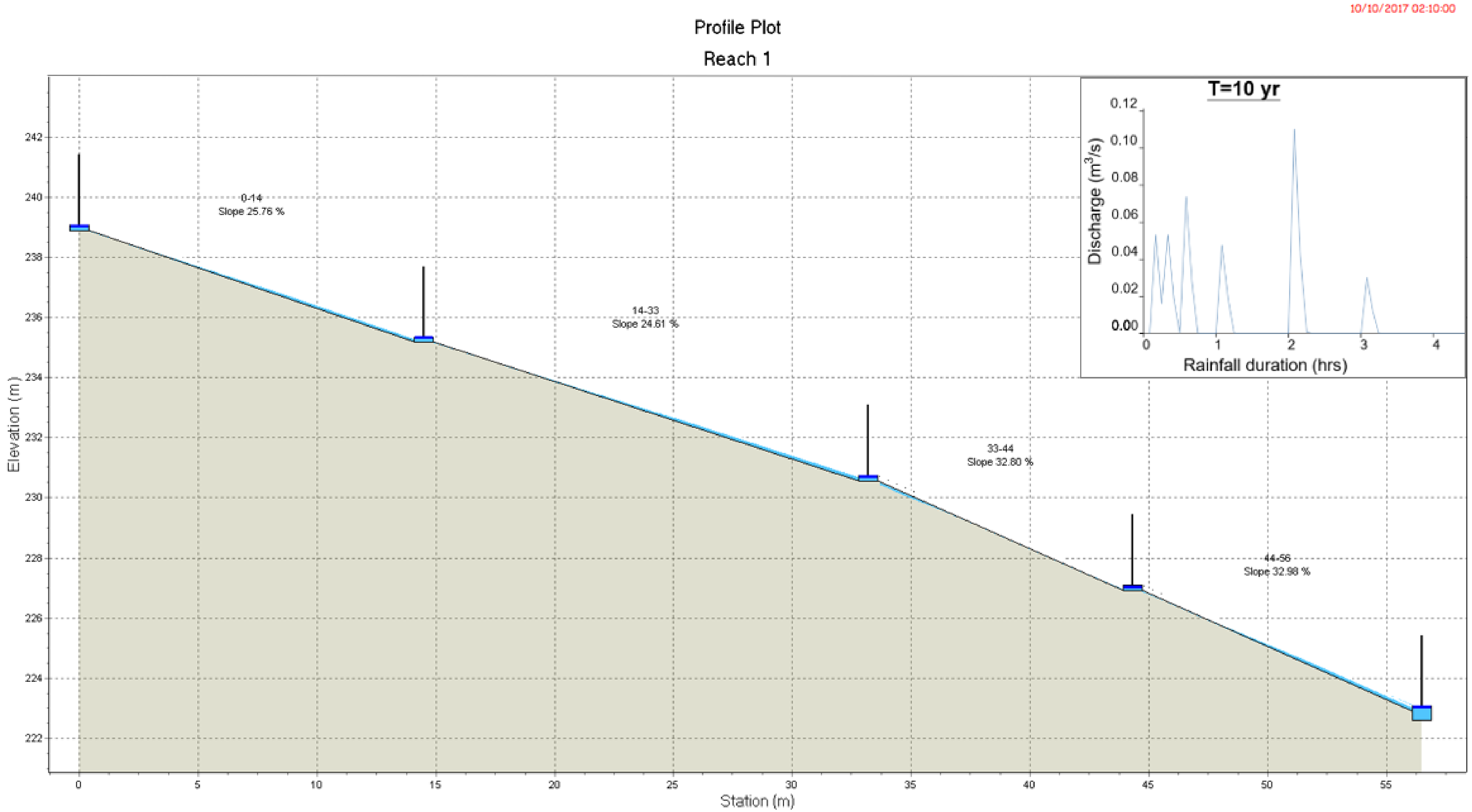
3.3. Simulation results

3.3.1. Overview of the drainage network

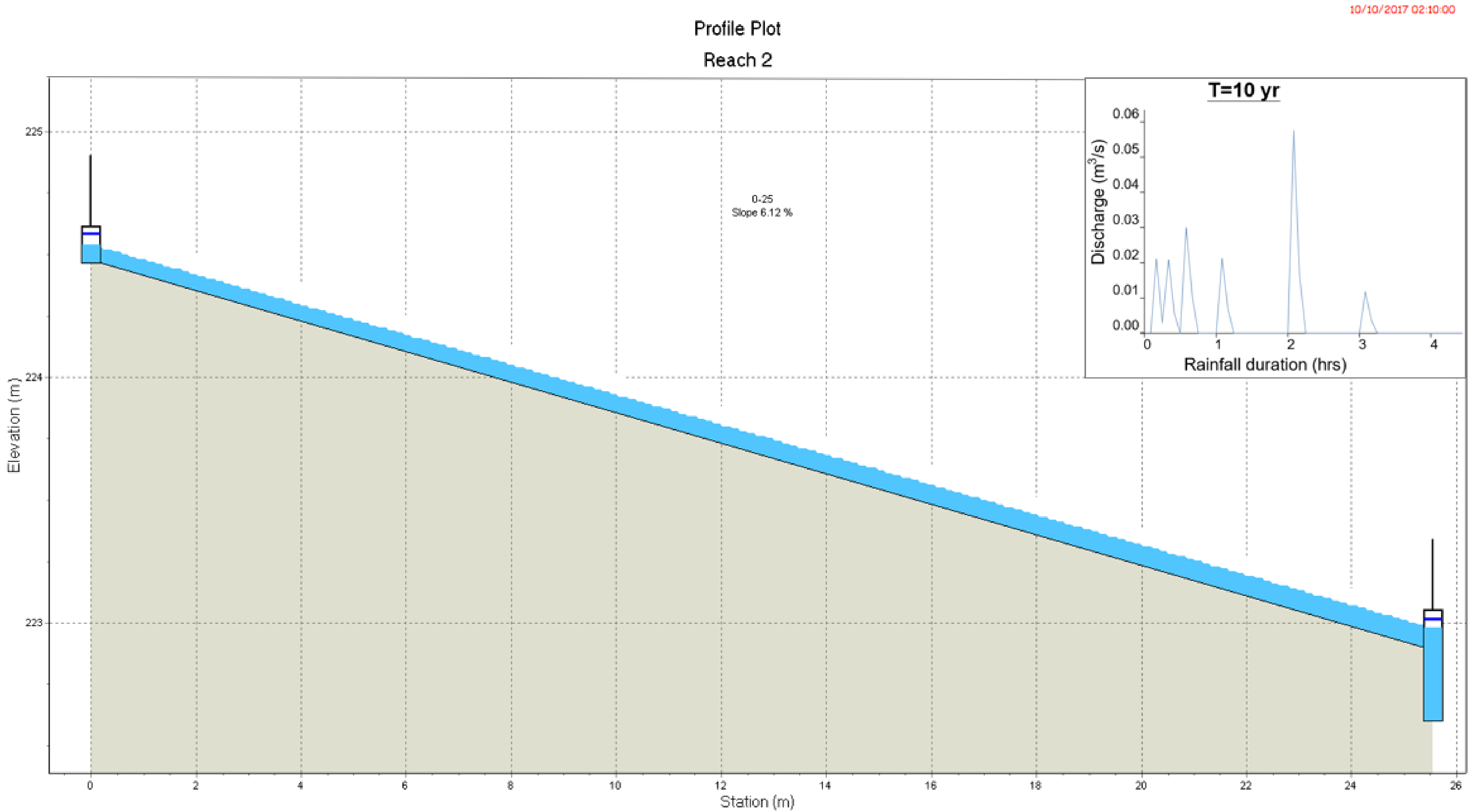




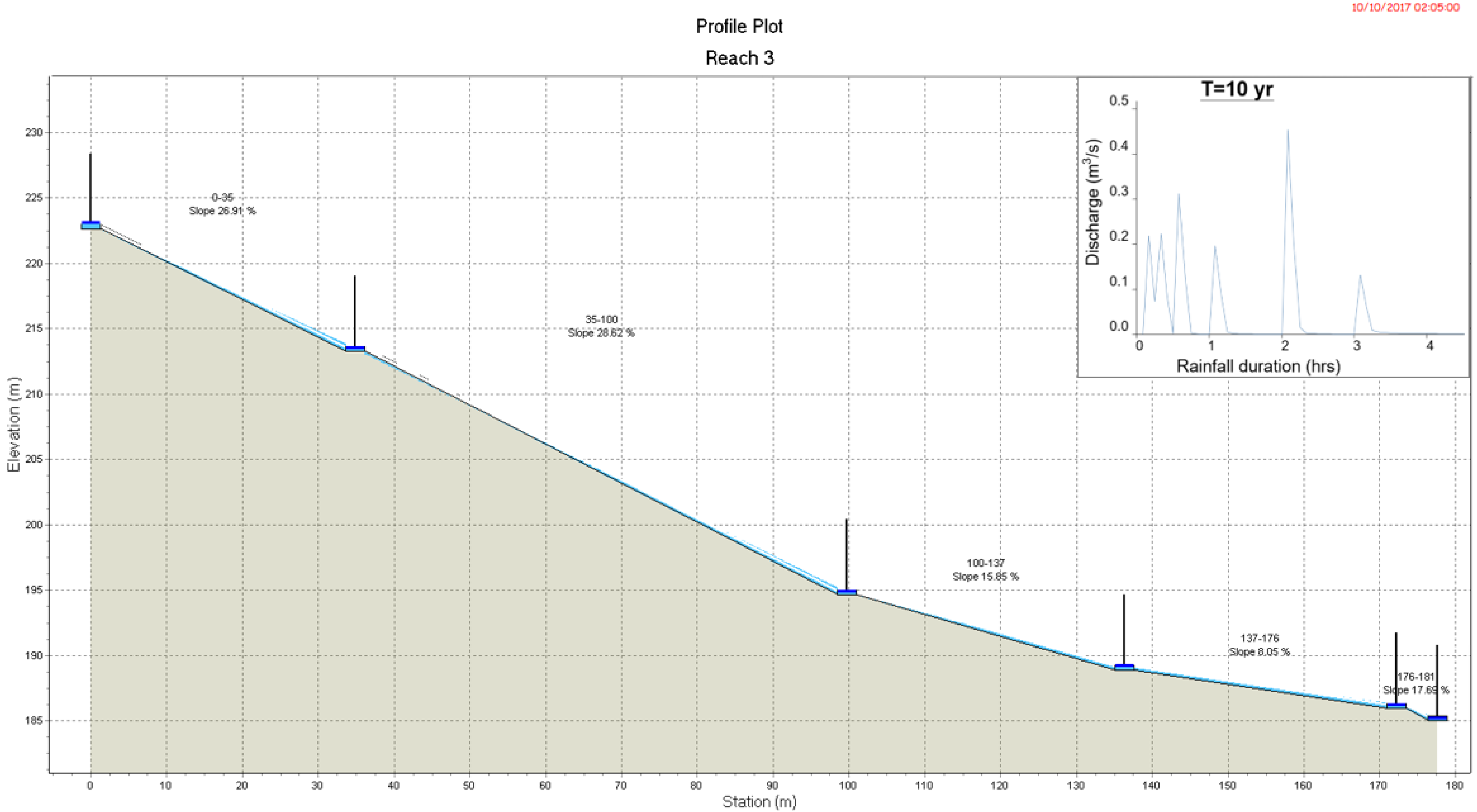
3.3.2. Longitudinal profile plots



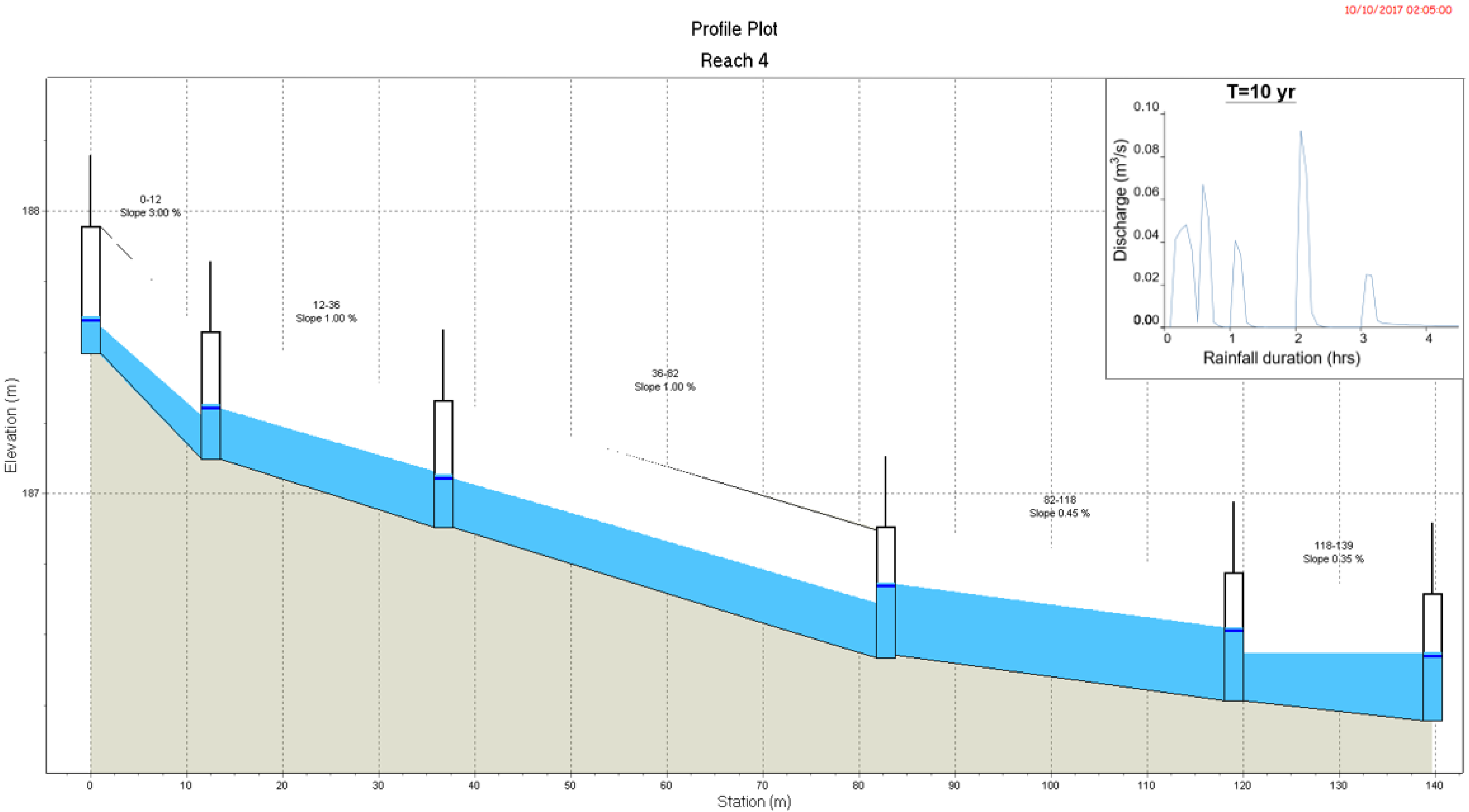
Chainage: (-)	0-14	14-33	33-44	44-56
Max vel (m/s)	3.99	4.10	4.45	4.37
Max depth (m)	0.13	0.13	0.12	0.12



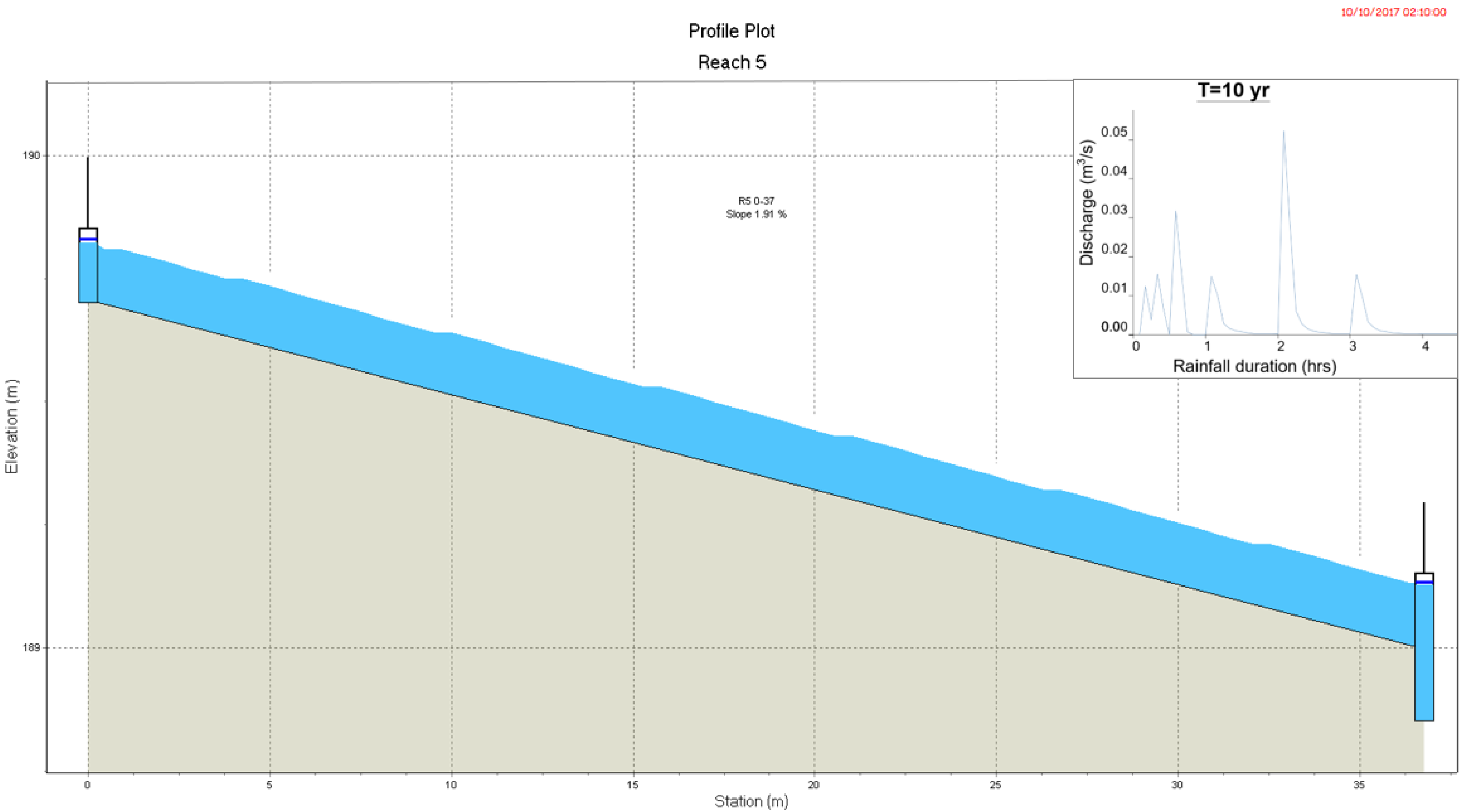
Chainage: (-)	0-25
Max vel (m/s)	3.40
Max depth (m)	0.12



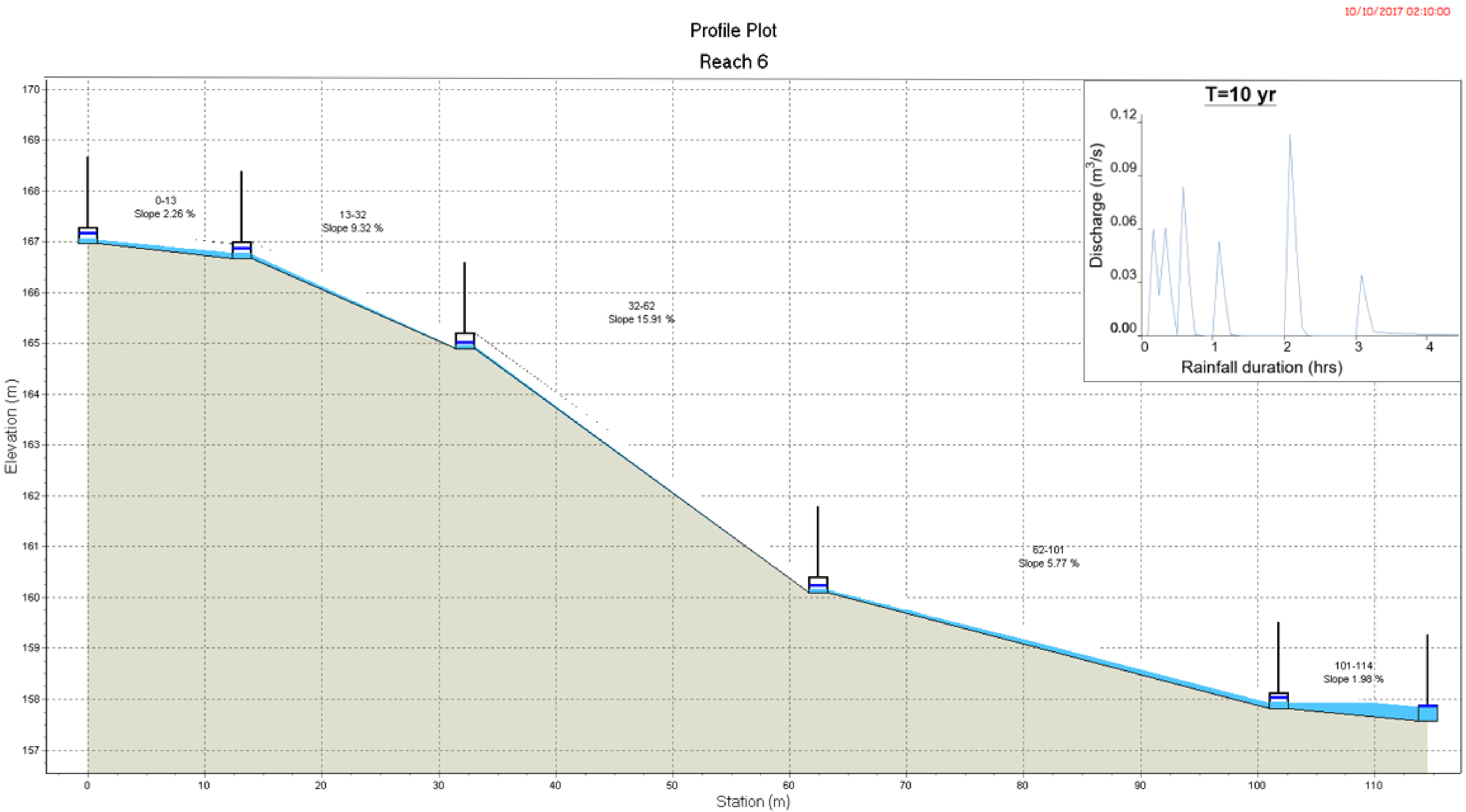
Chainage: (-)	0-35	35-100	100-137	137-176	176-181
Max vel (m/s)	5.70	5.11	5.87	4.85	5.96
Max depth (m)	0.13	0.13	0.19	0.24	0.19



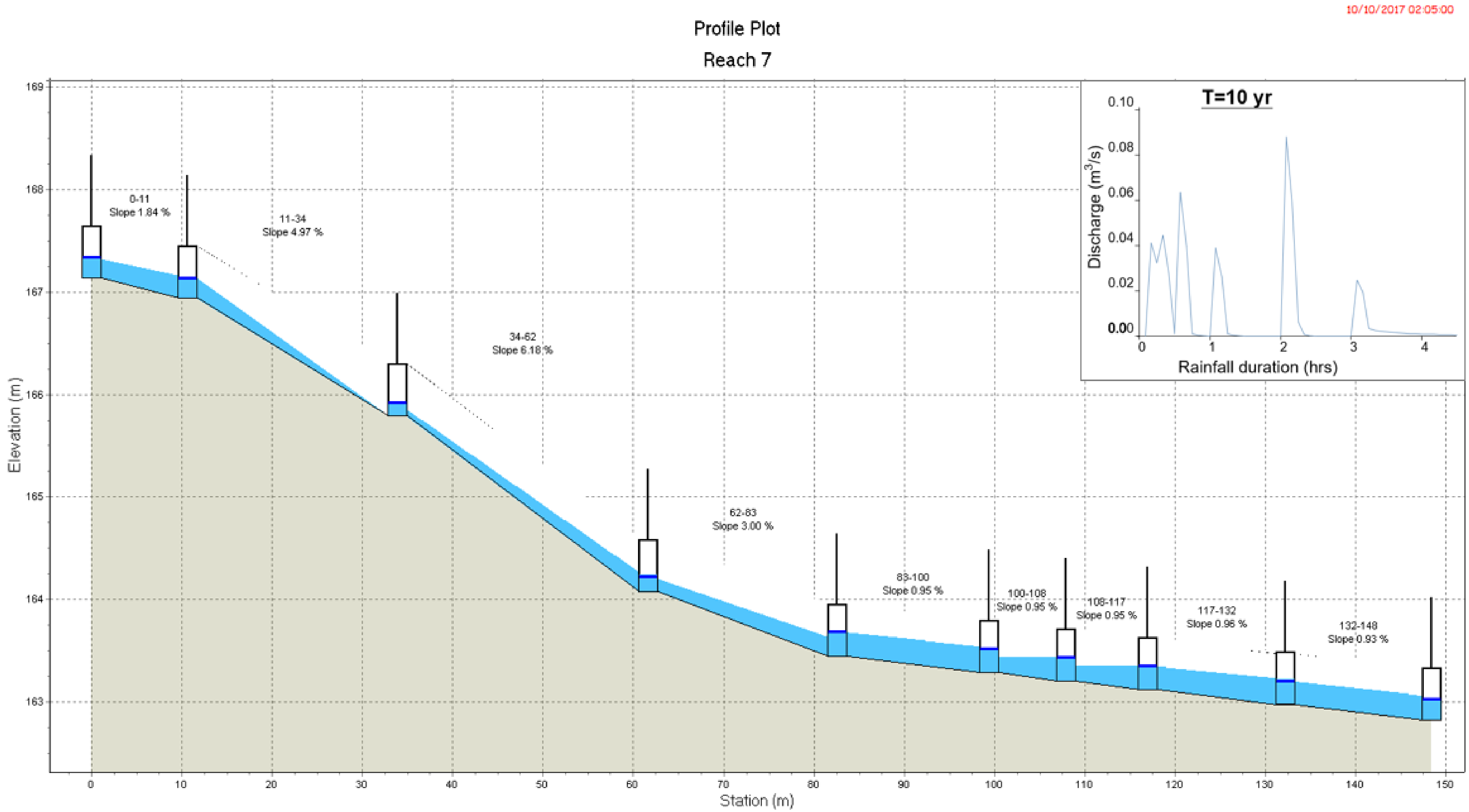
Chainage: (-)	0-12	12-36	36-86	82-118	118-139
Max vel (m/s)	2.51	1.69	1.70	1.18	1.05
Max depth (m)	0.13	0.19	0.18	0.24	0.25



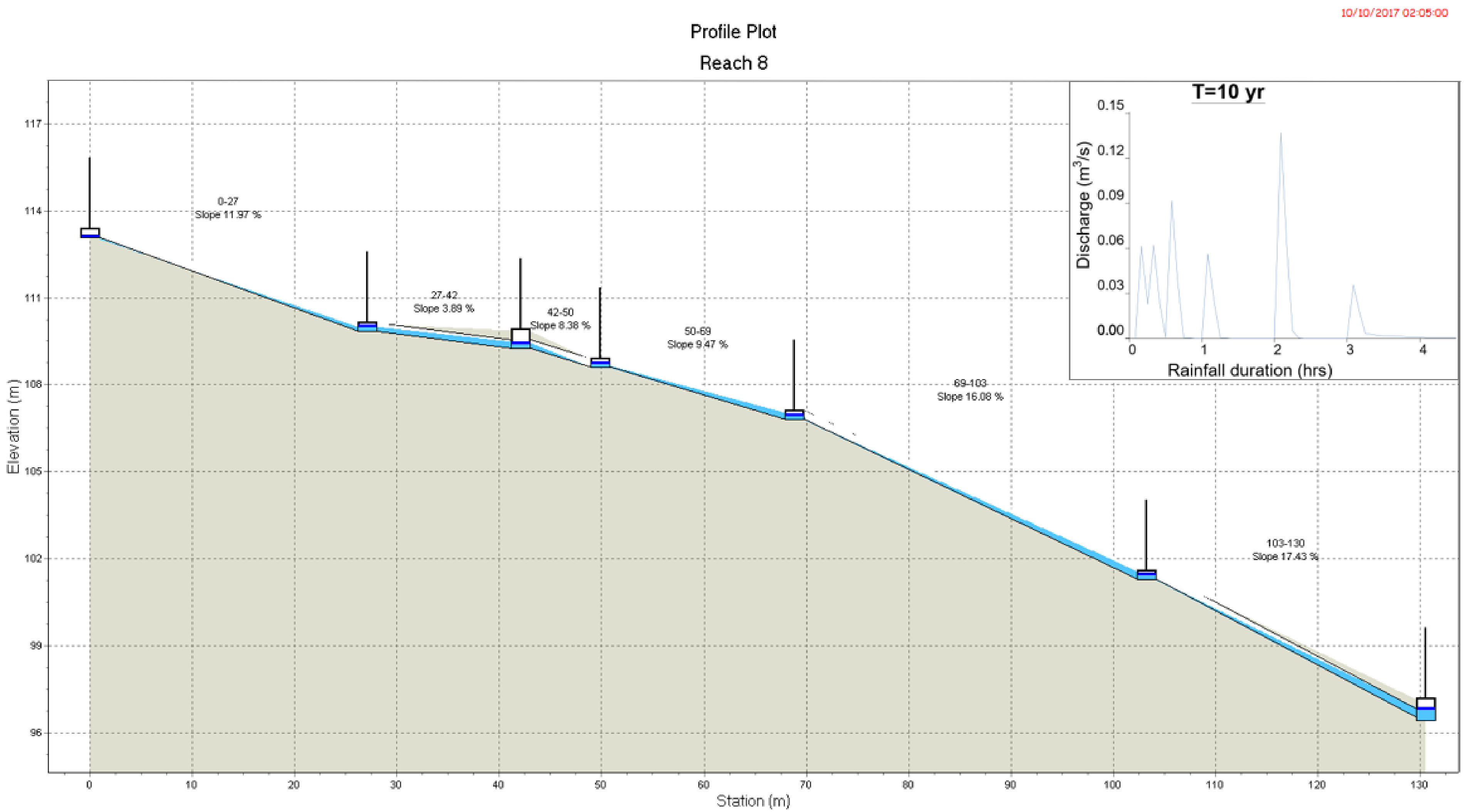
Chainage: (-)	0-37
Max vel (m/s)	2.05
Max depth (m)	0.14



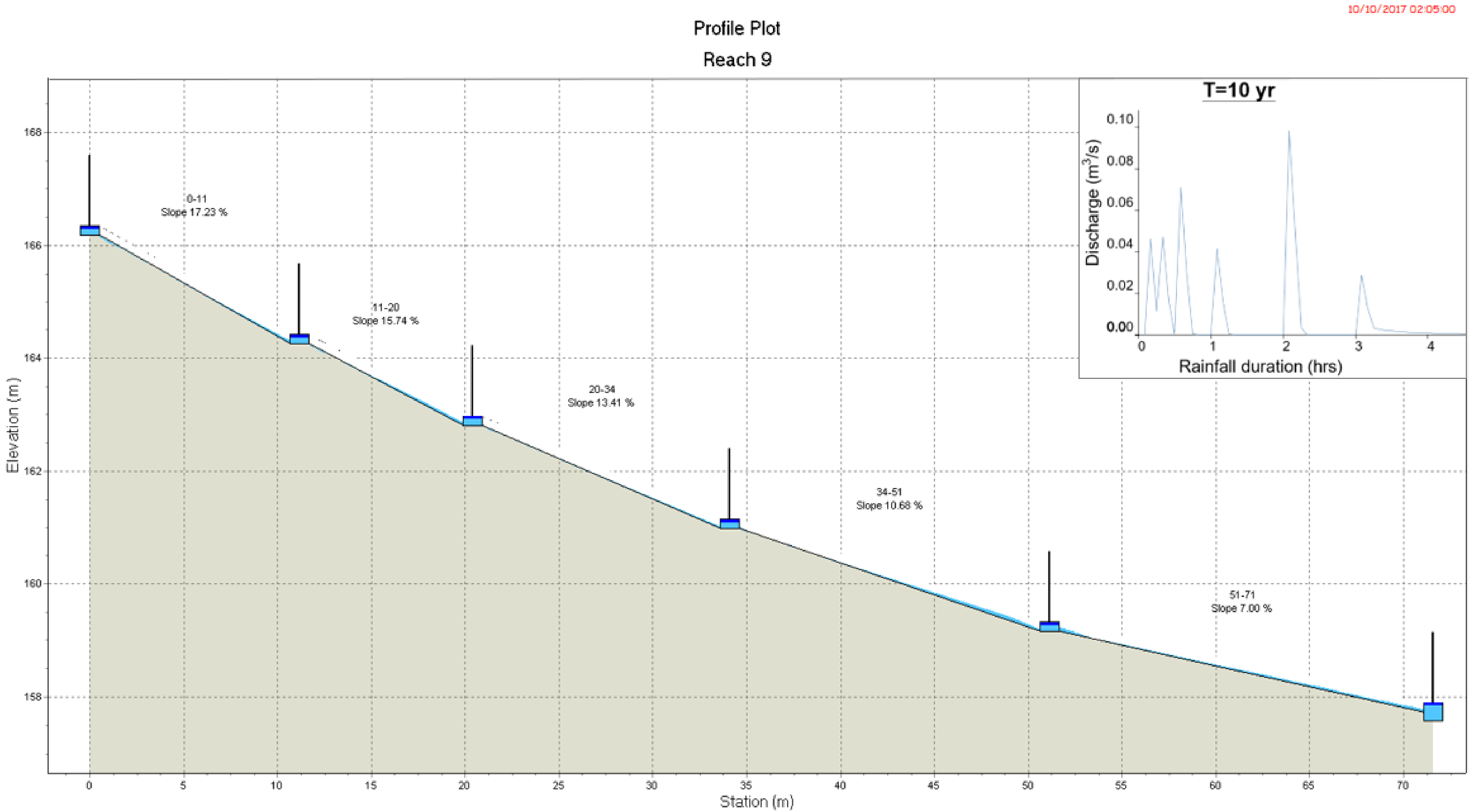
Chainage: (-)	0-13	13-32	32-62	62-101	101-114
Max vel (m/s)	2.11	3.64	4.41	3.05	2.02
Max depth (m)	0.20	0.12	0.09	0.13	0.20



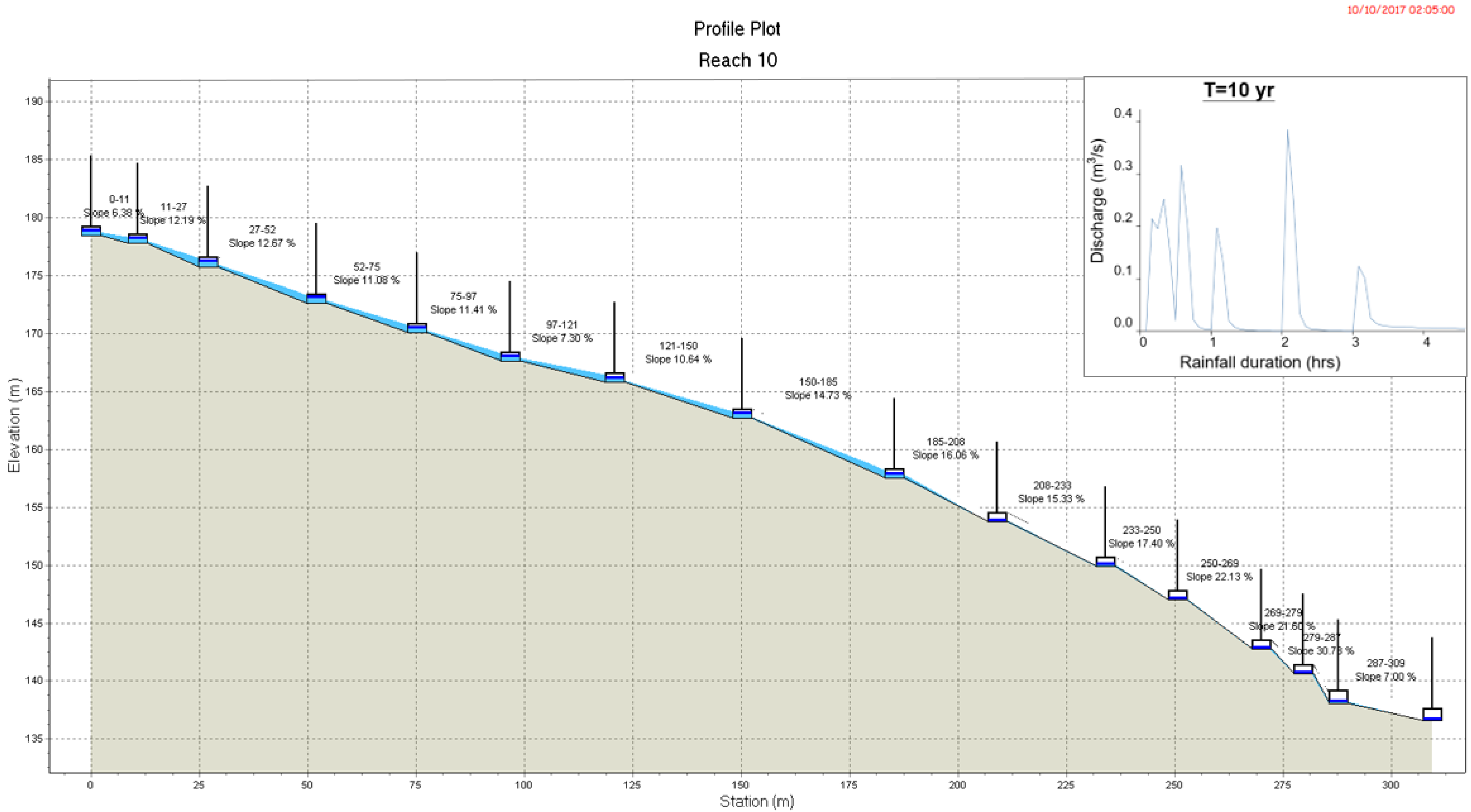
Chainage: (-)	0-11	11-34	34-62	62-83	83-100	100-108	108-117	117-132	132-148
Max vel (m/s)	1.92	2.81	3.05	2.32	1.48	1.47	1.47	1.43	1.50
Max depth (m)	0.20	0.13	0.12	0.16	0.24	0.23	0.23	0.23	0.21



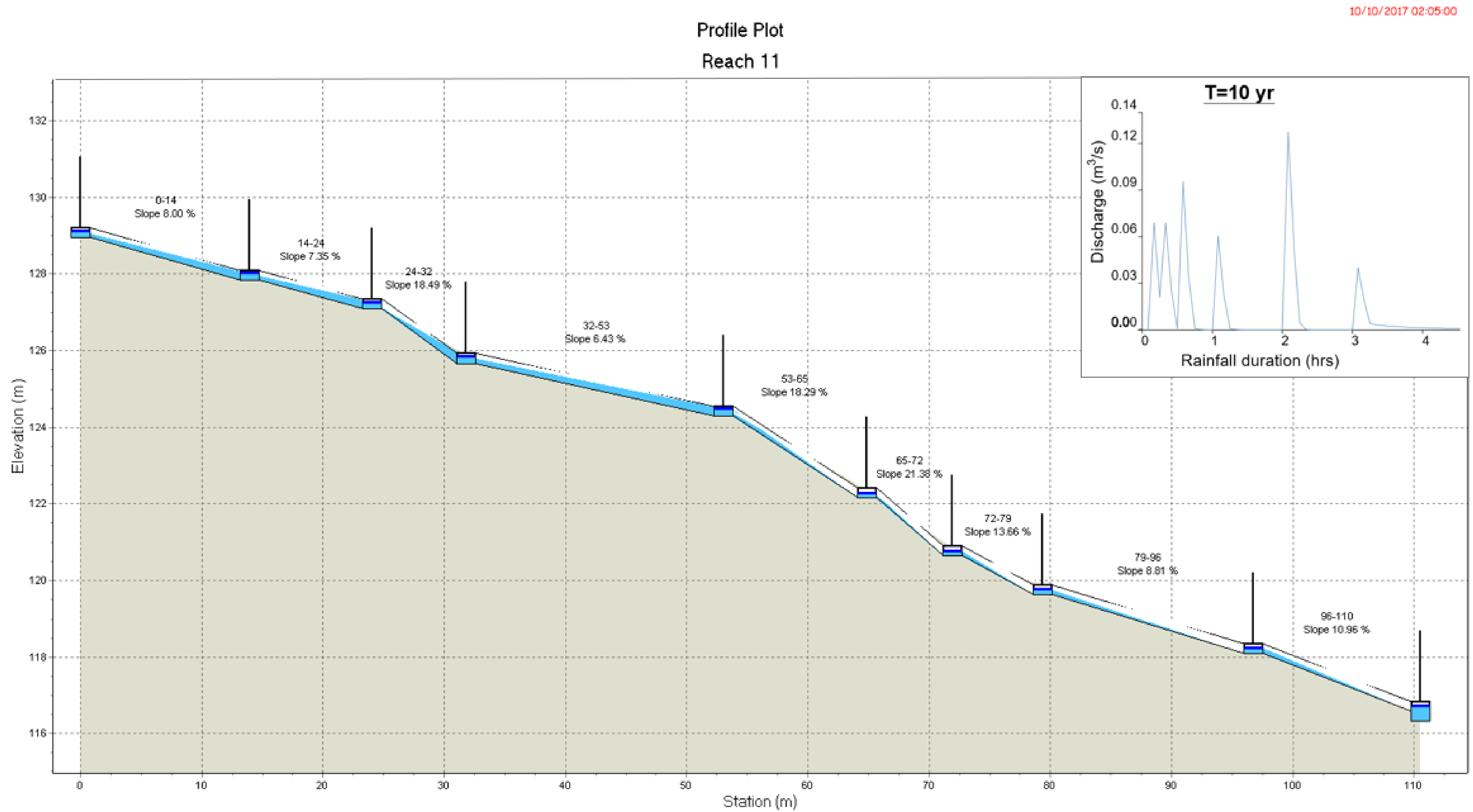
Chainage: (-)	0-27	27-42	42-50	50-69	69-103	103-130
Max vel (m/s)	2.97	2.76	3.70	3.86	3.15	2.59
Max depth (m)	0.05	0.19	0.14	0.13	0.16	0.19



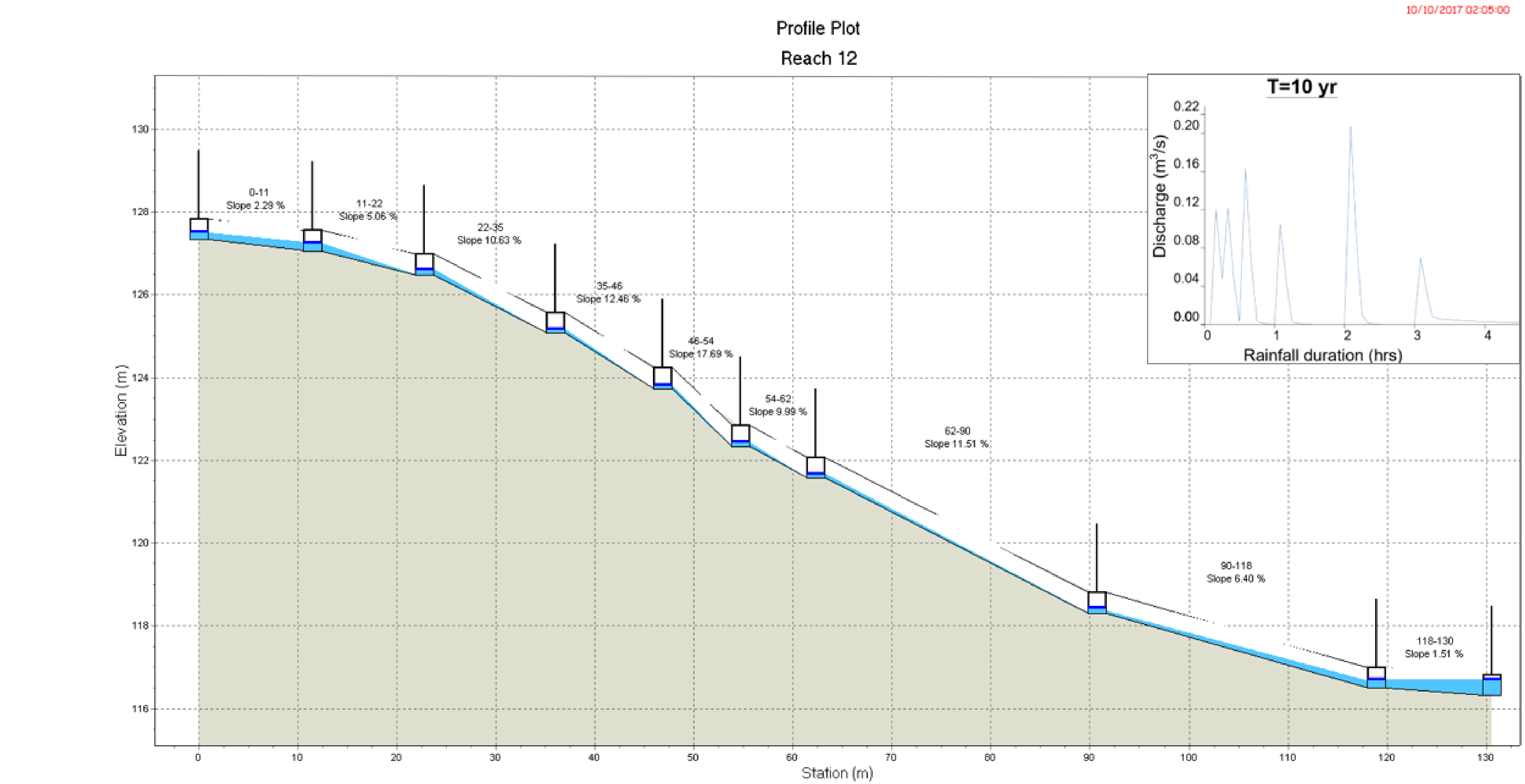
Chainage: (-)	0-11	11-20	20-34	34-51	51-71
Max vel (m/s)	3.23	3.11	2.91	2.72	2.33
Max depth (m)	0.13	0.13	0.13	0.13	0.14



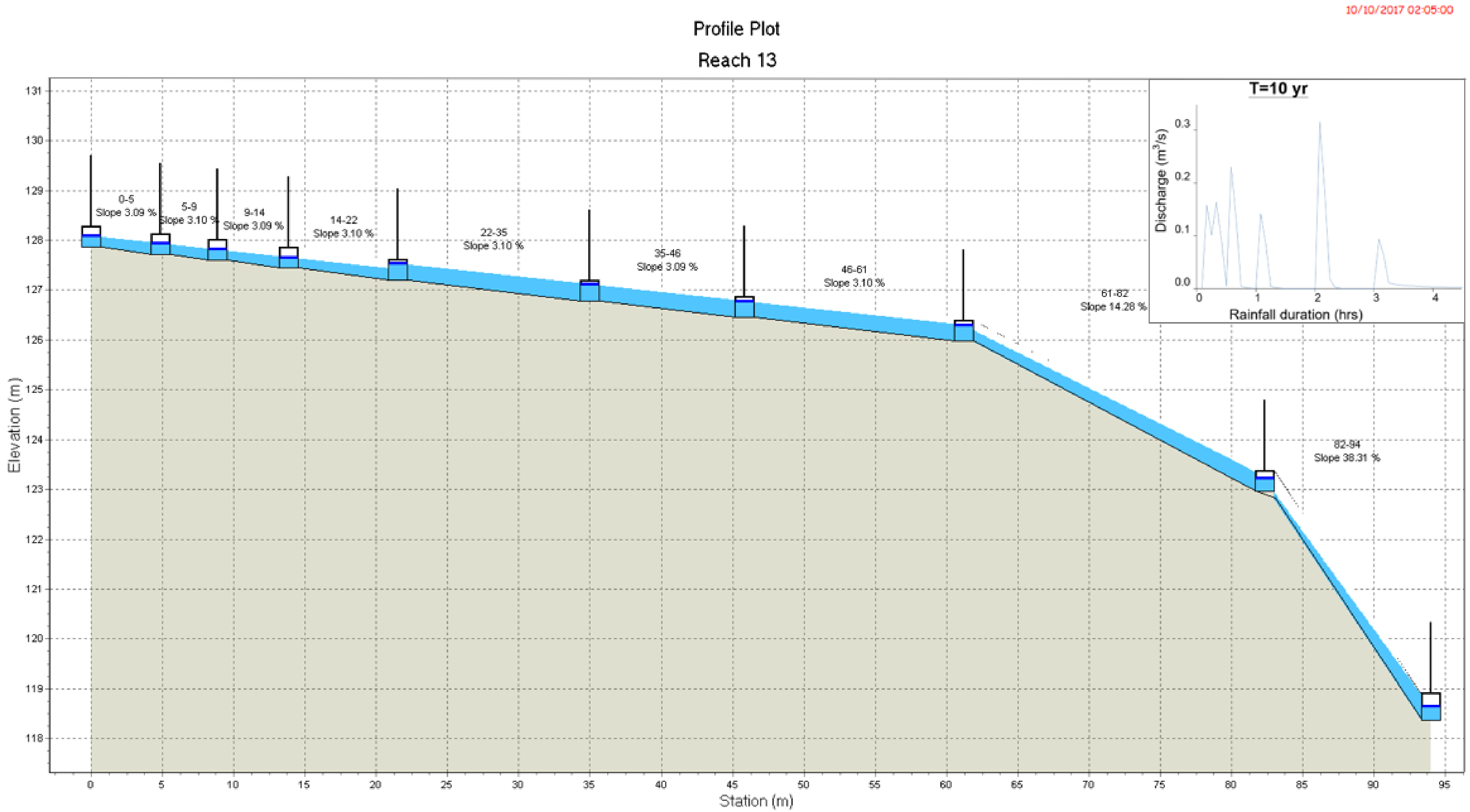
Chainage: (-)	0-11	11-27	27-52	52-75	75-97	97-121	121-150	150-185	185-208	208-253	233-250	250-269	269-279	279-287	287-309
Max vel (m/s)	4.13	3.44	3.51	3.30	3.34	4.38	5.12	3.74	3.98	3.92	4.09	4.44	4.40	4.96	4.21
Max depth (m)	0.40	0.47	0.46	0.48	0.46	0.35	0.29	0.39	0.15	0.15	0.14	0.13	0.13	0.11	0.12



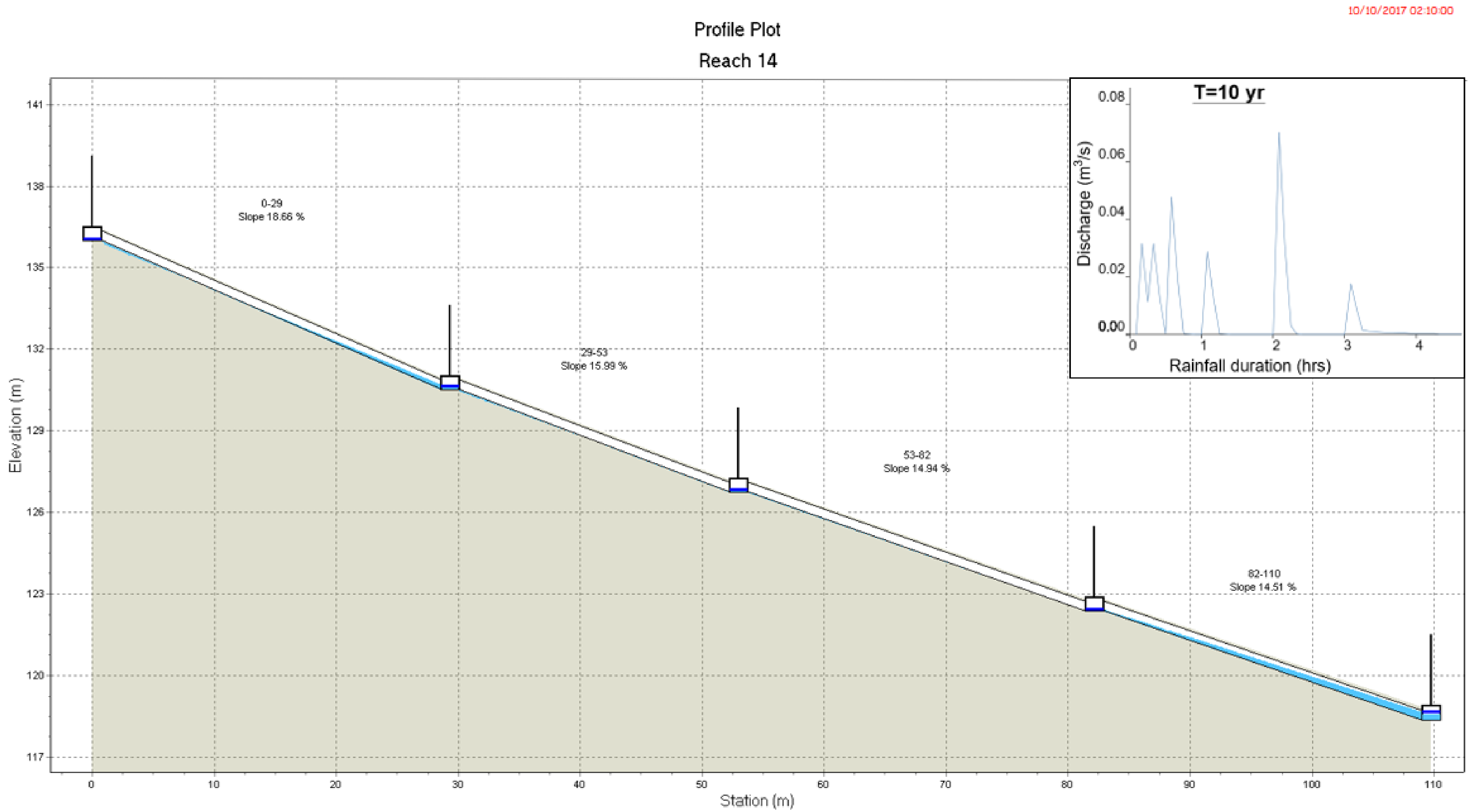
Chainage: (-)	0-14	14-24	24-32	32-53	53-65	65-72	72-79	79-96	96-110
Max vel (m/s)	3.50	3.39	4.83	3.22	4.80	5.08	4.29	3.63	3.95
Max depth (m)	0.16	0.16	0.11	0.17	0.11	0.11	0.13	0.15	0.13



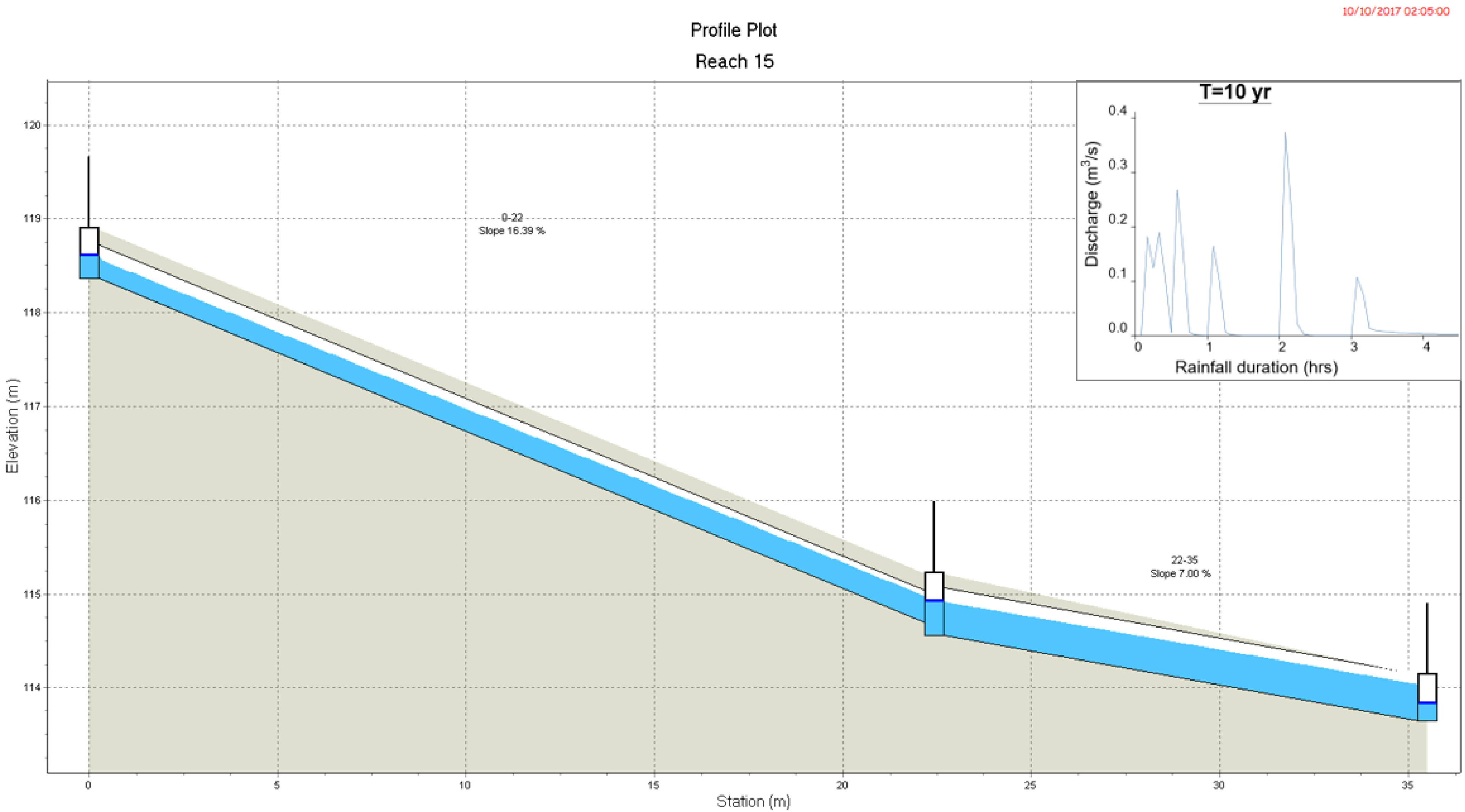
Chainage: (-)	0-11	11-22	22-35	35-46	46-54	54-62	62-90	90-118	118-130
Max vel (m/s)	2.47	3.28	4.25	4.48	5.05	4.16	4.36	3.55	2.12
Max depth (m)	0.19	0.14	0.11	0.10	0.09	0.11	0.10	0.12	0.20



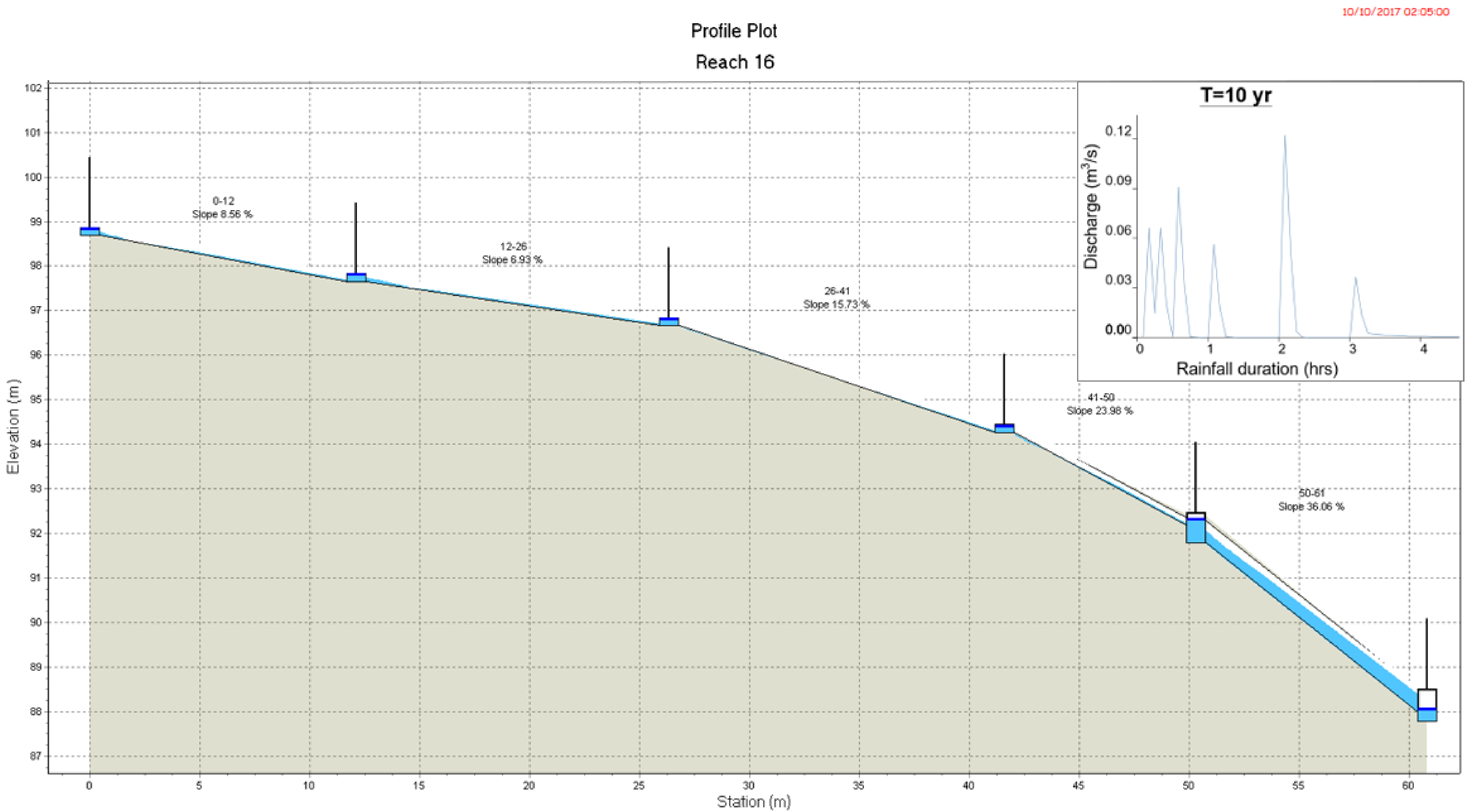
Chainage: (-)	0-5	5-9	9-14	14-22	22-35	35-46	46-61	61-82	82-94
Max vel (m/s)	2.78	2.78	2.78	3.05	3.05	3.05	3.05	3.66	5.37
Max depth (m)	0.21	0.21	0.21	0.32	0.32	0.32	0.32	0.26	0.13



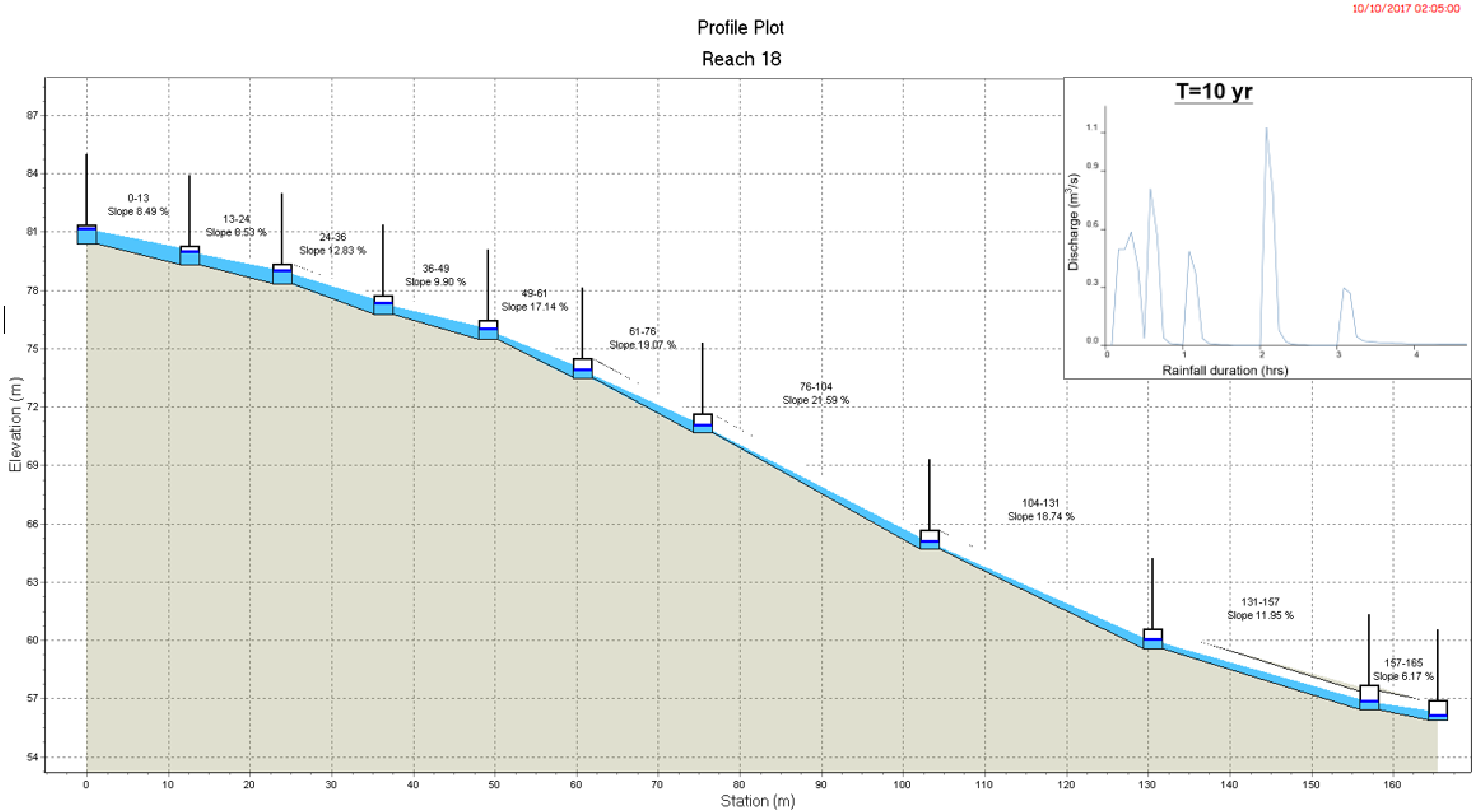
Chainage: (-)	0-29	29-53	53-82	82-110
Max vel (m/s)	3.96	2.13	3.68	3.64
Max depth (m)	0.06	0.10	0.06	0.06



Chainage: (-)	0-22	22-35
Max vel (m/s)	4.65	4.26
Max depth (m)	0.25	0.19

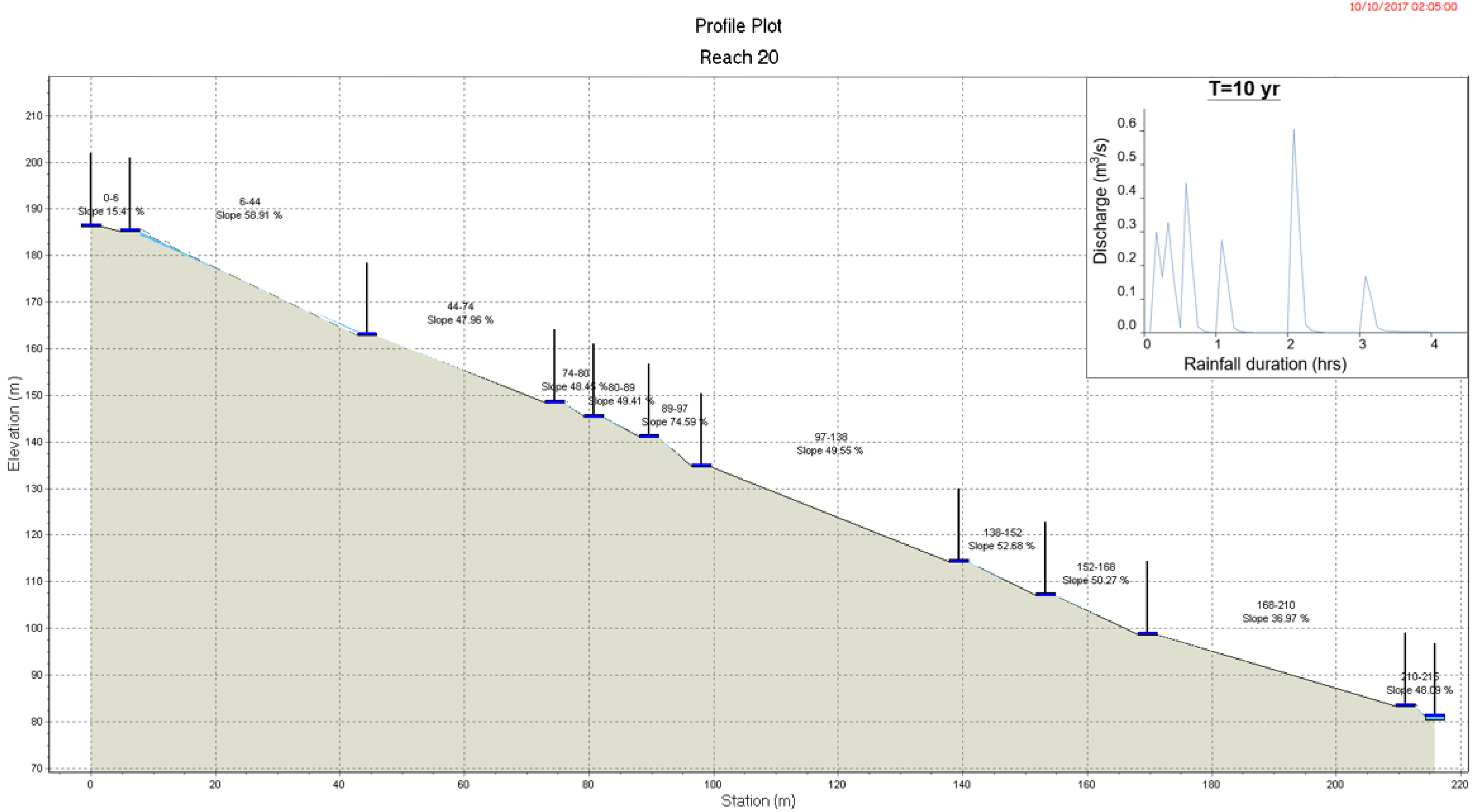


Chainage: (-)	0-12	12-26	26-41	41-50	50-61
Max vel (m/s)	2.34	2.21	3.31	3.77	5.31
Max depth (m)	0.14	0.15	0.14	0.13	0.05



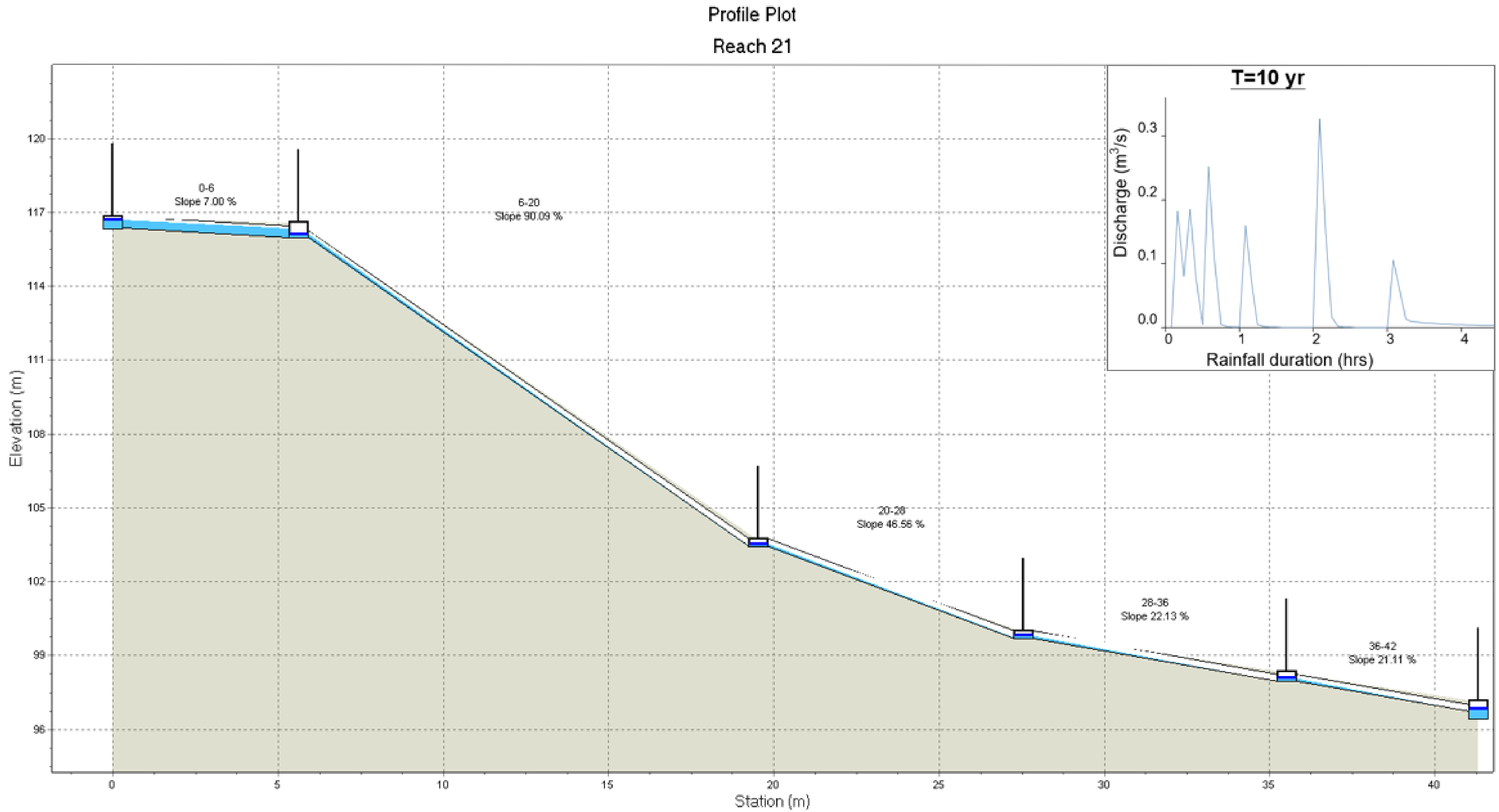
Chainage: (-)	0-13	13-24	24-66	36-49	49-61	61-76	76-104	104-131	131-157	157-165
Max vel (m/s)	4.15	3.29	3.90	4.44	5.53	5.77	5.84	5.74	4.80	5.59
Max depth (m)	0.51	0.65	0.55	0.48	0.38	0.37	0.34	0.36	0.42	0.22

10/10/2017 02:05:00

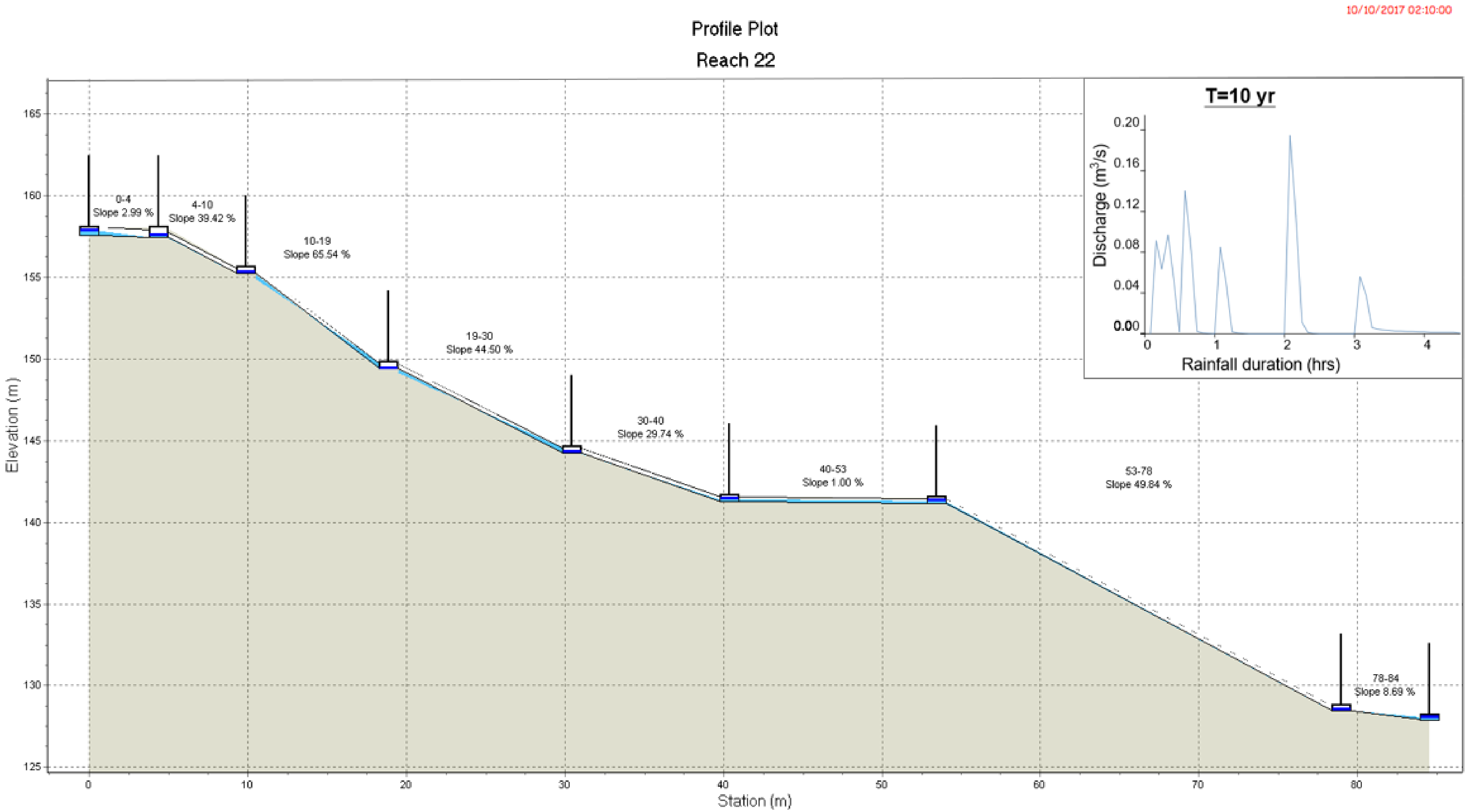


Chainage: (-)	0-6	6-44	44-74	74-80	80-89	89-97	97-138	138-152	152-168	168-210	210-215
Max vel (m/s)	2.07	3.77	4.45	4.66	4.69	5.34	4.91	5.00	5.96	5.41	5.88
Max depth (m)	0.07	0.04	0.06	0.06	0.06	0.05	0.07	0.06	0.10	0.11	0.10

10/10/2017 02:05:00



Chainage: (-)	0-6	6-20	20-28	28-36	36-42
Max vel (m/s)	4.22	5.68	5.29	4.13	4.06
Max depth (m)	0.16	0.09	0.09	0.12	0.12



Chainage: (-)	0-4	4-10	10-19	19-30	30-40	40-53	53-78	78-84
Max vel (m/s)	2.77	5.46	4.58	4.05	3.57	1.18	4.18	2.40
Max depth (m)	0.16	0.04	0.05	0.06	0.06	0.18	0.05	0.09

Appendix E. Drawings

Table of Contents

1. Introduction

The drawings presented in this appendix is pertaining for both road and stormwater drainage infrastructure upgrade for the Waymouth Hills. The design of these infrastructures was solely pertaining to the trajectory road or also refer to as side roads (such as Paradise Hills road, the Quil road, Brimstone road, Mouth Pele road, and Mount Souffriere road). However, due to the lack field surveyed data were available from Mouth Pele and Mount Souffriere road, drawings from these roads were not carried out and are not included in this appendix. Future more, the drawings pertaining to Mildrium road (the main road) were carried out by ICE hence were not included in this report.

Furthermore, to indicate the location of the storm drains presented in the drawings, Figure 1E and figure 2E presented below can be used as a guide.

Figure 1E: Road network of the Waymouth Hills.

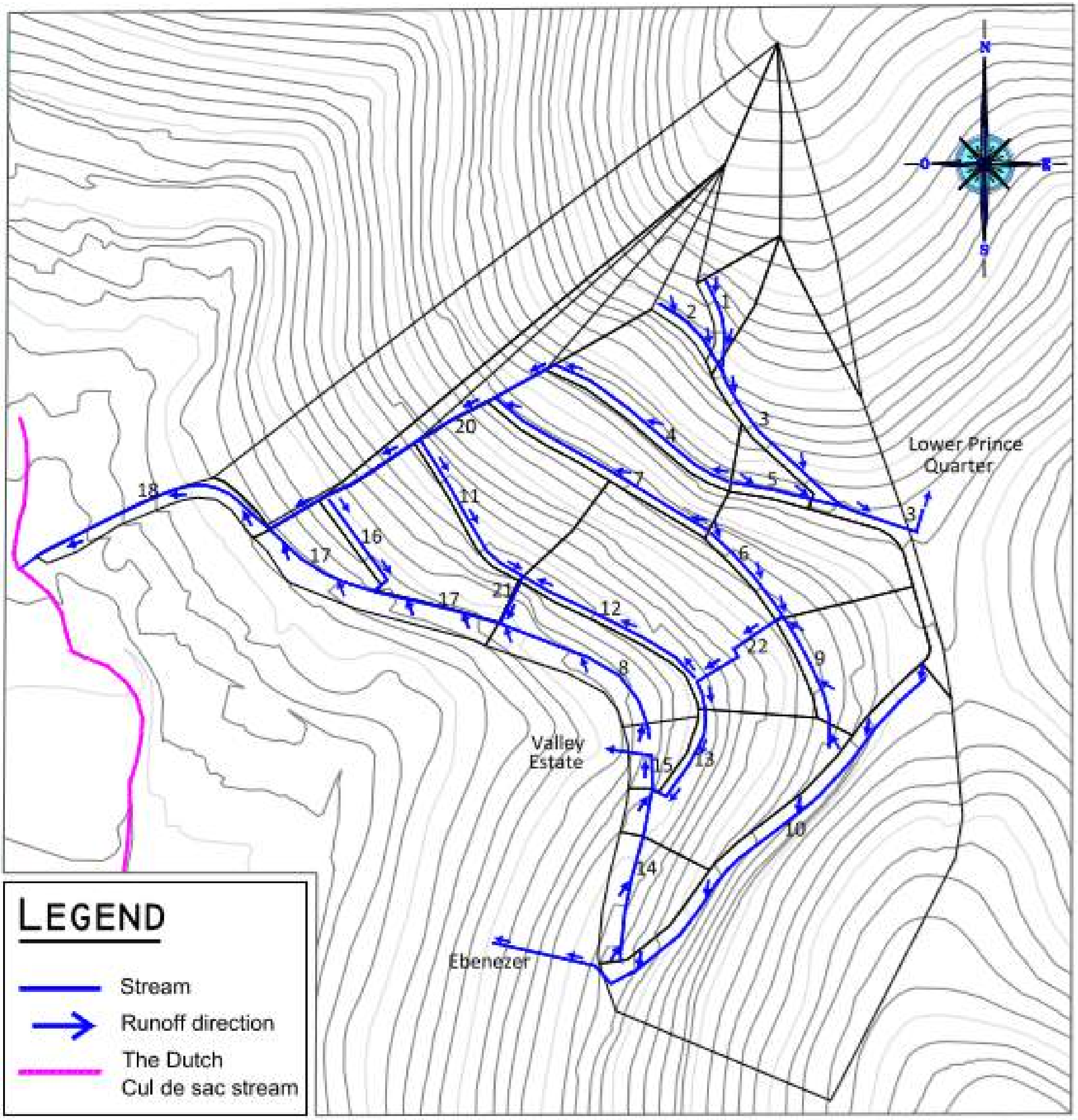


Figure 2E: Contour map of the Waymouth Hills illustrating the Reach paths.

2. Quil Road

Table 1E: Drawing list of Quil Road

Proj.nr.: **Client:**	**216-1380** **VROMI**	**Project:** **Link 6** **Division:** **Side roads**		**Date:** **18/07/2017**	
	CONCEPT DESIGN	**Quil Road**			
drawing nr.	**Subject**	**Format**	**Scale**	**Date**	**Rev.**
C35	Survey plan	A1	1:500	18-Jul-2017	
C36	Layout road plan	A1	1:500	18-Jul-2017	
C37	Typical road layout	Tabloid	1:50/2000	18-Jul-2017	
C38A	Typical road section [Alternative A]	A2	1:50/2000	18-Jul-2017	
C38B	Typical road section [Alternative B]	A2	1:1000	18-Jul-2017	
C39	Longitudinal profile	Tabloid	1:100	18-Jul-2017	
C40	Sections SL-1/ SL-5	Tabloid	1:100	18-Jul-2017	
C41	Sections SL-6/SL-10	Tabloid	1:100	18-Jul-2017	
C42	Sections SL-11/SL-15	Tabloid	1:100	18-Jul-2017	
C43	Detail crossing 2	Tabloid	1:100	18-Jul-2017	

3. Brimstone Hill Road

Table 2E: Drawing list of Brimstone Hill Road

Proj.nr.: Client:	216-1380 VROMI	Project: Division:	Link 6 Side roads	Date:	18/07/2017
	CONCEPT DESIGN	**Brimstone Hill Road**			
drawing nr.	**Subject**	**Format**	**Scale**	**Date**	**Rev.**
C50	Survey plan	A1	1:500	18-Jul-2017	
C51	Layout road plan	A1	1:500	18-Jul-2017	
C52A	Typical road section/ layout [Alternative A]	A2	1:50/2000	18-Jul-2017	
C52B	Typical road section/ layout [Alternative B]	A2	1:50/2000	18-Jul-2017	
C53	Longitudinal profile	Tabloid	1:1000	18-Jul-2017	
C54	Sections SL-1/ SL-5	Tabloid	1:100	18-Jul-2017	
C55	Sections SL-6/SL-10	Tabloid	1:100	18-Jul-2017	
C56	Sections SL-11/SL-15	Tabloid	1:100	18-Jul-2017	
C57	Sections SL-16/SL-17	Tabloid	1:100	18-Jul-2017	
C58	Detail crossing 3	Tabloid	1:100	18-Jul-2017	

4. Paradise Hill Road

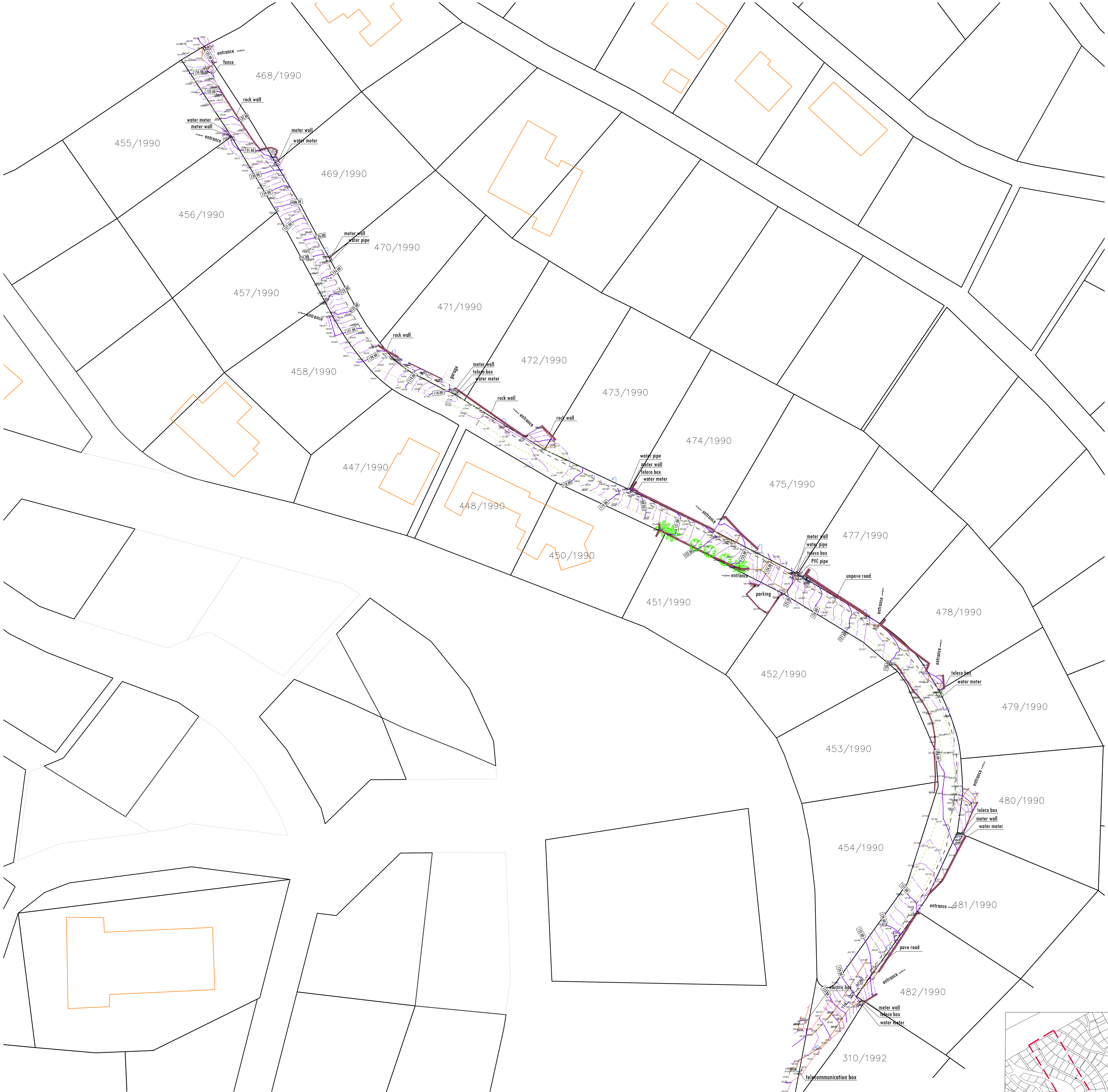
Table 3E: Drawing list of Paradise Hill Road

Proj.nr.: **Client:**	**216-1380** **VROMI**	**Project:** **Division:**	**Link 6** **Side roads**	**Date:**	**18/07/2017**
	CONCEPT DESIGN	**Paradise Hill Road**			
drawing nr.	**Subject**	**Format**	**Scale**	**Date**	**Rev.**
C60	Survey plan	Tabloid	1:250	18-Jul-2017	
C61	Layout road plan	Tabloid	1:250	18-Jul-2017	
C62A	Typical road section/ layout [Alternative A]	Tabloid	1:50/1000	18-Jul-2017	
C62B	Typical road section/ layout [Alternative B]	Tabloid	1:50/1000	18-Jul-2017	
C63	Longitudinal profile	Tabloid	1:250	18-Jul-2017	
C64	Sections SL-1/ SL-5	Tabloid	1:100	18-Jul-2017	
C65	Sections SL-6/SL-8	Tabloid	1:100	18-Jul-2017	
C66	Detail crossing 1	Tabloid	1:100	18-Jul-2017	

5. Drainage works

Table 4E: Drawing list of drainage works

Proj.nr.: **Client:**	**216-1380** **VROMI**	**Project:** **Division:**	**Link 6** **Side roads**	**Date:**	**18/07/2017**
	CONCEPT DESIGN	**Brimstone Hill Road**			
drawing nr.	**Subject**	**Format**	**Scale**	**Date**	**Rev.**
C70	Drainage plan	Tabloid	1:2000/25	18-Jul-2017	
C71	Drainage detail/ reinforcement	Tabloid	1:25	18-Jul-2017	
C72	Layout plan U-Gutter 20	Tabloid	1:500	18-Jul-2017	
C73	Longitudinal profile U-Gutter 20	Tabloid	1:750	18-Jul-2017	
C74	Layout plan U-Gutter 21	Tabloid	1:200	18-Jul-2017	
C75	Longitudinal profile U-Gutter 21	Tabloid	1:200	18-Jul-2017	
C76	Layout plan U-Gutter 22	Tabloid	1:200	18-Jul-2017	
C77	Longitudinal profile U-Gutter 22	Tabloid	1:250	18-Jul-2017	
C78	Detail culvert 1	Tabloid	1:50/25	18-Jul-2017	
C79	Detail culvert 2	Tabloid	1:50/25	18-Jul-2017	
C80	Detail culvert 3	Tabloid	1:50/25	18-Jul-2017	
C81	Detail culvert 4	Tabloid	1:50/25	18-Jul-2017	
C82	Layout plan traffic signs	Tabloid	1:2000	18-Jul-2017	
C90	Detention pond	A0	1:200/50/20	18-Jul-2017	

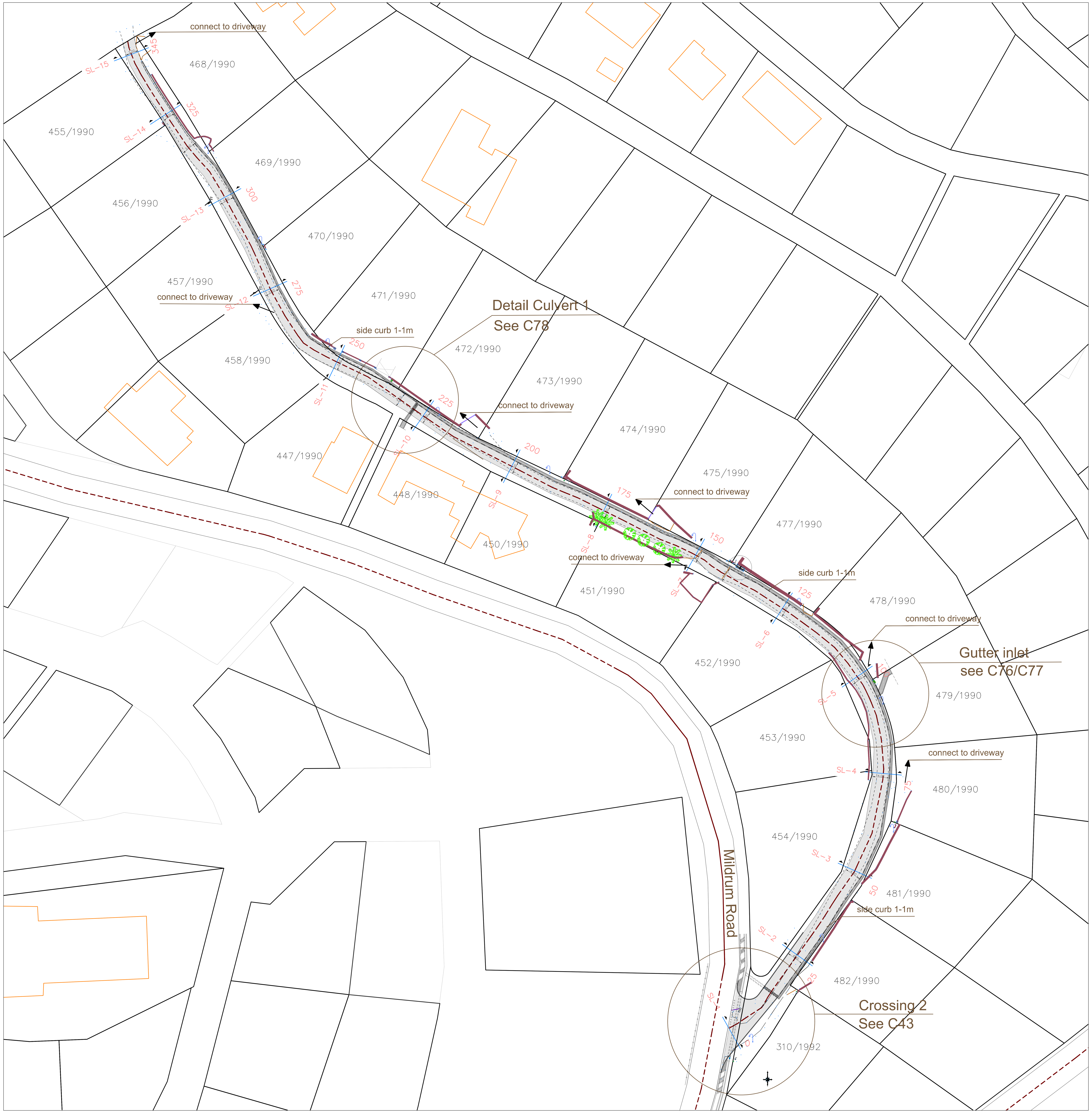


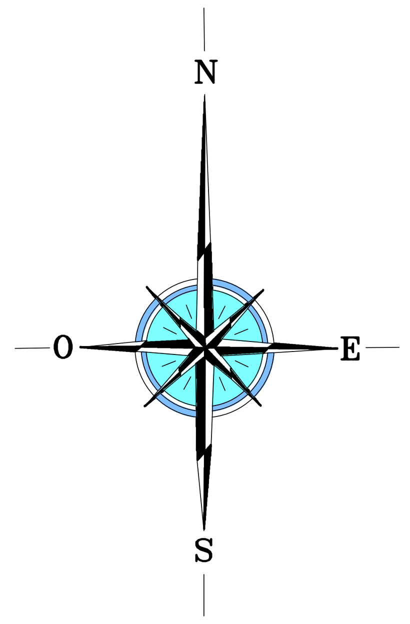
Survey layout
scale 1:500

0 15 30 Meters

Remarks:
- Existing features are based on survey drawings by Hunt's Topo Land.
- Dimensions in m, unless specified otherwise.
- Rebar in inches, rebar distance in mm.

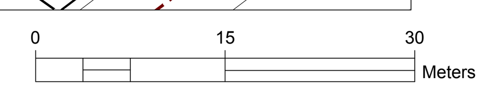
Project : Design and preparation for the construction of the Eastern part of link 6 [Side Roads]			Project nr : 216–1380
Principal : Ministry of VROMI	Date drawn : 18–07–2017	Revision :	
Subject : Survey layout Quill road	Drawn by : B.Z		
	Checked by : G.T		
	Format : A1		
	Scale : 1:500		
INDEPENDENT CONSULTING ENGINEERS. N.V.			Sheet number : C–35
p.o. box 390, zaegersgut road 13, philipsburg, st. maarten, n.a. phone 5422421, fax 5422597, e-mail icesxm@sintmaarten.net			File number : 216–1380–C35





Layout road plan new situation

scale 1:500



Remarks:
- Existing features are based on survey drawings by Hunt's Topo Land.
- Dimensions in m, unless specified otherwise.
- Rebar in inches, rebar distance in mm.

- For longitudinal section see C38
- For cross sections see C40 - C42

CONCEPT DESIGN

LEGEND

Road	
Gutter	
Curb	
Alignment	

Project : Design and preparation for the construction of the Eastern part of link 6 [Side Roads]		Project nr : 216–1380
Principal : Ministry of VROMI	Date drawn : 18–07–2017	Revision :
Subject : Layout road plan Quil Road New situation	Drawn by : BZ Checked by : GT Format : A1 Scale : 1:500	
INDEPENDENT CONSULTING ENGINEERS. N.V. p.o. box 390, zaegersgut road 13, philipsburg, st. maarten, n.a. phone 5422421, fax 5422597, e-mail icesxm@sintmaarten.net		Sheet number : C36 File number : 216–1380–C36

Typical sections layout

scale 1:2000

Remarks:

- Existing features are based on survey drawings by Hunt's Topo Land.
- Dimensions in m, unless specified otherwise.
- Rebar in inches, rebar distance in mm.

- For typical section see C38A & C38B

CONCEPT DESIGN

Project : Design and preparation for the construction of the Eastern part of link 6 [Side Roads]		Project nr : 216–1380
Principal : Ministry of VROMI	Date drawn : 18–07–2017	Revision :
Subject : Typical sections layout Quil Road	Drawn by : BZ	
	Checked by : GT	
	Format : Tabloid	
	Scale : 1:2000	
INDEPENDENT CONSULTING ENGINEERS. N.V.		Sheet number : C37
		File number : 216–1380–C37

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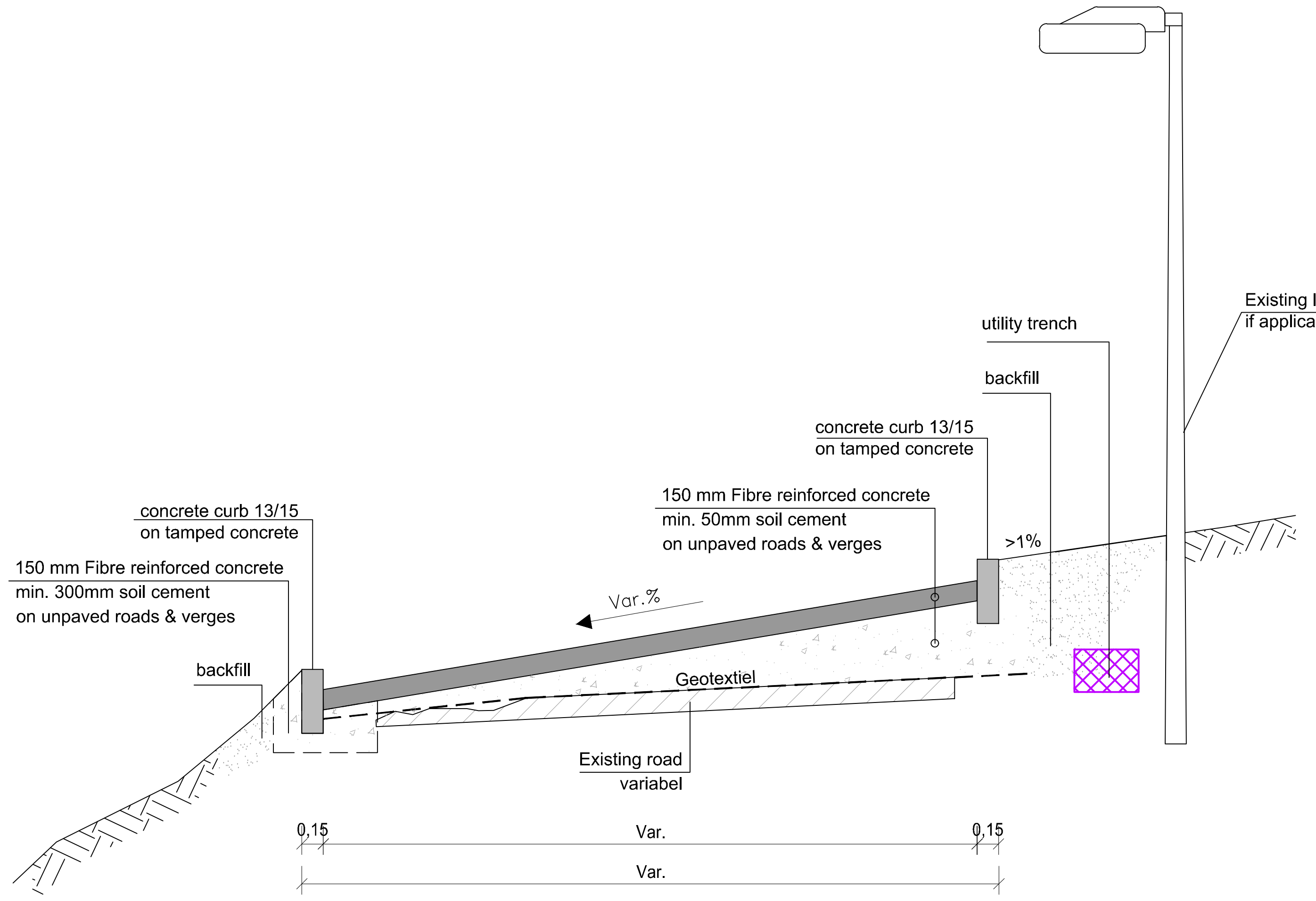
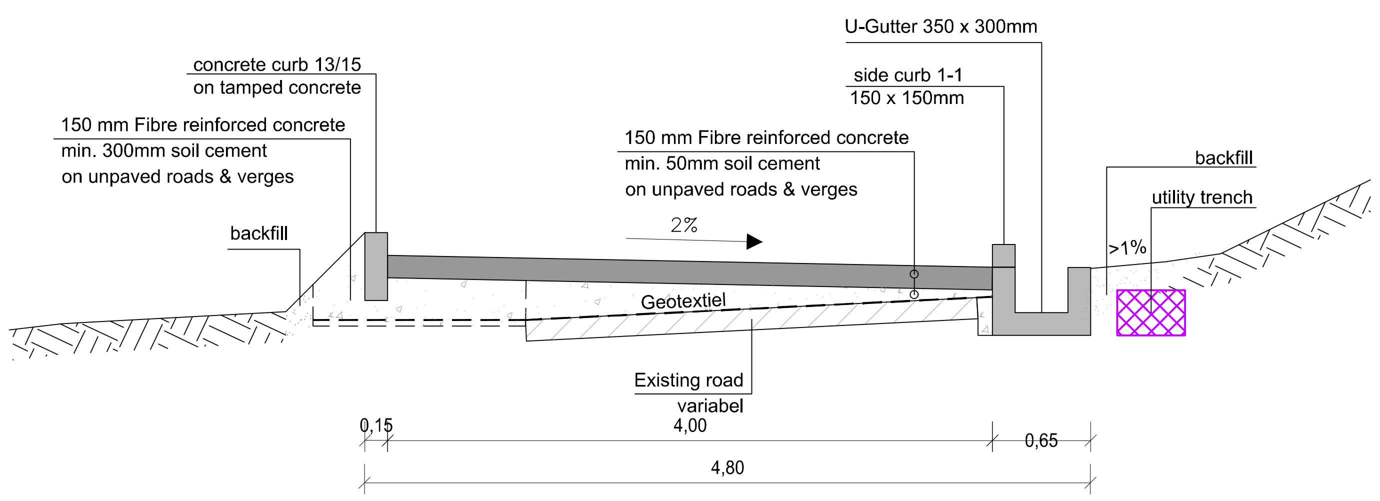
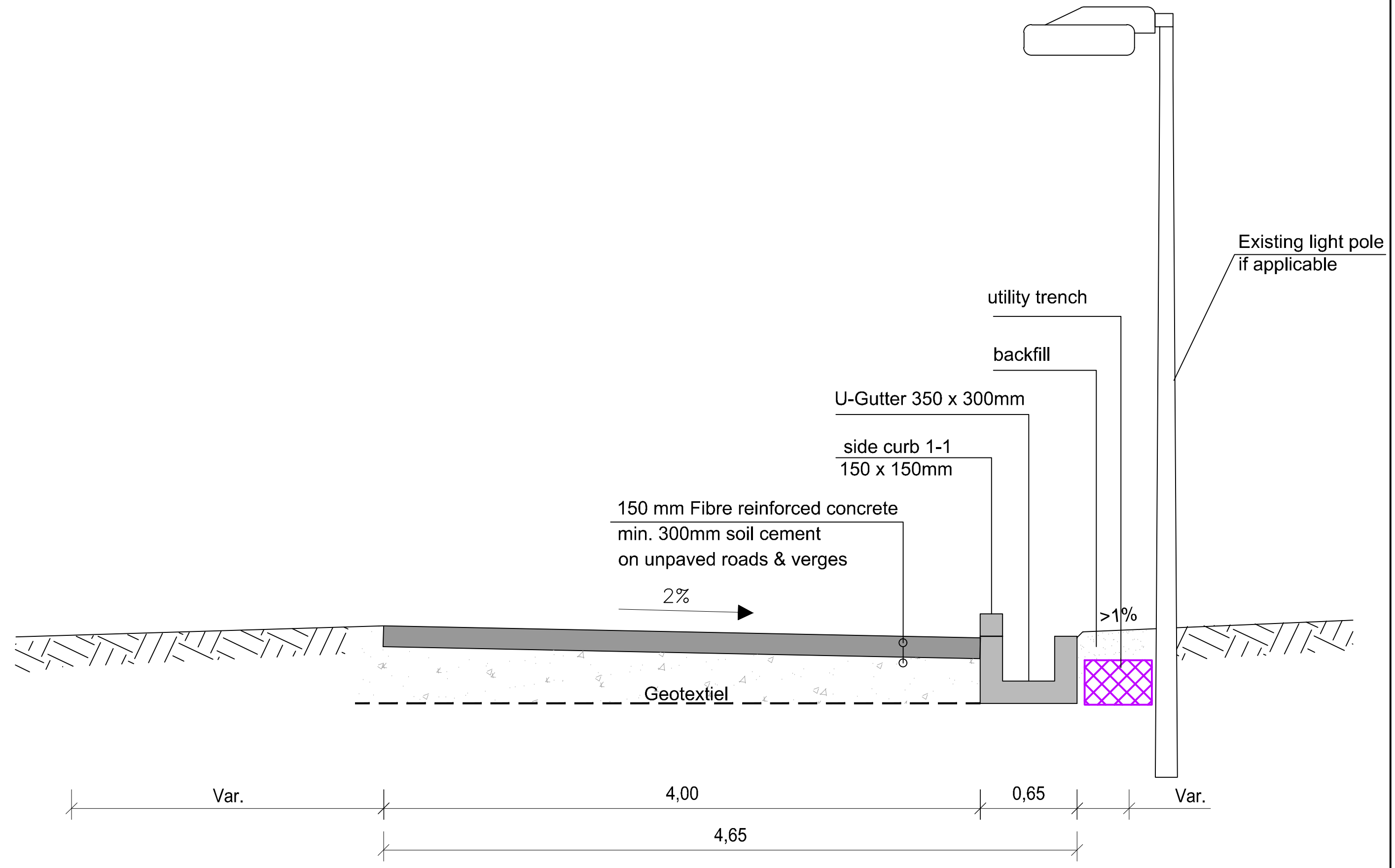


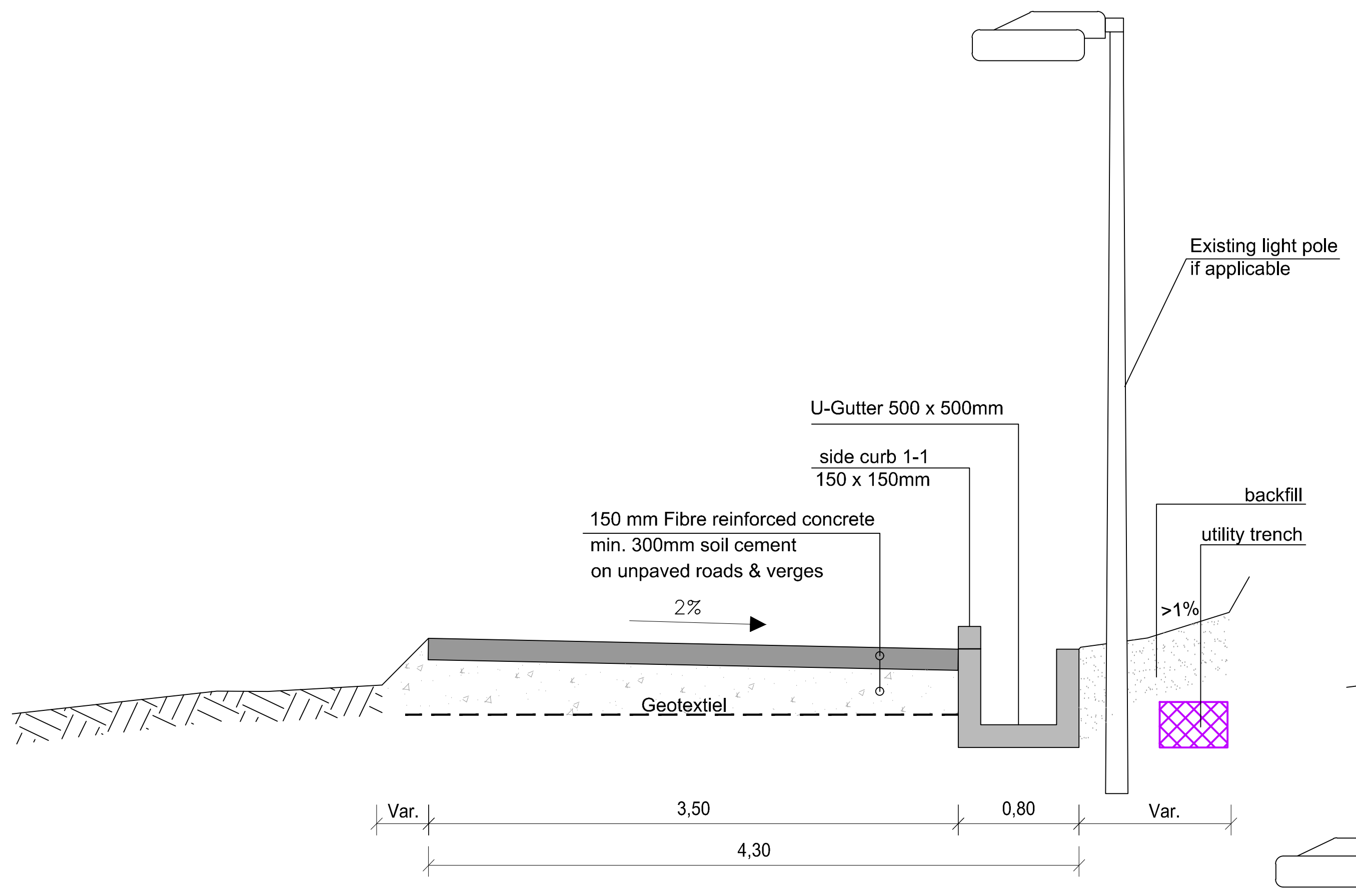
A1 typical cross section road (0-14m)
scale 1:50



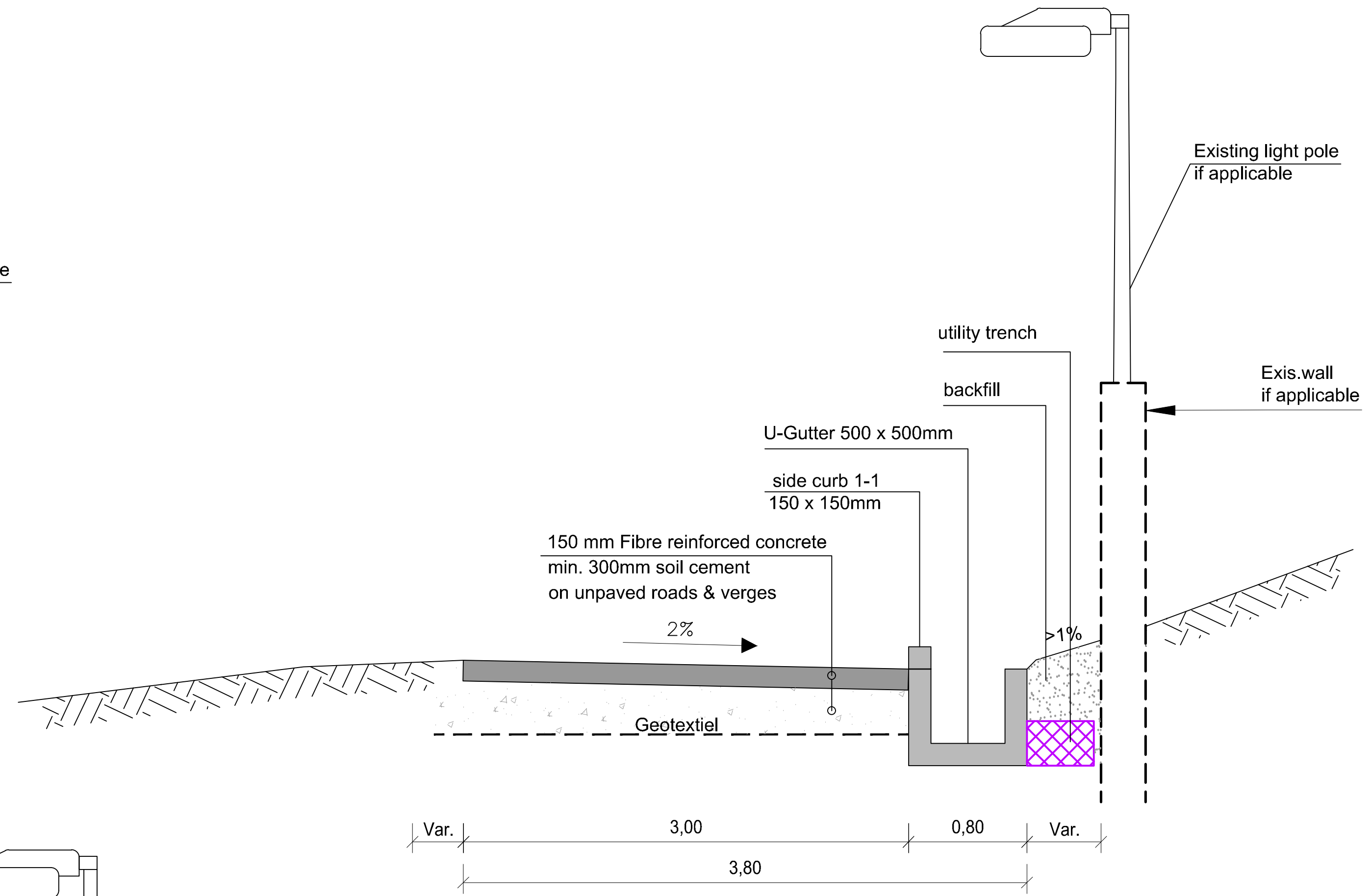
A2 typical cross section road (14-31m)
scale 1:50



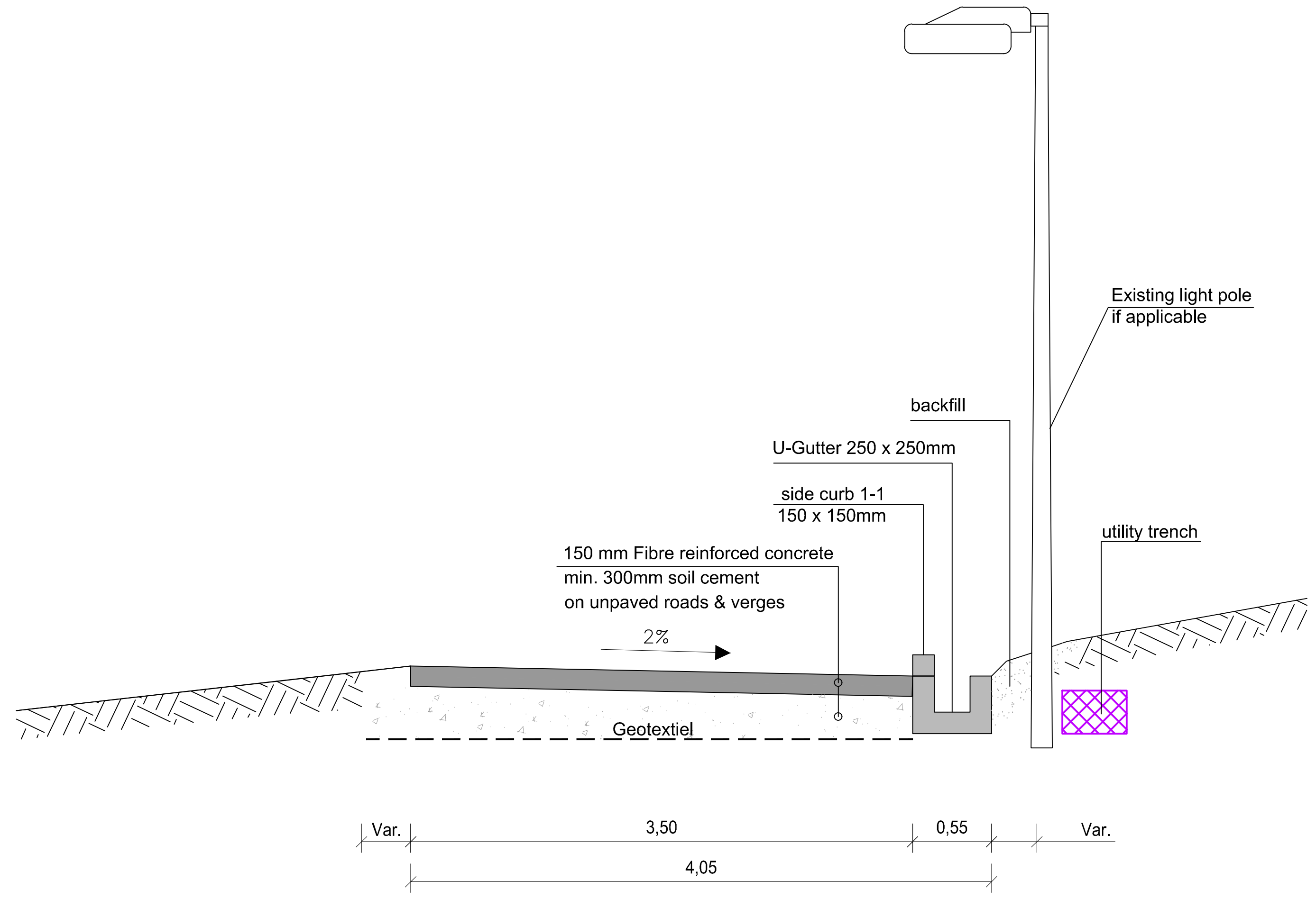
A3 typical cross section road (31-95m)
scale 1:50



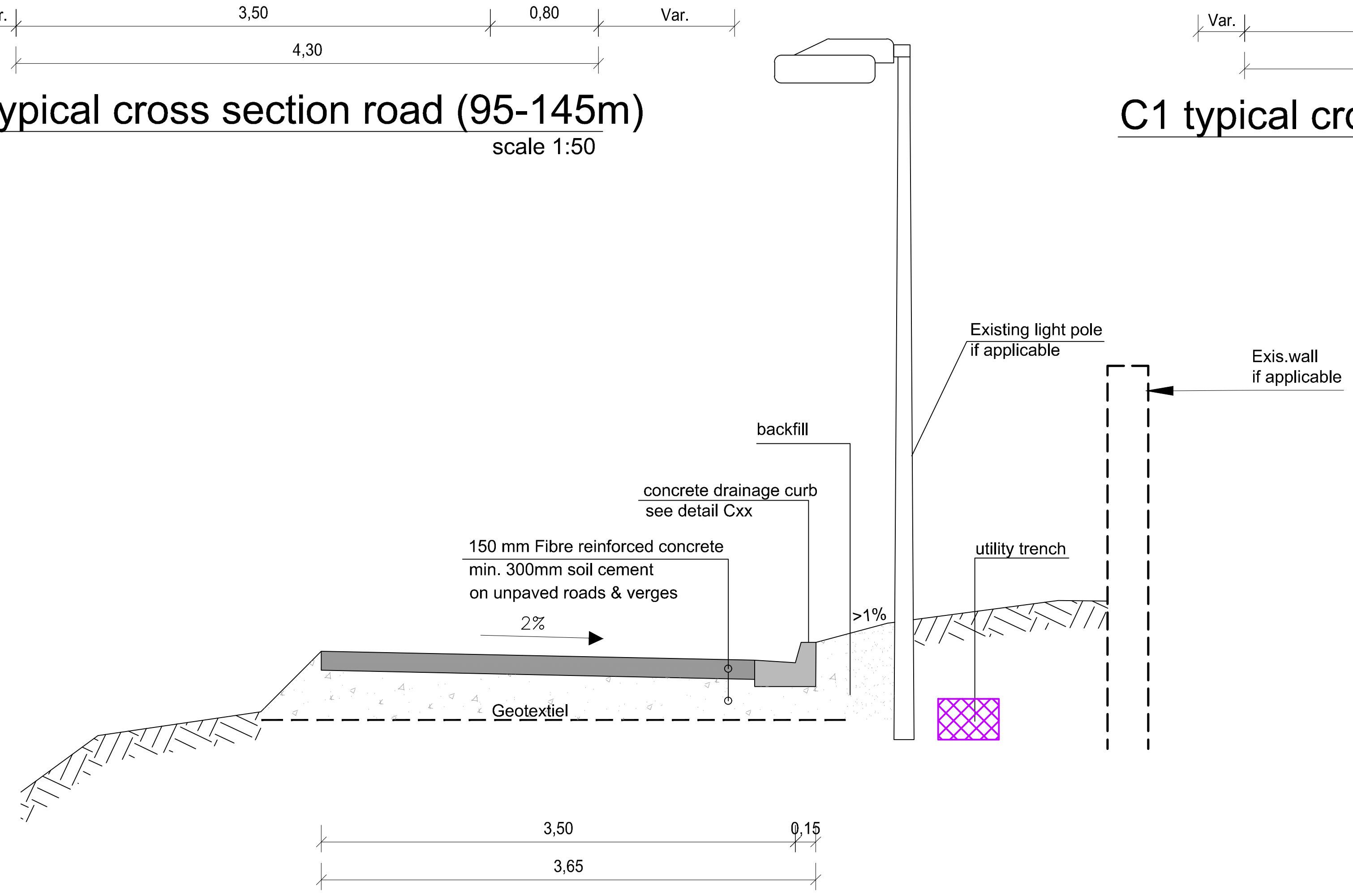
B1 typical cross section road (95-145m)
scale 1:50



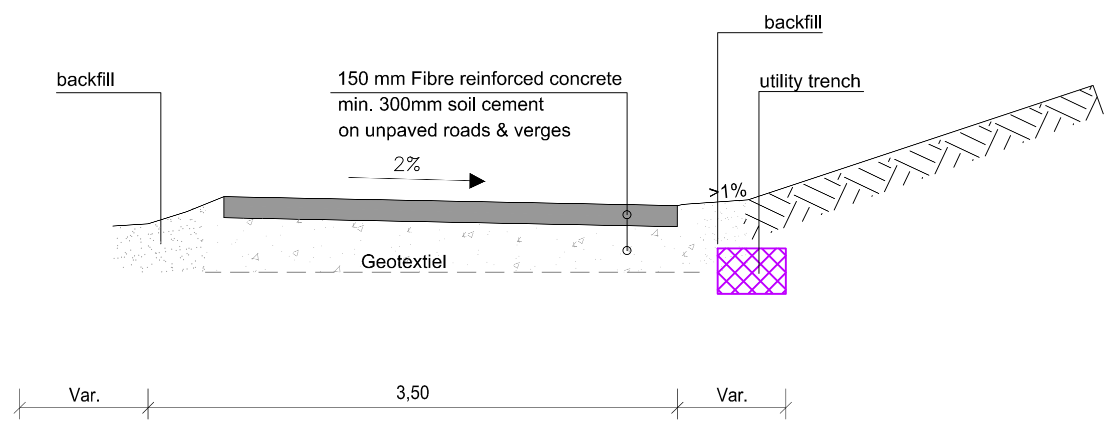
C1 typical cross section road (145-230m)
scale 1:50



D1 typical cross section road (230-315m)
scale 1:50



D2 typical cross section road (315-340m)
scale 1:50



D2 typical cross section road (340-346m)
scale 1:50

Remarks:
- Existing features are based on survey drawings by Hunt's Topo Land.
- Dimensions in m, unless specified otherwise.
- Rebar in inches, rebar distance in mm.
- For typical cross sections location see C37

CONCEPT DESIGN

Project : Design and preparation for the construction of the Eastern part of link 6 [Side Roads]		Project nr : 216-1380
Principal : Ministry of VROMI	Date drawn : 18-07-2017	Revision :
Subject : Typical cross sections Quil Road Alternative A	Drawn by : BZ	
	Checked by : GT	
	Format : A2	
	Scale : 1:50	
		Sheet number : C38 A
		File number : 216-1380-C38A

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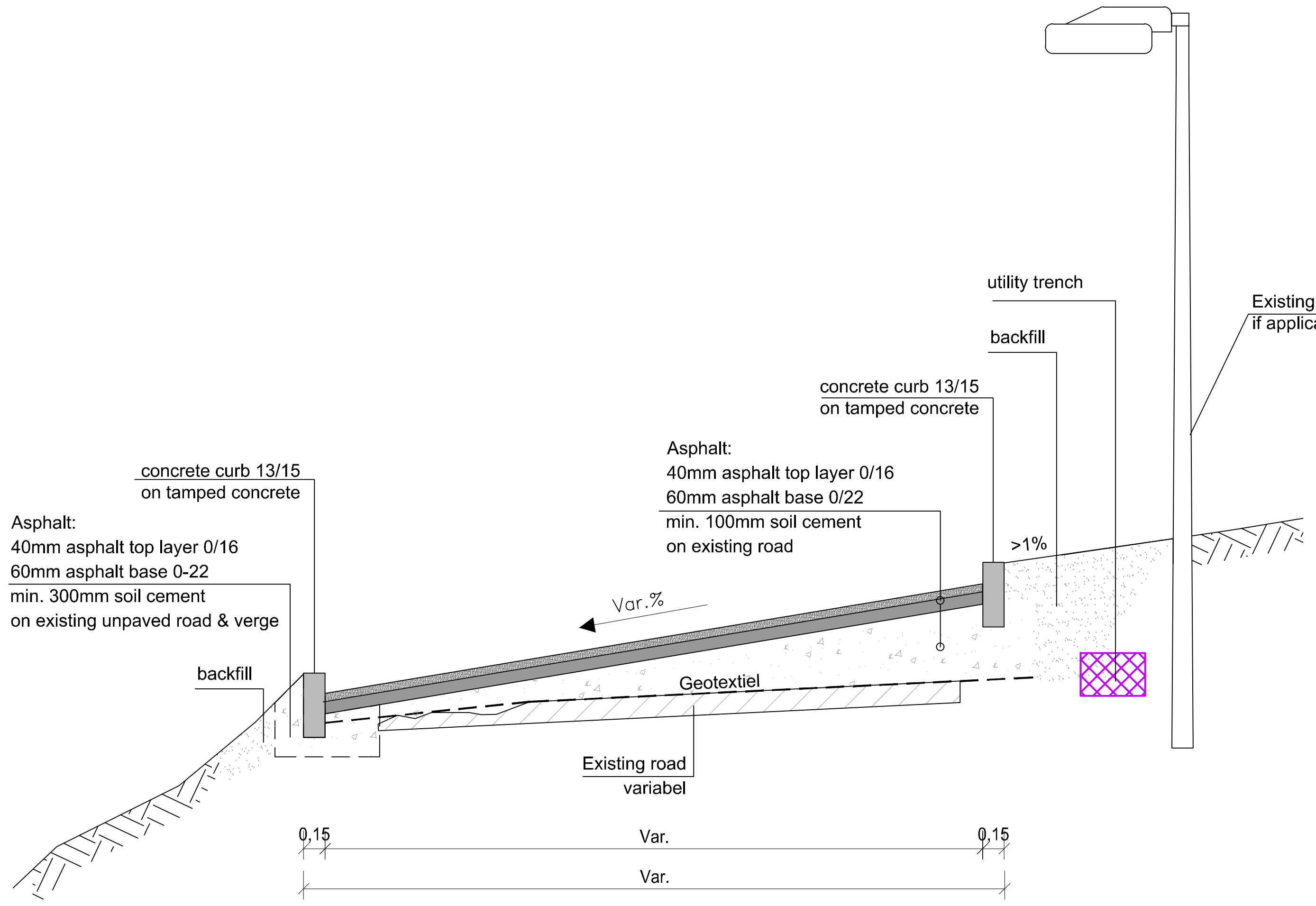
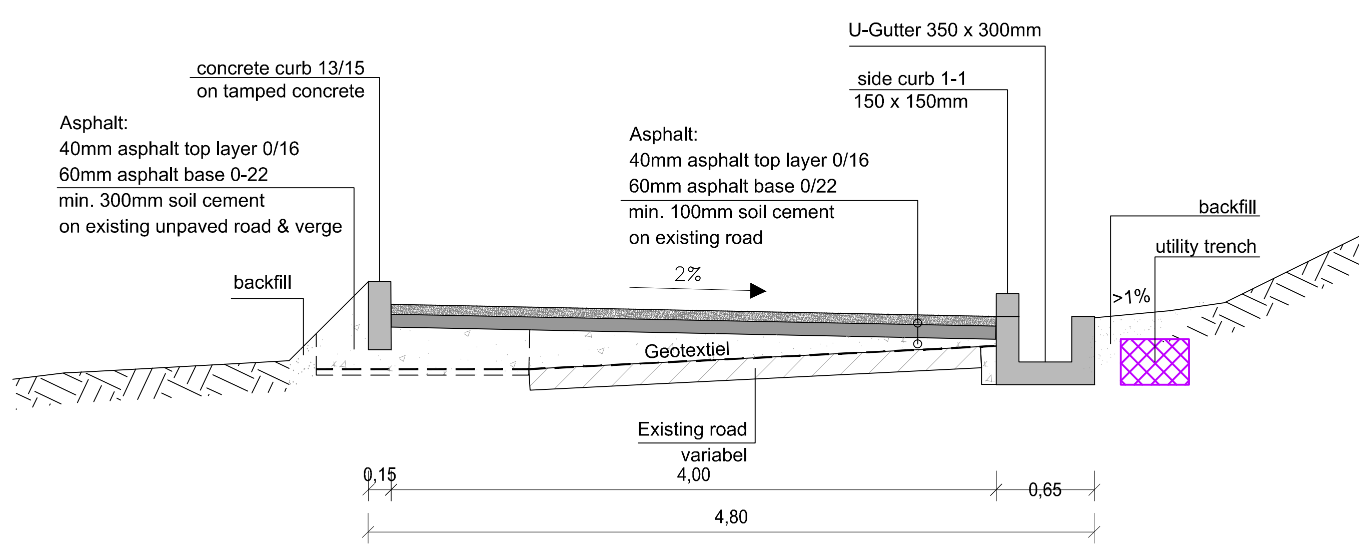
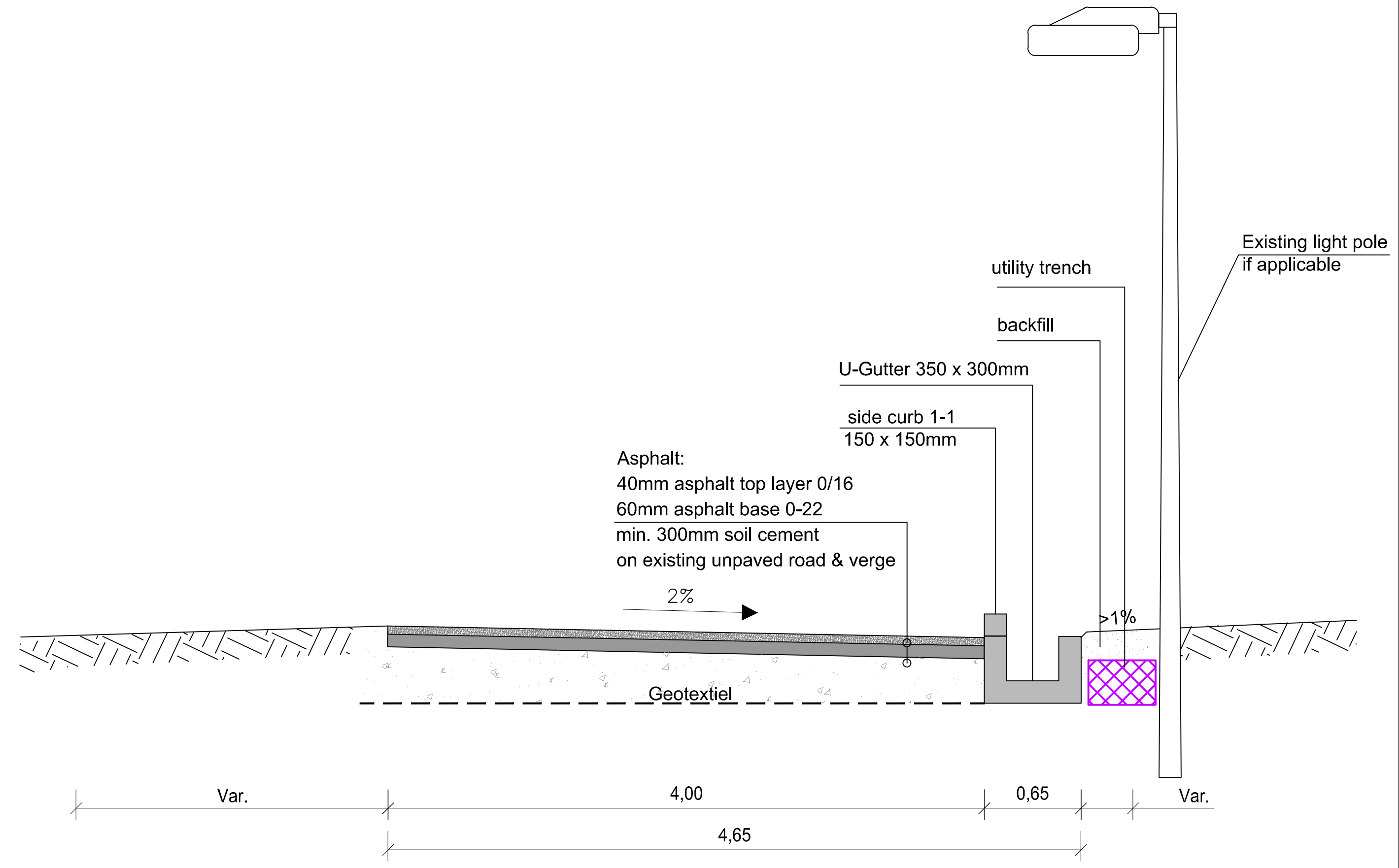


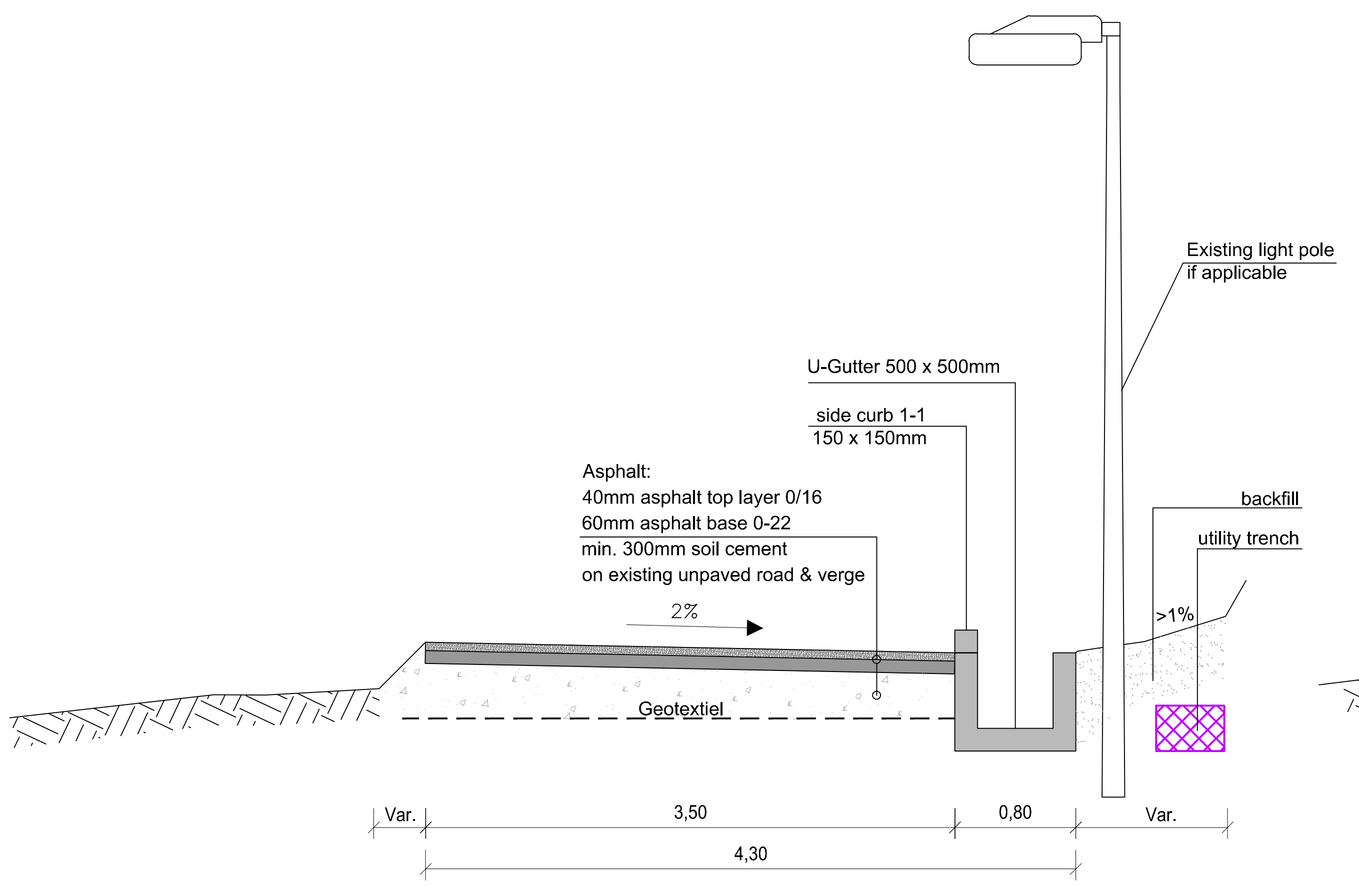
A1 typical cross section road (0-14m)
scale 1:50



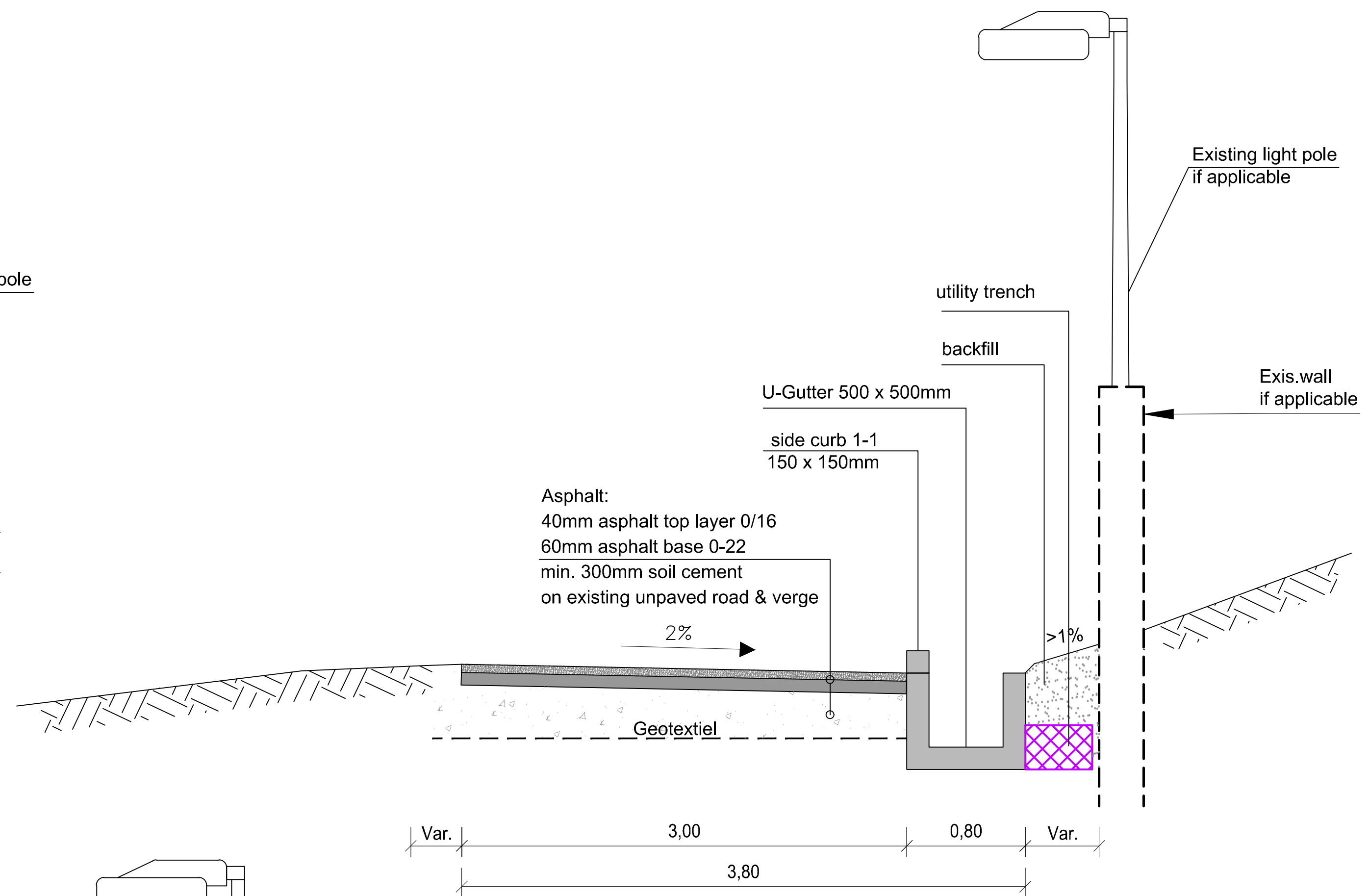
A2 typical cross section road (14-31m)
scale 1:50



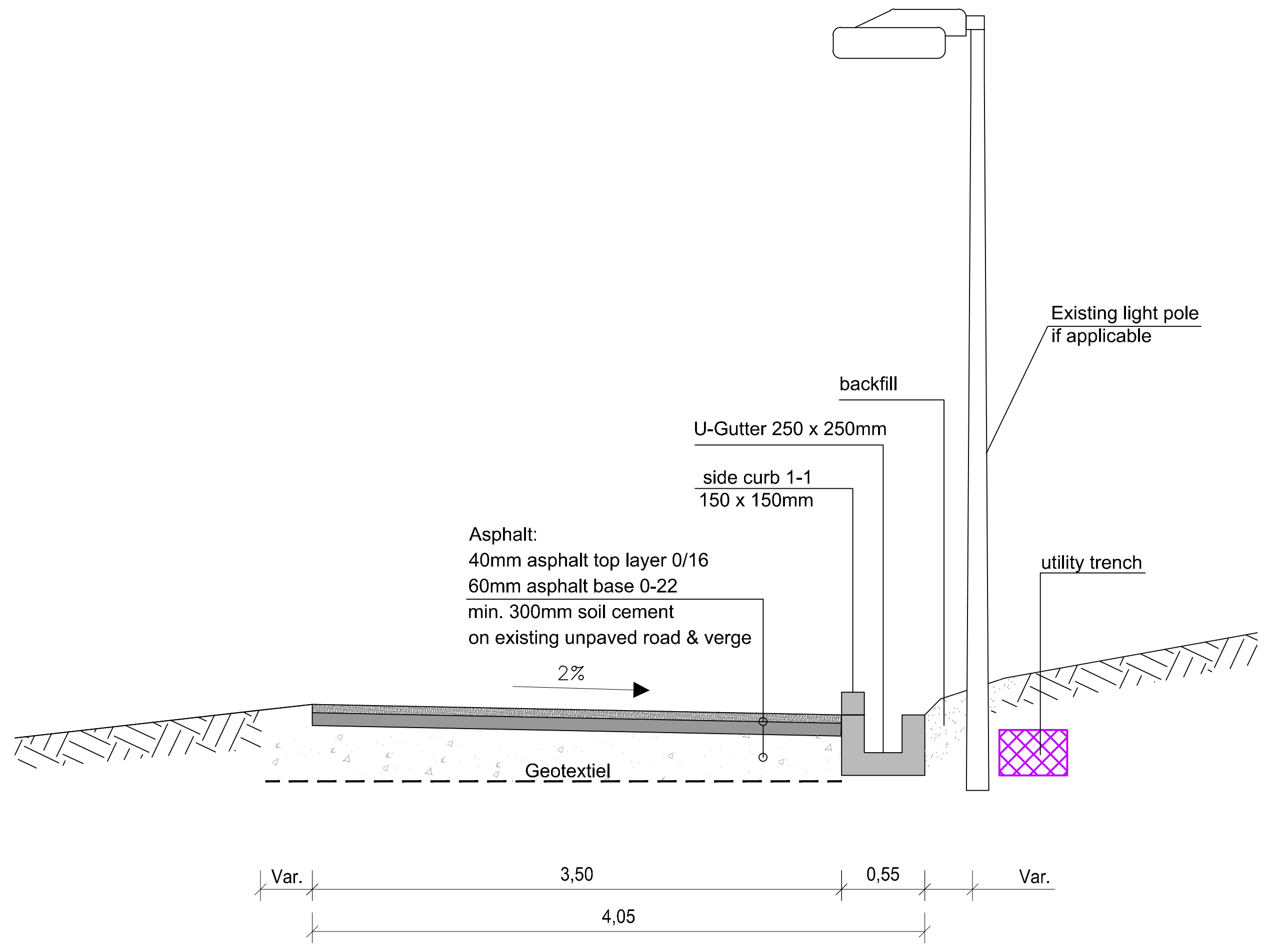
A3 typical cross section road (31-95m)
scale 1:50



B1 typical cross section road (95-145m)
scale 1:50

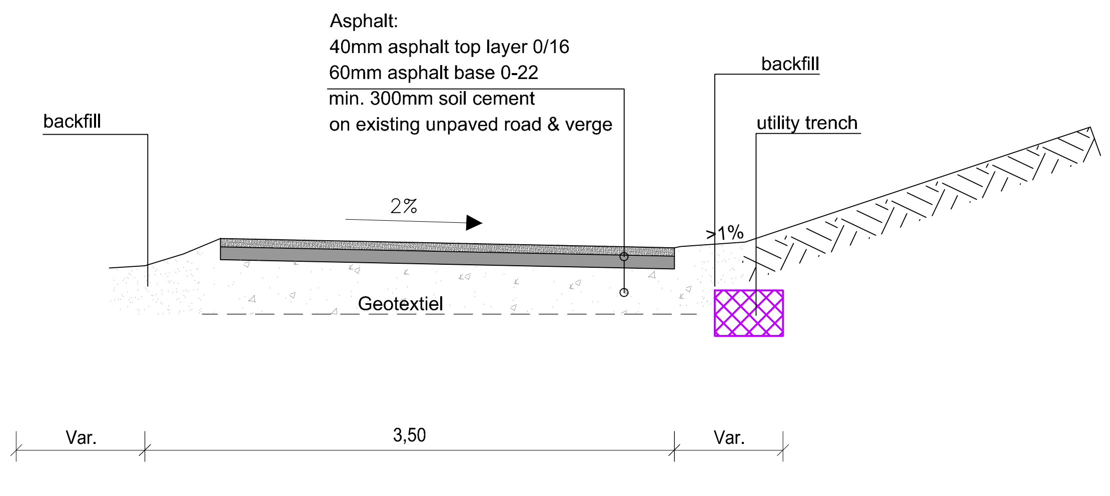


C1 typical cross section road (145-230m)
scale 1:50



D1 typical cross section road (230-315m)
scale 1:50

D2 typical cross section road (315-340m)
scale 1:50



D2 typical cross section road (340-346m)
scale 1:50

Remarks:
- Existing features are based on survey drawings by Hunt's Topo Land.
- Dimensions in m, unless specified otherwise.
- Rebar in inches, rebar distance in mm.
- For typical cross sections location see C37

CONCEPT DESIGN

Project : Design and preparation for the construction of the Eastern part of link 6 [Side Roads]		Project nr : 216–1380
Principal : Ministry of VROMI	Date drawn : 18-07-2017	Revision :
Subject : Typical cross sections Quil Road Alternative B	Drawn by : BZ	
	Checked by : GT	
	Format : A2	
	Scale : 1:50	
INDEPENDENT CONSULTING ENGINEERS. N.V.		Sheet number : C38 B
		File number : 216–1380–C38B

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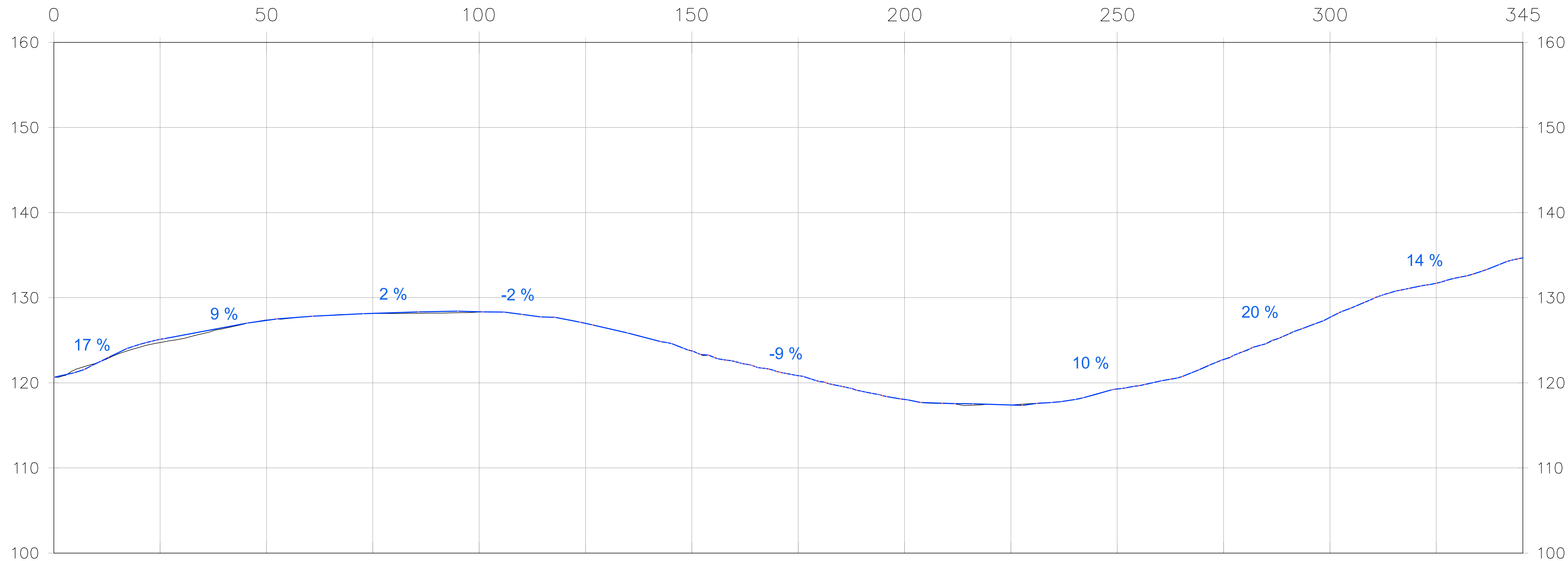

	Existing level	Design level
0+000.00	120.65	120.78
0+025.00	124.73	124.95
0+050.00	127.38	127.32
0+075.00	128.14	128.35
0+100.00	128.31	128.52
0+125.00	126.98	127.10
0+150.00	123.74	123.83
0+175.00	120.84	120.97
0+200.00	118.03	118.05
0+225.00	117.41	117.54
0+250.00	119.27	119.38
0+275.00	122.72	122.93
0+300.00	127.69	127.89
0+325.00	131.69	131.96
0+345.36		

Longitudinal profile road axis

CONCEPT DESIGN

Remarks:
- Existing level
- Design level

Remarks:
- Existing features are based on survey drawings by Hunt's Topo Land.
- Dimensions in m, unless specified otherwise.
- Rebar in inches, rebar distance in mm.

- For cross sections location see C36
- For cross sections see C40 -C42

Project :	Design and preparation for the construction of the Eastern part of link 6 [Side Roads]		Project nr : 216−1380
Principal :	Ministry of VROMI	Date drawn : 18−07−2017	Revision :
Subject :	Quil Road Longitudinal section	Drawn by : BZ	
		Checked by : GT	
		Format : Tabloid	
		Scale : 1:1000	
			Sheet number : C39
			File number : 216−1380−C39

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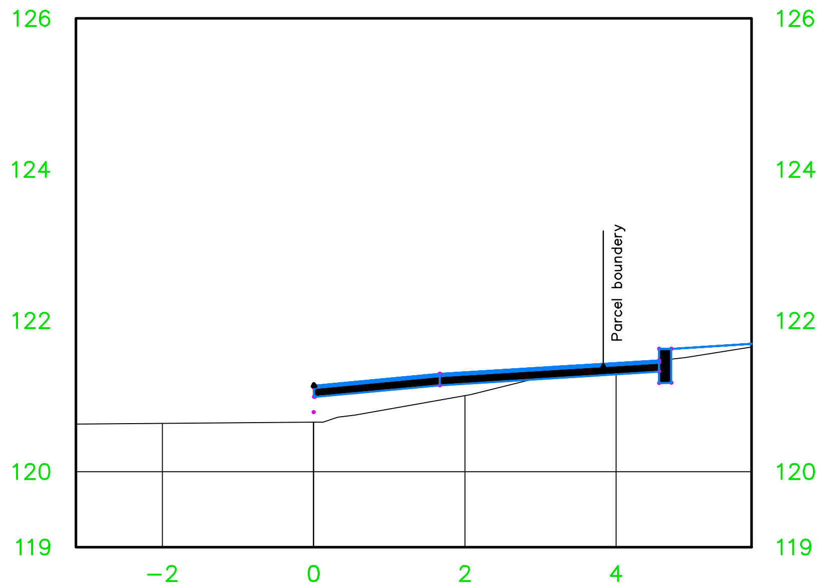
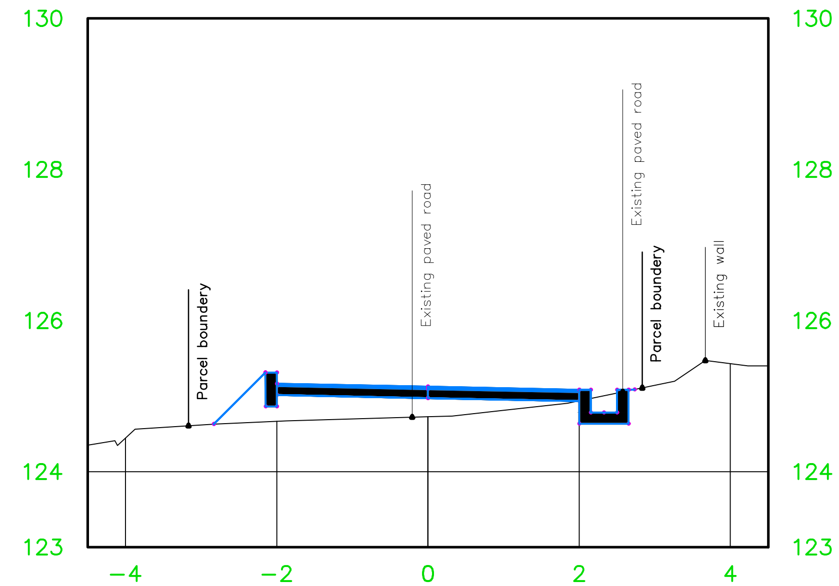
SL-1 (0.00m>



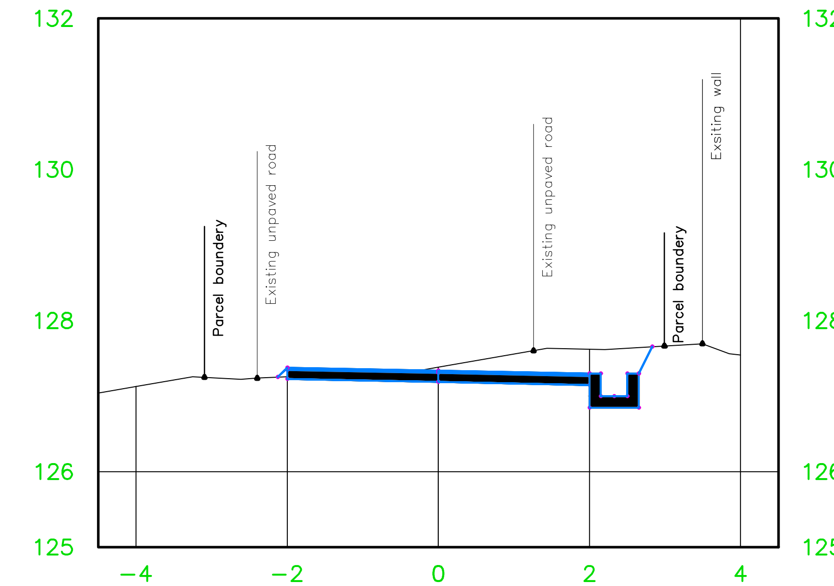
	-2	0	2	4
Existing level	120.64	120.65	121.00	121.40
Design level		121.14	121.32	121.48

SL-2 (25.00m>



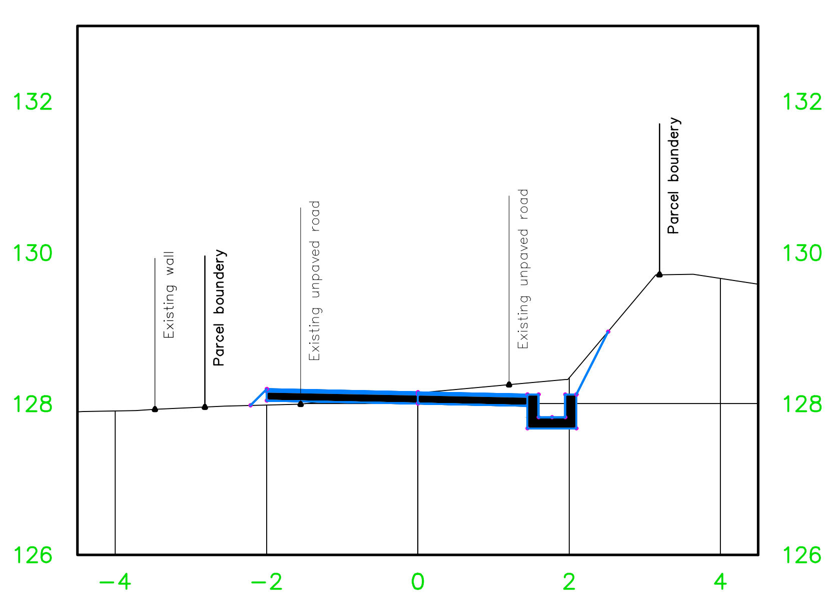
	-4	-2	0	2	4
Existing level	124.44	124.66	124.73	124.94	125.43
Design level		125.16	125.12	125.08	125.06

SL-3 (50.00m>



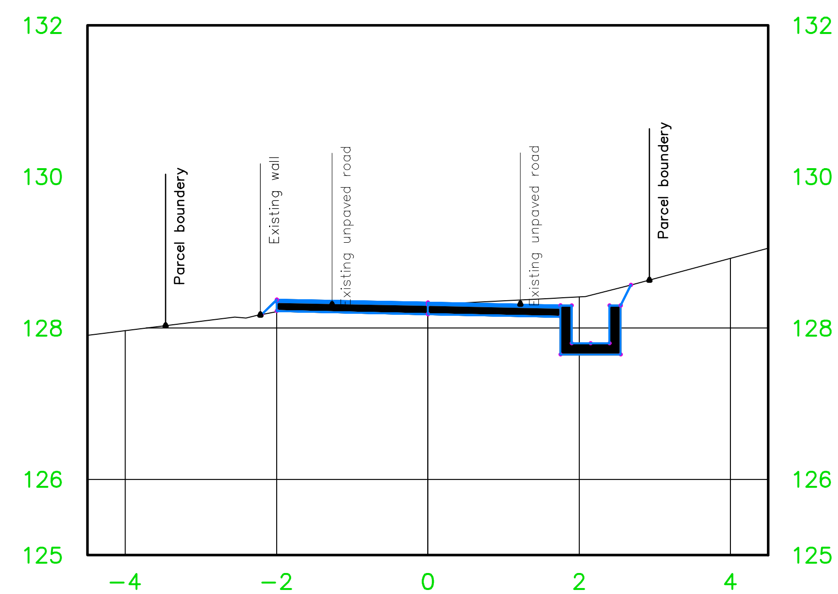
	-4	-2	0	2
Existing level	127.13	127.25	127.38	127.62
Design level		127.38	127.34	127.30

SL-4 (75.00m>



	-4	-2	0	2	4
Existing level	127.90	127.98	128.14	128.34	129.66
Design level		128.19	128.15	128.12	

SL-5 (100.00m>



	-4	-2	0	2	4
Existing level	127.97	128.22	128.31	128.41	128.92
Design level	127.97	128.37	128.33	127.80	

Remarks:
- Existing features are based on survey drawings by Hunt's Topo Land.
- Dimensions in m, unless specified otherwise.
- Rebar in inches, rebar distance in mm.

- For cross section location see C36
- For longitudinal section see C39

CONCEPT DESIGN

Project : Design and preparation for the construction of the Eastern part of link 6 [Side Roads]

Project nr : 216-1380

Principal : Ministry of VROMI

Date drawn : 18-07-2017

Revision :

Subject : Quil Road
Sections SL-1 / SL-5

Drawn by : BZ

Checked by : GT

Format : Tabloid

Scale : 1:100

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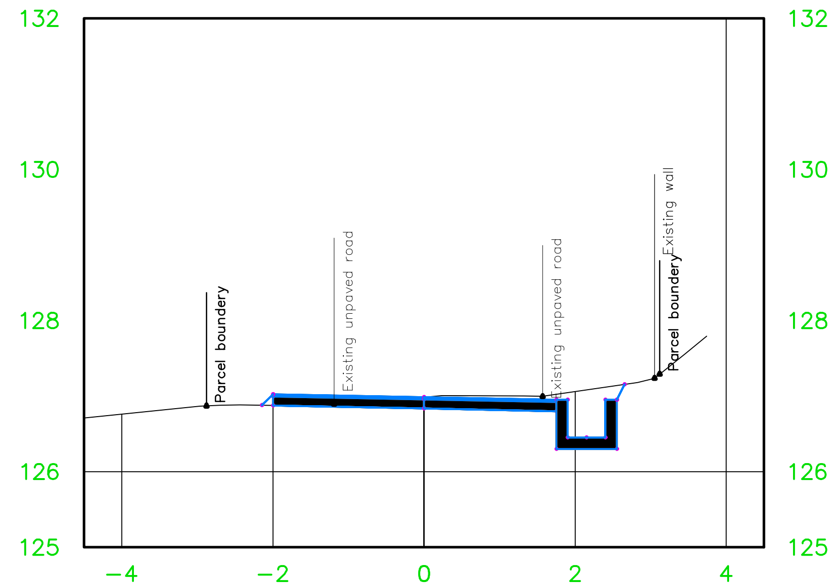
Sheet number : C40

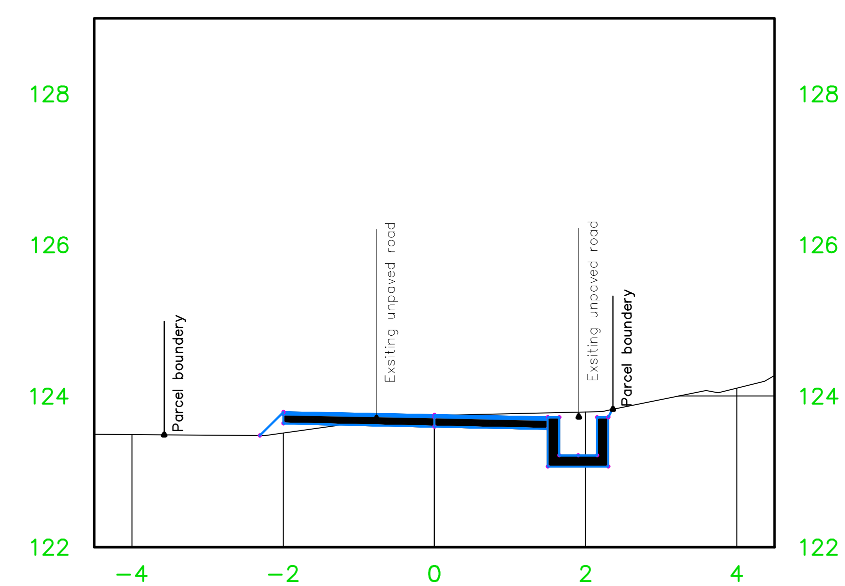
File number : 216-1380-C40

SL–6 (125.00m>



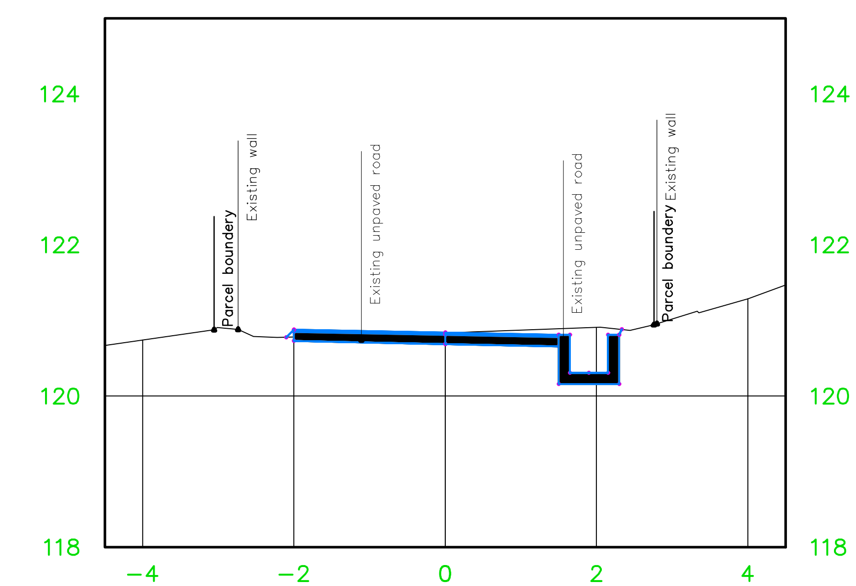
	–4	–2	0	2
Existing level	126.76	126.88	126.98	127.06
Design level	126.65	127.02	126.98	126.45

SL–7 (150.00m>



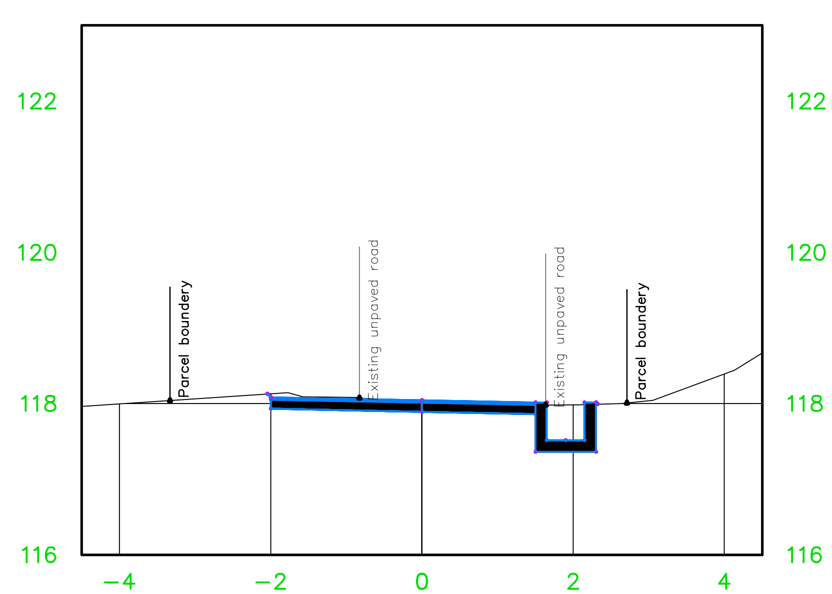
	–4	–2	0	2	4
Existing level	123.49	123.51	123.74	123.79	124.10
Design level	123.17	123.79	123.75	123.22	

SL–8 (175.00m>



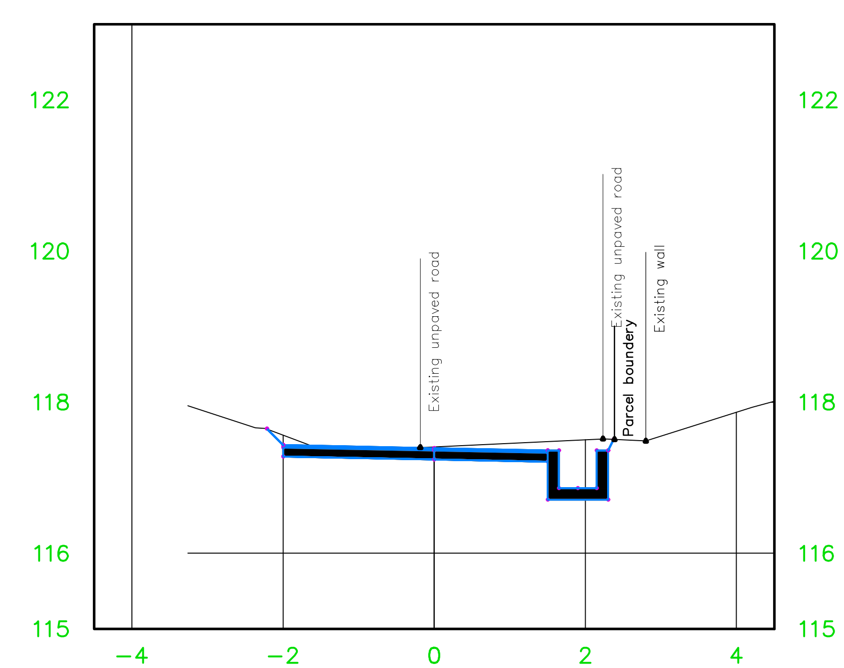
	–4	–2	0	2	4
Existing level	120.74	120.78	120.84	120.91	121.29
Design level		120.88	120.84	120.31	

SL–9 (200.00m>



	–4	–2	0	2	4
Existing level	117.99	118.13	118.03	117.98	118.39
Design level		118.08	118.04	117.51	

SL–10 (225.00m>



	–4	–2	0	2	4
Existing level		117.56	117.41	117.50	117.87
Design level		117.43	117.39	116.86	

Remarks:
- Existing features are based on survey drawings by Hunt's Topo Land.
- Dimensions in m, unless specified otherwise.
- Rebar in inches, rebar distance in mm.

- For cross section location see C36
- For longitudinal section see C39

CONCEPT DESIGN

Project : Design and preparation for the construction of the Eastern part of link 6 [Side Roads]

Project nr : 216–1380

Principal : Ministry of VROMI

Subject : Quil Road Sections Sl–6 / SL–10

Date drawn	18–07–2017
Drawn by	BZ
Checked by	GT
Format	Tabloid
Scale	1:100

Revision :

Sheet number : C41

File number : 216–1380–C41

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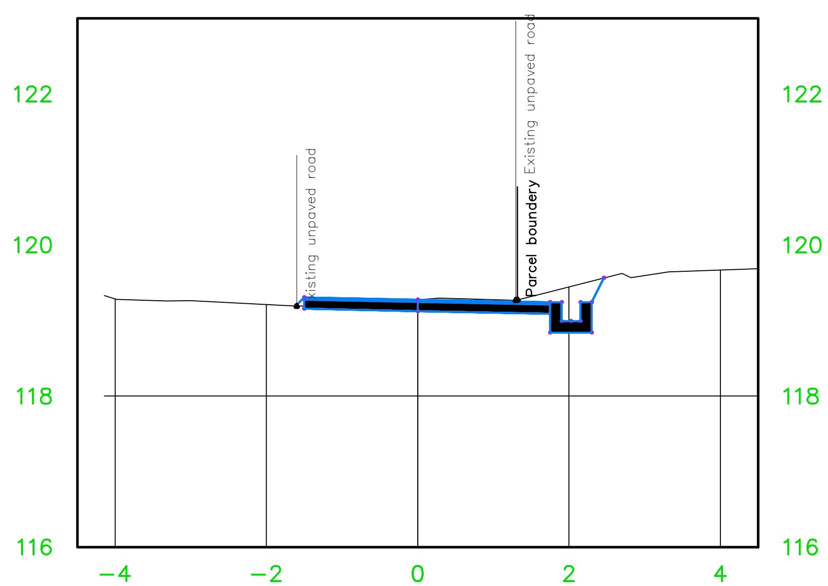
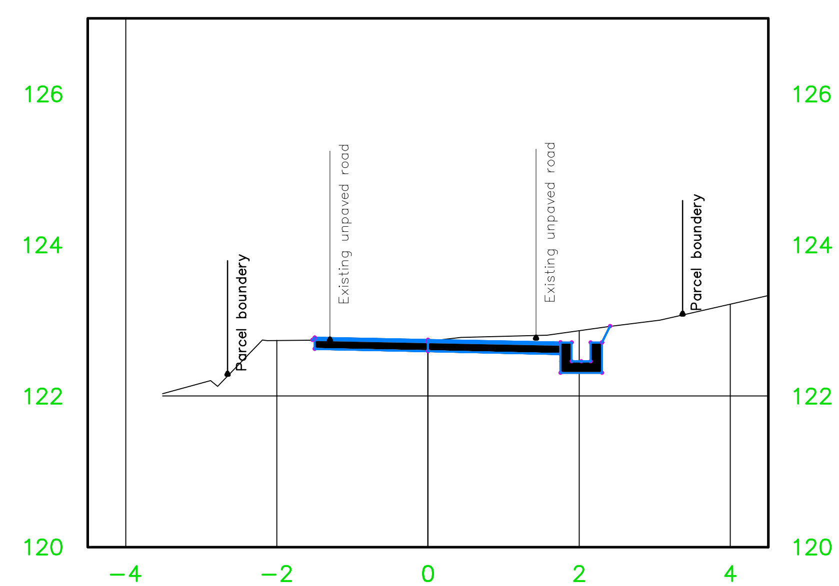
SL–11 (250.00m>



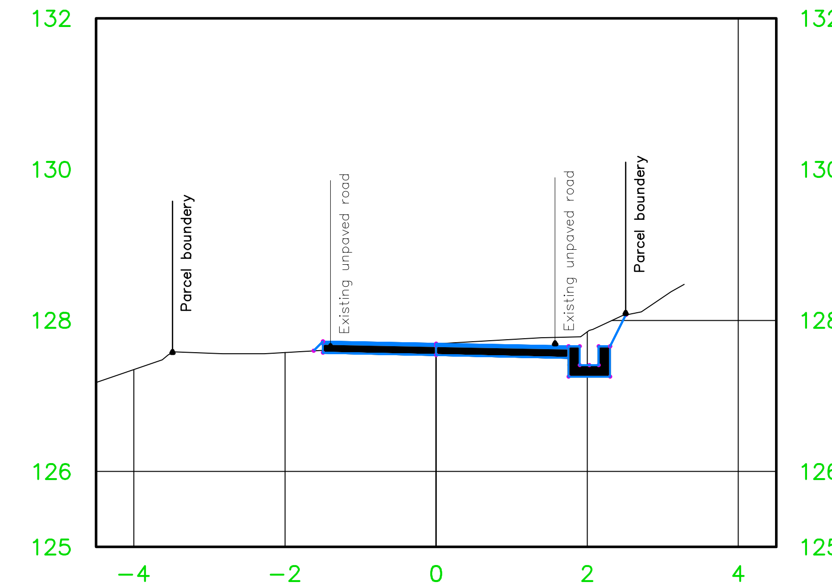
Existing level	119.28	119.21	119.27	119.45	119.67
Design level			119.27	118.99	119.65

SL–12 (275.00m>



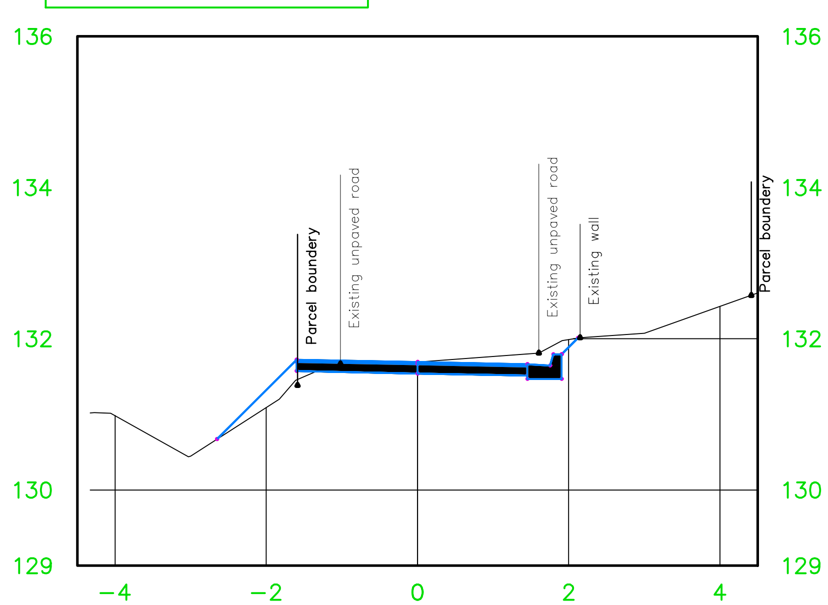
Existing level		122.74	122.72	122.86	123.21
Design level		122.40	122.74	122.46	123.14

SL–13 (300.00m>



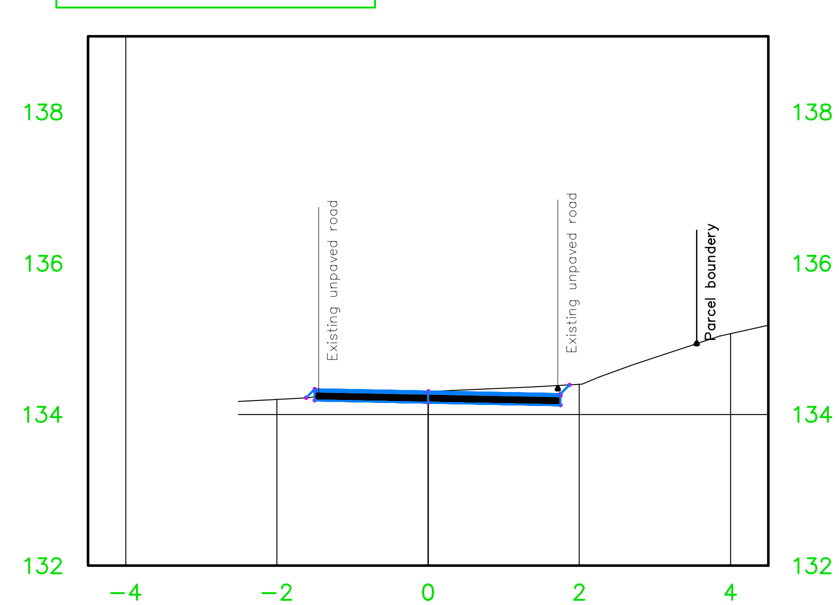
Existing level	127.35	127.57	127.69	127.85	
Design level	126.73	127.65	127.69	127.41	

SL–14 (325.00m>



Existing level	130.98	131.09	131.69	131.99	132.43
Design level		131.32	131.69	131.89	

SL–15 (341.98m>



Existing level		134.20	134.31	134.40	135.07
Design level			134.31	134.41	

Remarks:
- Existing features are based on survey drawings by Hunt's Topo Land.
- Dimensions in m, unless specified otherwise.
- Rebar in inches, rebar distance in mm.

- For cross section location see C36
- For longitudinal section see C39

CONCEPT DESIGN

Project : Design and preparation for the construction of the Eastern part of link 6		Project nr : 216–1380
Principal : Ministry of VROMI	Date drawn : 18–07–2017	Revision :
Subject : Quil Road Sections SL–11 / SL–15	Drawn by : BZ	
	Checked by : GT	
	Format : Tabloid	
	Scale : 1:100	
INDEPENDENT CONSULTING ENGINEERS. N.V.		Sheet number : C42
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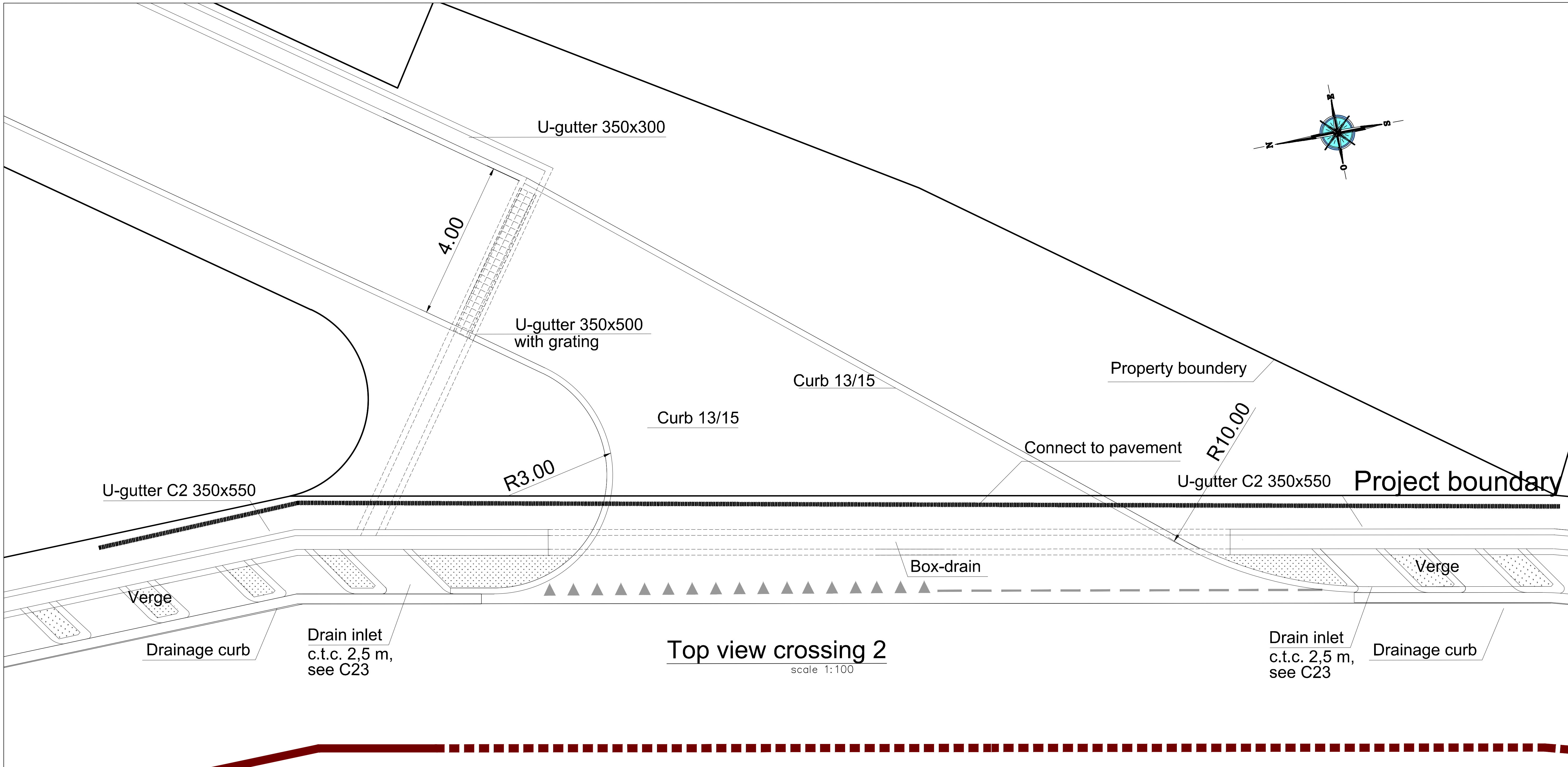
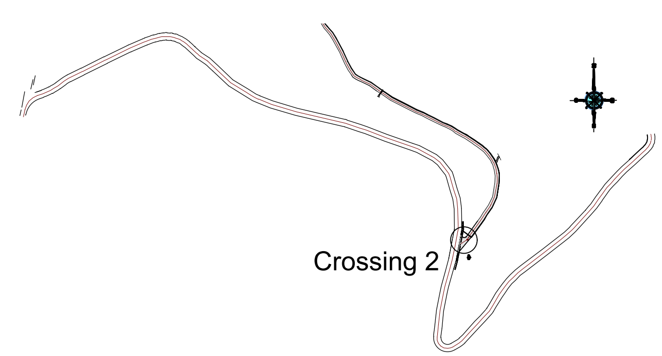


Top view crossing 2

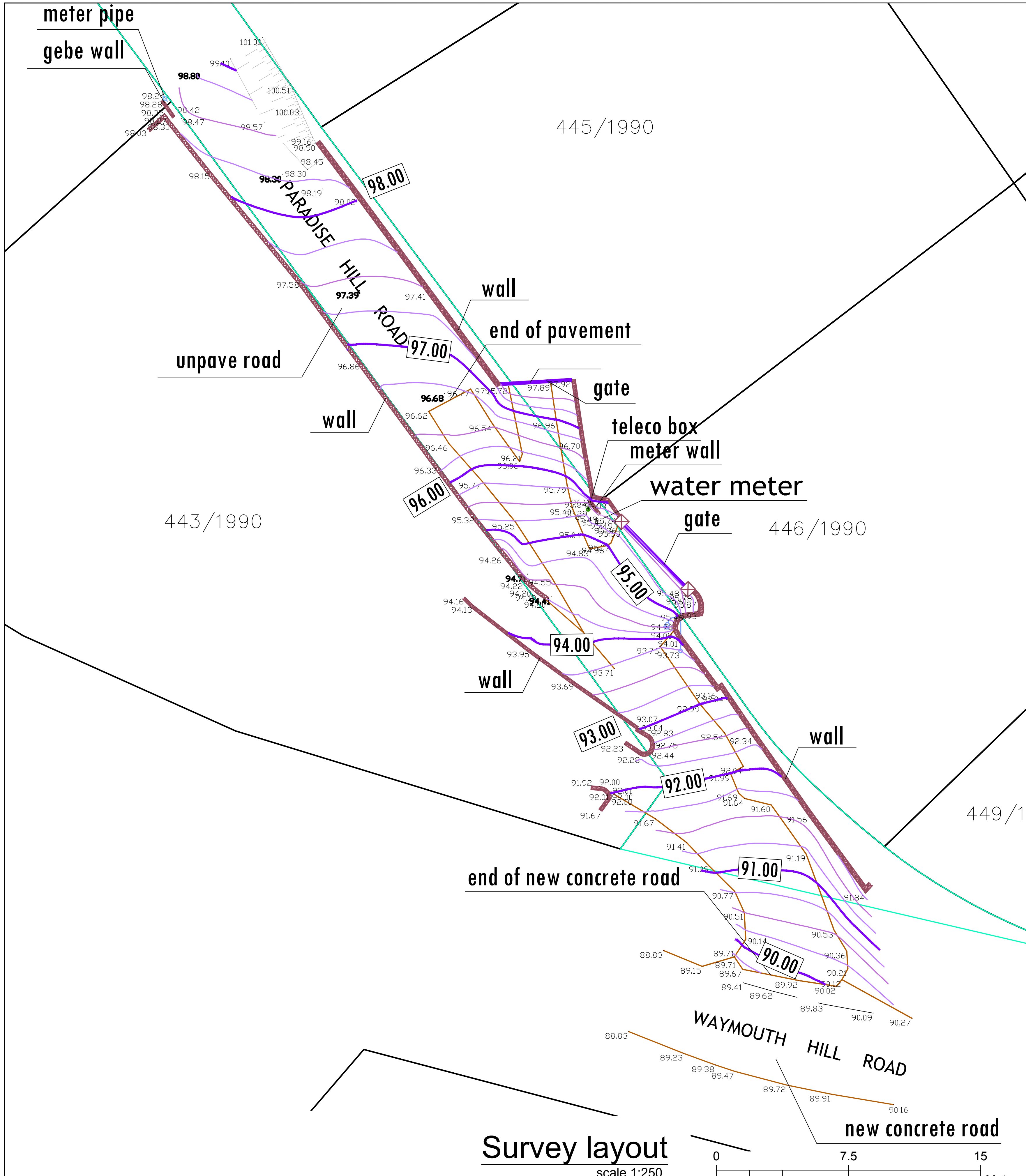
scale 1:100

Remarks:
- Existing features are based on survey drawings by Hunt's Topo Land.
- Dimensions in m, unless specified otherwise.
- Rebar in inches, rebar distance in mm.



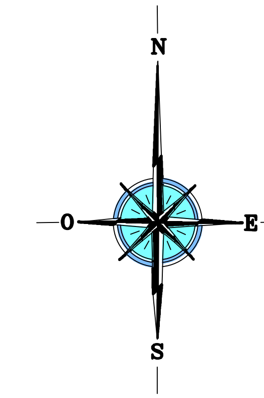
CONCEPT DESIGN

Project : Design and preparation for the construction of the Eastern part of link 6 [Side Roads]		Project nr : 216–1380
Principal : Ministry of VROMI	Date drawn : 18–07–2017	Revision :
Subject : Detail crossing 2	Drawn by : BZ	
	Checked by : GT	
	Format : Tabloid	
	Scale : 1:100	
INDEPENDENT CONSULTING ENGINEERS. N.V. p.o. box 390, zaegersgut road 13, philipsburg, st. maarten, n.a. phone 5422421, fax 5422597, e-mail icesxm@sintmaarten.net		Sheet number : C43
		File number : 216–1380–C43



Survey layout

scale 1:250



Remarks:
- Existing features are based on survey drawings by Hunt's Topo Land.
- Dimensions in m, unless specified otherwise.
- Rebar in inches, rebar distance in mm.

CONCEPT DESIGN

Project : Design and preparation for the construction of the Eastern part of link 6 [Side Roads]

Project nr : 216–1380

Principal : Ministry of VROMI

Subject : Survey layout Paradise Hill road

Date drawn	18–07–2017
Drawn by	BZ
Checked by	GT
Format	Tabloid
Scale	1:250

Revision :

Sheet number : C60

File number : 216–1380–C60

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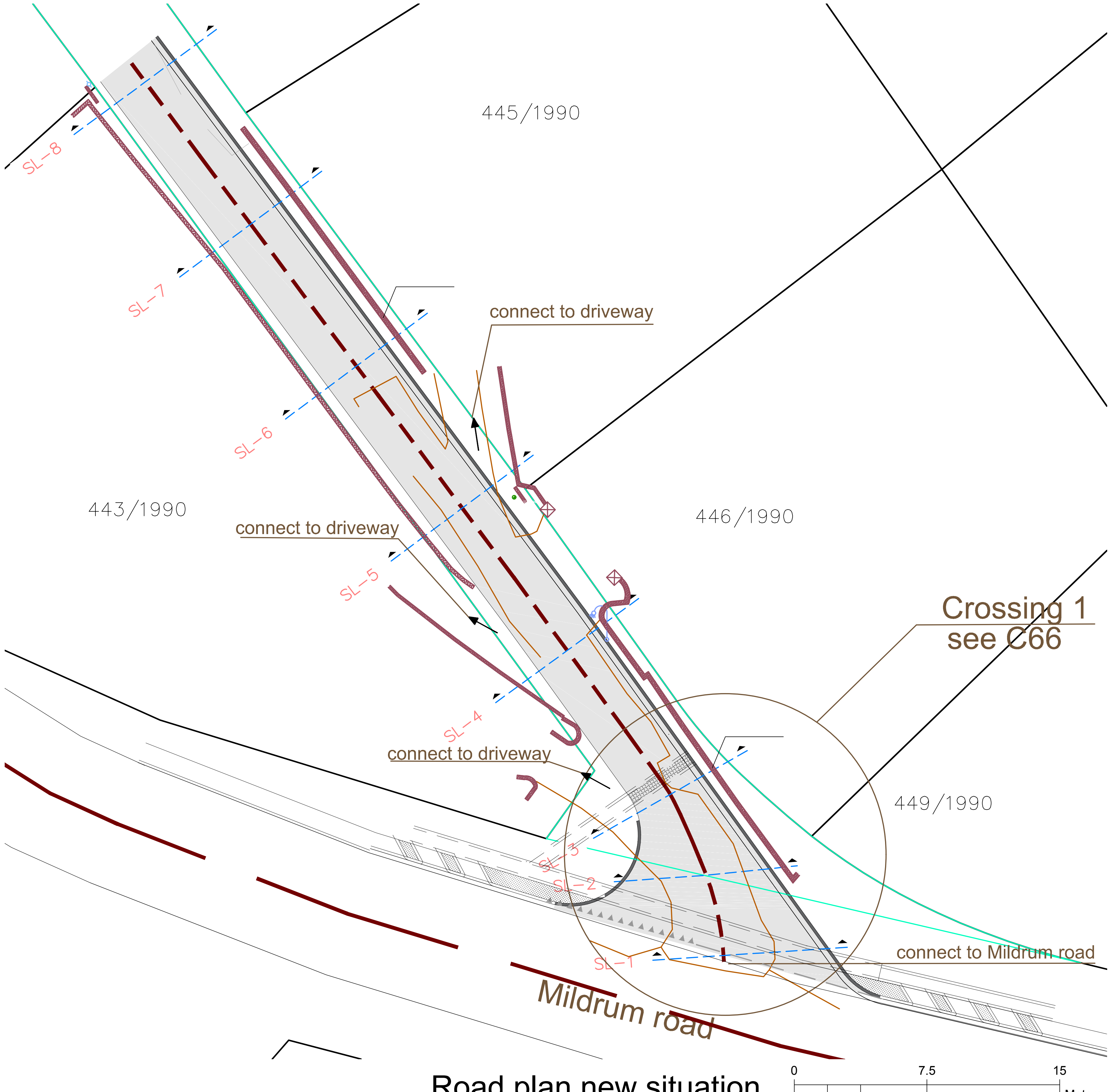
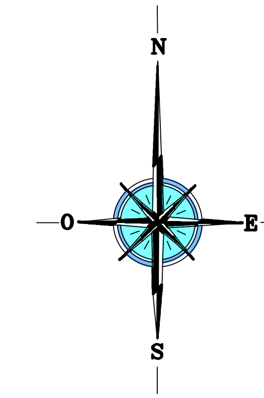
phone 5422421, fax 5422597, e–mail icesxm@sintmaarten.net



Road plan new situation

scale 1:250



Legend

	Road
	Curb
	Alignment

Remarks:

- Existing features are based on survey drawings by Hunt's Topo Land.
- Dimensions in m, unless specified otherwise.
- Rebar in inches, rebar distance in mm.

CONCEPT DESIGN

Project : Design and preparation for the construction of the Eastern part of link 6 [Side Roads]			Project nr : 216–1380
Principal : Ministry of VROMI	Date drawn : 18–07–2017	Revision :	
Subject : Layout road plan Paradise Hill road new situation	Drawn by : BZ		
	Checked by : GT		
	Format : Tabloid		
	Scale : 1:250		
			Sheet number : C61
			File number : 216–1380–C61

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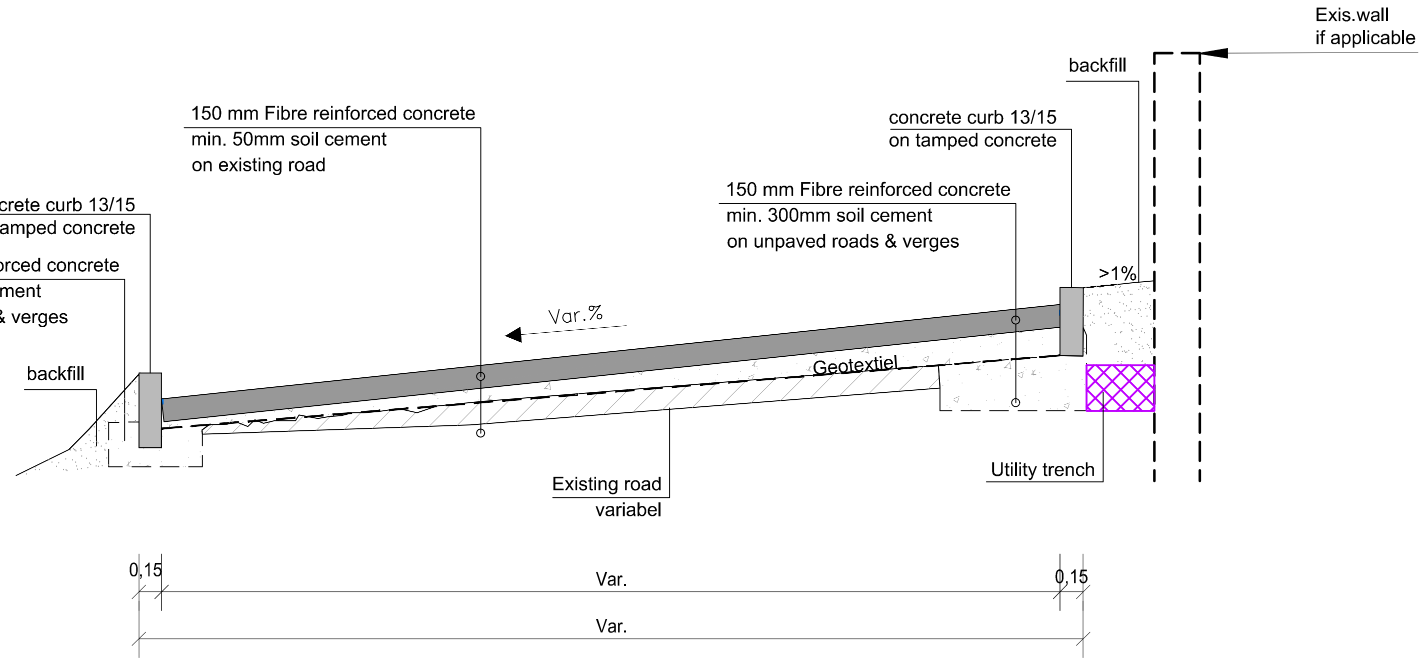
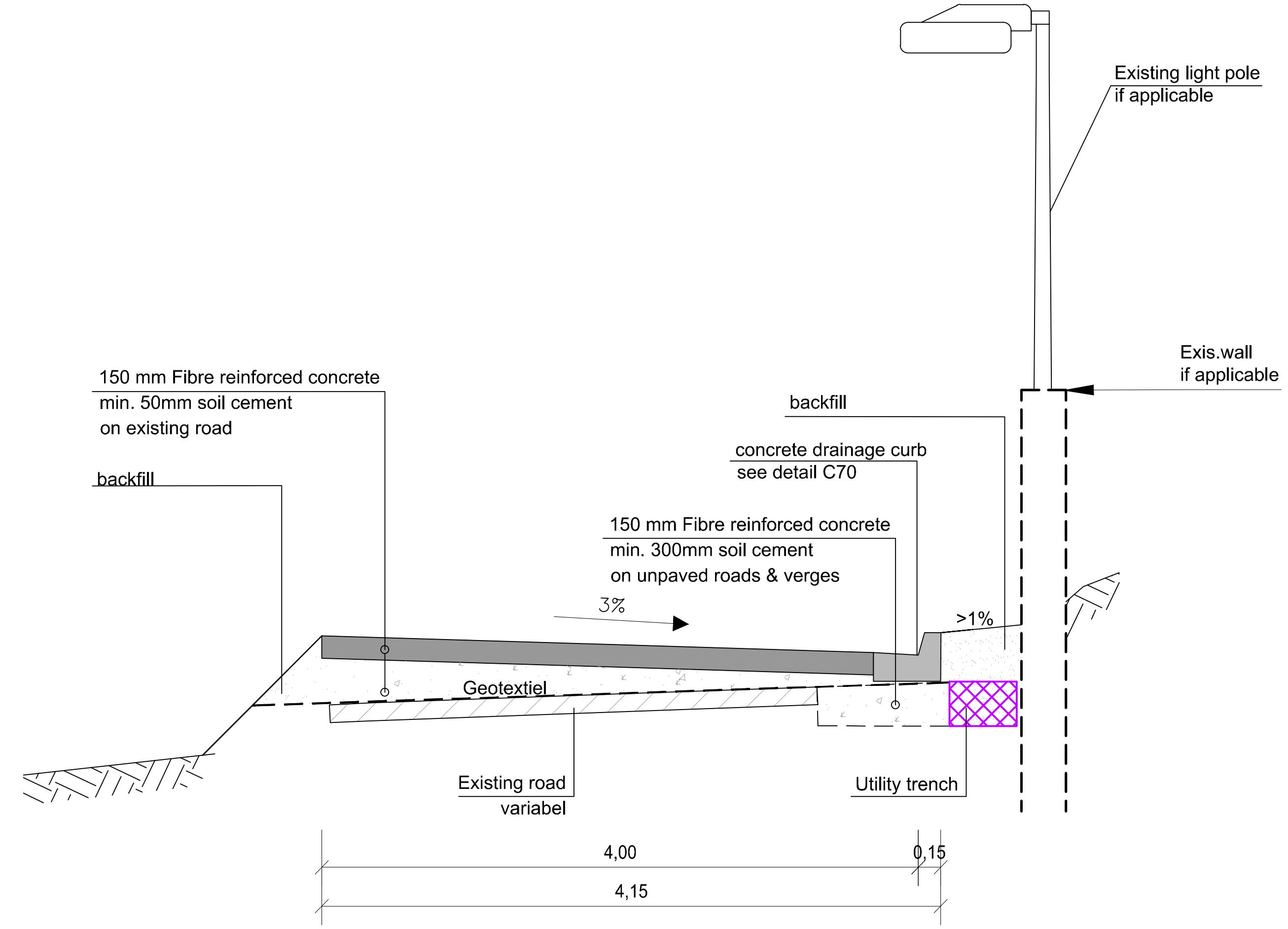
phone 5422421, fax 5422597, e-mail icesxm@sintmaarten.net



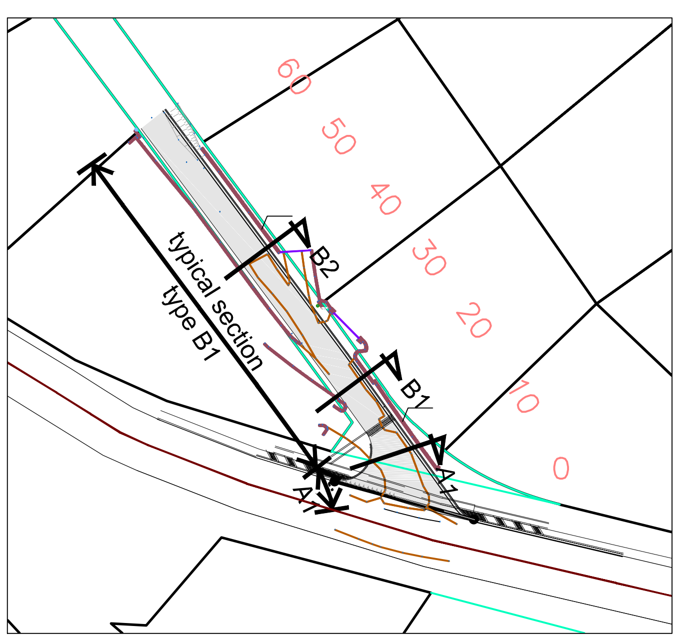
Typical cross section road A1 (0-9m)

scale 1:50



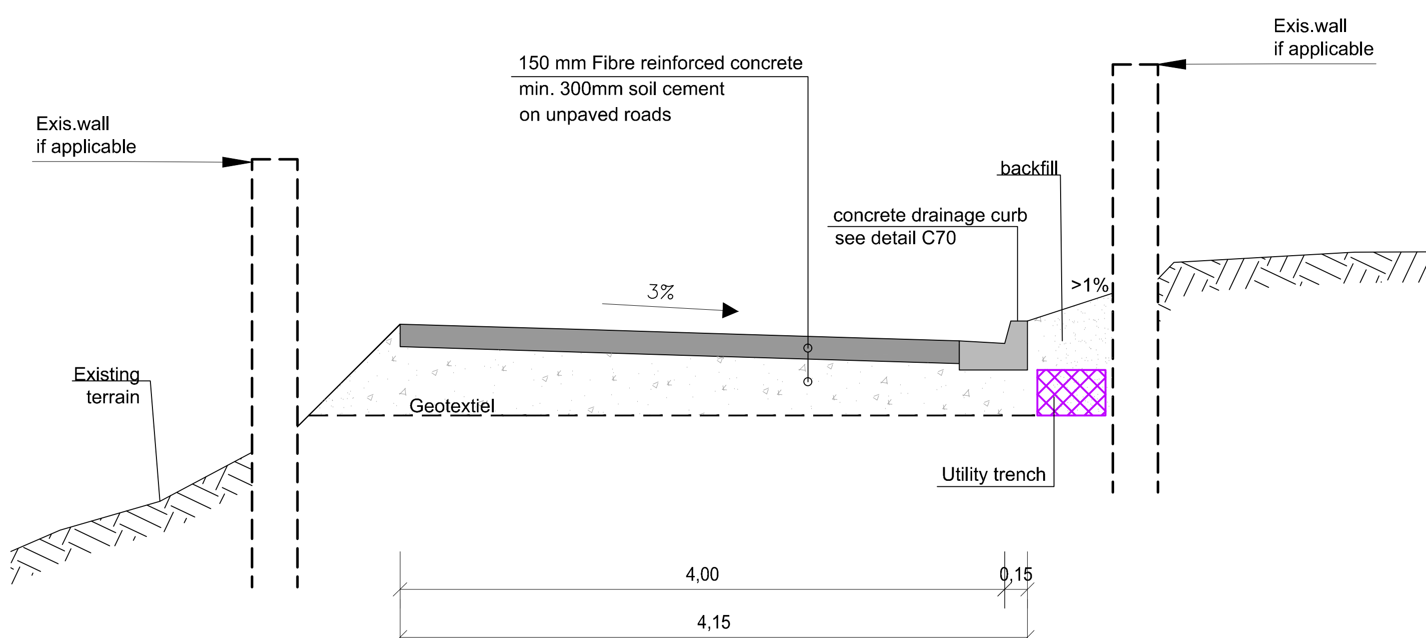
Typical cross section road B1 (9-38m)

scale 1:50



Typical sections layout

scale 1:1000



Typical cross section road B2 (38-61m)

scale 1:50

Remarks:
- Existing features are based on survey drawings by Hunt's Topo Land.
- Dimensions in m, unless specified otherwise.
- Rebar in inches, rebar distance in mm.

CONCEPT DESIGN

Project : Design and preparation for the construction of the Eastern part of link 6 [Side Roads]		Project nr : 216–1380
Principal : Ministry of VROMI	Date drawn : 18-07-2017	Revision :
Subject : Typical cross sections Paradise Hill road Alternative A	Drawn by : BZ	
	Checked by : GT	
	Format : Tabloid	
	Scale : 1:50	
		Sheet number : C62A
		File number : 216-1380-C62A

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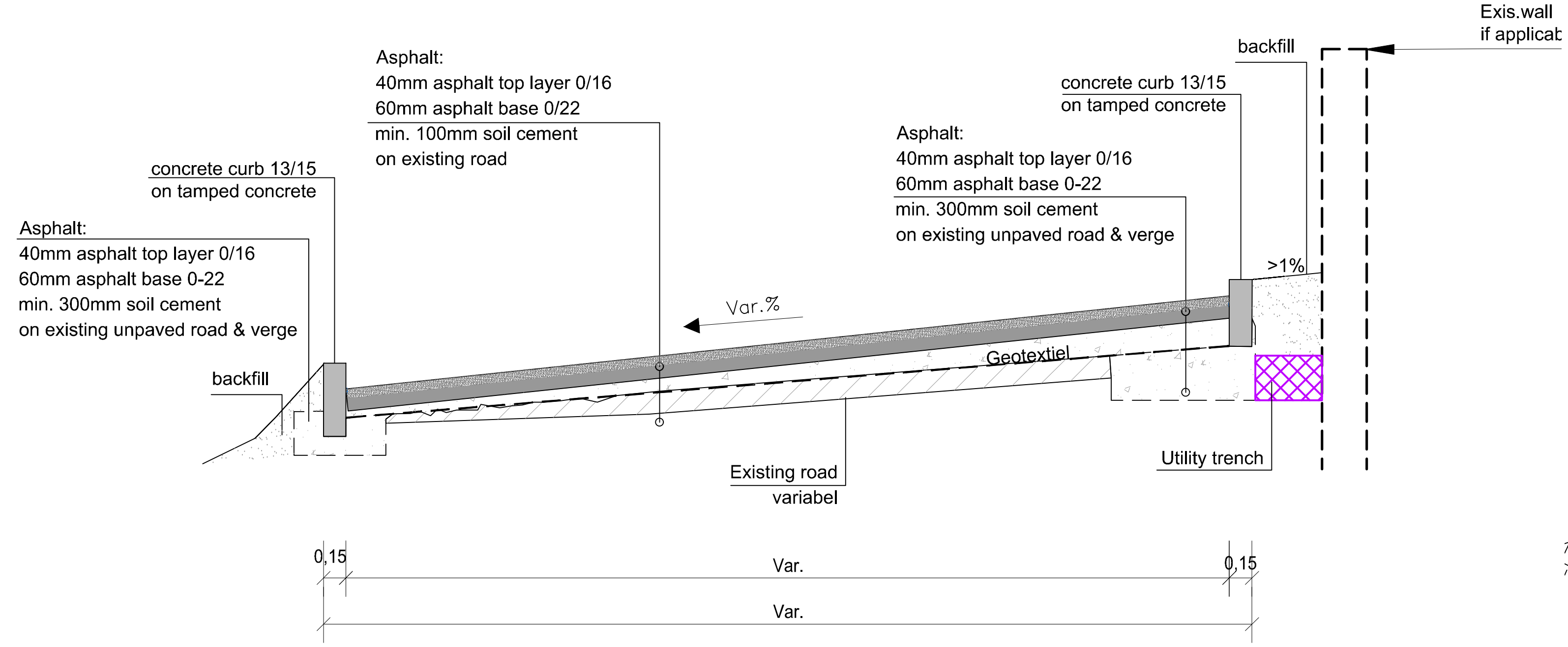
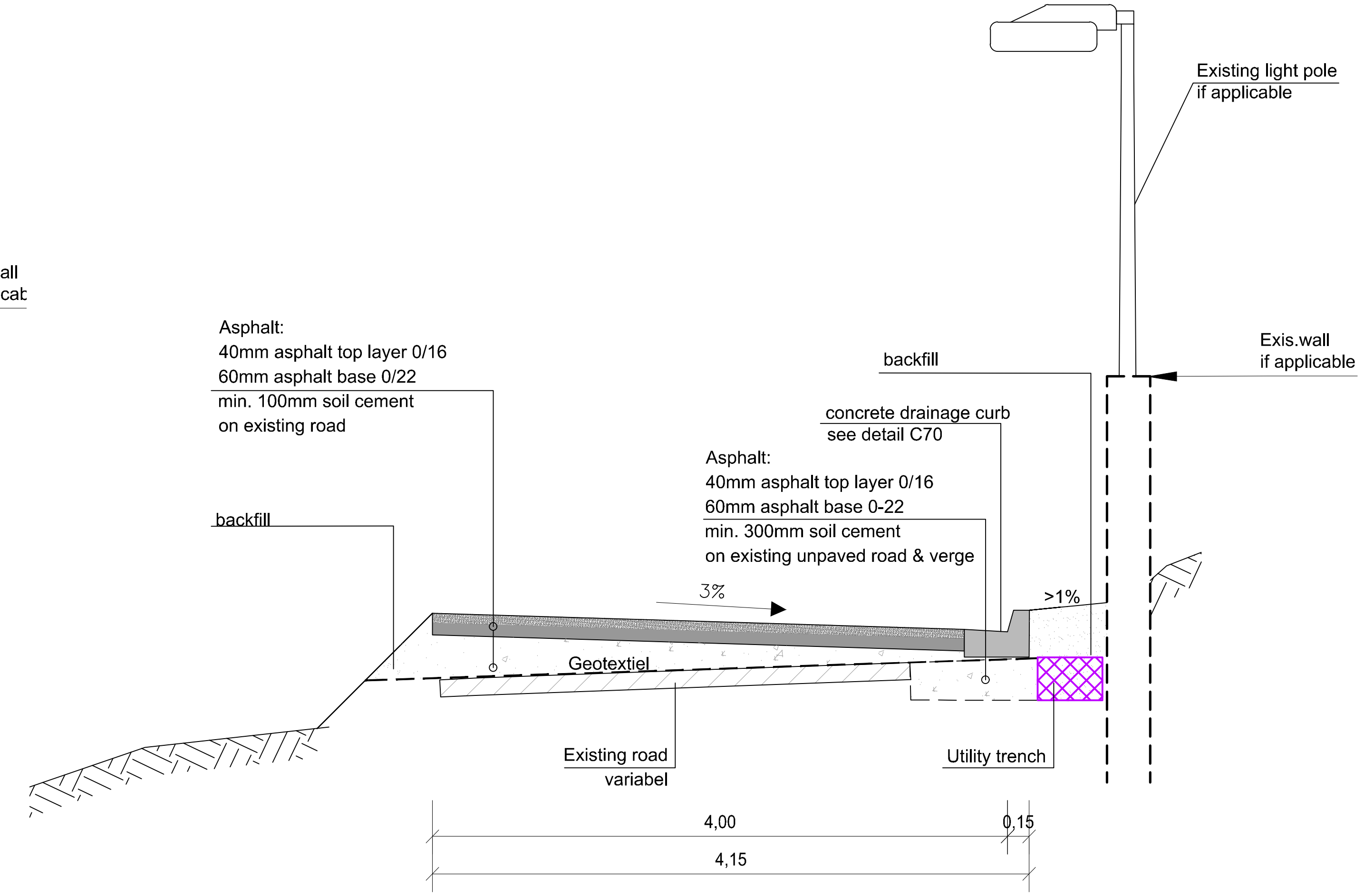
phone 5422421, fax 5422597, e-mail icesxm@sintmaarten.net



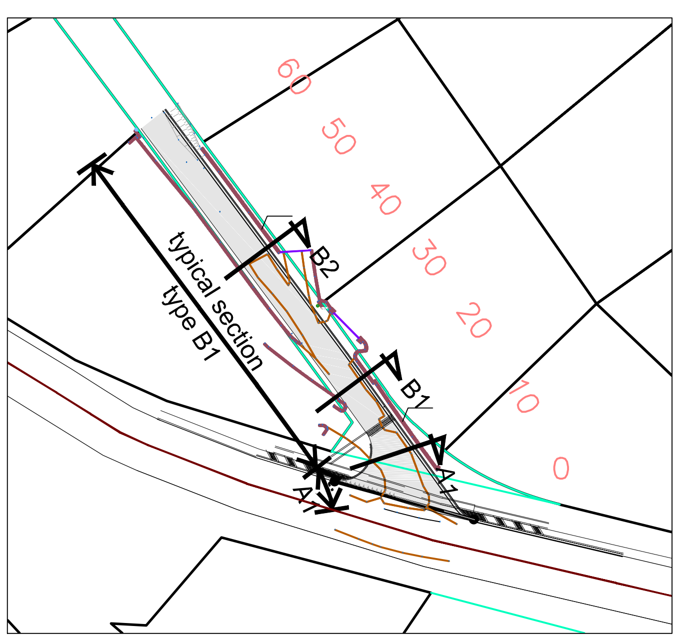
Typical cross section road A1 (0-9m)

scale 1:50



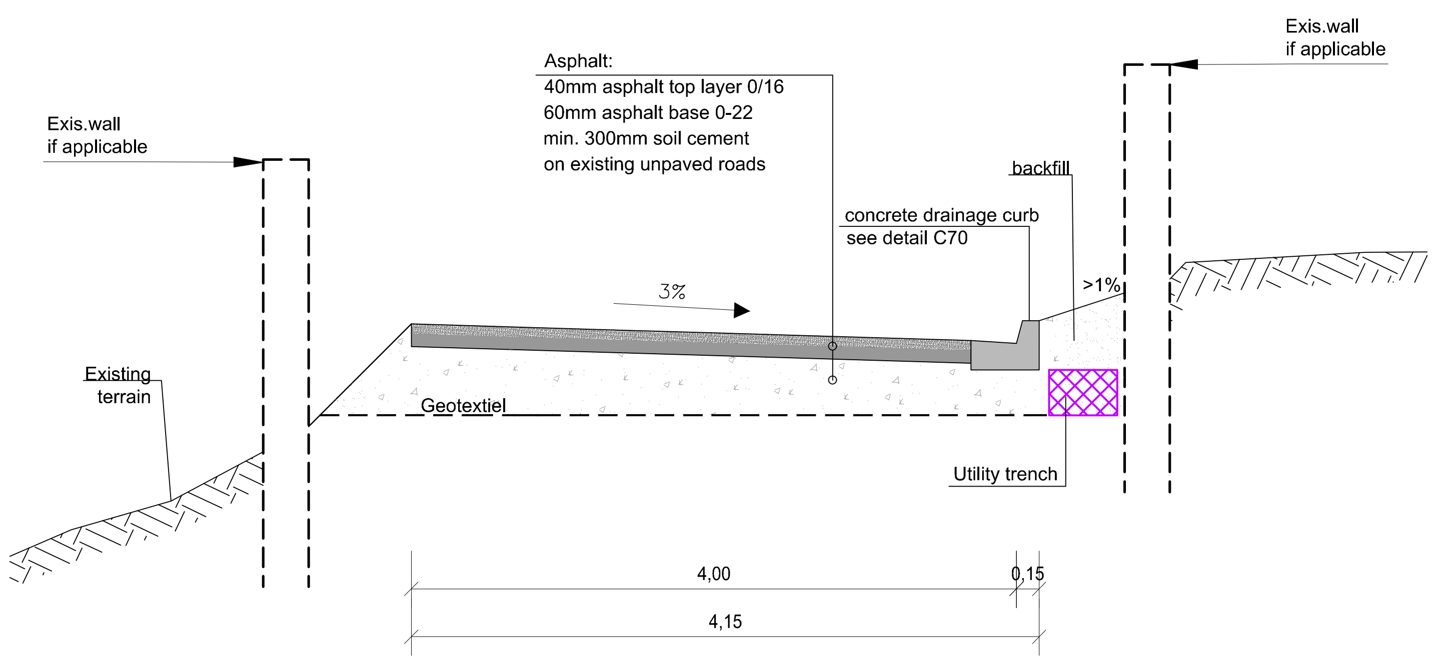
Typical cross section road B1 (9-38m)

scale 1:50



Typical sections layout

scale 1:1000



Typical cross section road B2 (38-61m)

scale 1:50

Remarks:

- Existing features are based on survey drawings by Hunt's Topo Land.
- Dimensions in m, unless specified otherwise.
- Rebar in inches, rebar distance in mm.

CONCEPT DESIGN

Project : Design and preparation for the construction of the Eastern part of link 6 [Side Roads]		Project nr : 216–1380
Principal : Ministry of VROMI	Date drawn : 18-07-2017	Revision :
Subject : Typical cross sections Paradise Hill road Alternative B	Drawn by : BZ	
	Checked by : GT	
	Format : Tabloid	
	Scale : 1:50	
		Sheet number : C62B
		File number : 216-1380-C62B

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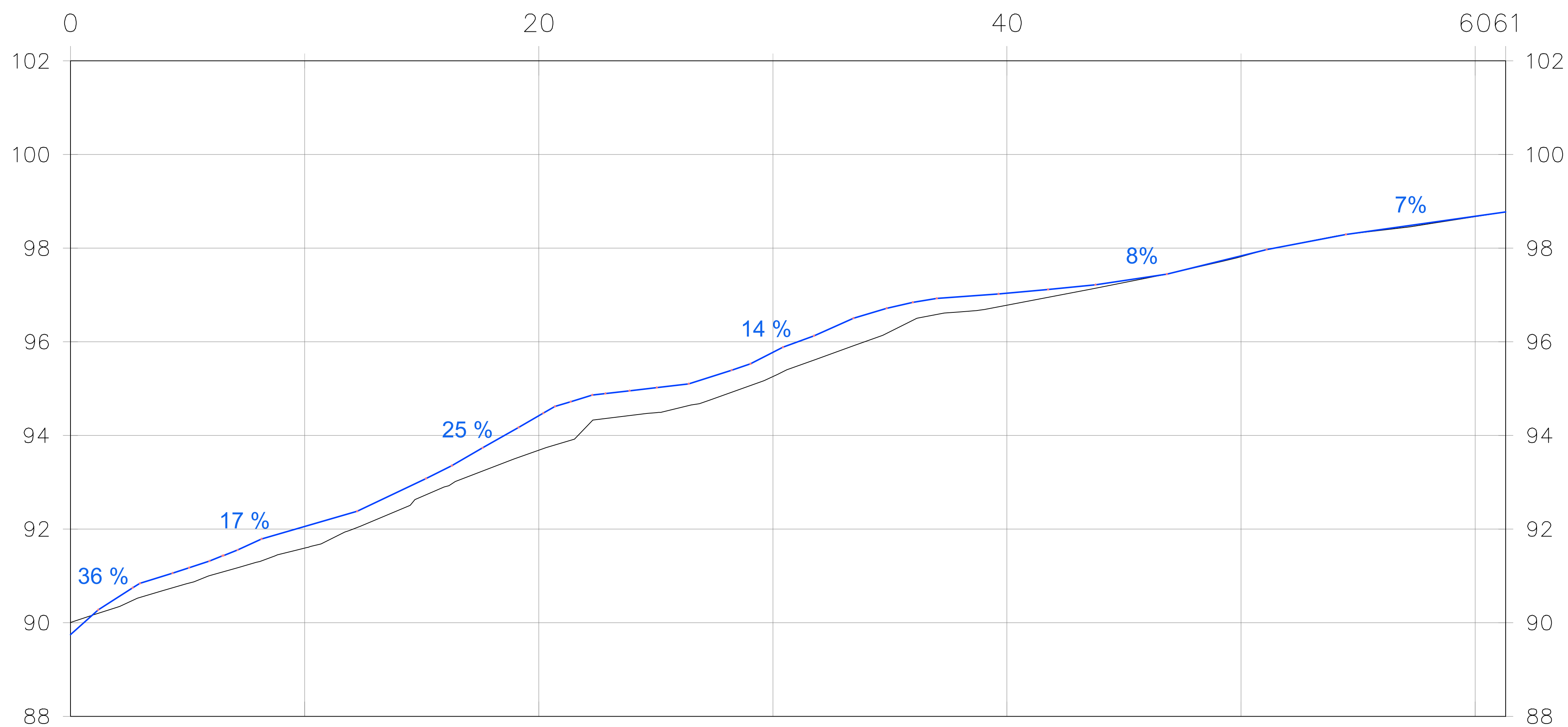
phone 5422421, fax 5422597, e-mail icesxm@sintmaarten.net



Existing level	0+000.00 90.01	0+010.00 91.59	0+020.00 93.68	0+030.00 95.26	0+040.00 96.78	0+050.00 97.82	0+060.00 98.67
Design level	0+000.00 89.75	0+010.00 92.05	0+020.00 94.42	0+030.00 95.77	0+040.00 97.04	0+050.00 97.83	0+060.00 98.68

Longitudinal profile road axis

CONCEPT DESIGN

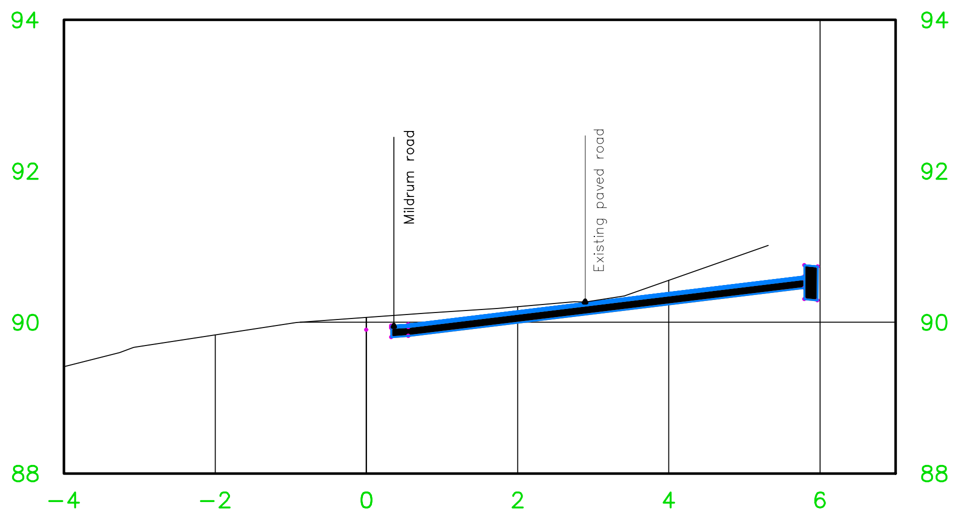
Remarks:
- Existing level
- Design level

Remarks:
- Existing features are based on survey drawings by Hunt's Topo Land.
- Dimensions in m, unless specified otherwise.
- Rebar in inches, rebar distance in mm.

- For cross sections location see C61
- For cross sections see C64 -C65

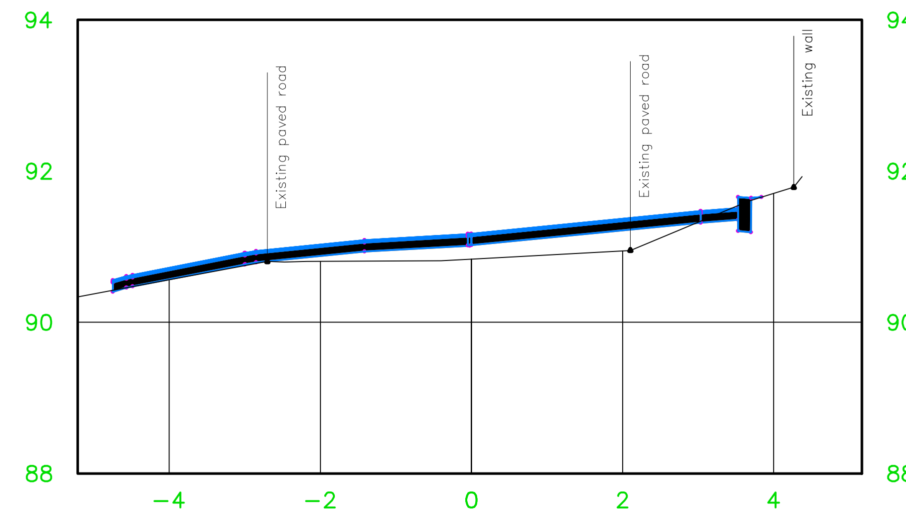
Project : Design and preparation for the construction of the Eastern part of link 6 [Side Roads]		Project nr : 216–1380
Principal : Ministry of VROMI	Date drawn : 18-07-2017	Revision :
Subject : Paradise Hill Road Longitudinal section	Drawn by : BZ	
	Checked by : GT	
	Format : Tabloid	
	Scale : 1:250	
INDEPENDENT CONSULTING ENGINEERS. N.V.		Sheet number : C63
p.o. box 390, zaegersgut road 13, philipsburg, st. maarten, n.a. phone 5422421, fax 5422597, e-mail icesxm@sintmaarten.net		File number : 216-1380-C63

SL-1 (0.35m>



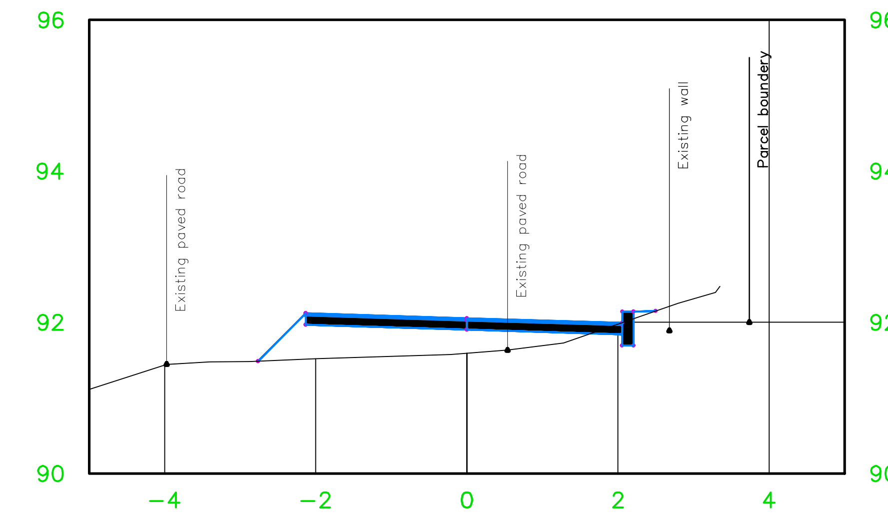
	-4	-2	0	2	4	6
Existing level	89.41	89.83	90.06	90.21	90.56	
Design level				90.23	90.60	

SL-2 (5.00m>



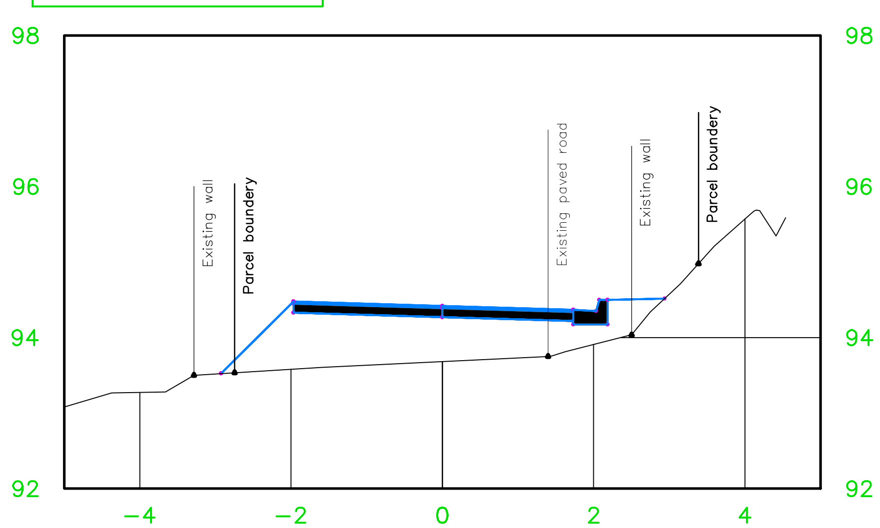
	-4	-2	0	2	4
Existing level	90.56	90.81	90.83	90.94	91.70
Design level	90.72	91.00	91.17	91.36	

SL-3 (10.00m>



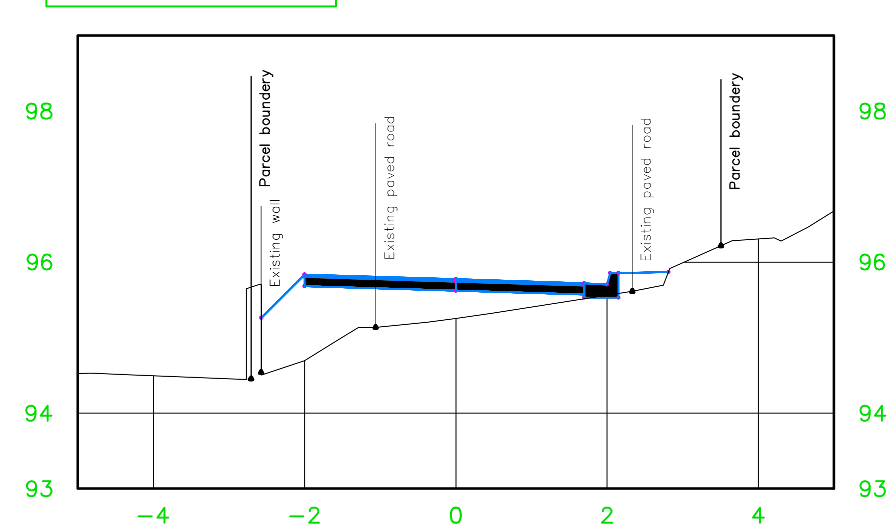
	-4	-2	0	2	4
Existing level	91.44	91.52	91.59	91.98	
Design level		92.11	92.05	91.99	

SL-4 (20.00m>



	-4	-2	0	2	4
Existing level	93.27	93.58	93.68	93.91	95.57
Design level		94.45	94.42	94.35	

SL-5 (30.00m>



	-4	-2	0	2	4
Existing level	94.50	94.69	95.26	95.56	96.30
Design level		95.83	95.77	95.70	

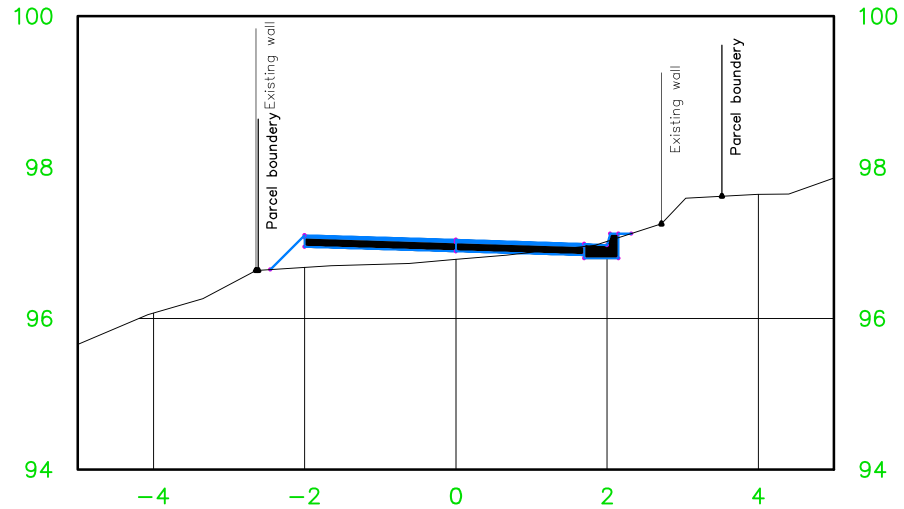
Remarks:
- Existing features are based on survey drawings by Hunt's Topo Land.
- Dimensions in m, unless specified otherwise.
- Rebar in inches, rebar distance in mm.

- For cross section location see C61
- For longitudinal section see C63

CONCEPT DESIGN

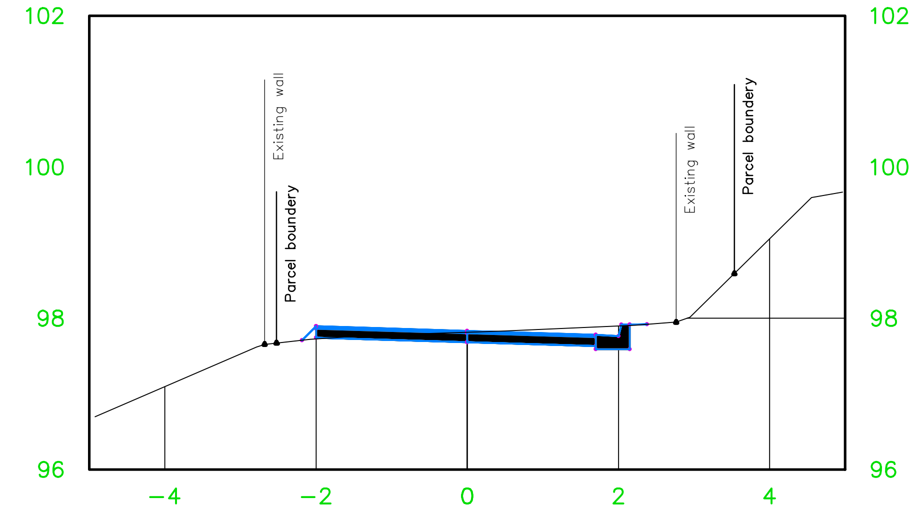
Project : Design and preparation for the construction of the Eastern part of link 6 [Side Roads]			Project nr : 216-1380
Principal : Ministry of VROMI		Date drawn : 18-07-2017	Revision :
Subject : Paradise Hill Road Sections SL-1 / SL-5		Drawn by : BZ	
		Checked by : GT	
		Format : Tabloid	
		Scale : 1:100	
INDEPENDENT CONSULTING ENGINEERS. N.V.			Sheet number : C64
p.o. box 390, zaegersgut road 13, philipsburg, st. maarten, n.a. phone 5422421, fax 5422597, e-mail icesxm@sintmaarten.net			File number : 216-1380-C64

SL–6 (40.00m>



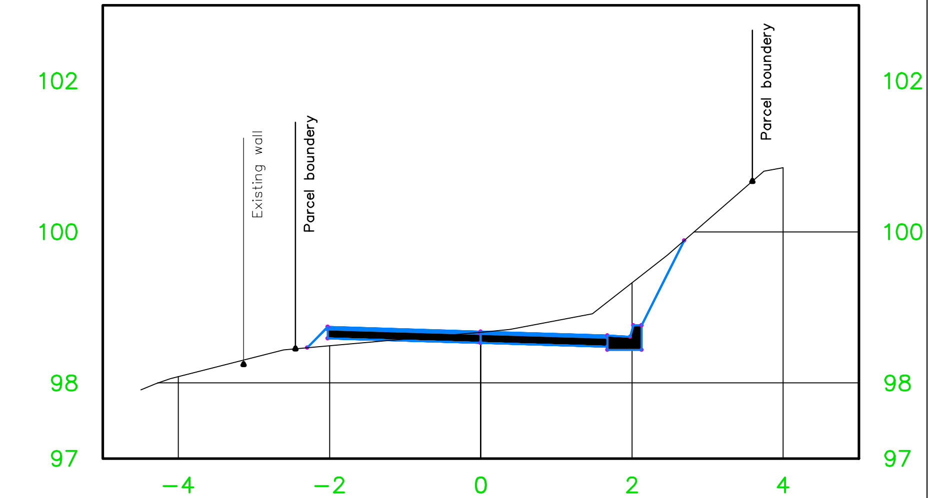
	–4	–2	0	2	4
Existing level	96.07	96.67	96.78	97.02	97.64
Design level		97.10	97.04	96.97	

SL–7 (50.00m>



	–4	–2	0	2	4
Existing level	97.10	97.73	97.82	97.90	99.05
Design level		97.89	97.83	97.76	

SL–8 (60.00m>



	–4	–2	0	2	4
Existing level	98.08	98.49	98.67	99.33	100.85
Design level		98.74	98.68	98.70	

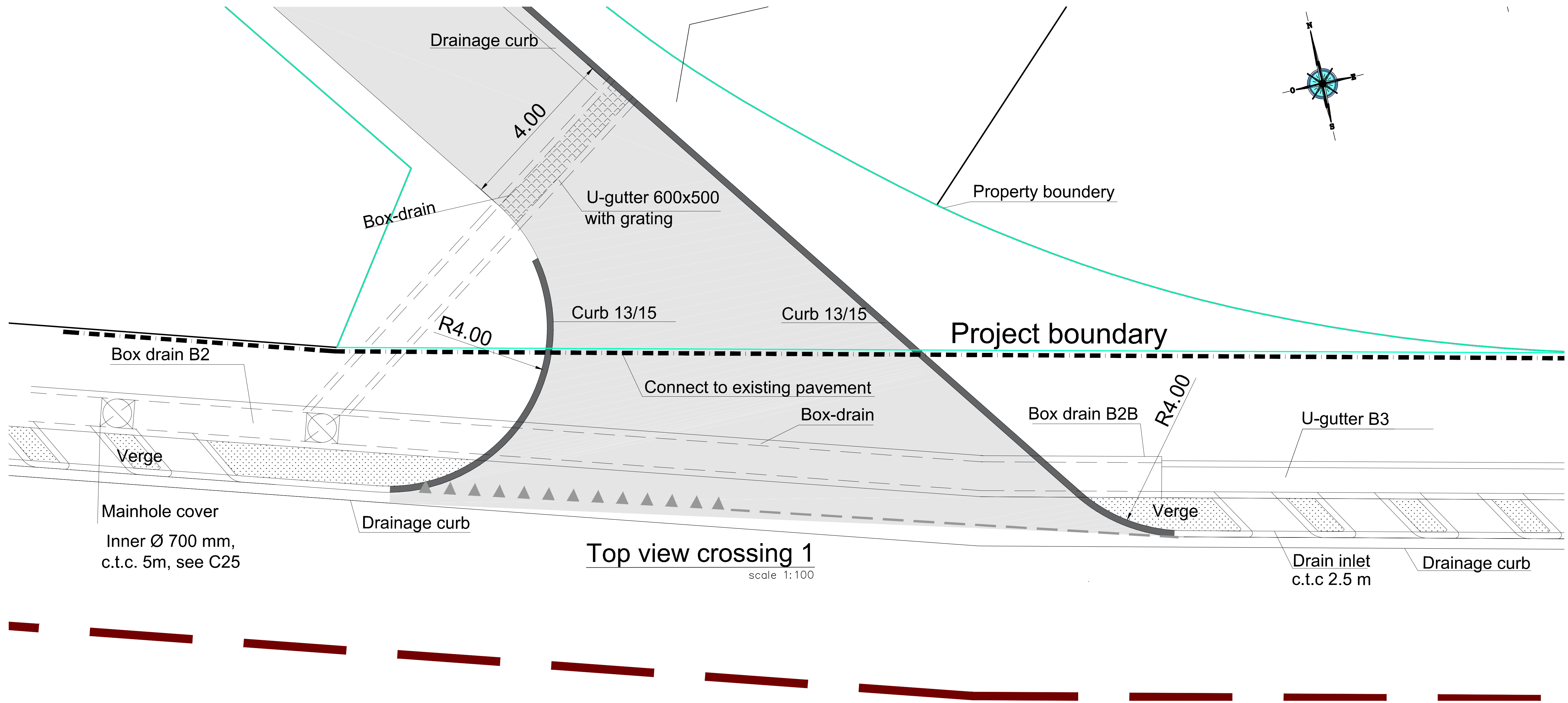
Remarks:

- Existing features are based on survey drawings by Hunt's Topo Land.
- Dimensions in m, unless specified otherwise.
- Rebar in inches, rebar distance in mm.

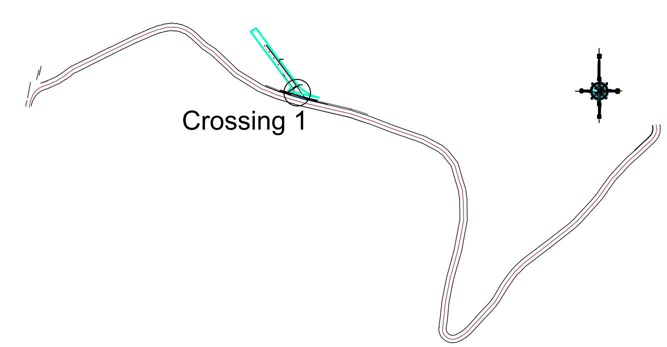
- For cross section location see C61
- For longitudinal section see C63

CONCEPT DESIGN

Project : Design and preparation for the construction of the Eastern part of link 6 [Side Roads]			Project nr : 216–1380
Principal : Ministry of VROMI	Date drawn : 17–07–2017	Revision :	
Subject : Paradise Hill Road Sections SL–6 / SL–8	Drawn by : BZ		
	Checked by : GT		
	Format : Tabloid		
	Scale : 1:100		
INDEPENDENT CONSULTING ENGINEERS. N.V.			Sheet number : C65
p.o. box 390, zaegersgut road 13, philipsburg, st. maarten, n.a. phone 5422421, fax 5422597, e-mail icesxm@sintmaarten.net			File number : 216–1380–C65

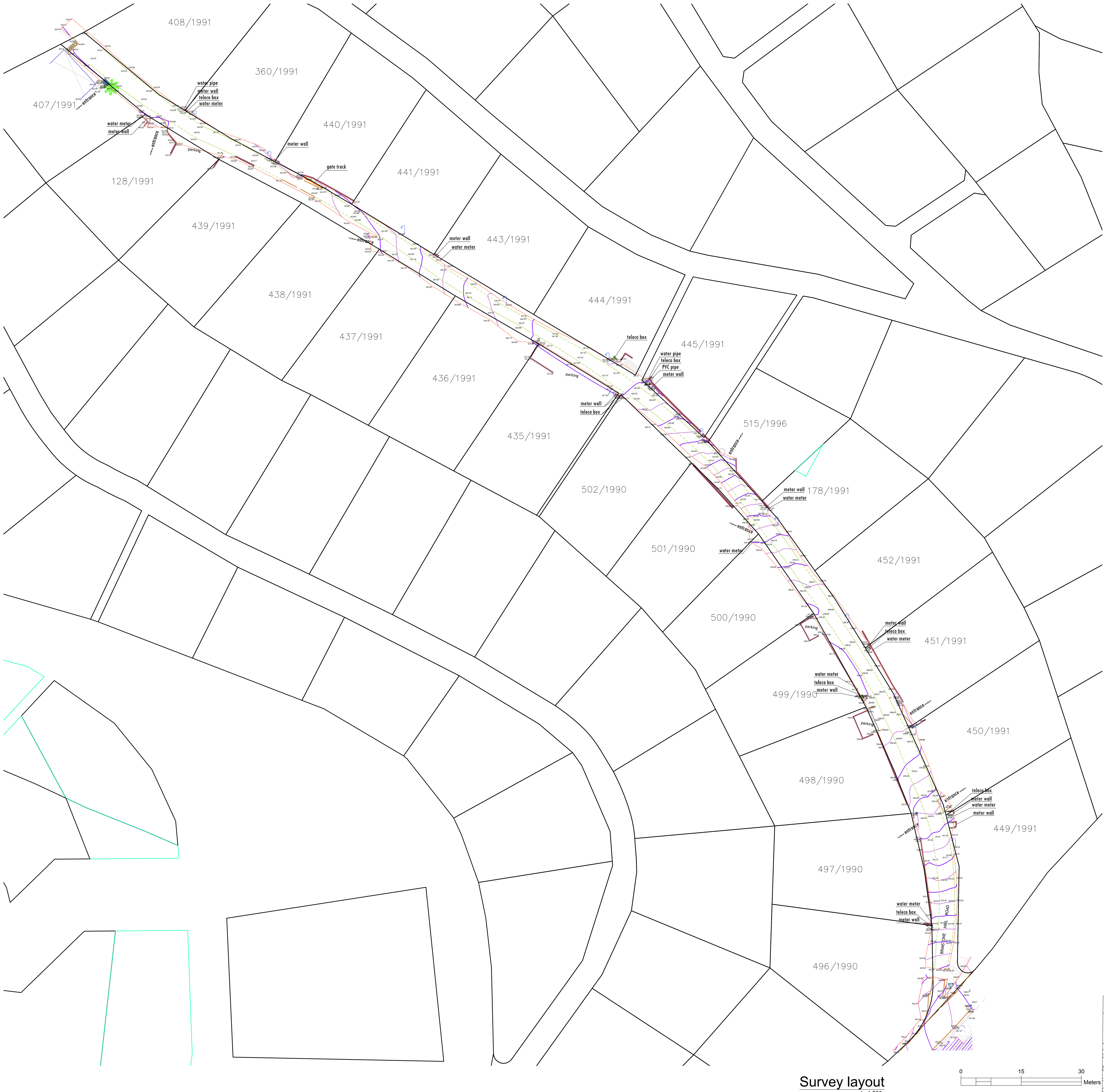


Remarks:
- Existing features are based on survey drawings by Hunt's Topo Land.
- Dimensions in m, unless specified otherwise.
- Rebar in inches, rebar distance in mm.



CONCEPT DESIGN

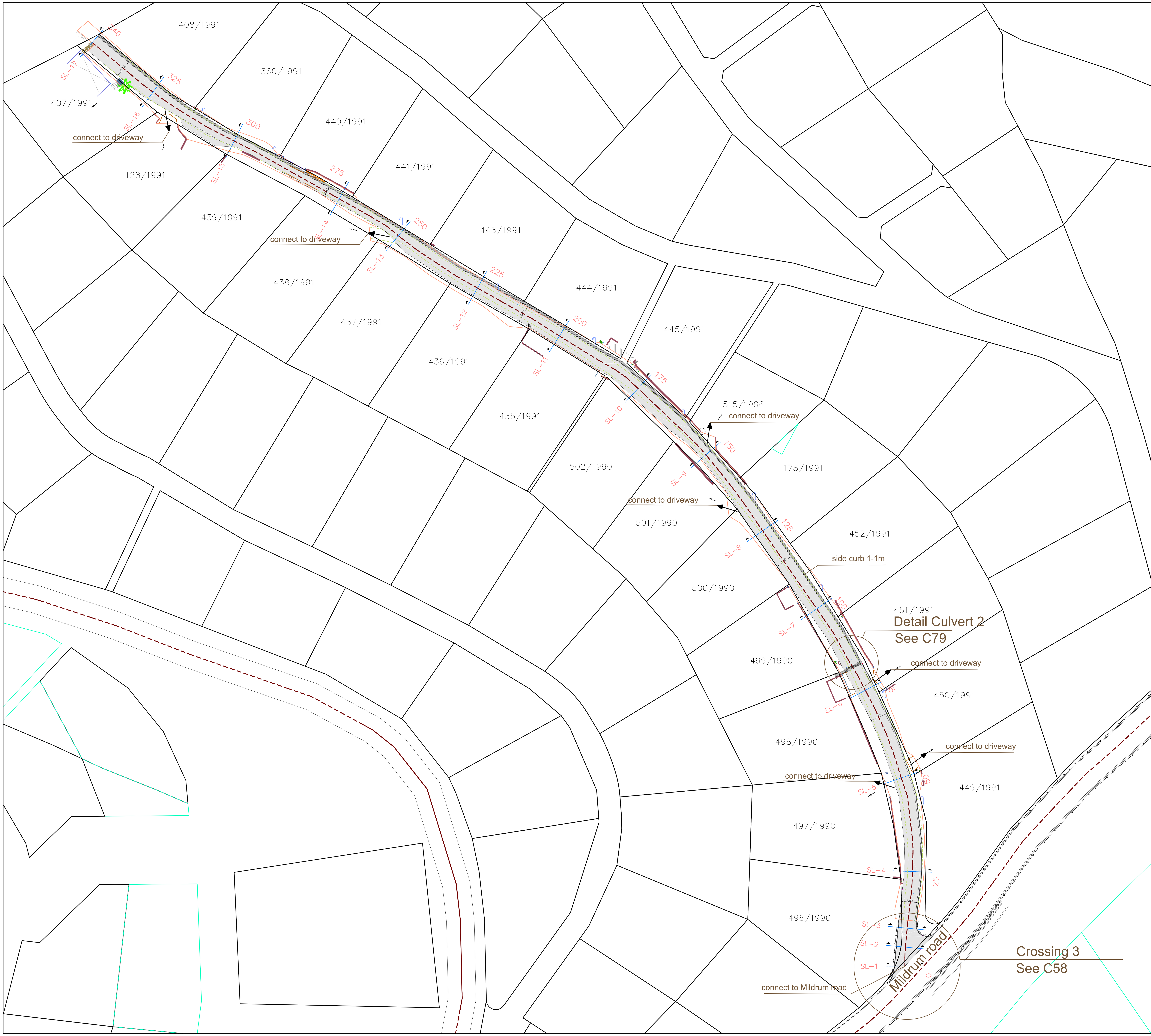
Project : Design and preparation for the construction of the Eastern part of link 6 [Side Roads]		Project nr : 216−1380
Principal : Ministry of VROMI	Date drawn : 30−06−2017	Revision :
Subject : Detail crossing 1 Paradise Hill Road	Drawn by : BZ	
	Checked by : GT	
	Format : Tabloid	
	Scale : 1:100	
INDEPENDENT CONSULTING ENGINEERS. N.V.		Sheet number : C66
p.o. box 390, zaegersgut road 13, philipsburg, st. maarten, n.a. phone 5422421, fax 5422597, e-mail icesxm@sintmaarten.net		File number : 216−1380−C66



Survey layout
scale 1:500

Remarks:
- Existing features are based on survey drawings by Hunt's Topo Land.
- Dimensions in m, unless specified otherwise.
- Rebar in inches, rebar distance in mm.

Project : Design and preparation for the construction of the Eastern part of link 6 [Side Roads]			Project nr : 216-1380
Principal : Ministry of VROMI	Date drawn : 18-07-2017	Revision :	
Subject : Survey layout Brimstone Hill road	Drawn by : B.Z		
	Checked by : G.T		
	Format : A1		
	Scale : 1:500		
INDEPENDENT CONSULTING ENGINEERS. N.V. p.o. box 390, zaegersgut road 13, philipsburg, st. maarten, n.a. phone 5422421, fax 5422597, e-mail icesxm@sintmaarten.net			Sheet number : C-50
			File number : 216-1380-C50



Layout road plan new situation

scale 1:500

0 15 30 Meters

LEGEND

- Road
- Gutter
- Curb
- Alignment

Remarks:

- Existing features are based on survey drawings by Hunt's Topo Land.
- Dimensions in m, unless specified otherwise.
- Rebar in inches, rebar distance in mm.

- For longitudinal section see C53
- For cross sections see C54 -C57

CONCEPT DESIGN

Project : Design and preparation for the construction of the Eastern part of link 6 [Side Roads]	Project nr : 216-1380
Principal : Ministry of VROMI	Date drawn : 18-07-2017
Subject : Layout road plan Brimstone Hill road New situation	Drawn by : BZ
	Checked by : GT
	Format : A1
	Scale : 1:500
	Revision :

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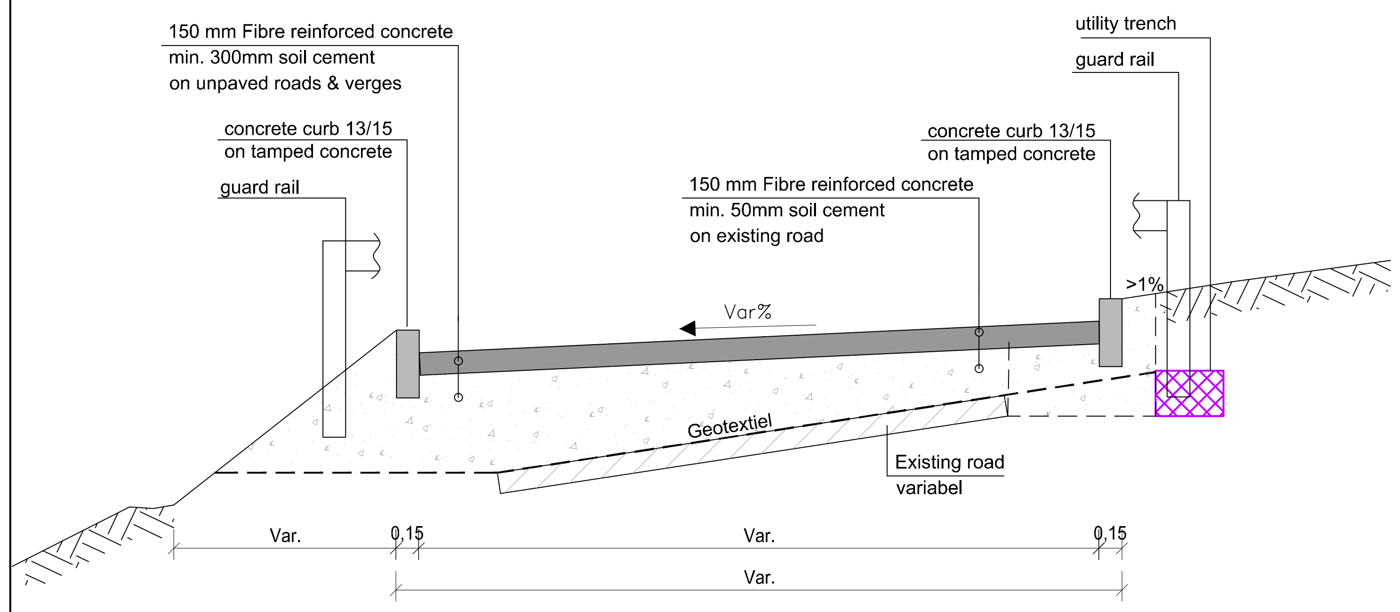
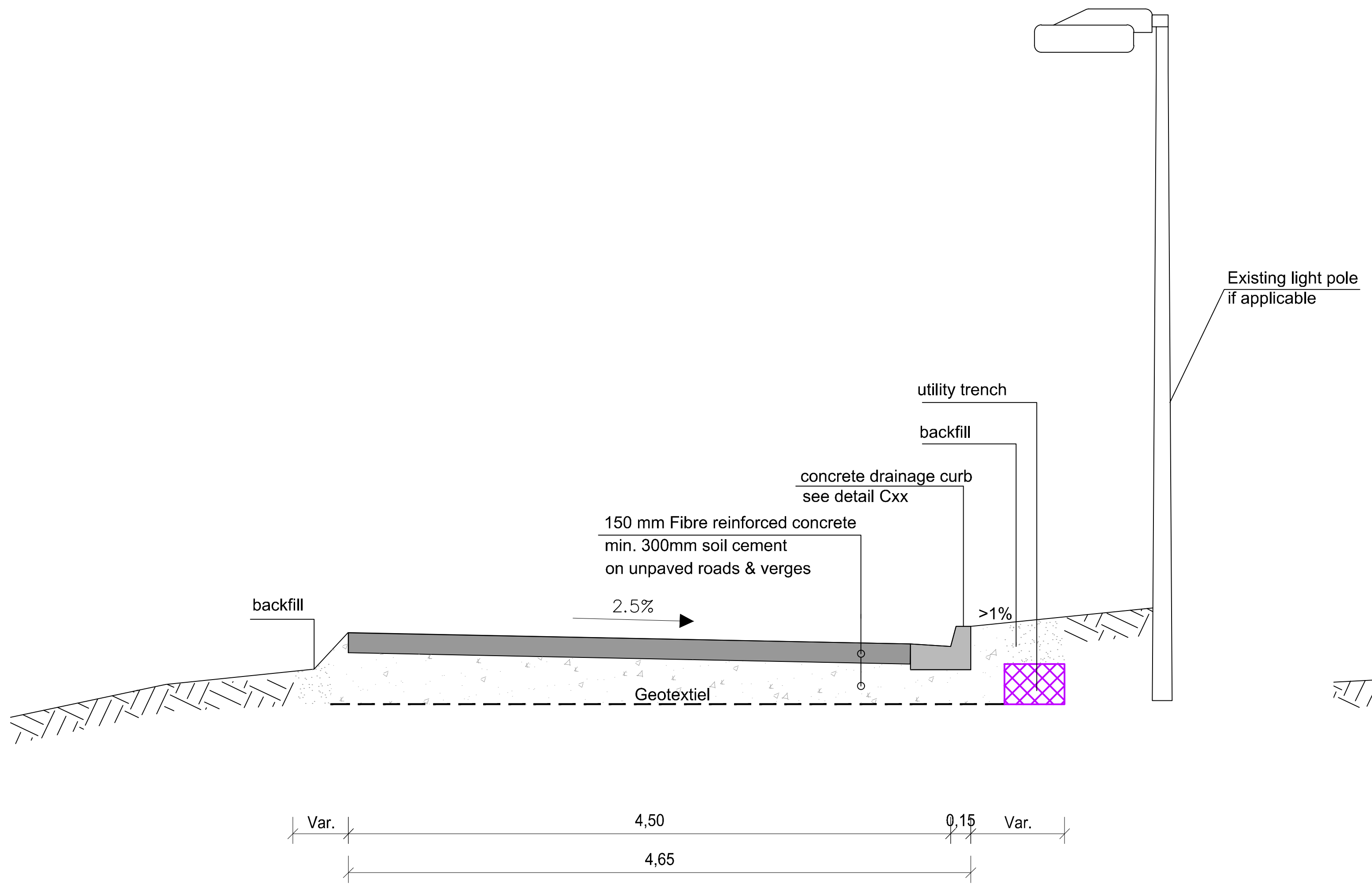
phone 5422421, fax 5422597, e-mail icesxm@sintmaarten.net

Sheet number : C51

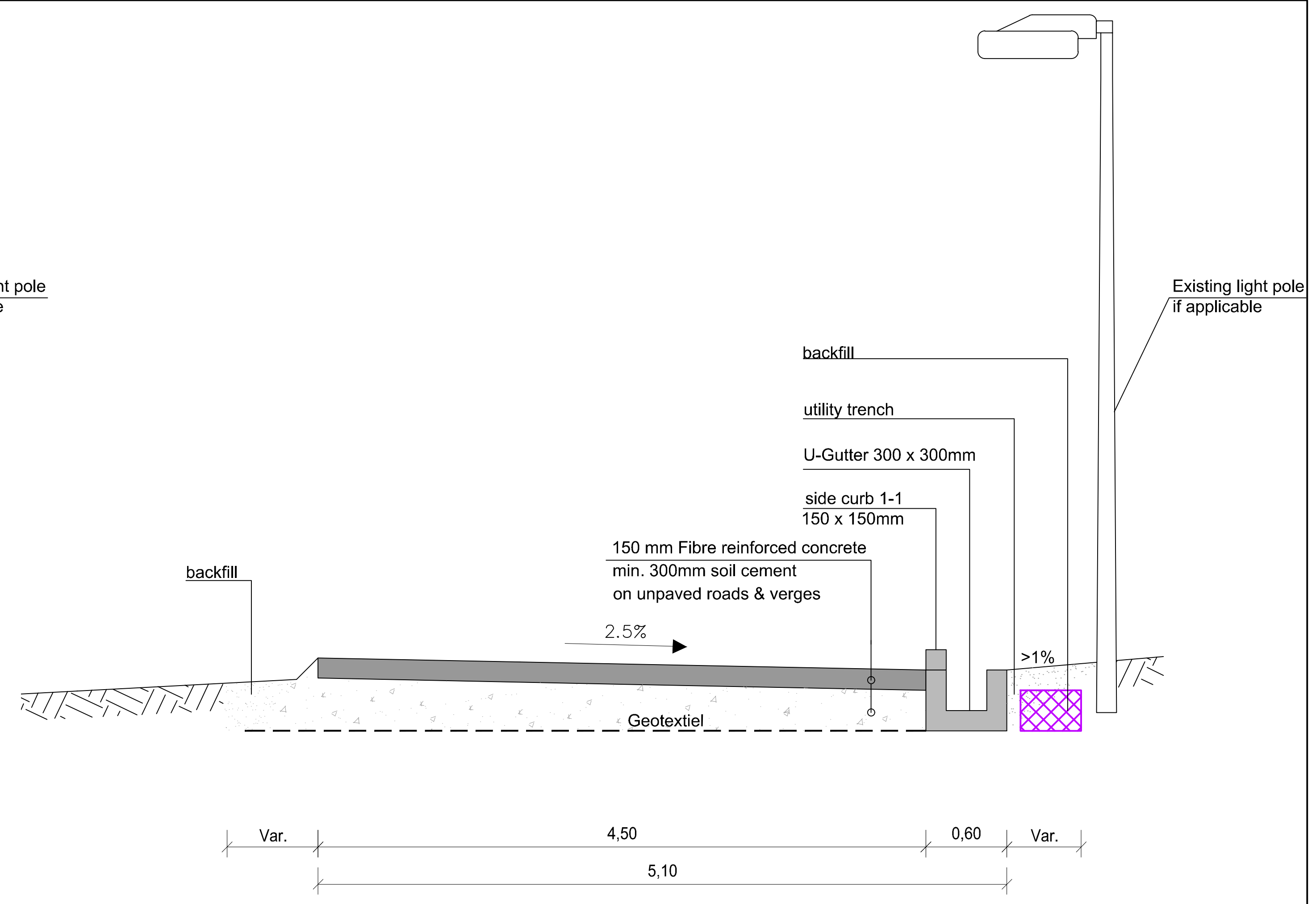
File number : 216-1380-C51



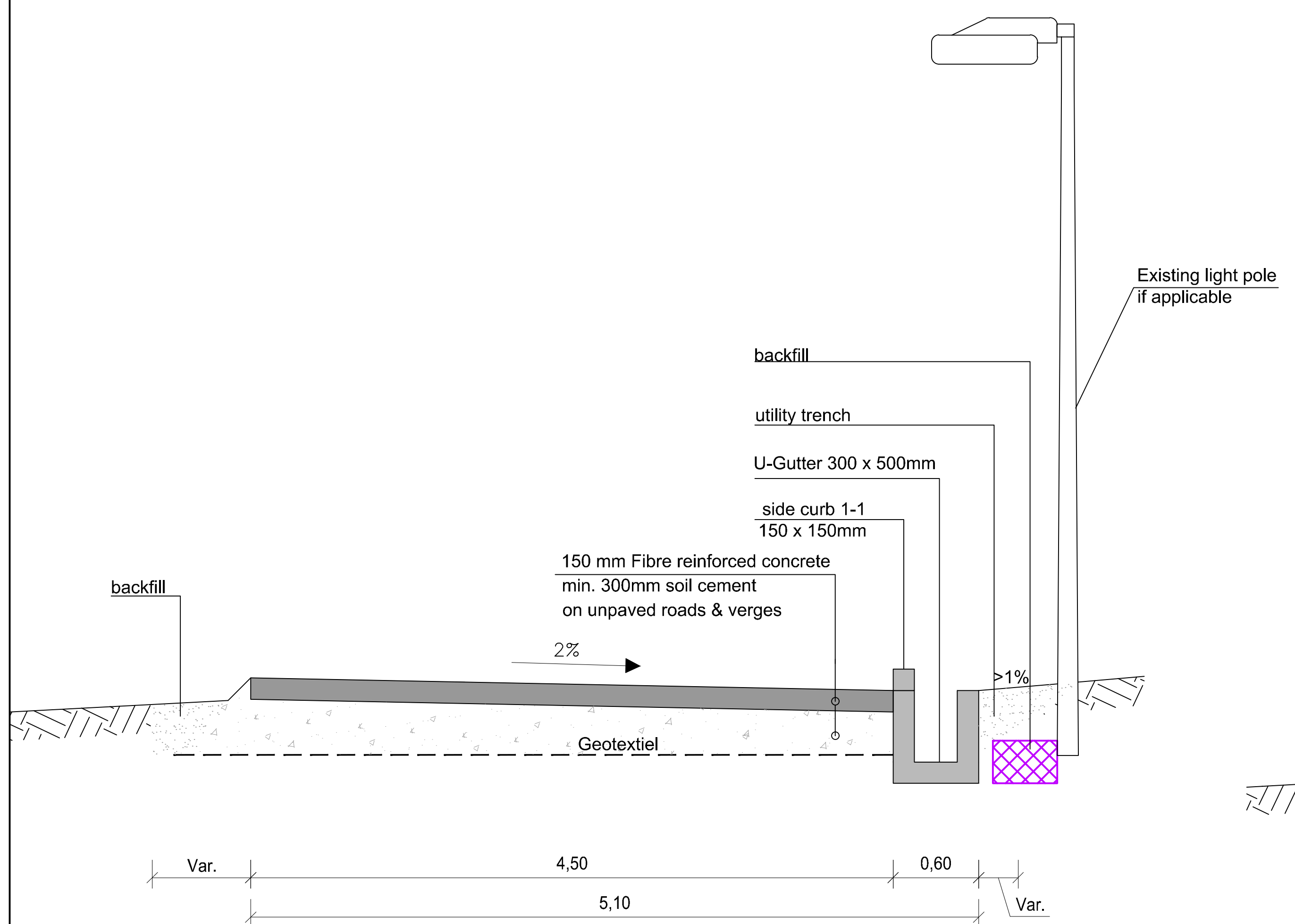
A1 typical cross section road (0-12m)
scale 1:50



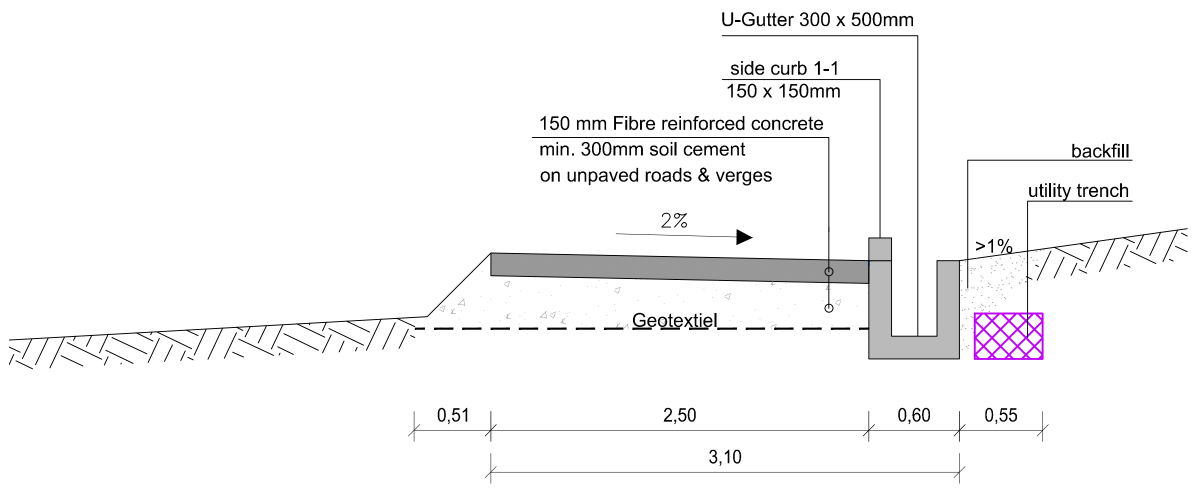
A2 typical cross section road (12-82m)
scale 1:50



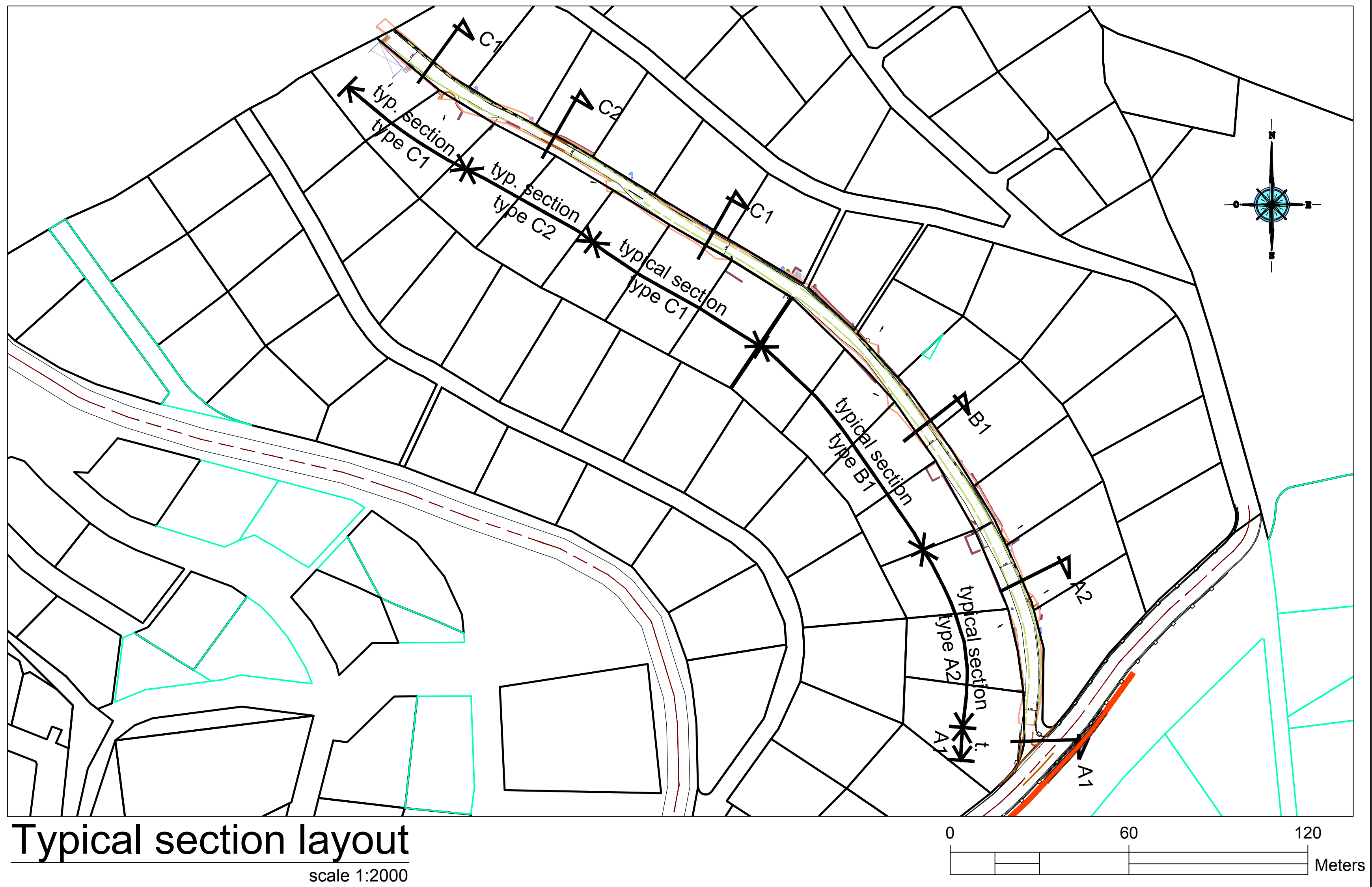
B1 typical cross section road (82-160m)
scale 1:50



C1 typical cross section road (160-245m & 300-346m)
scale 1:50



C2 typical cross section road (245-300m)
scale 1:50



Typical section layout
scale 1:2000

Remarks:
- Existing features are based on survey drawings by Hunt's Topo Land.
- Dimensions in m, unless specified otherwise.
- Rebar in inches, rebar distance in mm.

CONCEPT DESIGN

Project : Design and preparation for the construction of the Eastern part of link 6 [Side Roads]

Project nr : 216-1380

Principal : Ministry of VROMI

Date drawn : 18-07-2017
Drawn by : BZ
Checked by : GT
Format : A2
Scale : 1:50/2000

Revision :

Subject : Typical cross sections & layout
Brimstone Hill road
Alternative A

Sheet number : C52A

File number : 216-1380-C52A

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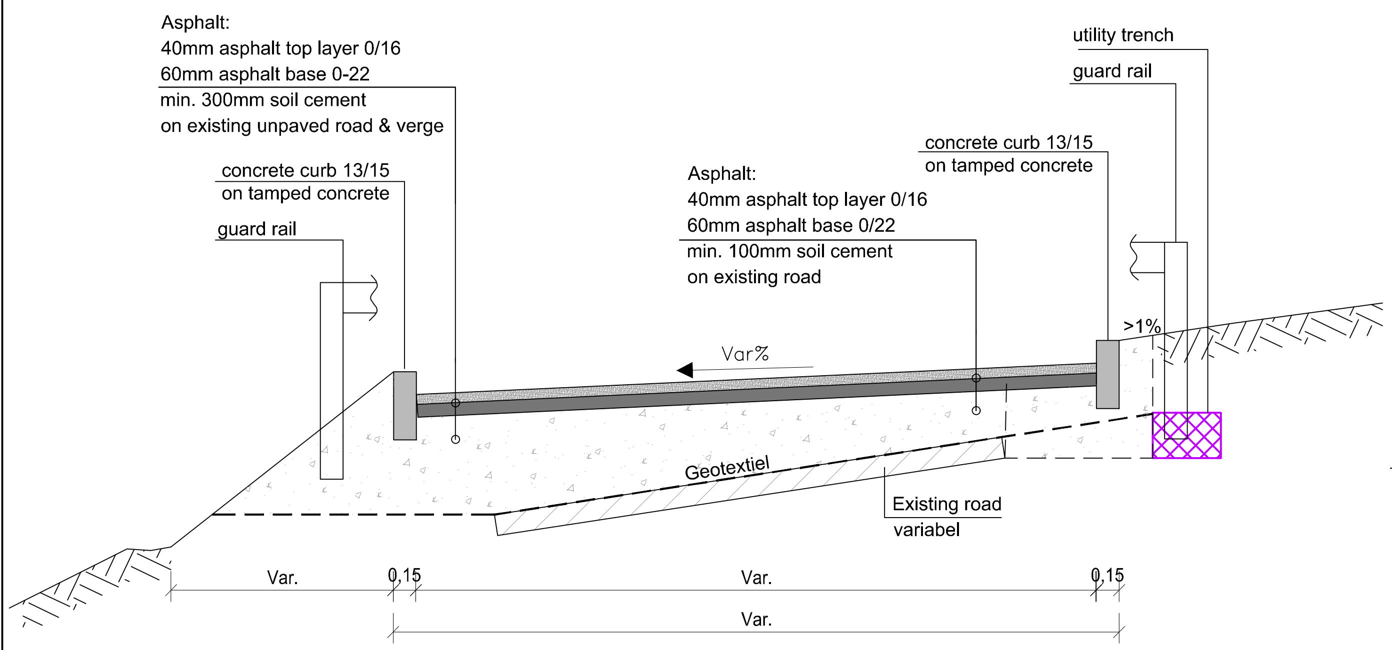
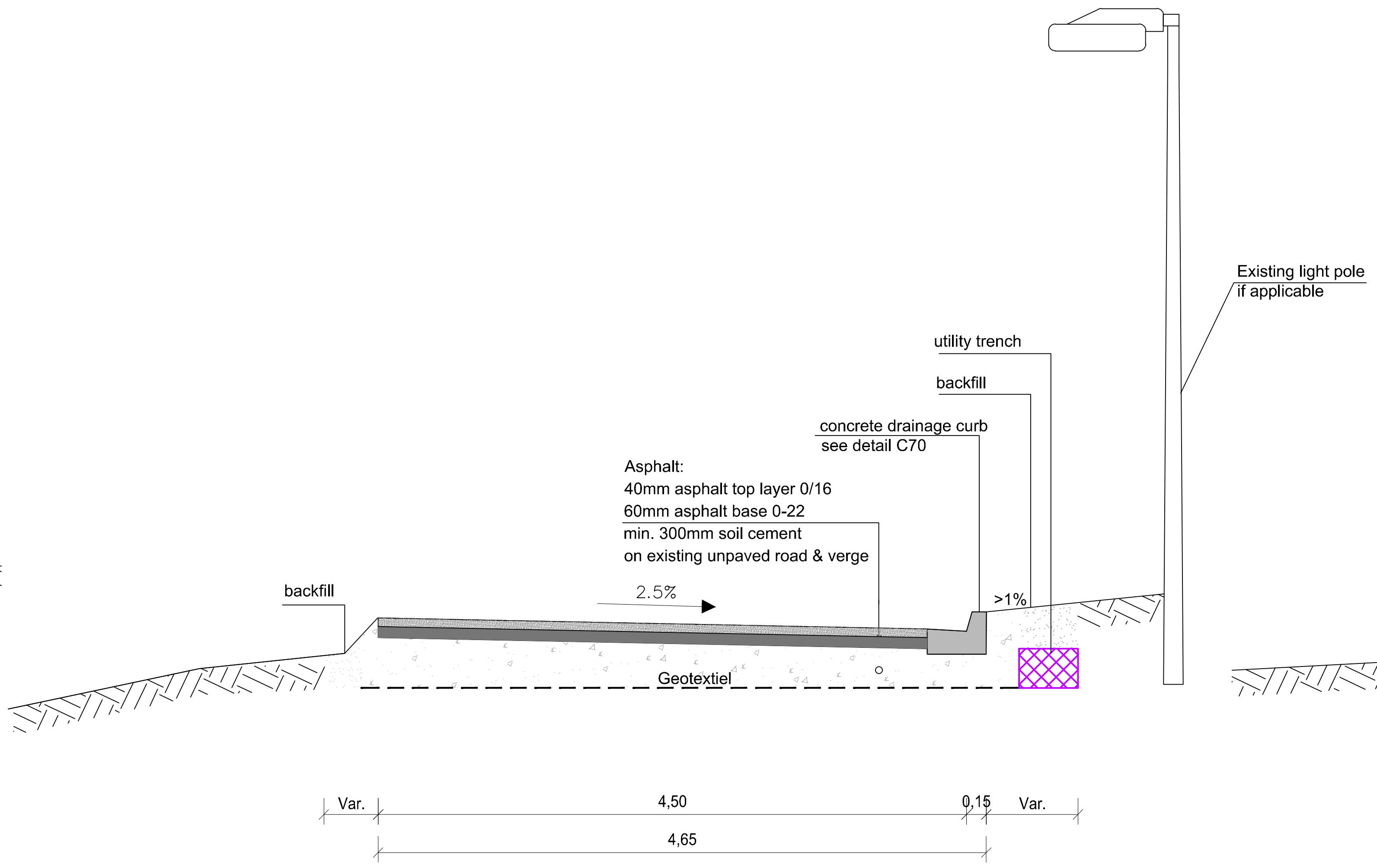
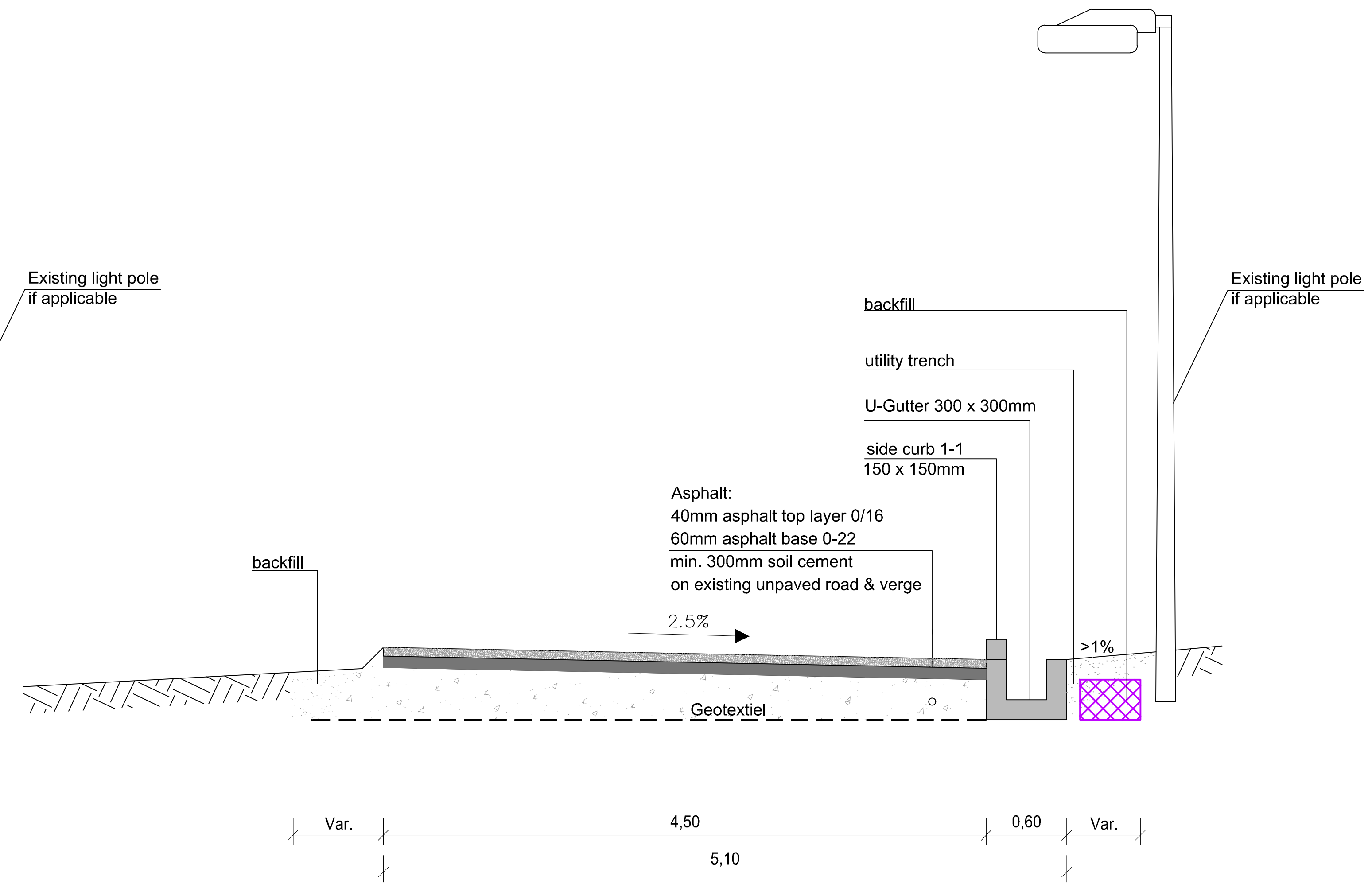
phone 5422421, fax 5422597, e-mail icesxm@sintmaarten.net



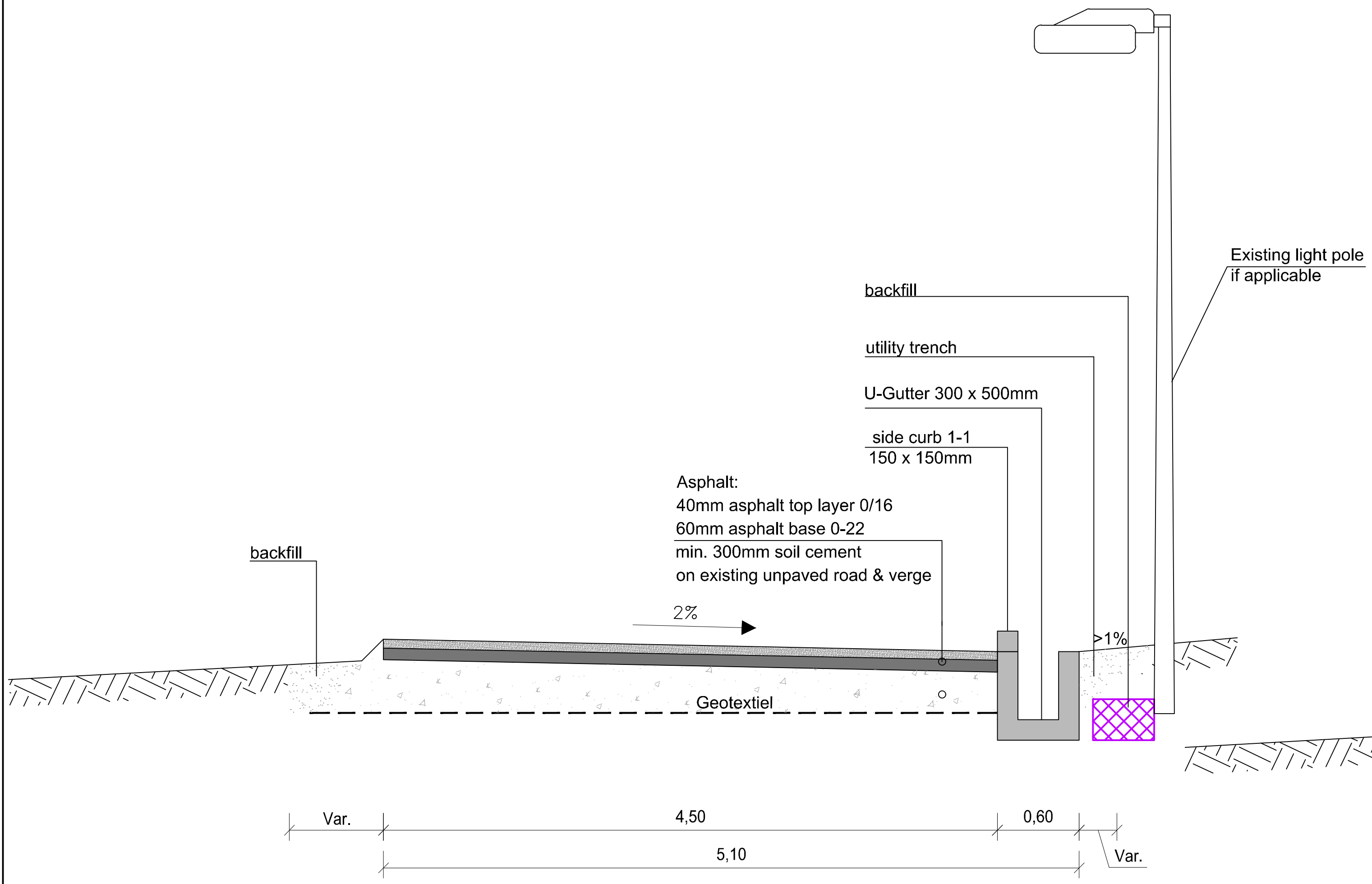
A1 typical cross section road (0-12m)
scale 1:50



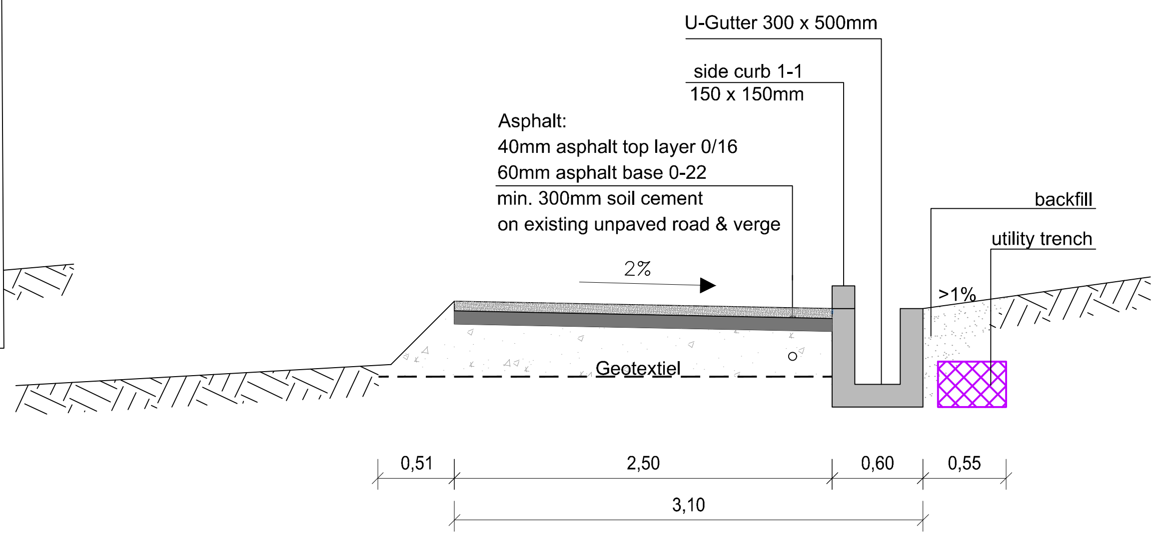
A2 typical cross section road (12-82m)
scale 1:50



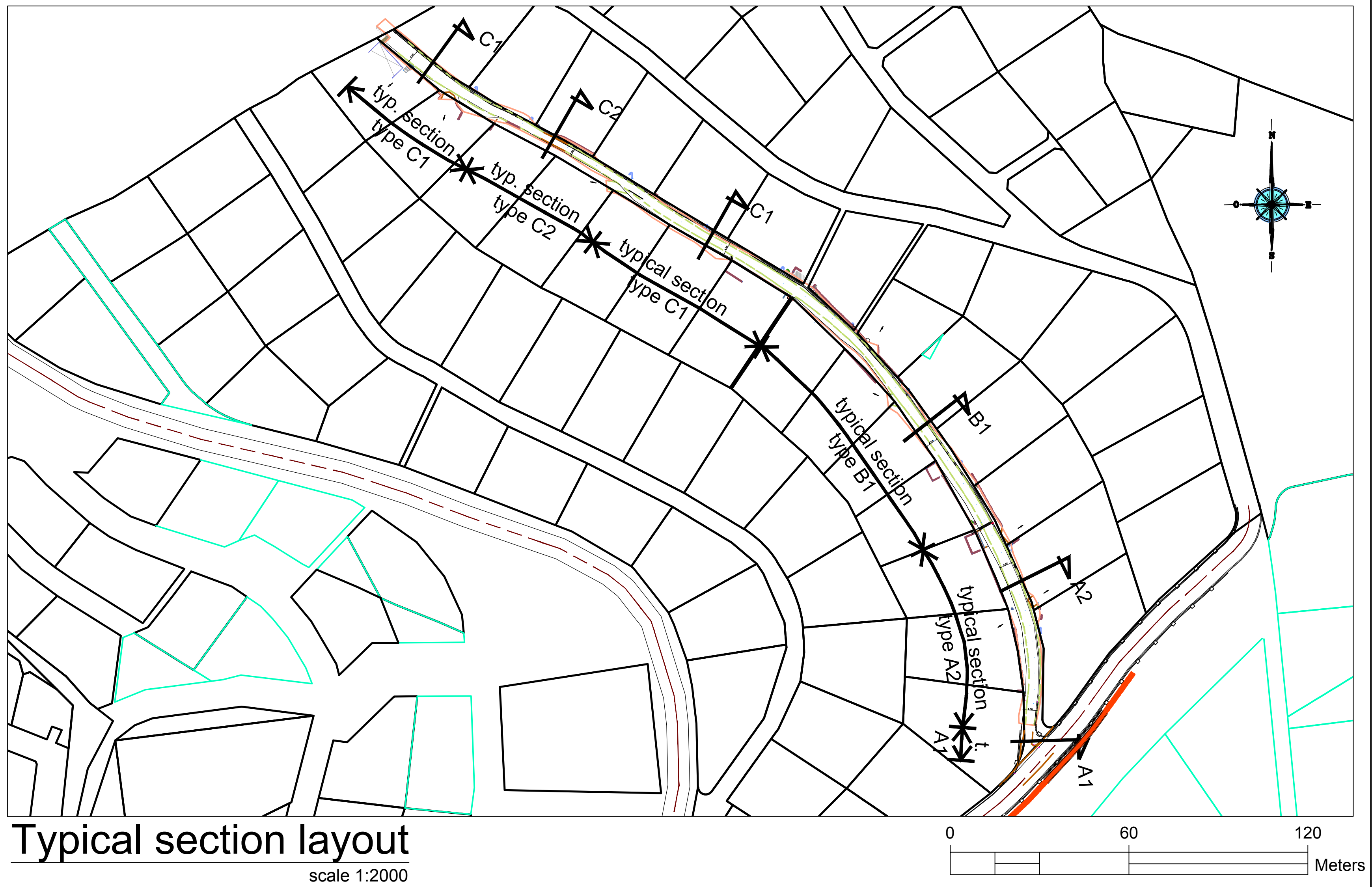
B1 typical cross section road (82-160m)
scale 1:50



C1 typical cross section road (160-245m & 300-346m)
scale 1:50



C2 typical cross section road (245-300m)
scale 1:50



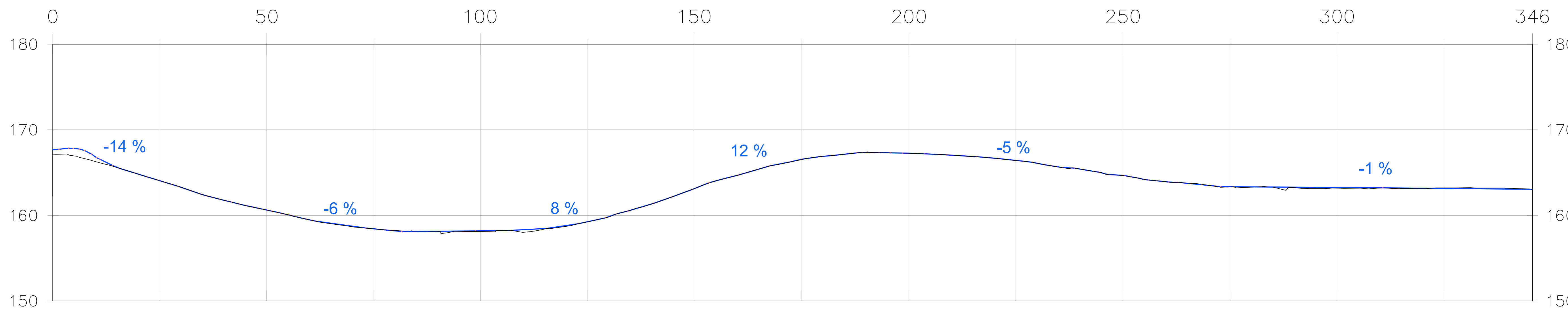
Typical section layout
scale 1:2000

Remarks:
- Existing features are based on survey drawings by Hunt's Topo Land.
- Dimensions in m, unless specified otherwise.
- Rebar in inches, rebar distance in mm.

CONCEPT DESIGN

Project : Design and preparation for the construction of the Eastern part of link 6 [Side Roads]		Project nr : 216-1380
Principal : Ministry of VROMI	Date drawn : 18-07-2017	Revision :
Subject : Typical cross sections & layout Brimstone Hill road Alternative B	Drawn by : BZ Checked by : GT Format : A2 Scale : 1:50/2000	
INDEPENDENT CONSULTING ENGINEERS. N.V.		Sheet number : C52B
		File number : 216-1380-C52B

p.o. box 390, zaegersgut road 13, philipsburg, st. maarten, n.a.
phone 5422421, fax 5422597, e-mail icesxm@sintmaarten.net



Existing level	0+000.00 167.16	0+025.00 164.05	0+050.00 160.62	0+075.00 158.43	0+100.00 158.11	0+125.00 159.26	0+150.00 163.15	0+175.00 166.54	0+200.00 167.25	0+225.00 166.41	0+250.00 164.66	0+275.00 163.31	0+300.00 163.18	0+325.00 163.22	0+345.65 163.05
Design level	0+000.00	0+025.00 164.05	0+050.00 160.62	0+075.00 158.43	0+100.00 158.19	0+125.00 159.27	0+150.00 163.15	0+175.00 166.57	0+200.00 167.25	0+225.00 166.42	0+250.00 164.66	0+275.00 163.35	0+300.00 163.26	0+325.00 163.15	0+345.65 163.05

Longitudinal profile road axis

CONCEPT DESIGN

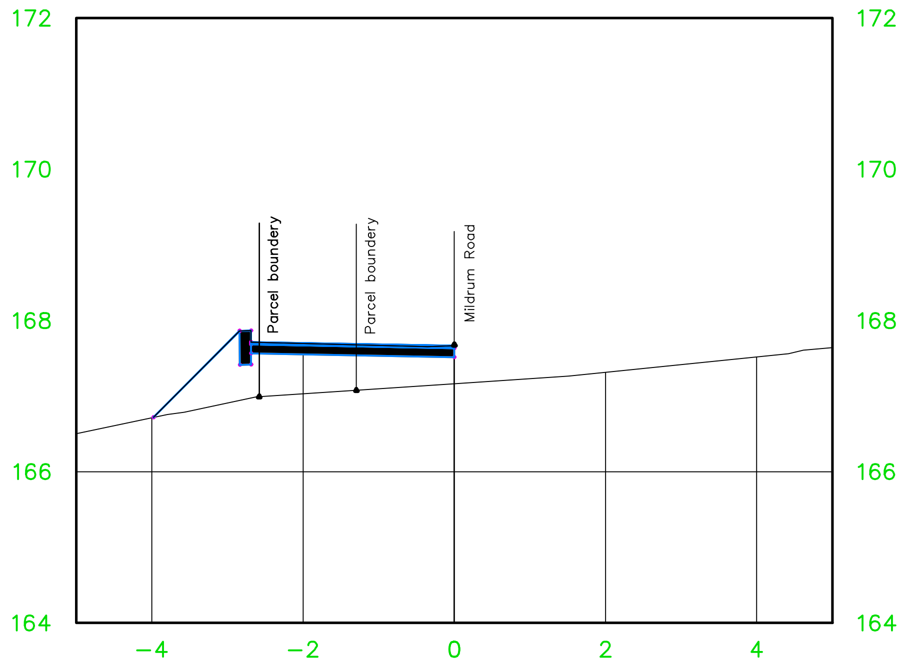
Remarks:
- Existing level
- Design level

Remarks:
- Existing features are based on survey drawings by Hunt's Topo Land.
- Dimensions in m, unless specified otherwise.
- Rebar in inches, rebar distance in mm.

- For cross sections location see C50
- For cross sections see C54 -C57

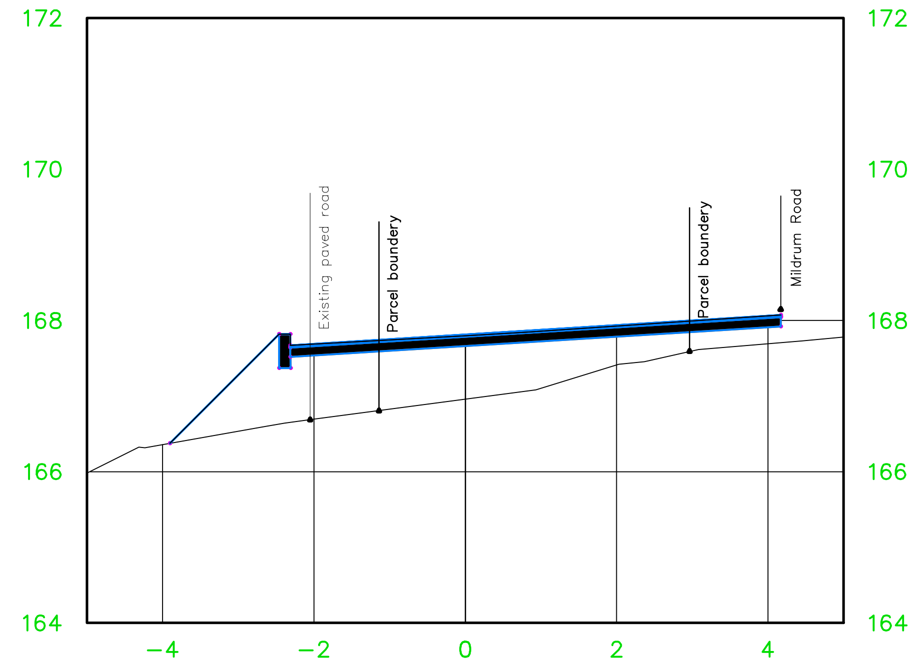
Project : Design and preparation for the construction of the Eastern part of link 6 [Side Roads]		Project nr : 216–1380
Principal : Ministry of VROMI	Date drawn : 18–07–2017	Revision :
Subject : Brimstone Hill road Longitudinal section	Drawn by : BZ	
	Checked by : GT	
	Format : Tabloid	
	Scale : 1:1000	
INDEPENDENT CONSULTING ENGINEERS. N.V. p.o. box 390, zaegersgut road 13, philipsburg, st. maarten, n.a. phone 5422421, fax 5422597, e-mail icesxm@sintmaarten.net		Sheet number : C53
		File number : 216–1380–C53

SL–1 (0.00m>



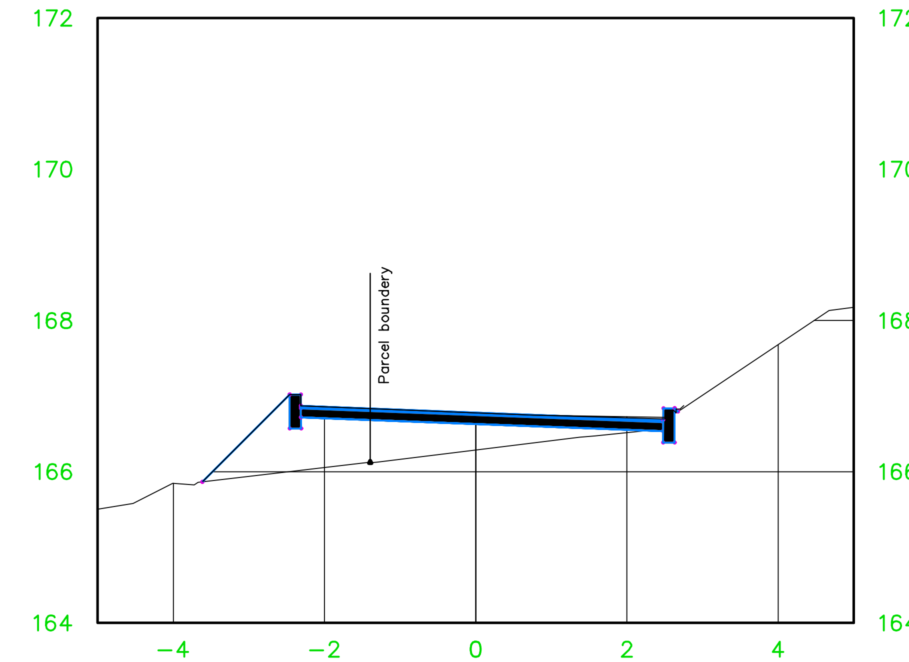
	-4	-2	0	2	4
Existing level	166.71	167.03	167.16	167.31	167.52
Design level		167.70	167.66		

SL–2 (5.00m>



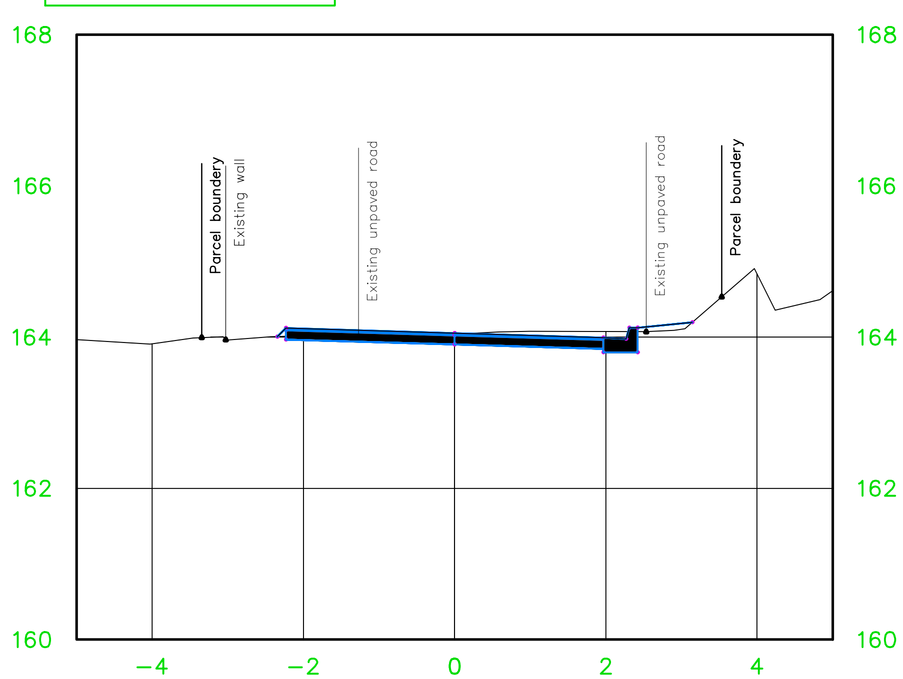
	-4	-2	0	2	4
Existing level	166.36	166.69	166.96	167.41	167.69
Design level		167.69	167.80	167.93	168.06

SL–3 (10.00m>



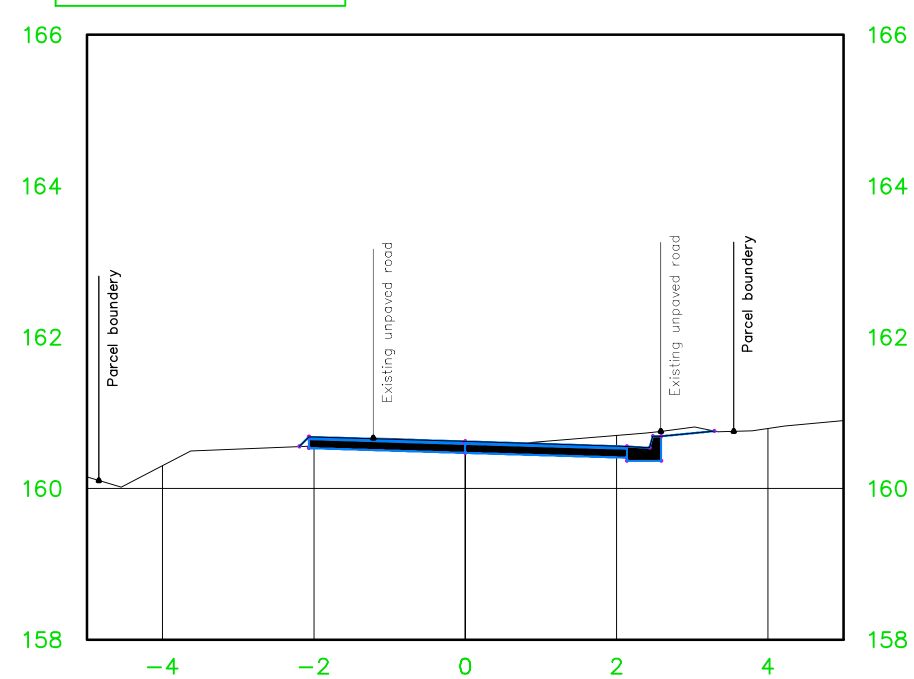
	-4	-2	0	2	4
Existing level	165.85	166.05	166.29	166.51	167.68
Design level		166.86	166.78	166.73	

SL–4 (25.00m>



	-4	-2	0	2	4
Existing level	163.91	164.02	164.05	164.07	164.84
Design level		164.11	164.05	163.99	

SL–5 (50.00m>



	-4	-2	0	2	4
Existing level	160.30	160.56	160.62	160.70	160.79
Design level		160.68	160.62	160.56	

Remarks:

- Existing features are based on survey drawings by Hunt's Topo Land.
- Dimensions in m, unless specified otherwise.
- Rebar in inches, rebar distance in mm.

- For cross section location see C51
- For longitudinal section see C53

CONCEPT DESIGN

Project : Design and preparation for the construction of the Eastern part of link 6 [Side Roads]

Project nr : 216–1380

Principal : Ministry of VROMI

Subject : Brimstone Hill road Sections SL–1 / SL–5

Date drawn : 18–07–2017
Drawn by : BZ
Checked by : GT
Format : Tabloid
Scale : 1:100

Revision :

Sheet number : C54

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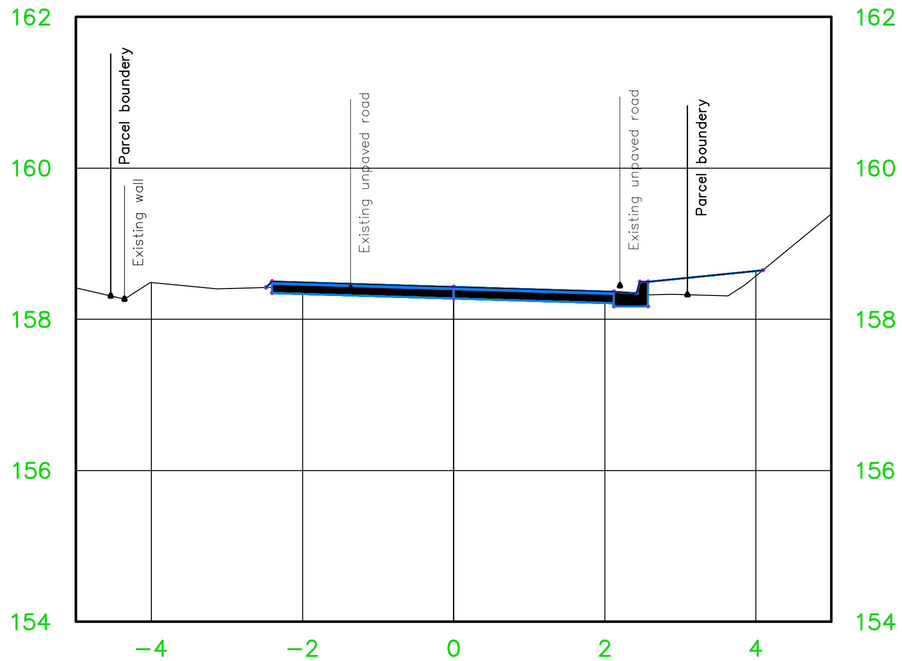
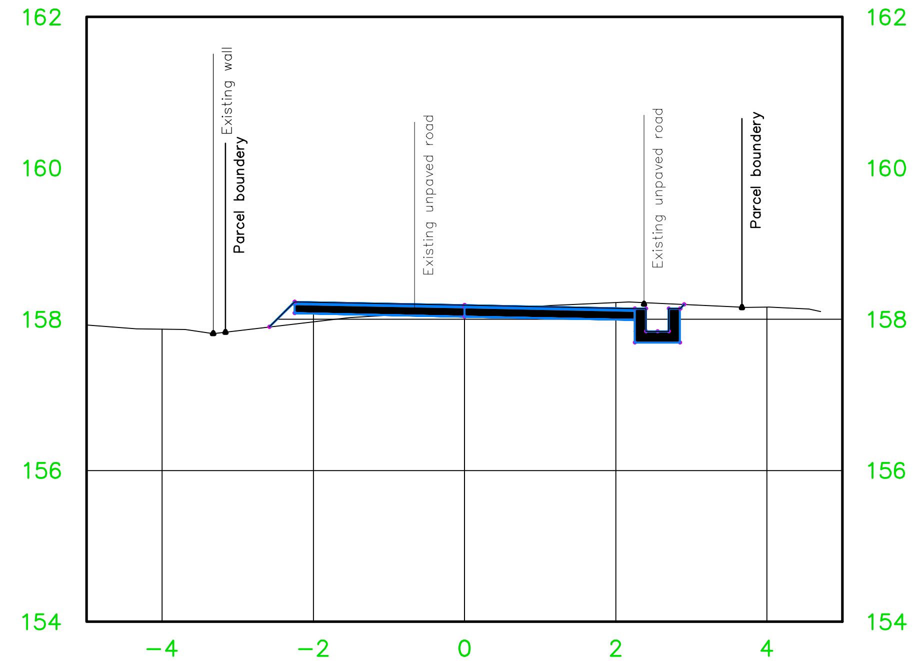
File number : 216–1380–C54

SL–6 (75.00m>



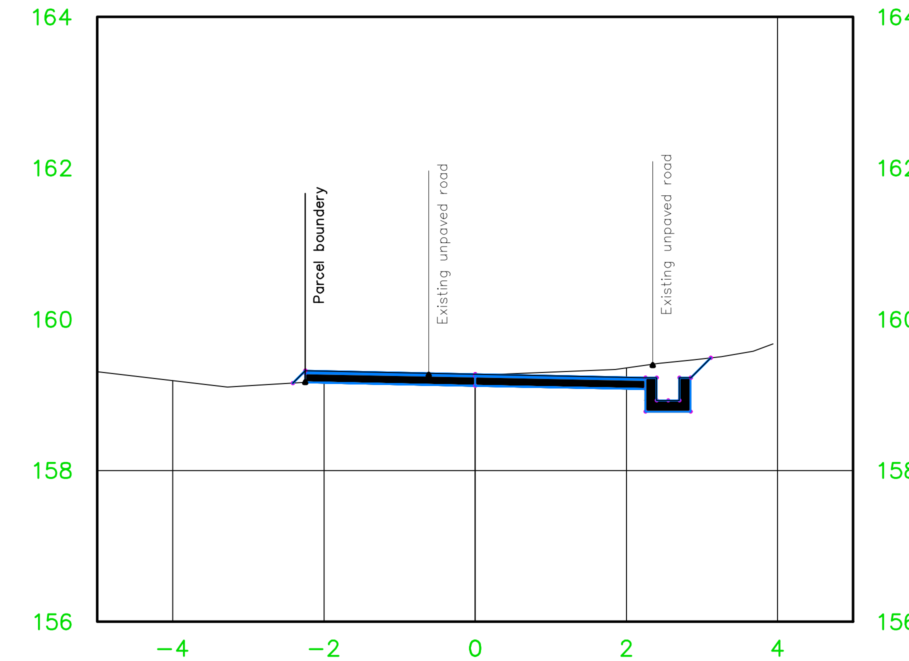
Existing level	158.49	158.42	158.43	158.33	158.57
Design level		158.49	158.43	158.37	158.64

SL–7 (100.00m>



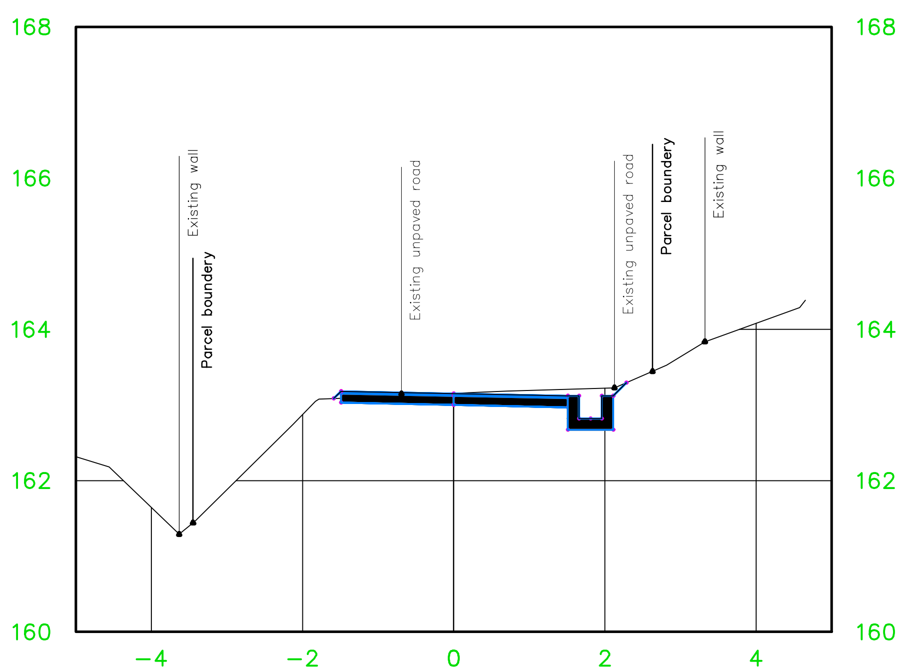
Existing level	157.87	157.97	158.11	158.22	158.16
Design level		158.23	158.19	158.15	

SL–8 (125.00m>



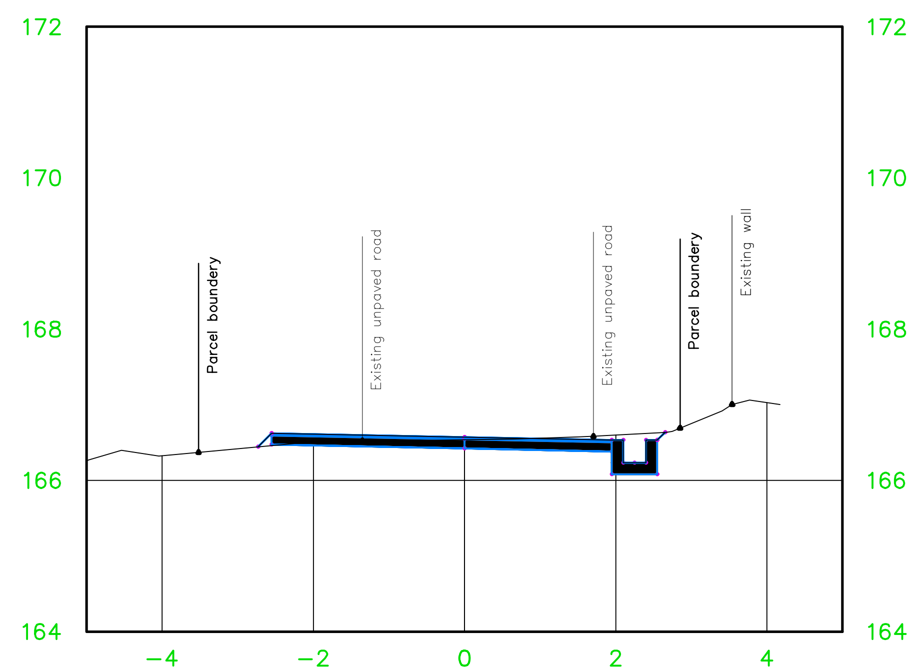
Existing level	159.19	159.18	159.26	159.36	
Design level		159.31	159.27	159.23	

SL–9 (150.00m>



Existing level	161.64	162.87	163.15	163.22	164.08
Design level			163.15	163.12	

SL–10 (175.00m>



Existing level	166.33	166.51	166.54	166.60	167.03
Design level		166.61	166.57	166.53	

Remarks:
- Existing features are based on survey drawings by Hunt's Topo Land.
- Dimensions in m, unless specified otherwise.
- Rebar in inches, rebar distance in mm.

- For cross section location see C51
- For longitudinal section see C53

CONCEPT DESIGN

Project : Design and preparation for the construction of the Eastern part of link 6 [Side Roads]		Project nr : 216–1380
Principal : Ministry of VROMI	Date drawn : 18–07–2017	Revision :
Subject : Brimstone Hill road Sections SL–6 / SL–10	Drawn by : BZ	
	Checked by : GT	
	Format : Tabloid	
	Scale : 1:100	
INDEPENDENT CONSULTING ENGINEERS. N.V.		Sheet number : C55
		File number : 216–1380–C55

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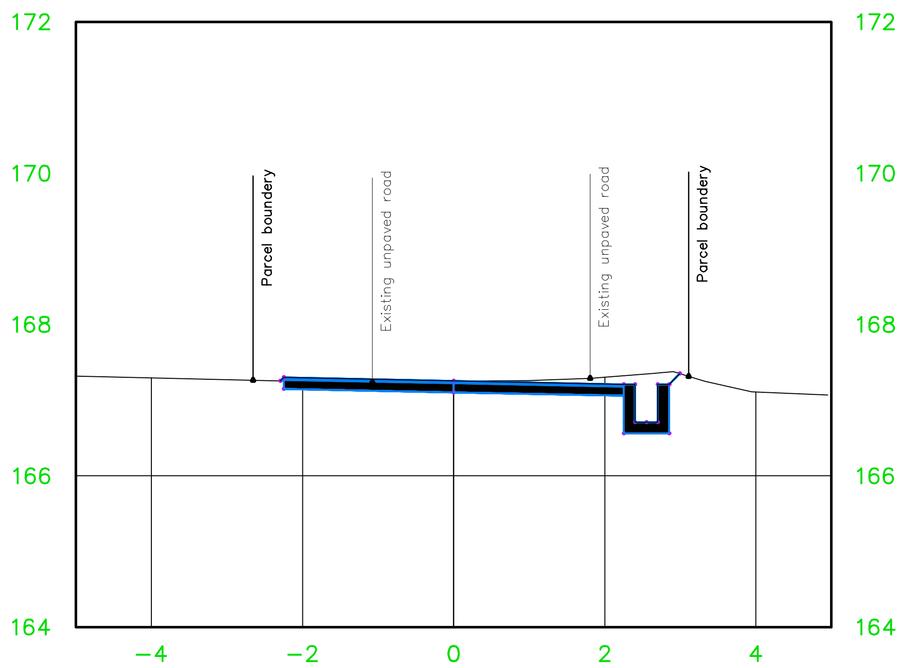
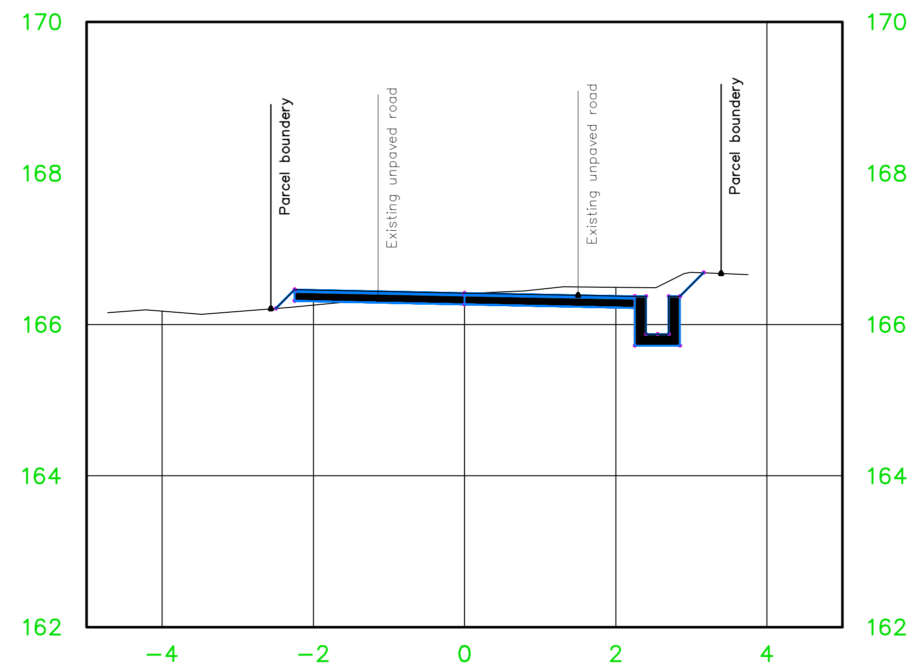
SL-11 (200.00m>



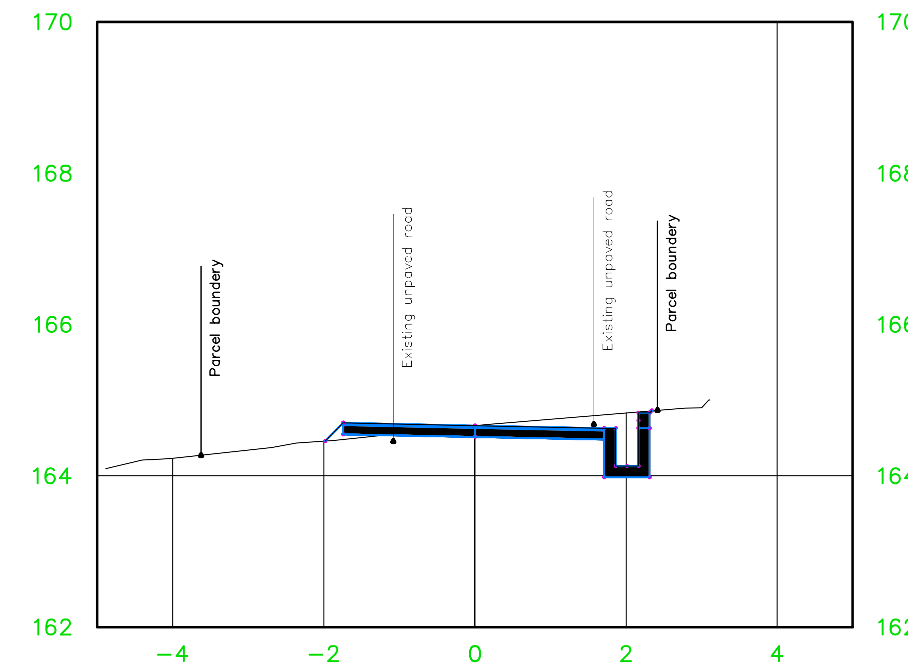
	-4	-2	0	2	4
Existing level	167.29	167.25	167.25	167.30	167.11
esign level		167.29	167.25	167.21	

SL-12 (225.00m>



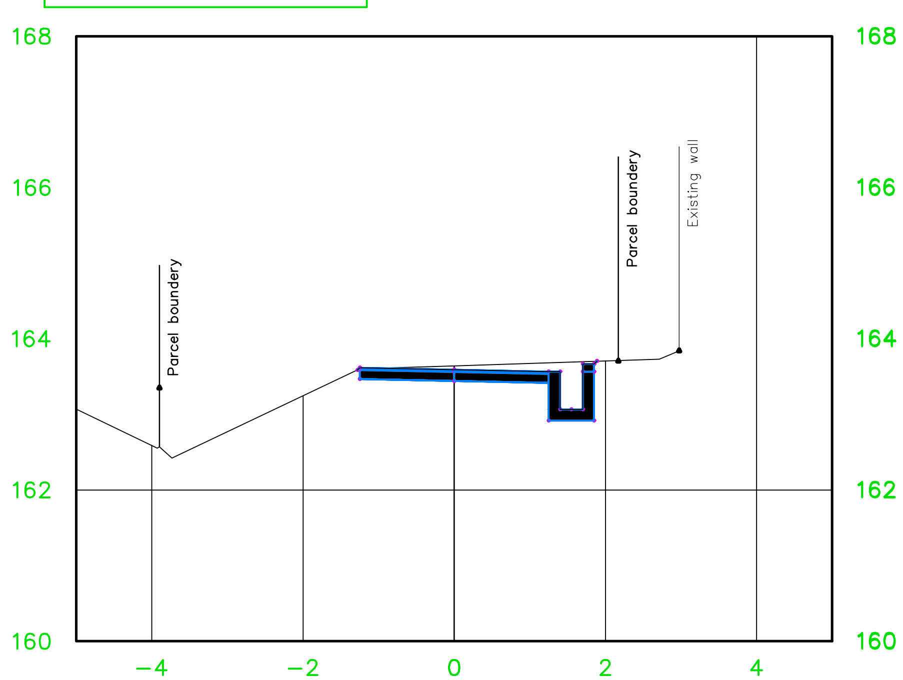
	-4	-2	0	2	4
Existing level	166.18	166.26	166.41	166.49	
Design level		166.46	166.42	166.38	

SL-13 (250.00m>



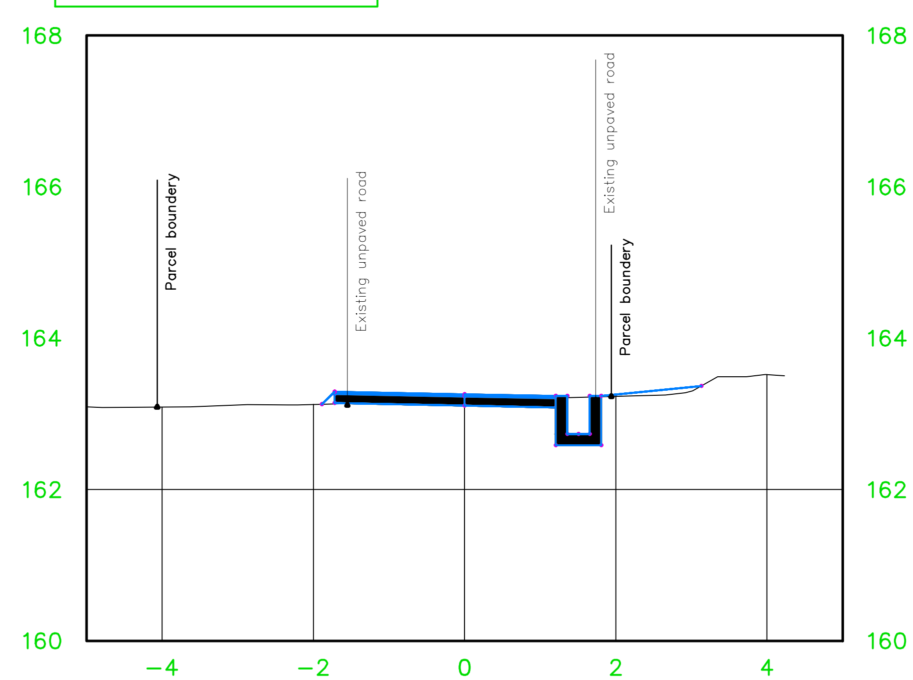
	-4	-2	0	2	4
Existing level	164.23	164.46	164.66	164.83	
Design level			164.66	164.13	

SL-14 (268.42m>



	-4	-2	0	2	4
Existing level	162.59	163.24	163.64	163.71	
Design level			163.59		

SL-15 (300.00m>



	-4	-2	0	2	4
Existing level	163.09	163.12	163.18	163.23	163.52
Design level			163.26	163.24	

Remarks:
- Existing features are based on survey drawings by Hunt's Topo Land.
- Dimensions in m, unless specified otherwise.
- Rebar in inches, rebar distance in mm.

- For cross section location see C51
- For longitudinal section see C53

CONCEPT DESIGN

Project : Design and preparation for the construction of the Eastern part of link 6 [Side Roads]		Project nr : 216-1380
Principal : Ministry of VROMI	Date drawn : 18-07-2017	Revision :
Subject : Brimstone Hill road Sections SL-11 / SL-15	Drawn by : BZ	
	Checked by : GT	
	Format : Tabloid	
	Scale : 1:100	
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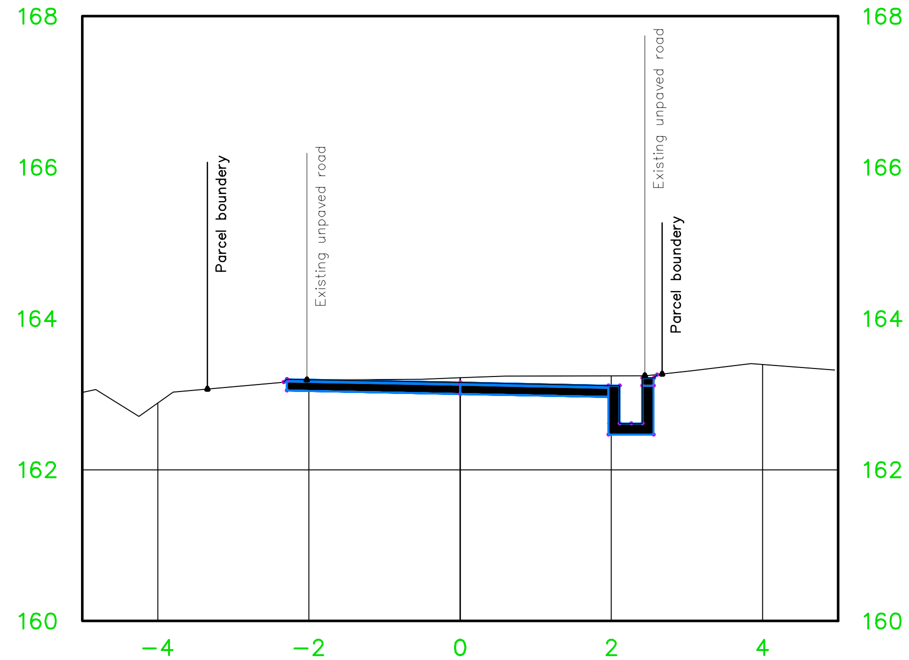
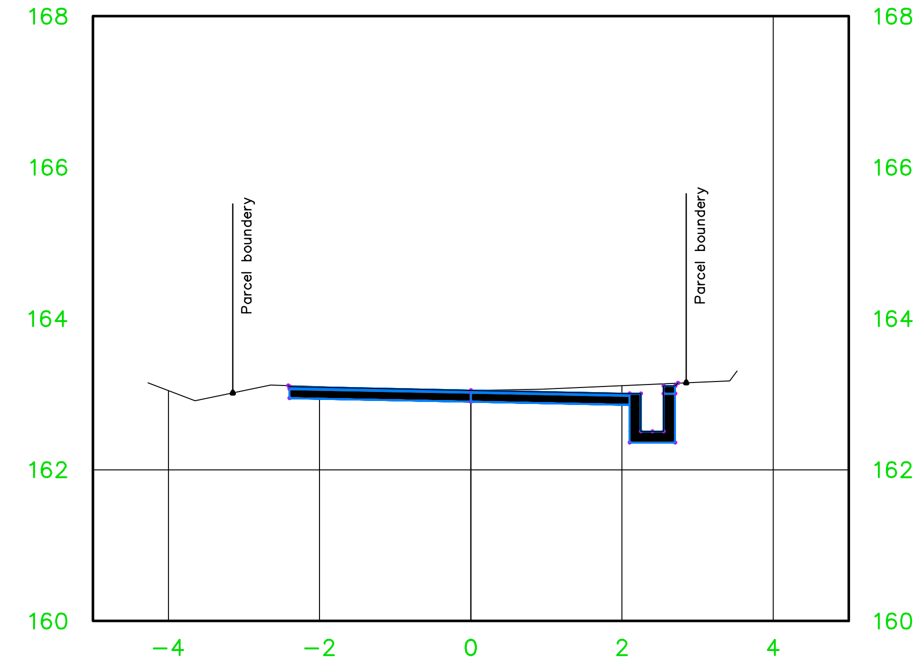
		File number : 216-1380-C56

SL-16 (325.00m>



Existing level	162.88	163.19	163.22	163.24	163.39
Design level		163.19	163.15	163.11	

SL-17 (345.65m>



Existing level	163.05	163.10	163.05	163.11	
Design level		163.09	163.05	163.01	

Remarks:

- Existing features are based on survey drawings by Hunt's Topo Land.
- Dimensions in m, unless specified otherwise.
- Rebar in inches, rebar distance in mm.

- For cross section location see C51
- For longitudinal section see C53

CONCEPT DESIGN

Project :	Design and preparation for the construction of the Eastern part of link 6 [Side Roads]		Project nr : 216-1380
Principal :	Ministry of VROMI	Date drawn : 18-07-2017	Revision :
Subject :	Brimstone Hill road Sections SL-16 / SL-17	Drawn by : BZ	
		Checked by : GT	
		Format : Tabloid	
		Scale : 1:100	
			Sheet number : C57
			File number : 216-1380-C57

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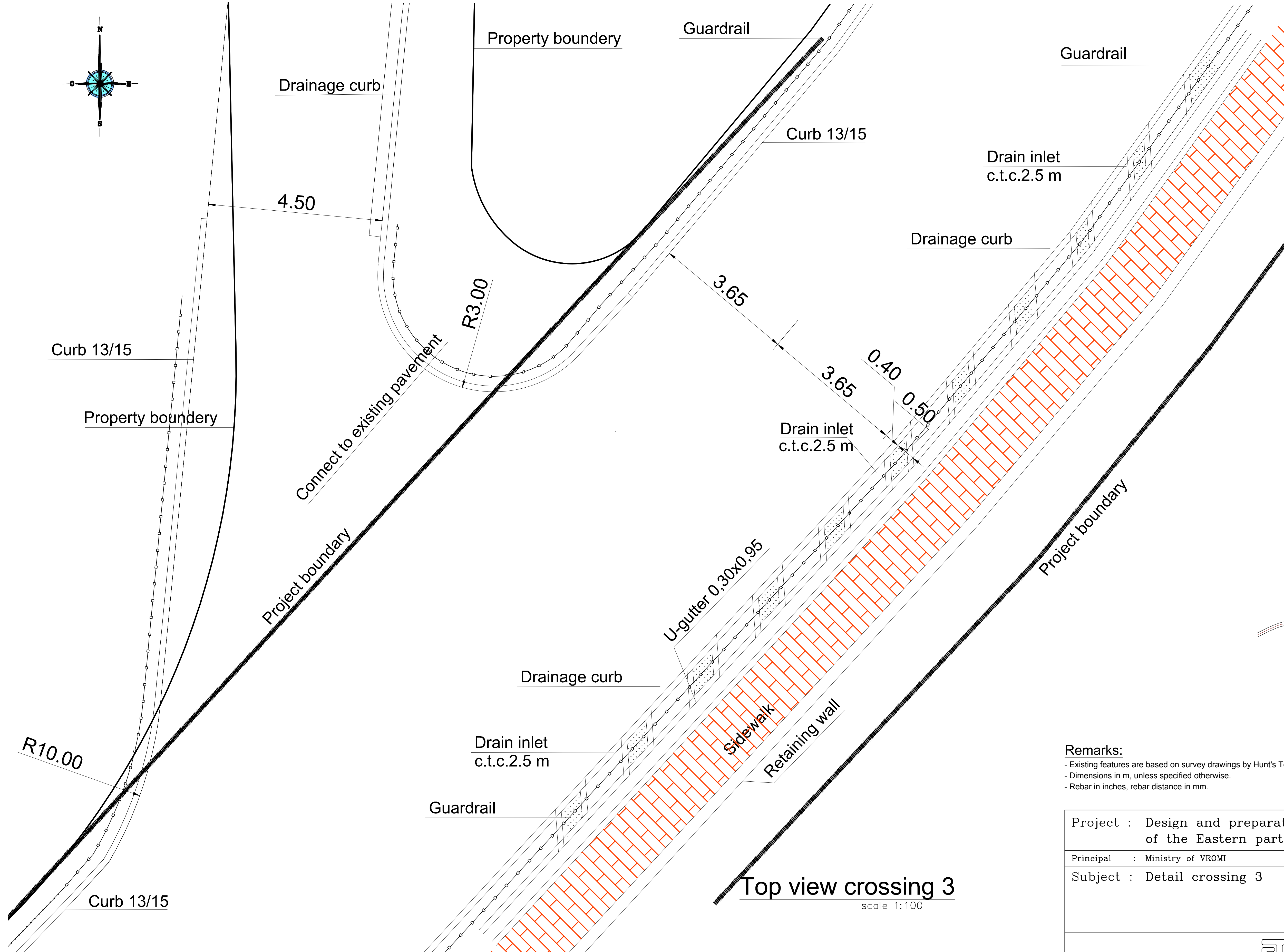
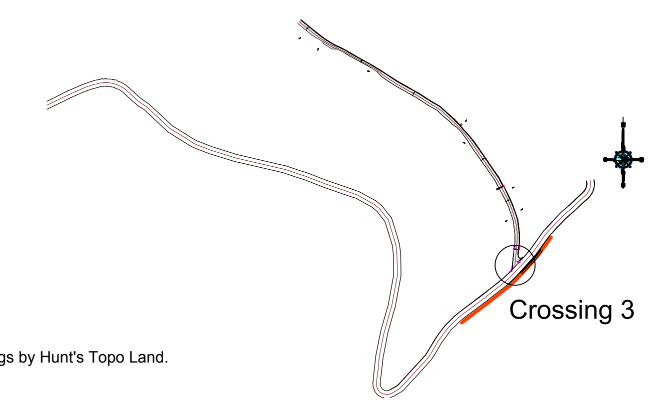
phone 5422421, fax 5422597, e-mail icesxm@sintmaarten.net





Top view crossing 3

scale 1:100

Remarks:

- Existing features are based on survey drawings by Hunt's Topo Land.
- Dimensions in m, unless specified otherwise.
- Rebar in inches, rebar distance in mm.

CONCEPT DESIGN

Project : Design and preparation for the construction of the Eastern part of link 6 [Side Roads]			Project nr : 216–1380
Principal : Ministry of VROMI	Date drawn : 18–07–2017	Revision :	
Subject : Detail crossing 3	Drawn by : BZ		
	Checked by : GT		
	Format : Tabloid		
	Scale : 1:100		
INDEPENDENT CONSULTING ENGINEERS. N.V.			Sheet number : C58
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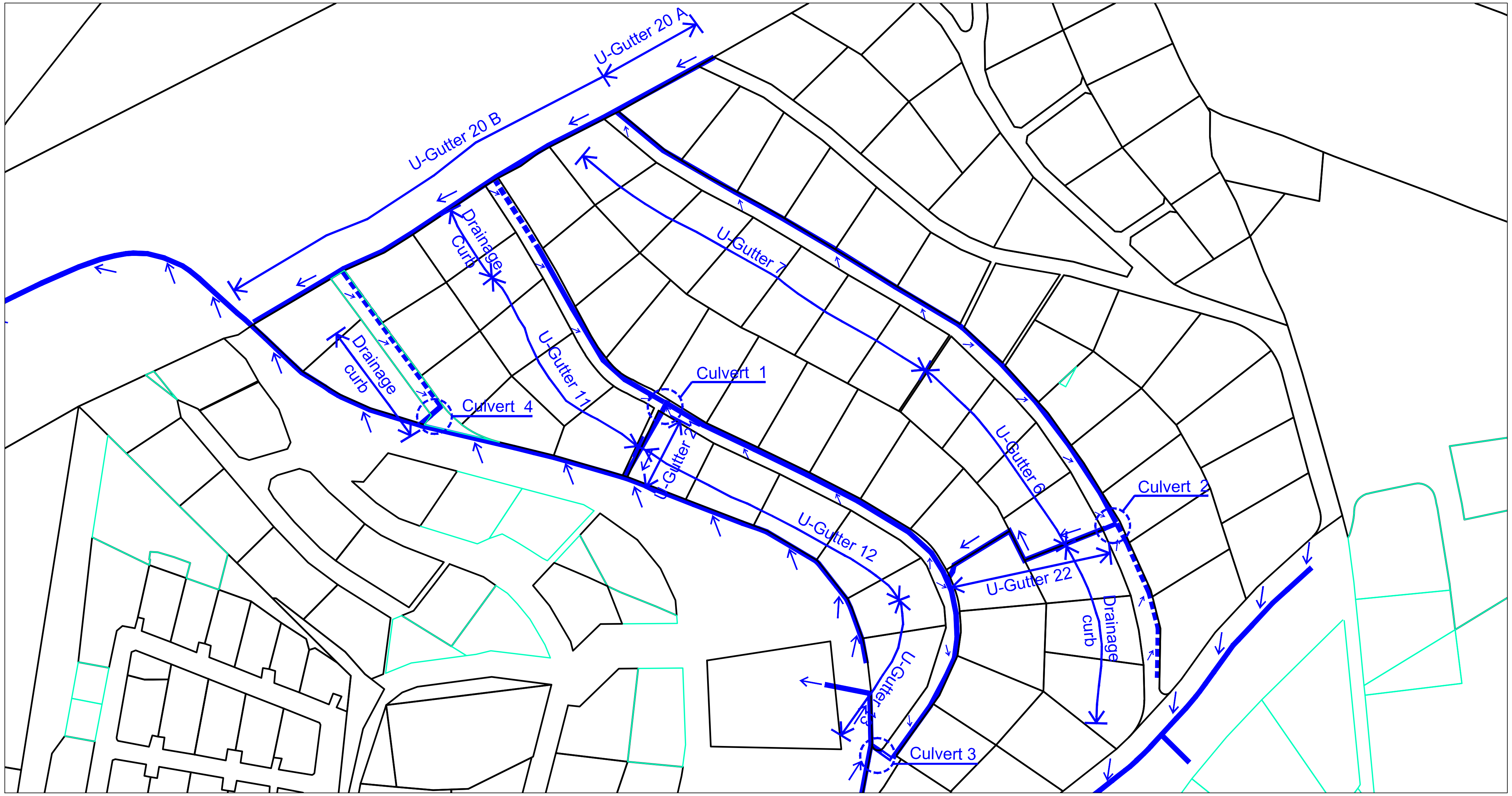
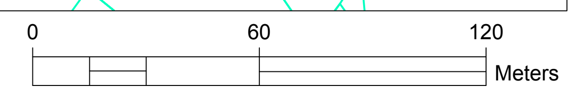
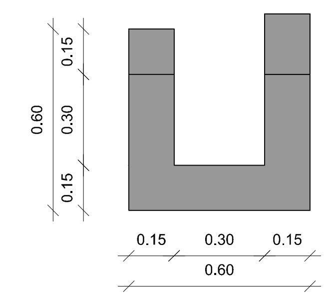


Drainage layout

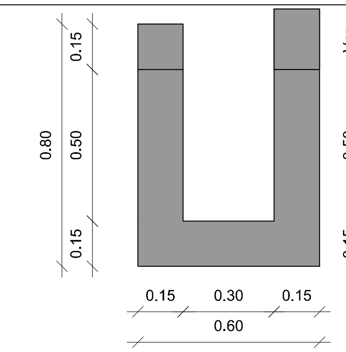
scale 1:2000





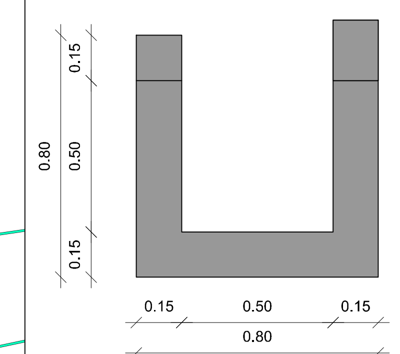
U-Gutter 6

300 x 300 mm
scale 1:25



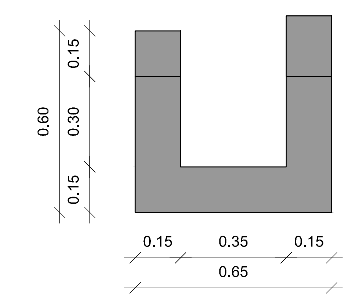
U-Gutter 7

300 x 500 mm
scale 1:25



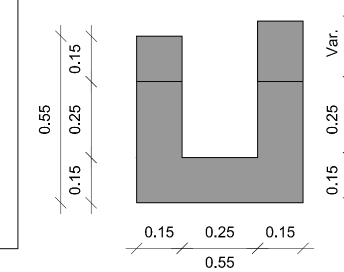
U-Gutter 12

500 x 500 mm
scale 1:25



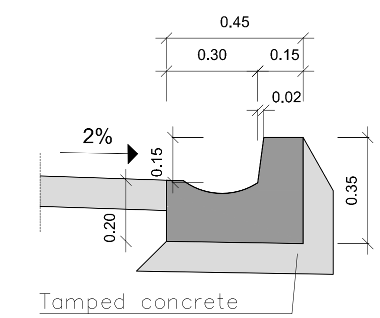
U-Gutter 13

350 x 300 mm
scale 1:25



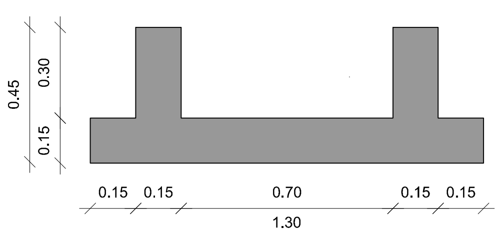
U-Gutter 11

250 x 250 mm
scale 1:25



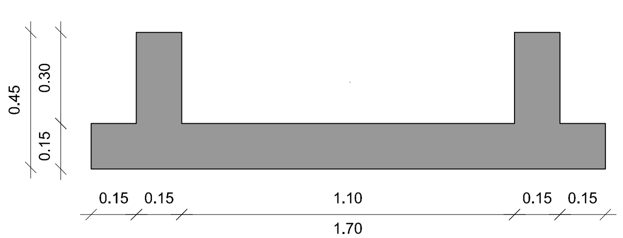
Detail: Drainage Curb

scale 1:25



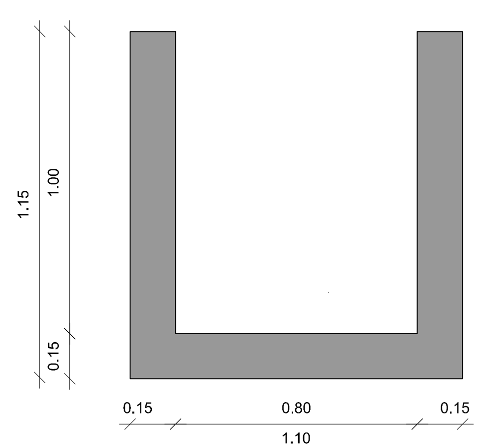
U-Gutter 20 A

700 x 300 mm
scale 1:25



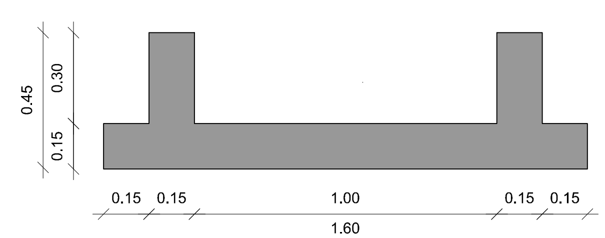
U-Gutter 20 B

1100 x 300 mm
scale 1:25



U-Gutter 21

800 x 1000 mm
scale 1:25



U-Gutter 22

1000 x 300 mm
scale 1:25

Remarks:

- Existing features are based on survey drawings by Hunt's Topo Land.
- Dimensions in m, unless specified otherwise.
- Rebar in inches, rebar distance in mm.
- For other drain details see C71

- For U-Gutter 20 see C72/C73
- For U-Gutter 21 see C74/75
- For U-Gutter 22 see C76/C77

- For Culvert 1 see C78
- For Culvert 2 see C79
- For Culvert 3 see C80
- For Culvert 4 see C81

Legend

- Drainage flow direction
- Concrete drainage u-gutter
- U-gutter with grating
- Drainage curb

CONCEPT DESIGN

Project : Design and preparation for the construction of the Eastern part of link 6	Project nr : 216–1380
Principal : Ministry of VROMI	Date drawn : 11–07–2017
Subject : Drainage plan	Drawn by : BZ
	Checked by : GT
	Format : Tabloid
	Scale : 1:2000/25
Revision :	Sheet number : C70
	File number : 216–1380–C70

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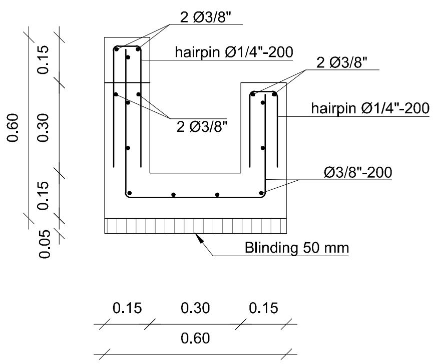
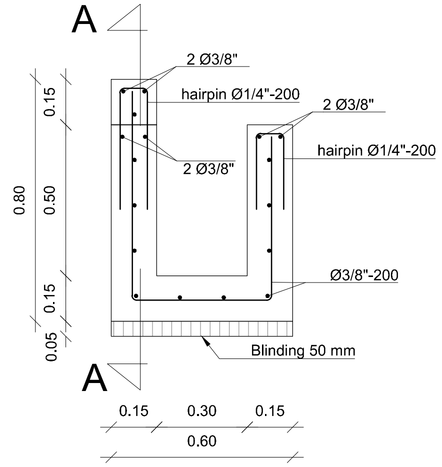
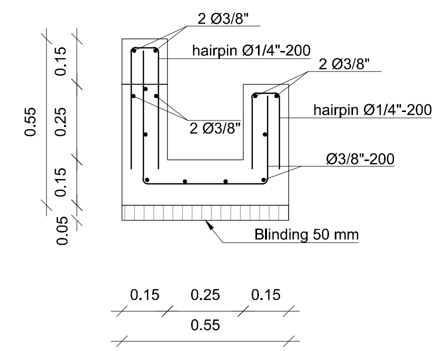


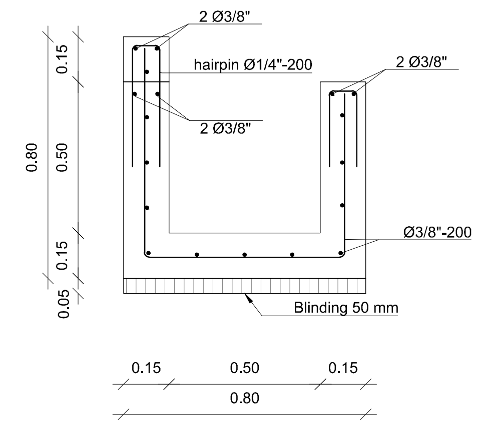
U-Gutter 6
Typical reinforcement
scale 1:25



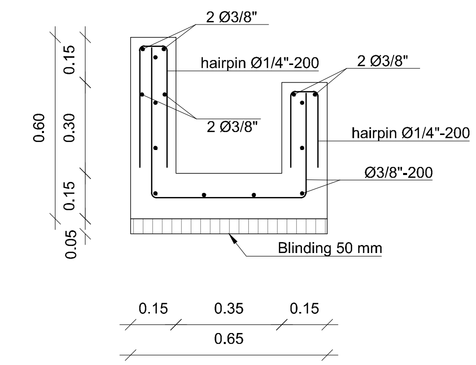
U-Gutter 7
Typical reinforcement
scale 1:25



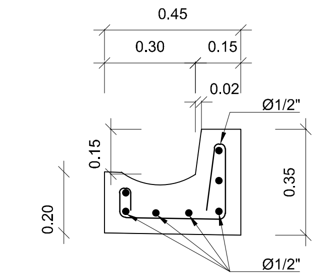
U-Gutter 11
Typical reinforcement
scale 1:25



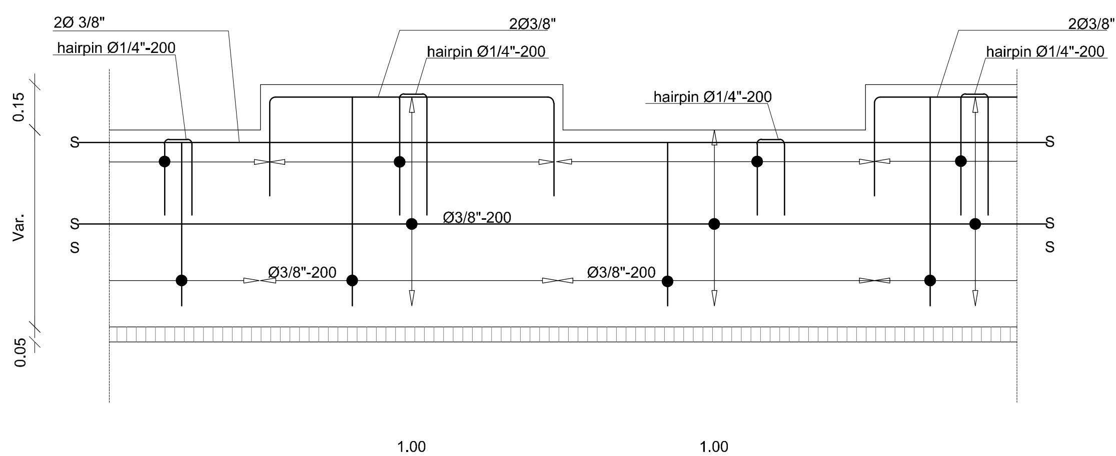
U-Gutter 12
Typical reinforcement
scale 1:25



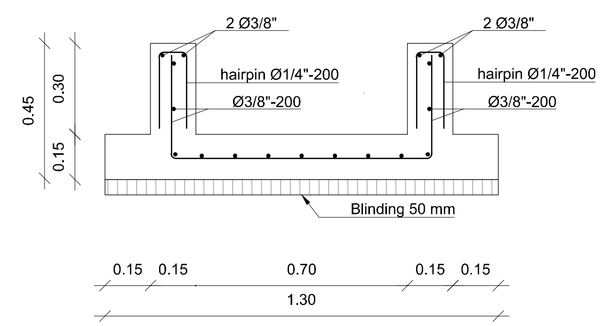
U-Gutter 13
Typical reinforcement
scale 1:25



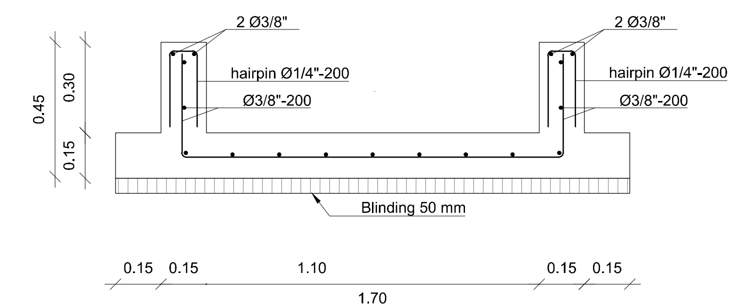
Detail: Drainage Curb
scale 1:25



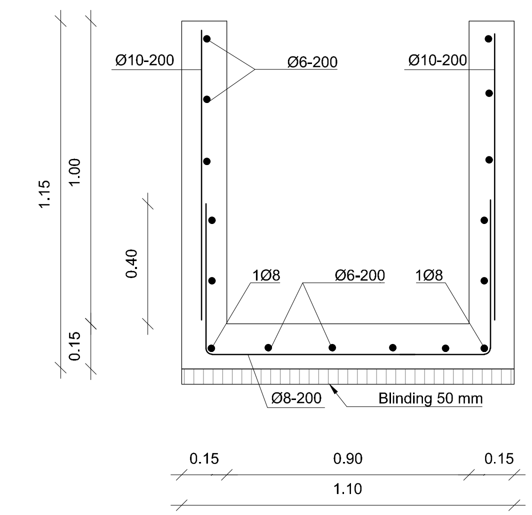
Section A-A
Typical U-Gutter & Curb reinforcement
scale 1:25



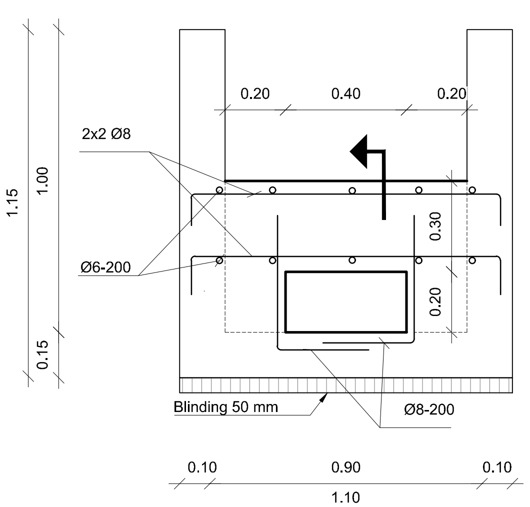
U-Gutter 20A
Typical reinforcement
scale 1:25



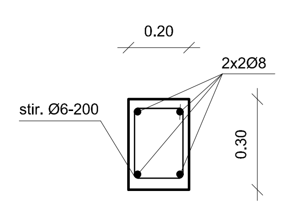
U-Gutter 20B
Typical reinforcement
scale 1:25



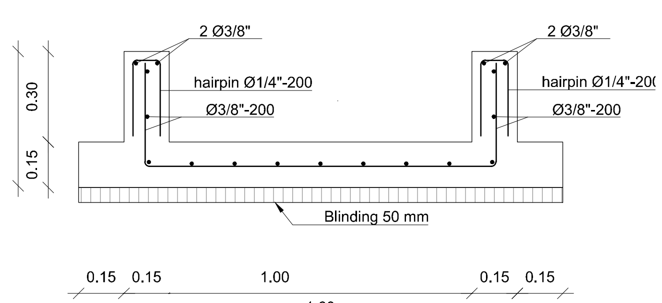
U-Gutter 21
Typical reinforcement
scale 1:25



U-Gutter 21- Weir
Typical reinforcement
scale 1:25



Cross section beam
scale 1:25



U-Gutter 22
Typical reinforcement
scale 1:25

Concrete Quality	C30/37
Reinforcement Quality	Grade 60
Traffic load	Class 30
Cover	40 mm

Remarks:
- Existing features are based on survey drawings by Hunt's Topo Land.
- Dimensions in m, unless specified otherwise.
- Rebar in inches, rebar distance in mm.

CONCEPT DESIGN

Project :	Design and preparation for the construction of the Eastern part of link 6		Project nr : 216-1380
Principal :	Ministry of VROMI	Date drawn : 18-07-2017	Revision :
Subject :	Drainage details Reinforcement & dimensions U-gutters with curb & drainage curb	Drawn by : BZ Checked by : GT Format : Tabloid Scale : 1:25	
			Sheet number : C71
			File number : 216-1380-C71

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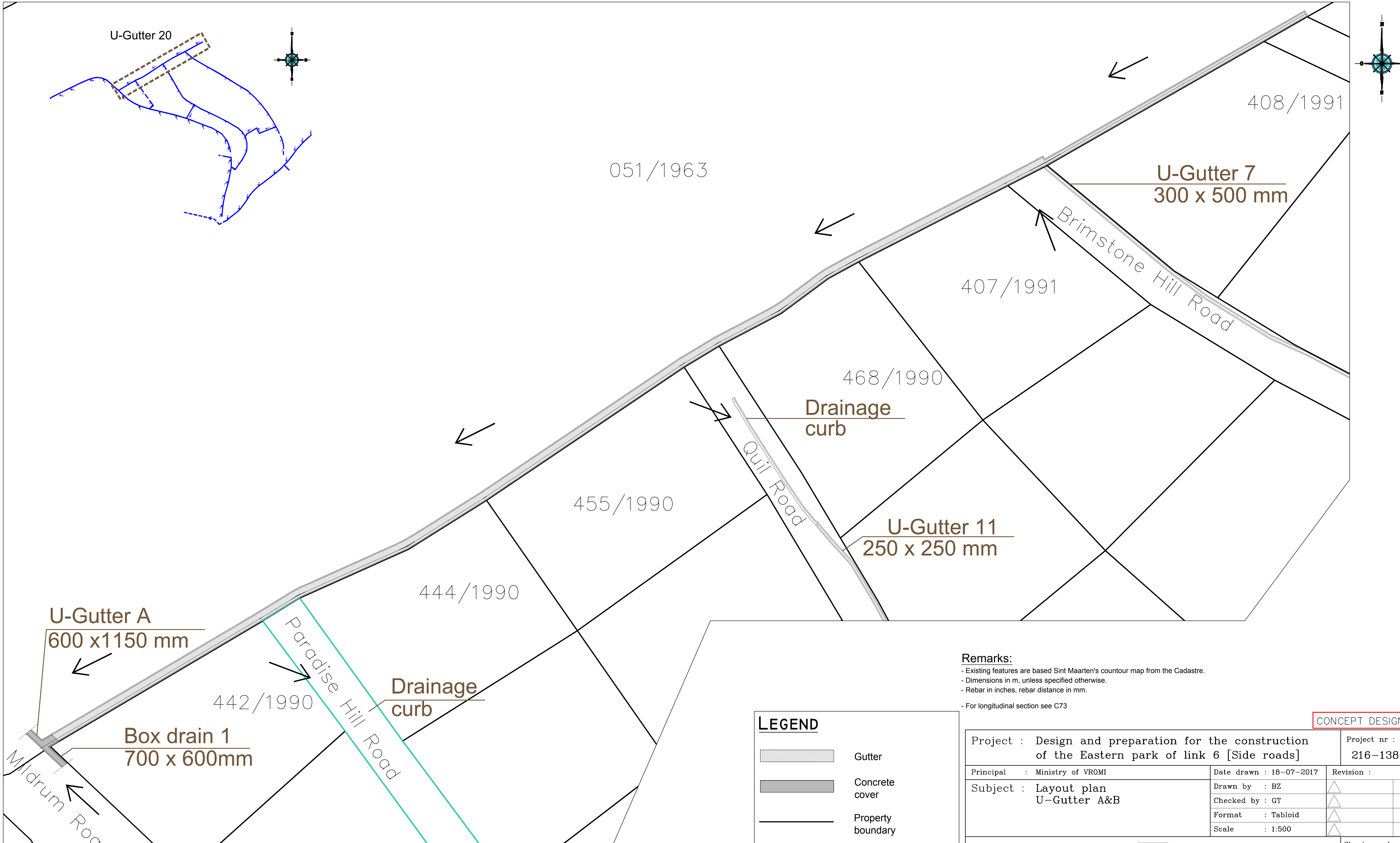
phone 5422421, fax 5422597, e-mail icesxm@sintmaarten.net



Layout plan U-Gutter 20A & B

scale 1:500

Remarks:
- Existing features are based Sint Maarten's countour map from the Cadastre.
- Dimensions in m, unless specified otherwise.
- Rebar in inches, rebar distance in mm.

- For longitudinal section see C73

Legend

- Gutter
- Concrete cover
- Property boundary
- Alignment
- Drainage flow direction

CONCEPT DESIGN

Project : Design and preparation for the construction of the Eastern park of link 6 [Side roads]		Project nr : 216–1380
Principal : Ministry of VROMI	Date drawn : 18–07–2017	Revision :
Subject : Layout plan U–Gutter A&B	Drawn by : BZ Checked by : GT Format : Tabloid Scale : 1:500	
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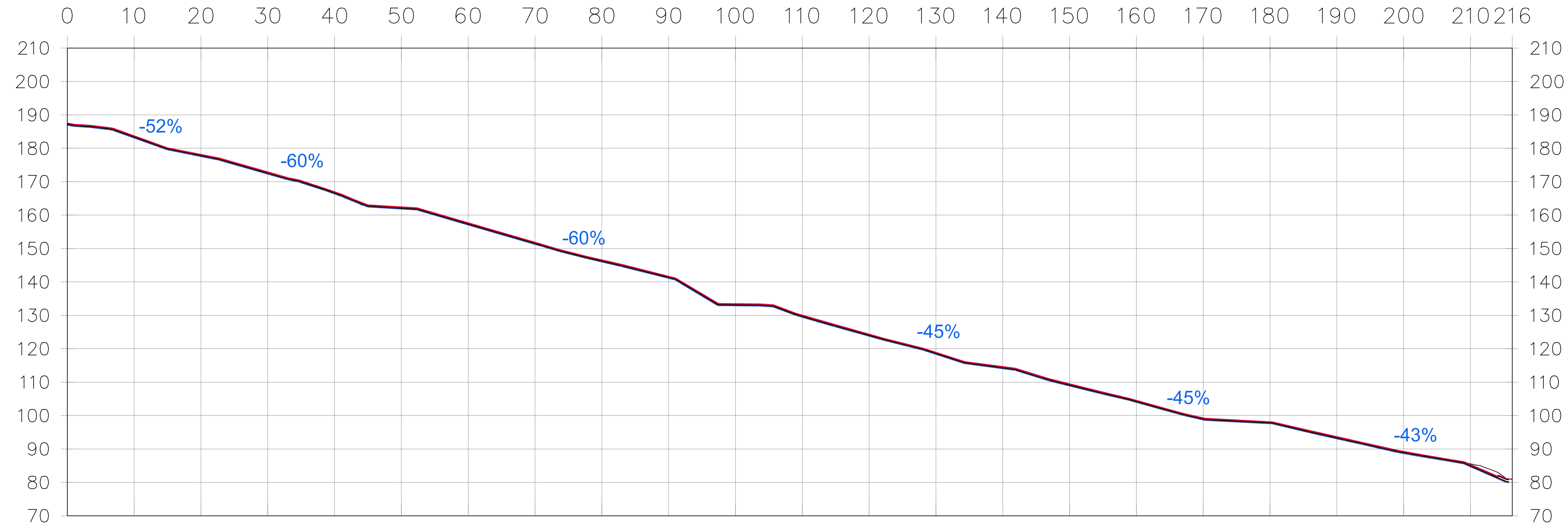

Chainage	Existing level	Design level
0+000.00	187.42	187.01
0+010.00	183.56	183.15
0+020.00	177.99	177.59
0+030.00	172.80	172.33
0+040.00	166.56	166.40
0+050.00	162.30	161.88
0+060.00	157.52	157.10
0+070.00	151.70	151.30
0+080.00	146.39	146.04
0+090.00	141.58	141.12
0+100.00	133.37	132.91
0+110.00	129.84	129.47
0+120.00	124.19	123.79
0+130.00	118.73	118.37
0+140.00	114.62	114.09
0+150.00	109.18	108.91
0+160.00	104.38	103.98
0+170.00	99.11	98.72
0+180.00	98.03	97.60
0+190.00	93.52	93.12
0+200.00	88.97	88.69
0+210.00	85.58	84.75
0+216.25	80.99	

Remarks:
- Existing level
- Design level
- U-Gutter Top Level

Remarks:
- Existing features are based Sint Maarten's countour map from the Cadastre.
- Dimensions in m, unless specified otherwise.
- Rebar in inches, rebar distance in mm.

CONCEPT DESIGN

Project : Design and preparation for the construction of the Eastern part of link 6 [Side Roads]

Project nr : 216–1380

Principal : Ministry of VROMI

Subject : U–Gutter 20 Longitudinal section

Date drawn : 18–07–2017
Drawn by : BZ
Checked by : GT
Format : Tabloid
Scale : 1:750

Revision :

Sheet number : C73

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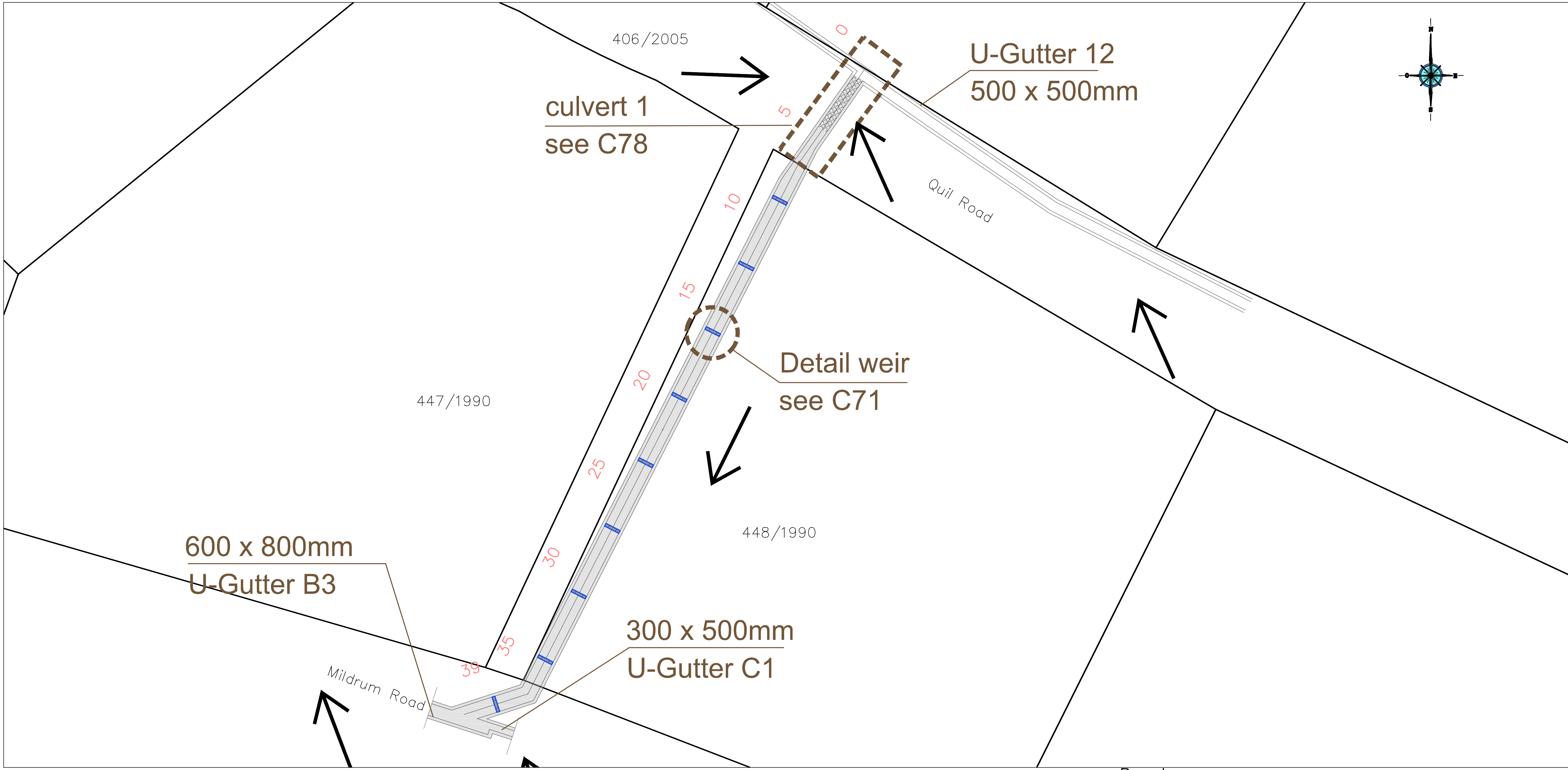
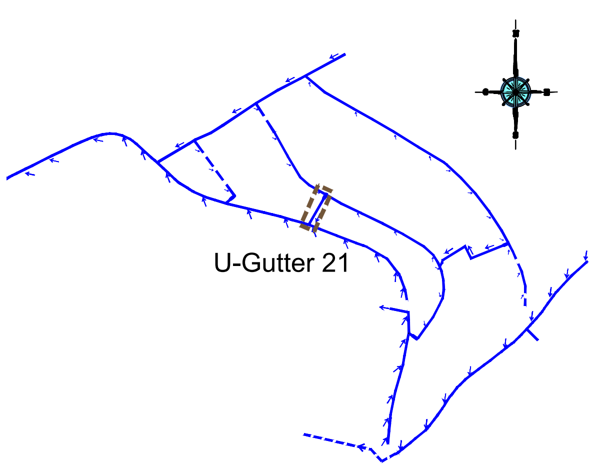
File number : 216–1380–C73



Layout plan U-Gutter 21

scale 1:200



LEGEND

- Gutter
- Weir
- Grate
- Property boundary
- Alignment
- Drainage flow direction

Remarks:

- Existing features are based Sint Maarten's countour map from the Cadastre.
- Dimensions in m, unless specified otherwise.
- Rebar in inches, rebar distance in mm.
- For longitudinal section see C75

CONCEPT DESIGN

Project : Design and preparation for the construction of the Eastern park of link 6 [Side roads]			Project nr : 216-1380
Principal : Ministry of VROMI	Date drawn : 18-07-2017		Revision :
Subject : Layout plan U-gutter 21	Drawn by : BZ		
	Checked by : GT		
	Format : Tabloid		
	Scale : 1:200		
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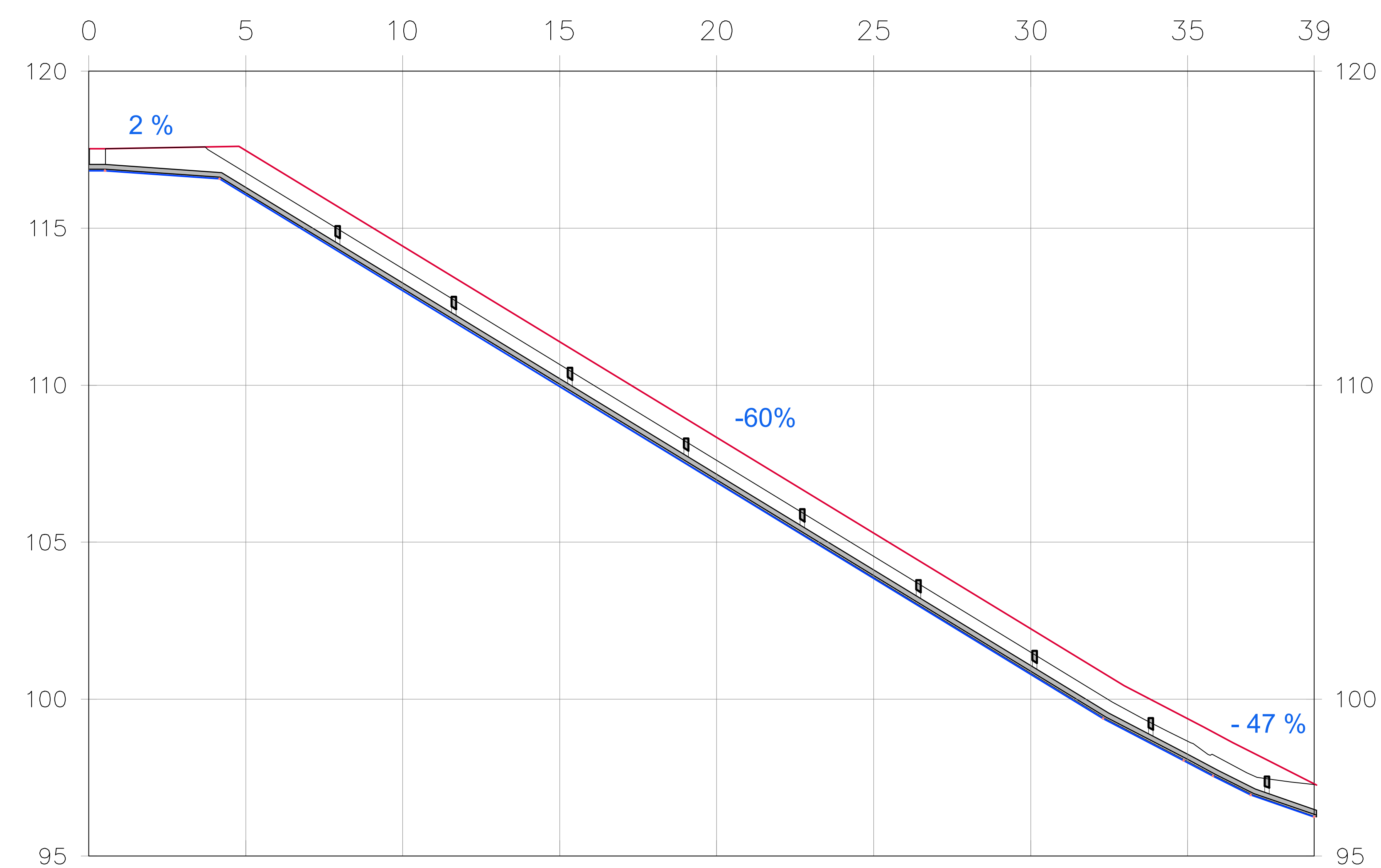
			File number : 216-1380-C74



	0+000.00	0+005.00	0+010.00	0+015.00	0+020.00	0+025.00	0+030.00	0+035.00	0+039.03
Existing level	117.53	116.77	113.71	110.66	107.60	104.55	101.49	98.66	97.28
Design level		116.06	113.01	109.95	106.90	103.84	100.79	97.98	96.25

Remarks:
- Existing level
- Design level
- U-Gutter Top Level

Remarks:
- Existing features are based Sint Maarten's countour map from the Cadastre.
- Dimensions in m, unless specified otherwise.
- Rebar in inches, rebar distance in mm.

CONCEPT DESIGN

Project : Design and preparation for the construction of the Eastern part of link 6 [Side Roads]

Project nr : 216–1380

Principal : Ministry of VROMI

Subject : U–Gutter 22 Longitudinal section

Date drawn : 18–07–2017
Drawn by : BZ
Checked by : GT
Format : Tabloid
Scale : 1:200

Revision :

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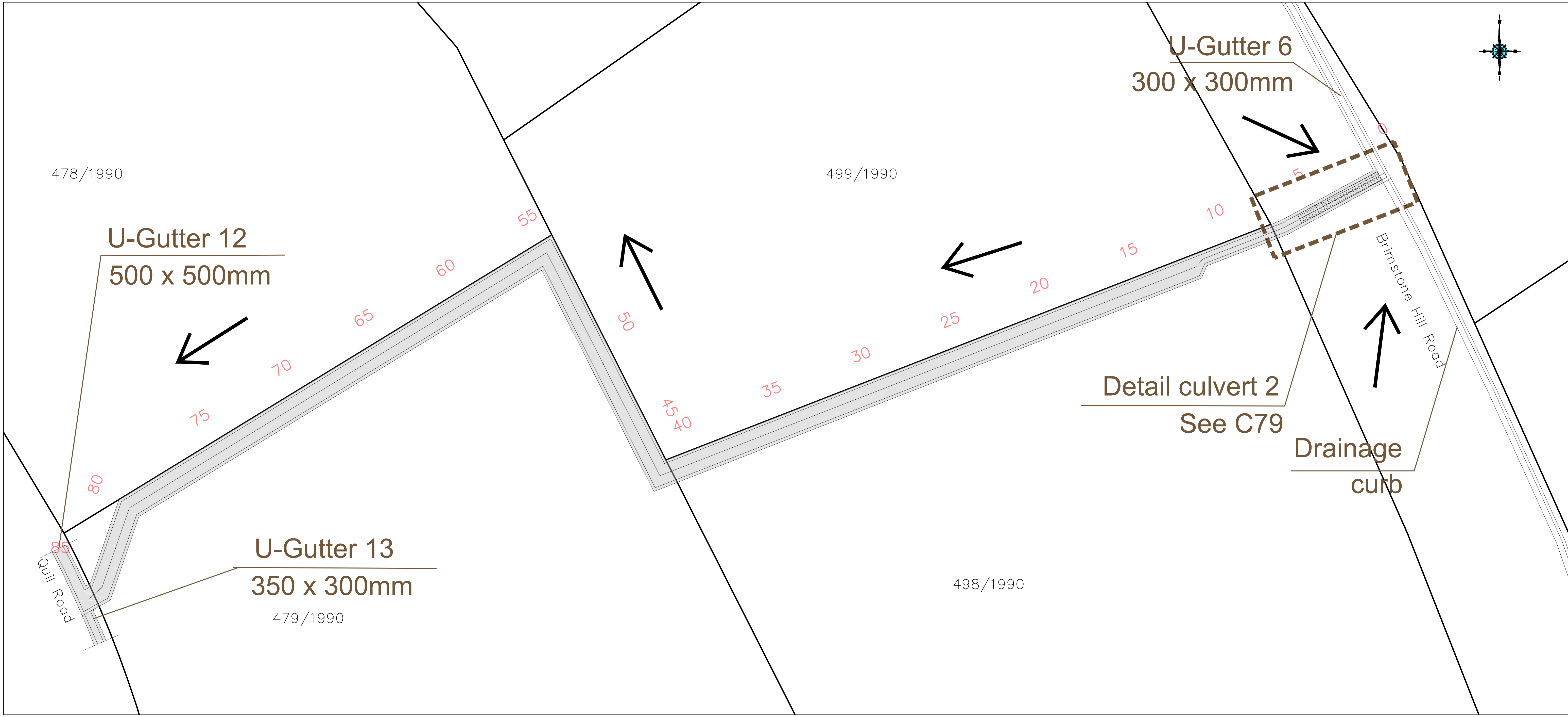
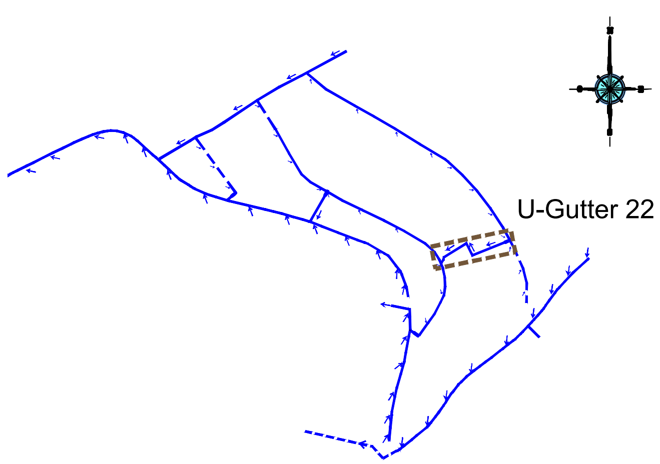
Sheet number : C75

File number : 216–1380–C75



Layout plan U-Gutter 22

scale 1:200



Remarks:

- Existing features are based Sint Maarten's countour map from the Cadastre.
- Dimensions in m, unless specified otherwise.
- Rebar in inches, rebar distance in mm.

- For longitudinal section see C77

LEGEND

(grey band)	Gutter
(hatched band)	Grate
(thick line)	Property boundary
(thin line)	Alignment
→	Drainage flow direction

CONCEPT DESIGN

Project : Design and preparation for the construction of the Eastern park of link 6 [Side roads]		Project nr : 216–1380
Principal : Ministry of VROMI	Date drawn : 18–07–2017	Revision :
Subject : Layout plan U-gutter 22	Drawn by : BZ	
	Checked by : GT	
	Format : Tabloid	
	Scale : 1:200	
		Sheet number : C76
		File number : 216–1380–C76

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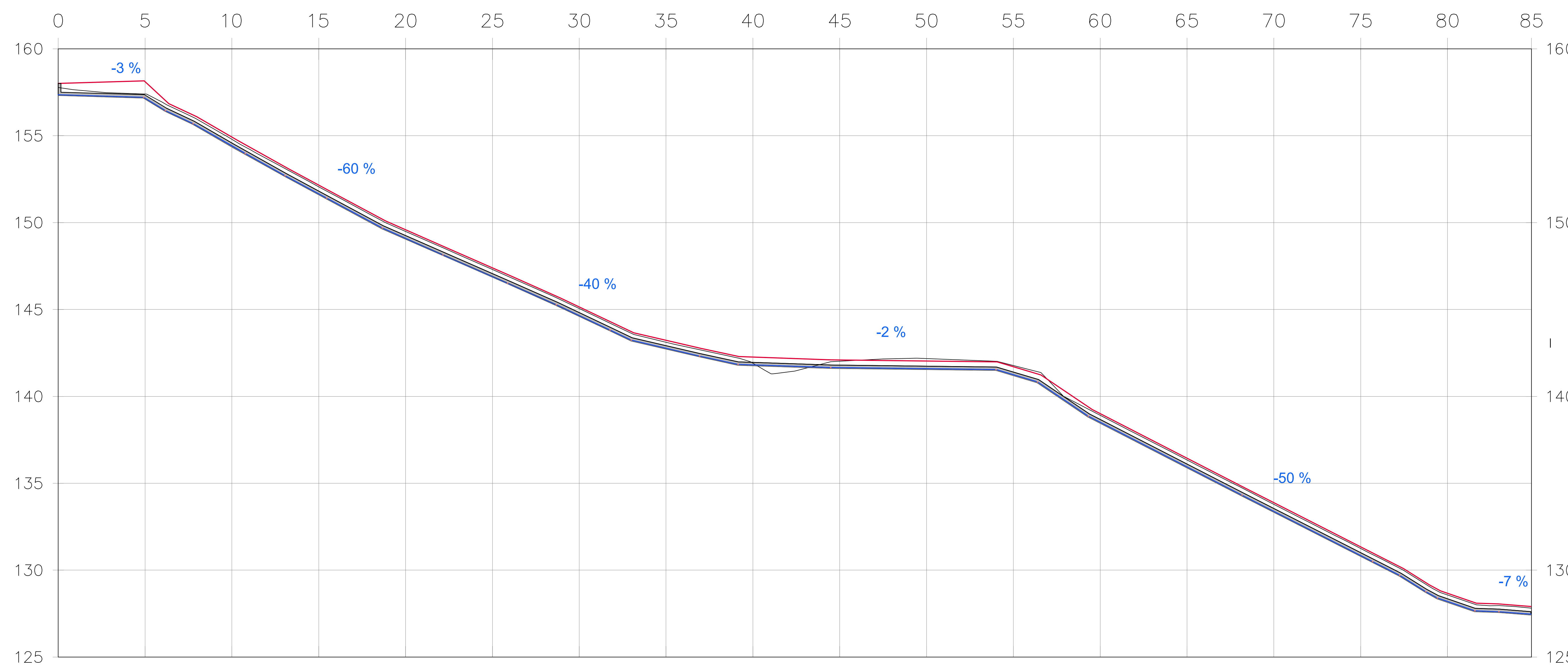

	0+000.00	0+005.00	0+010.00	0+015.00	0+020.00	0+025.00	0+030.00	0+035.00	0+040.00	0+045.00	0+050.00	0+055.00	0+060.00	0+065.00	0+070.00	0+075.00	0+080.00	0+084.84
Design level	157.77	157.40	154.79	152.06	149.49	147.30	145.05	143.11	141.91	142.03	142.18	141.78	138.91	136.34	133.79	131.24	128.57	127.81
Design level	157.35	157.15	154.41	151.65	149.09	146.89	144.64	142.77	141.81	141.65	141.59	141.24	138.50	135.94	133.38	130.84	128.19	

Remarks:
- Existing level
- Design level
- U-Gutter Top Level

Remarks:
- Existing features are based Sint Maarten's countour map from the Cadastre.
- Dimensions in m, unless specified otherwise.
- Rebar in inches, rebar distance in mm.

CONCEPT DESIGN

Project : Design and preparation for the construction of the Eastern part of link 6 [Side Roads]			Project nr : 216–1380
Principal : Ministry of VROMI	Date drawn : 18–07–2017		Revision :
Subject : U–Gutter 22 Longitudinal section	Drawn by : BZ		
	Checked by : GT		
	Format : Tabloid		
	Scale : 1:250		
INDEPENDENT CONSULTING ENGINEERS. N.V.			Sheet number : C77
			File number : 216–1380–C77

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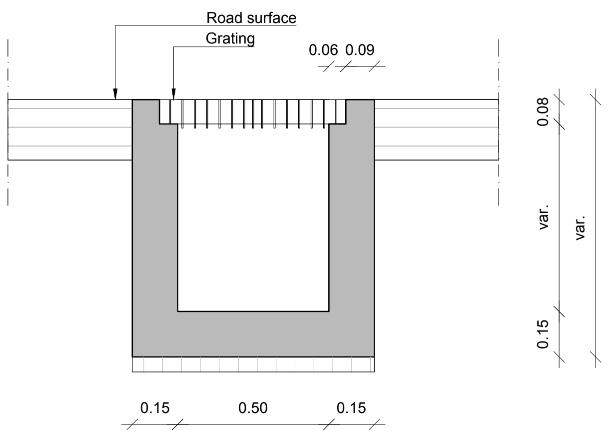
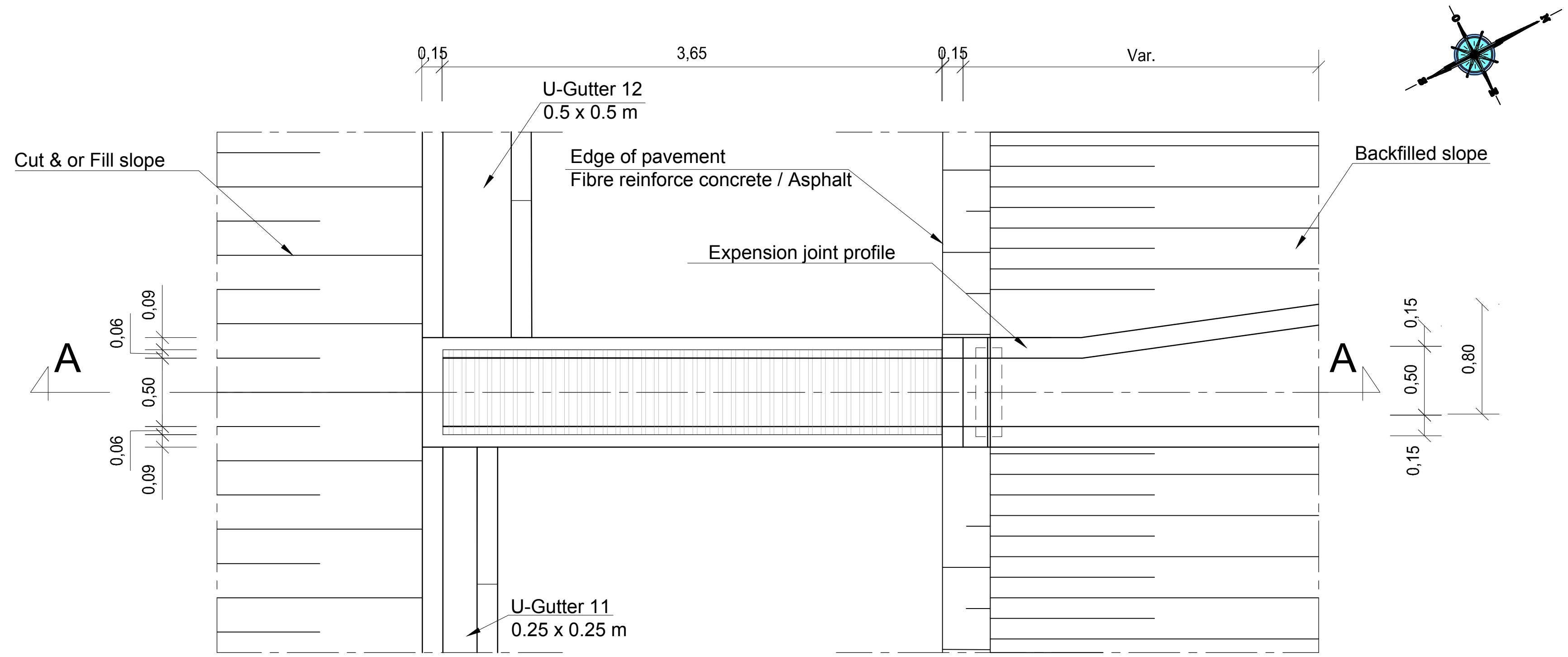
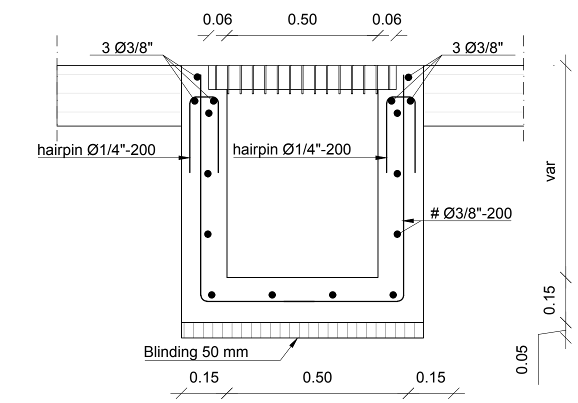


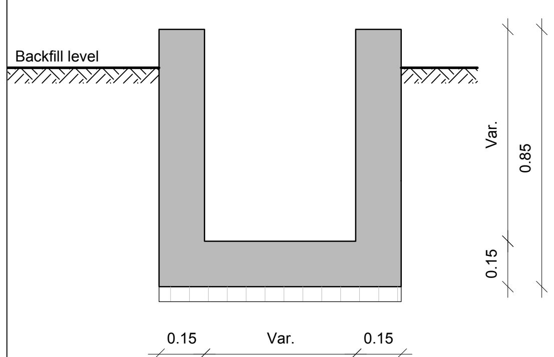
Cross section B
500 x 500 mm
scale 1:25



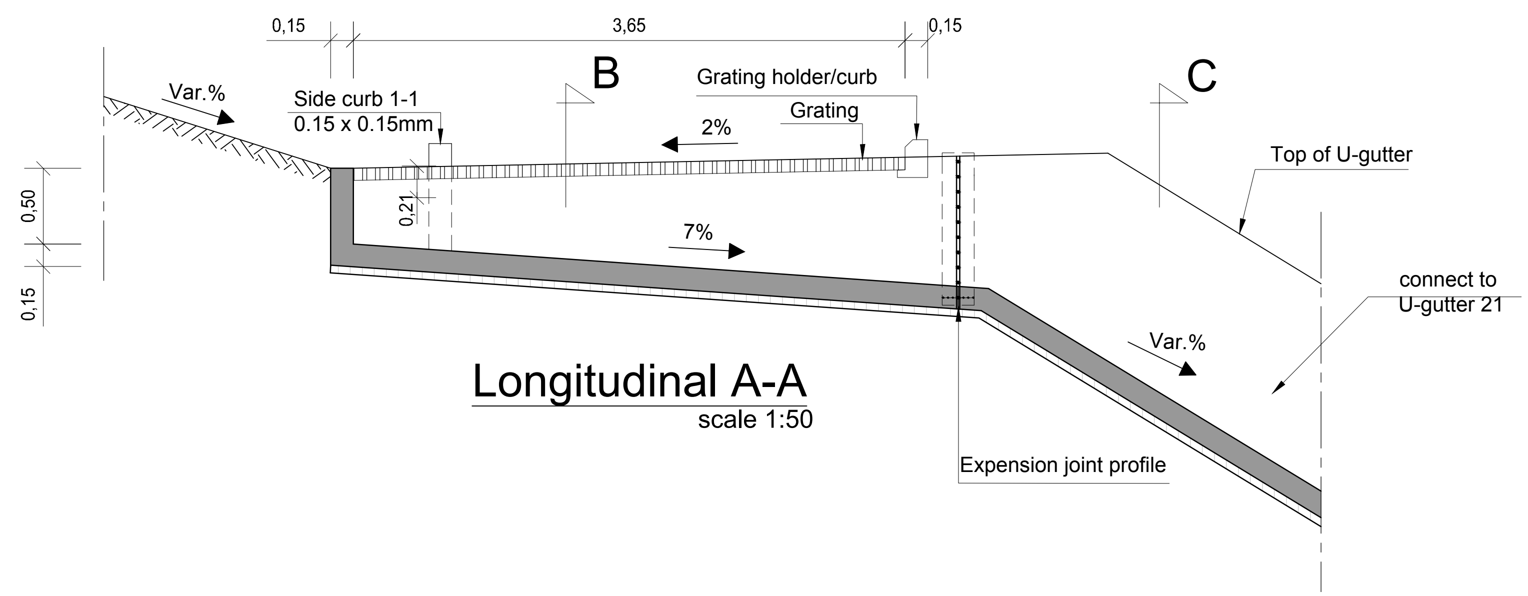
Top view culvert 1
scale 1:50



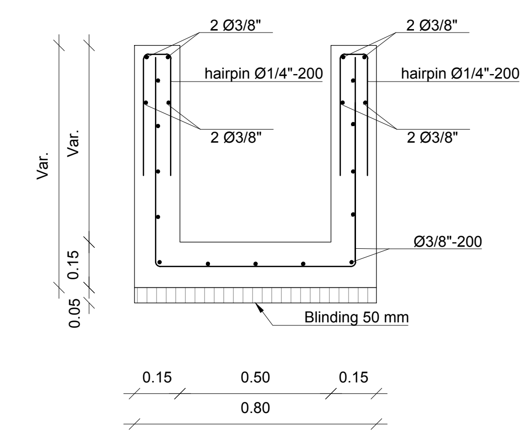
Grated gutter
Typical reinforcement
scale 1:25



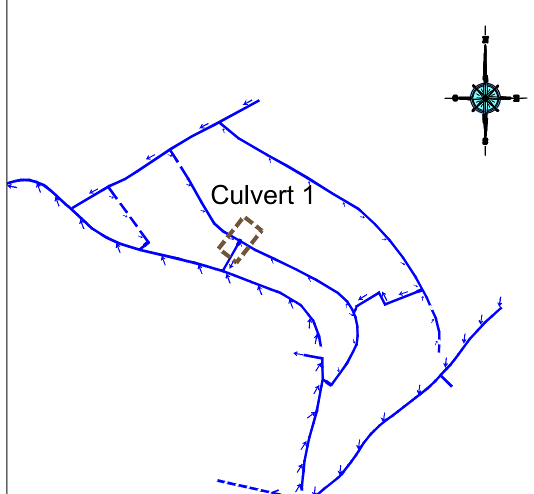
Cross section C
500 x 500 mm
scale 1:25



Longitudinal A-A
scale 1:50



Cross section C
Typical reinforcement
scale 1:25



Remarks:
- Existing features are based Sint Maarten's countour map from the Cadastre & survey drawing by Hunt's Topo Land
- Dimensions in m, unless specified otherwise.
- Rebar in inches, rebar distance in mm.

CONCEPT DESIGN

Project : Design and preparation for the construction of the Eastern park of link 6 [Side roads]

Project nr : 216-1380

Principal : Ministry of VROMI

Subject : Detail
Culvert 1
Quil Road

Date drawn : 18-07-2017
Drawn by : BZ
Checked by : GT
Format : Tabloid
Scale : 1:50/25

Revision :

Sheet number : C78

File number : 216-1380-C78

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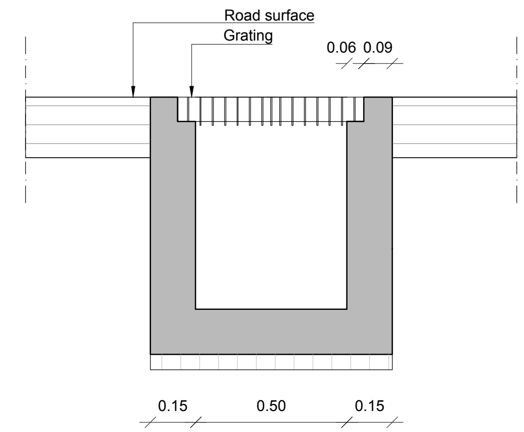
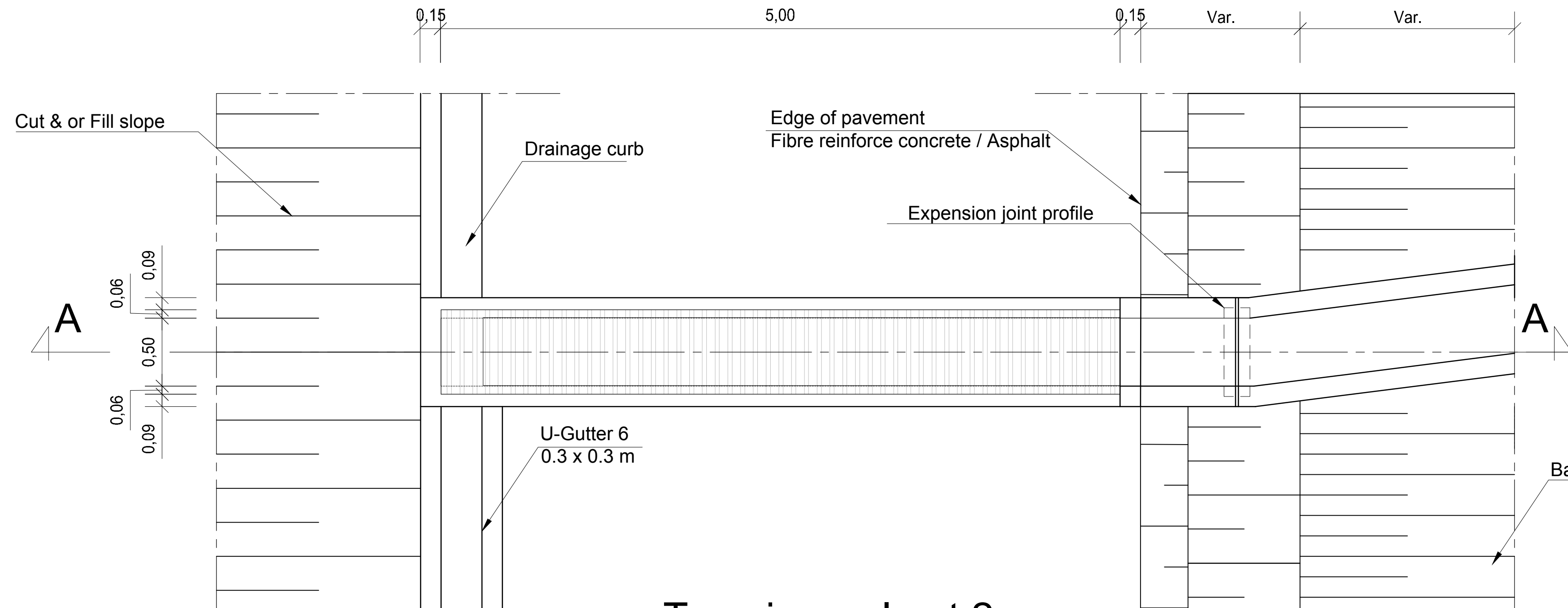
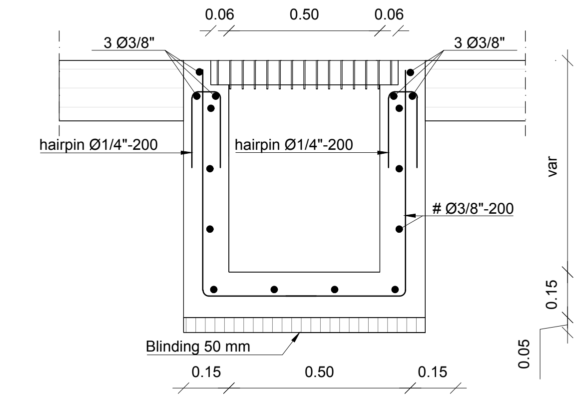


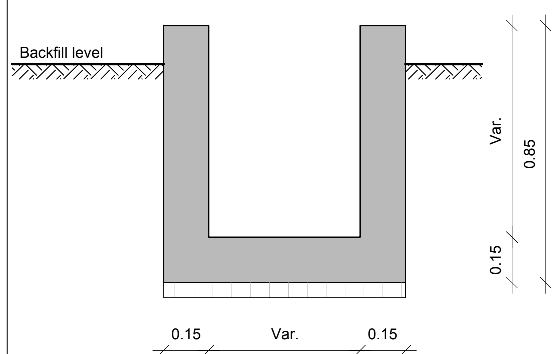
Cross section B
500 x 500 mm
scale 1:25



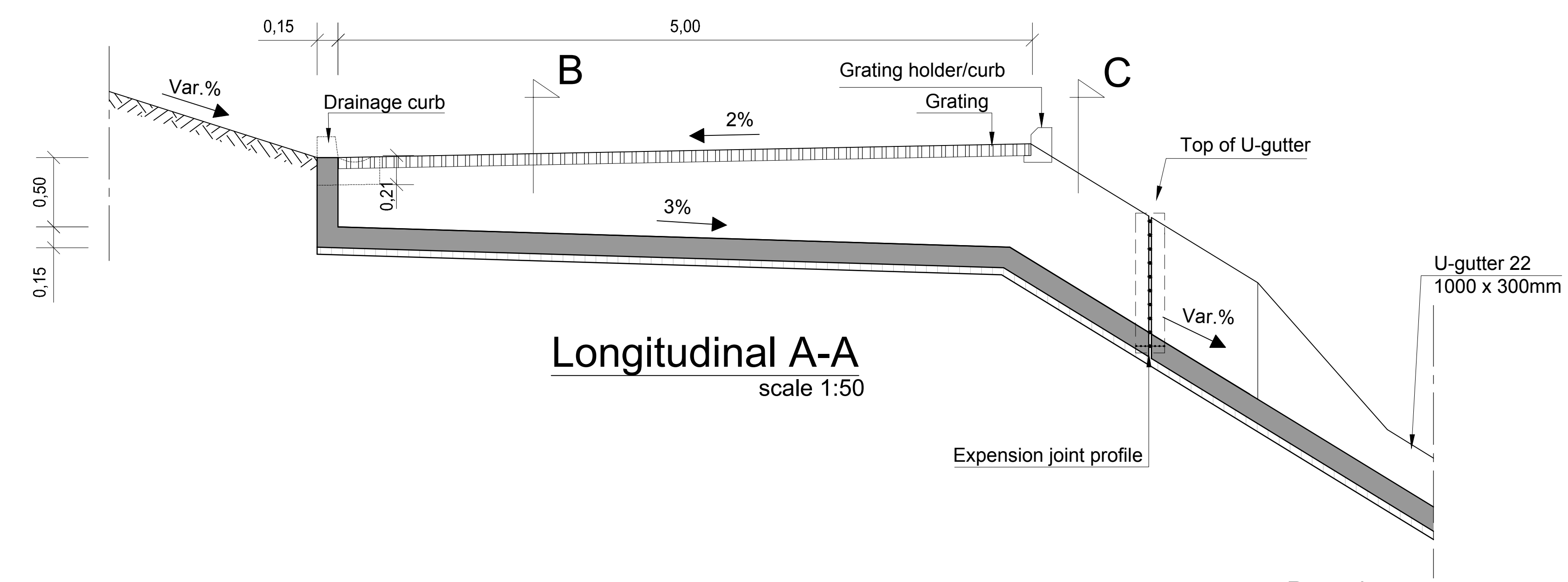
Top view culvert 2
scale 1:50



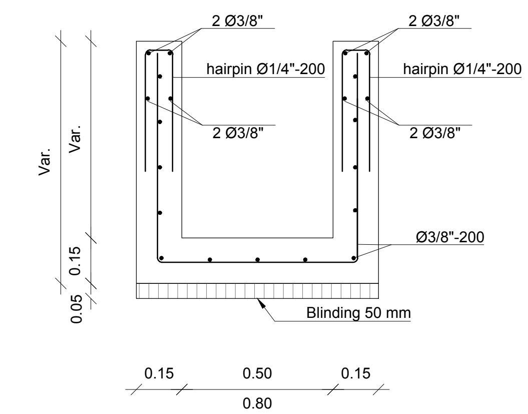
Grated gutter
Typical reinforcement
scale 1:25



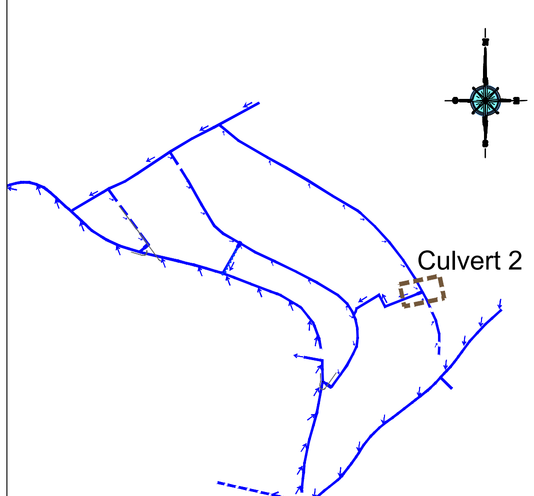
Cross section C
500 x 500 mm
scale 1:25



Longitudinal A-A
scale 1:50



Cross section C
Typical reinforcement
scale 1:25



Remarks:
- Existing features are based Sint Maarten's countour map from the Cadastre & survey drawing by Hunt's Topo Land
- Dimensions in m, unless specified otherwise.
- Rebar in inches, rebar distance in mm.

CONCEPT DESIGN

Project : Design and preparation for the construction of the Eastern park of link 6 [Side roads]			Project nr : 216–1380
Principal : Ministry of VROMI	Date drawn : 18–07–2017		Revision :
Subject : Detail Culvert 2 Brimstone Hill Road	Drawn by : BZ		
	Checked by : GT		
	Format : Tabloid		
	Scale : 1:50/25		
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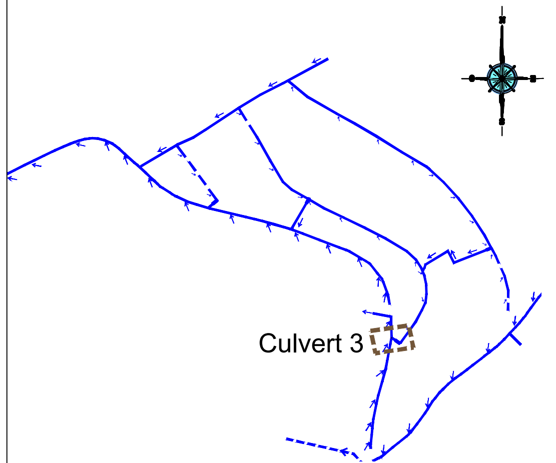
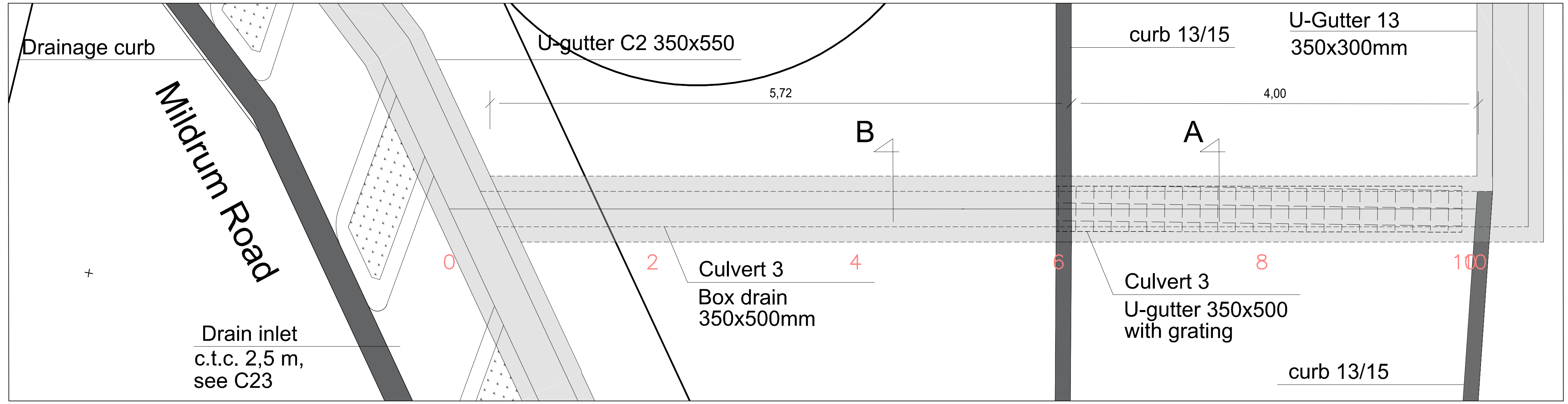
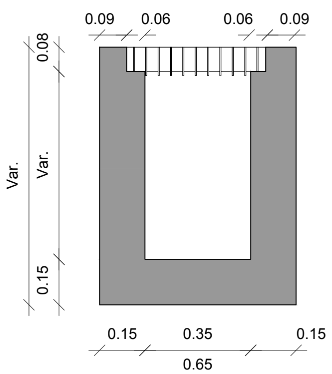
			File number : 216–1380–C79





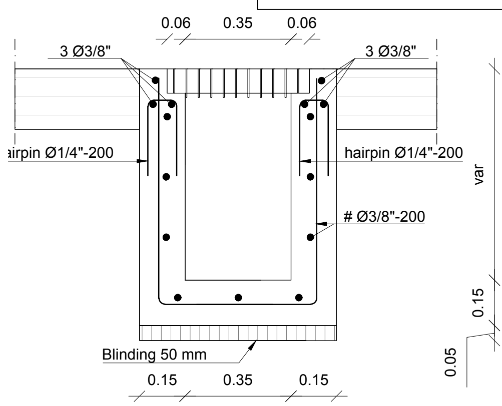
Top view Culvert 3

scale 1:50



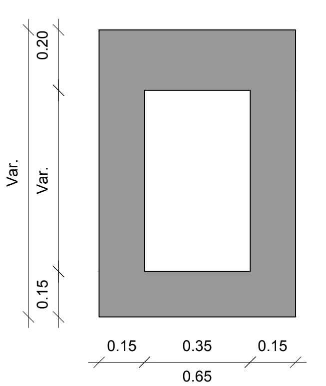
Section A

Grated U-gutter
350 x 500 mm
scale 1:25



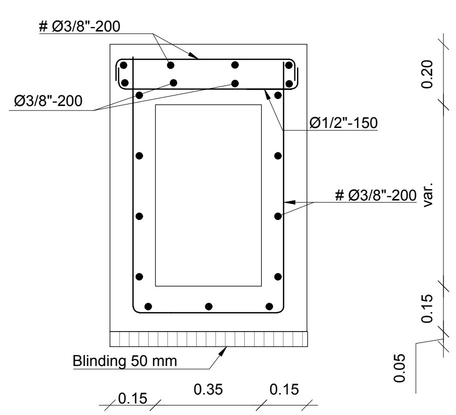
Grated gutter
Typical reinforcement

scale 1:25



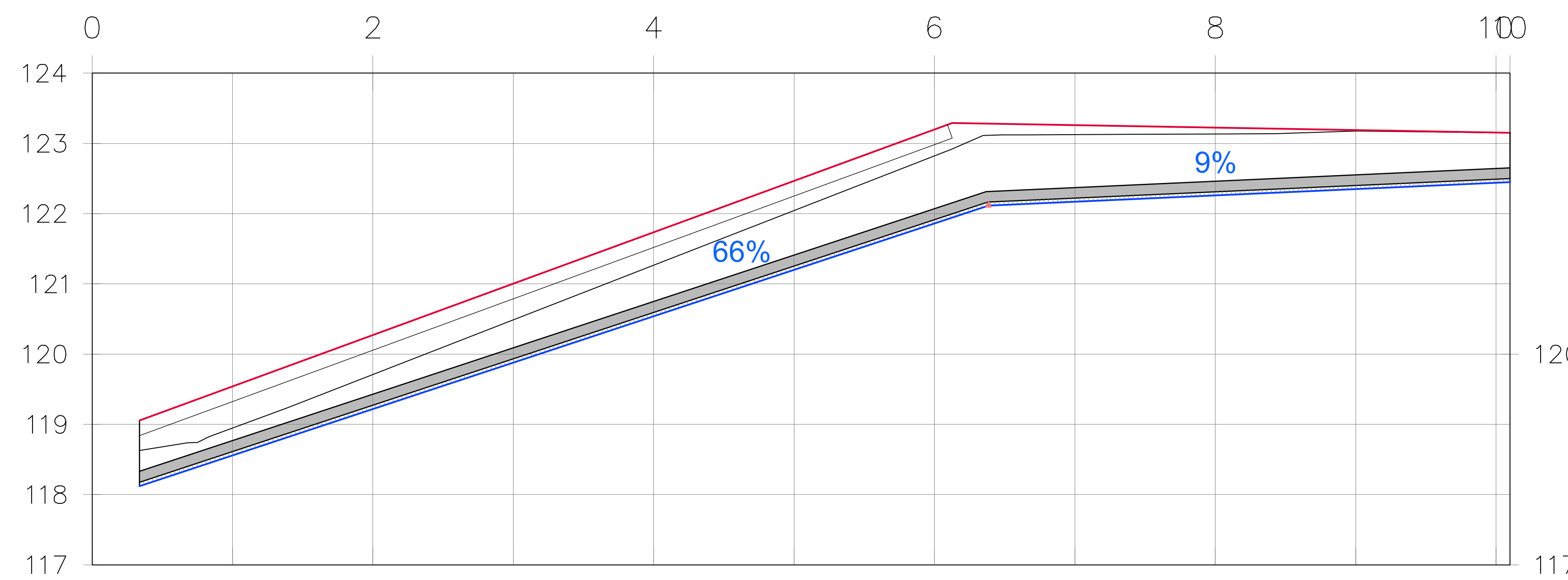
Section B

Box drain
300 x 500 mm
scale 1:25



Box drain
Typical reinforcement

scale 1:25



	0+000.00	0+001.00	0+002.00	0+003.00	0+004.00	0+005.00	0+006.00	0+007.00	0+008.00	0+009.00	0+010.00
Existing level		118.95	119.71	120.49	121.27	122.04	122.82	123.13	123.13	123.18	123.15
Design level		118.56	119.22	119.88	120.54	121.20	121.86	122.17	122.26	122.35	122.44

Longitudinal section

scale 1:50

LEGEND

- Gutter
- Grated U-gutter
- Curb
- Property boundary
- Alignment

Remarks:
- Existing level
- Design level
- Culvert Top Level

- Existing features are based Sint Maarten's countour map from the Cadastre & survey drawings by Hunt's Topo Land.
- Dimensions in m, unless specified otherwise.
- Rebar in inches, rebar distance in mm.

CONCEPT DESIGN

Project :	Design and preparation for the construction of the Eastern park of link 6 [Side roads]		Project nr : 216-1380
Principal :	Ministry of VROMI	Date drawn : 18-07-2017	Revision :
Subject :	Detail Culvert 3 Quil Road	Drawn by : BZ	
		Checked by : GT	
		Format : Tabloid	
		Scale : 1:50/25	
			Sheet number : C80
			File number : 216-1380-C80

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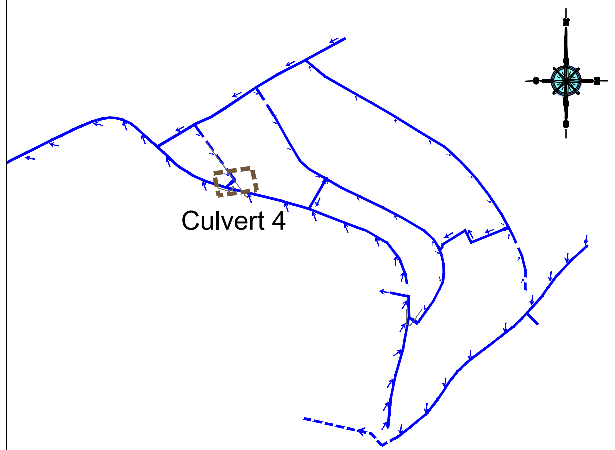
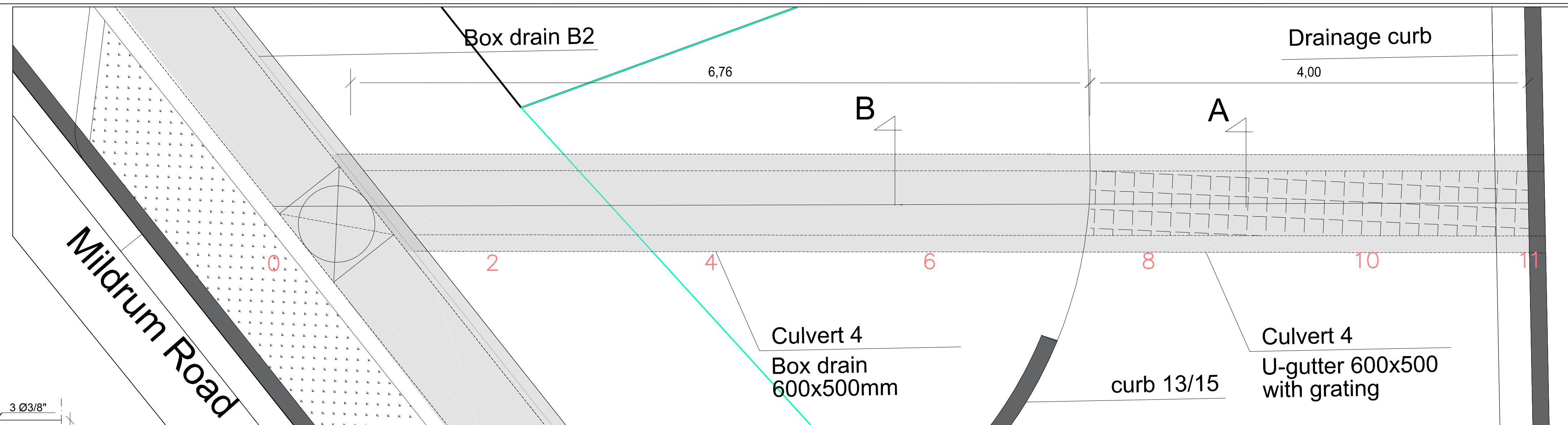
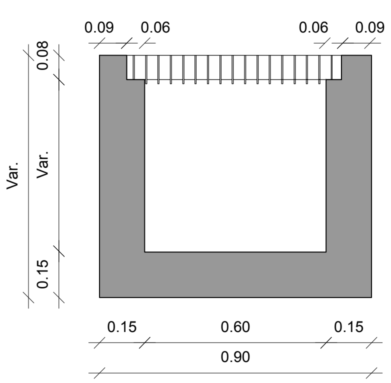
phone 5422421, fax 5422597, e-mail icesxm@sintmaarten.net





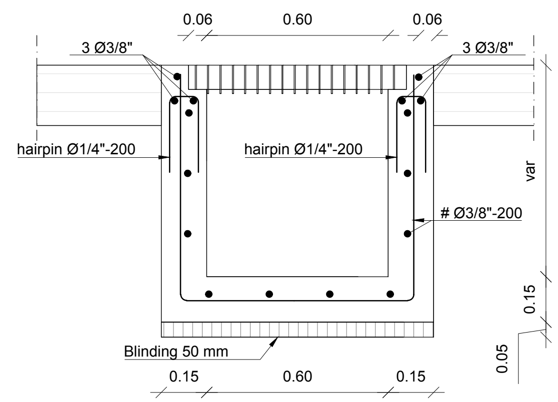
Top view Culvert 4

scale 1:50



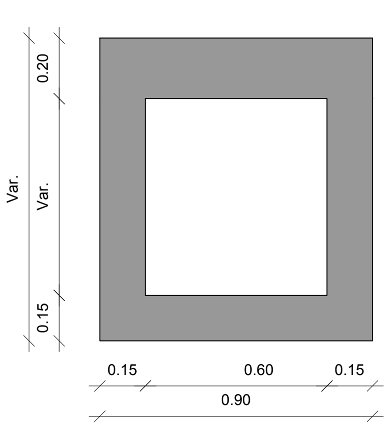
Section A

Grated U-gutter
600 x 500 mm
scale 1:25



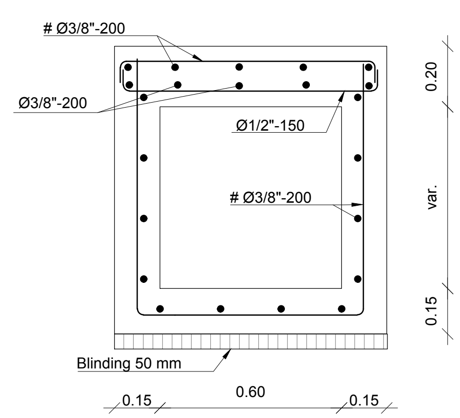
Grated gutter
Typical reinforcement

scale 1:25



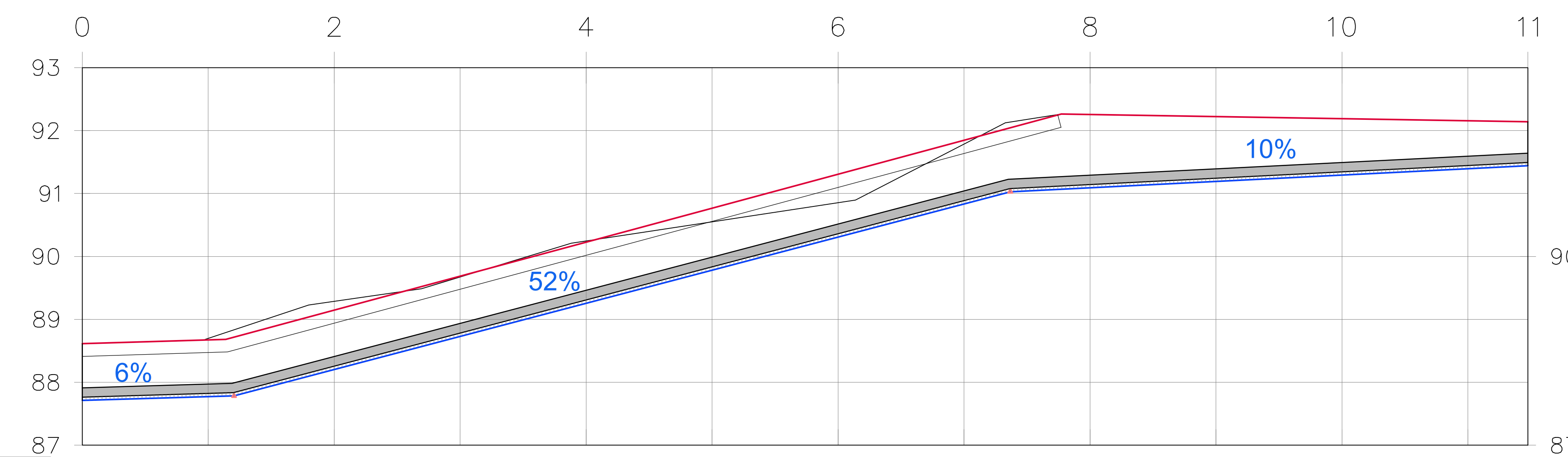
Section B

Box drain
600 x 500 mm
scale 1:25



Box drain
Typical reinforcement

scale 1:25



Existing level	0+000.00	0+001.00 88.70	0+002.00 89.29	0+003.00 89.67	0+004.00 90.25	0+005.00 90.55	0+006.00 90.86	0+007.00 91.79	0+008.00 92.25	0+009.00 92.22	0+010.00 92.19	0+011.00 92.16	0+011.48 92.14
Design level	0+000.00 87.71	0+001.00 87.77	0+002.00 88.20	0+003.00 88.73	0+004.00 89.26	0+005.00 89.78	0+006.00 90.31	0+007.00 90.84	0+008.00 91.09	0+009.00 91.19	0+010.00 91.29	0+011.00 91.39	0+011.48 91.44

Longitudinal section

scale 1:50

CONCEPT DESIGN

Remarks:
- Existing level
- Design level
- Culvert Top Level

- Existing features are based Sint Maarten's countour map from the Cadastre & survey drawings by Hunt's Topo Land.
- Dimensions in m, unless specified otherwise.
- Rebar in inches, rebar distance in mm.

LEGEND

- Gutter
- Grated U-gutter
- Curb
- Property boundary
- Alignment

Project : Design and preparation for the construction of the Eastern park of link 6 [Side roads]	Project nr : 216–1380
Principal : Ministry of VROMI	Date drawn : 18–07–2017
Subject : Detail Culvert 4 Paradise Hill Road	Drawn by : BZ
	Checked by : GT
	Format : Tabloid
	Scale : 1:50/25
	Revision :
	Sheet number : C81
	File number : 216–1380–C81

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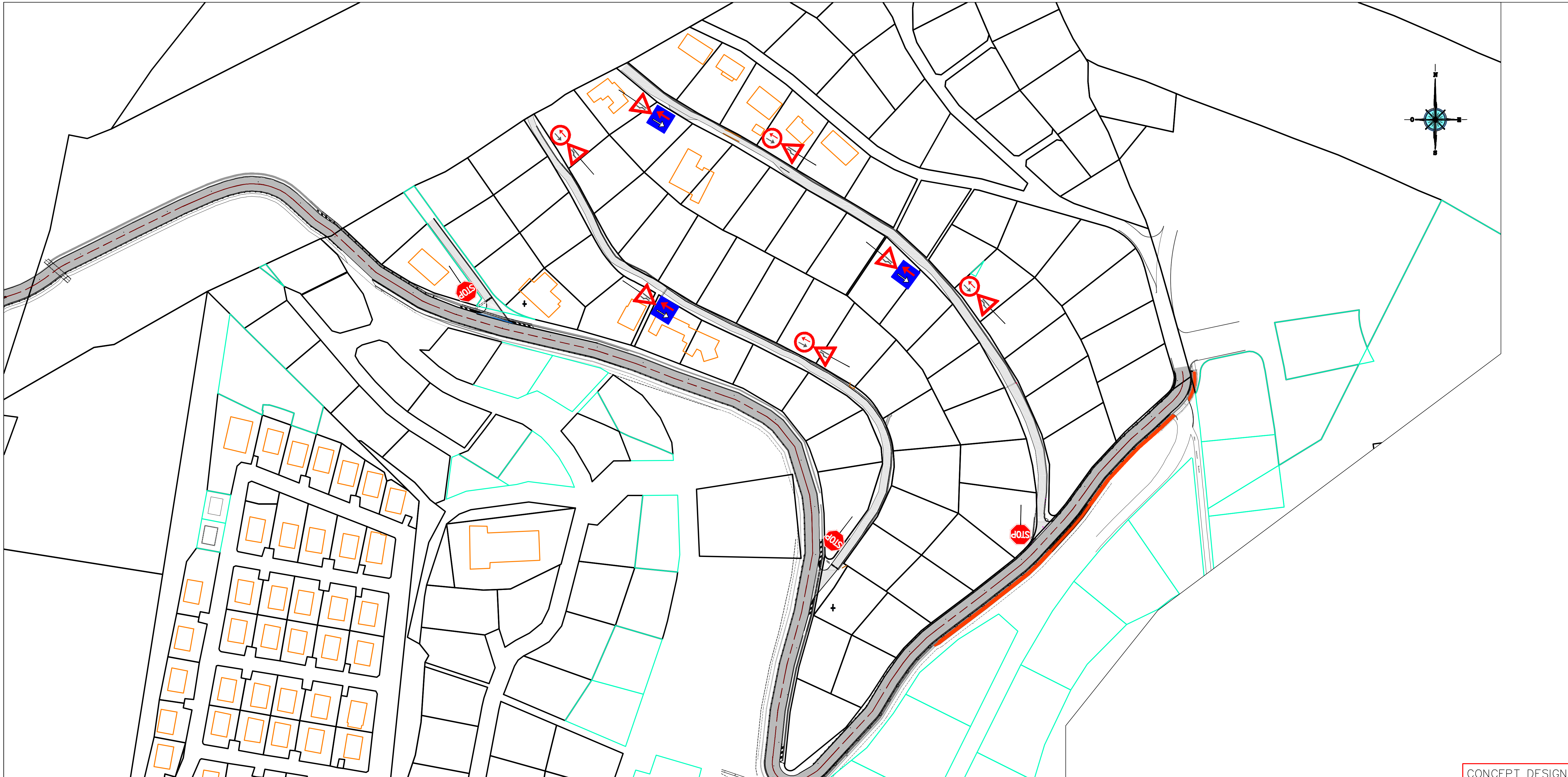

Layout road plan new situation traffic signs

scale 1:2000

Project :	Design and preparation for the construction of the Eastern part of link 6 [Side Roads]
Project nr :	216–1380
Principal :	Ministry of VROMI
Subject :	Layout road plan New situation Traffic signs
Date drawn :	18–07–2017
Drawn by :	BZ
Checked by :	GT
Format :	Tabloid
Scale :	1:2000
Revision :	
Sheet number :	C82
File number :	216–1380–C82

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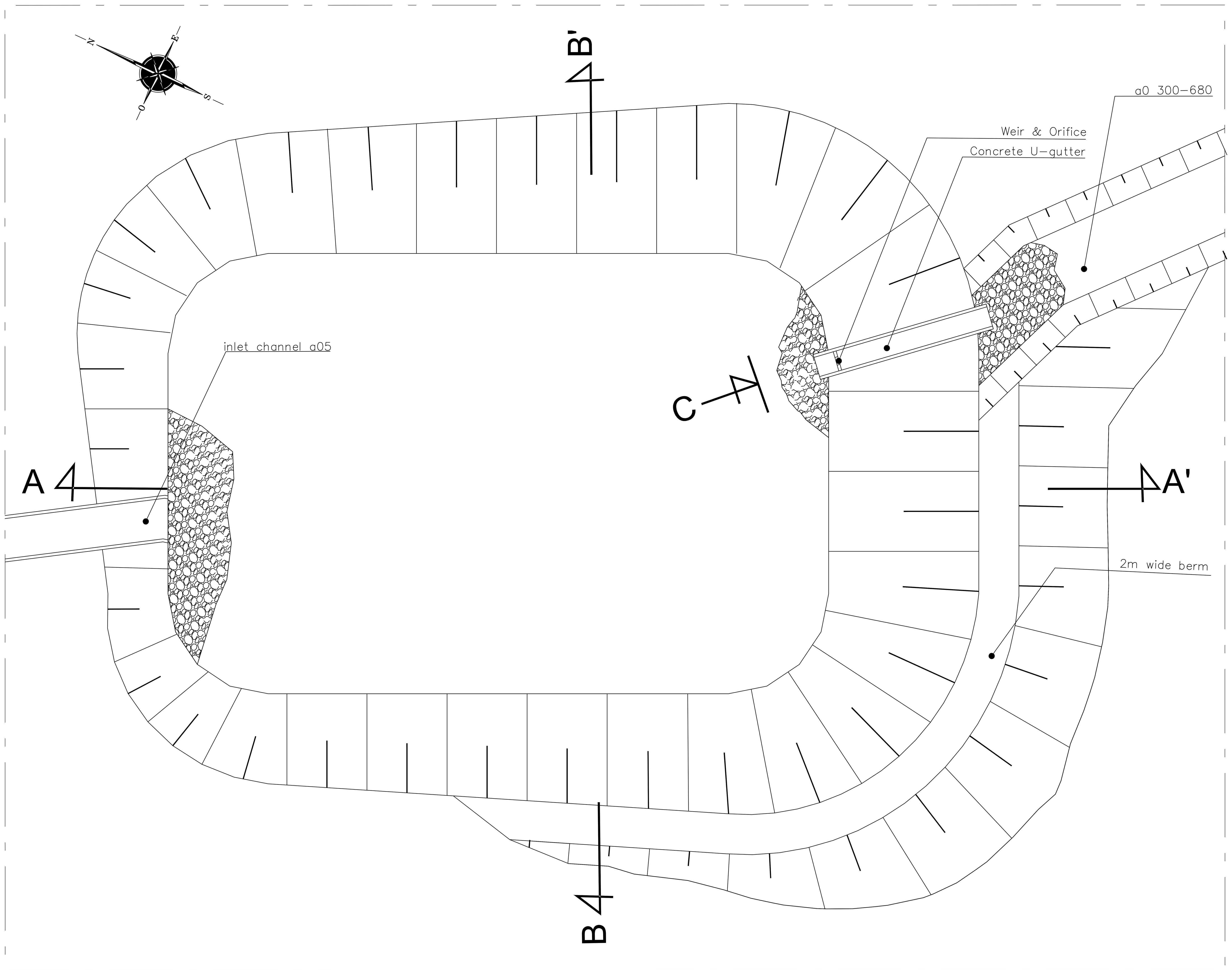
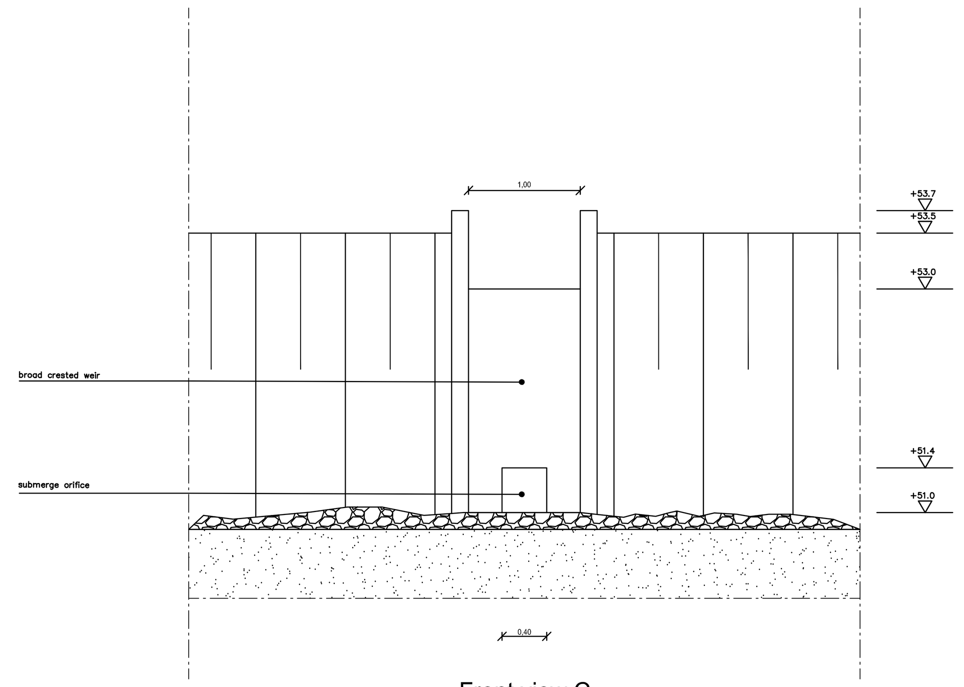


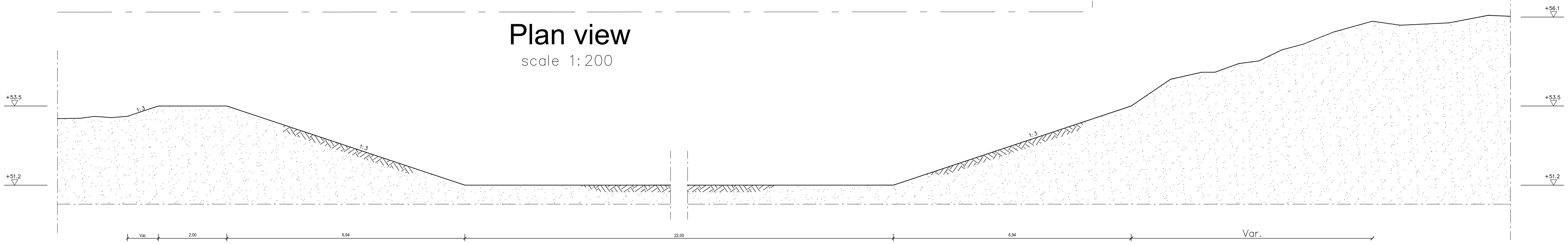
Plan view

scale 1:200



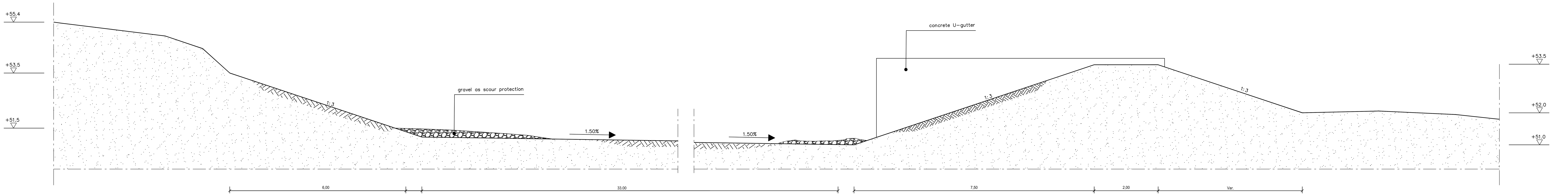
Front view C

scale 1:25



Cross section A-A'

scale 1:50



Cross section B-B'

scale 1:50

Project :	Design and preparation for the construction of the Eastern part of link 6 [Side Roads]		Project nr : 216–1380
Principal :	Ministry of VROMI	Date drawn : 18–07–2017	Revision :
Subject :	Plan view & cross sections of detention pond Front view of outlet structure	Drawn by : BZ	
		Checked by : GT	
		Format : A0	
		Scale : 1:200,1:50,1:25	

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Sheet number : C90

File number : 216–1380–C51

Appendix E. Technical Requirement for Infrastructure Execution

Table of Contents

1 INTRODUCTION TO SECTION VI

1.1 Introduction

The Easter part of Link 6 is an infrastructure project for the upgrading of the existing Mildrum Road to a primary connections road and the side roads of Mildrum Road to decent secondary roads. The project is roughly delineated by L.B. Scott Road, Valley Estate and Prima Vista.

The Employer (also known as the principal), the Ministerie van Volkshuisvesting, Ruimtelijke Ontwikkeling, Milieu en Infrastructuur (Ministry of VROMI) of Sint Maarten plans on developing the infrastructure needed. Independent Consulting Engineers N.V. has been commissioned by VROMI to make the design for the infrastructure.

The works shall be carried out in accordance with the details shown in the Contract and no deviation shall be allowed unless approved or directed by the Engineer.

All dimensions in all documents, drawings, calculations and information shall be expressed in metric SI unless stated otherwise.

1.2 Location Site

The location is the existing Mildrum Road and the side roads of Mildrum Road in Waymouth Hills: Paradise Hill Road, The Quill Road, Brimstone Hill Road, Mount Scenery Road and Mount Soufriere Road, as well as some additional gutters. The project is roughly delineated by L.B. Scott Road, Valley Estate and Prima Vista.

1.3 Work description and parcels:

The work consists of the following parcels:

(I). all works Mildrum Road
(II). all works Paradise Hill Road
(III). all works The Quill Road
(IV). all works Brimstone Hill Road
(V). all works Mount Scenery Road
(VI). all works Mount Soufriere Road

The works shall consist among others of the following:

1.3.1 Parcel i

1. Removal of existing trees and bushes, clearing and grubbing;
2. Earth works for roads, drainage, utilities cables trench;
3. Install sand under and around pipes and cables;
4. Install soil cement and backfilling;
5. Install road crossings;
6. Install utilities cables, (high tension cable, low tension cable, streetlight cable, water line, telecommunication cables);
7. Construction of fiber concrete road, sidewalks;
8. Construction of street gutters / drainage curbs;
9. Construction of rectangular concrete gutters, gutters with grating and culverts.
10. Construction of retaining walls;
11. Install and or move meter walls, streetlight poles and fire hydrants and such;

12. Install landscaping;
13. Preparing as-build drawings;
14. Commissioning and testing;
15. Preliminary take over by Employer;
16. Defect liability period (maintenance for 6 months after take over).

2 GENERAL CONDITIONS FOR EXECUTION

2.1 General

1. All to be provided and/or to be processed materials must be new, free of damage and unused.
2. During the execution, the contractor must take measures to restrict hindrance to third parties to a minimum.
3. During the execution, the contractor must take measures to avoid damage to properties of third parties. The contractor must directly repair any introduced damage. These costs are at the expense of the contractor.
4. Properties, etc. outside or in the immediate vicinity of the project areas shall not be damaged. Eventual damages to these supplies during the implementation shall be repaired, at the expense of the contractor. This shall be done within 24 hours after the first demand by the Engineer. Damages shall be reported to the Engineer immediately.
5. Areas bordering to the work area must remain as much as possible untouched.
6. The completed work, finished under profile, must be connected to the connecting areas, slopes, ways, paths, drainage, sewerage, utilities in consultation with the Engineer.
7. If during the execution of the work damage occurs to trees or other growth, which are to be preserved, cost of repair or replacement shall be at the expense of the contractor. The Engineer shall replace these damaged trees etc. by similar trees. The costs of this shall be implied on the firstly next payment term.
8. The contractor is responsible for missing, theft or damage of all materials.
9. The contractor is responsible for and shall bear the cost of all quality testing. Testing shall be done by a qualified expert or laboratory independent from the contractor, subcontractor or supplier unless the Engineer explicitly permits otherwise.

2.2 Data

1. Part of these specifications are the drawings mentioned in section **Error! Reference source not found. Error! Reference source not found.**.
2. The Contractor has to provide as-built drawings and product specifications within 2 calendar weeks after completion of the project.
3. The as-built drawings have to be checked by the Engineer and after approval by the Engineer the drawings shall be provided in 3-fold on paper as well as digital (PDF and AutoCAD 2010).
4. Product specifications shall be provided in 3-fold on paper as well as digital (PDF). On the as-built drawing the following shall be indicated:
    1. The exact location, elevation, dimensions and invert levels of drainage constructions and gratings;
    2. The location and elevation of new roads, sidewalks, pavers or landscaping;
    3. The location and elevation of other (retaining) structures;
    4. The exact location and depth of the road crossings for the cables and pipes of the utility companies;
    5. The exact location of the meter walls, street lights poles, signs and fire hydrants;
    6. The exact location of the utilities cables and splices;
    7. All other relevant information
5. Elevations on the as-built drawings shall be in SMP.

2.3 Survey, stake - out and detail design

The design of the development has been made based mainly on survey works and partially on the "island topographical information" as available. As this topographical information, the procedure described below shall be followed by the Contractor.

1. The contractor shall employ the services of a land surveyor for the duration of the project. This surveyor shall have to be the same person or person(s) within a firm for the duration of the project.
2. The contractor shall clear the terrain of trees and growth if and where necessary.
3. The contractor's surveyor shall stake out all constructions and confirm the elevations of the existing terrain.
4. Deviations due to the surveyed elevations shall under no circumstances be basis for claims for extra or lesser work. The contractor shall maintain his original contract sum also for the adjusted plans.
5. The contractor shall use the same surveyor during the entire project. During the excavation the z-coordinates must be measured directly from the staked-out reference points.
6. The contractor shall prepare drawings for the approval of the Engineer of details that are not covered in the design drawings and shop drawings, when necessary.

2.4 Permits and approvals

2.4.1General

1. The Infrastructure / building permit shall be arranged by the Employer.
2. The Contractor shall comply with any conditions or restrictions on construction imposed by approvals and permits at the cost of the Contractor
3. The Contractor shall supply any available information necessary for permits and approvals to the Engineer if necessary at no additional cost.

2.4.2 Traffic

1. The Contractor shall obtain approval from concerned authorities for road closures, road diversions and notifications. The Police and Fire Authority shall also be notified by the Contractor.
2. The Contractor shall keep all roads, over which construction traffic shall pass, clear of all dirt and mud and shall ensure that a safe and adequate route is available to vehicular and pedestrian traffic at all times. The Contractor shall agree optimum haul routes for delivery of materials to the work site with the relevant authorities concerned.
3. The cost of obtaining this approval is for the Contractor.

2.4.3 Excavation

1. The Contractor shall obtain approval from Authorities for all excavation work in and near public roads and other public spaces.
2. The cost of obtaining this approval is for the Contractor.

2.4.4 Private property

1. The Contractor shall inform and make detailed arrangements with the proprietors of private properties, if access to, or work on the private property is necessary for the execution of the Works.
2. The Contractor shall vouch for continuously accessibility to private properties. Temporary constructions shall be applied if required to maintain accessibility to private properties during execution of the Works.
3. Any damage at private properties is the full responsibility of the Contractor. Damage to any construction or object shall be repaired and restored to the original state at the full expense of the Contractor.
4. The Contractor shall prepare a detailed pre-construction survey, abundantly illustrated with pictures of any (parts of) private property that are used by the Contractor. The pre-construction survey shall be made in consultation with, and at the Engineers request in the presence of, the Engineer.

2.4.5 Service of Utilities

1. The Contractor shall be responsible for identifying the location and nature of all services on the construction locations, liaising with utilities and other organizations or bodies whose services may be affected by the works and obtaining the necessary permits and approvals for design and construction of the works. the necessary approval periods shall be allowed for in the Contractor's scheduling program.
2. Notwithstanding any approvals, before excavation commences the Contractor shall ascertain the accurate location of existing services using safe methods of pipe locations, cable detection or hand digging of trial holes as appropriate. Notwithstanding any services information supplied to the Contractor the responsibility to locate all services and prevent any damage to existing services shall rest with the Contractor, and no claim for extra costs shall be accepted
3. The Contractor shall be responsible for all works as may be required in the interrelation with existing utilities and services, such as the realignment, adjustment, disconnection, relaying and reconnection, for all and any delay occasioned thereby and making payment to the relevant statutory bodies for utility services.
4. The Contractor shall ensure that all utility service providers can gain access to that equipment to inspect, repair and renew the same without restriction.

3 CLEANING- AND DEMOLITION WORKS

3.1 General

1. No cleaning or demolition works shall be started without the approval by the Engineer.

2. The demolition works consist of the demolition of (retaining) structures, pavement et cetera and the removing of street furniture, such as fire hydrants, meter walls and light poles for later reinstallation.

3. Clearing and grubbing shall consist of clearing the surface of the ground of the designated areas of all trees, stumps, down timber, logs, snags, brush, undergrowth, hedges, heavy growth of grass or weeds, asphalt, fences, structures, debris, and rubbish of any nature, natural obstructions including the grubbing of stumps, roots, matted roots, foundations.

4. In areas to be cleared and grubbed, all stumps, roots, buried logs, brush, grass, and other unsatisfactory materials shall be removed.

4 EARTH WORKS

4.1 General

1. Under earthworks it is understood the excavation and backfilling of areas according to the indicated profiles and measurement on the drawings.
2. The contractor shall carry out all those earthworks and take all measures that are necessary for a good execution and safety, also these that are not described separately for the relevant activities.
3. Slopes, excavations, backfilling must be finished tightly and under profile as indicated on drawing.

4.2 Excavations

1. All obstacles encountered during the excavations must be removed by the contractor and transported, as far as the Engineer considers necessary.
2. The contractor shall remove all, upon judgment of the Engineer, unsuitable soil. Depending on size and depth, to the judgment of the Engineer, set off shall take place as extra work in accordance with the agreed unit price for soil improvement.
3. Trenches and excavations shall be supported where necessary to prevent landslides or collapsing. During the excavation the contractor is responsible for the bracing of the existing resident boundary walls and fences.
4. The contractor shall take measures to prevent erosion of the existing ground, trenches or excavations during heavy rainfall.
5. The contractor has to excavate up to 10 cm under the new utility cables and pipes.
6. Slopes with a steepness of 1.5:1 or steeper shall have a 1:1 slope at the top of the cut, to accommodate the existing top layer of approximately 1 m. The height of the 1:1 slope shall be more than 1 m if necessary, at the discretion of the Engineer.

4.3 Backfilling

A. Back filling under road pavement and other constructions

1. The plans provide for the road pavement to be situated almost everywhere in excavation into solid and stable material. However, variations in the terrain grade and the presence of unsuitable material might necessitate backfilling under the road pavement envelope. Other constructions, such as sidewalks, are often not in excavation.
2. The contractor has to use soil that is coming out of the excavation for the project for backfilling. Before reusing the excavated soil the contractor has to filter it so the big stones are removed from the soil
3. For backfilling under road pavements and other constructions, the materials released from the excavation shall be used as much as possible, as far as they exists of gravelly, sandy or other hard and coarse granular material. If necessary, make these materials suitable by sifting. All this subject to approval by the Engineer.
4. Under no circumstances silt, clay, peat and/or other similar material may be processed in back fillings. When the contractor sees that he is digging in silt, clay, peat/or other similar material, the contractor has to consult with the Engineer.
5. Backfilling has to be done in layers of 0,25m; each layer has to be compacted with an adjustable roller, upon approval by the Engineer. Fill has to be compacted to a value of 98% modified proctor density.
6. If there are any stones with size above 10 cm found in a backfilling layer, which shall hamper good compaction, these stones must be removed from the filling layer. The resulting holes shall be filled up directly with suitable backfilling materials.
7. Back fillings have to be finished tight and smooth.

B. Backfilling outside of road pavement and side walk envelope

1. Backfilling outside of road pavement, side walk, drainage gutter and other construction's envelopes shall not require compaction

C. Backfilling of trenches for utility lines

1. Backfilling of trenches around cables as indicated on plans has to be done with clean sand, to the approval by the utility companies. Sand rejected by the utility companies' representatives on site has to be considered as rejected by the Engineer
2. For backfilling of trenches, other than sand around cables, the materials released from the excavation shall be used as much as possible, as far as they exist of gravelly, sandy or other hard and coarse granular material. If necessary, make these materials suitable by sifting. All this to the approval by the Engineer.
3. Backfilling has to be done in layers of 0,25m; each layer has to be compacted with an adjustable roller, upon approval by the Engineer. Fill has to be compacted to a value of 95% modified proctor density. Testing shall be done at a qualified expert independent from the contractor or subcontractors at the Contractor's cost

4.4 Testing

1. The compaction degree of the sub-soil and of the backfilling must be checked by means of nuclear measuring.
2. For each type of backfill material used, a modified proctor test must be carried out.
3. Control compaction by means of nuclear measuring: 1 per 100 square meters , for each layer of backfill.
4. Contractor has to provide a drawing in which is indicated the exact place of the test locations.

5 SOIL-CEMENT STABILISATION

5.1 General

1. Where indicated on the drawings, or other locations mentioned in the specifications, back filling or foundations of soil cement have to be applied.
2. Where the road is not in excavation, but on compacted backfill, the road shall have a foundation of soil-cement.
3. Before installing foundations the necessary excavations or filling operations have to be completed
4. Before installation of the soil cement stabilization the subsoil has to be compacted. During compaction the degree of compaction achieved has to be monitored continuously. As soon as the required degree of compaction is achieved the compaction shall be stopped. The subsoil has to be compacted to 95 % of Modified Proctor Density.
5. The Contractor can propose to use base course instead of soil cement, subject to the approval by the Engineer. The Contractor's proposal shall include the specifications of the base course.

5.2 Preparation and installation

1. Soil cement shall be a mixture of soil, cement and water, with a cement content of 8 % of weight of soil.
2. The contractor can use the soil that is coming out of the excavations to make soil cement. Before reusing the excavated soil the contractor has to filter it so the big stones are removed from the soil. The contractor has to mix the existing soil with quantity 8 weight % for soil type A2 (AASHO Classification). In case other soil types are used the cement quantity shall have to be adjusted accordingly to the approval by the Engineer. Before backfilling a modified proctor test has to be made of the material.
3. The components to be mixed such that a homogenous mixture is achieved.
4. Soil cement to be applied in layers of maximum 20 cm, and to be compacted to at least 95 % of Modified Proctor Density

5.3 Materials

Soil shall not contain PAH's (Dutch PAK's), other chemical pollution or organic materials.

5.4 Testing

1. The compaction degree of the sub-soil and of the backfilling must be checked by means of nuclear measuring.
2. For each type of backfill material used, a modified proctor test must be carried out.
3. Control compaction by means of nuclear measuring: 1 every 100 square meters per layer.
4. Contractor has to provide a drawing in which is indicated the exact place of the test locations.
5. Per 40 m3 of soil cement 3 cubes 150x150x150 to be made, to be tested on compressive strength after 28 days. Cubes need to achieve a minimum compressive strength of 3 N/mm2.

6 CONCRETE WORKS

6.1 General conditions

1. The works as specified in these specifications consist mainly of the following:
   - The concrete curbs;
   - The drainage gutters and culverts;
   - The meter walls: prefabricated concrete meter walls in accordance to GEBE specifications and dimensions. If GEBE specifications conflict with underlying specifications, the Contractor shall inform the Engineer. The Engineer shall decide on how to resolve the conflict;
   - Retaining walls;
2. Unless otherwise stated all reinforced concrete and its components shall comply with the European and Dutch Regulations for concrete, NEN 6722 (VBT 1995) via A3, VBU 2002, NEN-EN 206-1 (nl) and the NEN 8005 (nl) Dutch supplement to NEN-EN-13791, “Europese beproevingsnormen en grondstofnormen” or equivalent.

6.2 Concrete composition and preparation

1. The contractor submits, at the start of the work, a proposal for the composition of concrete to the Engineer. Approval does not dismiss the contractor of his responsibility for the quality of the concrete. If during the work it appears that the composition of concrete must be adapted, this must be done in consultation with the Engineer. For this concrete mix composition test cubes must be manufactured in accordance with the VBC 1995 and NEN-EN 206-1 or equivalent Costs for concrete testing to be borne by contractor. Based on the results of the test cubes the project manger decides if the concrete composition can be considered suitable.
2. Aggregates shall comply with VBT (NEN8005). Deviations shall only be permitted after written approval by the Engineer
3. The type of cement to be used shall be Portland cement, Class A, NEN-EN 197-1 (or Portland cement Type I, ASTM).
4. The cement to be processed must be fresh to the mark, to the approval of the Engineer and must within four months after sending from the factory be processed.
5. Use of ready mix concrete is permitted. The Engineer can require a gradation analysis and a calculation of the mix composition of the concrete of each delivery.
6. At each delivery waybill shall be presented, on which the composition of the concrete with indication of possible additives, the slump and the departure time from the plant is indicated. This waybill shall be handed over before unloading to the Engineer.
7. For all concrete constructions the requirements, according to the NEN-EN 206-1, apply or equivalent.

6.3 Concrete qualities and types

1. The following minimum concrete qualities and types shall be used for the elements of the works as mentioned hereunder:
   - Blinding or backing C12/15, class XS1, (1500 PSI) according NEN/EC or equivalent;
   - For constructions C30/37, and according NEN/EC, unless stated otherwise.
2. All additions to the concrete need approval by the Engineer.

6.4 Concrete quality control

1. The quality of the concrete must comply with the regulations VBT 1995(nl) and NEN-EN 13791(en) or equivalent.
2. The slump of the mortar, for a certain construction component, defined with the “kegel van Abrams”, and as a rule shall lie between 60 and 80 mm. Slump tests shall be made in accordance to NEN-EN206-1 and NEN8005, whenever the engineer may require

   For the slump test, the following equipment is required:
   - 1 “Abrams cone”;

- 1 steel bar, long 600 mm, Ø 16 mm with rounded end;
- 1 flat steel plate 600 x 600 mm, thick 4 mm, with handle.

3. The contractor is responsible for making of the concrete cubes.

6.5 Preparation before placing concrete

1. The contractor has to verify the drawings on measurements. At least 5 days before every pouring, a written and complete pouring plan must be handed over for approval to the Engineer that shows:

   a. The date, the starting time and the expected duration of the pouring;
   b. The quantity of the concrete and the hourly capacity to process;
   c. The equipment to be used;
   d. The number of skilled employees with indication of their functions.

2. Before pouring any concrete, formwork and rebar have to be approved by the Engineer. The Engineer shall be notified at least 48 hours before pouring the concrete regarding changes in the pouring plan. The contractor remains, in all cases, responsible for the proper execution of concrete works. Also the contractor must ensure that all preparatory activities for pouring are completed a half working day in advance.

3. Before pouring the concrete:

   a. all the prepared components, the blinding, the rebar and formworks shall be hosed clean;
   b. the remaining wood and tie-wire must be removed;
   c. the formwork and blinding must be wetted with water.

6.6 Placing concrete

1. Transporting the mortar across prepared reinforcement shall be carried out with great care, to avoid damaging or displacing of the reinforcement.
2. The Contractor must ensure that no walking on prepared reinforcement takes place.
3. There may not be any pools of water in the formwork.
1. In case of ready mixed concrete, the mixer should turn on mixing speed for at least 2 minutes, directly before pouring the concrete.
2. Pouring concrete directly out of the mixer shall not be allowed, except with the permission of the Engineer.
3. Mechanical vibrators shall carry out compaction of concrete in order to avoid honeycombs. Mechanical vibrators shall be of the immersion or exterior type and shall be demonstrated to be of sufficient capacity. Vibrators must be present on the site in sufficient numbers and must be handled by qualified personnel, according to NEN 6722. There must be at least 1 spare vibrator, ready to use, present on the site. No concrete shall be poured unless said vibrators and personnel are present and available.
4. During pouring of concrete the Contractor’s supervisor must always be present.
5. No water may be added to the concrete mortar after delivery on the site.
6. The necessary measures have to be taken (elephant's trunk or other) to prevent free fall of concrete of more than 1.50 m.
7. Horizontal movement of already poured concrete by means of vibrators shall be avoided.
8. Pouring shall be maintained continuously for each component except for short interruptions of not more than half an hour.
9. The forming of honeycombs shall be prevented carefully. During rainfall no concrete mortar may be processed.
4. The poured concrete shall be covered with plastic and shall be kept wet with clean fresh (not salt) water.

6.7 Formwork

1. Formwork shall be of steel, aluminum or timber and shall be constructed to:

   1. Remain rigid during casting of the concrete;

    2. Be sufficiently watertight to prevent liquid loss during the hardening of the concrete;
    3. Allow easy removal, without shock, vibration or damage to the hardened concrete.

2. For formwork for concrete surfaces that will be visible, may be used:
    1. Either, steel;
    2. Or, concrete plex (“betonplex”) with a minimum thickness of 18mm;
    3. Or, plywood covert with 4mm concrete plex (”betonplex”).
3. Timber to be used for formwork shall have a minimum thickness of 25 mm. Timber that is not planed shall not be used. Formwork shall be completely even and smooth on the inside.
4. For concrete surfaces under the ground, deeper than 150 mm under the ground level, that will not be visible, wooden planks with a maximum of 100 mm width and a minimum of 22 mm thickness may be used. Butt joints shall be avoided as much as possible.
5. Contact of rebar with form oil shall be avoided at all times. Soiled rebar shall be removed immediately at the expense of the Contractor.
6. Concrete shall be even and built to the correct dimensions.
7. Formwork shall be removed in a simple manner without damaging the concrete surface. Time of removing the formwork according to the relevant regulations.
8. The hardened concrete may not have any sharp edges.
9. After every pouring the Contractor has to clean the formwork. Broken or damaged forms shall not be reused.

6.8 Installation of reinforcement steel

1. Bending of rebar shall not take place on the formworks, but must be done in advance. The rebar must be placed straight and tight to the correct measurements. At intersections the reinforcement bars shall be tied together with soft iron tie-wire, with the end of the wire turned towards the inside of the construction.
2. If the Engineer, having inspected the reinforcement, finds that insufficient rebar is placed, the additional steel will be at the expense of the Contractor.
3. Reinforcement shall not be straightened or bent for a second time. However, the straight parts can be used for short rebar or support steel.
4. Possible connections are indicated in the concrete construction drawings and/or are stipulated in consultation with the Contractor. The Contractor is obligated to supply rebar in suitable trade length approved by the Engineer.
5. The Contractor must design, produce and place supporting constructions concerning the upper reinforcement. The design for supporting must be submitted to the Engineer for approval. This approval does not release the Contractor from his full responsibility to build the project according to drawings.
6. Rebar which is not casted in for a period longer than 2 months must be covered with cement paste.
7. If the reinforcement appears dislocated at the removal of the formwork, the Engineer can reject the work which must then be redone at the expense of the Contractor.

6.9 Reinforcement steel

1. Reinforcement steel shall be of quality Grade 60 (ASTM), unless specified otherwise.
2. All reinforcement shall be free from loose scale, rust, oil, grease or any other harmful matter. If required by the Engineer, the reinforcement shall be thoroughly cleaned with wire brushes.
3. At least 4 spacing blocks per m^2 must be used to provide the necessary concrete cover to reinforcement. These spacing blocks must be cone- shaped, watertight concrete cubes, provided with 2 double bended wires, with the correct concrete cover.
4. The general concrete cover is 50mm, unless stated otherwise.
5. The quantity of the reinforcement must follow the calculations in the drawings, or be calculated at 125 kg/m3.

6.10 Fiber concrete pavement

1. Steel reinforcement replacement fiber: Adfil durus S400 and adfil fibrin XT (www.adfil.co.uk) or equivalent.
2. Fiber dosage: Durus S400 4 kg/m3 and fibrin XT 0.91kg/m3
3. Handling and mixing of fibers according to manufactures specifications.

6.11 Water

1. Water from other sources than tap water needs approval of the supervisor. The maximum chloride content is 100 mg Cl/l.

6.12 Curing and finishing

1. All the concrete must be flat, smooth and be finished immediately, except for driving surfaces which shall be rough instead of smooth.
2. Surfaces, which are not covered by formwork, must be finished with a ruler, directly after pouring of the concrete, unless indicated otherwise.
3. In rainy conditions measures must be taken against washing out of concrete on newly poured surfaces.
4. In case honeycombs have occurred despite careful execution during pouring, these must be repaired immediately at the expense of the Contractor after removal of the formwork in accordance with the instructions by the Engineer.
5. Repairing honeycombs without approval by the Engineer is not allowed. If the Engineer finds the honeycombs of such size, quantity of seriousness that greater measures are necessary, it can require partial or complete replacement of the affected component at the expense of the Contractor.
6. All the concrete drainage constructions must have a flow profile from mortar.
7. The inside and outside of the concrete constructions below ground level have to be treated with a heavy-duty bituminous paint.

6.13 Concrete roads

6.13.1 General

1. Below the new constructed concrete road like, a plastic sheet shall be placed on the sub-soil foundation.
2. Before installing concrete, the necessary excavations or filling operations shall be completed.
3. Before installation concrete, the subsoil shall be compacted. During compaction the degree of compaction achieved shall be monitored continuously. As soon as the required degree of compaction is achieved the compaction shall be stopped. The subsoil shall be compacted to 98 % of Modified Proctor Density.
4. The sub-soil foundation shall be dressed properly to the indicated height. The deviation value can be 1cm positive or negative.
5. The foundation is 25cm wider on both sides of the road construction.
6. To prevent dust particles blowing around in dry condition, the sub-soil foundations must be kept wet.

6.13.2 Testing

1. The compaction degree of the sub-soil and of the backfill materials shall be tested by means of nuclear measuring.
2. For each type of sub-soil and backfill material used, a Modified Proctor Test shall be carried out.
3. For each layer of sub-soil and backfill, one test per 100 m^2 shall be carried out, at the locations approved by the Engineer.
4. The Contractor shall ensure that the test locations are precisely indicated on a dedicated drawing to be provided to the Engineer as soon as the testing has been completed.

5. The density and humidity control methods shall be the “Troxler Nuclear” or the “Chicago Nuclear” (or equivalent). In case the required proctor density is not reached, the layer shall be compacted again until the required proctor density is reached.

7 ASPHALT PAVEMENTS

7.1 General

1. The general construction of the road shall be:
    a. Top-course, fine asphalt, thick 40 mm
    b. Binder, coarse asphalt, thick 60 mm

7.2 Adhesive layer (tack coat)

1. An adhesive layer shall be applied:
    a. between the binder and the asphalt top layer;
    b. against the concrete curbs.
2. The material for the adhesive layer shall be bituminous emulsion 40/50, the quantity shall be 0,5 kg/m2.
3. The adhesive layer shall be applied only when the existing surface is dry or contains sufficient moisture to get uniform distribution of the bituminous emulsion, and when the weather is not foggy or rainy. Immediately before applying the adhesive layer, the full width of surface to be treated shall be swept with a power broom and/or air blast to remove all loose dirt and other objectionable material.
4. The equipment used by the Contractor shall include a distributor for applying the bituminous emulsion, upon approval of the Engineer. The distributor shall be designed, equipped, maintained and operated so that bituminous emulsion may be applied uniformly on variable widths of surface at the specified rate.
5. The bituminous emulsion shall be heated to between 135 and 180 °C before application. If the emulsion has reached a temperature higher than 180 °C, it will be not allowed to use the emulsion anymore.
6. Unequal spraying of the adhesive layer, double spraying or not spraying of road sections shall be prevented.
7. The Contractor shall take measures to avoid pollution of surrounding pavements, curbstones, vehicles etc. caused by bituminous emulsions. In case of pollution of surrounding pavements or curbstones, contractor shall thoroughly clean such to remove any trace of the pollution.
8. In case the adhesive layer gets damaged, caused by rain, vehicles or otherwise, the layer shall be applied again.
9. Following the application, the coated surface shall be allowed to dry. The Engineer shall determine this period. The Contractor shall then maintain the surface until the surfacing has been placed. Suitable precautions shall be taken by the Contractor to protect the coated surface against damage during this interval.

7.3 Materials

1. Materials for fine asphalt and course asphalt will be in conformity of the specifications of articles 7.4.
2. Bituminous material for top layers shall consist of Trinidad Modified Asphalt (TMA) with a penetration of 40-55. TMA will be composed of 30-36 % Trinidad Lake Asphalt (TLA) and 64-70 % refinery bitumen with a penetration of 120-140. TLA is produced by Lake Asphalt of Trinidad & Tobago (1978) Ltd, Brighton La Brea, Trinidad, West Indies. The refinery bitumen will satisfy ASTM D 946.
3. Before gravels, sands or fillers are delivered, the contractor shall inform the Engineer in full (by writing), regarding the origin of the materials, the tests which are executed on the materials and the results of these tests which have to proof that the materials are suitable (compared with the specifications). If the information obtained does not make clear whether the materials are suitable, the Engineer will decide which complementary tests will be necessary. The costs for these complementary tests will be for the account of the Contractor.

4. The Contractor shall, for every load of bitumen delivered on the working site, provide the Engineer a certificate with the origin of the bitumen. The certificate shall give the following information:

   a. a declaration that the load bitumen is prepared out of mineral oil and a mention of the type of bitumen;;

   b. the name and the location of the refinery;

   c. a reference to the tests made by the manufacture/supplier

   d. the mixing temperature (equiviscosity temperature 170 mm2/s);

   e. the compaction temperature (equiviscosity-temperature 280 mm2/s).

5. In case this certificate is missing, or when reasonable doubt exists about the correctness of the certificate, the Contractor shall examine the load through an independent laboratory, to determine whether the bitumen complies with the requirements and to obtain the information required under par 4a, d and e.

6. The gravel, the sand and the filler for bituminous mixtures shall originate from homogeneous hard stones, the crushing factor shall be at least 0,70.

7. The rubble stones shall contain less than 5% flat pieces. To reach the required gradations, the gradation shall, if necessary be sieved or mixed with other gradations.

8. The different gradations shall be stored in such a way that no mixture between gradations can occur.

9. In case it is necessary to add to the filler from subsection 2. another filler, to arrive at the required gradation, the added filler shall need approval of the Engineer, a minimum of 75 % has to pass the sieve d = 0,063 um.

10. The asphalt shall exist out of a homogeneous mixture of aggregates, with a regular gradation, according paragraph 7.4. At the same time the amount of bitumen will be such that the grains are completely enclosed and that no “sweating” occurs.

7.4 Composition of asphalt concrete

1. Closed asphalt concrete 0/16 for top-layer with nominal thickness of 40 mm:

Sieve nr.	percentage remaining on the sieve
C 16	0-2
C 11,2	5-25
C 8	-
C5,6	30-55
C 2	57-63
C 0,063	93-97

   Asphalt bitumen percentage shall be between 6,0 and 6,4

2. Open asphalt 0/22 for base courses with nominal thickness of 60 mm:

Sieve nr.	percentage remaining on the sieve
C 16	0-4
C 11,2	30-40
C 8	-
C5,6	-
C 2	69-75
C 0,063	94-96

   Asphalt bitumen percentage shall be between 4,0 and 5,0

3. The Contractor may suggest an alternative mixture composition. An alternative mixture needs the approval of the Engineer.

7.5 Pre-research

1. The asphalt mixture to be processed shall be determined by the Engineer, based on the results of pre-research supplied by the Contractor. Hereto the Contractor shall submit a Marshall pre-study to the Engineer with design mixtures with:

   a. the desired bitumen percentage and also bitumen percentages which are resp. 0,5 % higher and 0,5 % lower then the desired percentage

   b. gradations which agree with respectively the upper limit and under limit of the allowed gradations.

2. The mixtures shall be tested on:

   a. Marshall stability;

   b. Percentage voids in mineral;

   c. Flow in mm.

3. The results of the pre-study shall be submitted to the Engineer, at least 1 week before starting the production.

7.6 Design criteria for mixtures

1. The to be selected mixtures shall comply with the following design requirements:

	asphalt-concrete for base courses	asphalt-concrete for top layers
Marshall stability, minimum	5.000 N	7.500 N
Voids in mineral, maximum	7 %	6%
Flow	1.5-3 mm	2-4 mm
Marshall quotient minimum	2500 N/mm	3000 N/mm

7.7 Asphalt preparation

1. The materials for the preparation of asphalt shall be stored separately and clearly marked close to the asphalt installation. Storage shall be done in such a way that mixing and pollution of different types of materials is avoided.
2. The storage capacity for sand and gravels shall be such a way that at least for one working day, production can be continued uninterrupted while producing at maximum capacity.
3. In the asphalt mixing installation, the dosage and the mixing of materials shall happen in such a way that a homogeneous product with a constant structure and gradation is produced.
4. If so requested, the Contractor shall supply the Engineer with a report of the composition of the batches (mixtures), based on the amounts and the gradation of the materials.
5. Dosage of the different types of aggregates shall be done by way of separate compartments, taking their moisture content into account.
6. The drying drum has to be heated in such a way, that materials are being dried and heated, and no pollution occurs. The drying progress of the mineral aggregate shall be in such a way that the moisture content of the asphalt mixture is not higher is then 0,1% (mass percent), at the point where it leaves the mixer.
7. The heated materials shall be stored in heated silos, trough sieves to ensure the required gradations.
8. The temperature of the binder in the bitumen heater and the temperature of the mineral aggregate in the heated silo shall be measured. The temperature of the asphalt concrete shall be checked regularly immediately after mixing.
9. The temperature of the aggregate in the heated silo shall be between 200 °C and 160 °C. Asphalt concrete with a temperature higher than 190 °C may not be processed.

10. The asphalt-concrete shall be prepared in separate batches. The filler, in case this is added separately, and the other aggregates shall be weighted with separate scales. The weighting of the aggregates has to be done in at least two separate gradation fractions. The binder shall be weight or measured. In case of measuring, the volume of the binder shall be calculated based on the density of the binder and the temperature during the measuring.
11. The time of the mixing process, calculated from the moment that all materials are in the mixing bin till the moment of unloading the bin, shall be given to the Engineer if so requested. The mixing time shall be the same as the recommendation of the manufacture.
12. The use of gas-oil as anti- adhesive remedy in the bin of the asphalt installation shall be limited to a regular spraying on the surface, with a maximum of 50 g/m2.
13. The installation shall be installed in such a way that in a simple way representative samples can be taken from the separate silos.

7.8 Processing

1. To comply with the requirements for layer thickness, profiles and smoothness, the subgrades and base courses shall be filled up or repaired, as far as required.
2. In case of rainfall, no asphalt may be applied. Asphalt concrete may not have a temperature below 145 °C when it is installed. Asphalt with a temperature below 145 °C may not be processed.
3. During transportation, a tarpaulin shall cover the asphalt.
4. Profile deviations shall be corrected directly behind the finishing machine, by removing the surplus asphalt or to fill up the lower parts with fine asphalt-concrete mixture. Water will not stay on the asphalt deck because of profile deviations. A maximum deviation under a 3 meter straightedge of 5 mm is allowed.
5. Rolling shall be continued until further consolidation is not possible. Adhesion to the under layer shall be secured. The speed of the roller shall be constant and slow.
6. In case greasy spots occur during the rolling-process, these spots shall be removed immediately by sprinkling and rolling of asphalt. Rough spots in the surface, also shall be erased immediately by sprinkling and rolling of asphalt.
7. The asphalt at the length-seams and the cross-seams shall be applied with an overlap and enough over height. Surplus or rough aggregate shall be removed. It is not allowed to spread the released material over the top-layer.
8. Before installation of a connecting course, cross – joints shall be coated with an adhesion coat.
9. In case asphalt paving works shall be executed after dark, adequate artificial lighting has to be provided.
10. The required compaction of the asphalt shall be at least 98 % .
11. Opening the road for traffic after finishing the asphalt works, can only be done after explicit and written approval is obtained from the Engineer.
12. At the end of the maintenance period, level differences, which might have appeared in the newly constructed pavement, shall be repaired, and restored to the correct profile. These repairs are for the account of the Contractor.

7.9 Testing

1. The Contractor shall be able to execute at least the tests as described below in a laboratory equipped with the proper equipment:
   a. Sieve analysis
   b. Marshall test
   c. Density of mineral aggregates
   d. Bitumen content of hot asphalt
   e. Compaction degree of asphalt
   f. Density of asphalt test specimens
   g. Density of asphalt mixtures

2. Tests as described above under a to g above shall be executed in accordance with “Eisen Rijkswaterstaat 78”.

3. The contractor shall drill cores of the completed asphalt pavements at his expense for regular quality control and eventually, control for completion certification. The Contractor will have a core drill machine available on site.

4. With the stipulations under subsections 2 and 3 above, the Contractor shall prepare at least the following studies and tests:

   a. Pre-study of mixture compositions, not older than one year;

   b. quality control during production and installation .

5. Quality control during production and installation shall include at least the following tests and verifications:

   a. Sieve analysis for determination of gradation of materials;

   b. Per day of production, at least one extraction and gradation analysis of a sample taken at the asphalt plant;

   c. Compaction of asphalt concrete installed in the works.

6. All tests and research have to be executed under the direction of an experienced laboratory technician, at the expense of the Contractor. The Engineer will at all times have access to the laboratory and be allowed to be present at all testing.

7. The Engineer is authorized to execute tests himself, with the use of the laboratory equipment available to the Contractor. The Contractor laboratory personal or the personal of an external laboratory if this is employed by the Contractor, shall provide assistance at the execution of tests if so requested by the Engineer.

8. Results of all tests will be handed over within 48 hours after execution of the tests.

9. Cores:

   a. The Contractor will drill and extract cores of the asphalt pavements, two cores diameter 10 cm, per 200 m road length, except when otherwise instructed by the Engineer.

   b. Core holes have to be filled with asphalt concrete with a temperature satisfying the requirements of paragraph 7.8 above, and compacted, in layers. The core hole walls shall be coated with an adhesion coat before filling of the holes.

   c. Pairs of cylinders have to be marked 1A, 1B, 2A, 2B etc, and stored for future testing if so required.

   d. Testing shall consist of:

      i. minimum and maximum thickness of layers

      ii. composition of asphalt concrete, percentage bitumen, percentage voids, compaction

7.10 Foundation

7.10.1 General

1. Below the new constructed asphalt road a foundation, like on the drawings, shall be constructed with a minimal thickness of:

   a. 50 mm of soil cement on the existing road.

   b. 300 mm of soil cement in other locations.

2. Before installing the asphalt, the necessary excavations or filling operations shall be completed.

3. Before installation asphalt, the subsoil shall be compacted. During compaction the degree of compaction achieved shall be monitored continuously. As soon as the required degree of compaction is achieved the compaction shall be stopped. The subsoil shall be compacted to 98 % of Modified Proctor Density.

4. The sub-soil foundation shall be dressed properly to the indicated height. The deviation value can be 1cm positive or negative.

5. The foundation is 25cm wider on both sides of the road construction.

6. To prevent dust particles blowing around in dry condition, the foundations shall be kept wet.

7.10.2 Testing

1. The compaction degree of the sub-soil and of the backfill materials shall be tested by means of nuclear measuring.
2. For each type of sub-soil and backfill material used, a Modified Proctor Test shall be carried out.
3. For each layer of sub-soil and backfill, one test per 100 m^2 shall be carried out, at the locations approved by the Engineer.
4. The Contractor shall ensure that the test locations are precisely indicated on a dedicated drawing to be provided to the Engineer as soon as the testing has been completed.
5. The density and humidity control methods shall be the “Troxler Nuclear” or the “Chicago Nuclear” (or equivalent). In case the required proctor density is not reached, the layer shall be compacted again until the required proctor density is reached.

8 CURBS

8.1 General.

1. The works as specified in these specifications consist mainly of the following:
   a. The drainage curb of the road;
   b. The concrete curb of the road;
   c. The small curb for the sidewalks.

8.2 Execution

1. The curbs can be either prefabricated or cast in place
2. Curbs shall be placed on a tamped concrete basis of at least 0,10 m and supplied with a concrete back support.
3. Concrete works according to chapter 6 Concrete Works of these specifications.
4. Drainage curbs slope towards discharge point, preventing standing water.
5. Curbs shall be lowered locally to enable access to lots in consultation with the Engineer. Lowering of the curbs shall be done in such a way that drainage is maintained.

8.3 Material specifications

1. Dimensions of the curbs as stated on the drawings.
2. Prefabricated curbs have to be applied with joggle joints (hol en dol);
3. Curb concrete minimum quality: in accordance with 6.3 Concrete qualities and types;
4. The backing of concrete has a minimum quality in accordance with 6.3 Concrete qualities and types.

9 GREEN ZONE / LANDSCAPING

9.1 General

1. Under verges is understood: the relatively flat area next to roads, sidewalks and drainage constructions.
2. Landscaping has to be executed if and as indicated on drawing.
3. Verges
   shall be finished tightly and under profile.
4. The upper 0.10 m of verges shall be stripped of parts larger than 30 mm.
5. The verges shall be maintained 2 times a month for 6 months by the contract

10 SURPLUS MATERIALS

10.1 General

1. Disposal of materials shall be as follows:

   A. Surplus excavated material has to be used as fill at the valley side of roads if possible.

   B. All other surplus materials, which have no value (to the judgment of the Engineer) shall be removed and disposed of by the Contractor to the garbage dump on the Great Salt Pond, or another location designated by the Engineer, at less or equal distance as the Great Salt Pond dump area. Dump and transport costs are at the expense of the Contractor.

2. Sludge spilled on public road must be removed immediately. The Engineer can instruct the contractor to remove the sludge directly, if he considers being necessary. The sludge must be transported by the contractor, to the garbage dump on the Great Salt Pond.

3. No materials may be burnt in the project area.

11 UTILITY CABLES, WATERLINES, FIRE HYDRANTS AND STREET LIGHTS

11.1 General

1. The contractor is responsible for the installation of the electricity and telecommunication cables, waterlines, meter walls and fire hydrants
2. The contractor is fully responsible for the coordination between contractor and the utility companies.
3. The Contractor shall complete the installation of the underground utility lines and of the meter walls ready for connection by the relevant utility company, in conformity with its requirements and to its approval.

11.2 Water installation

11.2.1 Work description

1. The works as specified in these specifications consist mainly of the following:
   a. Installation of the main supply pipes (HDPE2" & HDPE4") with the pressure regulators for the GEBE meters.
   b. The complete installation of the fire hydrants.
2. The works shall be carried out with due observance of the requirements of the utility company, other applicable local regulations and in accordance with NEN 1006 *"Algemene voorschriften voor leidingwaterinstallatie".*

11.2.2 Water lines

1. HDPE water lines
   a. Brand: DYKA or equivalent
   b. Type: HDPE, SDR13.6, PN10, KIWA keur
   c. Installation: According to the manufacturer's specifications.

11.2.3 Fire hydrants

1. The approximate location of the fire hydrants as indicated on the drawings. The exact location of the fire hydrants at the approval of the Engineer.
2. The Contractor has to submit fire hydrant catalogue cuts.
3. Fire hydrants shall meet the requirements of the Fire Department and be installed accordingly. For additional information see the fire department web-site: http://www.brandweersxm.net
4. All fire hydrants should be placed in such a manner that fire trucks have immediate access to the hydrant at all times.
5. No type of object(s) of any kind should be placed around the fire hydrant which could restrict its immediate use.
6. Each connection of the fire hydrants to the main water lines shall be provided with a non-return valve.

11.2.4 Testing:

1. All pipes must be pressure tested continuously during the construction process to detect defects.
2. Before the water installation is put into use it must undergo a pressure test. the test shall be done according to the method described in the NEN1006 (chapter 2.3).
3. If the installation doesn't work properly the contractor needs to replace the defective components before the project finished at no extra costs.

11.3 Street lights

11.3.1 Light poles

1. Brand: KAAL masten

2. Type: CVL 8.0 PTU48 EU/60- GST168
3. Top type: EU 1.23360 1000/15/5-PTU48
4. Light fixtures type: LED LSR3 (75W) supplied by Profilic, or equivalent

11.3.2 General

1. The feeder cable for connection of masts shall be a 4-core direct burial cable. Core insulations shall be black, light blue, dark brown and black/white. Core dimensions as per the specifications of the manufacturer/ supplier.
2. The cable to the light fixture shall be a 3-core cable. This cable to be secured with fuses behind the hatch-opening as per NEN 1010.
3. Electrical cables shall be of the brands DRAKA, PIRELLI or equal (to be demonstrated by test certificates, to the approval by the Engineer).
4. The feeder cable between masts and the cable to the light fixtures each have to be connected to a ground in the mast.
5. Contractor shall have the installation presented for inspection by a recognized installation contractor.
6. Contractor shall further take the following into account:
    a. Inspection costs
    b. Connection costs of the lights to the main distribution room
    c. Purchase costs of switch cabinets
7. Masts have to be installed in accordance to the manufacturer's specifications.
8. The feeder cable between two masts has to consist of one cable length, without splices.
9. Only one splice per 500 meter cable length is permitted.
10. On top of the cables a warning band has to be placed, with English text, type CWBE, with a width of 100 mm and a thickness of 0.15 mm.
11. The location of splices to be indicated on the As-built drawings.
12. Masts have to be placed so that the inspection hatch is on the sidewalk-side.
13. After completion of the works three keys for the mast hatches have to be delivered to the Engineer.
14. The installation has to be operated by a photocell, location to the approval by the Engineer.
15. Before start of the execution the contractor shall submit a detail design drawing for approval to the Engineer.
16. Execution and design shall be in accordance to NEN 1010.

11.4 Moving and reuse

1. Existing street lights, meter walls and fire hydrants et cetera that have to be removed shall be stored for reuse within the project, if possible.
2. For Mildrum Road (parcel i): the existing utility trench, including all cables, pipes, warning band and other provisions shall be moved from the hill side of the road to the valley side, in conformity with the drawings.
3. For the side roads (parcel ii-vi) the existing utility trench, including all cables, pipes, warning band and other provisions shall remain in place if possible and shall only be moved if necessary. A provisional sum for adjustable quantities of the digging of test trenches / pits and the moving of utility trenches is included in the tender.

12 SLEEVE PIPES

12.1 Sleeve pipes

1. In accordance with the specifications and the applicable drawings, sleeve pipes shall be placed.
2. The sleeve pipes must be assembled out of schedule 40 PVC-tubes.
3. Connections between tubes shall be made by means of PVC sockets with rubber rings, in conformity with the manufacturers requirements.
4. The sleeve pipes must be provided with a nylon pull wire, with a tensile strength of 250 kg over the complete length of the sleeve pipe, and an extra length of 2.00 m at both sides.
5. The length of sleeve pipes must be of such size, in consultation with Engineer, so that the sleeve pipes are extending 0.50 m outside the side of the pavement.
6. Road crossings to be inspected and approved by the Engineer and / or GEBE.

12.2 Execution

1. Excavations for the installation of sleeve pipes may only be carried out mechanically. Excavations near existing cables and/or piping must be carried out manually.
2. The sleeve pipes must be placed under a slight slope, so that water is drained automatically.
3. Sleeve pipes for cables must be placed as much as possible at a right angle to the axis of the road, however not at a lower angle than 60°.
4. The sleeve pipes must be placed at a mutual distance of at least 0.05m and must be embedded in a layer of clean white sand of at least 0.05m on all sides.
5. After the installation and immediately prior to backfilling, each of the sleeve pipes shall subsequently:
    1. Be fully cleared from any obstacles and dirt;
    2. Be marked and numbered, both on the exterior and interior ends on either side, by means of a waterproof felt-tip pen, clearly indicating the type of cable or line for which the sleeve pipe is meant;
    3. Be closed off with fitting PVC caps that are duly attached with tape.

12.3 Installation of sleeve pipes underneath existing roads

1. For installation of sleeve pipes under existing roads, the pavement must be sawed in a straight line at a right-angle to the axis of the road.
2. The moment of applying the sleeve pipes underneath the road must be determined in consultation with the Engineer, the road manager and the utility companies.
3. The pavement must be repaired immediately. If necessary a temporary pavement may be applied after consultation with the Engineer and the road manager. Temporary pavements are allowed for up to 7 days (provided that it shall be maintained properly), unless compromised differently. After this period the slot must be provided with a permanent pavement.

13 ROAD MARKINGS

13.1 General

1. Road markings shall be applied with high quality road paint and prefabricated strips, 2-component reflecting, cold-curing acrylate based mass free from solvents for manual application, or equal at the approval of the Engineer.
2. Road markings shall be applied within 2 weeks after installment of pavements, unless agreed otherwise with the Engineer.
3. The road markings shall be handled and applied in conformity with the manufacturers specifications and the Standaard RAW bepalingen, 2015 and ASVV 2012.
4. The staking out of the road markings have to be inspected and approved by the Engineer, before road markings are allowed to be applied.
5. The road markings shall be applied with a quantity of at least 4.5 kg/m2 and a thickness of at least 3.0 mm.
6. For immediate retro reflection 200 gr/m2 glass beads shall be applied afterwards.
7. The road surface shall be clean and dry and free of old markings and loose material upon application of the road markings.
8. Road markings shall have neat, straight and smooth lines.
9. Renewed or repaired road markings shall be applied in the original location, unless specified otherwise in the drawings or by the Engineer.
10. Wrongly applied road markings shall be removed immediately. Resulting damage to the pavement shall be repaired by the Contractor immediately.
11. Road markings shall be applied by hereto qualified personnel.

14 TRAFFIC SIGNS

14.1 General

1. Traffic signs shall be installed on poles, which are made from seamlessly drawn steel pipes. The outer diameter of the lower part of the tube shall be ø 75 mm, the outer diameter of the upper part shall be at least ø 48 mm.
2. The poles shall be fitted with a double ground anchor.
3. The entirety shall be hot dip galvanized in accordance with NEN 1461.
4. The poles shall be placed 0.80 m deep in the ground and be so long that the bottom of the traffic sign is 2.10 m above the ground level.
5. The poles shall be coated black and white c.t.c. 40 cm above ground level.
6. The road signs shall comply with NEN 3381 regarding requirements for colors, retro-reflection and material requirements. The front sides shall be completely retro -reflective.
7. The back of the signs shall be muffled.
8. The signs shall be made of tension free 2mm thick aluminum and have double-bend edges with drainage hole.
9. For each sign, 2 hot dip galvanized fastening brackets (so called hinge brackets, Dutch: scharnierbeugels) shall be supplied, complete with bolts and nuts or 2 stainless steel ties.
10. The location of the traffic signs on drawings is approximate. The exact location will be determined during the execution in consultation with the traffic police and the Engineer.
11. Sharp curve signs shall be at least 600x1180 mm.

15 GUARD RAIL

15.1 General

1. Guard rails shall be installed, as indicated on the drawings.
2. Contractor to submit shop drawings and the necessary calculations of the aforementioned guard rails for stability to the approval of the Engineer.
3. Any damaged guard rails shall be rejected and replaced at the Contractors expense at the Engineers discretion. This applies to damages to both the structure and appearance of the guard rails.
4. Transport, storage and installation shall be in conformity with the RAW 2015 and the manufacturer's specification.

15.2 Guard rails

1. Installation of the guard rails include installation of all parts needed for the construction as well as aligning.
2. The guard rails mainly consists of:
    a. Hot dip galvanized steel plates type A, supplier Eurorail, Hasselt or equal with steel plates with standard length 4.00 m. Connect to supports with hot dip galvanized lock bolts M12.
    b. Supports: hot dip galvanized INP-160 steel profiles, length at least 1.40 m, c.t.c. 2.00 m
    c. In-situ concrete footings for supports, with dimensions 0.5x0.5x0.5 m, concrete quality C12/15. Support are poured 50 cm into concrete.
    d. Endings shall be equipped with hot dip galvanized end pieces.
    e. Footings and anchors to install supports on concrete structures, where necessary.
3. Damaged parts of the guard rail and parts with corrosion shall be cleared of rust, grease and dirt and treated with zinc compound, to the approval of the Engineer
4. Hot dip galvanization shall be in accordance with NEN 1461.
5. Concrete footings shall be in conformity with chapter 6.
6. Supports and plates shall be placed perpendicular to the edge of pavement.
7. Deviation of the support distance to the approval of the Engineer.
8. Directly after placement of the bolts, nuts shall be tightened completely.
9. It is not permitted to skip, enlarge or make additional bolt holes.
10. Supports shall be placed vertically.
11. Aligning shall be done as follows:
    a. Loosen bolts and nuts to allow alignment;
    b. Adjust guard rail to achieve smooth vertical and horizontal alignment
    c. Tighten bolts and nuts in such a way that contact areas connect.
12. If raising or lateral moving of supports is necessary for the alignment, the support and footing shall be completely removed and reinstalled and the resulting hole shall be filled and compacted appropriately.
13. The zinc compound shall be composed appropriately for this purpose.

16 HANDRAILS

16.1 General

1. Aluminium rails shall be installed on retention walls, as indicated on the drawings.
2. Aluminium rails shall be in accordance with NPR-CEN/TR 1317-6
3. Aluminium posts shall be installed c.t.c. 2000 mm.
4. Contractor to submit shop drawings and the necessary calculations of the aforementioned rails for stability to the approval of the Engineer.
5. Any damaged rails shall be rejected and replaced at the Contractors expense. This applies to damages to both the structure and appearance of the rails.

16.2 Aluminum rails

1. Manufacturer: TECHNAL (www.technal-int.com) or equivalent to the approval of Engineer.
2. Model: banisters round bars or equivalent to the approval of Engineer.
3. Dimensions: minimum height 1.0 m.
4. Finishing: factory primed, powder coated.
5. Installation shall be in conformity with the manufacturer's specification

17 DRAINAGE WORKS

17.1 General

6. The drainage works as specified in these specifications consist mainly of the following:
    a. Open concrete gutters;
    b. Open concrete gutters with grating;
    c. Closed concrete gutters;
    d. Culverts
7. All concrete drainage structures shall be in accordance with chapter 6 Concrete Works of these specifications;
8. All drainage shall be finished water tight to prevent erosion of the subsoil.
9. All drainage shall be finished tightly to prevent standing water and obstacles. U-Gutters with a grade over 7% shall not be finished smoothly
10. All drainage shall slope towards the discharge point. Under no circumstances are counter slope or flat sections allowed to prevent standing water.
11. All drainage shall be equipped with a flow profile.

17.2 Prefab elements

1. The Contractor shall provide shop drawings and detail engineering calculations of any prefab elements for approval by the Engineer.
2. All the prefab elements shall have a number corresponding to the shop drawing.
3. The contractor shall provide a time schedule with the exact pouring date of the elements.
4. Only after 3 weeks of curing the elements may be transported from the yard to the site.

17.3 Gratings

1. The gratings and corner profiles shall be of hot dip galvanized steel
2. Hot dip galvanization in accordance with NEN 1461.
3. The grating shall comply with traffic class 45 strength requirements.
4. The grating shall have a mesh opening of 50 mm ctc.
5. Any welds that are made after construction to the brackets and gratings shall be finished immediately with zinc compound and epoxy.
6. All gratings shall be locked to a corner profile, which shall be poured in with the concrete of the gutter.

17.4 Inspection

1. The Contractor shall clear the complete drainage of all obstacles and dirt before inspection.
2. If, during inspection, the Engineer has reasonable doubts about the slope or tight finish. The Contractor shall be required test the drainage slope or finish by supplying water to the drainage at the Contractors expense.

18 SAFETY, SECURITY AND HEALTH

18.1 General

1. The Contractor presents a final safety plan to the Engineer for approval. The Engineer shall review the plan with regard to the following items:
   a. Phone list for who to ring at calamities;
   b. Personnel protective equipment (helmet, boots, clothing, glasses, ear protection, et cetera);
   c. Safety instructions to personnel;
   d. Presence of first-aid kit at site office and person(s) responsible for first-aid;
   e. Provisions against dust;
   f. Communication with and accessibility for police, ambulance, fire department;
   g. Cleaning up the working area.
2. The Contractor shall not start any activity before the Engineer approves the safety plan.
3. The Contractor shall take care that the accepted safety plan is observed at all times.
4. During the execution period the Contractor is responsible for the safety of all people concerned in the project area, including third parties.
5. The Contractor must notify the Engineer without delay of any accident, of whatever nature, that may occur in connection to the project and provide the Engineer with all relevant information.

19 GUARANTEE OF REALIZED WORK

19.1 General

1. The Contractor guarantees the works for a period of three year after the date of delivery. After this period of guarantee, the entire project must be in such a state of satisfaction, that the project can be expected to remain in satisfactory condition for several years, without major repairs, while also taking in to account the experience with the project during the period of guarantee.

2. Possible defects in the construction, appearing in the period of guarantee, such as cracks, poor spots, collapses; disintegration of materials, etc. must be repaired on first demand of the Engineer, within a period determined by the Engineer.