Research Article

Investigating the Effect of Climate Change on Stormwater Networks: Capital Ankara Case

İklim Değişikliğinin Yağmursuyu Şebekeleri Üzerine Etkisinin Araştırılması: Başkent Ankara Örneği

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Abstract

In this study, we applied the analysis results of past and future variations of extreme precipitation and land use/cover changes in Ankara province for a newly built stormwater network of a pilot study area in Etimesgut, Ankara. We investigated the performances of the system under current and changing extreme rainfall conditions and different approaches such as stationary and nonstationary extreme precipitation assumptions. The system operated in a satisfactory state and it can be said that according to climate change projections for the extreme rainfall, the maximum volume that the system face will not exceed baseline design criteria throughout the projection period. Combination of changing climatic and land use/cover conditions also reveal a satisfactory performance for the baseline design which used 15 minutes storm duration and 2 years return period rainfall intensity and a runoff coefficient of 0,8 as design input. On the other hand, the system may fail under the loads derived separately or together with longer storm duration (such as 30 minutes or more) or higher return periods (such as 5 years and more) that is computed from stationary and nonstationary observed data analysis which is a preferred design input for such a critical facility and area.

Keywords: *Climate change, stormwater, extreme rainfall*

Öz

Bu çalışmada Ankara İlinin geçmiş dönem ve gelecekteki aşırı yağış ve arazi kullanımı/örtüsü değişim analiz sonuçları, Ankara İli Etimesgut ilçesinde yeni inşa edilen bir pilot çalışma alanının yağmur suyu şebekesi için uygulanmıştır. Sistemin performansı, mevcut ve değişen koşullar ile durağan ve durağan olmayan aşırı yağış varsayımı gibi farklı yaklaşımlar altında araştırılmıştır. Sistemin iklim değişikliği projeksiyonları altında yeterli ve beklenen servisi sağlayabileceği ve aşırı yağış için yapılan iklim değişikliği projeksiyonlarına göre sistemin temel tasarımda öngörülen maksimum kapasiteyi projeksiyon süresi boyunca aşmayacağı söylenebilir. Değişen iklim ve arazi kullanım/örtü koşulları birlikte değerlendirildiğinde sistem, 15 dakikalık yağış süresi ve 2 yıl tekerrür periyodu ile akış katsayısı olarak 0,8 kullanılan temel tasarım için tatmin edici bir performans ortaya koymaktadır. Öte yandan sistem gözlem verisi ile yapılan durağan ve durağan olmayan analizlerden elde edilen daha uzun yağış süreleri (30 dakika veya daha fazla gibi) veya daha yüksek tekerrür süreleri (5 yıl ve daha fazlası gibi) ile ayrı ayrı veya birlikte hesaplanan yükler altında beklenen servisi sağlayamayabilir ki bu süreler kritik bir altyapı tesisi ve alan için tercih edilen bir tasarım girdisidir.

Anahtar kelimeler: *İklim değişikliği, yağmur suyu, a*şırı *yağış*

Introduction

Climate change and its potential impacts gained importance due to projected changes in temperature and precipitation which can significantly affect the hydrological cycle, land use, extremes and the related infrastructure (Elshorbagy et al., 2018). The alterations in land cover and rainfall characteristics that cause urban flooding increase the need for impact assessments on design and management of stormwater networks (Hailegeorgis & Alfredsen, 2017). Stormwater networks can be sensitive to climate change, in particular to extreme rainfall events as they are one of the main variables for design. On the other hand, the design and expected performance of stormwater infrastructure become questionable with the changing climate (nonstationarity) because the conventional design (stationary assumption) may not consider the changing climatic conditions (Rosenberg et al., 2010).

Impacts studies on urban storm water drainage systems gain high attention at locations where these systems are vulnerable. Osman (2014) analyzed future rainfall characteristics and modelled the output to explore the impacts of climate change on the urban drainage system in the Northwest of England during the 21st Century. The results implied that potential changes in rainfall intensity in the future are expected to alter the performance and serviceability of the system, causing more challenges such as surface flooding and increase in surcharge level in sewers. Bahadur et al. (2016) reports that stormwater management systems will be overwhelmed by the rising intensity of rainfall, and extreme events will damage the infrastructure systems in specific regions of Asia. Hence, urging for urban climate change resilience is considered.

Urbanized areas become more vulnerable to flood hazard under conditions of high precipitation intensity (Sun et al., 2011). During the last century the number of people living in urban areas has globally increased rapidly. At the beginning of the twentieth century, only 14% of the world population lived in urban areas, today 55% of the global population resides in urban areas (United Nations [UN], 2018). This increase in urban population is expected to continue until at least 2050 and reach 68% (UN, 2018). Since urbanization is one of the main consequences of urban population growth, it is expected that increase in the urban population lead to urbanization over that period. As a consequence of urbanization impervious surface area increases and this in turn brings significant effects on the hydrological cycle in the urban areas. Increased proportion of impervious surface results in shorter lag times between onset of precipitation and end up with higher runoff peaks and total volume of runoff (Shuster, Bonta, Thurston, Warnemuende, & Smith, 2005). The conversion of pervious (permeable) land to impervious (non-permeable) surfaces changes the hydrologic characteristics of the landscape by reducing infiltration into the soil and evapotranspiration from vegetation which results in a dramatic increase in the rate and volume of stormwater runoff (Guidelines for NYC, 2012).

The rising trend of rainfall intensities as a result of changing climate or variability is a challenge for infrastructure systems that use particular return levels and periods as design parameters (Zhou, 2014). Furthermore urbanization raises the impervious areas, changes land cover types resulting in increase in discharge, volume, and frequency of floods which together with climate change induced intensified rainfalls will have amplified effects on urban stormwater systems (Thakali et al., 2016). The adaptation process is not as fast as the changing environmental conditions and this also increases exposure to floods therefore, as a result vulnerability is increased (Kundzewicz, 2003; Trenberth, 1998). In addition to extreme rainfall events and urbanization effects, inadequate investment and maintenance of infrastructure further increases exposure to flooding (Simonovic et al., 2016). While older stormwater networks have not been designed to withstand extreme rainfall events, urbanization further increases impervious surfaces and exacerbates the effect of extreme rainfall events.

In the flooding observations of last 20 years, heavy rainfall and flash flooding caused significant damages to the properties and even loss of life in Ankara, the Capital of Turkey (Supplementary Document of Official Letter of Turkish State Meteorological Service [TSMS], 2017a). Therefore, a strong need has emerged to study extreme precipitation events to reveal potential frequency and intensity alteration under changing climate conditions, reveal the effect of land use/cover change and investigate the impact of these changes on the urban stormwater networks. The current and projected IDF analysis of Ankara with transient climate change effects was already investigated in Oruc et al. (2019). However, their impact on storm sewerage system has not been carried out so far not only in the capital of Turkey, Ankara but also any other province throughout the country. It'll be a pioneer to document the response of a storm water infrastructure to changing climate condition.

In this study the results of extreme precipitation and land use/cover change analyses from Ankara province are applied for a newly built stormwater network of a pilot study area in Etimesgut, Ankara. The effect of changing climatic and land use/cover conditions are studied in pilot study area to incorporate climate change into urban stormwater network design by using stationary and non-stationary GEV models for extreme rainfall analysis together with land use/cover change. Performance of the system is investigated by the analysis in which stationary (St) and nonstationary (Nst) maximum storm intensities are converted to peak discharges using rational formula and they are conveyed from the current pipeline system under current and future conditions. As a result, the resulting fullness capacity of the system with and without revision is compared with the baseline (current). Finally, the basic economic assessment is also performed after suggested revisions to the system.

Methodology

Data and Study Area

The annual maximum precipitation data of 5, 10, 15, 30 minutes and 1, 2, 3, 6 hours duration is used for the observation period (1950-2015) while the data of 10 min, 15 min, 1 hour and 6 hour durations is used for the projection period in extreme value analysis. The future data consists of daily estimates from three global climate models (GCM) namely HadGEM2-ES, MPI-ESM-MR, and GFDL-ESM2M based on RCP 4.5 and RCP 8.5 emission scenarios. A fine-scaled regional climate model (RCM) coupled to these GCMs provides the daily precipitation of 2015-2099 period. As it is documented in Oruc et al. (2019), the maximum precipitation data for sub daily scales (10 min, 15 min, 1 hour and 6 hour) were obtained by applying a disaggregation method to projected daily precipitation values. Table 1 the location of meteorological stations and nearest RCM grids to these stations together with their altitudes are given.

Table 1

Projection Data Stations & Grids (Supplementary Document of Official Letter of TSMS, 2017b)

No	Station	Grid	Station			Grid		
			Latitude	Longitude	Altitude mt	Latitude	Longitude	Altitude mt
17129	Etimesgut Airport	2733	39,9558	32,6854	806	39,9661	32,6608	1028
17131	Ankara Güvercinlik Airport	2733	39,9343	32,7387	820	39,9661	32,6608	1028

Stationary and nonstationary rainfall return levels (in mm) for return periods 2, 5, 10, 25, 50, 100, 200 years are derived for observed and projected data for extreme rainfall time series of the sub-hourly and hourly annual maximum data.

Extreme precipitation analysis results are transformed and applied to test the runoff and hydraulic performance of an actual stormwater network of a Railway Maintenance Complex (Figure 11) located in Etimesgut, Ankara Province (Turkish State Railways, 2018).

Figure 1. Railway Maintenance Complex in Etimesgut,Ankara.

Ankara is located in the northwest of Central Anatolia. The city is like a pot surrounded by four mountains of Anatolia Plateau with an altitude of 850-1000 meters. A population of 5.3 million people (Turkish Statistical Institute [TURKSTAT], 2016) lives in the capital Ankara and 88% of the population lives in the city center (Governorate of Ankara, 2018). The growth of Ankara displayed a typical example of modernization efforts at the beginning and later (the second half of this century) it showed the uncontrollable expansion and transformation of the city with expanding squatter areas due to heavy migration. Then, the city shaped by urban regeneration projects in the last decades (Batuman, 2013).

The pilot study area is mostly flat and has an average altitude of 796 meters above mean sea level. The rail service maintenance plant has border on the northern side with the Ankara River, on the southern side with the existing railway line, on the eastern side with the Sugar Factory, and on the western side with the E89 road (Figure 2).

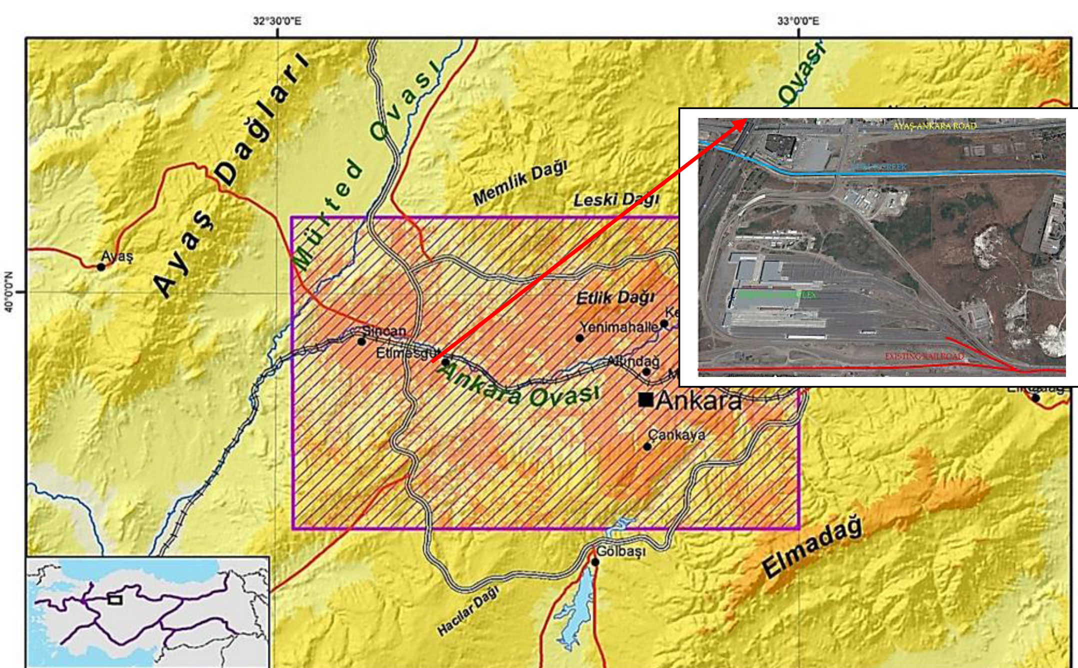

Figure 2. Location of Ankara (Yilmaz, 2013).

The new rail service maintenance plant is located approximately 20 km west of the center of Ankara. The stormwater network of this area consists of drainage channels and drainage manholes. The drainage manholes are connected by circular pipes; the drainage channels and the circular pipes discharge the flow in two main sewer pipes, in an existing sewer, or in the ditches. In addition to these solutions, some ditches have been included in the project to protect the rail service maintenance plant and the road. In these ditches is also conveyed part of the rain water of the rail service plant. The stormwater network covers a 342,000 m^2 basin area which is divided into 38 sub basins for the calculations. Single runoff coefficient of 0.8 is applied to the basin. Kutter formula is used to calculate the velocity of the discharge according to General Directorate of Ankara Water and Sewage Administration (AWSA) regulations. The stormwater network used for the calculations consists of 218 separate pipes in various length and size.

Intensity-duration-frequency analyses of maximum precipitation under stationary and nonstationary conditions were evaluated in Oruc et al. (2019) and therefore the readers are addressed to this article for further information about precipitation analysis that are used in this current study.

Land Use/Cover Analysis

Maps produced by the General Command of Mapping (GCoM) were analyzed as the most suitable data set by Özkil (2015). It has been identified that there are 4 different versions of maps (late 50s, early 80s, mid 90s and lately the year of 2013) covering the area of interest, available in the GCoM archives. Initially, the classification of land is done by extracting the image bands with the pixels grouping to each image segment. The land cover classification (e.g. urban area, green area, soil, road etc.) is performed based on the land use conditions. Polygons are created in advance during the digitizing process and then appropriate projection system is defined for polygons to calculate the geometry. Finally, urban and green areas (impervious and pervious surfaces) are calculated as the surface area per parcel.

In the pilot study area, there are several types of land cover, which are and their coverage areas are shown in Table 2. These cover types have different range of runoff coefficients (TDT, 2016; Burke & Burke, 2015) not only related with the material but also related with the slope of the cover (Table 2).

Table 2

Cover Types and Runoff Coefficient – Pilot Study Area

Type	Area m²	Runoff Coefficient
Building (Roof)	51.629	0.95
Car Park	18.500	0.95
Landscaped Area	4.650	0.50
Green Area	16.500	0.30
Asphalt-Sub-Ballast	149.000	0.95
Slab on Grade	49.000	0.95
Undeveloped	5.272	0.30
Potential Development Area	47.449	0.95
Composite Runoff Coefficient		0.90

A drainage area is composed of subareas with different runoff coefficients. The summation of the products of corresponding runoff coefficients and subareas is divided by the total area and then, a composite coefficient for the total drainage area is computed. In order to find a scenario type runoff coefficient, possible future land use types for the potential development of the undeveloped area are also considered. Finally, a composite runoff coefficient is calculated by using weighted average of the current and future land cover types. In this study, it is found as 0.9 (Table 2), which represents highly urbanized or impervious surface conditions in the future.

Hydraulic Calculations

Rational formula is used to recalculate the peak runoff of pilot study area where stormwater network information is available. The rational method makes the basic assumption that the peak rate of surface outflow from a given watershed is proportional to the watershed area and average rainfall intensity over a period of time (time of concentration) just sufficient for all parts of the watershed to contribute to the outflow (Burke & Burke, 2015). The rational formula is written as:

$$Q=CIA \tag{1}$$

Where Q is the peak runoff, C (runoff coefficient), is the ratio of peak runoff rate to average rainfall rate over the watershed during the time of concentration (T_c), I is the rainfall intensity and A is the contributing area of watershed under consideration.

In the application of rational formula, an intensity-duration-frequency curve is used to identify intensity for the selected return period and storm duration that equals to the T_c. This is then multiplied by the drainage area and runoff coefficient to determine the peak discharge rate.

Data from the results of precipitation analyses for observation and projection periods are used to calculate the peak discharge. Time of concentration and return period are chosen 15 minutes and 2 years, respectively according to design standards of AWSA. Moreover the 15 minutes - 5 years storm depths were also used for the performance analysis of baseline storm sewer design. For the calculations, rainfall intensities (return levels in mm) are first converted to mm/minutes and then to l/sec/ha in this study. This is then multiplied by the area and runoff coefficient to determine the peak discharge rate.

Furthermore design parameters such as percent fullness, maximum and minimum velocity and minimum water height in the channels are calculated for the new intensities to observe their change. Maximum discharge (Q_{full}) and discharge of the pipe (Q_{design}) are calculated by using Q_{design}/Q_{full} ratio. V_{design} is calculated with respect to V_{design}/V_{full} ratio where V_{full} is the velocity at the full capacity. Finally, water depth (h) is calculated by using depth ratio (h/D). Kutter formula is used to calculate the velocity of the maximum discharge.

$$V = \frac{100\sqrt{R}}{m+\sqrt{R}}\sqrt{RJ} \qquad (2)$$

Where R is hydraulic radius, J is slope (hydraulic gradient) and m is Kutter constant (0.35 in this study for concrete and rainforced concrete pipes). Then, the maximum discharge (Q_{full}) is calculated using the formula below where A is the cross-sectional area of the drain:

$$Q_{full} = V_{full} \text{ x } A \qquad (3)$$

The maximum and minimum velocity and minimum water height in the channels are investigated considering the baseline design criteria (Vmin=0.5 m/s, Vmax=5 m/sand h_{min} > 20 mm) to identify whether the system is overdesigned or exceed capacity. If the system is overdesigned pipe diameter can be decreased, the slope must be increased or both can be applied in order to stay in the design ranges. In this study only diameters are changed to find a better allocation of pipes.

Analysis conducted for the following cases;

Case 1; only the design intensity has changed and the behavior of existing stormwater network is observed.

Case 2; only design runoff coefficient has changed (from 0.8 to 0.9) and the effect is observed.

Case 3; both design intensity and runoff coefficient have changed and the changes in pipe capacities (percent fullness) of existing network is observed.

Case 4; design intensity has changed and pipe diameters are optimized.

Case 5; design intensity and runoff coefficient have changed and pipe diameters are optimized.

Results and Discussion

In order to figure out the performance of current (existing) stormwater network of pilot study area rainfall return levels that are obtained from observation and projection periods with stationary and nonstationary models are used to calculate peak runoff in rational formula. In addition, land use/cover change effect is integrated as composite runoff coefficient (0.9) for the pilot study area.

Existing network design considered the time of concentration 15 minutes and the return period 2 years and therefore, the same duration and return period is used for the analyses in this study. On the other hand, the minimum return periods of 5 - 10 years were also recommended for urban stormwater networks where such a critical facility was established (Burke & Burke, 2015; Efe, M., 2006). If there is greater possibility of damage and loss, then the risk can be reduced by preferring greater frequencies. Thus, the baseline network capacity is also investigated for 15 minutes 5 years return period values. These (15 minutes - 5 years) design storm depths were obtained from the observation and projection results. However, only the results from observation period were used for the analyses because the 15 minutes - 5 years storm intensities of projections were lower than the baseline design intensity (15 minute – 2 year). The design rainfall intensity (baseline design intensity) for 15 min and 2 year frequency used in the existing network design is 116 lt/s/ha. Generally, the operational status of the stormwater network is described as unsatisfactory if the specified hydraulic criteria (e.g. excess pipe capacity, velocity is out of the range) is violated (Nanos & Filion, 2016; Gouri & Srinivas, 2015) In this study, the capacity surcharge of pipes, which is the excess of percent fullness ratio, is described as a failure.

Results from Case 1, 2, and 3

Figure 3 shows the percent of the network pipe volume as function of percent fullness for each projection model, observation and baseline design (Case 1). In addition, Figure 4 demonstrates maximum percent fullness capacity experienced by each pipe for each entry in Case 1. Considering the current climate conditions, 96% of the network pipe volume is under the 60% capacity for the baseline design as shown in Figure 4.1. Only the 4% of the network pipe volume shows a maximum pipe capacity higher than 60%. The 69% of the network pipe volume is reaching a maximum capacity lower than 20% (Figure 3).

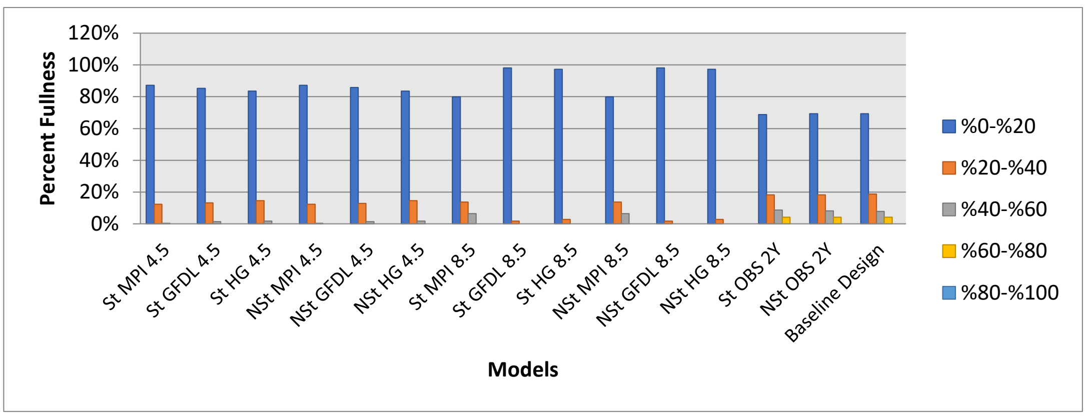

Figure 3. Percent fullness of projection model, observation and baseline design without Pipe Revision-Case 1**.**

With regard to the climate change scenario (Case 1), remarkable changes in the system performance can be observed when compared with current conditions in terms of pipe capacity ranges. Specifically, the maximum pipe capacity reached about the 85% of the network pipe volume stay within the %0-%20 range, while about %15 falls in the %20-%40 range (Figure 3). Even flows at %0-%20 capacity increases to around 98% pipe volume from GFDL and HG with 8.5 scenarios. There are no pipe flows that exceed 60% capacity for all the climate change scenarios. On the other hand observed data in stationary and nonstationary conditions exhibit a parallel capacity range with the baseline design. In Figure 4.2, observed data with and without nonstationarity provided the highest maximum fullness capacity (75%) while all scenarios showed lower than 60% maximum fullness capacity. Lowest fullness capacities occurred with RCP8.5 models.

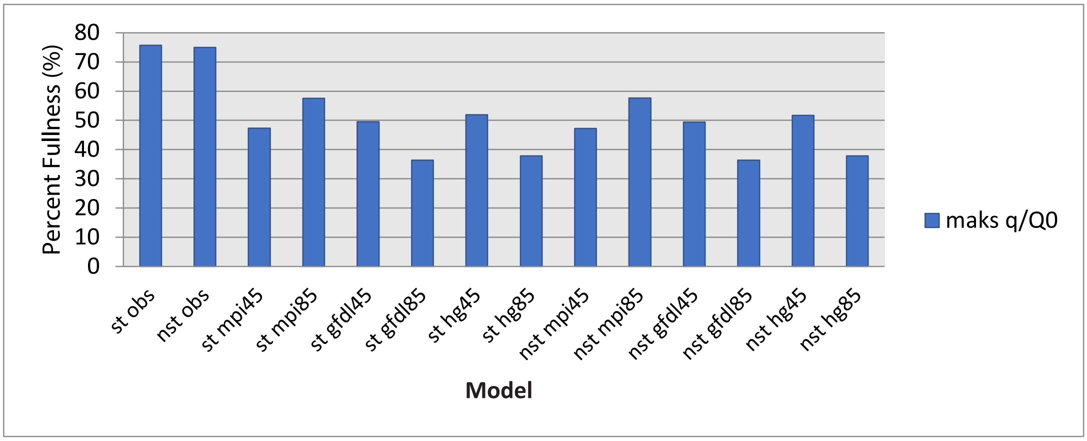

Figure 4. Maximum capacity experienced during the projection and observed period by each pipe.

Figure 5 demonstrates the percent fullness change for baseline design when only runoff coefficient is changed from 0.8 to 0.9 in Case 2. Rainfall intensity for the existing storm network remained the same. With increasing runoff coefficient, the flows at %0-%20 range slightly decrease (4%) and they contribute to higher percent fullness range (%80-%100) with 2 percent. However, the general effect of surface runoff coefficient is negligibly small for this study area. Figure 6 exhibits the pipe capacities for land use change combined with climate change scenarios without any revision for the existing network as Case 3.

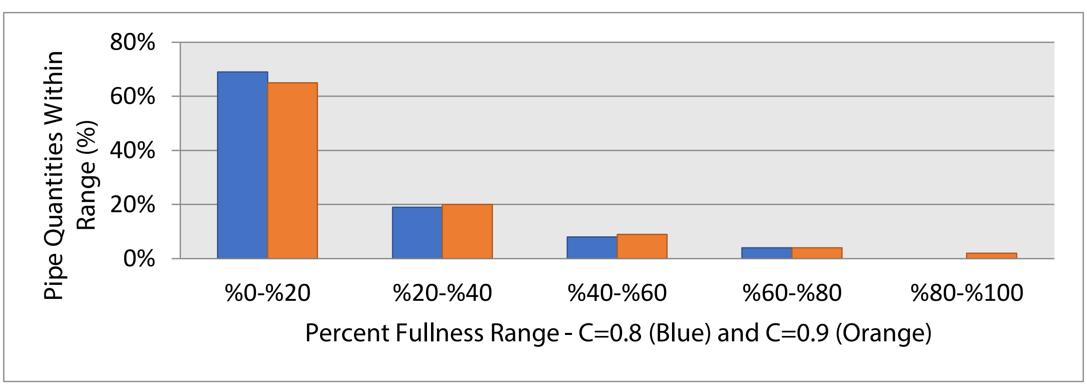

Figure 5. Baseline design percent fullness with surface runoff coefficients of C=0.8 (Blue) and C=0.9 (Orange) in Case 2.

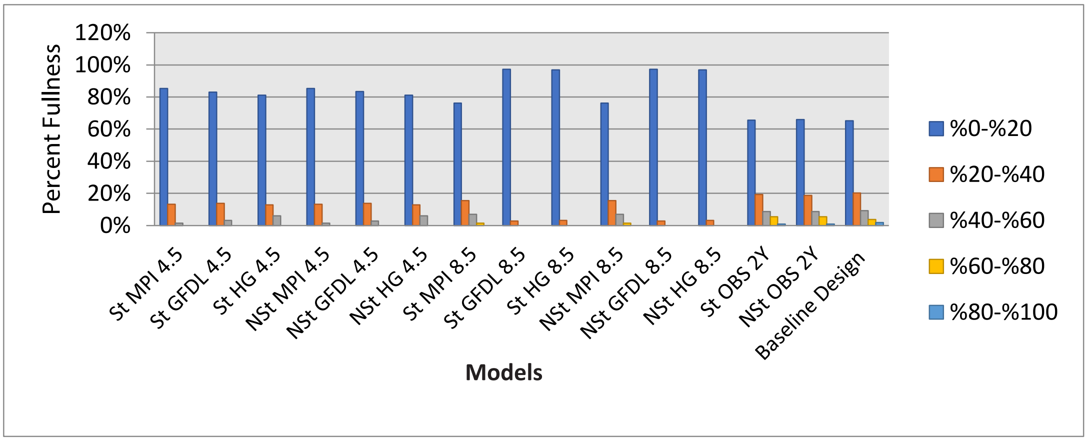

Figure 6. Percent fullness of models without pipe revision-climate change and land use change (C=0.90) – Case 3.

The system operated in a satisfactory state refers the condition that the maximum volume that the network system conveys as result of climate change projections for the extreme rainfall will decline throughout the projection period. With the combined effect (or the effect of 10 percent higher surface runoff coefficient) the flows in pipes shift to higher fullness capacity ranges with projection models and observed conditions. However, there is still no flow at fullness range of %80-%100 and even at the range of %60-%80 in any scenario models (Figure 6). Only observed stationary and nonstationary analyses show pipes with 1% within %80-%100 fullness range. But, they are still less than pipe volumes of baseline design (2%). The outputs produced from the simulations show that performance of the case study stormwater network was observed to operate in a satisfactory state for the climate change scenario experiments, where an unsatisfactory state is defined as an occurrence of the conduit/pipe capacity (percent fullness) exceeding 90%. These results indicate that system performance will be satisfactory by the end of the century.

Results from Case 4 and 5

The existing stormwater network design is observed to operate in satisfactory conditions considering the projected rainfall data however a better allocation of pipe diameters can be achieved when the percent fullness data is examined. For this reason other design parameters such as velocity and minimum depth are compared. The maximum and minimum velocity and minimum water height in the channels are presented in Table 3 for Case 1. Stationary and nonstationary projection results reveal closer values while observed period shows the highest velocity, depth and fullness percent for stationary and nonstationary conditions.

Minimum velocity values indicate that the system is over designed and pipe diameter must be decreased or the slope must be increased in order to stay in the design ranges. RCP 8.5 based results for GFDL and HG models show lower rainfall intensities so the velocity (0.1 m/s for minimum) in the pipes and the fullness (~36%) decrease for these models. Also, the water depth in the pipes reaches its lowest value (0.3 cm) for these models. With regard to models results, decreasing trend of extreme precipitation, which is the outcome of projection results, can be one of the reasons that existing system stay satisfactory over time. That means stationary assumption reveals more conservative design conditions for the future but with higher economy of scale.

Table 3

Velocity, Percent Fullness, H Minimum Results For Models Obtained For Model Driven Intensities For Baseline Design System

Model	Maximum velocity (m/s)	Minimum velocity (m/s)	Max q/Q_0 (%)	H minimum (cm)
St Obs 2 Years	3.03	0.26	75.7	1.62
Nst Obs 2 Years	3.02	0.25	75	1.62
St Mpi 4.5	2.44	0.2	47.3	0.54
St Gfdl 4.5	2.51	0.2	49.5	0.75
St Hg 4.5	2.59	0.21	51.9	0.75
St Mpi 8.5	2.76	0.23	57.6	1.5
St Gfdl 8.5	1.95	0.1	36.4	0.3
St Hg 8.5	2.03	0.1	37.8	0.3
Nst Mpi 4.5 2.43		0.2	47.2	0.54
Nst Gfdl 4.5	2.51	0.2	49.4	0.75
Nst Hg 4.5	2.59	0.21	51.7	0.75
Nst Mpi 8.5	2.76	0.23	57.7	1.5
Nst Gfdl 8.5	1.95	0.1	36.4	0.3
Nst Hg 8.5	2.03	0.1	37.8	0.3

Furthermore the existing stormwater network is redesigned in terms of pipe diameter to obtain more optimal and economical solutions and compare them with the current quantities. Figure 7 and Figure 8 exhibit the pipe capacities for only climate change effect and combined effect of climate change with land use change scenarios, respectively with revision made to the existing network (Case 4 and Case 5).

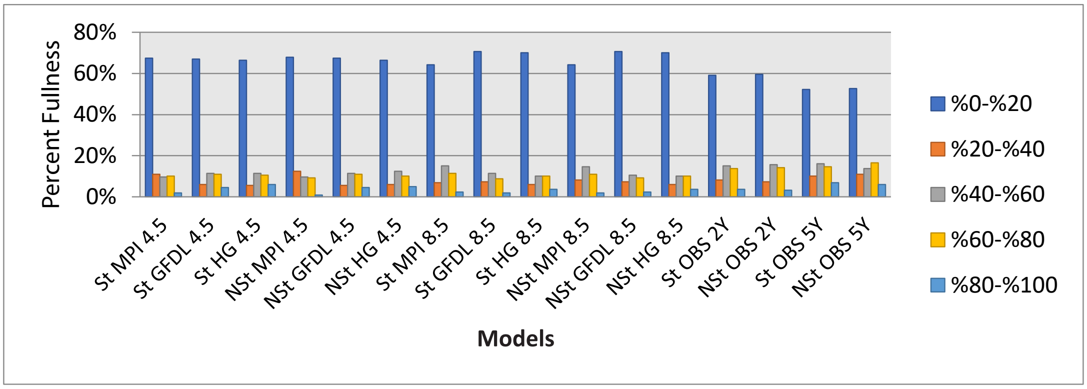

Figure 7. Percent fullness of models with pipe revision to climate change – Case 4.

In the climate change and revised system scenario (Case 4), the system performance (capacity of fullness) increases. Approximately the 15% and 25% of the network pipe volumes show a maximum pipe capacity higher than 60% and 40%, respectively for the projection scenarios. The surcharged range (80%-100%) network pipe volume is equal to 3% for RCP8.5 and %5 for RCP4.5 but these pipe volumes do not exceed the 90% capacity ratio significantly (only 1 or 2 pipes do exceed) (see Figure 7). In the 15 minutes 5 years scenario that is originated from observed stationary and nonstationary return levels, the revised system produces also a better drainage system performance, if compared with the baseline design. For example, the maximum pipe capacity within 0%-20% dropped to 52%. About 40% of the network pipe volume has a maximum pipe capacity higher than 40% capacity ratio and almost the 16% of the pipes is within 60%-80% capacity range (Figure 7).

In addition to climate change scenario, also the new composite runoff coefficient (0.9) is applied to the design (Case 5). The revised system performance simulated under these new conditions and its results are given in Figure 8. The surcharged range (80%-100%) of network pipe volume is further increased and there are pipe volumes that exceed the 90% (1-6% for projections and 4-7% for observation) and 100% (1% for projections and 3% for observed 5-year stationary frequency) capacity ratio significantly. On top of climate change effect, adding runoff coefficient increases the pipe volumes 4 percent for projections and 6 percent for observed 2-year and 5- year frequency at the capacity range of %80-%100 (see Figure 7 and Figure 8). Prior to revision applied to the system, either climate change effect or joined effect has no significant change on the system performance but after the revision it becomes critically important. For the 15 minutes 5 years return levels, increase in the rainfall intensity cause system failure; about 10% of the pipes excess capacity for the baseline (existing) network.

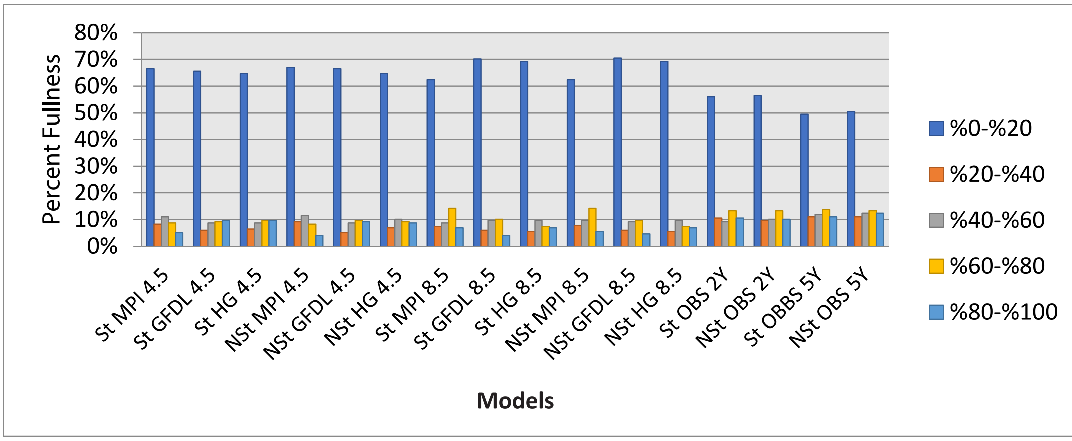

Figure 8. Percent fullness of models with pipe revision to climate change and land use – Case 5.

The hydraulic performance of the baseline and revised system for the observed and projected rainfall data has been compared in terms of the pipe capacity ratio associated with various pipe capacity range (0-20%, 20-40%, 40-60%, 60-80% and 80-100%) for five cases. It can be seen that system continuity can be satisfied with various design conditions, including climate change and land use, for all return level results. The remarkable point is that the system can perform with lower pipe diameters than it is designed. Figure 9 shows quantities of small size and large size pipes in meters for every diameter (200 mm to 1600 mm) and total quantities of small (200 mm to 600 mm) and large (800 mm to 1600 mm) size pipes for each model entry and baseline design. The revision of the system by pipe diameter, which is represented in Figure 9, results in a reduction of the oversized network.

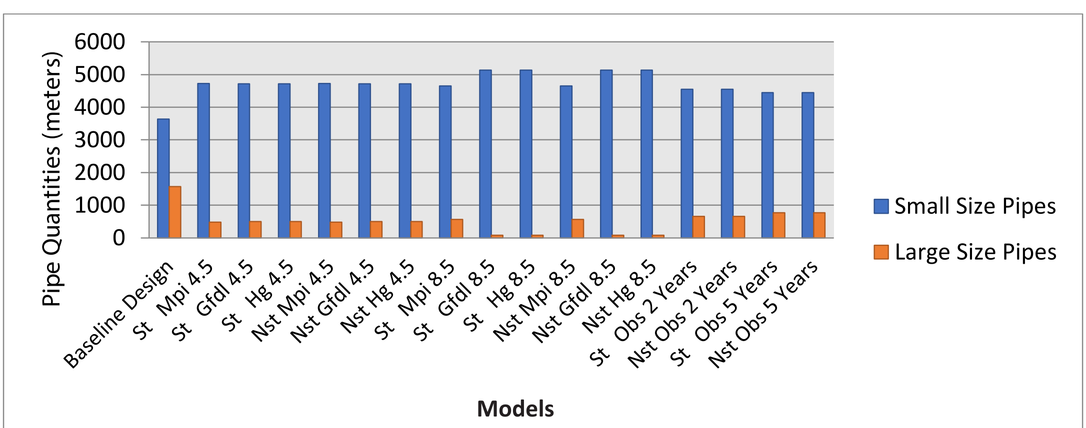

Figure 9. Small vs. large size pipe quantities – Baseline and revised system pipe lengths in meters - Small (200-600 mm) and Large (800-1600 mm).

Table 4 exhibits the % change in pipe length for every diameter after revision compared with baseline design. The negative and positive signs show decrease and increase in the pipe length change, respectively. Figure 9 also shows the total small and large size pipe quantities (lengths in meters) of baseline (existing) design and revised network's small and large size pipe quantities due to climate change scenarios. The current system also can perform well under the 15 minutes 5 years return period loads with revisions in pipe diameter such as increase in large size and decrease in small size quantities considering the revised system which can be seen from Figures 7, 8 and Table 4. The revised network for 2 years - 15 minutes rainfall intensities can also be designed with several changes in pipe diameter so that 15 minutes - 5 years return period loads can be tolerated which at the end result increase in small size pipes and decrease in large size pipes with respect to existing (baseline) design pipe quantities (see Figure 9 and Table 4). The largest decrease (95%) and increase (41%) in pipe sizes occurred with model projections of GFDL 8.5 and HG 8.5 for stationary and nonstationary conditions. Changes (22-25% and 51-58%) in observed cases with 2-yr and 5-yr return periods stayed below the changes (>28% and >64%) for all projected cases after revision.

Economical Analysis

Furthermore the effect of design revision due to changing climatic conditions is reflected in terms of cost in Figure 10. Only the cost associated with pipe diameter is calculated by using actual project unit price. The pipe diameters (small size or large size group) determine the choice of concrete or reinforced concrete for which unit prices are different. Therefore, it directly affects total cost, which decreases with model results. In other words, with the revision large size pipes (reinforced concrete) are generally replaced with small size pipes (concrete) and this reduces the total cost.

The models that reveal lower rainfall intensity have the lowest total costs such as stationary GFDL with RCP8.5 or HG with RCP8.5. In general, RCP4.5 results reveal higher total cost than RCP8.5 results probably because of the less warming that may cause higher precipitation, besides MPI model stationary and nonstationary results. Observed data driven design alternatives have the higher cost among the all alternatives, yet they are still lower than the existing design. Almost 400K euro the baseline design, which was designed with 15-min and 2-yr storm is more costly and even increasing the return period to 5-yr it is still more expensive. In baseline design, the cost for small size pipes (300-600 mm) is almost four times lower than the cost for large size pipes (800-1200 mm) but in all projected designs the cost for small diameter pipes is higher. The cost distribution for observed data driven designs (2-yr and 5-yr) is similar to the baseline design but with very low total cost.

Table 4

Pipe Diameter Change for the Models Compared with Baseline Design

Model	Ø300	Ø400	Ø500	Ø600	Total Change (Small Ø)	Ø800	Ø1000	Ø1200	Total Change (Large Ø)
St Mpi 4.5	37%	-52%	469%	-51%	30%	-58%	-95%	-100%	-69%
St Gfdl 4.5	37%	-2%	245%	-54%	30%	-59%	-90%	-100%	-69%
St Hg 4.5	35%	-19%	335%	-41%	30%	-64%	-77%	-100%	-69%
Nst Mpi 4.5	32%	-29%	469%	-51%	30%	-65%	-77%	-100%	-69%
Nst Gfdl 4.5	35%	7%	245%	-54%	30%	-59%	-90%	-100%	-69%
Nst Hg 4.5	35%	-23%	357%	-41%	30%	-64%	-77%	-100%	-69%
St Mpi 8.5	31%	-45%	496%	-50%	28%	-58%	-77%	-100%	-64%
St Gfdl 8.5	55%	0%	229%	-66%	41%	-95%	-95%	-100%	-95%
St Hg 8.5	55%	0%	229%	-66%	41%	-95%	-95%	-100%	-95%
Nst Mpi 8.5	29%	-36%	496%	-50%	28%	-58%	-77%	-100%	-64%
Nst Gfdl 8.5	55%	0%	229%	-66%	41%	-93%	-100%	-100%	-95%
Nst Hg 8.5	55%	0%	229%	-66%	41%	-95%	-95%	-100%	-95%
St Obs 2 Years	26%	-18%	190%	36%	25%	-48%	-77%	-100%	-58%
Nst Obs 2 Years	26%	-20%	190%	36%	25%	-48%	-77%	-100%	-58%
St Obs 5 Years	24%	-62%	91%	139%	22%	-70%	-6%	-48%	-51%
Nst Obs 5 Years	24%	-56%	64%	138%	22%	-70%	-6%	-48%	-51%

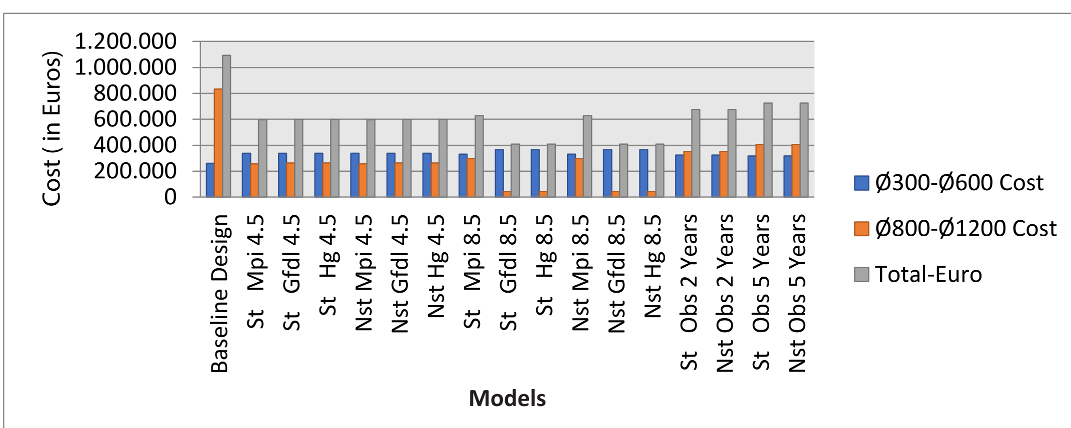

Figure 10. Cost Comparison of Models after Revision (in Euros).

Conclusions

The impact of climate and land use changes on the capacity of a storm water drainage system in Etimesgut, Ankara of Turkey was investigated. The capacity of the system was evaluated under the current condition using the data from 1950 to 2015 and future condition using the data from three different GCM results with two different emission scenarios. The rainfall intensity of 15 min and 2 year frequency from all data entries was conveyed through the system. The resulted capacity (percent fullness) was compared with the capacity available from existing network that also used 15 min – 2 year rainfall frequency. The higher load obtained from 15 min – 5 year storm for current period was also added to the analyses. The nonstationarity effect in maximum rainfall depth was also considered in this impact study. Finally, alternative solutions (reduction in pipe sizes) were applied to the existing sewerage system and their implications were evaluated.

The system operated in a satisfactory state totally under current and future conditions when climate change (case 1) and land use change (case 2 case 3) were considered. It can be concluded that according to climate change projections for the extreme rainfall with and without nonstationarity, the maximum volume that the system face will not exceed baseline design criteria throughout the projection period. Combination of climate change and land use change conditions also stay satisfactory for the baseline design which used 15 minutes storm duration and 2 years return period as intensity input. Increase in runoff coefficient from 0.8 to 0.9 did not yield important changes in flow amount but joining it to climate change produced significant flow loads in the current system towards higher depth ratio values. The current network system can perform appropriately under the loads from 1950-2015 and 2015-2098 periods that are delivered by stationary and nonstationary assumption. The stationary assumption reveals more conservative design conditions for the future but with higher cost. The capacity of the system with both stationary and nonstationary is expected to be not overload in the future. On the other hand the system may fail under the loads derived separately or together by longer storm duration (over 15 minutes and more) or higher return periods (such as 5 years and more) for such a critical facility and area. Under these conditions, selecting the design parameters that satisfy the demand and requirements gains importance. Dikici (2018) indicates the importance of proper and optimum solution for water sewer system of the major urban areas and draw attention to updated design criteria considering the increasing population and urbanization.

Overall, the total cost results of alternative design options indicate that the storm sewer system can be built at a lower total cost not only for all climate change and land use options but also for the 2 and 5 years 15 minutes observed data driven return level results. On the other hand the unit cost of pipes are unique to the project (regarding this, information that have been presented in this study cannot be used or reproduced without permission) due to tender method and cannot be generalized.

By contrast with this study, the general outcome of the future period studies with regard to stormwater networks in literature is that current systems will probably fail and cannot withstand considering the future climate conditions. For instance, Thakali et al. (2016) specify that the present capacity of most urban drainage systems is expected to be overload in the near future and the analysis of the present stormwater facilities of the Flamingo and Tropicana watershed showed that these facilities are unable to sustain their performance under the loads resulting from the projected climate scenario. Furthermore, Osman (2014) also showed that in the future the urban drainage system of the area that he studied could react differently in terms of increase in number of surcharged sewers and from manholes surface flooding. Larsen et al. (2008) pointed out that a 100-year event in the control period for Sweden will be shorten due to climate change scenarios and damages caused by urban flooding will probably occur more frequently.

With regard to stormwater network design in Ankara, there are two important aspects; one is selection of the design parameters and the other is application of these parameters in construction phase in an appropriate environment. For instance selecting an appropriate design load (e.g. storm duration and return period) is important for the network design on the other hand if the runoff cannot be routed correctly to the system then surface flooding occurs. Infrastructure is relatively a long term investment and conditions may change during the proposed design life such as decrease in pervious land, population growth etc. lasting with increasing load exposure for the system, so monitoring is an essential part for a vital stormwater management process.

Selecting the optimum parameter is another issue, for the changing environmental conditions together with urbanization bring out the need for a new approach. Design parameters cannot be assumed stationary for such a long term design lives so temporal, spatial or other changes must be considered for the design process which makes involvement of multiple bodies to the design process necessary. While selecting the design approach and parameters also a risk based approach should be applied; for this not only physical damage but also environmental and social cost of the event must be considered. The cost of designing a network for 10 minutes - 10 years return period rainfall will probably be higher than a 15 minutes - 2 years return period rainfall based design as the design intensity increases which brings out the larger pipe diameters. Nevertheless decision making mechanism must take into account not only the extra cost derived by the design parameter but also the cost of loss of life, reputation, interruption of business etc. Because Ankara is the capital of Turkey, center of the bureaucracy, transportation hub for high speed rail and host of many entities that are determining bodies of economic and social state of affairs design process of stormwater network must consider the above mentioned conditions.

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https://sciforum.net/paper/view/conference/5807
https://sciforum.net/paper/view/conference/5808

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Extended Turkish Abstract
(Genişletilmiş Türkçe Özet)

İklim Değişikliğinin Yağmursuyu Şebekeleri Üzerine Etkisinin Araştırılması: Başkent Ankara Örneği

Kentsel su altyapı sistemleri, örneğin yağmur suyu sistemleri ve barajlar gibi taşkın kontrol yapıları ekstrem yağış özelliklerine göre tasarlanırlar ve bu yağış özellikleri de Şiddet-Süre-Tekerrür eğrileri ile ifade edilir (Peck vd., 2012; Hosseinzadehtalaei vd., 2017). Şiddet-Süre-Tekerrür eğrileri, farklı yağış sürelerinde belirli bir şiddette bir yağışın meydana gelme sıklığını ölçmektedir. Şiddet-Süre-Tekerrür eğrileri, genel olarak geçmiş döneme ait yağış analizleri ve istatistiklerine dayanmaktadır. Altyapı tasarımı da uzun süreden beri zaman içinde ekstrem olayın sıklığında bir değişiklik olmadığını varsayarak oluşturulmuş kriterlere dayanmaktadır. Ancak ekstrem yağışların sıklığı ve şiddetinin değişiklik göstermekte olduğu ve bu değişikliklerin gelecekte de muhtemelen devam edeceği anlaşılmıştır (Cheng ve Aghakouchak, 2014). Ayrıca, değişen frekansın ihmal edilmesinin ekstrem olayları hafife alan Şiddet-Süre-Tekerrür eğrileri ile sonuçlandığı görülmüştür (Cheng ve Aghakouchak, 2014). Sarhadi vd. (2017) de yaptıkları çalışmalarda Cheng ve Aghakouchak (2014) ile benzer sonuçlar ortaya koymuştur. Sonuçlara göre durağan yani zaman içerisinde değişikliğin ihmal edildiği yaklaşımın ekstrem yağış olaylarını hafife alabileceği dolayısıyla da değişen koşullarda Şiddet-Süre-Tekerrür eğrilerinin ve tasarım kriterlerinin güncellenmesinin gerekliliği ortaya konmuştur. Hosseinzadehtalaei vd. (2017), ekstrem yağışların özelliklerinin değişmekte olduğunu vurgulamış ve zamanla herhangi bir değişiklik göstermeyen Şiddet-Süre-Tekerrür eğrilerine dayanan mevcut tasarım standartlarına dikkat çekmiştir.

Altyapı sistemlerini, özellikle de yağmur suyu sistemlerini, etkileyen önemli bir faktör de kentleşmedir. Kentleşmenin sonucu olarak geçirimsiz alanların artış göstereceği, bunun da değişen yüzey örtüsü ile birlikte deşarj hacminde artışa neden olacağı, iklim değişikliğinin de etkisi ile yağışların frekansı ve şiddeti göz önüne alındığında kentsel yağmur suyu sistemlerinin daha yoğun etkilere maruz kalacağı belirtilmiştir (Thakali vd., 2016). Son veriler, Avrupa'daki tarım arazilerinin, kentsel alanlara ve altyapı tesislerine kalıcı olarak dönüştürüldüğünü ve kentsel genişlemenin büyüklüğünü teyit etmektedir (AÇA, 2017). Arazi örtüsü ve kullanımı ile iklim değişmekte olup; arazi kullanımı ve özelliklerinde değişiklikler ile kentsel taşkınlara neden olan ekstrem yağış olayları birlikte ele alındığında bu değişikliklerin yağmur suyu şebekelerinin tasarımı ve yönetimi üzerindeki etkilerinin değerlendirilmesine duyulan ihtiyaç ortaya çıkmaktadır (Hailegeorgis ve Alfredsen, 2017).

Türkiye'de yıllık ekstrem olay sayısı, Meteoroloji Genel Müdürlüğü 2017 İklim Değerlendirme Raporu'na (2018) göre 1940-2017 döneminde artış eğilimi göstermektedir. 2017 boyunca en tehlikeli ekstrem olayların şiddetli yağmur / sel (% 31), rüzgar fırtınası (% 36), dolu (% 16), yoğun kar (% 7) ve yıldırım (% 4) olduğu görülmektedir. Türkiye'nin başkenti olan Ankara, Sakarya ve Kızılırmak Havzaları içerisinde yer almaktadır. Ankara yarı kurak bir iklime sahiptir ve sürekli bir nüfus artışına dolayısıyla da geçirimsiz yüzeylerin artmasına neden olan yüksek bir kentleşme oranına sahip olup iklim değişikliğinden önemli ölçüde etkilenmiştir. Bu da kentsel altyapı üzerinde artan bir baskıya neden olmaktadır. Ayrıca, son 20 yıldaki sel olayları gözlemlendiğinde, yoğun yağış ve ani selin, mülklere çeşitli zararlar vermiş olduğu ve hatta kentte can kaybına neden olduğu görülebilir. Bununla birlikte, tüm bu olumsuz olaylara rağmen, özellikle iklim değişikliği etkisinde ekstrem yağışta meydana gelebilecek değişikliklere ve Şiddet-Süre-Tekerrür eğrilerinin bu değişikliklerden nasıl etkileneceğine odaklanan detaylı çalışmalar Türkiye'nin Başkenti Ankara için eksik kalmıştır

İklim değişikliği ve arazi kullanımından kaynaklı değişikliklerin Ankara ili, Etimesgut ilçesindeki bir yağmur suyu drenaj sisteminin kapasitesi üzerindeki etkileri incelenmiştir. Sistemin kapasitesi 1950›den 2015›e kadar olan gözlem verileri ile iki farklı emisyon senaryosuna sahip üç farklı iklim modeli sonucundaki projeksiyon verileri kullanılarak değerlendirilmiştir. Etki analizi yapılırken tüm model ve gözlem sonuçlarından elde edilen 15 dakikalık süreli ve 2 yıllık tekerrüre sahip yıllık maksimum yağış verileri kullanılmıştır. Elde edilen kapasite verileri (doluluk oranı), mevcut şebekenin tasarımında kullanılan kapasite oranları ile karşılaştırılmıştır. Analizlere ayrıca gözlem periyoduna ait 15 dakikalık-5 yıl tekerrür süresine sahip yağış şiddeti de eklenmiştir. Bu çalışmada hidrolik hesaplamalarda kullanılan yıllık maksimum yağış yüksekliğine durağanlığın etkisi de dikkate alınmıştır.

Bir drenaj alanı, farklı akış katsayıları olan alt alanlardan oluşur. Alanların kullanımı / arazi örtüsü özelliklerine karşılık gelen akış katsayıları ve bu alanların akış katsayıları ile çarpımlarının toplamı, toplam alana bölünür ve daha sonra toplam drenaj alanı için bir bileşik akış katsayısı hesaplanır. Çalışma alanında gelecekte meydana gelebilecek akış katsayısını bulmak için, herhangi bir düzenlemeye maruz kalmamış alanların gelecekteki olası arazi kullanım tipleri dikkate alan potansiyel maksimum gelişim senaryosu ile alanların arazi örtüleri belirlenmiştir. Son olarak, mevcut ve gelecekteki arazi örtüsü tiplerinin ağırlıklı ortalaması kullanılarak bileşik bir akış katsayısı hesaplanmıştır. Çalışma alanı için gelecekte yüksek oranda kentleşmiş ve geçirimsiz yüzey koşullarını temsil eden akış katsayısı 0.9 olarak bulunmuştur. Son olarak, mevcut yağmursuyu sistemine alternatif çözümler (boru çaplarında varyasyonlar) uygulanmış ve etkileri değerlendirilmiştir.

Tüm analizler göz önüne alındığında, mevcut sistemin, durağan ve durağan olmayan yaklaşımla tespit edilen 1950-2015 ve 2015-2098 dönemleri arasındaki yükler altında uygun şekilde çalışabildiği görülmüştür. Gelecek dönem için tespit edilen yükler göz önünde bulundurulduğunda durağan varsayım ile yapılan tasarımın daha konservatif olduğu anlaşılmıştır ancak bunun maliyetinin daha yüksek olacağını da unutmamak gerekir. Hem durağan hem de durağan olmayan koşullar için sistemin kapasitesinin gelecekte aşırı yüklenmeye maruz kalması beklenmemektedir. Öte yandan, sistem, böyle kritik bir tesis ve alan için ayrı ayrı veya birlikte daha uzun yağış süreleri (15 dakika ve üzeri) veya daha yüksek tekerrür periyotları (5 yıl ve daha fazlası gibi) ile elde edilen yükler altında istenen ve beklenen performansı gösteremeyebilir. Bununla birlikte, modellerin yağış şiddeti ve sıklığı gibi tahminlerindeki farklılıklarının, mevcut tasarım parametrelerini en yeni veriler ve yaklaşımlarla güncelleme ihtiyacını desteklediğine dikkat edilmelidir. Ortaya çıkan farklılıklar ayrıca iklim modellerinden elde edilen gelecek dönemlere ait verileri de kullanarak analiz yapma ihtiyacını ortaya koymaktadır.