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Hydraulic Performance of Al-Kut's Stormwater Drainage Network under Iraqi Code and 100-Year Extreme Rainfall Scenarios Using SewerGEMS and GIS

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ABSTRACT

This study assesses the hydraulic performance of the entire stormwater drainage network of Al-Kut City, Iraq, covering both banks of the Tigris River (7,571 PVC pipes, 328.3 km, 33 sub-networks), using Bentley OpenFlows SewerGEMS integrated with GIS. Two steady-state scenarios were simulated with Rational Method inflows: the Iraqi Code design intensity of 13 mm/h (20-min duration) and a 100-year extreme intensity of 91.42 mm/h (30-min duration) derived from the local lognormal Intensity–Duration–Frequency (IDF) relationship. Results were screened against the Iraqi Building Code limits of 0.9–3.0 m/s for velocity and 80% for the depth-to-diameter ratio (d/D), with pressurised flow identified from the hydraulic grade line (HGL) relative to the pipe crown. Under the code storm, 98.5% of pipes were pressurised, 25.1% carried more than their full-flow capacity, and 70.8% flowed below 0.9 m/s, mainly because of backwater from overloaded trunk sewers. Under the extreme storm, the overloaded share rose to 74.5%, and 49.4% of pipes were overloaded only in this scenario. The simulated HGL reached ground level at 4,135 manholes (54.6%) and 6,946 manholes (91.7%), respectively, indicating potential overflow locations. These screening results identify priority areas for rehabilitation and support climate-resilient drainage planning in Iraqi cities.

1. INTRODUCTION

The urban stormwater drainage system is the most important man-made structure designed to protect against surface flooding, with runoff from rainfall events transported from built-up areas using a series of interdependent pipe conduits and manhole nodes. These hydraulic systems are designed using the well-validated Manning equation, where n is Manning's roughness coefficient, R is the hydraulic radius, and S is the sewer slope [1]. The Manning's n coefficient of 0.009 to 0.011 is recommended for PVC sewer pipes and is significantly lower than the 0.013 value recommended for concrete pipe [2]; PVC pipes make up the whole of the stormwater network evaluated in this study. The lining material and the roughness index play a role in flow velocity and discharge capacity [3]. The lower roughness of PVC means that it can be made to have a significantly higher hydraulic capacity for a given pipe diameter and slope of pipe, which is relevant to the velocity and capacity performance of the Al-Kut City network examined in this study. Standard wastewater engineering references state that the minimum self-cleansing velocity of a stormwater sewer system should be capable of preventing deposition of particulate matter on the pipe invert, which over time will decrease the effective pipe cross-section and could ultimately lead to complete blockage [4]. The Iraqi Building Code 2021 (101/6) [5] sets a minimum design velocity of 0.9 m/s for stormwater pipes (0.6 m/s for sanitary sewers) and a maximum of 3 m/s for all pipes (Clause 2-4/5/1). In addition, gravity flow requires the flow depth not to exceed 80% of the internal pipe diameter (depth-to-diameter ratio (d/D) ≤ 80%, Clause 2-4/3/1) [5]; above this depth, free-surface gravity flow is no longer ensured, and the pipe may become surcharged, with hydraulic grade line (HGL) elevations propagating upwards and potentially above the ground surface at the manhole [1]. This dual Iraqi Code criterion — velocity adequacy and d/D compliance — is the normative basis for the present study. Urban flooding incidents in Iraq have increased in incidence and severity due to hydroclimatic forcing and the lack of urban drainage infrastructure [6]. Studies using SWMM have evaluated the effect of climate change on flooding in Kerbala City [7, 8] and modelled the combined impacts of land-use and climate change on the sewer system in Al-Najaf [9]. A study on stormwater drainage management in Iraqi urban conditions identified hydraulic deficiencies and provided recommendations for rehabilitation [10]. These studies consistently highlight that existing Iraqi drainage networks operate below design capacity under current and projected rainfall conditions. Shamkhi et al. [11] obtained locally calibrated Intensity–Duration–Frequency (IDF) curves for Al-Kut City from rainfall records for 1992–2019 (20 years of available data). Similar IDF analyses for Baghdad [12],

Babylon [13], and Basrah [14] confirmed significant spatial variability in design rainfall intensities across Iraqi governorates and the need for city-specific IDF data for hydraulic network assessment. SewerGEMS has been widely applied to the design and assessment of sewer networks, as reviewed by Bhagat et al. [15], including GIS-integrated network assessment in Kabul [16], and its results have been verified against standard Manning equation-based gravity-flow analysis [17]. In Baghdad, velocity deficiencies leading to sediment deposition and progressive capacity reduction have been documented in combined trunk sewers [10, 18]. Multi-scenario rainfall assessment has been shown to be necessary to characterise the full range of network performance from adequacy to capacity deficiency [19], motivating the comparative approach used in the present study, which contrasts the Iraqi Code design intensity with a 100-year extreme intensity. Previous Iraqi studies have mostly applied SWMM to selected districts, trunk sewers, or sanitary systems [7-9, 18, 20-22], or have addressed network design optimisation rather than the assessment of an existing network; in Al-Kut, the only drainage-network study identified by the authors optimised the layout of a combined system for two districts [23]. These studies generally report flooding volumes or flooded nodes for a single network area and do not evaluate compliance with the Iraqi Code velocity and d/D criteria at the level of individual pipes. To the best of the authors' knowledge, no published study has assessed the complete stormwater network of Al-Kut, covering both banks of the Tigris River, under both the Iraqi Code design intensity and a locally derived 100-year intensity. The present study differs from previous work in three respects: (i) network coverage — the entire city network of 7,571 pipes in 33 sub-networks is modelled rather than a single district or trunk line; (ii) rainfall scenarios — the code design storm is contrasted with an extreme event derived from the local IDF relationship [11]; and (iii) hydraulic risk assessment — a pipe-by-pipe cross-scenario comparison separates pipes that exceed the design limits in both scenarios, which require structural rehabilitation regardless of storm severity, from pipes that exceed them only in the extreme scenario, for which flood-resilience measures are more relevant, while manholes where the simulated HGL reaches ground level are screened as potential overflow locations. For Al-Kut, this distinction provides a basis for prioritising rehabilitation and climate-adaptation investments across the city. The objectives of this study are therefore to: (1) build a city-wide hydraulic model with SewerGEMS integrated with GIS; (2) apply the Iraqi Code design intensity (13 mm/h, 20-min duration) and a 100-year extreme intensity (91.42 mm/h, 30-min duration) derived from the local IDF relationship [11]; (3) evaluate flow velocity and d/D performance against the Iraqi Building Code 2021 thresholds [5]; and (4) classify pipes and manholes by cross-scenario deficiency to support rehabilitation prioritisation.

2. LITERATURE REVIEW

2.1 Iraqi Building Code 2021 — Design criteria for stormwater networks

The required framework for hydraulic design of all stormwater pipe networks in Iraq is given in 101/6, Ministry of Construction, Housing and Public Municipalities, First Edition, 2021 [5]. The following requirements are given in the Code: (i) a minimum design velocity of 0.9 m/s for storm water pipes to ensure self-cleansing without sediment deposition (Clause 2-4/5/1, item 2); (ii) a maximum design velocity of 3 m/s to avoid erosive damage to pipe walls and joints (Clause 2-4/5/1, item 4); (iii) a maximum flow depth of 80% of the internal pipe diameter for gravity flow, beyond which free-surface gravity flow is no longer ensured (Clause 2-4/3/1, items a-1 and b); and (iv) a design storm for central and southern Iraq of not less than 37.06 L/s/ha (≈ 13 mm/h) lasting 20 min with a 2-year return period (Clauses 2-3/1/1/1 to 2-3/1/1/3, Eq. 2-3/1). The Code also allows the client's own study to be used to estimate the design storm intensity under changing climate conditions (Clause 2-3/1/1/2).

The minimum velocity criterion in the Iraqi Code 2021 [5] is 0.9 m/s. This high threshold is needed due to high sediment loads and high ambient temperatures in Iraqi urban areas, which allow faster biological and chemical consolidation of sediments in trunk sewer systems, as reported by Al-Ani et al. [18] in relation to sewer systems in Baghdad. The Manning's roughness coefficient values for PVC pipe (n = 0.009 to 0.011) determined by the Uni-Bell PVC Pipe Association [2] in the field for in-service PVC pipe confirm that PVC networks are less rough than concrete or clay networks and can therefore have lower slopes to achieve self-cleansing velocities if pipe gradients are considered adequate.

2.2 Rainfall intensity-duration-frequency analysis in Iraq

The generation of accurate IDF relationships is the basic element of any hydraulic design and evaluation of an urban stormwater drainage system. Researchers [11] obtained IDF curves specifically for Al-Kut City using rainfall records for 1992–2019 (28 years of available data) and fitted three types of probability distributions (Gumbel, lognormal, Log-Pearson Type III) using EasyFit 5.5 software; they concluded that the lognormal distribution was the most statistically robust fit. Mahdi and Mohamedmeki [12] performed an IDF analysis on rainfall data from 30 years of daily records for the city of Baghdad, and calculated the coefficient of the Sherman equation for return periods of 2 to 100 years, confirming that design rainfall varies from one Iraqi governorate to another. Al-Jalili et al. [13] developed IDF curves for the city of Babylon over 32 years of daily rainfall data (1991–2022) using the Indian Meteorological Department (IMD) reduction formula, and Al-Aboodi et al. [14] prepared IDF curves for return periods ranging from 2 to 100 years for the city of Basrah using the Sherman equation. These studies highlight the need for city-specific IDF data for hydraulic network assessment.

2.3 Hydraulic modelling using SewerGEMS

Bentley OpenFlows SewerGEMS is one of the most widely proven hydraulic modelling, design, and performance assessment platforms for stormwater drainage systems worldwide. The software is used for hydraulic calculations, which calculate pipe diameters, pipe invert levels, pipe flow depth, and pipe velocity for each network element by using the design discharge data and pipe attribute data [24] as input for Manning's equation [1]. Research [15] presented a comprehensive review of sewer network design by SewerGEMS, summarising its use for sizing pipes, setting slopes, and checking velocities against design criteria. Research [17] verified that SewerGEMS V8i is accurate and

reliable, as it gives the same results in the design of the sewerage network as those obtained by gravity-flow analysis using the Manning equation. Research [16] applied ArcGIS and SewerGEMS to assess the sewer network in Kabul City, Afghanistan, using GIS-derived network geometry in building the hydraulic model; they concluded that the integrated GIS–SewerGEMS approach was a viable solution for data-poor urban areas. The study by Abd-Elhamid et al. [19] showed that multi-scenario rainfall assessment is needed to determine whether the network will be adequate or not as storm severity increases, using StormCAD — a similar Bentley platform — integrated with GIS.

2.4 Stormwater and sewer network performance in Iraq

Al-Ani et al. [18] assessed the performance of the combined sewage trunk system in Al-Rusafa region of Baghdad and concluded that the design capacity was not operating at the maximum practical conditions and hydraulic gradients were not sufficient to maintain self-cleansing velocities, leading to the sedimentation and progressive clogging of the trunk system. Research [10] performed a hydraulic analysis of combined trunk sewers in Baghdad city and reported the existence of many hydraulic deficiencies due to ageing infrastructure and overloading caused by population growth.

Hussain et al. [9] modelled the impacts of land-use and climate change on the performance of the stormwater sewer system for the City of Al-Najaf using SWMM, concluding that the impacts of climate change had a larger adverse hydraulic impact than the impacts of land-use change. Research [20, 21] studied flooding and overflow in Al-Samawah, finding that inadequate capacity causes widespread sewer overflows even under moderate rainfall. Studies [7, 22] assessed flood mitigation using SWMM in Kerbala and Najaf City, respectively, both showing that existing urban drainage facilities in Iraqi cities are generally insufficient for the design flooding conditions.

Hassan et al. [8] simulated the impact of climate change on flooding of stormwater networks in Kerbala City and found that the surcharge in the network due to extreme rainfall events with future climate projections is significantly higher than that of the historical design rainfall events, which highlights the need to assess the existing Iraqi stormwater networks for all the design rainfall events specified in the Iraqi Building Code 2021 [5]. Study [6] showed, at a national scale, that the interaction between hydroclimatic forcing and chronic infrastructure deficiency is the primary driver of flood hazard in Iraq.

2.5 Pipe materials and hydraulic roughness

The hydraulic performance of gravity-flow sewer systems is highly sensitive to the Manning's roughness coefficient (n) attributed to each pipe material, which directly affects the relationship between pipe velocity, pipe slope, and pipe hydraulic radius in the Manning equation [1]. Chow [1] provided the theoretical background for the Manning equation in open channel flow hydraulics, which states that for a given pipe diameter, the hydraulic radius for a partially full circular pipe is dependent on the flow d/D, and that peak velocity occurs at about 81% of the pipe diameter rather than full pipe depth, which is counterintuitive and has direct relevance to the d/D assessment of pipe flow performance in this study.

The Uni-Bell PVC Pipe Association, based on 25 field measurements on in-service PVC sewer pipe, found that PVC pipe retains its low n value for smooth pipe, and there is no measurable slime build-up on the inside of the pipe during service life. Metcalf & Eddy and AECOM [4] reported Manning's n values of approximately 0.009–0.011 for smooth plastic/PVC sewer pipes, and provide extensive design criteria for gravity sewer systems such as minimum self-cleansing velocities, maximum permissible velocities, and maximum full-flow capacity ratios that is in compliance with the Iraqi Code 2021 thresholds [5].

3. STUDY AREA

3.1 Geographical and administrative context

Al-Kut City is the administrative capital of Wasit Governorate, located on the Tigris River in central-southern Iraq, 180 km southeast of Baghdad. There are two main urban sectors in the city — the left side and the right side — which are divided by the Tigris River. This study covers the complete stormwater drainage system of Al-Kut City, including both the left and right sides, which together contain the main residential, commercial, and administrative areas of the city. The study area has a total drainage surface of about 25.54 km² and serves a population of about 464,920 people.

The topography of the study area has the characteristics of the Mesopotamian alluvial plain, with ground surface elevations in the surveyed network ranging from about 8.4 m to 24.7 m above mean sea level (AMSL). This limited overall relief, coupled with the locally variable micro-topography, results in a system of generally low pipe gradients — a direct relevance to the self-cleansing velocity compliance results presented in Section 5.

3.2 Climate characteristics

Al-Kut experiences a hot semi-arid (BSh) climate under the Köppen classification, with long, intensely hot, dry summers and short, mild winters during which the majority of annual rainfall occurs. As established by Shamkhi et al. [11], rainfall intensity in Al-Kut increases systematically with increasing return period and decreases with increasing storm duration. This pattern is consistent with the IDF characteristics documented by Mahdi and Mohamedmeki [12] for Baghdad City and by Al-Jalili et al. [13] for Babylon City, confirming the broad regional consistency of Iraqi rainfall frequency characteristics. The pronounced rainfall seasonality means that stormwater network adequacy must be verified against short-duration, high-intensity events — the central design consideration addressed by the comparative assessment in this study.

3.3 Existing stormwater network of Al-Kut City

The stormwater drainage system of Al-Kut City is a gravity-fed, separate storm sewer system, consisting of all polyvinyl chloride (PVC) pipes. The pipe network analysed has 7,571 pipe segments that are approximately 328.3 km in total length, connected to 7,571 manhole nodes that receive the runoff. All pipe segments were assigned a Manning's roughness coefficient, n = 0.011, which is consistent with Manning's n values found in the literature for smooth-walled PVC sewer pipes [2]. The internal diameters of pipes range from 300 mm

to 1,400 mm, with 65.5% of pipe segments being 315 mm in diameter. The topological structure of the network consists of thirty-three separate sub-networks (22 on the left bank and 11 on the right bank), each draining a distinct part of the city catchment to its own wet well and pumping station. The main characteristics of the network are summarised in Table 1.

Table 1. Summary characteristics of the Al-Kut City stormwater network covering both banks of the Tigris River (SewerGEMS inventory)

Parameter	Value	Unit
Drainage area (both banks)	25.54	km²
Served population	464,920	inhabitants
Total pipe network length	328.3	km
Number of pipe segments modelled	7,571	—
Number of manholes modelled	7,571	—
Number of distinct sub-networks	33	—
Pipe diameter range	300–1,400	mm
Ground elevation range	8.4–24.7	m AMSL
Mean pipe slope	0.0065	m/m
Pipe material	PVC	—
Manning's n coefficient	0.011	—

4. MATERIALS AND METHODS

4.1 Overall methodological framework

The methodology integrates four principal components: (1) GIS data compilation and network digitisation; (2) determination of design rainfall intensities from the Iraqi national code; (3) construction of a steady-state hydraulic model using Bentley OpenFlows SewerGEMS [24]; and (4) comparative hydraulic simulation under two design rainfall scenarios, with all performance evaluation conducted against the mandatory criteria of the Iraqi Building Code 2021 [5].

4.2 GIS data compilation and network digitisation

Following the integrated GIS–SewerGEMS approach demonstrated by Noori and Singh [16] in sewer network modelling under data-constrained urban conditions, the network geometry, comprising pipe alignment, pipe diameters, pipe lengths, invert elevation, ground elevation, and manhole locations, was obtained in GIS format from the as-built records of the Wasit Sewerage Directorate. Ground elevations were surveyed by the Directorate in 2025. Coordinates are in UTM Zone 38N (WGS 84), and elevations are in m AMSL. The GIS network dataset (7,571 pipe segments and 7,571 manhole nodes) was imported into SewerGEMS with the ModelBuilder tool, ensuring full consistency between the spatial network representation and the hydraulic model topology.

4.3 Design rainfall intensities

Two rainfall scenarios were adopted. The code scenario uses the design storm of the Iraqi Building Code 101/6 for central and southern Iraq, $I = 3336/(t + 70)$, which gives not less than 37.06 L/s/ha (≈ 13 mm/h, 0.013 m/hr) for a 20-min duration and a 2-year return period, as specified in Clauses 2-3/1/1/1 to 2-3/1/1/3 and Eq. 2-3/1 of the Code [5]; it represents the condition the network is intended to accommodate. The extreme scenario represents a 100-year event derived from the locally calibrated IDF relationship of Shamkhi et al. [11], as permitted by Clause 2-3/1/1/2 of the Code, which allows the design intensity to be estimated from the client's own study to account for changing climate conditions [5]. Reference [11] fitted Gumbel, lognormal, and log-Pearson Type III distributions to annual maximum rainfall at Al-Kut (record period 1992–2019) and derived short-duration intensities from daily rainfall using the IMD reduction formula; the lognormal distribution gave the best fit (lowest LSSMIC = 0.829) and yields 91.42 mm/h for the 100-year return period at 30-min duration (Table 6 in research [11]). The two intensities differ by a ratio of about 7.03.

Storm duration and time of concentration. In the Rational Method, the storm duration is taken equal to the time of concentration, $t = t_i + t_f$, where the inlet time t_i is about 10 min for paved streets in Iraqi cities (Clause 2-3/1/1/2 [5]). In the present model, inflows were assigned at each manhole from its local contributing area, for which the time of concentration is dominated by the inlet time and is of the order of the 20-min minimum duration specified by the Code. The 30-min duration was used for the extreme scenario because it is the shortest duration tabulated by Shamkhi et al. [11]. Because rainfall intensity decreases with duration, the 30-min value is a conservative (lower) estimate of the 100-year intensity at 20 min; applying the IMD relationship ($P \propto t^{1/3}$) used by Shamkhi et al. [11] gives about 120 mm/h at 20 min, i.e., about 30% higher. In the steady-state simulation, the duration does not enter the hydraulic computation directly: it only determines the intensity applied, and each scenario represents a constant peak-flow loading. Since local peak inflows are summed without routing attenuation, downstream trunk flows are upper-bound estimates in both scenarios, so the two scenarios are compared on a consistent basis.

Uncertainty of the 100-year estimate. The 100-year intensity is an extrapolation well beyond the record length: although the record period is 1992–2019, data for 2003–2010 are missing, so research [11] fitted the distributions to a sample of 20 annual maximum daily rainfalls (Tables 1 and 7 in research [11]), which include an exceptional event in 2018 (222 mm). Refitting a lognormal distribution to the 20 annual maxima of 30-min rainfall depth listed in Table 1 of research [11] reproduces the 100-year value within about 2% (46.7 mm vs. 45.71 mm) and, using the standard error of a lognormal quantile [25], gives an approximate 90% confidence interval of 67–130 mm/h for the 100-year, 30-min intensity. The extreme scenario should therefore be interpreted as a stress test of the network rather than a precise estimate of the 100-year event.

4.4 Hydraulic model construction in SewerGEMS

4.4.1 Software platform and Manning's equation

The hydraulic model was created in the Bentley OpenFlows SewerGEMS [24] software, which is used for gravity-flow hydraulic modelling using Manning's equation [1]. If the pipe is partially filled, the flow depth and velocity will not change linearly with the d/D ratio, and the maximum velocity will occur at about 81% of the full-pipe flow depth in accordance with the experimental findings by Chow [1]. The steady-state hydraulic simulation was used as the main type of analysis [15]. A Manning's n of 0.011 was used for all PVC pipes; this is the upper end of the range of 0.009–0.011 measured for in-service PVC sewers [2] and recommended by Metcalf & Eddy and AECOM [4], and was selected as a conservative value to allow for joints, minor deposits, and ageing.

4.4.2 Sub-catchment delineation and rational method

Sub-catchments were delineated with the Thiessen polygon tool of SewerGEMS, which assigns to each manhole the part of the study area that is closer to it than to any other manhole. The 7,571 resulting polygons cover the entire study area of 25.54 km², with a median area of 2,813 m² (5th–95th percentile: 1,100–7,002 m²). The peak inflow of each sub-catchment was computed with the Rational Method, which is specified by the Iraqi Code for stormwater networks (Clause 2-3/1/2 [5]) and is intended for small urban catchments such as these [25]:

$$Q = C \times i \times A \quad (1)$$

where, Q is the peak inflow, C is a dimensionless runoff coefficient, i is the design rainfall intensity, and A is the sub-catchment area. The inflows were assigned to the manholes as constant loads with the LoadBuilder tool. The expected total inflow is therefore 0.7×0.013 m/h $\times 25.54 \times 10^6$ m² $\approx 232{,}414$ m³/h; the total actually loaded in the model, from the 7,571 Thiessen polygons (25.541 km²), is 232,424 m³/h (64.6 m³/s) for the code scenario and 1,634,117 m³/h (453.9 m³/s) for the extreme scenario, which equals the total discharge simulated at the 33 network outlets in both scenarios. A uniform runoff coefficient of $C = 0.7$ was adopted because a parcel-level land-use map was not available. This value lies within the ranges given by the Iraqi Code for the land uses that dominate Al-Kut — 0.6–0.75 for attached residential housing and 0.7–0.95 for the city centre (Table 2-3/1 [5]) — and is consistent with values used for similar urban catchments [19]. Because the Rational Method is linear in C, a change of ±0.1 in C changes all inflows by about ±14%; the sensitivity of the results to C (0.6–0.8) is presented in Section 4.6 and does not alter the main conclusions.

Thiessen polygons give a geometric allocation of the contributing area to each manhole, but do not account for all surface drainage pathways and inlet-capture processes. An assumption for the local inlet time is taken as being approximately 10 minutes and is not a time of concentration measured along the entire drainage network. Similarly, the uniform runoff coefficient $C = 0.7$ is an assumed value, and not a uniform surface characteristic. The resulting nodal inflows applied simultaneously will support steady-state screening, but will not include network travel-time effects or transient storage.

4.4.3 Solver, boundary conditions and treatment of overflow

The model was run in Bentley OpenFlows SewerGEMS CONNECT Edition [24] with the GVF-Convex (SewerCAD) numerical solver in steady-state analysis mode (pipe sizes were not redesigned), which computes gradually varied flow profiles in each conduit and switches to pressure-flow computation where a conduit flows full. Ground elevations were taken from a terrain model based on a survey by the Wasit Sewerage Directorate (2025), and pipe invert elevations from its as-built records (Section 4.2). Each of the 33 sub-networks drains by gravity to its own wet well at a stormwater pumping station (33 wet wells in total), from which flow is pumped to outfalls on the Tigris River, modelled as free outfalls. Because the gravity network discharges into pumped wet wells rather than directly into the river, Tigris River stages do not propagate as backwater into the gravity pipes; they affect only the pumping head, which is outside the scope of this study. Where the computed HGL reaches the rim of a manhole, the solver limits it to the rim (ground) elevation; the volume, depth, and extent of any resulting surface overflow are not simulated. Manholes where the HGL reaches ground level are therefore reported as potential overflow locations, and all such results are presented as screening indicators.

Direct Tigris River tailwater to the pumped wet wells is not included in the simulated gravity networks in the free outfall representation. This is an assumption for boundary conditions and does not imply that actual wet well operating levels are not related to pump performance or river stage. The following factors were not simulated in the pump operation: variations in wet-well level, sensitivity of the water level downstream of the pump. Gravity-network HGLs are thus limited by their possible effect on gravity networks.

Pipe direction, adverse gradients and full-flow capacity. In the GIS inventory, 4,167 pipes (55.0%) have a lower invert at their start node than at their stop node, so they appear to have an adverse gradient when read in the digitising direction. They were identified by comparing the start and stop invert elevations of every pipe. Their direction was then verified with the exact link topology of the model: the 7,571 pipes and 7,604 nodes form 33 branched networks without loops, one per wet well, so each pipe has a single downstream direction, towards its wet well. Read in this direction, none of the 4,167 pipes has an adverse gradient. All of them were digitised from their downstream to their upstream manhole, and their invert falls in the direction of flow; only one pipe (P-1284) is flat. SewerGEMS orients each pipe in the direction of flow: the HGL (Out) of every pipe equals the HGL of its downstream manhole. The reference slope for Q_{full} is the invert difference between the upstream and downstream ends divided by the pipe length. For the 4,167 pipes, this equals the absolute value of the slope in the digitising direction, so using the absolute slope does not overestimate their capacity; an independent Manning calculation with this slope reproduces the SewerGEMS Q_{full} (median ratio 1.000). The flat pipe has zero or near-zero capacity and is classified as $Q > Q_{full}$ in both scenarios, whatever small reference slope is used. The pressurised-flow test (Section 5.3) also uses the inverts of the upstream and downstream ends in the direction of flow. The apparent adverse gradients are therefore a digitising artefact and do not affect the $Q > Q_{full}$ classification or the main conclusions.

4.5 Hydraulic performance assessment criteria — Iraqi Building Code 2021

The assessment of all hydraulic performances is carried out only against the obligatory requirements of the Iraqi Building Code 2021 [5], as summarised in Table 2.

(a) Minimum Design Flow Velocity: 0.9 m/s, which is the minimum velocity specified by Clause 2-4/5/1 of the Iraqi Code [5] for stormwater pipe networks, with a risk of sediment deposition that is not acceptable.

(b) Maximum Design Flow Velocity: 3.0 m/s — maximum velocity prescribed by Clause 2-4/5/1 of the Iraqi Code [5] for preventing erosive damage to pipe walls and joints.

(c) Maximum Flow Depth Ratio (d/D): 80% is the maximum flow depth ratio for gravity flow specified by Clause 2-4/3/1 of the Iraqi Code [5]; flow depths above this limit indicate the loss of free-surface gravity flow, while pressurised flow is identified separately from the HGL relative to the pipe crown (Section 5.3).

(d) HGL and Manhole Freeboard (FB): The ground

elevation was compared to the HGL level at each manhole. A freeboard of zero or less indicates a potential overflow location (Section 5.4).

Table 2. Hydraulic assessment criteria: Iraqi Building Code 2021 requirements, model parameters and screening thresholds adopted in this study

Criterion	Threshold	Type	Source
Min. design velocity (storm pipes)	≥0.9 m/s	Code	[5], Cl. 2-4/5/1(2)
Max. design velocity	≤3.0 m/s	Code	[5], Cl. 2-4/5/1(4)
Max. depth ratio for gravity flow	d/D ≤ 80%	Code	[5], Cl. 2-4/3/1(a-1)
Design storm (central and southern Iraq)	≥37.06 L/s/ha, 20 min, 2-yr	Code	[5], Cl. 2-3/1/1/1–3
Runoff coefficient, C	0.7 (code range 0.6–0.95)	Model	[5], Table 2-3/1
Manning's n (PVC pipe)	0.011	Model	[2]
Potential overflow	Freeboard ≤ 0 m	Screening	This study, [20]
Critical freeboard	0 < Freeboard < 0.3 m	Screening	[24]
Warning/safe freeboard	0.3–1.0 m/≥ 1.0 m	Screening	This study

Note: Depth-to-diameter ratio (d/D).

4.6 Model verification and sensitivity

No field measurements of flow, water level, or flooding are available for the Al-Kut stormwater network, so the model could not be calibrated or independently validated. Instead, the numerical consistency of the model was verified, and the sensitivity of the results to the main uncertain parameter was tested; the results are therefore presented as screening indicators. (i) Mass balance: in both scenarios, the total inflow loaded at the manholes (232,424 and 1,634,117 m³/h) equals the total discharge simulated at the 33 outlets to within 0.001%, and their ratio (7.031) equals the ratio of the rainfall intensities, as expected from the linear Rational Method. (ii) Manning-equation consistency: the full-flow capacity reported by SewerGEMS agrees with an independent Manning calculation (n = 0.011, upstream-to-downstream pipe slope; Section 4.4.3) to within 0.1% (median ratio 1.000); for pipes flowing full, the simulated velocity equals Q/A; and for the 101 pipes with free-surface flow in the code scenario (d/D ≤ 80%, positive slope and HGL below the pipe crown), simulated velocities agree with the partially full Manning velocity computed from the circular-section relationships of Chow [1] (median ratio 0.94, r = 0.74, MAE = 0.14 m/s). These are internal checks of the hydraulic computation, not validation against observations. (iii) Sensitivity to the runoff coefficient: the code scenario was re-run with C = 0.6 and C = 0.8. The share of pressurised pipes changed only from 98.1% to 98.8%, the share of pipes carrying more than their full-bore capacity from 22.2% to 28.2%, and the share of manholes where the HGL reached ground level from 49.7% to 58.2% (54.6% for C = 0.7), so the main conclusions are not sensitive to C within this range. Sensitivity to outlet tailwater was not tested because the outlets are pumped wet wells rather than free river outfalls (Section 4.4.3); this is stated as a limitation in Section 5.8. The findings are also qualitatively consistent with documented capacity deficiencies of Iraqi drainage networks, including insufficient hydraulic gradients and capacity in Baghdad trunk sewers [18] and system failure under critical rainfall in Al-Najaf [9]; this consistency is supporting evidence only. Calibration against flow and water-level monitoring and against documented flooding events is recommended as future work.

Rainfall-to-inflow ratio is a consistency check and not a validation of observed network performance, nor is an independent Manning calculation. The finding of hydraulic deficiency everywhere in C = 0.6–0.8 is retained in the runoff-coefficient sensitivity runs, with a range of 49.7% to 58.2% of manholes in each of the runs indicating potential overflow. The interpretation is thus sound in terms of the extent of deficiency, and the number of affected locations is sensitive to the runoff coefficient assumed. There is a need for field measurements for calibration and independent validation.

5. RESULTS AND DISCUSSION

5.1 Overview of simulation outputs

The steady-state hydraulic simulation using Bentley OpenFlows SewerGEMS [24] provided outputs for 7,571 pipe segments and 7,571 manhole nodes under both the code and extreme scenarios (total length of the network modelled was about 328.3 km). All performances are reported in comparison to the required conditions in the Iraqi Building Code 2021 [5]: minimum velocity ≥ 0.9 m/s, maximum velocity ≤ 3.0 m/s, and maximum d/D ≤ 80%. All PVC pipe sections have n = 0.011.

5.2 Flow velocity analysis against Iraqi Code 2021

The simulated velocities were classified against the limits of the Iraqi Code 101/6 (2021), clause 2-4/5/1 (2) and (4), p. 30/2 [5]: velocities between 0.9 and 3.0 m/s inclusive are treated as compliant, and velocities above 3.0 m/s as exceeding the maximum limit (Table 3 and Figure 1). Under the code scenario, 70.8% of the pipes (5,364 pipes, 225.2 km) flowed below the 0.9 m/s minimum, 22.3% (1,689 pipes, 77.3 km) were compliant, and 6.8% exceeded 3.0 m/s. Under the extreme scenario, the share below 0.9 m/s fell to 19.1% (1,443 pipes, 58.1 km), while 46.2% of the pipes exceeded 3.0 m/s.

The velocity results should be read in two groups. The first group comprises the pipes whose discharge does not exceed their full-flow capacity ($Q \le Q_{full}$; 5,668 pipes under the code scenario). These pipes are surcharged mainly by backwater from downstream, and 92.9% of them flow below 0.9 m/s, with a median of 0.25 m/s. This combination of full pipes and low velocities indicates conditions favourable to sediment deposition under the code storm. The second group comprises the overloaded pipes ($Q > Q_{full}$; 25.1% under the code scenario and 74.5% under the extreme scenario), with median velocities of 2.03 and 4.33 m/s, respectively. Because the steady-state solver caps the hydraulic grade at ground level but does not remove the overflowing volume, the whole inflow is still routed through these pipes. Their computed velocities are therefore upper-bound indicators of hydraulic overload rather than realistic estimates, and should not be interpreted as a confirmed erosion risk.

The velocity classes are also defined in a consistent manner; a velocity range of $0.9 \le V \le 3.0$ m/s is considered compliant; $V > 3.0$ m/s is considered an exceedance of the maximum

criterion. The velocities in overloaded pipes are conditional steady-state screening results, as the solver will hold prescribed discharge in the event of potential overflow. These are not to be used as a measured velocity or as a precise prediction of the flow conditions during surface flooding.

Table 3. Flow velocity distribution under the code and extreme scenarios, classified against the velocity limits of the Iraqi Code 101/6 (2021), clause 2-4/5/1 [5]

Velocity Range (m/s)	Code (n)	Code (%)	Extreme (n)	Extreme (%)	Iraqi Code
< 0.3 (Critically low)	3,168	41.8%	148	2.0%	Below min.
0.3 to < 0.6 (Very low)	1,421	18.8%	549	7.3%	Below min.
0.6 to < 0.9 (Below code)	775	10.2%	746	9.9%	Below min.
Total < 0.9 (non-compliant)	**5,364**	**70.8%**	**1,443**	**19.1%**	Below min.
0.9 ≤ V ≤ 3.0 (Compliant)	1,689	22.3%	2,629	34.7%	PASS
> 3.0 (Exceeds code max.)	518	6.8%	3,499	46.2%	Above max.
Total	**7,571**	**100%**	**7,571**	**100%**	

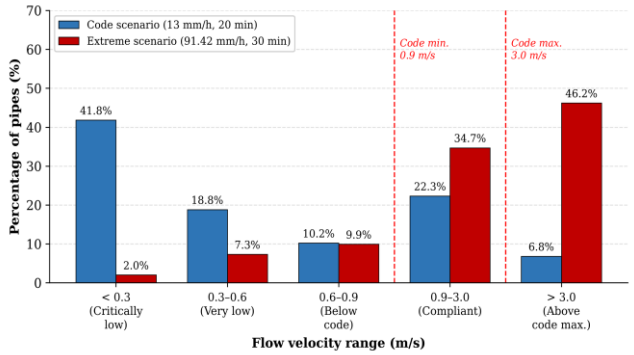

Figure 1. Comparative flow velocity distribution under the code and extreme scenarios against the Iraqi Code 101/6 (2021) velocity limits of 0.9 and 3.0 m/s [5]

5.3 Flow depth ratio and pressurised flow against Iraqi Code 2021

In the SewerGEMS output, d/D (depth/rise) is the computed flow depth divided by the pipe rise and is capped at 100%. A value of 100% therefore means that the pipe is flowing full, not that it is necessarily under pressure. A pipe was classified as pressurised when the simulated hydraulic grade at either end exceeded the pipe crown (invert + D) by more than 5 mm. Its discharge was also compared with its full-bore Manning capacity (Q/Q_{full}). These criteria give four mutually exclusive flow states (Table 4 and Figure 2): (i) gravity flow complying with the code limit of d/D ≤ 80% (clause 2-4/3/1, pp. 18–19/2 [5]); (ii) d/D > 80% with the HGL still below the crown; (iii) pressurised flow with $Q \leq Q_{full}$, where the pipe is surcharged from downstream although its own capacity is sufficient; and (iv) pressurised flow with $Q > Q_{full}$, where the capacity of the pipe is exceeded.

Pipe filling, pressurisation, and discharge relative to Q_{full} represent different hydraulic properties. The d/D indicator is used for reporting filling; pressurisation is identified as a positive HGL below the crown, and Q/Q_{full} compares the simulated discharge with the reference full-flow Manning capacity. Some of the indicators may overlap. Downstream backwater can cause a pipe to have a flow less than Q_{full} when it is full and pressurized. The reported 73.4% is for the same code-scenario group, but is not a percentage of only the pressurised pipes in the inventory.

Table 4. Pipe flow states under the code and extreme scenarios: Mutually exclusive classes and overlapping indicators, against the Iraqi Code 101/6 (2021) limit of d/D ≤ 80% [5]

Flow State	Code (n)	Code (%)	Extreme (n)	Extreme (%)	Iraqi Code
Gravity flow, d/D ≤ 80%	101	1.3%	0	0.0%	PASS
d/D > 80%, HGL below crown	12	0.2%	3	< 0.1%	FAIL
Pressurised, $Q \leq Q_{full}$ (backwater surcharge)	5,557	73.4%	1,927	25.5%	FAIL
Pressurised, $Q > Q_{full}$ (capacity exceeded)	1,901	25.1%	5,641	74.5%	FAIL
Total	**7,571**	**100%**	**7,571**	**100%**	
Overlapping indicators:					
All pipes with d/D > 80%	7,451	98.4%	7,570	> 99.9%	
Pipes flowing full (d/D = 100%)	7,417	98.0%	7,565	99.9%	
All pressurised pipes (HGL > crown)	7,458	98.5%	7,568	> 99.9%	
All pipes with $Q > Q_{full}$	1,903	25.1%	5,641	74.5%	

Note: depth-to-diameter ratio (d/D); hydraulic grade line (HGL).

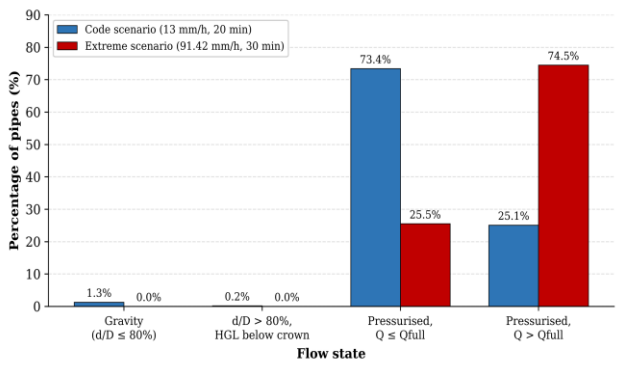

Figure 2. Distribution of pipe flow states under the code and extreme scenarios (mutually exclusive classes; pressurised = hydraulic grade line (HGL) above the pipe crown)

Under the code scenario, only 101 pipes (1.3%, 3.8 km) operate as gravity flow within the code limit. Overall, 98.4% of the pipes exceed d/D = 80%, 98.0% flow full, and 98.5% are pressurised. Most of the pressurised pipes (5,557 pipes, 73.4%, 232.7 km) carry less than their full-bore capacity, whereas 1,901 pipes (25.1%, 91.4 km) are overloaded. Under the extreme scenario, no pipe remains in gravity flow, and the overloaded share rises to 74.5% (5,641 pipes, 250.3 km). The increase in rainfall intensity therefore changes the dominant cause of surcharge more than its extent. Under the code storm, surcharge is consistent with backwater propagated from downstream trunk sections and wet wells (tested in Section 5.7). Under the extreme storm, it is dominated by local capacity exceedance. This widespread surcharge is consistent with the capacity deficiencies reported for other Iraqi storm networks [8, 10, 18].

Figures 3 and 4 map the four flow states on the 33 sub-networks, numbered from the largest (1) and labelled by river bank. The pattern is similar on both banks. Under the code scenario, 98.1% of the left-bank pipes (5,057 pipes, 215.6 km; 22 sub-networks) and 99.3% of the right-bank pipes (2,514 pipes, 112.7 km; 11 sub-networks) are pressurised. The overloaded share is higher on the right bank (31.7% vs. 21.9%). Under the extreme scenario, it rises to 71.5% on the left bank and 80.5% on the right. Sub-network 1, the largest (1,081 manholes, 52.7 km, draining to wet well W-32 on the left bank), is fully pressurised in both scenarios. However, only 18.6% of its pipes are overloaded under the code scenario, rising to 63.6% under the extreme scenario. A sub-network that is fully surcharged while most of its pipes have sufficient capacity points to a downstream constraint at the trunk sewer or the wet well. Pressurised conditions commonly precede surface flooding in Iraqi networks [7], but pressurised flow does not by itself establish flooding. The related potential overflow at manholes is examined in Section 5.4.

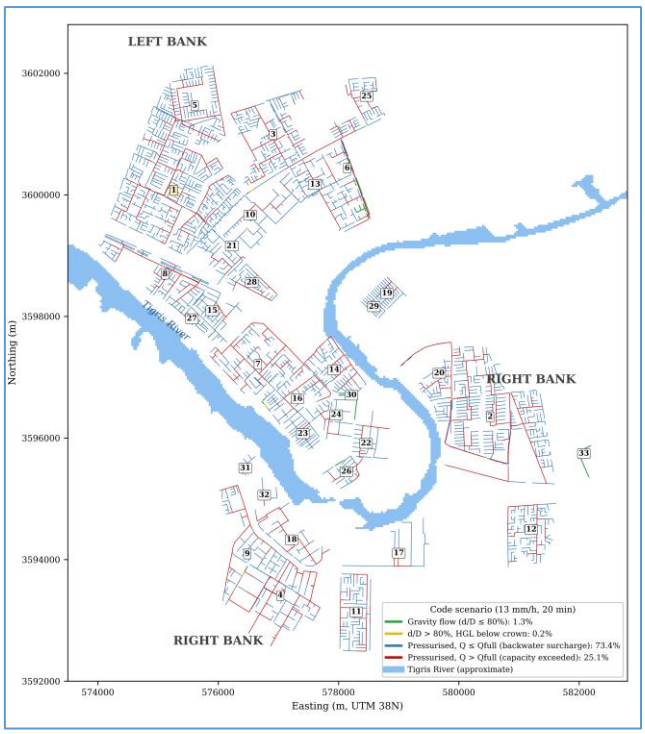

Figure 3. Spatial distribution of pipe flow states under the code scenario (13 mm/h, 20 min)
Note: Numbers identify the 33 sub-networks (1 = largest, highlighted); bank labels refer to the Tigris River (river outline approximate).

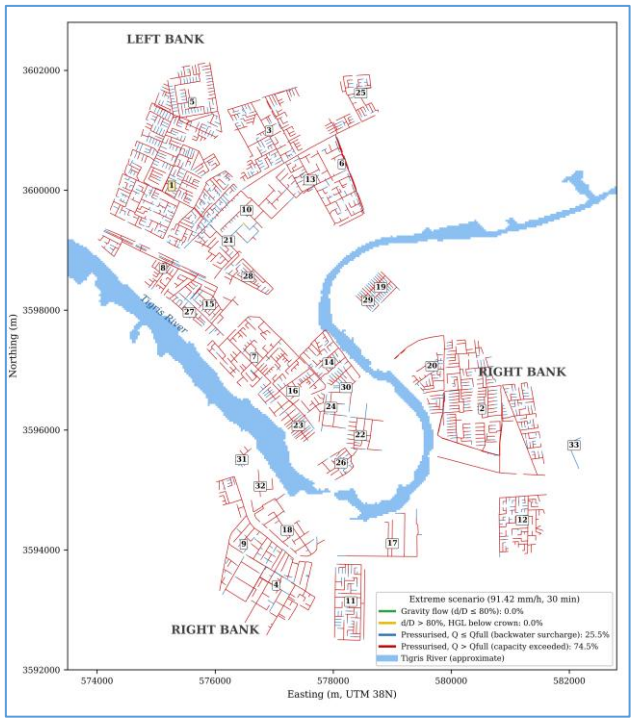

Figure 4. Spatial distribution of pipe flow states under the extreme scenario (91.42 mm/h, 30 min)
Note: Numbers identify the 33 sub-networks (1 = largest, highlighted); bank labels refer to the Tigris River (river outline approximate).

5.4 Hydraulic grade line and manhole freeboard analysis

The FB at each manhole was computed as FB = ground elevation − HGL and classified into four mutually exclusive classes (Table 5 and Figure 5): potential overflow (FB ≤ 0), critical (0 < FB < 0.3 m), warning (0.3 ≤ FB < 1.0 m) and safe (FB ≥ 1.0 m). Under the code scenario, the HGL reached ground level at 4,135 manholes (54.6%); a further 2,105 (27.8%) had a critical FB, and only 384 (5.1%) were safe. Under the extreme scenario, potential overflow extended to 6,946 manholes (91.7%), and only 9 manholes (0.1%) remained safe. The pattern is similar on both banks: under the code scenario, potential overflow was found at 54.9% of the left-bank manholes (2,775) and 54.1% of the right-bank manholes (1,360), rising to 91.3% (4,616) and 92.7% (2,330), respectively, under the extreme scenario.

Table 5. Manhole freeboard (FB) classification under the code and extreme scenarios (mutually exclusive classes; FB = ground elevation − hydraulic grade line (HGL))

FB Class	Code (n)	Code (%)	Extreme (n)	Extreme (%)
Potential overflow (FB ≤ 0)	4,135	54.6%	6,946	91.7%
Critical (0 < FB < 0.3 m)	2,105	27.8%	497	6.6%
Warning (0.3 ≤ FB < 1.0 m)	947	12.5%	119	1.6%
Safe (FB ≥ 1.0 m)	384	5.1%	9	0.1%
Total	**7,571**	**100%**	**7,571**	**100%**

These results indicate where the network may overflow; they do not establish the actual extent or depth of surface flooding. Because the steady-state solver limits the HGL to the

ground elevation and does not route the excess volume over the surface, no negative FB is computed: all potential-overflow manholes have FB = 0, and the median FB is 0.00 m in both scenarios. The mean FB of the remaining manholes falls from 0.39 m under the code scenario to 0.20 m under the extreme scenario. Of the 6,946 potential-overflow manholes under the extreme scenario, 4,135 are already at ground level under the code storm, and 2,811 (37.1%) reach it only under the extreme storm (Section 5.6). Quantifying flood extent and depth would require a dynamic model with surface (1D–2D) routing, which is recommended as future work.

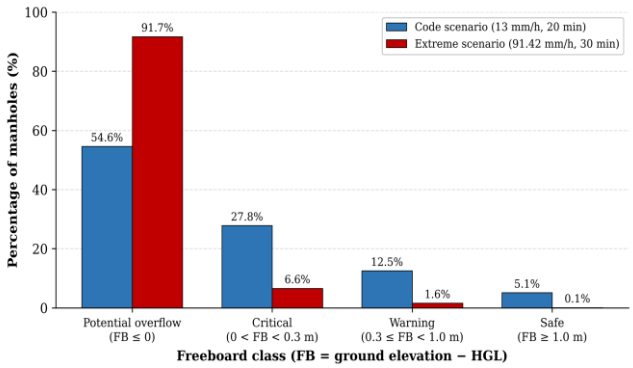

Figure 5. Comparative manhole freeboard (FB) classification under the code and extreme scenarios (FB = ground elevation − hydraulic grade line (HGL))

5.5 Network inflow loading comparison

The total inflow loaded at the manholes was 232,424 m³/h under the code scenario and 1,634,117 m³/h under the extreme scenario, equal to the total discharge simulated at the 33 outlets. The mean nodal inflow over all 7,571 manholes increased from 30.7 to 215.8 m³/h, and the maximum nodal inflow increased from 1,097.9 to 7,719.1 m³/h. The largest pipe discharge, in the outlet pipe of sub-network 1, increased from 29,754 to 209,191 m³/h. The ratio of the two loads (7.03) equals the ratio of the two rainfall intensities, as expected from the linear Rational Method; this is an internal consistency check rather than a validation (Section 4.6).

5.6 Cross-scenario deficiency classification (Iraqi Code 2021)

A pipe-by-pipe comparison of the two scenarios separates the deficiencies that already occur under the code storm from those that appear only under the extreme storm. (i) Pressurised flow: 7,458 pipes (98.5%, 324.1 km) are pressurised in both scenarios, and a further 110 (1.5%) only under the extreme scenario. (ii) Capacity: 1,903 pipes (25.1%, 91.5 km) carry more than their full-flow capacity in both scenarios, so they are under-capacity even for the code design storm; a further 3,738 pipes (49.4%, 158.8 km) are overloaded only under the extreme scenario. (iii) Manholes: 4,135 manholes (54.6%) reach potential overflow in both scenarios, and 2,811 (37.1%) only under the extreme scenario. (iv) Velocity: 1,437 pipes (19.0%, 57.8 km) remain below 0.9 m/s in both scenarios, and a further 3,927 pipes are below 0.9 m/s only under the code storm and reach higher velocities under the extreme storm.

These groups have different management implications. Pipes overloaded in both scenarios require capacity upgrading. Pipes and manholes affected only under the extreme storm are candidates for adaptation measures such as storage and source control. Pipes with persistently low velocities are priorities for inspection and cleaning.

Low velocities in surcharged pipes may favour sediment deposition. If deposition occurs, it would reduce the effective pipe capacity and could aggravate surcharge, as reported for Baghdad trunk sewers [18]. Sediment transport and blockage were not modelled in this study, so this is a possible mechanism rather than a demonstrated finding. It should be checked by CCTV inspection and sediment-depth surveys in the pipes with persistently low velocities.

5.7 Backwater test on a representative sub-network

To test the backwater explanation given in Section 5.3, Figure 6 shows the long profile of the longest flow path in sub-network 2. The path has 50 pipes and is 3,063 m long, running from manhole MH-886 to wet well W-10 on the right bank, with diameters of 315–1400 mm and surveyed ground elevations. Under the code scenario, the first three pipes (153 m) carry 24–58% of their full-flow capacity at velocities of 0.13–0.30 m/s. Yet the HGL at their manholes lies 0.5–0.7 m above the pipe crowns, while remaining 0.4–0.6 m below the ground. Downstream of this reach, 45 of the 50 pipes on the path (90%) carry more than their full-flow capacity (Q/Q_{full} up to 25.9), and the HGL reaches ground level at 43 of the 50 manholes. The surcharge of the upstream pipes is therefore imposed by the overloaded section downstream. This confirms the backwater mechanism and explains why full pipes with $Q \leq Q_{full}$ have low velocities. Under the extreme scenario, all pipes on the path are overloaded ($Q/Q_{full} \geq 1.68$), and the HGL reaches ground level at all 50 manholes. The performance of this sub-network is thus controlled by the capacity of its trunk sewer rather than by local collector pipes.

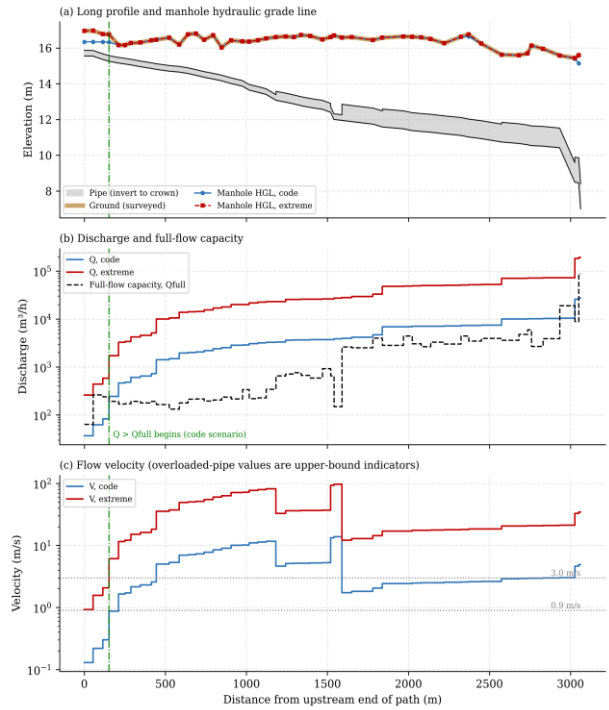

Figure 6. Long profile of the longest flow path in sub-network 2 (MH-886 to wet well W-10) under the code and extreme scenarios: (a) invert, crown, surveyed ground and manhole hydraulic grade line (HGL); (b) discharge and full-flow capacity; (c) velocity (values in overloaded pipes are upper-bound indicators)

Finally, as the code-based and 100-year rainfall scenarios are compared, the amount of rainfall for each simulation is assigned. The severity of the 100-year situation is put into context by studies of rainfall extremes in Baghdad [26, 27] and return-period rainfall depths for the Tigris basin [28]. The extent of network overload as estimated is dependent on the selected rainfall input for Al-Kut. The GIS analysis can be used to gain insight into where runoff is focused and where hydraulic limitations show up. This interpretation is confirmed by the drainage delineation using DEM [29], spatial mapping of flood impacts [30], and soil- and climate-based runoff modelling [31]. The requirement for both design capacity and pipe condition is also emphasized by regional conveyance assessments [32] and sediment buildup in the trunk sewer lines of Baghdad [33] that were observed. Al-Ani and Al-Obaidi [33] evaluated combined trunk sewers in the city of Baghdad during dry weather conditions by using flow velocity and water depth. These indicators are used to interpret the simulated hydraulic response of the Al-Kut network for the two rainfall scenarios.

5.8 Limitations

The results should be interpreted as steady-state screening indicators, subject to the following limitations.

(i) Tailwater. Each sub-network was modelled with a free outfall at its pumped wet well, so the effect of the wet-well water level on the gravity network was not represented. Tailwater sensitivity was not tested; this would require wet-well operating levels (pump start/stop levels and their variation during storms), which were not available for this study. Because a free outfall gives the lowest possible downstream water level, a higher wet-well level could only raise the HGL in the downstream reaches. The reported pressurised fractions and potential overflow counts are therefore likely to be lower-bound estimates for the surcharged conditions.

(ii) Overflow. The solver caps the HGL at ground level and does not route the overflowing volume, so flood extent and depth are not quantified, and velocities in overloaded pipes are upper-bound indicators.

(iii) Loading. The Thiessen allocation, the inlet time of about 10 min, and the uniform C = 0.7 are simplifications. The sensitivity runs with C = 0.6–0.8 show that the conclusions hold, although the number of affected locations varies.

(iv) Calibration. No flow, water-level, or flood records were available, so the model was verified but not calibrated or validated.

6. CONCLUSIONS

In this study, the hydraulic performance of the Al-Kut stormwater drainage network (33 sub-networks on both banks of the Tigris) was evaluated with SewerGEMS and GIS under two steady-state rainfall inputs: the Iraqi Code design storm of 37.06 L/s/ha (≈ 13 mm/h) for 20 min, and the 100-year, 30-min intensity of 91.42 mm/h [11]. The main conclusions are:

1. Under the code design storm, the network does not meet the Iraqi Code criteria. Only 1.3% of the pipes operate as gravity flow with d/D ≤ 80%; 98.5% are pressurised; 25.1% carry more than their full-flow capacity; and the HGL reaches ground level at 54.6% of the manholes.

2. Under the code storm, 70.8% of the pipes flow below the 0.9 m/s minimum velocity. Most of these are full pipes surcharged from downstream (median 0.25 m/s), which may favour sediment deposition. Sediment processes were not modelled, so this needs field confirmation.

3. The main difference between the scenarios is a shift from backwater surcharge to capacity exceedance. Under the extreme storm, 74.5% of the pipes are overloaded, and 91.7% of the manholes reach potential overflow; 3,738 pipes (49.4%) and 2,811 manholes (37.1%) are deficient only under the extreme storm. Results on the two banks are similar.

4. The long profile of a representative sub-network confirms that surcharge in pipes with sufficient capacity is imposed by overloaded trunk sewers downstream. Rehabilitation should therefore give priority to trunk sewers and to the 1,903 pipes that are already overloaded under the code storm.

5. For urban flood risk management and sustainable drainage, the results distinguish three groups: pipes needing capacity upgrades; areas exposed only to extreme rainfall, where storage and source-control measures can reduce risk; and pipes with persistently low velocities that need regular cleaning. The results are steady-state screening indicators. Dynamic 1D–2D modelling calibrated against flow, water-level and flood records is recommended to quantify flood extent and depth.

REFERENCES

[1] Chow, V.T. (1959). Open Channel Hydraulics. McGraw-Hill, New York.

[2] Uni-Bell PVC Pipe Association. (2016). Manning's "n" for PVC gravity sewer pipe. Technical Report. Uni-Bell PVC Pipe Association, Dallas, Texas, USA. https://www.uni-bell.org/Portals/0/ResourceFile/manning-s-n-for-pvc-gravity-sewer-pipe.pdf.

[3] Haleem, N.A., Al-Sulttani, A.O., Ameen, A.M.S. (2026). Hydraulic implications of canal lining types: A case study of the Bani Hassan irrigation system. Journal of Engineering, 32(5): 144-160. https://doi.org/10.31026/j.eng.2026.05.07

[4] Metcalf & Eddy, AECOM. (2014). Wastewater Engineering: Treatment and Resource Recovery. 5th ed. McGraw-Hill Education, New York.

[5] Ministry of Construction, Housing and Public Municipalities, Republic of Iraq. (2021). Iraqi Building Code for Sanitary Sewerage, Sewer Networks, and Lift, Pumping, and Treatment Stations in Cities (Code 101/6, 1st ed., in Arabic). Baghdad, Iraq. https://investpromo.gov.iq/wp-content/uploads/2026/11/iraqi-building-code-25-ar.pdf.

[6] Savelli, E., Mustafa, A.M., Koutroulis, A. (2026). When moderate storms become Disasters: Operationalising the co-production of flood hazard in Iraq. International Journal of Disaster Risk Reduction, 136: 106061. https://doi.org/10.1016/j.ijdrr.2026.106061

[7] Nile, B.K., Hassan, W.H., Esmaeel, B.A. (2018). An evaluation of flood mitigation using a Storm Water Management Model [SWMM] in a residential area in Kerbala, Iraq. IOP Conference Series: Materials Science and Engineering, 433: 012001. https://doi.org/10.1088/1757-899X/433/1/012001

[8] Hassan, W.H., Nile, B.K., Kadhim, Z.K. (2024). Effect of climate change on the flooding of storm water

networks under extreme rainfall events using SWMM simulations: A case study. Modeling Earth Systems and Environment, 10: 4129-4161. https://doi.org/10.1007/s40808-024-01998-1

[9] Hussain, S.N., Zwain, H.M., Nile, B.K. (2022). Modeling the effects of land-use and climate change on the performance of stormwater sewer system using SWMM simulation: Case study. Journal of Water and Climate Change, 13(1): 125-138. https://doi.org/10.2166/wcc.2021.180

[10] Hamza, J.N., Ameen, A.M.S., Mohammad, T.A. (2023). The effects of intake water quality and filtration unit performance on the efficiency of the Al-Rasheed Water Treatment Plant in Baghdad, Iraq. Iraqi Journal of Science, 64(12): 6158-6174. https://doi.org/10.24996/ijs.2023.64.12.7

[11] Shamkhi, M.S., Azeez, M.K., Obeid, Z.H. (2022). Deriving rainfall intensity-duration-frequency (IDF) curves and testing the best distribution using EasyFit Software 5.5 for Kut City, Iraq. Open Engineering, 12(1): 834-843. https://doi.org/10.1515/eng-2022-0330

[12] Mahdi, E.S., Mohamedmeki, M.Z. (2020). Analysis of rainfall intensity-duration-frequency (IDF) curves of Baghdad city. IOP Conference Series: Materials Science and Engineering, 888(1): 012066. https://doi.org/10.1088/1757-899X/888/1/012066

[13] Al-Jalili, S.K., Zwain, H.M., Hayder, A.M. (2024). Generating rainfall IDF curves using IMD reduction formula and choosing the best distribution for Babylon City, Iraq. Engineering Reports, 7(1): e13053. https://doi.org/10.1002/eng2.13053

[14] Al-Aboodi, A.H., Hassan, A.A., Ibrahim, H.T. (2019). Rainfall intensity probability for design of drainage system in Basrah City, south of Iraq. ARPN Journal of Engineering and Applied Sciences, 14(5): 954-961.

[15] Bhagat, R.M., Borse, A.S., Mukherjee, O., et al. (2025). A comprehensive review of sewer network design by SewerGEMS. In Innovations in Technologies: Pioneering Sustainable Infrastructure for a Resilient Future. IC-RTIDSM 2023. Sustainable Civil Infrastructures, Springer, Cham, pp. 441-455. https://doi.org/10.1007/978-3-031-82133-2_33

[16] Noori, A.R., Singh, S.K. (2021). Assessment and modeling of sewer network development utilizing Arc GIS and SewerGEMS in Kabul city of Afghanistan. Journal of Engineering Research, Special Issue, pp. 22-31.

[17] Rajpurohit, P.H., Raol, J.D. (2016). Design of sanitary sewer network for Gandhinagar City using SewerGEMS V8i software. International Journal of Advance Research in Engineering, Science & Technology, 3(2): 164-170.

[18] Al-Ani, F.H., Al-Bayati, S.A.F., Jasim, A.N. (2018). Evaluation of combined sewage trunk system (East Bank Trunk) in Al-Rusafa area, Baghdad City, Iraq. MATEC Web of Conferences, 162: 05014. https://doi.org/10.1051/matecconf/201816205014

[19] Abd-Elhamid, H.F., Zeleňáková, M., Vranayová, Z., Fathy, I. (2020). Evaluating the impact of urban growth on the design of storm water drainage systems. Water, 12(6): 1572. https://doi.org/10.3390/w12061572

[20] Mohammed, M.H., Zwain, H.M., Hassan, W.H. (2021). Modeling the impacts of climate change and flooding on sanitary sewage system using SWMM simulation: A case study. Results in Engineering, 12: 100307. https://doi.org/10.1016/j.rineng.2021.100307

[21] Mohammed, M.H., Hassan, W.H., Zwain, H.M. (2023). Overflow sanitary sewer systems during the rainy season: A case study in Al-Samawah city. AIP Conference Proceedings, 2631: 020012. https://doi.org/10.1063/5.0133043

[22] Hussein, S.N., Nile, B.K., Zwain, H.M. (2023). Evaluation and determination of flood areas using SWMM and the creation of a mathematical model using dimensional analysis: A case study of the city of Najaf. AIP Conference Proceedings, 2631: 020023. https://doi.org/10.1063/5.0131176

[23] Aziz, M.K., Shamkhi, M.S. (2022). Optimal design of drainage networks in Kut city using genetic algorithms and spectral analysis of the river to know the impact of network pollutants. Wasit Journal of Engineering Sciences, 10(2): 119-128. https://search.emarefa.net/detail/BIM-1572449.

[24] Bentley Systems Inc. (2023). OpenFlows Sewer. Computer Software. Bentley Systems, Exton, Pennsylvania, USA. https://www.bentley.com/software/openflows-sewergems/.

[25] Chow, V.T., Maidment, D.R., Mays, L.W. (1988). Applied Hydrology. McGraw-Hill, New York.

[26] Al-Baldawi, T.H.K., Al-Zuabidi, Z.Z.A. (2016). Statistical analysis of extreme rainfall data in Baghdad City. Iraqi Journal of Science, 57(1C): 713-718.

[27] Hassan, A.S., Zeki, K.N., Salih, N.S. (2018). Determination the quantity of extreme rainfall and calculation of the climatology mean for Baghdad City. Iraqi Journal of Science, 59(1B): 447-455. https://doi.org/10.24996/ijs.2018.59.1B.25

[28] Oleiwi, A.S., Abed, B.Sh., Hasan, B.F. (2023). Rainfall prediction and runoff modelling under climate change scenarios for Tigris River from Mosul to Baghdad cities. International Journal of Design & Nature and Ecodynamics, 18(3): 537-546. https://doi.org/10.18280/ijdne.180305

[29] Al-Shammari, A.A., Al-Dabbas, M.A. (2015). Extraction drainage network for Lesser Zab River Basin from DEM using Model Builder in GIS. Iraqi Journal of Science, 56(4A): 2915-2926.

[30] Prihantarto, W.J., Purnamasari, E., Ismail, M., Putri, S.K., Putri, E.W., Yudinugroho, M. (2025). Integration of remote sensing and GIS for mapping of land cover change due to flash floods in Simangulampe, North Sumatra, Indonesia. Iraqi Journal of Science, 66(12): 5831-5840. https://doi.org/10.24996/ijs.2025.66.12.44

[31] Al-Juhaishi, M.R., Oleiwi, A.Sh., Abed, B.Sh. (2024). Modeling surface runoff in Al-Mohammadi Valley: Influence of climate and soil parameters. International Journal of Design & Nature and Ecodynamics, 19(3): 1043-1049. https://doi.org/10.18280/ijdne.190333

[32] Al-Saadi, T.S., Al-Thamiry, H.A. (2021). Evaluation and development of Shatt Al-Diwaniya and the diversion canal of Shatt Al-Diwaniya. Journal of Physics: Conference Series, 1973(1): 012180. https://doi.org/10.1088/1742-6596/1973/1/012180

[33] Al-Ani, R.R.A., Al-Obaidi, B.H.K. (2019). Hydraulic analysis and performance evaluation of combined trunk sewers: A case study of Baghdad City. Journal of Engineering, 25(3): 1-10. https://doi.org/10.31026/j.eng.2019.03.01