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Hydraulic Design Audit and Preliminary Energy Assessment of an Urban Stormwater Pumping Station in Wasit, Iraq

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ABSTRACT

Stormwater pumping stations are used to supplement urban drainage in areas where gravity flow is not adequate, but little local data may make it difficult to reconcile hydrologic loading, wet-well storage, hydraulic losses, and pump operating requirements. An energy assessment and a preliminary hydraulic design audit were performed for a proposed station located in the Al-Zahraa neighborhood of the Governorate of Wasit in Iraq based on the available information on the project and the steady-state calculations. Assumptions used were a catchment area of 915,000 m^2, a runoff coefficient of 0.45, a baseline rainfall intensity taken from the literature of 10 mm/h, and a 1,000 m force main of 0.70 m diameter. The estimated peak inflow was 1.1438 m^3/s, and the assumed ten-minute balancing requirement was 686.25 m^3. In this case, the operating-level range of the adopted dimensions of the wet well (9.6 m × 7.2 m) is estimated to be about 9.93 m, which is not a complete civil design. Inlet elevation, pump submergence, free storage, and freeboard checks are still to be done to confirm the practical feasibility. The baseline calculations showed that the total dynamic head (TDH) was around 23.7 m, of which 70.5% was dynamic loss. The sensitivity runs showed the effect of the force main diameter on friction loss. The estimated peak inflow would be balanced with three pumps at 1372.5 m^3/h, with no net storage recovery if the peak inflow continued. At a preliminary estimated system head of 41.2 m, where it is assumed that inflow has ceased, and a duration of seven minutes, the rate of drawdown will be about 1,960.7 m^3/h per pump. These calculated requirements are subject to the assumptions and inputs used, and have not been tested in use. Prior to design confirmation, local rainfall and topographic information, certified pump curves, parallel-pump evaluation, dynamic, transient pressure and suction-condition checks are necessary. The findings reveal design inconsistencies for flood risk, operational reliability, and hydraulic energy requirements, but do not prove station performance.

1. INTRODUCTION

Rising urban flooding is a problem of infrastructure performance and not just rainfall. An increase in impervious surfaces leads to increased runoff, decreased infiltration, and stormwater channelizing to finite conveyance and storage networks. Reliable free-surface drainage may be impeded in low-relief districts by local depressions or high receiving-water levels or by impractically deep gravity sewers. A stormwater lifting station, therefore, becomes a key link in the flood protection system and its effectiveness is not solely dependent on the size of the pumps installed, but on how well the hydrologic loading, wet-well storage, force-main resistance, pump duty, controls, and protective devices work together during the critical storm event [1].

These variables are strongly coupled. The higher the runoff, the more the wet well will have to be filled, power will be consumed, pipelines will need more discharge from the pump, and friction loss will increase. A wider pipe will cause a decrease in resistance and result in an increase in capital cost, while a larger storage buffer will cause inflow to the storage tank but will not be able to cover up for an insufficient pumping capacity indefinitely; and the use of aggressive drawdown targets will cause the station to operate at high-flow operating points where the head of the system will be much higher. The final pump duty is therefore dictated by the full system curve and operating logic, and depends on the intake geometry, screening, valve arrangement, air management, net positive suction head (NPSH) margin, and transient protection [2, 3].

The need for integrated design is strengthened by urbanization and non-stationary rainfall. Recent research suggests that the effects of precipitation extremes can lead to higher drainage pressure and larger capacity thresholds, while the value of storage, pumping, pipe enlargement, and low-impact strategies is site-specific [4, 5]. Research studies on flood resilience also reveal other factors that are important besides the rainfall exceedance, such as blockage, failure of components, recovery time, and service continuity [6, 7]. Green and grey interventions can be used to enhance

resilience, but in different ways, and not as capacity additions and substitutes per se on the hydraulic system [8, 9].

Pumped drainage is controlled by energy and reliability, related to the head the pump must cover [10]. Total dynamic head (TDH) is the sum of static head + major losses + minor losses. In long force mains, the Darcy-Weisbach loss is quite dependent on velocity and diameter, and a host of other items – such as valves, tees, elbows, branch pipes, screens, and discharge fittings – can significantly increase resistance. These losses represent the system curve and thus the actual operating point of the system when it crosses a pump curve. If the system curve is not complete or operating flows are changing, a selected catalog pump that is based on nominal flow and head will work off of the efficiency region. Recurring energy demand and bigger installed power as a result of avoidable hydraulic resistance [11, 12].

Despite the fact that the hydraulic sizing of stormwater pumping stations is a well-known procedure, the size of the pumping station may not be consistent with the amount of inflow, the volume of wet-well storage, the duty of the pump, and the operating criteria. The unanswered question in this case is whether these design elements provide a coherent foundation for station operation under the assumptions given [13]. This study is thus a transparent and auditable check of the existing engineering design of Al-Zahraa neighborhood in Wasit, Iraq, without proposing a new hydraulic method. Its role is to make sure that design calculations are consistent, to find inconsistencies between the inflow balancing duty and the seven-minute drawdown requirement, and to look at how sensitive the hydraulic losses are to certain design parameters. The following issues have practical implications for urban flood risk, operational reliability and energy use: if the net pumping capacity is insufficient, there may be limits on recovery for storage during rainfall; the operating criteria are inconsistent, which may influence the scheduling of operations; and excessive hydraulic losses may increase pumping energy requirements. These implications are noted as design concerns, but have not yet been confirmed for flood protection, operating reliability, and energy consumption.

The study conducts an audit of the proposed Al-Zahraa stormwater pumping station located in Wasit, Iraq, with a catchment area of 915,000 m², a runoff coefficient of 0.45, and baseline rainfall intensity of 10 mm/h. The goals are: Recalculate peak inflow; evaluate the balancing volume assumed for the ten-minute drawdown; Balance hydraulic losses and TDH; and separate peak-inflow balancing from the seven-minute drawdown. The contribution is a check of consistency between these design elements and will yield conditional estimates and identify unfulfilled requirements. Future work is still to be done—certified pump verification, dynamic modelling, transient analysis, and NPSH checks.

2. LITERATURE REVIEW

2.1 Urban flooding, drainage pressure, and resilience

The overall philosophy of urban drainage is shifting away from a purely fail-safe approach and towards resilience and safe-to-fail concepts [14]. Traditional design typically rates infrastructure based on a given event and sizes infrastructure to prevent hydraulic failure during that event. Resilience research, on the other hand, focuses on the size and the length of the loss of services, the impact of service failure, and the ability to bounce back [15, 16]. In addition to hydraulic overloading, Mugume et al. [17] demonstrated that structural failures (blockage and pipe malfunction) can be as relevant as hydraulic overloading. For an urban stormwater drainage system, the purpose of the system is to carry stormwater during high-intensity rainfall and not allow flooding when exceeded. To minimize the danger of drainage failure and the intensity of floods, Rohith and Sudheer [18] came up with a safe-fail design framework. The Storm Water Management Model (SWMM) can be used for drainage analysis to simulate the rainwater–runoff process and flow through drainage networks [19]. Another use of simulation in water infrastructure was by Jasim [20], who modelled a wastewater treatment plant. To minimize urban flood risk, Sun et al. [21] integrated a stormwater storage approach that utilizes forecasts during storms and real-time control.

Hydraulic analysis can be used in addition to network modeling to describe flow and pressure at the component level. Swamee and Jain [22] developed explicit equations based on the Colebrook–White relationship to simplify pipe-flow calculations. Jasim [23] designed an experimental apparatus to show how to locate the CoP on a submerged surface. Wang et al. [24] proved that resilience metrics are dependent on the benchmark used to compare intervention schemes. The study by Haghbin and Mahjouri [9] also extended the definition of resilience assessment, using both technical and socio-ecological indicators. The studies provide a rationale for a design process, where redundancy, recoverability, and failure consequences are taken into account, along with hydraulic capacity. This is strengthened by climate and urbanization uncertainty. Under the CMIP6 projections, Wang et al. [25] simulated that drainage pressure would increase in many cities in the future, and they determined that local adaptation and climate mitigation would significantly influence the future drainage pressure. Li et al. [13] integrated climate projections with the Storm Water Management Model (SWMM) to determine critical decades and spatial hot spots where drainage pressure might exceed critical thresholds. While these studies have much more comprehensive data than the case in question, the message remains relevant: a deterministic design intensity should not be seen as a full description of future risk. At a minimum, sensitivity analysis should be performed in data-sparse situations along with a baseline calculation.

2.2 Grey infrastructure, pumping capacity, and storage

Grey drainage systems will continue to be essential in areas where gravity drainage is not possible [26]. However, pumping capacity is not directly linked to the reduction in flood risk [27]. A non-linear relationship between pumping capacity and urban flood susceptibility was quantified by Wu et al. [28], showing that the marginal value of the increased capacity may change once threshold values are surpassed. This finding contradicts the misguided policy of providing pumps that are oversized without undertaking a system-level analysis; capital costs, motor demands, and the severity of transients could increase while providing no corresponding flood risk benefit. Storage can alleviate the need for pumping in a momentary fashion by buffering inflow, but this value is dependent upon the operating logic. Sun et al. [21] demonstrated that performance of limited storage can be enhanced by releasing water in accordance with the anticipated storm hydrograph in real-time. The overall design

principle is that the volume of the wet-well, the level of the pump start/stop, and control rules are inextricably linked.

2.3 Runoff estimation under limited data

The Rational Method is still popular for small urban catchments because it uses a runoff coefficient, rainfall intensity, and catchment area to calculate peak flow. It is simple and appealing where detailed rainfall-runoff calibration is not available, but it is an approximation of the peak rate only. Wang and Wang [26] moved beyond the original Rational Method and showed how the concept can still be useful when applied to on-site runoff controls, while highlighting the importance of understanding the assumptions and limitations. Dynamic tools, on the other hand, allow United States Environmental Protection Agency Storm Water Management Model (EPA SWMM) to route time-varying runoff through pipes, storage nodes, pumps, and control devices, and can include surcharge, ponding, reverse flow, and dynamic pump rules [19]. The available information in the case study supports a baseline using the Rational Method and does not support a calibration using a SWMM model.

2.4 Force-main hydraulics and pump operating point

The Darcy–Weisbach equation is used to represent pipe friction, and the friction factor can be obtained from the Colebrook equation, the Moody diagram, or the validated explicit equations such as Swamee–Jain [6, 16, 22]. If the flow is turbulent, the roughness becomes more significant, and if the system curve changes, it could be due to deposits, corrosion, or aging. K coefficients are used to represent the local losses and may be significant at high velocities or if a station has several valves and fittings. System curve selection should be based on the point where the system curve intersects a manufacturer-certified pump curve, and not on the number of nominal heads or flows alone. The importance of pump application, intake uniformity, piping arrangement, operating regions, and margins for NPSH is addressed in current Hydraulic Institute [10-12] guidance because hydraulic instability and air entrainment can negatively impact efficiency and reliability. A wet-well intake should ensure an adequate degree of uniformity and swirl-free approach flow, and the pump should work within an acceptable operating range around its best efficiency point (BEP).

2.5 Infrastructure adaptation and design trade-offs

Portfolios are often a part of resilient stormwater design, not as a single solution. Under uncertain future conditions, Dong et al. [8] compared green and grey interventions and demonstrated that there are different ways to enhance resilience. Azari and Tabesh [2] combined sustainability indices with optimization of stormwater networks and LID measures. Safe-fail design criteria, which take account of the consequences of flooding, have been suggested by Rohith and Sudheer [18], which do not just rely on a common return period. These studies indicate that a pumping station is best considered part of a catchment-wide adaptation solution. However, many municipalities have to fix immediate hydraulic restrictions before they can effect the more complex hydraulic solutions necessary for the projects in question. In the above context, an auditable mechanical-hydraulic design is still useful as long as the design assumptions can be seen, and as later stages of the design process involve dynamic simulation, green-infrastructure options, and climate-adjusted rainfall. The present case is located on that fringe: it establishes a viable baseline and outlines the analysis needed to move from deterministic design to resilient asset planning.

3. METHODOLOGY

3.1 Study area and design data

The case study is Al-Zahraa neighbourhood, Wasit Governorate, Iraq. According to the engineering record, Al-Zahraa is the application area, and the contributing drainage area is 915,000 m². The source data does not contain a georeferenced catchment polygon, survey coordinates, rainfall-gauge identifier, or a full Digital Elevation Model. Accordingly, this manuscript does not provide a georeferenced catchment boundary or survey-grade coordinates for the study area. Acquiring a GIS catchment boundary and benchmarked elevation data is identified as a prerequisite for subsequent detailed design and pre-construction verification.

The rainfall intensity (I = 10 mm/h) and runoff coefficient (C = 0.45) are assumed in the baseline design. They were cited as typical urban surface and regional design rainfall values. The precipitation intensity is treated as an adopted design input, as the rainfall time series itself is not part of the record and neither is the selected return period. The difference between these two is significant: the hydraulic values can be checked from the values registered in the report, but the frequency of the design storm can never be checked from the available project file independently.

The literature review was used to establish a baseline rainfall intensity of 10 mm/h. This value has not been validated with a locally established intensity–duration–frequency (IDF) relationship and is not assigned a local return period. The estimated peak flow is thus dependent on the intensity used. At a fixed catchment area and runoff coefficient, a 20% increase in rainfall intensity leads to a 20% increase in the estimated peak flow, ranging from about 0.915 to 1.373 m³/s.

A 1,000 m long pipeline having an internal diameter of 0.70 m and a supposed absolute roughness of 0.26 mm is used as a force main. The source design philosophy suggests one more duty-pump branch, and the baseline total inflow is through three duty-pump branches. The reported diameter of the branches is 0.30 m, and the branch length is 8 m. The static lift is 7.0 m.

Existing survey information on the engineering design of the Al-Zahraa neighbourhood stormwater pumping station was used to reconstruct the baseline hydraulic parameters. To ensure traceability, parameters were grouped into three provenance categories: project-provided, author-assumed, and study-calculated. The peak inflow was estimated by the Rational Method; the effective storage volume and wet-well depth were obtained from the adopted storage duration and wet-well dimensions. Project-specific verification was not available for parameters, and these were assumed as modelling parameters. The issuing organisation, the document date and revision number, and the formal availability date of the original engineering record could not be determined independently and are therefore considered to be a limitation of the available documentation. The project-provided, author-assumed, and quantities calculated in this study are indicated in Table 1. The values adopted for each parameter and their associated units are provided in the table, as well as the provenance category, supporting source and, where relevant,

the reference for the assumption or calculation.

3.2 Hydrologic loading by the Rational Method

The Rational Method was used to estimate peak stormwater runoff for the level of data available. In SI units, the area unit is m², and the intensity is expressed in mm/h; the conversion is as follows:

$$Q = \frac{CIA}{3.6 \times 10^6} \quad (1)$$

The peak runoff rate, Q (m³/s), is proportional to the dimensionless runoff coefficient, C, the rain intensity (I, mm/h), and the catchment area (A, m²). Substitution of C = 0.45, I = 10 mm/h, and A = 915,000 m² yields Q = 1.14375 m³/s. The calculation was independently recalculated, taking into account an ambiguity in the notation of the source worksheet, but without altering the outcome. The Rational Method is only applied as a minimum (baseline) peak-flow estimator and is not used to generate the event hydrograph or time-varying storage function [26].

Table 1. Baseline case-study inputs reconstructed from the engineering design

Parameter	Symbol	Adopted Value	Role in Analysis	Provenance Category
Contributing urban area	A	915,000 m² (91.5 ha)	Rational Method runoff	Project-provided
Runoff coefficient	C	0.45	Imperviousness/runoff response	Author-assumed
Design rainfall intensity	I	10 mm/h	Baseline hydrologic load	Project-provided
Peak baseline inflow	Q_{in}	1.14375 m³/s	Station inflow-balancing case	Study-calculated
Effective storage duration	t	10 min	Wet-well balancing volume	Author-assumed
Wet-well effective volume	V_e	686.25 m³	Operational storage	Study-calculated
Wet-well plan dimensions	—	9.6 m × 7.2 m	Storage geometry	Project-provided
Calculated effective depth	—	≈9.93 m	$V_e/(9.6 \times 7.2)$	Study-calculated
Force-main length	L	1,000 m	Main friction loss	Project-provided
Force-main diameter	D	0.70 m	Velocity and friction	Project-provided
Absolute roughness	ε	0.26 mm	Friction factor	Author-assumed
Kinematic viscosity	ν	1.004×10^{-6} m²/s	Reynolds number	Author-assumed
Reynolds number	Re	—	Friction calculation	Study-calculated
Branch diameter	D_b	0.30 m	Per-pump branch loss	Project-provided
Branch length	L_b	8 m	Per-pump branch loss	Project-provided
Static head	H_s	7.0 m	TDH	Project-provided
Duty pumps	n	3	Baseline flow distribution	Project-provided
Standby philosophy	—	≥1 standby unit	Operational redundancy	Project-provided

A post hoc robustness analysis, in the form of a hydrologic stress test, was then applied. Note that the study is not based on a rainfall projection with known validity for the site; thus, it is a test scenario and not a prediction. The rainfall intensity was modified from 6 mm/h to 30 mm/h and C from 0.35 to 0.65 to show the linear relationship between the peak flow in the Rational Method and the hydrologic parameters used. The analysis is performed to determine which design is sensitive rather than to determine return periods.

The initial balancing assumption was taken to be a ten-minute period, which implies an estimated inflow volume of 686.25 m³. This volume is an inflow that is added over the assumed time period with no credit to simultaneous pumping and is therefore not a validated flood-storage capacity. Verification of its suitability must be made through an inflow hydrograph, characteristics of pump operation, and start–stop control levels. For the dimensions adopted, this volume would represent a calculated active water-depth range of ~9.93 m. This range should be considered between the minimum and maximum operating water levels, and is not necessarily equal to the total structural depth. The minimum operating level should meet the specified pump submergence requirements, and provision for dead storage below this level. The maximum operating level should be verified against the invert of the inlet and the permissible upstream surcharge; and there shall be sufficient freeboard above the maximum design water level. Some of these geometric and operational checks are outstanding.

Rainfall intensity and runoff coefficient were selected as 10 mm/h and 0.45, respectively, and the Rational Method was used. The available documentation does not, however, shed light on the duration of the rainfall, the return period, or the time of concentration. The runoff coefficient is the assumed overall response of the urban catchment, including both impervious and pervious surfaces. Hence, these values are assumed to be baseline inputs, and the computed peak discharge of 1.14375 m³/s is assumed to be a baseline scenario and not a local design discharge.

3.3 Wet-well effective storage and operational levels

The design is based on 10 minutes of effective storage at the base flow peak inflow. Balancing volume is then:

$$V_e = Q\,t \quad (2)$$

Using Q = 1.14375 m³/s and t = 600 s gives V_e = 686.25 m³. The corresponding effective depth is about 9.93 m, with a plan area of 9.6 m × 7.2 m. The volume is the active volume between the pump-control levels, not the total depth of excavation of the structure. Freeboard, pump submergence, sump geometry, and dead storage should be detailed-designed.

The continuity equation ($dV/dt = Q_{in} - Q_{out}$) was used for the evaluation of wet-well storage. The 10-minute allowance (686.25 m³) is used for inflow accumulation when pumping is not done and is used as a first allowance for short-duration start-up or interruption of pumping. At the time of the presumed peak inflow, all three pumps balance the net storage change is close to zero. This volume, therefore, is regarded here as a preliminary storage allowance, not as an actual operating storage requirement.

The level control philosophy is based on four main states: all pumps off at the minimum safe submergence; lead pump start; lag/assist pump start when inflow is greater than the lead pump; and high level alarm – maximum pumping available. To distribute run hours, duty alternation must be performed. Other design features of the source focus on reducing the

number of starts, keeping the pump cool, avoiding the formation of vortices, and retaining emergency storage. These are the same as the general requirements for stable pump-intake flow and controlled operating regions [12, 13].

3.4 Flow velocity, Reynolds number, and friction factor

The mean flow velocity was calculated from continuity:

$$v = \frac{4Q}{\pi D^2} \tag{3}$$

For Q = 1.14375 m³/s and D = 0.70 m, v = 2.972 m/s. The Reynolds number is:

$$Re = \frac{vD}{\nu} \tag{4}$$

Which gives $Re \approx 2.07 \times 10^6$ for $\nu = 1.004 \times 10^{-6}$ m²/s. The flow is consequently totally turbulent and can be treated as such in the design. Relative roughness is $\varepsilon/D \approx 3.71 \times 10^{-4}$. From the Moody diagram, the Darcy friction factor (f) obtained for the project is 0.01560. The value of f obtained from an independent explicit-explicit correlation audit is quite similar; the same explicit relation is used in the sensitivity analysis, but used at various diameters for the baseline [16, 22].

3.5 Major and branch losses

Major loss in each straight pipe segment was determined with the Darcy–Weisbach equation:

$$h_f = f \frac{L}{D} \frac{v^2}{2g} \tag{5}$$

Here, the Darcy friction factor f is dimensionless, L is the length of the pipe segment (m), D is the inside diameter of the pipe segment (m), v is the mean velocity of the flow in the pipe segment (m/s), and g is the acceleration of gravity (9.81 m/s²).

The reported baseline values for the 1,000 m main yield a loss of 10.041 m for the main line. In order to present a meaningful comparison, the pump discharge branch was evaluated separately since it has a 0.30 m diameter, which allows for a much higher local velocity. The Darcy–Weisbach equation predicted the head loss to be 0.756 m in the 8 m branch pipe, with a Darcy friction factor of f = 0.01560 and the volumetric flow rate through the branch is Q is about 0.38125 m³/s, and the branch friction loss for 8 m is 0.756 m. When the branch is also considered a fitting, it may be omitted in the arithmetic, which is avoided by separating the main and branch losses. The discharge per pump is 0.38125 m³/s. When the diameter of the branch is 0.30 m, the mean velocity of the branch is given by 5.39 m/s from Eq. (3).

3.6 Minor losses and mechanical components

Minor losses were calculated by the conventional coefficient method:

$$h_m = \sum K \frac{v^2}{2g} \tag{6}$$

where, K is the dimensionless local head-loss coefficient for each pipe fitting or component (e.g. bend or valve) and the sum of the $\sum K$ is the sum of all of them that are included in the analysis.

The source design provides a total combined fitting-loss contribution of 5.937 m, which comprises direction changes and valve/fitting effects. The mechanical configuration further includes check valves to prevent backflow, isolation valves (gate valves or penstocks), tees and elbows to distribute the flow, air release valves at the highest points, dismantling joints for maintainability, and pipe supports/thrust restraint at direction changes. The components are included in the methodology because local losses are related to maintenance access and transient response: A valve arrangement that is hydraulically efficient but cannot be isolated safely is not a complete station design.

3.7 Total dynamic head and pump duty

The reconciled TDH is calculated as the static head plus all identified dynamic components:

$$TDH = H_s + h_{f,\text{main}} + h_{f,\text{branch}} + h_m \tag{7}$$

where, TDH is the total dynamic head required from the pump (m), H_s is the static head (m), h_f and $h_{f,\,branch}$ are the frictional head losses along the main and branch pipes, respectively (m), and h_m is the total local head loss from fittings, valves, and other components along the flow path (m).

For the case study, TDH = 7.000 + 10.041 + 0.756 + 5.937 = 23.734 m. This is the value that is utilized in all the results. It is not the final result for the Al-Zahraa case study, but is a higher value (29.825 m) that appears in another worked methodology example and relates to a different set of pipe length and design inputs. If the inflow is the same and the duty pump is divided equally, then the nominal per-pump flow is equal to:

$$Q_p = (Q \times 3600) / n = 1{,}372.5 \text{ m}^3/\text{h} \tag{8}$$

The corresponding hydraulic power at the reconciled TDH is approximately 88.8 kW per duty pump before efficiency losses. This is hydraulic power only; no electrical motor rating is inferred because the exact pump efficiency, motor efficiency, service factor, and certified manufacturer curve are not documented. Final selection must verify the system-curve intersection, acceptable operating region around BEP, NPSH margin, solids-passage requirement, and motor rating using certified data [11].

The static head was determined by considering the difference in elevation between the receiving-water level and the operating water level of the wet-well. The static head is not necessarily constant due to the change in the level of the water in the wet-well during pump operation. Accordingly, the upper and lower operating levels should be considered separately when developing the system curves. The adopted 7 m static head should be coupled to the operating water level, and the resulting static head and friction losses should be considered when assessing pump performance.

3.8 Screening, intake, and wet-well mechanical design

Primary trash screening at the inlet of the wet-well is included in the station methodology since stormwater may carry plastics, vegetation, sediment, and other materials that can cause impeller blockage or destabilize rotating equipment. Coarse and, as needed, fine screening should be sized based on the pump solids-passage capability but also provide acceptable approach velocity and screen head loss. The

geometry of the wet wells should reduce dead zones, sediment settling, swirl, and air entrainment. The submersible pumps are mounted on guide rails and self-coupling discharge bases so that the units can be moved out of the wet well without the need for personnel entering the wet well. To withstand hydraulic and start-up loads, riser pipes are supported. For each pump line, there is a non-return valve, isolation valve, and a set of fittings that are connected to the common force main. Hydraulic high points have air-release valves. These features ensure that the hydraulic design of a pump piping system can be maintained and are consistent with current guidance regarding the design of pump piping and intake systems to ensure stable hydraulic performance instead of simply meeting available space requirements [12].

3.9 Control, automation, and safety systems

The water level can be monitored using ultrasonic measuring equipment or by using multiple float switches that send a signal to the PLC/control panel. The lead/lag sequence is programmed with inlet isolation conditions to prevent the pumps from running against a closed or improperly programmed inlet isolation system. High-level alarms provide an independent state when a reaction is happening. In addition to the above, the source project includes manual/actuated penstocks, access ladders, service platforms, guardrails, and lifting equipment for pump maintenance. The auxiliary systems are considered part of hydraulic reliability because an unsafe or non-maintainable station is unable to provide the level of service designed for its life cycle.

3.10 Water-hammer and cavitation provisions

Water-hammer and cavitation provisions are provided for in accordance with the following: Qualitative protection against fast pressure changes is provided by the inclusion of check valves, controlled isolation, supports/thrust restraint and air-release devices in the baseline design. These measures are needed but not sufficient to assess transient pressure. No transient pressure envelope is claimed in the project as it lacks a numerical water-hammer model, pump inertia data, valve closure laws, and surge-vessel analysis. The detailed design should include a transient simulation of the system for the following events: pump trip, power failure, start-up, and valve manoeuvres, with the pipe wave speed and equipment characteristics verified [4]. Likewise, a formal NPSH available versus required check is needed for cavitation avoidance at the pump duty and minimum wet-well level chosen.

3.11 Post hoc robustness and consistency audit

Two analyses were added for traceability. First, the diameter of the force main (diameter of the main pipe) varied between 0.55 and 1.00 m while the other factors (fluid properties, branch loss, minor losses, static head, and roughness) were held constant. For all scenarios, the explicit correlation of Swamee–Jain was consistently used to estimate f. Not a lifecycle cost optimization; it is just isolating the hydraulic penalty of diameter selection. Secondly, the drawdown statement for the wet-well was audited against the pump capacity. The source design suggests that 686.25 m^3 be removed in 7 minutes, or 1.6339 m^3/s if inflow has stopped. If the pumps were all the same size, they would all need about 1,960.7 m^3/h. In contrast, the reported 1,372.5 m^3/h per pump results in a total of 1.14375 m^3/s, which is the same as the baseline peak inflow. If a net drawdown of 686.25 m^3 is required within 7 minutes while the peak inflow (1.14375 m^3/s) continues, the required combined pump discharge is $1.14375 + 686.25/420 \approx 2.7777$ m^3/s. The cases were, therefore, split instead of being combined into a vague design stream. The design sequence was then reduced to the decision sequence displayed in Figure 1, after the hydrologic, wet-well, conveyance, loss, pump-duty, protection, and audit calculations were completed. The flowchart illustrates the preliminary hydraulic design and the validation gate that should be met before the final procurement and construction of equipment. Figure 1 shows the schematic layout of the pumping station and discharge pipeline, including the wet well, pump branches, common discharge header, pipeline routing, and flow direction, and a hydraulic design methodology is shown, with the design calculations being performed sequentially, followed by an evaluation of the system and the requirement to validate the design.

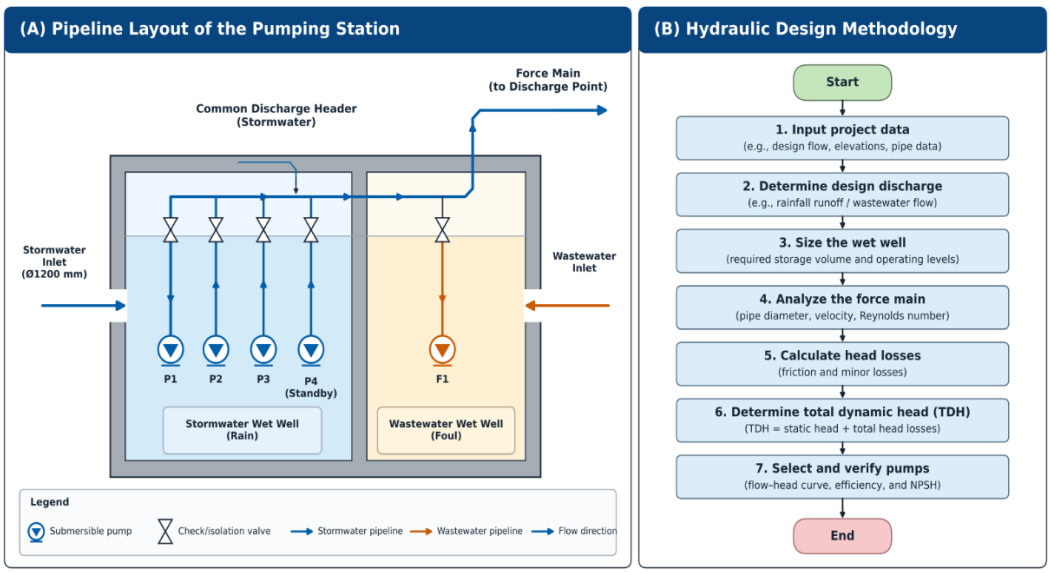

Figure 1. (A) Schematic layout of the pumping station, including stormwater wet wells, wastewater wet wells, pump discharge branches, common stormwater discharge header and flow direction; and (B) hydraulic design methodology and validation procedure

Figures 2 and 3 show the physical layout and characteristics of the pumping station studied. The two structurally separate wet wells collect stormwater and wastewater, respectively, at the station. The cross-sectional arrangement, as shown in Figure 2, indicates the location of the pumping units, vertical pipes, and other mechanical components. The stormwater pumping section has four pump installations and an additional pumping system for wastewater collection. This physical arrangement establishes the main hydraulic and mechanical elements of the station and specifies the basis for the hydraulic analysis of the pumping system arrangement.

Operating Water Levels and Design Elevations for the stormwater wet well are shown in Figure 3. The project engineering drawings indicate that the first pump will start at +10.00 m, the second pump at +11.00 m, and the third pump at +12.00 m. All-pumps-on operating level is set at +14.50 m, while the high-water level (HWL) is set at +15.50 m. The lower water level (LWL) where the pumps are turned off is +9.00 m, and the sump floor elevation is +7.20 m. These elevations will be used for the staged pump-operation arrangement and for defining the vertical range with respect to wet-well storage and operating-level evaluation.

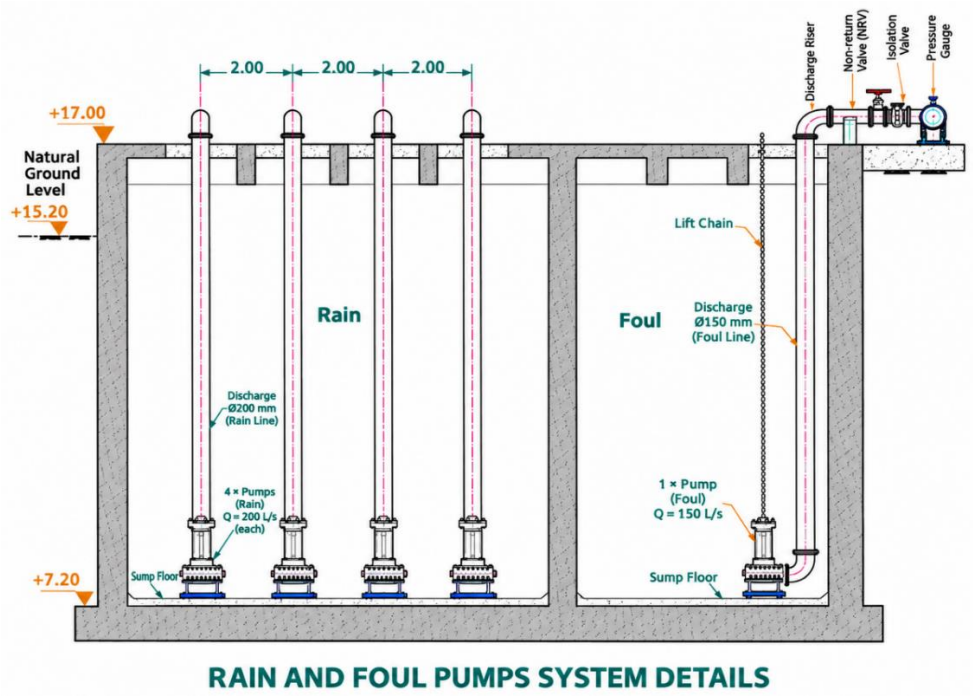

Figure 2. Cross section of the pumping station with structurally separated stormwater and wastewater wet wells, installed pumping units, vertical discharge pipelines and mechanical components

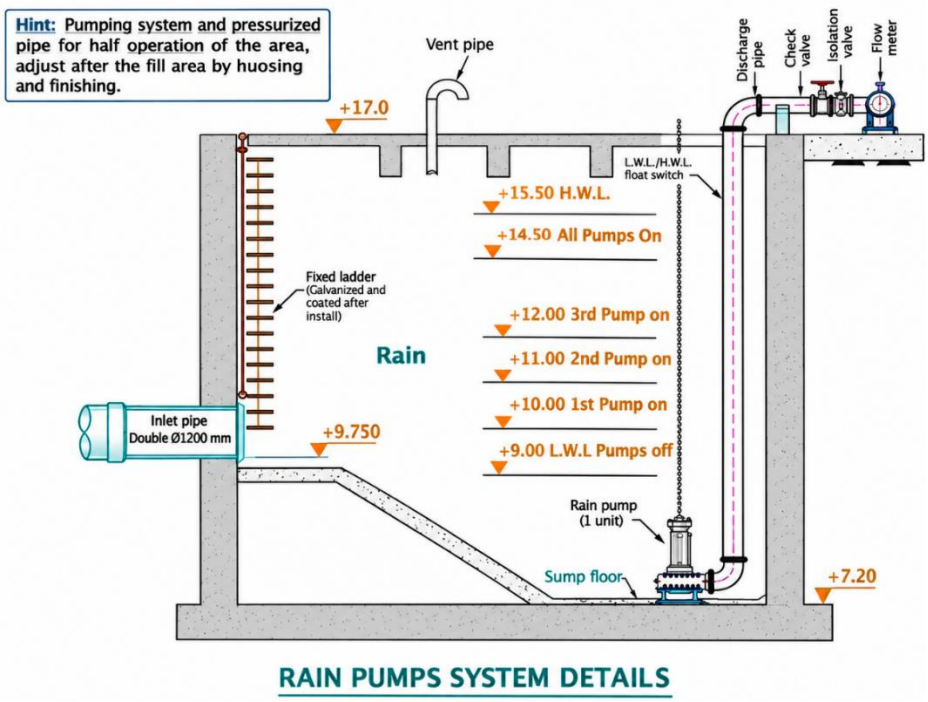

Figure 3. Operational water levels and elevations of the stormwater pumping wet well to illustrate the sequence of pump start water levels, minimum operating water level, high water level and sump floor elevation

The combination of Figures 2 and 3 gives complementary views of the station's physical layout and hydraulic operating conditions. Figure 2 shows the mechanical and structural layout of the pumping system, and Figure 3 shows the elevation of the water at which the stormwater pumping system activates and deactivates. The arrangements are project-specific and offer supporting information for the wet-well storage assessment and the following hydraulic assessment of the pumping and discharge system.

4. RESULTS

The adopted catchment area of 915,000 m², runoff coefficient of 0.45, and literature-based baseline rainfall intensity of 10 mm/h resulted in the estimated peak inflow of 1.1438 m^3/s using the Rational Method. This estimate is subject to the input that was used; it has not been checked against a local intensity – duration – frequency relationship or given a return period for Wasit. For a catchment area and a runoff coefficient of 416 m² and 0.50, respectively, an increase or decrease of 20% in the intensity of rainfall results in an estimated peak flow of 1.3725 m³/s, respectively, 0.9150 m³/s. These illustrative sensitivity scenarios show the proportional effect of the different rainfall intensities on the calculated peak flow and are not statistical limits. In addition, the lack of surveyed catchment boundaries and detailed elevation data adds uncertainty to the contributing area and runoff-routing assumptions. The calculated peak flow is thus not a validated design discharge to achieve a specified level of flood protection but a basis for auditing the design.

4.1 Hydrologic loading and wet-well storage

The peak rate verified by the Rational Method is 1.1438 m^3 per second, which is the same as 4,117.5 m^3 per hour if the peak rate were to be maintained. The adopted effective storage is 686.25 m³ for 10 minutes. Effective depth is found by dividing the volume by the wet-well plan area of 69.12 m², which gives around 9.93 m of effective depth. The calculation is internally consistent, but a final civil design should differentiate the elements of active depth, dead storage, submergence, freeboard, structural slab thickness, and emergency surcharge volume. The baseline force-main diameter is 0.70 m, carrying a total design discharge of approximately 1.1438 m³/s.

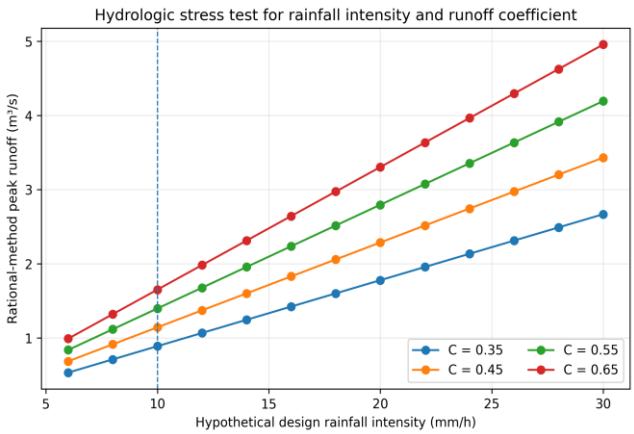

Figure 4. Post hoc hydrologic stress test
Note: Curves are hypothetical sensitivity cases derived from the Rational Method; they are not climate projections for Wasit.

The baseline hydrologic result is very sensitive to I and C because the Rational Method is linear with respect to both. Figure 4 shows the responses, but not the probabilities of the potential scenarios. At the baseline C = 0.45, doubling intensity from 10 to 20 mm/h doubles the peak flow from 1.1438 to 2.2875 m³/s. When the C is increased from 0.45 to 0.65 at I = 10 mm/h, the peak flow increases about 44%. This emphasises the importance of confirming the rainfall return period and surface characteristics prior to the final purchase of equipment.

The storage calculation also makes it clear on what time scale the design is made. The 686.25 m³ active volume equals one-sixth of the one-hour runoff volume in the baseline peak flow rate case, thus giving 10 minutes of buffering if the peak flow is maintained. This is a balancing volume, not independent flood storage. The calculated active depth of 9.93 m is large when compared to the plan size and should therefore be checked with the actual inlet invert, minimum submergence of the pump, dead storage, freeboard and the depth of structural excavation before the geometry of the wet-well is frozen.

4.2 Force-main hydraulic state

The baseline force-main diameter is 0.70 m, carrying a total design discharge of approximately 1.1438 m³/s. The Reynolds number is approximately 2.07×10^6, which is in the turbulent regime. The Moody friction factor is reported as 0.01560, and the friction head is calculated to be 10.041 m for the main line. The pump branch of the separate branch is 0.756 m. If you add the two straight-pipe losses together, you will get 10.797 m of head—before the local fittings are taken into account.

4.3 Loss decomposition and reconciled total dynamic head

Static head, elevation datum, wet-well operating levels

The static pumping head is dependent upon the instantaneous water-surface elevation in the wet well and the downstream hydraulic boundary. The project engineering drawings indicate that the operating water level of the stormwater wet wells is from an LWL of +9.00 m to a HWL of +15.50 m. The start dates for the first, second, and third pumps are set at elevations of +10.00 m, +11.00 m, and +12.00 m, respectively, and the all-pumps-on is set at +14.50 m. The sump-floor elevation is +7.20 m (see Figure 5). The static head is defined as the difference between the hydraulic boundary elevation at the downstream end of the system and the instantaneous water-surface elevation in the wet well, referenced to a common vertical datum ($H_s = Z_d - Z_w$), where Hs is the static pumping head (m), Z_d is the hydraulic boundary elevation at the downstream end of the system (m), and Z_w is the water-surface elevation in the wet well at a given instant (m). Therefore, the static head is dependent on the operating level of the wet well. The static head listed in Table 2 for the reported condition is the reference design condition and is intended to be coupled with the wet-well operating elevation and downstream hydraulic boundary.

The difference in static head across the operating range is significant for assessing pumping needs for different wet-well situations. The static head will rise with a drop in the water level in the wet-well and decline with an increase in the water level in the wet-well for a fixed downstream hydraulic boundary. The TDH and hydraulic performance of the pumping system should, therefore, be considered as a variation

of this.

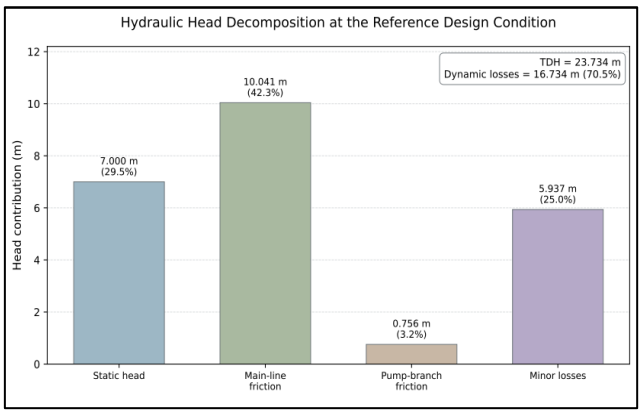

Figure 5. Decomposition of the reconciled 23.734 m total dynamic head (TDH)

Table 2. Reconciled hydraulic-head balance for the Al-Zahraa case

Hydraulic Head Component	Head (m)	Contribution to TDH (%)	Description
Static head	7.000	29.5	Reference static elevation difference
Main-line friction losses	10.041	42.3	Friction losses along the main discharge pipeline
Pump-branch friction losses	0.756	3.2	Friction losses in individual pump branches
Minor losses	5.937	25.0	Local losses associated with fittings and discharge arrangements
Total dynamic losses	**16.734**	**70.5**	Sum of main-line, branch, and minor losses
Total dynamic head (TDH)	**23.734**	**100.0**	Static head plus total dynamic losses

Note: The static head of 7.000 m represents the reference design condition. The corresponding wet-well operating elevation and downstream hydraulic boundary elevation must be specified relative to a common project vertical datum. TDH: Total dynamic head.

Table 3. Estimated operating points for different pump operating configurations

Operating Configuration	Number of Operating Pumps	Total Station Discharge, Q (m^3/s)	Operating Head, H (m)
Single-pump operation	1	0.618	13.65
Two-pump parallel operation	2	0.971	19.73
Three-pump parallel operation	3	1.161	24.23

Note: The operating points are only preliminary estimates derived from the reconstructed candidate pump characteristic and the assumed values of the system loss coefficients. They are NOT certified operating points. For confirmation, manufacturer-certified pump performance data and verification of the actual distribution of system losses must be provided.

The local loss due to the discharge arrangement is 5.937 m, and the static elevation difference is 7.000 m. The TDH, therefore, is reconciled to 23.734 m. The dynamic losses add up to 16.734 m (70.5% of head). The major contribution by a single source is main-line friction at 42.3% of TDH, followed by static lift at 29.5%, minor losses at 25.0%, and branch friction at 3.2% (Table 3 and Figure 5). Because of the dominance of dynamic resistance, the size of the pipe and the arrangement of fittings have a direct impact on the recurring energy demand. Table 2 shows the reconciled hydraulic-head balance for the Al-Zahraa case.

4.4 Pump-duty interpretation

Three duty pumps, each delivering 1,372.5 m^3/h, provide a combined flow of 4,117.5 m^3/h, or 1.14375 m^3/s. This is equal to the baseline Rational Method inflow and is thus an inflow-balancing duty. The theoretical hydraulic power is about 88.8 kW/duty pump and 266.3 kW for the three duty pumps at TDH = 23.734 m. The actual motor input will be higher and should be calculated, not assumed, based on certified pump and motor efficiencies.

The source project has a candidate performance chart in the hydraulic range required, but it does not specify the model of the pump, the certified test tolerances, the impeller diameter, the motor rating, or the NPSH datum. It should be noted that this manuscript does not specify a commercial pump but rather represents the base procurement duty specification of 1,372.5 m^3/h at 23.734 m. The final pump should be tested in accordance with its full system curve and allowable operating area.

A station system curve was reconstructed directly from the project losses that had been reconciled, using a 7.0 m static lift and a 16.734 m dynamic lift that was scaled approximately with Q^2 for the fixed geometry. As is evident in the figure, the baseline point 1.14375 m^3/s, 23.734 m is just one operating condition on a steeply rising resistance curve. The same resistance model yields a system head of ~41.2 m if the station needs to discharge 1.6339 m^3/s to restore the entire active storage in seven minutes following the cessation of inflow. Therefore, the seven-minute recovery requirement can not be met by increasing pump flow while maintaining the baseline TDH value of 23.734 m.

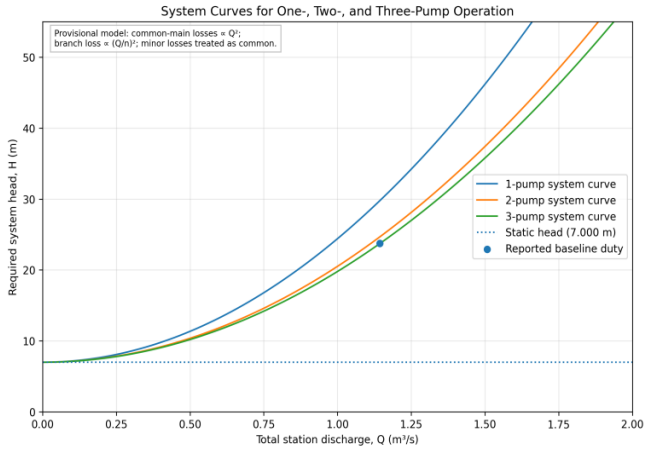

Figure 6. The system characteristic curves for one-, two-, and three-pump operating configurations, showing the relationship between total station discharge and required system head

The hydraulic performance of the pumping station was checked by comparing the pump characteristic curves to the respective pumping and system curves in the pumping operating cases of one, two, and three pumps. In order to be consistent, all pump and system characteristics were referred to the total station discharge, Q, not to the discharge of a single pump. If all the pumps are of the same size and are functioning in parallel, the sum of the individual pump discharges equals the total discharge; the developed head is approximately the same for all the pumps. Thus, the characteristic curve of n parallel pumps is given by $H_{p,n}(Q) = H_p(Q/n)$, where H_p is the characteristic head of one pump, and n is the number of pumps running. Wastewater pump (F1) design duty point is 60 L/s with a total dynamic head of 39.2 m from the available project data. The combined design duty point of the stormwater pumping system is 700 L/s at a total dynamic head of 9.9 m and the system consists of 3 duty pumps (pumps P1, P2 and P3) in parallel. There is a fourth storm water pump (P4) that is kept as a backup. The preliminary linear approximation is in the form of $H_p(q) = 41.9469 - 45.7813q$, where H_p is the developed head in metres, and q is the discharge of an individual pump in m³/s. The equivalent characteristics of the parallel pumps are shown as a common total-discharge basis in Figure 6.

To distinguish between common discharge-main losses and individual pump-branch losses, the system curves were developed. Reconciled hydraulic-head balance at the baseline design condition is a static head of 7.000 m, main-line friction losses of 10.041 m, pump-branch friction losses of 0.756 m, and minor losses of 5.937 m, adding up to a TDH of 23.734 m at a total discharge of 1.14375 m³/s. The loss coefficients for the pump branches were calculated based on the baseline pump design condition, assuming quadratic head loss relationships and identical pump branches. The main line coefficient of loss is 7.6756, the common minor loss is 4.5384, and the individual pump-branch friction coefficient is 5.2012 (with discharge in m³/s and head in metres). Thus, the system head for n operating pumps is given by $H_{sys,n}(Q) = H_s + (K_m + K_{min})Q^2 + K_\beta(Q/n)^2$, where the static head is H_s, the main-line friction coefficient is K_m, and the common minor-loss coefficient is K_{min}. The minor losses reported are considered to be typical discharge-system losses for the purposes of this preliminary assessment; the actual location of fittings and valves should be confirmed on the project drawings. The system characteristics for one-, two-, and three-pump operation are shown in Figure 7.

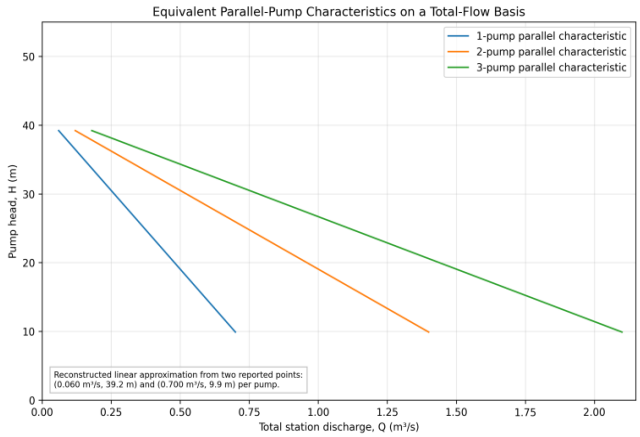

Figure 7. Reconstructed pump performance characteristics for one-, two-, and three-pump parallel operation, expressed on a consistent total-station-discharge basis

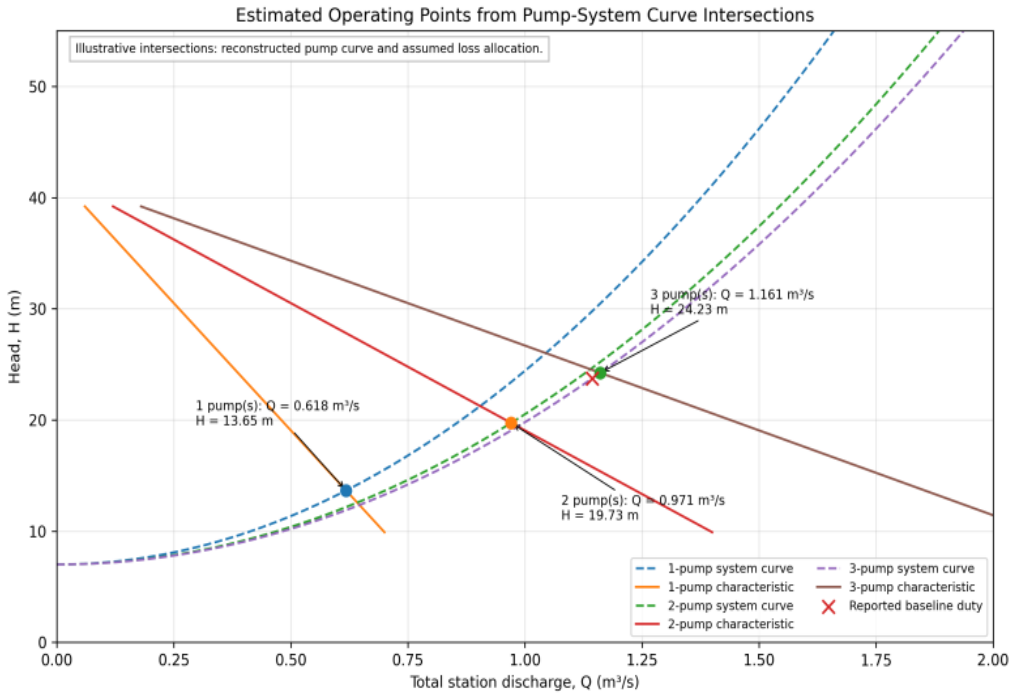

Figure 8. Estimated operating points determined from the intersections of the parallel-pump characteristic curves and corresponding system curves for one-, two-, and three-pump operation

For each pumping configuration, an operating point is determined by the intersection of its corresponding parallel-pump characteristic and system curve based on the condition, $H_{p,n}(Q) = H_{sys,n}(Q)$. The preliminary estimated discharge and head for operating in one-pump, two-pump, and three-pump modes are approximately 0.618 m³/s and 13.65 m, 0.971 m³/s

and 19.73 m, and 1.161 m³/s and 24.23 m, respectively, based on the reconstructed candidate pump curve and the adopted system-loss coefficients. These intersections are shown in Figure 8 and illustrate the different discharges at each station and head required for different numbers of working pumps. Three pumps is the highest estimated station discharge configuration and is used as the primary configuration to validate the design-duty requirement. The estimated discharge of the three pumps is reasonably close to the baseline design discharge of 1.14375 m³/s, but the estimated head is marginally different from the reconciled design TDH of 23.734 m.

Table 4. Post hoc force-main diameter sensitivity using a consistent Swamee–Jain friction-factor relation

Diameter (m)	Velocity (m/s)	Main Loss (m)	Approx. TDH* (m)
0.55	4.814	35.99	49.69
0.60	4.045	22.91	36.60
0.65	3.447	15.13	28.82
0.70	2.972	**10.31**	**24.00**
0.75	2.589	7.21	20.91
0.80	2.275	5.17	18.86
0.90	1.798	2.82	16.51
1.00	1.456	1.64	15.33

Note: *Static, branch, and minor losses are held at baseline values for isolation of the diameter effect. TDH: Total dynamic head.

The results show that the operating parameters of the pumping station vary depending on the combination of the pump performance and the number of pumps in operation, as well as the hydraulic resistance of the discharge system. The difference between common main-line losses and individual branch losses is critical for achieving accurate assessments of the different operating modes. The curves only approximate the operating conditions that are to be expected, but the actual operating point must be determined from the system-loss distribution and the manufacturer's provided pump performance curves. It is therefore recommended that the operating points presented should be regarded as preliminary engineering approximations and should not be viewed as certified operating conditions for the pump.

The project also contains a candidate pump H-Q performance chart which is reproduced at higher legibility in Figure 5. The flow-head combination at 381.25 L/s per pump at 23.734 m is in the plot range and can be considered as a starting point in the preliminary screening process. The three-pump, seven-minute recovery case is much more intensive: About 545 L/s per pump is needed, with the reconstructed station head increasing to ~ 41.2 m; the source H-Q curve at this flow rate offers much less head. This is an engineering consistency check and not final pump acceptance, as the manufacturer, pump model, impeller diameter, test tolerance, and certified NPSH data are not exactly documented.

The operating points estimated from the intersections of the pump and system curves are shown in Table 4. The results show that the total station discharge and operating head increase with the number of operating pumps. The estimated total discharge with the three-pump configuration is 1.161 m³/s at 24.23 m operating head.

4.5 Diameter sensitivity and energy implication

This is validated by the post hoc diameter analysis. The same explicit friction factor correlation is used for the estimation of the main-line loss. On the other hand, if the diameter is increased to 0.80 m, the estimated main loss becomes approximately 5.17 m, which is roughly 50% less compared to the audit value of 0.70 m. The sensitivity study was carried out with the explicit correlation (Swamee–Jain equation), instead of by graphical interpolation from the Moody chart. The comparison shows that the key hydraulic design parameter that affects the frictional head loss is the force-main diameter and that the optimal diameter for the lowest economic cost still needs to be determined using life-cycle costing. The friction head loss obtained was 10.31 m for a force-main diameter of 0.70 m, as illustrated in Table 4. It is 0.269 m (around 2.68%) above the baseline value of 10.041 m. A small difference is due to two different approaches used to calculate the Darcy friction factor: the approach of Swamee–Jain in the sensitivity analysis and the approach of graphical interpolation from the Moody chart in the baseline calculation. So the two values are fairly similar, and the difference is not a material factor in the overall design conclusion.

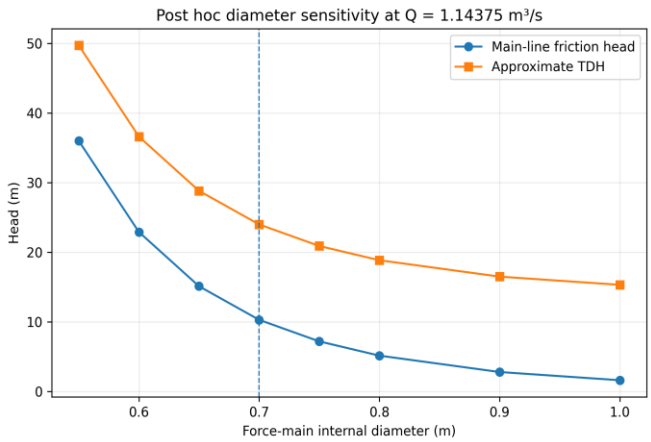

Figure 9. Sensitivity of main-line friction head and approximate total dynamic head (TDH) to force-main diameter at the baseline peak flow

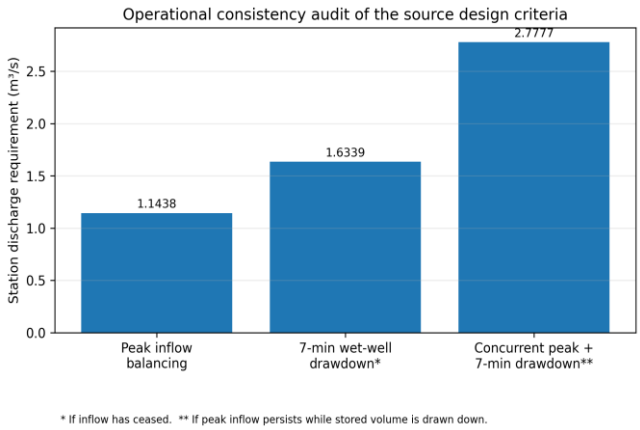

Figure 10. Operational consistency audit distinguishing peak-inflow balancing from post-event and concurrent wet-well drawdown requirements

4.6 Operational consistency audit

The biggest internal design problem is how to understand the seven-minute drawdown requirement. After inflow has stopped, the required 1.6339 m³/s will be needed to extract the

stored 686.25 m³ in 420 s. Assuming an equal distribution of the work among the three duty pumps, the criterion would mean 1,960.7 m³/h/pump, or approximately 42.9% more than the reported 1,372.5 m³/h/pump. However, more importantly, the system curve is reconstructed, and it can be seen that at this larger total discharge, the station head would rise from 23.734 m to ~41.2 m. Assuming a peak inflow of 1.14375 m³/s and that the volume stored was to be removed in 7 minutes, the theoretical station discharge would be 2.7777 m3/s with the resistance being even higher. Figure 9 shows the sensitivity of main-line friction head and approximate TDH to force-main diameter at the baseline peak flow. The flow criteria are thus isolated in Figure 10, and the reason for using separate system heads for each criterion is shown in Figure 4.

Table 5 compares peak-inflow balancing and wet-well recovery requirements. This total discharge of 1.14375 m³/s from the three duty pumps equals the assumed peak inflow but results in no net storage drawdown. If there is no inflow to the tank, then this pumping capacity needs about 10 minutes to pump the 686.25 m³ of volume stored in the tank. A higher discharge requirement of 1.63393 m^3/s was used for a recovery period of seven minutes; however, no documented project needs supporting this recovery period were available. The seven-minute criterion, therefore, is not considered a design criterion, but rather an alternative assessment scenario. Use the system curves to determine the corresponding TDH for each operating flow and wet-well level.

Table 5. Pump-capacity interpretations derived from the reported wet-well and flow criteria

Operating Interpretation	Total Station Flow	Per-pump Flow for 3 Duty Units	Design Meaning
Peak inflow balancing	1.14375 m³/s	1,372.5 m³/h	Maintains level approximately steady at baseline peak inflow
7-min drawdown after inflow ceases	1.63393 m³/s	1,960.7 m³/h	Recovers 686.25 m³ active volume within 420 s
Peak inflow + 7-min concurrent drawdown	2.77768 m³/s	3,333.2 m³/h	Very conservative theoretical combined criterion; not established by source design

4.7 Mechanical, control, and planning outputs

The design for the proposed pumping station uses submersible wet-well pumps with guide-rail retrieval systems, individual discharge branches, non-return valves, isolation devices, and a common discharge force main. Primary trash screens are offered at the pumping unit inlet to keep large debris and suspended materials from entering the pumping unit, minimizing the threat of clogging and mechanical damage. Controlled by level sensors and programmable logic controller (PLC) logic, the pump will operate in a lead–lag sequence, duty alternation, and high water level alarm. Mechanical lifting devices are also included with guardrails, access platforms, and ladders to ensure equipment can be retrieved, inspected, and carried out for routine maintenance in a safe manner. These mechanical and operational features contribute to system reliability and are required to meet maintenance needs of the hydraulic operating conditions.

Two similar pumping stations, with a combined nominal capacity of 20,000 m³/day, and an indicative capital cost of 400 million Iraqi dinars (IQD) are considered in the preliminary planning proposal. The nominal discharge is 833.33 m^3/h (0.2315 m^3/s), which is significantly less than the calculated peak stormwater discharge of 4117.5 m^3/h (1.14375 m^3/s). Thus, the nameplate pumping capacity should not be equated with the pumping capacity that is needed during peak storm events. In addition, the proposed capital cost is a planning estimate and not a validated economic assessment, as detailed bills of quantities, equipment quotes, price reference dates, station-specific inflow hydrographs and life-cycle cost calculations are not completed. Event-based pumping duty cycles, civil construction quantities, equipment and installation costs, electric tariffs, discount rates and expenditure assumptions for maintenance would be necessary for a comprehensive economic evaluation.

Two different hydraulic operating scenarios were considered to examine the pumping needs in peak-inflow balancing and accelerated wet-well storage recovery scenarios. Three operating pumps deliver a combined discharge of 4,117.5 m³/h or 1.14375 m³/s, which is the estimated peak inflow in the first scenario. With this perfectly balanced situation, the pumping discharge is equal to the inflow, so that the total amount of water stored does not decrease. The reconciled hydraulic-head balance results in a reference static head of 7.000 m, and the sum of the dynamic hydraulic losses is 16.734 m: 10.041 m of main-line friction losses, 0.756 m of pump-branch friction losses, and 5.937 m of minor losses. Consequently, the TDH for the baseline design condition is 23.734 m (TDH = 7.000 + 10.041 + 0.756 + 5.937 = 23.734 m).

In the second scenario, the active storage volume for the wet-well is assumed to be 686.25 m³, which should be emptied within 7 minutes, with no additional inflows during the recovery period. The corresponding discharge is calculated as $Q = V/t = 686.25/(7 \times 60) = 1.63393$ m^3/s, equivalent to approximately 5,882.14 m^3/h. The required recovery discharge is 1.63393 m^3/s, which is about 42.86% higher than the baseline discharge of 1.14375 m^3/s. This increased discharge requirement means that the seven-minute recovery condition will significantly exceed the hydraulic demand of the peak-inflow balancing condition.

The dynamic hydraulic losses in the baseline condition were scaled by the discharge ratio squared to estimate the dynamic hydraulic losses under the accelerated recovery condition. The following method is preliminary and assumes that pipeline geometry and hydraulic resistance coefficients, pipeline friction factors, and flow distribution between operating pump branches remain constant. Thus, the dynamic head loss under the recovery condition is expressed as $H_{dyn,2} = H_{dyn,1} \times (Q_2 / Q_1)^2$, where $H_{dyn,1} = 16.734$ m is the baseline dynamic head loss, $Q_1 = 1.14375$ m^3/s is the baseline discharge, and $Q_2 = 1.63393$ m^3/s is the required recovery discharge. Substituting these values gives $H_{dyn,2} = 16.734 \times (1.63393/1.14375)^2 = 34.15$ m. As a result, the approximate total system head for the seven-minute recovery condition is $H_{sys} = 41.2$ m for the seven minute recovery condition, with H_{static} being assumed as 7.00 m.

The corrected dynamic head of 34.15 m has been reported,

rather than the previous 12.2 m reported, which was inconsistent with the total system head. The revised calculation indicates that the approximate recovery head of 41.2 m is made up of 7.00 m of reference static head, and 34.15 m of estimated dynamic hydraulic losses. The accelerated recovery condition has a TDH of 23.734 m, which is about 73.4% higher than the baseline condition. This substantial rise is due to the fact that under the screening assumptions, the hydraulic resistance is quadratically dependent on the discharge.

It should be noted that the estimated recovery head of 41.2 m is a preliminary system-head requirement, not a proven operating point of the pump. The actual static head will depend on the water level in the wet well and the dynamic losses will be dependent on the distribution of flow through the individual pump branches and common force main. Furthermore, the pumping system's capacity to provide the desired recovery discharge should be demonstrated by the interaction of the pumping system's characteristic and the pumping system's system curve. It should, therefore, be treated as a representative hydraulic recovery scenario until the actual fitting-loss distribution and manufacturer-calculated performance characteristics for pumps are known, and until operating water levels are known. The considerations also make the baseline peak-inflow balancing condition different from the accelerated storage recovery requirement and make the interpretation of the associated hydraulic and operational demands consistent.

5. DISCUSSION

5.1 Hydraulic adequacy and meaning of the design point

The case shows the importance of considering a pumping station as a system, and not as a pump catalogue selection. The maximum inflow verified is consistent with the reported Rational Method parameters, and main hydraulic calculations can be reproduced from the pipe geometry and losses reported. The corrected case-study TDH is also physically interpretable since all the major head components can be identified. This traceability is a more solid foundation for design review than a nominal manufacturer curve. Meanwhile, hydraulic adequacy is dependent on the rainfall intensity assumed. The project does not offer IDF derivation or return period, so this value (10 mm/h) may not be seen as a full local derivation or return period. The immediate result is the stress test: If the intensity of stress is 20 mm/h under the same catchment size and runoff coefficient, the peak using the Rational Method would be doubled. Recent urban-drainage studies also highlight that future precipitation extremes can lead to pressure build-up in the network and accelerate the triggering of critical capacity thresholds [18, 24]. The next technical step for Al-Zahraa is, then, not a more sophisticated manipulation of the same individual intensity, but rather retrieval of the original gauge record and creation or validation of local IDF curves. The second constraint to hydraulic adequacy is added by the wet-well result. The result of this 10-minute active-storage assumption is 686.25 m³, but the consequence is a non-trivial geometric end result of 9.93 m active depth. Adjust the storage interval or plan dimensions if the final civil layout does not provide for that active depth, along with the dead storage, minimum submergence, and freeboard. Therefore, hydrology, station footprint, operating levels, and pump cycling should all be determined as one design task – otherwise the correct volume numerically could be geometrically or operationally impractical.

5.2 Energy significance of hydraulic losses

The most significant quantitative result is that 70.5% of the necessary pump head is used for dynamic resistance. The static lift is 7 m, and pipe/branch/loss of local lift is 16.734 m. The energy performance is very sensitive to diameter, fitting configuration, roughness, and operating flow for this composition. That is, a high-efficiency pump does not overcome an unnecessarily restrictive conveyance system. The energy-aware design should minimize unnecessary system resistance prior to optimizing the motor-pump unit. The diameter stress test is a great example of the size design lever. A calculated main friction loss is about doubled when the main is reduced from 0.70 m to 0.60 m, and the main is increased to 0.80 m, the loss is approximately halved. This is not conclusive, as it might suggest that 0.80 m is economically optimal, as pipe capital cost increases with diameter and may not be used at maximum flow around the clock. It does state that a life-cycle cost analysis is warranted. This should be an analysis that combines the system curve with average duty time, pump efficiency as a function of flow, electricity price, discount rate, and pump maintenance costs. Such an assessment should couple the system curve with the flow–efficiency relationship from certified pump curves, representative duty cycles, electricity tariffs, discount rate, and maintenance/overhaul and replacement assumptions based on operating hours or design life. Although the economic optimum is not quantified yet, the project's qualitative recommendation not to reduce the 0.70 m main is hydraulically sound. This is supported by the reconstructed system curve. Dynamic losses already represent 70.5% of the baseline head and will result in a higher pump requirement if the discharge is increased. The post-event recovery time of 7 minutes adds a 42.9% increase in discharge, although there is a 73% (23.734 to ~41.2 m) increase in reconstructed head for the fixed-geometry resistance model. This is not a linear gain, and that's why storage-recovery targets need to be assessed on the entire system curve. It also implies that simply increasing the pump capacity does not necessarily result in a more economical hydraulic regime, if the force-main diameter is not reconsidered.

5.3 Pump sizing, storage recovery, and operational logic

Peak-inflow balancing is different from wet-well storage recovery, which is what the hydraulic audit differentiates. An estimated peak inflow of 1.14375 m³/s would be balanced by three pumps, each with a capacity of 1,372.5 m³/h if the discharge can be achieved at the same system head. If inflow and pumping discharge were equal, the stored volume would not decrease, but remain constant. If flow is stopped, an average discharge of 1.6339 m³/s will be needed to remove 686.25 m³ in seven minutes, which corresponds to 1,960.7 m³/h per pump with a uniform discharge of three pumps. Therefore, it is important to remember that the balancing discharge and the drawdown discharge are different operating flows and should not be treated as interchangeable design flows.

The initial case of drawdown is about 545 L/s per pump at an estimated system head of 41.2 m (with baseline head of

23.734 m). The drawdown head estimate assumes constant static head and quadratic scaling of dynamic losses. Further evaluation is needed for changing wet-well levels and flow-dependent friction factors. Available source-project pump chart not identified by manufacturer, not certified, does not establish compliance to operating requirements, cannot justify final pump acceptance/rejection.

Consider the storm inflow hydrograph, available active storage, and potential for additional precipitation peaks when determining storage-recovery requirements. Cycling, maximum water levels, and recovery time can be evaluated using dynamic simulation that includes wet well geometry, pump characteristics, start–stop elevations, and varying inflow over time. Final pump selection is subject to the manufacturer-provided pump curves and parallel pump analysis for the pump family for the specified operating cases.

5.4 Proposed reliability measures and outstanding verification

Design provisions include standby pumps, level-based controls, alarms, and protective devices. This has not been proven to be effective under failure conditions by dynamic simulation, failure-scenario analysis, or operational testing. Therefore, the findings from the presence or absence of protective features should not be interpreted as evidence of verified station resilience.

In the event of a power failure, pumping may be interrupted, and inflow may continue to cause wet-well levels to rise. There should be consideration of available storage, backup-power capacity, transfer delay, and pump restart sequencing in assessing. If one duty pump failed, the other would need to take up the required duty without failure; otherwise, there is a loss of available discharge. The remaining pumping capacity should be determined from the appropriate parallel-pump and system curves. If the wet-well is experiencing high levels, then the alarm and standby start set points, inlet surcharge, available freeboard, and possible overflow routes need to be evaluated under a changing inflow.

Transient pressure analysis is still excellent in the areas of pump trips, starts, power restoration, and check-valve closure. Net positive suction head available must also be verified against manufacturer's requirements with a suitable margin over the operating range, and especially at the minimum operating water level. Operational reliability and resilience are not yet verified until failure scenarios are simulated and analysed in transient-pressure analysis and NPSH is verified.

5.5 Resilience, redundancy, and control

The source design is correctly identifying redundancy and level control as resilience measures. These features reduce the likelihood that a single piece of equipment will cause a flood: A standby pump, alternating duty sequence, high-level alarm, bypass/isolation, and maintainable screens. This is consistent with the wider resilience studies which indicate that drainage system performance is influenced not just by overload on the system due to rain but also by drainage blockage and failure of drainage components [17]. The proposed two decentralised stations, however, should be tested using network routing before being quantified as a resilience benefit. True redundancy requires that the diverted flow be received by the other station hydraulically and that the conveyance methods are available in the event of failure. Automation can increase reliability but also creates dependency on power, instrumentation, and control integrity. Therefore, the proposed PLC and level sensors should be backed up with independent high-high level alarms, manual override, fail-safes, and power redundancy. Sun et al. [21] showed that higher levels of predictive control will enhance the performance of stormwater storage, although these solutions depend on the availability of rainfall or inflow predictions and validated models of the system. A strong baseline for the present station would be robust rule-based lead/lag control, while predictive control is a future enhancement once instrumentation and data collection are in place.

5.6 Pump intake, cavitation, and transient risk

The qualitative aspects of the design covered here (vortex avoidance, submergence, check valves, air release, and gradual isolation) are suitable but not a replacement for a full hydraulic verification. The approach conditions should be uniform and free of excessive swirl and entrained air, or the actual operating state can be influenced by the approach conditions and move away from the intended curve, and can also lead to vibrations and maintenance demands [12]. To ensure the final wet-well configuration meets the current intake-design criteria, the final pump bell or suction geometry should be compared with them. Computational and/or physical modelling may be required if the geometric criteria are exceeded. Similarly, steady-state TDH is not a means to quantify water hammer. A 1,000 m force main carries a considerable momentum, and a power failure may result in a fast deceleration, backflow, closing of check valves, and propagation of a pressure wave. Chaudhry [4] demonstrates that the transient response is influenced by the wave speed, pipe elasticity, valve characteristics, pump inertia, and boundary conditions. The current design has adequate protection, but a surge analysis is needed to establish maximum and minimum pressures, vacuum risk, and the requirement for surge vessels, air/vacuum valves, controlled check valves and/or variable-speed ramping.

5.7 Comparison with recent urban-drainage literature

Methodologically the present case is simpler than recent model-studies, as seen in the comparison. As a result, the publishable value of its lies in transparency and a data-driven audit, not in its purported equivalent predictive sophistication. The paper shows how a traditional municipal calculation can be turned into a design and how self-consistency can be identified prior to embedding it in procurement. The design should be extended with observed rainfall, added up GIS network geometry, dynamic simulation, and at least one observed or independently benchmarked event to achieve a more comprehensive predictive resilience assessment. Table 6 shows the positioning of the case study relative to selected urban-drainage and resilience literature.

5.8 Economic and sustainability interpretation

Pumped drainage is a regular sustainability cost that is related to energy use. The theoretical hydraulic power at the peak point of 266.3 kW for three duty pumps is NOT an annual energy estimate but is an instantaneous lower bound, before pump and motor inefficiencies. The frequency and duration of operation are important in determining annual energy. However, the loss decomposition indicates where energy is being lost: almost three-quarters of the energy is dissipated in

head. Minimising avoidable friction can reduce the cost of installed power and operational cost of the system without reducing flood protection. This is not enough to make the 400 million Iraqi dinars cost estimate made for the project sustainable. An economic analysis that is credible should separate out civil works, pumps, electrical systems, controls, force main, standby power, screening, land, and maintenance; and compare alternative diameters and pump configurations using a net present cost. A larger main may cost more at set-up, but it will be lower in energy cost; a smaller main will be lower in set-up cost and higher in energy cost. The optimum is related to duty-cycle distribution and not just with peak-flow hydraulics.

5.9 Integrated design implications for data-constrained urban drainage

Combined, the results indicate three dominant controls: the hydrologic loading is how fast storage is being depleted, the hydraulic resistance is the head loss resulting from the required discharge, and the operating logic is the condition of the discharge flow. If there is any ambiguity in one, it spreads throughout the station. The baseline peak flow and 0.70 m force main are a consistent and coherent preliminary design for Al-Zahraa, while the 7-minute drawdown statement is a different duty because both discharge and system head are increasing. The interpretation can be applied to other data-rich municipalities. The calculations may be audited, the head balance and system curve reconstructed, normal/recovery/emergency cases separated, pumps tested over the entire operating range, and datasets for rainfall, geospatial, transient, and economic data checked through a validation gate, without the need to invent missing datasets. The progression from preliminary design to validated procurement is detailed in Figure 1.

Table 6. Positioning of the case study relative to selected urban-drainage and resilience literature

Study	Primary Focus	Relevant Finding	Relation to Present Case
Mugume et al. [17]	Structural resilience	Failures beyond hydraulic overload can drive flooding	Supports standby, blockage and component-failure thinking
Dong et al. [8]	Green vs grey resilience	Resilience depends on infrastructure portfolio and future uncertainty	Pumping should later be assessed with catchment-scale alternatives
Wang and Wang [26]	Extended Rational Method	Peak-flow methods can be extended but require careful interpretation	Supports use of Rational Method as baseline, not a hydrograph
Wang et al. [24]	Drainage resilience benchmark	Intervention ranking depends on resilience benchmark	Supports explicit operating cases and baseline definition
Azari and Tabesh [2]	Stormwater sustainability optimization	Network rehabilitation and BMPs can be jointly optimized	Motivates lifecycle/network optimization beyond pump sizing
Sun et al. [21]	Predictive stormwater control	RTC can improve limited storage performance	Future pathway for wet-well/storage control
Wu et al. [28]	Pumping capacity and flood susceptibility	Pump benefit can exhibit nonlinear thresholds	Warns against capacity increases without system analysis
Rohith and Sudheer [18]	Safe-fail stormwater design	Flood consequences should inform design return periods	Supports robustness and failure-consequence framing
Wang et al. [25]	Future drainage pressure	Climate pathways can increase drainage pressure	Supports rainfall-sensitivity and IDF revalidation

6. CONCLUSIONS

This research recreated and examined the whole chain of the hydraulic design process for an urban stormwater lifting station in Al-Zahraa neighborhood in Wasit Governorate, Iraq. Using the available project inputs, the Rational Method peak runoff is 1.14375 m^3/s for A = 915,000 m^2, C = 0.45, and I = 10 mm/h. The selected plan area for the wet well is 9.6 m × 7.2 m. For the above 9.6 m × 7.2 m plan area, the 10-minute effective storage equals 686.25 m^3 with an effective depth of about 9.93 m. The measured velocity for the 1000 m force main is about 2.972 m/s, with the diameter of the force main being 0.70 m, which is in the turbulent regime. The main friction is 10.041 m, the pump-branch friction is 0.756 m, and the minor losses are 5.937 m. With all components installed and a static lift of 7.0 m, the correct case-study TDH will be 23.734 m. The dynamic resistance is 70.5% of that head, showing that it is the force-main and fitting design that is most important to the energy performance of a station. The baseline peak inflow is balanced by three duty pumps operating at 1,372.5 m^3/h each, but this is not the same as the separately declared requirement to pump down 686.25 m^3 in 7 minutes. The latter will need 1.6339 m^3/s when no inflow occurs, which is equal to 1,960.7 m^3/h for each of 3 equal pumps. If the project resistance is presented as a system curve, then that bigger total flow is approximately equal to 41.2 m of head, instead of the baseline of 23.734 m of head. The project pump chart is therefore only used for a preliminary consistency check, and certified pump curves will be used to define the final operating point. The diameter sensitivity also shows that if the diameter of the main is reduced below 0.70 m, then there is a significant friction and energy loss, but if the diameter is increased then the head loss can be reduced significantly. That leaves the current 0.70 m as a defensible minimum baseline selection, for which economic optimisation will call for lifecycle analysis. Overall, it is demonstrated that a data-constrained design can be made auditable through reconciliation of TDH, reconstruction of the system curve, separation of operating cases, and correlation of pump selection to a validation gate. Construction is still underway, and the design rainfall intensity is tentative until it is verified with local IDF data. A topographic survey of the site and hydraulic simulation of the hydrologic network have yet to be completed. The performance curves for the pumps are pending

certification and the hydraulic transient analysis and life-cycle cost analysis are still under way.

7. RECOMMENDATIONS AND FUTURE WORK

Keep the minimum baseline force-main diameter of 0.70 m until a life-cycle cost optimization has shown that increasing the force-main diameter results in a lower net present cost, without causing adverse low-flow operation. Specify 3 distinct hydraulic operating cases in the design basis: (1) Peak-Inflow Balancing, (2) Post-Event Storage Recovery, and (3) Emergency/High Level Operation. The 7-minute recovery flow should not be assessed at the baseline level of TDH, and each case should be assessed individually with the system head. The reconstructed baseline duty of 1,372.5 m^3/h per pump at 23.734 m TDH should only be used as a starting point for procurement. Finally, certified manufacturer H-Q, efficiency, power, NPSH curves, pump model/impeller data, solids-passage requirements, and allowable operating region around BEP must be considered. Verify the proposed three duty/one standby configuration using dynamic storm simulations and parallel-pump curves. Seeking only the nominal redundancy percentage doesn't demonstrate standby capacity; it is important to demonstrate standby capacity under credible pump-outage and high-level scenarios. Re-check the active volume of the wet-well (686.25 m^3) and the implied active depth in the wet-well (9.93 m) with the surveyed level of the inlet invert, minimum pump submergence, dead storage, freeboard, sediment allowance, access and structural requirements before finalizing civil dimensions. Ensure inlet penstocks, isolation valves, and pump operation have redundant level sensing, automatic lead/lag alternation, independent high-high alarm, manual override, standby power, and fail-safe interlocks. Finalize check-valve characteristics, air/vacuum valves, surge-control devices, pipe pressure class, and thrust restraint after performing numerical transient analysis of pump trip, power failure, pump start-up, and valve closure. Maintain strong screening, guide-rail pump retrieval, dismantling facilities, lifting equipment, and safe access measures to ensure that debris management and hydraulic maintenance are not the constraint for hydraulic reliability.

Future research

Use documented rainfall-gauge data to establish a site-specific IDF relation and clearly specify the design return period; determine uncertainty surrounding the rainfall intensity rather than a single adopted value. Generate a georeferenced catchment and drainage-network model based on surveyed coordinates, invert levels, and a digital elevation model, and then adjust the model to match observed rainfall and water-level/flood events using EPA SWMM or other similar models. Assess pump enlargement and climate-adjusted, non-stationary rainfall scenarios to identify when the existing pumping/storage arrangement will reach service thresholds and if more pumping storage will be effective or distributed green infrastructure will work better. Apply the system-curve analysis to any system to perform a complete lifecycle optimization of the force-main diameter, pump number, variable-speed operation, energy consumption, maintenance, replacement times, and carbon emissions. Install existing or newly installed instrumentation to measure rainfall, wet-well level, pump runtime, pressure/discharge, and electrical energy to validate design requirements based on actual field performance for predictive maintenance. Contrast ductile iron and HDPE force-main alternatives on the basis of roughness, surge response, installation constraints, durability, and lifecycle cost; comparison should not only involve steady-state friction but transient pressure as well. Determine if the final geometry clearly meets the intake uniformity and submergence requirements and, if not, investigate wet-well intake modeling – both computational and physical – paying special attention to vortices, air entrainment, sediment deposition, and pump-to-pump interaction. Assess hybrid solar or other distributed energy sources mainly for controls, instrumentation, and auxiliary services, following main pumping duty and emergency backup-power requirements, and ensure resilience in case of grid outage remains the key consideration.

DATA AVAILABILITY

In this manuscript, all numerical parameters used in the steady-state calculations are reported in the manuscript. Supporting project records are held by the authors' institution and may be made available from the corresponding author upon reasonable request, subject to project confidentiality and data-sharing restrictions.

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NOMENCLATURE

A	contributing catchment area, m^2
BEP	best efficiency point
C	runoff coefficient
D	pipe internal diameter, m
f	Darcy friction factor
g	gravitational acceleration, m/s^2
h_f	major friction head loss, m
h_m	minor/local head loss, m
H_s	static head, m
I	design rainfall intensity, mm/h
K	minor-loss coefficient
L	pipe length, m
NPSH	net positive suction head, m
PLC	programmable logic controller
Q	flow rate, m^3/s
Re	Reynolds number
RTC	real-time control
SWMM	Storm Water Management Model
TDH	total dynamic head, m
V_e	effective wet-well volume, m^3
ν	kinematic viscosity, m^2/s
ε	absolute pipe roughness, m