

## Safety-Reliability Index for Evaluating Urban Corridor Performance

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### ABSTRACT

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Urban road safety assessments are often performed separately from traffic operational analysis, limiting decision-makers' ability to understand the interactions between congestion, capacity constraints, and crash risks. This study proposes an integrated framework that combines safety and reliability evaluations for urban road corridors under uncertain demand and capacity conditions. We quantify capacity reliability using probabilistic performance density functions derived from estimated traffic demand and operational capacity calculations. Road safety is evaluated using the Level of Service of Safety (LOSS) measure as defined in the Highway Safety Manual (HSM, 2010). The methodology is applied to a 3-km urban corridor in Al-Kut City, Iraq, which connects three major intersections. Results indicate severe oversaturation, an 83% probability of capacity failure, and poor reliability ( $Z = -0.94$ ), all correlating with suboptimal safety performance (LOSS III–IV). A reliability-safety matrix is developed to identify critical locations, assisting decision-makers in integrated planning. Potential capacity improvements and safety countermeasures are evaluated, showing significant benefits in both capacity reliability and crash reduction. This study emphasizes the importance of integrating reliability analysis with safety assessments to support sustainable urban traffic management.

## 1. INTRODUCTION

Modern urban transportation systems have undergone continuous advancement across design, traffic control, and vehicle technology. On the other hand, traffic demand has increased while roadway capacity has remained limited. These have led to critical challenges; increased crash risk is the most challenging issue among them [1].

Traffic demand has increased due to rapid urbanization, population growth, and rising vehicle ownership. This has resulted in increased pressure on existing road infrastructure, creating congestion and unstable traffic operations. This is also accompanied by higher rates of conflict and more aggressive maneuvers. Consequently, road safety cannot be addressed independently of traffic operations and the balance between demand and capacity [2].

Conventional road safety performance measures rely on crash frequency and severity, neglecting the traffic operation performance [3]. In addition, conventional traffic performance measures primarily quantify operational efficiency but neglect the safety measures [4]. Modern safety assessment methodology, such as predictive models adopted by the Highway Safety Manual [5], considers roadway geometry, traffic control, and traffic volume. This framework is integrated with valid safety performance measures such as the Level of Service of Safety (LOSS). Fluctuating traffic demand reflects driver behavior, land-use patterns, and temporal factors. Therefore, it should not be ignored in traffic and safety studies, as it.

Reliability refers to the consistency of service performance. Capacity reliability refers to the consistency of a designed system in accommodating demand. Reliability analysis is important for monitoring the performance of facilities within a jurisdiction over time. It is also important in prioritizing interventions, identifying causes, and assessing the impact of a specific improvement on a facility post-implementation [5]. Although travel time and connectivity reliability have been widely investigated, these studies overlook capacity limitations. Therefore, this study will focus on capacity reliability as an index for assessing the current traffic situation at a selected facility.

Integrating reliability with road safety measures is likely to yield a more realistic assessment of traffic and safety performance. It also enables traffic engineers and planners to assess how often a roadway or intersection operates in a risky state [6].

Previous studies have developed reliability-based approaches to analyze road network performance. However, they primarily focus on travel time or network capacity reliability under simplified assumptions. Many models rely on probability distributions, but they rarely use operational traffic statistics from the real world. Additionally, the integration of dependability analysis with traffic safety evaluation at the corridor or intersection level has not been thoroughly studied. To assist safety-oriented transportation planning, there is still a need for useful frameworks that integrate operational traffic modeling with reliability-based analysis.

This study aims to develop a comprehensive framework for

integrating road safety with probabilistic reliability assessment. The proposed approach supports the development of balanced strategies that enhance both operational efficiency and safety. This research contributes to the development of safer, more resilient, and sustainable urban road networks by integrating safety, reliability, and demand management.

Capacity Reliability was selected for the study because it best captures the network's robustness to varying traffic demand while maintaining stable service levels. This is critical in contexts like Iraq, where infrastructure is often limited and unstable. Capacity Reliability enables us to account for unexpected breakdowns and selected capacity fluctuations. It also enables us to compare them with the expected capacity level. The main objectives of such an analysis include monitoring the reliability of a collection of facilities within a jurisdiction or region over time to prioritize them for operational or physical interventions. It is also used to identify the fundamental causes of reliability issues. In addition, it can be employed to assess the impact of a specific treatment or improvement on a facility post-implementation.

The novelty of this study lies in integrating capacity reliability analysis with the LOSS through a probabilistic reliability–safety matrix, enabling simultaneous assessment of congestion risk and crash potential under traffic demand uncertainty.

To estimate capacity reliability, road capacity and traffic demand are required. Therefore, the methods of estimating both were reviewed. Based on simplicity, accuracy, availability, and validity, the Highway Capacity Software (HCS) [7] and TranCAD will be used to estimate road capacity and traffic demand, respectively.

Regarding the road safety analysis, LOSS was selected as the safety indicator. LOSS provides a more consistent approach to identifying high-risk locations than simple crash counts and crash counts per traffic volume. The latter often exhibit random statistical bias and fluctuating volumes.

## 2. THEORETICAL CONCEPT AND LITERATURE REVIEW

### 2.1 Applications of reliability in transportation systems

Reliability measures the extent to which the current design meets objectives. It is also used to measure the extent to which the supplied design meets the demand [8, 9]. Higher reliability indicates reasonable road infrastructure with stable operational traffic. With increasing urban congestion and limited infrastructure capacity, transportation reliability has become a key performance measure [10-13]. Yu et al. [14] conducted research to investigate the feasibility of integrating reliability analysis with traffic safety analyses using data from a mountainous freeway in Colorado. Finding the risky portion using traffic flow and real-time crash data was the main goal. The rate of failure was used to determine the reliability index. According to the findings, crash rates were highest in segments with poor reliability. They came to the conclusion that in situations where crash data is lacking, the reliability index can be utilized. However, this study employed statistical crash prediction models such as logistic regression, Poisson, or negative binomial. Although these techniques are effective in identifying critical crash-related components, they do not explicitly assess the probability of system failure or the operational reliability of traffic infrastructure. Li et al. [15]

created a synthesis method for combining reliability analysis with road safety system assessments. They used crash data for ASEAN nations throughout various time periods. Policymakers in these nations have utilized the results to create strategic plans. Travel time and connectivity reliability have historically been the main subjects of research on transport network reliability [16, 17], followed by capacity reliability [18].

Connectivity reliability refers to the likelihood that all network nodes remain connected. Determining whether a certain origin-destination pair is still connected is a crucial step. If there is at least one path connecting any two nodes, the network is deemed functional in this sense [19]. Nevertheless, it does not take capacity constraints into consideration and is not relevant to small networks [11, 20]. Travel-time reliability refers to the consistency of travel time across various traffic conditions. It is particularly valuable for assessing network performance under typical daily traffic fluctuations [21]. Asakura [22] developed this concept to account for road deterioration, defining reliability as the ratio of travel times under degraded versus normal conditions. The concept presented in this research was the basis for setting level-of-service standards. Chen and Recker [21] used travel time to examine the effect of different risk levels on route choice under supply and demand uncertainties.

Capacity reliability refers to the capability of the supplied road infrastructure to accommodate travel demand under prevailing operational conditions and acceptable service standards without causing congestion and forming queues. It provides transportation engineers and planners with a structured, quantitative approach for evaluating system capacity and identifying areas for improvement [17, 20]. Reliable capacity enhances operational efficiency and reduces the likelihood of unexpected delays caused by traffic-flow breakdowns [23]. Therefore, as a direct reflection of road network capacity, capacity reliability has recently received greater attention from network planners and traffic management departments worldwide [17, 24].

In terms of applications, Sumalee and Kurauchi [25] used capacity reliability to assess post-disaster traffic regulation. Chootinan et al. [26] proposed an alternative reliability index focused on daily route-choice reliability, which Chen et al. [27] later incorporated into a new reserve-capacity model for signal-controlled networks. This work was further extended by Chen et al. to a bi-objective model that integrates both capacity and travel-time reliability under demand uncertainty. Chen et al. [20] subsequently proposed a methodology that combines reliability analysis, equilibrium modeling, sensitivity analysis, and Monte Carlo simulations for degradable networks. A bi-level programming model for network capacity reliability was proposed by Fang and Pan [28] to incorporate elastic demand and travel-time reliability into route-choice models. They found that route-choice behavior significantly affects capacity reliability. Ji and Ma [29], Hosseini and Pishvaei [30], and Hao et al. [24] then incorporated the capacity reliability index into route-choice models at a larger-scale network, based on capacity and demand at a given time.

Bao et al. [31] introduced a data-driven, temporally adaptive methodology for evaluating and enhancing the reliability of urban transportation systems. An integrated real-world passenger flow data into a percolation-based reliability evaluation was investigated. The methodology was applied to the Shanghai metro system. They proposed reliability metrics that account for both traffic-flow preservation and travel-time

variation under disruptions. By conducting extensive numerical simulations across different traffic patterns, they found that failing to account for travel quality can lead to an underestimation of network vulnerability. Pemberthy et al. [32] addressed poor traffic performance and low safety in Medellín, Colombia, by integrating statistical analysis and optimization modeling. The proposed methodology was built using statistical analysis of historical RTA data to evaluate transport network reliability. This study highlighted the value of combining RTA analysis and network reliability perspectives for data-driven strategic transportation planning. The approach offered actionable insights for policymakers and urban planners seeking to reduce accidents and enhance urban mobility through optimized resource allocation.

A study assessing the structural safety of current road bridges in Brazil using the semi-probabilistic and full probabilistic approaches was given by Santos et al. [33]. The primary goal was to meet Brazil's demand for precise and impartial safety evaluations of bridges in light of the country's growing traffic and inadequate infrastructure financing. According to the findings, the dependability study can ensure the safety of the bridges even with higher traffic volumes without incurring further expenses for structural reinforcement. As a result, the study emphasizes how crucial it is to advance reliability analysis in Brazil in order to guarantee the security of current bridges and to facilitate affordable, impartial safety evaluations.

The most widely used method to determine the reliability index is the probability density function (PDF). It is usually used to estimate the probability distribution over a link by measuring deviation from standard values or a threshold; the mean is typically used as the threshold. The probability of failure (non-compliance) can then be diagnosed [24, 29, 34, 35]. Soltani-Sobh et al. [12] used the PDF, assuming normal distributions for demand and capacity along the link. When the ratio of the estimated Capacity PDF to the demand density function (Demand PDF) is less than 1 (Capacity PDF < Demand PDF), the situation is identified as a failure situation (non-compliance) [36]. It was found that the demand-capacity uncertainty method is more effective than the travel time and connectivity reliability indices.

However, Huang et al. [17] stated that capacity reliability is more complex than travel time reliability because travel time varies independently of traffic volume. They failed to take into account how uncertainty impacts link capacity and how low reliability can change road users' route choices. In contrast to travel time reliability, which deals with temporal uncertainty from the viewpoint of the passenger, capacity reliability concentrates on the infrastructure's operational and physical consistency. In order to improve system performance, this distinction aids in prioritizing actions at crucial bottlenecks.

## 2.2 Capacity estimation

The Highway Capacity Manual (HCM) [7] defines two types of capacity: Geometric Capacity and Operational Capacity. Geometric Capacity is defined as a constant value and mainly depends on the physical characteristics of the road, such as lane width, road geometry, and vehicle type. Operational Capacity is estimated to consider the effect of the proportion of heavy vehicles, obstruction, interference among vehicle traffic, road lane width, vehicle parking on both sides of the road, and other factors.

There are various methods for estimating road capacity,

including HCM, model fitting, stochastic distribution, and breakdown-related methods [37, 38]. These can be broadly categorized into empirical, statistical, and simulation methods. Empirical methods are considered easier but are less accurate. Statistical methods are more accurate only when based on high-quality data. Simulation methods are the most precise but also the most computationally demanding and data-intensive. Many of the developed models lack validation with multiday observation data. Consequently, estimating traffic capacities at various stages of planning, design, and control is crucial for improving the quality of urban traffic management [39].

The HCS is a valid and widely used tool to estimate capacity when input data are accurate. Studies confirm its reliability and validity as a practical tool for traffic capacity analysis [40, 41]. It is based on the procedures described in the HCM. It permits a comprehensive analysis of signalized and unsignalized intersections, two-lane and multilane highways, roundabouts, and freeways. Many investigations have demonstrated HCS's ability to accurately assess traffic facility performance across different operational types and circumstances [17, 42].

## 2.3 Demand estimation

Travel demand is defined by Garber and Hoel [43] as the expected number of persons or vehicles per unit time travelling on a given transportation facility system under a set of given conditions. Traffic demand differs from traffic volume; traffic volume is the number of observed vehicles at a given time and location, while traffic demand is the number of vehicles desiring or attempting to travel along a roadway [9].

Travel demand modeling aims to forecast the origin, destination, mode, and route choice of trips. This process helps to comprehend the effects of current and future developments on the trip's characteristics [43]. Therefore, extensive research has been conducted on demand forecasting methods and models [44]. Four-step models, activity-based models, temporal categorization methods (e.g., kernel density estimation (KDE)), and statistical models (e.g., the upper percentile of observed volume in a given year) are among the most common methods for travel demand forecasting [45]. The selection of the most suitable method depends on the planning objectives and data availability.

The four-stage travel demand forecasting model is one of the widely used methods. It aims to estimate the number of trips between origins and destinations within a region. The model begins by estimating the total number of trips generated and attracted to each zone. These trips are then allocated from origin zones to destination zones using trip distribution models, creating an origin-destination trip matrix (O-D Matrix). Finally, each trip matrix is assigned to the route network of a particular mode using the trip assignment models.

TransCAD is a software application and GIS-transport modeling platform designed for comprehensive travel-demand modeling, compatible with Windows, and essential for engineers and professionals in planning and traffic management. TransCAD has been used in transport-planning studies for various purposes, including estimating origin-destination (O-D) flows, network coding and performance evaluation, and GIS-based accessibility/route assessment. It is often used to combine field traffic counts with zone-based socio-economic data and basic assignment procedures [46-48]. It has been employed at smaller spatial scales, reflecting a practical approach, especially when comprehensive

household travel surveys are not feasible [49]. In addition, it has been used to estimate the O-D matrix for a certain trip mode or for specifying trip purpose, for example, for truck freight demand estimation, to investigate the impact of freight flows on traffic congestion and logistics planning [50]. Qasim et al. [51] combined TransCAD with GPS/GIS data to assess traffic performance in Dhi Qar City by attaching traffic volumes and link attributes to the built road network to identify performance issues such as long travel times and low vehicle speed. Aboodi and Qasim [52] used TransCAD to evaluate the operational performance of a road network in Al-Kut City. They employed TransCAD to analyze capacity utilization, travel time, and connectivity. Overall, the reviewed studies used TransCAD as a planning tool when detailed survey data are limited; therefore, researchers entered traffic volume data, GIS layers, zoning, and assignment to produce the required O-D matrices and network performance indicators.

Traffic volume counts were collected during the peak period. These observed link volumes were used as inputs for the O-D matrix estimation procedure implemented in TransCAD. The software applies an iterative adjustment algorithm that reconstructs the most probable O-D demand pattern that reproduces the observed traffic flows while respecting the network topology and traffic assignment relationships. This process allows the estimation of corridor-level travel demand rather than relying solely on raw traffic counts.

Due to the absence of detailed household travel surveys or large-scale origin-destination datasets for the study area, the estimated O-D matrix is constrained by the available traffic count data. Consequently, the reconstructed demand reflects the best available approximation of travel patterns within the corridor based on observed flows. Although this approach is widely used in practical traffic studies, it may not fully capture latent or suppressed demand. Future studies could improve demand estimation by incorporating household travel surveys, mobile-phone mobility data, or automated traffic monitoring systems.

## 2.4 Level of Service of Safety

The LOSS is a safety performance measure; its concept was first introduced in 1972 and developed by Kaub in 2000 [53]. Kononov and Allery [54] have developed the concept of LOSS within the framework of the safety performance function (SPF) and employed it to quantify risk magnitude, describe the quality of safety level, and diagnose risky sites. It can also be used to frame a reference for decision makers. Kononov et al. [55] used a value of (1.5 times the standard deviation) to set the boundary limits of the LOSS.

Lu et al. [53] showed that the LOSS method was developed to address the issues of crash randomness and insufficient crash data, which result in biased safety assessments. The updated concept of LOSS by Kononov and Allery [54] is adopted by the Highway Safety Manual HSM 2010 [5] to use as an indicator of the safety level at a road facility. The main applications of LOSS are to categorize the study area, identify the highest-risk site, determine contributing factors, and propose countermeasures to improve the safety level [55].

Determining LOSS is based on the statistical difference between the observed crash frequency and the expected crash frequency, considering a confidence interval. The predicted

crash frequency is derived from the SPF and crash modification factors. The SPF is the expected crash frequency at a specific site, based on Annual Average Daily Traffic (AADT) and the assessed site's base conditions, representing the engineering characteristics used in deriving the SPF model. The base conditions vary by facility type. When the engineering characteristics of the study area differ from the base conditions, CMFs are used to adjust the SPFs to match the predicted crash frequency. Four categories of LOSS can be identified [5, 56]:

- LOSS I: represents a high safety level when the actual crash frequency is 1.5 standard deviations ( $\sigma$ ) value below the expected mean at a 95% confidence interval of the SPF.

- LOSS II: represents a moderate safety level when the actual crash frequency is 1.5 standard deviations ( $\sigma$ ) above and the expected mean within the lower 95% confidence interval.

At LOSS I and LOSS II, sites perform as predicted or better than predicted.

- LOSS III: represents low safety-level when the actual crash frequency is higher than the expected frequency and still within the upper 95% confidence interval. At this level, sites perform as predicted or slightly better than predicted; they may warrant attention.

- LOSS IV: represents too low a safety level (severe risk situation) when the actual crash frequency is significantly higher than expected within the upper 95% confidence interval. At LOSSIV, sites are experiencing significant safety problems and should be prioritized for treatments.

Hamilton [57] applied the LOSS to proactively assess the safety of the intersection of State Highway 75 and Plank Road (Cunty Road A) in Racine County, Wisconsin, US. The key factors affecting the corridor's safety were identified, and countermeasures were proposed based on the assessment results, including resurfacing the corridor with routine maintenance and correcting the intersection's geometric design. Abdulla and Karim [58] applied the LOSS measure, along with four other measures, to assess safety and identify the highest-risk site among 40 intersections in Sulaymaniyah, Iraq. It has been found that the LOSS approach offers an intuitive means of assessing safety performance.

## 3. METHOD OF THE RESEARCH

This section presents the methodology followed in this research, including a description of the study area and data collection. In the first step, the study area will be selected based on specific criteria. Then the needed data will be identified based on the input variables in the analysis step. The data will then be collected, including traffic volume, signal cycle details, road network's geometry data, vehicle speed, accident data, and other needed data. The ArcGIS 10.8.1 will be used to present data. The collected data will be analyzed using HCS 2010 to estimate the road capacity and TranCAD to estimate Demand. The performance density function will be used to estimate failure probability and reliability.

The accident data will also be collected from official documents and analyzed using the procedure of the Highway Safety Manual [5]. A reliability-safety matrix will be developed using the quadrant concept. Based on the findings, Improvements will be suggested, and the findings after improvements will be investigated.

The following subsections detail the research methodology.

### 3.1 The study area

A 3 km corridor connecting Al-Khajiya Street and Al-Zahra Commercial Street in Al-Kut City, Iraq, was selected as a case study. This corridor links Al-Khajiya Intersection, Al-Shuhada Intersection, and Al-Zahra Intersection. The black line in Figure 1 shows the corridor and the three intersections. The selected road starts at Badra Road, a highway connecting Al-Kut City to the Badra District on the border with Iran. It ends at Al-Amara Road, another highway connecting Wasit Governorate to Maysan Governorate. The study area was selected for its strategic location within a densely developed

urban area, directly influenced by numerous surrounding activities. These activities, including commercial establishments, educational institutions, and service facilities, collectively generate significant vehicular and pedestrian movements. These activities contribute to a substantial traffic demand, particularly noticeable during morning and evening peak hours, leading to increased congestion, vehicle queuing, and travel delays. As a result, the study area has become a critical node in the local traffic network, affecting both the operational efficiency of adjacent roadways and the corridor's overall LOS.

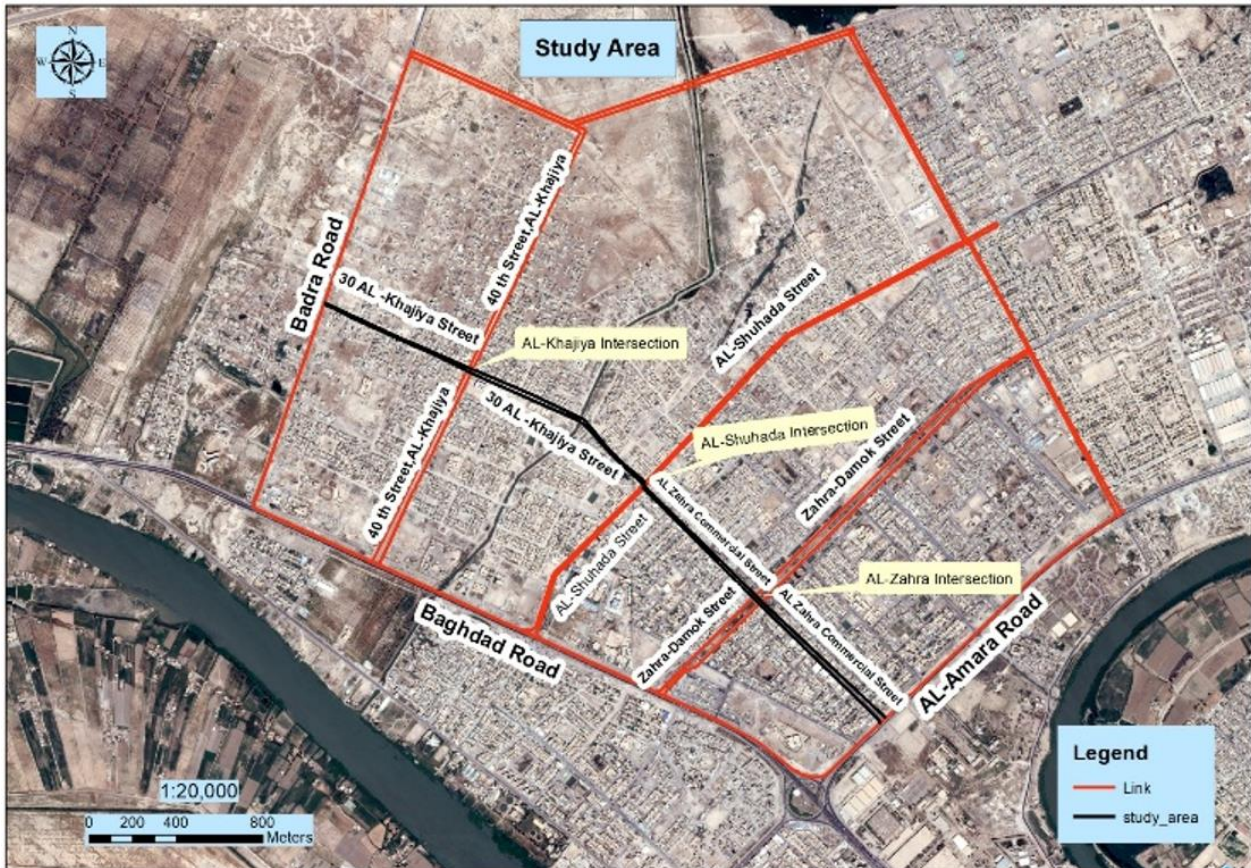


Figure 1. Google map image of the roads in the study area

Al-Khajiya intersection is a signalized four-leg intersection recognized as one of the most important in Al-Kut city. It facilitates the traffic flow from Badra Street to Al-Zahra Street and from Baghdad Street to Al-Khajiya Street. This intersection experiences congestion during multi-peak hours (morning and evening) due to high traffic volume.

Al-Shuhada Intersection is a signalized four-leg intersection connecting Al-Zahra main street with Al-Shuhada main street. The presence of various activities in the surrounding area of this intersection resulted in a high volume of traffic and delays, particularly during peak hours.

Al-Zahra Intersection is a four-leg, unsignalized intersection that serves as a junction for numerous city axes. Its significance is in the connection between residential and commercial areas. During peak hour, this intersection was characterized by increased traffic.

### 3.2 Data collection

The data collected for this study can be divided into four

parts: traffic data, geometric data, accident data, and other data. Traffic data includes traffic volume, operation speed, discharge headway, movement direction, signal cycle time, gap, and follow-up time. A video camera was used to capture traffic volume, and laser speed guns were used to measure vehicle speeds. Geometric data includes road width and length, lane number, median width, and other needed details. To determine the geometric parameters of each facility, a measuring wheel was used to measure lane width and segment length. Supplementary data were collected through observation, including lane count and the presence of a median. Accident data was collected from official documents.

#### 3.2.1 Traffic volume

To ensure consistency in traffic flow analysis, all vehicle types were converted to Passenger Car Units (PCUs) by multiplying the number of vehicles by the equivalent factors specified in the Iraqi Highway Design Manual (2014).

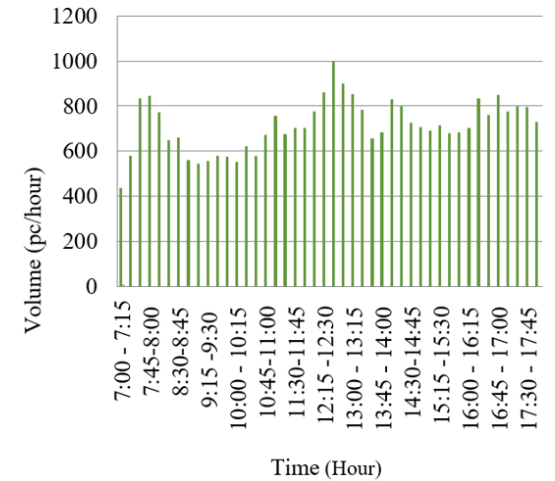
To determine the optimal period for traffic data collection during peak hours, traffic volumes were monitored at each

studied intersection over 11 hours from 7:00 a.m. to 6:00 p.m., from December 15 to 19, 2024. The peak traffic volume was observed between 12:15 p.m. and 1:15 p.m. at Al-Khajiya, between 12:00 p.m. and 1:00 p.m. at Al-Shuhada, and earlier

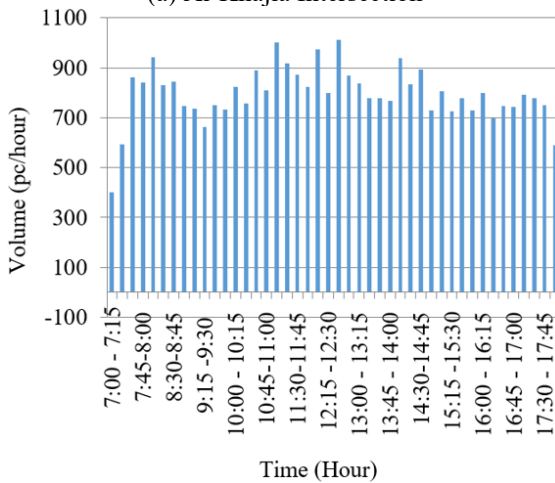
at Al-Zahra, between 10:00 a.m. and 11:00 a.m., as shown in Figure 2. The observed traffic volumes in PCUs by movement direction for each approach of the selected intersection in the study area are presented in Table 1.

**Table 1.** Movement volume for each approach (pc/hour)

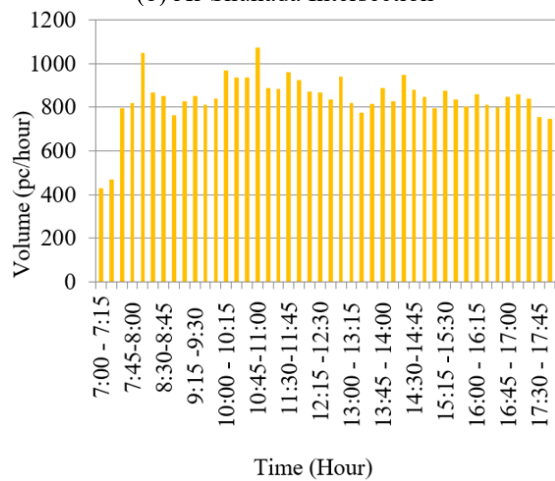
Time	Eastbound			Westbound			Northbound			Southbound			Total
	L	Th	R	L	Th	R	L	Th	R	L	Th	R	
Al-Khajiya Intersection	176	347	230	166	332	194	211	546	145	189	801	277	3614
Al-Shuhada Intersection	177	440	330	96	563	71	355	436	192	91	571	337	3659
Al-Zahra Intersection	78	709	250	147	992	259	204	355	174	185	431	134	3918



(a) Al-Khajiya Intersection



(b) Al-Shuhada Intersection



(c) Al-Zahra Intersection

**Figure 2.** The recorded traffic volume (pc/15 minutes)

### 3.2.2 Peak hourly factor

The ratio of the peak hour traffic volume to four times the peak traffic volume during the 15 minutes of the peak hour was calculated according to the traffic capacity guide using the following Eq. (1):

$$PHF = \frac{V}{4V_{15}} \quad (1)$$

$PHF$  = peak hour factor,  $V$  = hourly volume (pc/hour), and  $V_{15}$  = volume during the peak 15 min of the analysis hour (pc/15 min).

The value of PHF for each intersection in the study area is 0.903, 0.904, and 0.911 for Al-Khajiya Intersection, Al-Shuhada Intersection, and Al-Zahra Intersection, respectively.

### 3.2.3 Saturation flow rate

The traffic saturation flow for the study area was calculated according to the procedures outlined in the HCM [7], using data collected from video cameras installed at the selected signalized intersections. This approach ensures reliable, reproducible measurements of saturation flow under real-world traffic conditions. For each signal cycle, the recorded video was analysed to determine the departure times of vehicles during the queue-discharge phase (when the signal turns green and queued vehicles start to move).

The time at which the fourth vehicle crossed the stop line was subtracted from the time at which the final observed vehicle ( $n$ th) crossed the stop line. This provided the total headway time for vehicles 5 through  $n$  in the queue-discharge sequence. Then, the total headway was divided by the number of vehicles considered ( $n-4$ ) to obtain the average headway per vehicle:

$$\text{Average Headway} = (tn - t4) / (n - 4) \quad (2)$$

where,  $tn$  = departure time of the  $n$ th vehicle,  $t4$  = departure time of the 4th vehicle, and  $n$  = total number of vehicles observed in the queue

To ensure the statistical reliability of the results, a minimum of 15 signal cycles were analysed, each having a queue length of at least eight vehicles. This aligns with the HCM [7] recommendations for producing representative and stable estimates. The saturation flow rate ( $S$ ) was calculated as 3,600 (seconds per hour) divided by the average headway ( $h$ ).

Moreover, the smallest time interval in the primary street traffic flow that allows entry of a single secondary street vehicle at a junction is termed the critical gap,  $t_c$ . Consequently, the driver's crucial gap is the minimal permissible interval. A specific driver would dismiss any gaps smaller than the critical gap and take available gaps that are at least as large as the critical distance [7]. The collection

occurred during peak hours. The Critical Gap and Follow-Up Time of the study area are detailed in Table 2.

**Table 2.** Saturation flow rate for study area intersections (pc/hour)

Approach	NB	SB	EB	WB
Al-Khajiya Intersection	1989	1920	1809	2034
Al-Shuhada Intersection	1865	2293	1856	1800

### 3.2.4 Critical gap and follow-up time

The main method for observing the behavior of follow-up time and the critical gap in the study area is to use a video camera at the designated spot to collect the required data. The duration between a specific vehicle exiting the secondary street and the next vehicle occupying the same gap on the primary street during persistent queuing on the secondary street is termed the follow-up time, ft. [7].

The arithmetic mean critical gap was computed from the following Eq. (3):

$$\bar{u} = \frac{\sum u_i f_i}{\sum f_i} \quad (3)$$

The standard deviation was computed using Eq. (4).

$$S = \sqrt{\frac{\sum f_i (u_i - \bar{u})^2}{N - 1}} \quad (4)$$

where,  $\bar{u}$  = The arithmetic means critical gap,  $u_i$  = The recorded gap of vehicle  $i$ ,  $f_i$  = the number of vehicles with a recorded gap of  $u_i$ , and  $S$  = Standard deviation. The results are shown in Tables 3 and 4.

**Table 3.** Critical gap of Al-Zahra Intersection

Approach	Sample Size (veh)	Mean (Sec)	Standard Deviation
North	197	7.5	2.5
South	202	7.8	2.4

**Table 4.** Follow-up time of Al-Zahra Intersection

Approach	Mean (Sec)
North	4.2
South	4.5

### 3.2.5 Speed measurement

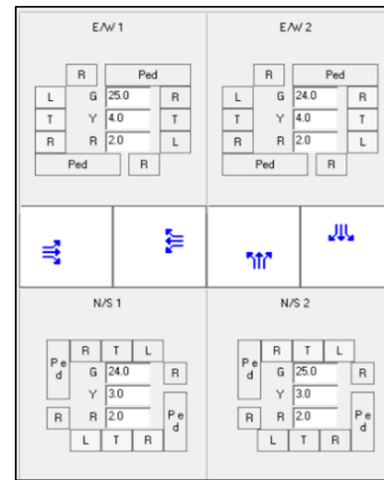
The arithmetic mean speed was calculated as the sum of the vehicles' passing speeds through a specific road section, divided by the total number of vehicles during a specified time period. In this study, the radar method was used to calculate the average speed for each lane for each segment of the facility. The speed was calculated for each segment at one point, in the middle, and away from the acceleration and deceleration areas. The mean speeds are shown in Table 5.

### 3.2.6 Intersection control methods and signal timing data

In the study area, Al-Khajiya and Al-Shuhada intersections operate on regular traffic signals (signalized intersections) that operate with specific traffic cycles, consisting of several stages. In contrast, the Al-Zahra intersection is unsignalized. For signalized intersections, signal timing parameters (cycle length, green time, yellow time) are recorded. Figure 3 shows the phasing design, which consists of four phases; each approach operates as a separate phase.

**Table 5.** The average speed for each lane for each segment of the study area

Segment Name	Speed (km/hr)
30 Al-Khajiya Street (Seg 1)	35
30 Al-Khajiya Street (Seg2)	40
Al-Zahra Commercial Street (Seg3)	30
Al-Zahra Commercial Street (Seg 4)	30



**Figure 3.** Phasing design for Al-Khajiya Intersection

**Table 6.** The number of vehicles that turned RTOR

Segment Name	RTOR (veh/h)			
	East	West	North	South
Intersection Name				
Al-Khajiya Intersection	121	72	87	101
Al-Shuhada Intersection	122	25	50	129

**Table 7.** The available queue storage length (m)

Segment Name	Queue Storage Length			
	East	West	North	South
Intersection Name				
Al-Khajiya Intersection	165	148	197	230
Al-Shuhada Intersection	195	164	230	165

### 3.2.7 Operational factors

A set of important data was collected in analyzing the performance of intersections according to the principles of HCM2010. This data includes:

- The percentage of right turns on red (RTOR), which represents the number of vehicles that turned right at a red light compared to the total number of vehicles going right. The recorded RTORs are shown in Table 6.
- The arrival pattern in the study area was random,
- The pedestrian speed represents the average speed at which pedestrians cross the intersection, which is typically 1.2 m/sec (approximately 4 ft/sec).
- The queue storage length was also determined, which represents the number of vehicles stopped at the beginning of the intersection. The recorded values are shown in Table 7.

### 3.3.8 Geometric data

Before conducting the capacity and operational analysis using HCS 2010 [7] and TransCAD, it was essential to document the geometric features of each segment and intersection included in the study. The geometric layout, such as the lane width, number of lanes, turning movements,

approach configuration, and presence of auxiliary lanes, directly influences the traffic flow and, consequently, the calculated LOS and capacity. All segments are classified as a divided minor arterial road, with three lanes, including a parking lane, and a lane width of 3 m. Tables 8 and 9 summarize the geometric characteristics of the selected intersections, which serve as the input parameters for the analytic tools.

**Table 8.** Geometric characteristics of the intersections in the study area

Intersection	Type of Intersection	Approach	Lane No	Approach Width (m)
Al-Khajiya Intersection	Signalized	NB	2	6
		EB	2	6
		SB	2	6
		WB	2	6
Al-Shuhada Intersection	Signalized	NB	2	6
		EB	2	6
		SB	2	6
		WB	2	6
Al-Zahra Intersection	Unsignalized	NB	3	9
		EB	3	9
		SB	3	9
		WB	3	9

**Table 9.** Length of streets in the study area

Street	Length (km)
30 Al-Khajiya Street (Seg1)	0.643
30 Al-Khajiya Street (Seg 2)	0.792
Al-Zahra Commercial Street (Seg3)	0.765
Al-Zahra Commercial Street (Seg 4)	0.8

### 3.2.9 Traffic accidents data collection

The number of accidents for each intersection and road section was obtained from the Wasit Governorate Traffic Directorate for the period (2023-2024). Table 10 demonstrates the data on accidents collected.

**Table 10.** No. of accidents in each segment in the study area

Street	No. of Accidents (crash/two years)	Average $N_{observed}$
30 Al-Khajiya Street (Seg1)	12	6
30 Al-Khajiya Street (Seg 2)	15	7.5
Al-Zahra Commercial Street (Seg3)	20	10
Al-Zahra Commercial Street (Seg 4)	26	13
Al-Khajiya Intersection	15	7.5
Al-Shuhada Intersection	12	6
Al-Zahra Intersection	18	9

### 3.2.10 On-street parking

Through field surveys, it was noted that there are no legal parking spaces on both sides of the road, but drivers park their vehicles randomly, which leads to a reduction in the width of the lanes and restricts traffic. Therefore, a parking lane was considered present. This phenomenon is one of the common traffic violations in the study area, as it causes an increase in delays and a decrease in the actual capacity of the lanes. These observations were taken into consideration while evaluating

the reliability and safety of the facility (study area).

### 3.2.11 Median island

Medians are considered important engineering elements in urban road design. Their primary function is to separate traffic directions. In the study area, it was noted that there was a difference in the characteristics of medians in the sections that make up the facility. In Al-Zahra Commercial Street, the median was relatively narrow, with a width of 2 m, while in 30 Al-Khajiya Street, the median was wider and more regular, with a width of 5 m.

### 3.2.12 Roadside fixed object

Roadside fixed objects represent any fixed object located on both sides of the road within the potential deviation area for vehicles. These obstacles usually include lampposts, trees, and traffic signs. In this study, fixed obstacles located on the right side of the road in each direction of travel were taken into consideration, and did not include any obstacles located within the median strips.

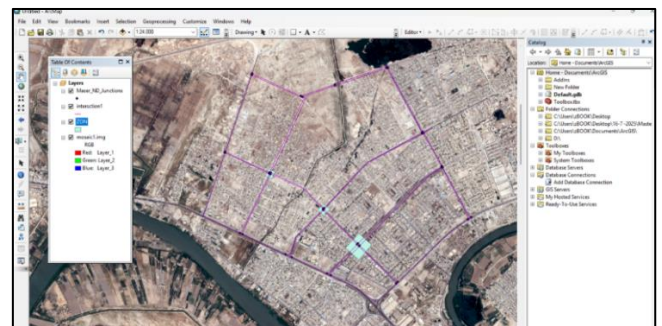
### 3.2.13 Lighting

In the study area, it was noted that the area was equipped with lighting poles along the facility and intersections, while the basic condition in HSM 2010 is the complete absence of road lighting.

## 3.3 Capacity estimation

The capacity estimation in this study was performed using Highway Capacity Software (HCS 2010), which followed the analytical procedures of the HCM 2010. This software allows evaluation of various roadway facilities, including signalized and unsignalized intersections, two-lane highways, and urban streets. The software analyses data and applies an adjustment series factor to account for prevailing conditions such as lane width, grade, parking presence, and heavy vehicle percentage. For signalized intersections, the effective green time and cycle length were incorporated to compute the capacity, volume-to-capacity ratio (v/c), LOS, and average control delay. For unsignalized intersections, the analysis was based on follow-up time and acceptable gap parameters. The results obtained from HCS 2010 were used to calculate the operational capacity of the facility.

The ideal saturation flow was entered as the saturation flow estimated for the discharge headway values, explained in section 3.2.3. Therefore, the adjusted factors are assumed to be 1.0, except for the adjustment factors for heavy vehicles and on-street factors. The reason is to account for the effect of heavy vehicles and parking maneuvers on the capacity of the facility before and after improvement.



**Figure 4.** Nodes and Links of the study area

### 3.4 Network building

This step was essential for estimating the demand of the study area. ArcGIS 10.8 software was used to construct the road network. It was separated into two phases: the initial phase involves network sketching, and the second task entails geometric data extraction and inputting it into the attribute table. The road network in the study area was developed using polylines to create links and points to create nodes, interpreting the intersections as demonstrated in Figure 4.

### 3.5 Traffic assignment model in TransCAD

Traffic assignment models are used to estimate network traffic demand. These models allocate trips to the study area based on a given travel demand, typically represented as a matrix that shows the journey volume between each potential pair of (O-D). The (O-D) matrix delineates the relationship between traffic origins and destinations [59]. Trips are typically planned along the shortest path from origin to destination. The resulting data are presented in a table showing the volume and characteristics of traffic on each network link. This information is crucial for predicting urban travel demand and was used in various applications, such as traffic safety modeling, travel choice analysis, air quality impact assessment, and benefit estimation [59].

Trans CAD (V4.5) necessitates two primary traffic assignment inputs:

- The (O-D) matrix, sometimes referred to as "demand". The subsequent sections provide additional details regarding the O-D matrix papers and the roadway network employed in these processes
- Document for the roadway infrastructure.

Trans CAD software (V4.5) employs a distinctive data structure known as a network file to encapsulate the characteristics, attributes, and spatial configuration of a transportation system. This file deviates from the conventional shape and geographic files employed in most GIS-based investigations, despite its application in several transportation planning contexts, including traffic assignment. The network file was produced from the node and line layers, which were preprocessed using the aforementioned ArcGIS (V10.8) program. The road and node networks were first exported as shapefiles in ArcGIS (V10.8) and then imported into TransCAD (V4.5).

### 3.6 Probability density function

To measure the extent to which the road capacity is reliable to accommodate the traffic demand, Chen et al. [60] introduced the capacity reliability index using probability theory. It can be represented by PDF as follows:

The Supply represents the capacity of the road and can be represented by PDF as

$$Supply\ PDF = g(C1, C2, C3, \dots, Ca) \quad (5)$$

A PDF can represent the Demand for traffic as

$$Demand\ PDF = h(D1, D2, D3, \dots, Db) \quad (6)$$

where,  $C1, C2, C3, \dots, Ca$  = the random capacities,  $D1, D2, D3, \dots, Db$  = The stochastic input variables for estimating

demand.

The overlapping area represents the inadequacy of the system when demand exceeds capacity ( $g(C) < h(D)$ ).

In terms of the probability function, the reliability index can be written as:

$$Reliability\ index = P[M(C, D) \geq 0] \quad (7)$$

where,  $M(C, D) = g(C) - h(D) = reliability\ margin$ . Moreover, represents the probability function.

If  $M(C, D) < 0$ , failure (non-compliance) occurs.

The probability of failure or non-compliance is an estimate of the probability that the road fails to perform its operational performance at the required level of service under anticipated operational conditions. The probability of non-compliance can be determined by drawing the performance distribution function [61].

### 3.7 Traffic safety analytic tool

To evaluate the expected safety performance of the study area, the methodology recommended in the HSM [5] was applied. The HSM adopted 13 performance measures to assess the safety level of any road facility. The LOSS is one of these measures; it was adopted in this study due to its advantages. It is a quantitative interpretation of the level of safety based on quantitative crash analysis. It produces an easy translation of actual crash data to easy-to-understand safety levels. Therefore, it was considered an ideal measure that combines the expected crash frequency (predicted crash frequency)  $N_{predicted}$  with the actual crash frequency  $N_{observed}$ . According to the HSM, the LOSS is classified into four levels shown in Table 11; Level I interprets the best safety level, and LOSS IV interprets the worst safety level, highlighting areas where safety interventions may be most needed. Locations exhibiting inadequate LOSS are identified for additional analysis in the study [5].

**Table 11.** Categories of Level of Service of Safety (LOSS) [5]

LOSS	Condition	Description Indicates
I	$0 < N_{observed} < (N_{predicted} - 1.5 \times (\sigma))$	Low probability of accident reductions
II	$(N_{predicted} - 1.5 \times (\sigma)) \leq N_{observed} < N_{predicted}$	Low to moderate probability for accident reductions
III	$N_{predicted} \leq N_{observed} (N + 1.5 \times (\sigma))$	moderate to significant probability for crash reductions

The required data for estimating the LOSS performance measure are traffic volume and crash data by location. There are five major steps for ranking sites by using the LOSS performance measure, which are as follows:

Step 1: Estimate the predicted crash frequency using the SPF

Step 2: Standard Deviation Calculation

Step 3: Calculation of LOSS category boundaries and comparison of observed crashes with LOSS criteria

Step 4: Sites ranking

#### 3.7.1 Predicted crash frequency using an safety performance function

The HSM methodology (par C) was adopted to estimate the average crash frequency based on traffic properties, geometric

design features, and site type. The predictive model combines an SPF, which represents the expected crash frequency under base conditions, with crash modification factors (CMFs) to account for the effects of real site-specific features. Additional terms are included to capture pedestrian and bicycle-related crashes, while a calibration factor adjusts the results to reflect local conditions [5].

### 3.7.2 Standard deviation calculation

The predicted crashes' standard deviation is determined by adopting Eqs. (3)-(24). This estimation is considered valid because the SPF is based on the assumption that crash counts follow a negative binomial distribution.

$$\sigma = \sqrt{(k \times N_{predicted}^2)} \quad (8)$$

where: *k*: the SPF Over-dispersion parameter (*k* = 0.86, 0.36 for segment and intersection (from HSM 2010). Each SPF also has an associated over-dispersion parameter, *k*; closer to zero indicates higher statistical reliability.

*σ*: Standard deviation.

### 3.7.3 Determining Level of Service of Safety categories, limits, and site ranking

The models in Table 11 are used to find the boundary limits of the four levels of LOSS. The observed crash frequency will then be compared with the limits of the LOSS categories to determine the LOSS.

## 4. RESULTS AND DISCUSSION

### 4.1 Capacity determination

The HCS 2010 program was used to analyze the operational performance of the intersection and determine the operational capacity for each approach at each intersection in the study area. Table 12 shows that the operational capacity varies across intersections and approaches due to differences in geometric design, traffic composition, and road conditions.

The western approach has the highest capacity, while the eastbound direction shows the lowest. Even so, the saturation rate (*v/c*) indicates that the traffic flow rate exceeds the operational capacity. The westbound has the lowest saturation rate; however, it still nearly approaches 1, indicating a near-capacity state. The results of the delay time indicate excessive values at the northbound and southbound, which indicates a

long vehicle queue over multiple cycles. The overall delay corresponds to level of service F, indicating the severe congestion that leads to operational failure due to north-south traffic flow.

**Table 12.** The capacity of each approach at Al-Khajiya Intersection

Intersection Name	Capacity			
	EB*	WB*	NB*	SB
Al-Khajiya Intersection	671	728	700	714
Al-Shuhada Intersection	690	662	699	809
Al-Zahra Intersection	446	630	576	333

\*EB, WB, NB, and SB mean east, west, north, and south bound, respectively. In HCM terminology, bound describes the direction of travel along a roadway segment, while the approach refers to traffic entering an intersection from a specific leg.

At Al-Shuhada Intersection, the southbound approach records the highest capacity, whereas the westbound direction again experiences the lowest. The saturation flow at all approaches exceeds 1.0, indicating a growing queue of created vehicles over multiple cycles. The too-long delay time corresponds to Level of service F, indicating oversaturated traffic flow and severe congestion at all approaches.

Al-Zahra Intersection consistently exhibits lower capacity values across all approaches, indicating a systematic limitation in its ability to accommodate traffic demand.

### 4.2 Estimating the reliability index

Table 13 compares the operational capacity of each road segment, obtained from the HCS analysis, with the estimated demand, derived from TransCAD modeling. The results indicate notable differences between capacity and flow across the segments. The capacity of each corridor segment was determined based on the controlling downstream intersection approach capacity listed in Table 12. For each segment, two travel directions were analyzed (Direction 1 and Direction 2), representing opposite traffic movements along the same roadway.

In Segment 1, the travel demand (617 and 682 veh/h) for both sides of the segment was below the operational capacity (728 veh/h). This indicates that the movement has enough capacity to serve all arriving vehicles within each signal cycle. It also indicates that no residual queue remains after the green phase, and queues grow and dissipate during the green timing.

**Table 13.** The capacity and flow for each segment in the facility

Street Name	Control Intersection/Bound	Capacity**	Demand
30 Al-Khajiya Street (Seg 1, Direction 1*)	Al-Khajiya Intersection/ WB	728	617
30 Al-Khajiya Street (Seg 1, Direction 2*)	Al-Khajiya Intersection /WB	728	682
30 Al-Khajiya Street (Seg 2, Direction 1*)	Al-Khajiya Intersection /EB	671	683
30 Al-Khajiya Street (Seg 2, Direction 2*)	Al-Shuhada Intersection /WB	662	741
Al-Zahra Commercial Street (Seg3, Direction 1*)	Al-Zahra Intersection / WB	630	1363
Al-Zahra Commercial Street (Seg3, Direction 2*)	Al-Shuhada Intersection /EB	690	984
Al-Zahra Commercial Street (Seg 4), Direction 1*	Al-Zahra Intersection / EB	446	995
Al-Zahra Commercial Street (Seg4, Direction 2*)	Al-Zahra Intersection / EB	446	1364
Mean***		625.125	928.625
Standard Deviation***		115.285	302.176

\* Direction 1 and Direction 2 refer to the two opposite directions along the same road segment of the corridor.

\*\* The segment capacities correspond to the controlling downstream intersection approach capacities reported in Table 12.

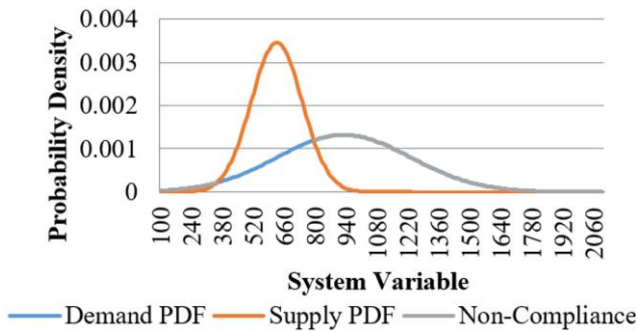
\*\*\*The reported mean and standard deviation summarize variation across corridor segments and are used only for descriptive comparison of demand and capacity levels.

However, the results of segment 2 show a demand of (683 and 741 veh/h), which exceeds its capacity (671 and 662 veh/h) for both sides. Similarly, Segment 3 has a demand of (1363 and 984veh/h) compared to a capacity of (630 and 690 veh/h) for both sides. Segment 4 had traffic demand (995 and 1364 veh/h), which exceeded its capacity on both sides (446 veh/h). These results indicate that these segments are experiencing oversaturation, the intersection cannot process all arriving vehicles, and queues grow continuously.

Overall, the results show that while Segment 1 operates below its design capacity, Segments 2,3, and 4 exceed their capacities. This indicates potential congestion problems and the need for capacity improvements or traffic management strategies in these areas.

The PDF concept has been employed to compare the distribution of traffic demand and available capacity across the analyzed corridor segments. The probability of failure (non-compliance) has been determined by drawing the performance distribution function of the capacity and demand results. Figure 5 shows the distribution of the observed demand values and the corresponding controlling capacities derived from the analysis.

It should be noted that the distributions shown in Figure 5 represent spatial variability across different corridor segments, rather than temporal variability at a single location.



**Figure 5.** Descriptive distribution of observed demand and capacity values across corridor segments used to identify oversaturated locations

It can be seen that the demand curve is flatter and wider compared with the supply curve, indicating a more fluctuated and higher mean value than the supply distribution. This indicates that several corridor segments operate under oversaturated conditions, which was consistent with the demand–capacity comparisons reported in Table 13. In particular, Segments 2, 3, and 4 exhibit demand levels exceeding their capacity, suggesting that these locations act as operational bottlenecks within the corridor.

The non-compliance curve was plotted only when the demand exceeded capacity and represents the failure region. It can also be noted that the demand curve shifts to the right while the supply curve lies to the left, interpreting the wider failure region. This means that the probability of failure was high and the reliability was too small.

The results, therefore, highlight the need for targeted operational and traffic management measures at these specific segments rather than indicating a probabilistic failure rate for the corridor as a whole.

Eq. (9) is used to estimate the reliability index based on the results of the estimated capacity and demand of the study area.

$$Z = \frac{M_C - M_D}{\sqrt{S_C^2 + S_D^2}} \quad (9)$$

where,  $M_C$  = Mean Capacity,  $M_D$  = Mean Demand,  $S_C$  Standard deviation of Capacity,  $S_D$  = Standard deviation of Demand. By substituting the values of means and standard deviation (from Table 13),  $Z = -0.9384 \approx -0.94$ .

According to Melchers and Beck [62], a Z value less than 1.5 was considered unacceptable with an unreliable system, as shown in Table 14.

**Table 14.** Interpretation of Z values

Z Value	Interpretation
> 3.0	Reliable
2.0 - < 3.0	Acceptable
1.5 - < 2.0	At Margins
≤ 1.5	Unreliable

This means that capacity was 0,94 standard deviation below demand, and the probability of reliability can be obtained from the Z-Table was 17.36%. This corresponds to a probability of failure (non-compliance) = 83%, meaning there was an 83% probability that demand exceeds capacity. As a result, the traffic capacity was unreliable, and improvements should be proposed to enhance capacity.

### 4.3 Safety level

The LOSS categories have been identified according to the boundaries shown in Table 11. For example, the predicted crash frequency ( $N_{\text{predicted}}$ ) obtained from applying the HSM methodology was 2.8 crashes/year for segment 2. Accordingly, the results of applying the boundary formulas in Table 11 are obtained. The predicted crashes' standard deviation was determined by substituting k: 0.86in Eq. (8), as 2.6. Table 15 shows the results by comparing the actual observed crash frequency (7.5 crashes/year) with the boundaries of the LOSS categories of 30 Al-Khajiya Street.

**Table 15.** Categories of Level of Service of Safety (LOSS) for 30 Al-Khajiya Street (Seg. 2)

LOSS	Boundaries (from Table 11)	Condition
I	$0 < N_{\text{observed}} < (2.8 - 1.5 \times (2.6))$	$0 > N_{\text{observed}} > -1.1$
II	$(2.8 - 1.5 \times (2.6)) \leq N_{\text{observed}} < 2.8$	$0 \leq N_{\text{observed}} < 2.8$
III	$2.8 \leq N_{\text{observed}} \leq (2.8 + 1.5 \times (2.6))$	$2.8 \leq N_{\text{observed}} \leq 6.7$
IV	$N_{\text{observed}} \geq (2.8 + 1.5 \times (2.6))$	$N_{\text{observed}} \geq 6.7$ $7.5 > 6.7$

The segment is at LOSS IV, indicating a high probability of collision reductions. The previous steps are repeated for all segments in the facility, and the details are shown in Table 16.

**Table 16.** Segment results of the Level of Service of Safety (LOSS)

Segment Name	LOSS
30 Al-Khajiya Street (Seg1)	IV
30 Al-Khajiya Street (Seg 2)	IV
Al-Zahra Commercial Street (Seg 3)	IV
Al-Zahra Commercial Street (Seg 4)	IV

According to the results in Table 16, all four segments fall under LOSS IV, indicating a high-risk level. This classification suggests that the observed number of crashes significantly exceeds the expected level, highlighting safety deficiencies along the facility.

The same procedure was also applied to intersections in the study area by substituting  $N_{\text{predicted}}$  with 4.64 crashes/year, obtained from applying the HSM methodology. The values of the crash standard deviation ( $\sigma$ ) were obtained as 2.784. Table 17 shows the boundaries of the LOSS categories of the Al-Khajiya intersection. The results of the remaining intersections are shown in Table 18.

**Table 17.** Categories of Level of Service of Safety (LOSS) for 30 Al-Khajiya Street

LOSS	Boundaries (from Table 11)	Condition
I	$0 < N_{\text{observed}} < 4.64 - 1.5 \times (2.784)$	$0 < N_{\text{observed}} < 0.44$
II	$(4.64 - 1.5 \times (2.784)) \leq N_{\text{observed}} < 4.64$	$0.44 \leq N_{\text{observed}} < 4.64$
III	$4.64 \leq N_{\text{observed}} \leq (4.64 + 1.5 \times (2.784))$	$4.64 \leq N_{\text{observed}} < (8.84)$
IV	$N_{\text{observed}} \geq (4.64 + 1.5 \times (2.784))$	$N_{\text{observed}} \geq 8.84$

**Table 18.** Intersection results of the Level of Service of Safety (LOSS)

Intersections	LOSS
Al-Khajiya	III
Al-Shuhada	III
Al-Zahra	IV

The results show that the observed crash frequencies at these intersections are significantly higher than the expected values, pointing to notable safety issues. These imply that all segments and intersections require immediate attention and safety interventions. Countermeasures should be considered to reduce collision risks and improve overall roadway safety.

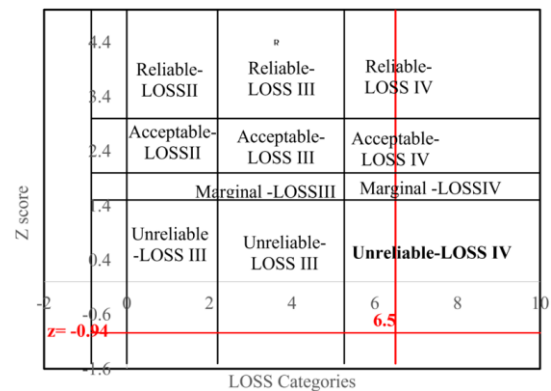
#### 4.4 Reliability-safety matrix

To integrate the reliability index results with the LOSS, the four-quadrant concept has been used. It was widely used to combine two indicator classes into quadrants and to inform decision-making. The matrix contains two axes. The horizontal axis divides the matrix into the upper two quadrants representing positive evaluation and the lower two quadrants representing negative evaluation, according to the scale of the vertical axis. While the vertical axis divides the matrix into two quadrants, the left two quadrants represent the positive evaluation, and the right two quadrants represent the negative evaluation according to the scale of the vertical axis. Usually, Quadrant 1, in the upper-left corner, represents the ideal situation in which both indicators are positive and at their highest levels. The lower-left quadrant is Quadrant 4, which represents the worst situation [63].

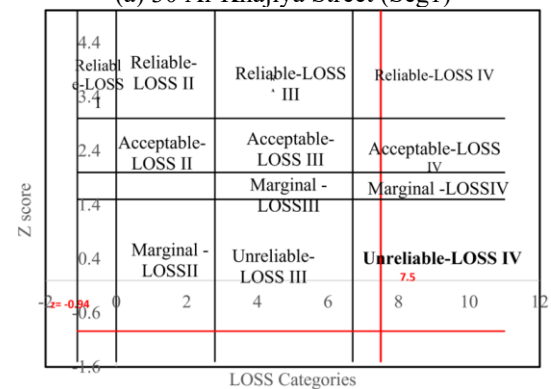
To employ this concept, a reliability-safety matrix was created as a multi-zone performance matrix. The vertical axis was scaled according to the lowest and highest limits of the Z score, which are shown in Table 14. Horizontal lines are drawn from the boundary values of Z-scores, dividing the matrix into four rows: Reliable, Acceptable, Margins, and Unreliable. The horizontal axis represents the scale of the LOSS, dividing the

matrix into four columns: LOSS I, LOSS II, LOSS III, and LOSS IV. The proposed matrix for segments and intersections is shown in Figure 6.

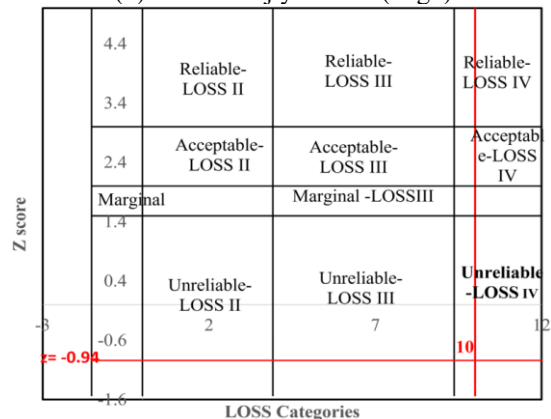
In the matrix of 30 Al-Khajiya Street (Seg1), the first zone, in the upper-left, represents the ideal situation. It was characterized by a z-score greater than 3, representing the highest level of reliability. It also has a higher level of safety. At the same time, the lowest-right zone represents the worst case, with a Z score less than 1.5 (unreliable) and LOSS greater than 6.7 (the highest risk level). By marking the Z-score (-0.94) and the observed crash frequency (6.5 crashes/year) of the case study in the proposed matrix by red lines, it can be seen that the reliability-safety zone of the case study is LOSS IV-unreliable. This indicates high travel demand relative to the operational capacity and a high risk level. All segments have the same rank: LOSSIV (unreliable). In contrast, intersections have a slightly lower risk level (LOSS III-Unreliable) for Al-Khajiya and Al-Shuhada Intersections and (LOSS IV-Unreliable) for Al-Zahra Intersection. Therefore, improvements are required to enhance the reliability and safety of the study area.



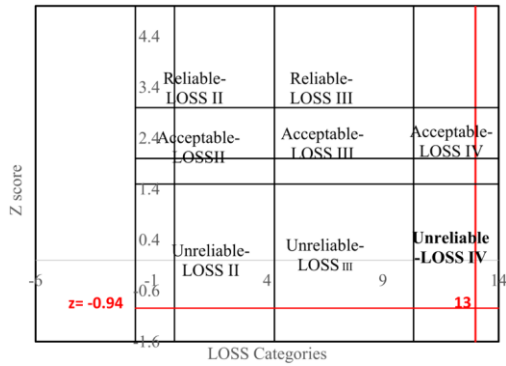
(a) 30 Al-Khajiya Street (Seg1)



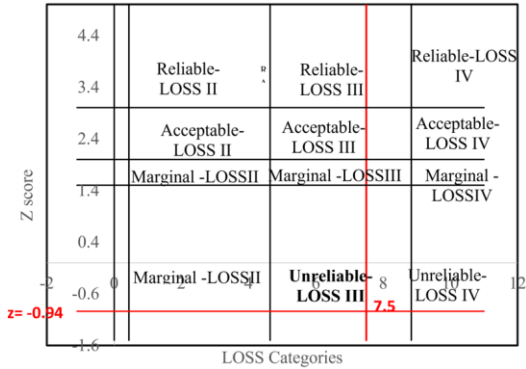
(b) 30 Al-Khajiya Street (Seg2)



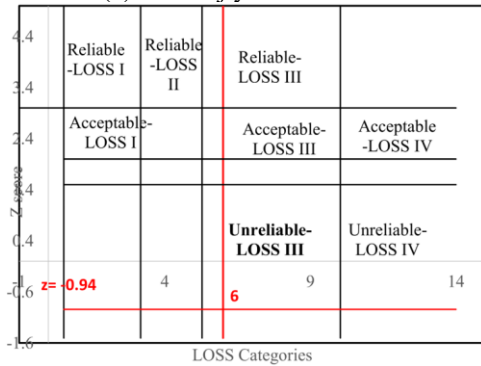
(c) Al-Zahra Commercial Street (Seg 3)



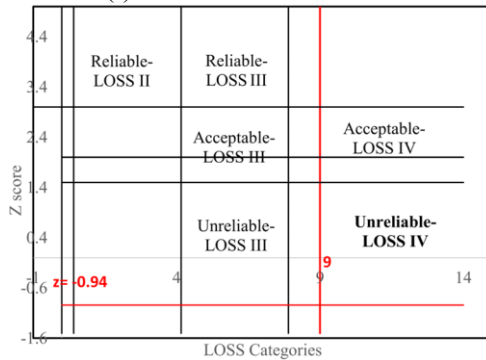
(d) Al-Zahra Commercial Street (Seg 4)



(e) Al-Khajiya Intersection



(f) Al-Shuhada Intersection



(g) Al-Zahra Intersection

Figure 6. Reliability-safety multi-zone matrix

#### 4.5 Suggested improvements

To enhance the reliability and safety in the segments and intersections, proposals for improvement are recommended. They have been suggested to enhance capacity and mitigate the probability of crash occurrence.

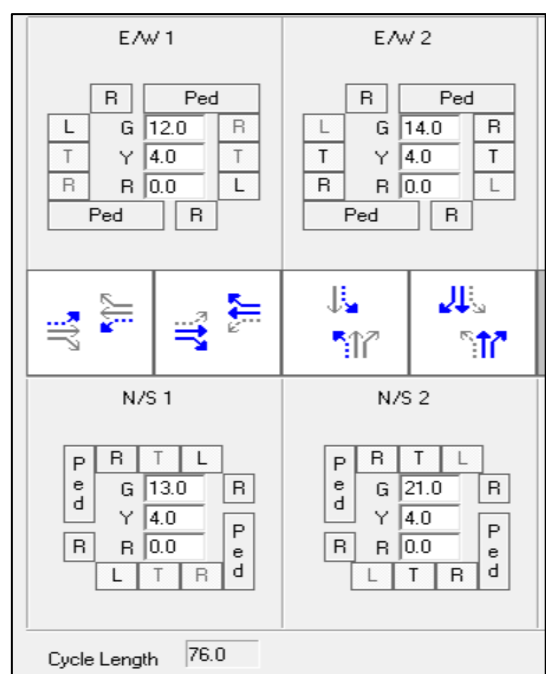
##### 4.5.1 Proposals for increasing capacity

To increase the operational capacity of the study area, the contributing factors and the field current characteristics are

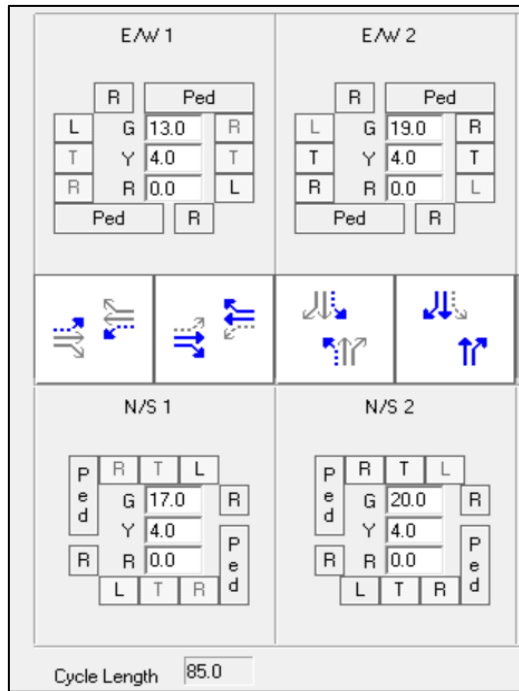
considered. The proposal includes:

- Increase the lane width of road segments to 12 ft.
- Prohibiting parking on both sides of the road by providing traffic signs and imposing fines on violators.
- Prohibiting the traffic of heavy vehicles at peak hour, especially the rubbish collection trucks, which cause interruptions in the traffic of other vehicles.
- Converting the traffic control of Al-Zahra to a signalized intersection.
- Optimizing and coordinating signal timing of the three intersections by employing the planning option in the HCS. Rephasing has been suggested to allocate an exclusive phase for left turns and an exclusive phase for through+right movements. It was also suggested to reduce the green time to 76 sec for AlKhajia Intersection and to 85 sec for Al-Shuhada section, as shown in Figure 7. The cycle length of Al-Zahra Intersection was 72 sec.

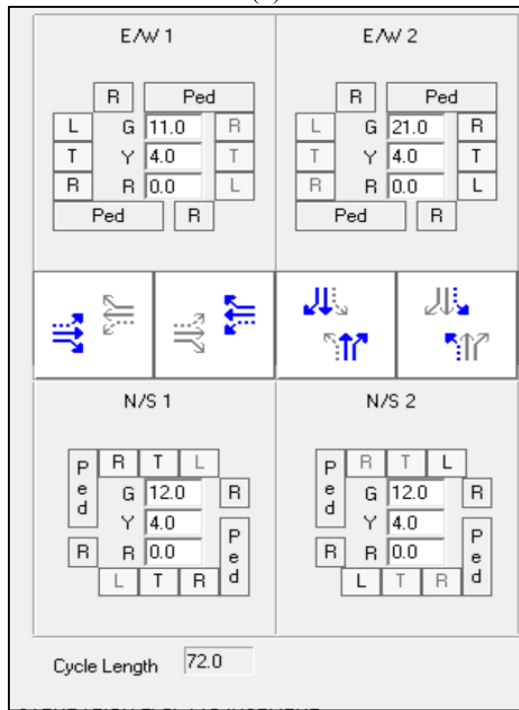
The above proposals have been considered to repeat the HCS and Safety analysis of the study area. Figure 8 shows a comparison of the capacity results before and after improvements. In contrast, Table 19 shows the rate of increase in the operational capacity after improvements at the three intersections. The results show a significant increase in the capacity at the three intersections due to applying the suggested improvements. For example, the capacity at Al-Khajiya intersection increases from 617-728 vph to 1109-1176 vph with an increase rate of 39.1% to 40.2%. This indicates that the proposed improvements contribute effectively to lower saturation levels and can accommodate the current traffic demand. The significant increase in capacity was also noticed at Al-Shuhada Intersection, with a range of rates between 37.5 and 40%. Despite that, it is not sufficient to accommodate the current traffic demand. Al-Zahra Intersection has the most significant improvement in the capacity, reaching a range of rates of 62.6 % to 79.6%, exceeding the rate of increase at other intersections by significant values.



(a)

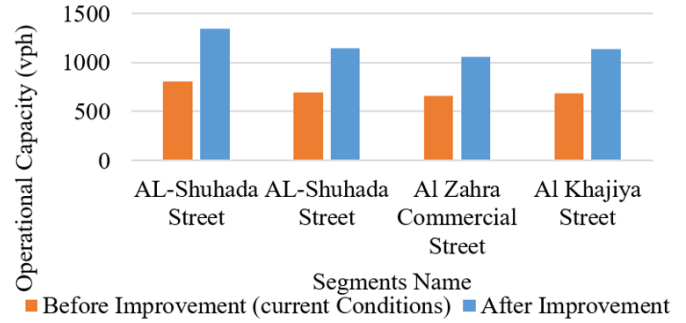
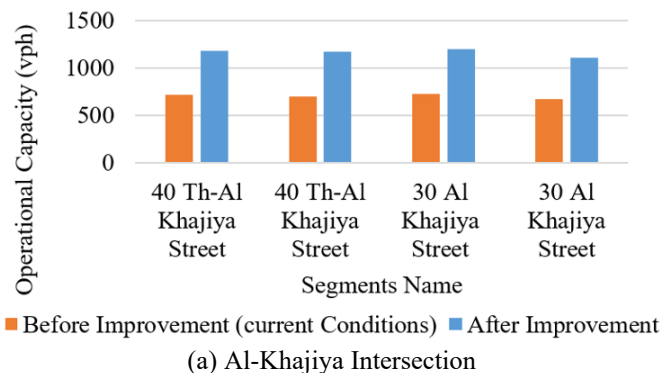


(b)

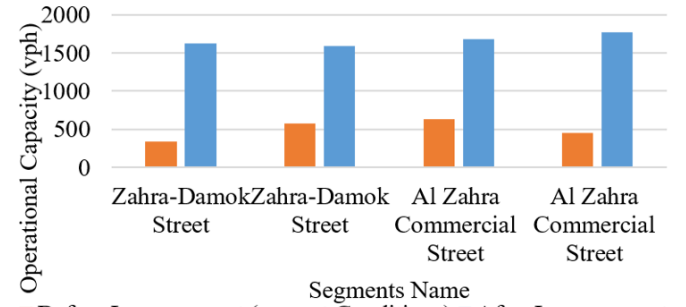


(c)

**Figure 7.** The suggested signal timing and phasing for (a). Al-Khajiya Intersection (b). Al-Shuhada Intersection (c). Al-Zahra Intersection



(b) Al-Shuhada Intersection



(c) Al-Zahra Intersection

**Figure 8.** The estimated operational capacity of the study area before and after improvements

Figure 9 illustrates the probabilistic relationship between traffic demand from TransCAD software and supply (capacity) from HCS analysis after improvements, using PDFs.

**Table 19.** The rate of increase in the operational capacity

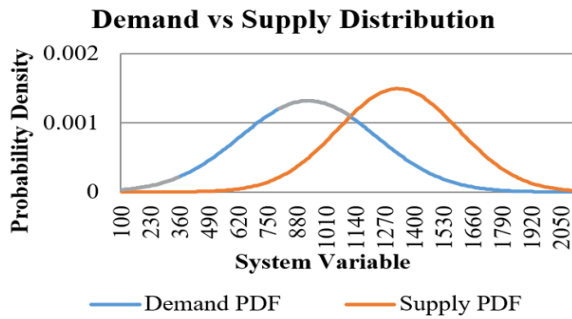
Intersection Name	Rate of Increase (%)			
	EB	WB	NB	SB
Al-Khajiya Intersection	39.4%	39.1%	40.2%	39.3%
Al-Shuhada Intersection	39.3%	37.5%	38.8%	40%
Al-Zahra Intersection	74.8%	62.6%	63.7%	79.6%

It can be noticed that the capacity PDF has been shifted to the right, indicating that the capacity mean was higher than the demand means. This interprets an overall enhanced operating condition, with partial overlap between the two distributions of capacity and demand, and a probability of failure (non-compliance). This may lead to a traffic queue forming and increased delay. Compared with the results of the PDF before improvements shown in Figure 5, the failure rate has been significantly reduced from 83% to 15.6%, and the reliability index has increased from 17.36% to 82.64%. However, according to Melchers and Beck [62] and the Z-score categories shown in Table 14, the Z score remains less than 1.5, indicating an unreliable situation.

The interesting finding of the data is that, while taking into account possible operational enhancements, demand levels in a number of corridor segments still approach or surpass available capacity. This implies that congestion issues along the study corridor might not be entirely resolved by geometric or capacity-based enhancements alone. In actuality, these circumstances could show up as persistent lines at junctions, protracted delays during busy times, and erratic traffic flow.

Therefore, in addition to localized capacity improvements, broader traffic management strategies may be required. These may include demand management measures, improved public

transport services, traffic signal coordination, and travel demand redistribution. Considering these complementary approaches is particularly important in rapidly growing urban areas where traffic demand may continue to increase over time.



**Figure 9.** Descriptive distribution of demand and capacity values across corridor segments is used to identify oversaturated locations after capacity improvement

#### 4.5.2 Proposals for safety enhancement

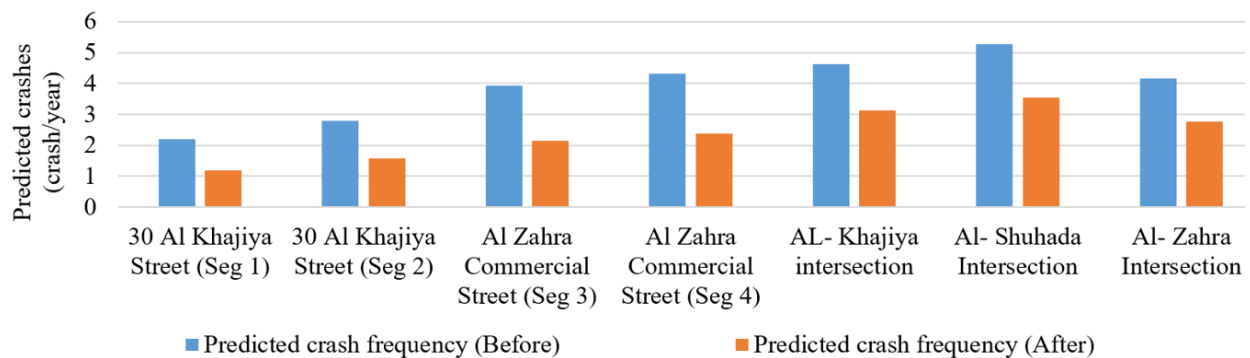
Field observations along the corridor indicated several localized factors contributing to safety risks. To enhance the safety level of the study area, these factors were considered to suggest improvement:

- Roadside encroachments such as informal parking, street vending activities, and pedestrian gathering near commercial frontages were observed at several segments. These are leading to reduced effective lane width and increased vehicle–pedestrian conflict points. Removing encroachments and obstacles on the median and sidewalks on both sides of the road. Sidewalk

encroachments by shop owners and other obstacles reduce pedestrian space, forcing individuals into the roadway and increasing vehicular–pedestrian conflicts. Additionally, the excessive width of medians and sidewalks, combined with their misuse, results in inefficient roadway space allocation.

- Some midblock segments showed comparatively high operating speeds, which almost resulted from limited enforcement. Installing automated speed enforcement systems may assist in reducing excessive speeding and improving adherence to posted speed limits. Such actions are expected to enhance safety performance in higher-traffic-volume and pedestrian-activity segments. This countermeasure could reduce the probability of crashes by 6% [5].
- Enhancing facilities for pedestrians and cyclists, such as marked crosswalks, pedestrian refuge islands, and protected bicycle lanes.

The before-and-after approach used by HSM [5] has been used to assess the efficacy of the recommended countermeasure. Using the HSM approach outlined in Chapter Three, the anticipated crash frequency under the current conditions and after implementing the recommended countermeasures has been calculated. The projected crash frequency before and after improvements is summarized in Figure 10. After implementing the recommended changes, the data show a considerable decrease in the expected crash frequency, which ranges from 32.7% to 33.6% at intersections and from 43.9% to 45.5% at segments. These decreases attest to the suggested countermeasures' efficacy in reducing collisions and generating appreciable safety gains at road segments and intersections.



**Figure 10.** The results of the predicted crash frequency before and after improvements

## 5. CONCLUSIONS

This study examined the operational and safety conditions of a 3-km urban corridor in Al-Kut City, Iraq, connecting three major signalized intersections. By combining traffic demand estimation, intersection capacity analysis, and safety performance evaluation, the study explored the potential of integrating operational reliability assessment with safety analysis to diagnose corridor-level transportation problems.

The results indicate that while certain segments of the corridor operate within their available capacity, several locations experience demand levels that exceed the controlling intersection capacities, leading to recurring congestion and

operational instability. These findings suggest that geometric or capacity-based improvements alone may not fully address corridor performance issues when traffic demand remains high.

An important implication of the results is that segments experiencing frequent demand–capacity exceedance are likely to exhibit unstable traffic flow conditions, including stop-and-go waves, extended queues at intersections, and increased delay variability. Such operational instability can also contribute to elevated safety risks by increasing rear-end conflicts, abrupt braking events, and driver frustration during peak periods. Therefore, reliability and safety should not be treated as independent performance dimensions. Locations

identified as both operational bottlenecks and safety concern areas require integrated interventions that address traffic demand, intersection control, and roadside conditions simultaneously. This finding highlights the value of combining reliability-based operational assessment with safety performance evaluation when diagnosing corridor-level transportation problems.

The integrated reliability–safety framework proposed in this study provides a structured approach for identifying locations where operational bottlenecks and safety concerns may occur simultaneously. Such an approach can support more informed decision-making by highlighting the interaction between traffic flow conditions and safety performance.

However, the present analysis also has limitations. In particular, the demand–capacity comparisons are based on single-period traffic observations rather than time-series data, which limits the ability to capture temporal demand variability and corridor reliability dynamics fully. Future research could extend the proposed framework by incorporating time-series traffic data and probabilistic demand modeling to assess uncertainty in corridor performance more accurately.

Overall, the study demonstrates the potential value of combining operational and safety analyses for urban corridor evaluation while highlighting the importance of considering both capacity improvements and broader traffic management strategies when addressing congestion and safety challenges.

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