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Comparative Analysis of Roundabouts and Traffic Signals for Unsignalized Intersection Treatment (Case Study: Wadung Asri–Brebek–Gedong Masjid Intersection, Sidoarjo)

Izzul Muhamad Hasyir Rachman.¹, Nugroho Utomo² and Achmad Dzulfikar Alfiansyah³

^{1,2,3} Civil Engineering Department, Faculty of Science and Technology; Universitas Pembangunan Nasional “Veteran” Jawa Timur

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CORRESPONDING AUTHOR

*E-mail: izzul.muhammadhr@gmail.com

A B S T R A C T

Unsignalized intersections are vulnerable to performance degradation as traffic volume increases, particularly at the Wadung Asri–Brebek–Gedong Masjid Intersection in Sidoarjo Regency. This study aims to analyze the intersection performance based on the 2023 Indonesian Highway Capacity Guidelines (PKJI 2023) and compare the effectiveness of a roundabout and a signalized intersection (APILL) as alternative treatments under projected traffic growth. The research method involved manual traffic surveys conducted over four days during peak hours, including traffic volume and intersection geometric data. The results show that the existing condition has a degree of saturation (DS) of 0.88 with an average delay of 15.16 s/SMP. Based on the traffic projection for the design year of 2030, the DS increases to 1.11, with an average delay of 26.08 s/SMP, indicating that the intersection does not meet the PKJI 2023 standard (DS < 0.85). The first treatment involved geometric widening of each intersection approach and designing a roundabout, resulting in a DS of 1.07 and an average delay of 52.00 s/SMP. The second treatment involved geometric widening of each intersection approach and the design of a four-phase signalized intersection, resulting in a DS of 0.79 and an average delay of 51.80 s/SMP. Therefore, geometric widening combined with a four-phase signalized intersection is recommended as the preferred treatment alternative because it achieves a DS that meets the PKJI 2023 standard, with a relatively comparable delay to the roundabout alternative.

Contribution to Sustainable Development Goals (SDGs):

SDG 9 : Sustainable Transportation Infrastructure

SDG 11 : Sustainable Urban Mobility

SDG 13 : Climate Action

1. INTRODUCTION

1.1. Research Background

As traffic demand continues to increase, selecting an appropriate intersection treatment becomes increasingly important to maintain intersection performance throughout the design period. Existing traffic conditions may not adequately represent future capacity requirements, particularly at intersections experiencing

continuous traffic growth [1]. An improvement alternative that performs adequately under current conditions may see performance decline as traffic demand increases. Therefore, evaluating intersection improvement alternatives must consider projected traffic conditions over the design period. [2].

Among the available intersection treatments, roundabouts and signalized intersections have different traffic control characteristics and movement patterns [3]. These differences may result in varying capacity, degree of saturation, delay, and queue performance when applied under the same traffic conditions.



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However, comparative evaluations of these two alternatives that simultaneously consider traffic growth over the design period and geometric widening remain limited, particularly for unsignalized intersections in areas experiencing high traffic growth. This condition highlights the need to evaluate both alternatives more comprehensively based on future traffic conditions [4], commercial activities, and the presence of schools around the intersection. Previous studies have shown that roundabouts and signalized intersections can improve intersection performance under various traffic conditions [5]. However, comparative evaluations that consider projected traffic growth and geometric widening simultaneously remain limited. Therefore, this study compares roundabout and signalized intersection alternatives using projected traffic demand up to 2030, while considering geometric widening to determine the most effective treatment for future traffic conditions..

1.2. Literature Review

Intersections are locations where vehicles from different road segments meet and move in various directions [6]. In principle, an intersection is an area consisting of several road approaches that converge at a single point, where roads and vehicle paths intersect [7]. Therefore, traffic management is crucial to ensuring smooth and safe traffic movement at intersections.

1.2.1. Unsignalized Intersection

unsignalized intersection is an intersection without traffic signal devices. Traffic management at such intersections relies on road markings, traffic signs, and other traffic regulations to establish priority and regulate the movement of vehicles passing through the intersection [8].

1.2.2. Signalized Intersection

A signalized intersection is an intersection equipped with traffic signals to regulate the movement of traffic flows from each approach [9]. Traffic signals function to control the flow of vehicles entering the intersection from each approach. The purpose of implementing traffic signals is to maintain intersection capacity during peak hours and reduce the risk of accidents caused by conflicts among traffic flows entering the intersection [10].

1.2.3. Roundabout Weaving Section

A roundabout is a type of intersection that features a circular traffic island at its center, which serves as the center of vehicle circulation. Traffic flows continuously around the central island, with priority given to vehicles already circulating within the roundabout. Therefore, traffic movements can be regulated without the use of traffic signals [11].

The method for determining weaving capacity is empirical; therefore, its application should be limited to the range of variation of the empirical data variables used to formulate the capacity equations in this guideline Indonesian Highway Capacity Guidelines (PKJI) 2023.

1.2.4. Traffic Volume

Traffic volume is an important parameter in traffic engineering that represents the number of vehicles passing through a specific observation point on a road cross-section over a given period of time [12].

1.2.5. Intersection Capacity (C)

Intersection capacity is the ability of an intersection to accommodate and serve the maximum traffic flow that can pass through it under specific conditions within a given period. Intersection capacity is expressed in passenger car units per hour (SMP/h) and is influenced by geometric conditions, traffic characteristics, and operational control at the intersection.

1.2.6. Degree of Saturation (DS)

The degree of saturation is a parameter indicating the level of intersection capacity utilization based on the ratio of traffic flow to intersection capacity (C) [13]. The degree of saturation is calculated using the following equation.

$$Ds = \frac{Q}{C}$$

Infomation:

DS : Degree of Saturation

Q : Capacity of unsignalized intersection arm (SMP/hour).

C : Traffic volume (SMP/hour).

1.2.7. Delay

Delay is the additional travel time experienced by vehicles due to road geometric conditions and traffic impedances during movement along a road segment or through an intersection [14].

1.2.8. Queue Probability

Queue probability (QP) is expressed as a percentage representing the range of possible queue occurrences based on the degree of saturation (DS). The QP value consists of lower and upper limits obtained through specific equations. In addition, queue probability can be determined from the relationship between QP and DS, where the graph represents the range of probabilities and the boundary lines indicate the minimum and maximum queue probability values [6].

1.3. Research Objective

This study aims to analyze the performance of the unsignalized intersection of Jalan Wadung Asri–Jalan Brebek–Jalan Gedong Masjid based on the Indonesian Road Capacity Guidelines (PKJI) 2023, considering traffic flow, capacity, degree of saturation, delay, and queue probability. Furthermore, this study evaluates and compares roundabout and signalized intersection alternatives under traffic conditions projected up to 2030, including geometric widening, to determine the most effective treatment alternative for improving intersection performance.

2. MATERIALS AND METHODS

2.1. Research Location

The study was conducted at the unsignalized intersection of Jl. Wadung Asri, Jl. Brebek, and Jl. Gedong Masjid, Sidoarjo Regency, Indonesia. The location was selected due to relatively high traffic volumes, particularly during peak periods, which may result in vehicle queues and increased delays. These conditions indicate the need to evaluate the intersection performance and analyze alternative improvement measures to enhance its

performance and capacity under both existing traffic conditions and projected conditions for the design year.

2.2. Data Collection

The data utilized in this study were classified into two categories, namely primary data and secondary data. The characteristics and sources of each type of data are described in the following sections.

a. Primary Data

Primary data were collected through direct field surveys at the study location over four days, namely Friday, Saturday, Sunday, and Monday, during peak traffic periods. The surveys were conducted during the morning peak period (07:00 to 09:00), the midday peak period (12:00 to 14:00), and the afternoon peak period (16:00 to 18:00). The collected data included traffic volume, vehicle movements, intersection geometric conditions, and side friction. These data served as the basis for evaluating intersection performance and determining appropriate improvement alternatives.

b. Secondary Data

Secondary data were obtained from relevant government agencies and other supporting sources. The data included the 2025 population of Sidoarjo Regency to determine the city-size classification, Average Daily Traffic (ADT) data for Sidoarjo Regency from the Sidoarjo Transportation Agency, and the research location map from Google Maps. These data supported the intersection performance analysis and traffic projection.

2.3. Data Processing Analysis

Data processing and analysis in this study were conducted based on the 2023 Indonesian Road Capacity Guidelines (PKJI 2023) [15]. The analysis evaluated the existing performance of the unsignalized intersection and identified appropriate improvement alternatives. The analysis consisted of the following stages:

- Traffic volume analysis was conducted based on vehicle type, movement direction, and observation period to determine traffic volume in passenger car units (sm/hour).
- Intersection capacity analysis was conducted by considering the basic capacity and adjustment factors based on the geometric and traffic characteristics of the intersection.
- Degree of saturation (DS) analysis was performed by comparing traffic flow (Q) with intersection capacity (C) to evaluate intersection performance.
- Delay and queue probability analysis was conducted to determine the level of delay and potential queue formation at the intersection.
- Traffic projection analysis was conducted to estimate traffic volume and intersection performance in the design year based on the traffic growth rate.
- Improvement alternative analysis was conducted by evaluating traffic signalization, roundabout implementation, and approach widening to improve intersection capacity and performance.

2.4. Traffic Projection and Design Year

This study set the design year at 2030, representing a five-year projection period from the base year. Traffic volume projections were conducted to estimate future traffic demand and

evaluate intersection performance at the end of the projection period. The projection used the logarithmic regression method based on historical traffic volume data from field observations. The logarithmic regression model used in this study is expressed as:

$$Y = a + b \ln(X)$$

Information:

Y = Projected traffic volume (SMP/hour).

X = Observation year.

a = Regression constant.

b = Regression coefficient.

$\ln(X)$ = Natural logarithm of the observation year.

The regression model was developed using historical traffic data to identify the growth trend and project traffic volume for the 2030 design year. The projected volume was then used to evaluate future intersection performance.

2.5. Intersection Improvement Alternative

Based on the intersection performance analysis and traffic projection for the design year, two improvement alternatives were evaluated to improve intersection capacity and performance: roundabout implementation and signalized intersection implementation. Both alternatives considered the existing intersection geometry and projected traffic demand. Each alternative incorporated approach widening to accommodate projected traffic volume and increase intersection capacity.

3. RESULT AND DISCUSSION

3.1. Existing Intersection Performance

The studied intersection is an unsignalized four-arm intersection consisting of north, east, south, and west approaches. The existing geometric condition is presented in Fig. 1. Existing Geometric Condition

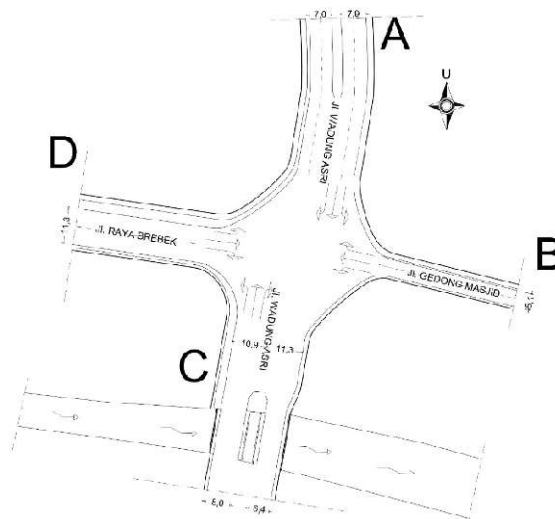


Fig. 1. Existing Intersection Geometry

The existing approach widths are summarized in Table 1..

Table. 1 Existing Geometric Condition

Approach	Approach Width (m)
North	7.90
East	2.55
South	10.90
West	5.65

3.1.1. Traffic Volume Data

Traffic volume surveys were conducted on May 30–31 and June 1–2, 2025, covering the morning, midday, and afternoon periods. The highest traffic volume was recorded on Monday, June 2, 2025, from 16:10 to 17:10, which was identified as the peak-hour condition for subsequent analysis based on PKJI 2023. The observed traffic volumes were converted into passenger car units (SMP/hour) using the vehicle equivalency factors specified in PKJI 2023.

Table. 2 Peak-Hour Traffic Volume by Approach and Movement

Approach Code	Movement	Passenger Cars (MP) Veh/h	Heavy Vehicles (KB) Veh/h	Motorcycles (SM) Veh/h
A (North)	Left Turn	38	6	294
	Through	310	23	2202
	Right Turn	65	9	740
	Total	413	38	3236
B (East)	Left Turn	70	6	321
	Through	52	19	277
	Right Turn	27	10	206
	Total	149	35	804
C (South)	Left Turn	190	27	1177
	Through	261	21	1727
	Right Turn	24	4	174
	Total	475	52	3078
D (West)	Left Turn	173	6	555
	Through	118	7	272
	Right Turn	178	22	1027
	Total	469	35	1854

The observed traffic volumes were converted into passenger car units (SMP/hour) using the vehicle equivalency factors specified in PKJI 2023. The resulting traffic volume of 3586 SMP/h was used to evaluate the existing intersection performance, particularly its capacity and degree of saturation (DS), and to determine the need for intersection improvement..

3.1.2. Existing Intersection Performance

a. Intersection Capacity (C)

Intersection capacity was determined based on the basic capacity and corresponding adjustment factors in accordance with PKJI 2023. The basic intersection capacity was 3,400 SMP/hour. The adjustment factors used in the analysis were:

- Approach width factor (F_{LP}) = 1.110
- Major-road median factor (F_M) = 1.05
- City size factor (F_{UK}) = 1.00
- Side-friction factor (F_{HS}) = 0.92
- Left-turn factor (F_{BK_i}) = 1.300
- Right-turn factor (F_{BK_a}) = 1.00
- Minor-to-total flow ratio factor (F_{Rmi}) = 0.860

Based on these adjustment factors, the intersection capacity was calculated using the following equation:

$$C = C_0 \times F_{LP} \times F_M \times F_{UK} \times F_{HS} \times F_{BK_i} \times F_{BK_a} \times F_{Rmi}$$

$$C = 3400 \times 1.100 \times 1.05 \times 1.00 \times 0.92 \times 1.300 \times 1.00 \times 0.860$$

$$C = 4076 \text{ SMP/hour}$$

Based on the calculation, the intersection capacity was obtained as 4,076 SMP/hour.

b. Degree of Saturation (DS)

The degree of saturation (DS) was calculated by comparing the peak-hour traffic flow (Q) with the intersection capacity (C)

$$DS = \frac{Q}{C}$$

$$DS = \frac{3586}{4076} = 0,88$$

The degree of saturation was 0.88, exceeding the recommended threshold of 0.85 specified in PKJI 2023. This indicates that the intersection is operating under relatively high traffic demand and is approaching its capacity limit.

c. Intersection Delay (T)

Intersection delay was calculated based on the traffic flow, degree of saturation, and relevant delay components in accordance with PKJI 2023. The intersection delay calculation is presented in the following equation.

$$T = T_G + T_{LL}$$

$$T = 4.06 = 11.10$$

$$T = 15.16 \text{ s/SMP}$$

The resulting intersection delay was 15.16 s/SMP, indicating that high traffic demand at the intersection affects traffic operations.

d. Queue Probability

The queue probability (PA) was determined based on the degree of saturation (DS) in accordance with PKJI 2023. The queue probability is calculated using the following equation.

$$PA = 31\% - 61.36\%$$

The resulting queue probability ranged from 31.09% to 61.36%, indicating a considerable likelihood of queue formation during the peak period. Therefore, further evaluation and appropriate intersection improvement alternatives are required.

Based on the analysis of traffic flow, intersection capacity, degree of saturation, delay, and queue probability, the existing unsignalized intersection performance can be assessed comprehensively. The resulting performance parameters are summarized in Table 3.

Table. 3 Existing Intersection Performance

Parameter	Value
Traffic Flow (SMP/hour)	3586
Basic Capacity (SMP/hour)	3400
Intersection Capacity (SMP/hour)	4076
Degree of Saturation (DS)	0.88
Intersection Delay, T (s/SMP)	15.16
Queue Probability, PA (%)	31.09 – 61.36

Overall, the existing intersection has a degree of saturation of 0.88, exceeding the recommended threshold of 0.85, with a delay of 15.16 s/SMP and a queue probability of 31.09%–61.36%. These results indicate that the intersection is approaching saturation and requires improvement. Therefore, the next analysis focuses on traffic growth projection and the evaluation of appropriate improvement alternatives.

3.2. Traffic Projection and Design Year

The design year was determined based on a five-year planning period, with 2025 established as the base year. Therefore, the design year was set as 2030. The base-year traffic volume was used to project future traffic demand throughout the planning period.

3.2.1. Traffic Growth Rate

Traffic volume projection was carried out using logarithmic regression. Based on the historical traffic volume data for 2022–2024, the regression equation for motorcycle (SM) volume was obtained using Microsoft Excel, with 2022, 2023, and 2024 represented as observation periods 1, 2, and 3, respectively. The projected year 2025 was assigned as observation period 4. The resulting logarithmic regression equation is presented as follows:

$$Y = 452621 \ln(x) + 976377$$

Information:

Y = Projected motorcycle volume (veh/h)

x = 4

ln(x) = natural logarithm of x

Substitution of the known values:

$$Y = 452621 \ln(4) + 976377$$

$$Y = 1.603.843$$

The coefficient of determination was obtained as 0.9279, indicating a strong relationship between the observation period and motorcycle volume. The projected motorcycle volume for 2025 was then calculated by substituting into the regression equation, as follows:

$$F = P(1 + i)^n$$

Information:

F = Projected traffic volume in the design year (veh/hr)

P = Current traffic volume (veh/h)

i = Annual traffic growth rate

n = Projection period

Substitution of the known values:

$$1970887 = 1603843(1 + i)^5$$

$$\sqrt[5]{\frac{1970887}{1603843}} = 1 + i$$

$$i = 1,0421 - 1$$

$$i = 0,0421$$

$$i = 4,21\%$$

The same calculation procedure was applied to the other vehicle types, including passenger cars (MP), medium vehicles (KS), and heavy vehicles (KB). For each vehicle type, we first estimated traffic volume using logarithmic regression, then determined the annual growth rate. The resulting growth rates were then used to project the traffic volume for each year from 2025 to the design year of 2030. The projected traffic volumes for all vehicle types are summarized in Table 4.

Table. 4 Projected Traffic Volume by Vehicle Type, 2025–2030

Approach Code	Movement	Passenger Cars (MP) Veh/h	Heavy Vehicles (KB) Veh/h	Motorcycles (SM) Veh/h
A (North)	Left Turn	43	11	358
	Through	357	42	2679
	Right Turn	75	19	901
	Total	475	72	3938
B (East)	Left Turn	80	11	390
	Through	60	38	337
	Right Turn	32	19	251
	Total	172	68	978
C (South)	Left Turn	219	74	1432
	Through	300	59	2102
	Right Turn	29	11	212
	Total	548	144	3746
D (West)	Left Turn	198	11	675
	Through	136	13	331
	Right Turn	204	43	1249
	Total	538	67	2255

Information:

MP = Passenger car

KB = Heavy vehicle

KS = Medium Vehicle

SM = Motorcycle

The projected traffic volumes for the five-year design period (2030) were converted into passenger car units (SMP/hour) using the vehicle equivalency factors for each vehicle type specified in PKJI 2023. The conversion resulted in a total projected traffic volume of 4548 SMP/hour (C).

3.2.2. Future Intersection Performance

The projected traffic volume for 2030 was subsequently used to evaluate the future performance of the existing unsignalized intersection. The intersection capacity was recalculated based on the projected traffic conditions, including the relevant adjustment factors affected by the traffic movement proportions. The capacity was calculated using the following equation:

$$C = C_0 \times F_{LP} \times F_M \times F_{UK} \times F_{HS} \times F_{BK_i} \times F_{BK_a} \times F_{Rmi}$$

$$C = 3400 \times 1100 \times 1.05 \times 1.00 \times 0.92 \times 1.301 \times 1.00 \times 0.860$$

$$C = 4079 \text{ SMP/hour}$$

Based on the recalculation, the intersection capacity for 2030 was obtained as 5282 SMP/hour. The degree of saturation was then determined using the projected traffic volume and the recalculated capacity:

$$Ds = \frac{Q}{C}$$

$$Ds = \frac{4548}{4079} = 1,11$$

The resulting degree of saturation of 1.11 exceeds the recommended threshold of 0.85, indicating that the existing intersection cannot accommodate the projected traffic demand in 2030. Therefore, improvement alternatives are required to improve intersection performance. The intersection delay was calculated in accordance with PKJI 2023 as follows:

$$T = T_G + T_{LL}$$

$$T = 4.00 = 22,08$$

$$T = 26,08 \text{ s/SMP}$$

The resulting intersection delay was 26.08 s/SMP, indicating significant traffic delay during the peak period. The queue probability ranged from 49.81% to 99.78%, indicating a high likelihood of queue formation.

3.3. Roundabout Alternative

The roundabout alternative was proposed to improve traffic flow and reduce conflicts at the existing unsignalized intersection. The design was developed based on the existing geometric conditions, with geometric widening applied to accommodate the projected traffic demand and improve intersection performance.

3.3.1. Roundabout Geometric Design

The first improvement alternative was the conversion of the existing unsignalized intersection into a roundabout. The initial design was developed based on the existing geometric condition, as shown in Figure Fig 2.

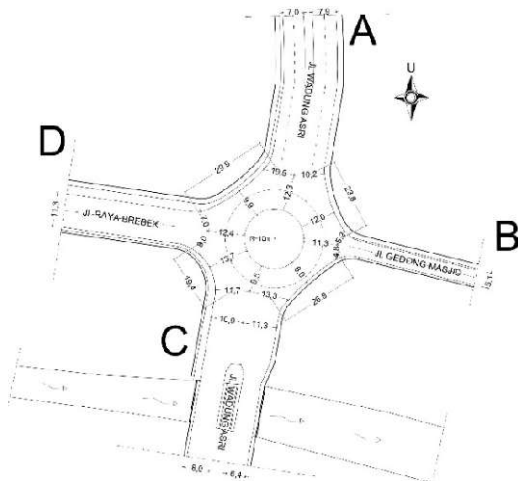


Fig. 2 Existing and Proposed Roundabout Layout

Based on the initial design, geometric widening was then applied to accommodate the projected traffic demand and improve vehicle circulation.

Table. 6 Geometric Parameters of Roundabout Weaving Sections

Section Weaving	Entry Width m		Average Entry Width (W_E) m	Weaving Width (W_W) m	W_E/W_W	Weaving Length (L_W) m	W_W/L_W
	Approach 1	Approach 2					
U-T	11.80	12.30	12.05	12.30	0.98	22.90	0.54
T-S	8.40	9.10	11.30	11.30	1.00	25.60	0.44
S-B	13.30	10.50	16.50	10.50	1.57	27.10	0.39
B-U	10.80	12.40	13.10	12.40	1.06	18.50	0.67

The geometric parameters in Table 6 were used to determine the capacity of each weaving section, which subsequently served as the basis for evaluating the degree of saturation, delay, and queue probability according to PKJI 2023.

3.3.2. Roundabout Capacity Analysis

The projected traffic volumes for the five-year design period (2030), previously presented in vehicle/hour, were converted into passenger car units (SMP/hour) using the vehicle equivalency factors specified in PKJI 2023. The resulting traffic volumes were then used as input for the roundabout capacity analysis. Table 7 presents the projected traffic volumes in SMP/hour.

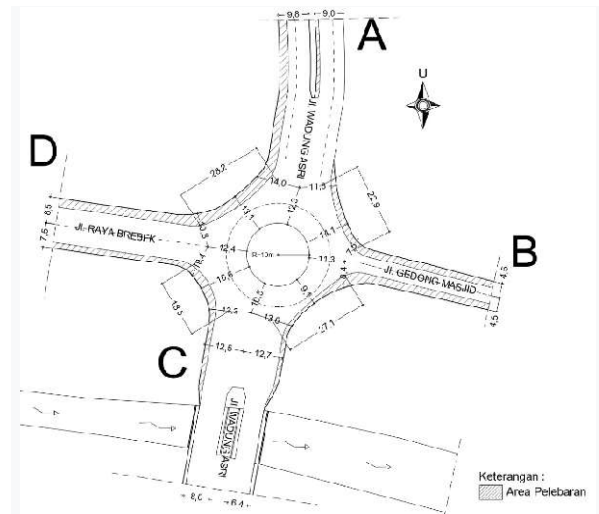


Fig. 3 Proposed Roundabout with Geometric Widening

The changes in approach width before and after widening are presented in Table 5.

Table. 5 Existing and Proposed Approach Widths

Approach	Existing (m)	Proposed (m)	Widening (m)
North	7,90	9,00	1,10
South	2,55	12,50	1,60
East	10,90	4,5	1,95
West	5,65	8,4	2,75

The performance analysis used the final roundabout geometry after widening and the projected traffic demand in the design year. We used the geometric data from Fig. 3 to determine the weaving section parameters presented in Table 6. These parameters were then used to evaluate the capacity, degree of saturation, delay, and queue probability based on PKJI 2023.

Table. 7 Projected Traffic Volume by Approach and Movement in SMP/hour

Approach Code	Movement	Passenger Cars (MP) SMP/h	Heavy Vehicles (KB) SMP/h	Motorcycles (SM) SMP/h
A (North)	Left Turn	43	14	179
	Through	357	55	1340
	Right Turn	75	25	451
	Total	475	94	1969
B (East)	Left Turn	80	14	195
	Through	60	49	169
	Right Turn	32	25	126
	Total	172	88	489

Table. 8 Projected Traffic Volume by Approach and Movement in SMP/hour (Continued)

Approach Code	Movement	Passenger Cars (MP) SMP/h	Heavy Vehicles (KB) SMP/h	Motorcycles (SM) SMP/h
C (South)	Left Turn	219	96	716
	Through	300	77	1051
	Right Turn	29	14	106
	Total	548	187	1873
D (West)	Left Turn	198	14	338
	Through	136	17	166
	Right Turn	204	56	625
	Total	538	87	1128

The converted traffic volumes were subsequently used as input for the roundabout capacity calculation, together with the geometric parameters of each weaving section. As an example, the calculation of the weaving section parameters is presented for the North–South (U–T) weaving section. The calculation includes determining the weaving width factor, entry-to-weaving width ratio factor, and weaving ratio factor, as shown below.

- Weaving width factor (F_{WW})

$$F_{WW} = 135 \times W_W^{1.3}$$

$$F_{WW} = 135 \times 12,30^{1.3}$$

$$F_{WW} = 3525,42$$

Information:

F_{WW} = Weaving width factor

W_W = Weaving width (m)

- Entry-to-Weaving Width Ratio Factor ($F_{WE/WW}$)

$$F_{WE/WW} = (1 + W_E/W_W)^{1.5}$$

$$F_{WE/WW} = (1 + 0,98)^{1.5}$$

$$F_{WE/WW} = 2,79$$

Information:

$F_{WW/WE}$ = Entry-to-weaving width ratio factor

W_W = Weaving width (m)

W_E = Average entry width (m)

- Weaving Ratio Factor (P_W)

$$fP_W = \left(1 - \frac{P_W}{3}\right)^{0.5}$$

$$fP_W = \left(1 - \frac{0,712}{3}\right)^{0.5}$$

$$fP_W = 0,87$$

Information:

FP_W = Weaving ratio factor

P_W = Weaving ratio

- Weaving Width-to-Length Ratio Factor ($F_{WW/LE}$)

$$F_{WW/LE} = \left(1 + \frac{W_W}{L_W}\right)^{-1.8}$$

$$F_{WW/LE} = (1 + 0,54)^{-1.8}$$

$$F_{WW/LE} = 0,46$$

Information:

$F_{WW/LE}$ = weaving width-to-length ratio factor

W_W = Weaving width (m)

L_W = Weaving length (m)

The resulting factors were then used to determine the basic capacity (C_0) of each weaving section. The final capacity (C) was subsequently obtained by applying the city-size and road-environment correction factors.

$$C_0 = (F_{WW}) \times (F_{WE/WW}) \times (P_W) \times (F_{WW/LE})$$

$$C_0 = (3525,24) \times (2,79) \times (0,87) \times (0,46)$$

$$C_0 = 3936$$

Information:

C_0 = Basic Capacity (SMP/hour)

F_{WW} = Weaving width factor

$F_{WE/WW}$ = Entry-to-Weaving Width Ratio Factor

P_W = Weaving Ratio Factor

$F_{WW/LE}$ = Weaving Width-to-Length Ratio Factor

Based on the study area's characteristics, the city-size correction factor (F_{UK}) was set at 1.00, while the road-environment correction factor (F_{RSU}) was set at 0.92. We applied these correction factors to the basic capacity to obtain the final capacity of each weaving section. The capacity calculation is presented as follows:

$$C = C_0 \times F_{UK} \times F_{RSU}$$

$$C = 3936 \times 1 \times 0,92$$

$$C = 3621$$

Information:

C = Capacity (SMP/hour)

C_0 = Basic Capacity (SMP/hour)

F_{UK} = City size correction factor

F_{RSU} = Side Friction Adjustment Factor

The calculated basic and final capacities for each weaving section are summarized in Table X. The results show the capacity of the proposed roundabout for the U–T, T–S, S–B, and B–U weaving sections after applying the corresponding correction factors.

Table. 9 Capacity of Roundabout Weaving Sections

Weaving Section	F_{WW}	$F_{WE/WW}$	FP_W	$F_{WW/LW}$	Basic Capacity	F_{UK}	F_{RSU}	Capacity
					C_0 (SMP/hour)			C (SMP/hour)
U-T	3525.42	2.79	0.87	0.46	3936	1	0.92	3621
T-S	3157.44	2.83	0.86	0.52	3996	1	0.92	3676
S-B	2869.99	4.12	0.88	0.55	5723	1	0.92	5265
B-U	3562.72	2.96	0.86	0.40	3628	1	0.92	3338

3.3.3. Degree of Saturation Analysis

The degree of saturation was subsequently used to determine the traffic delay of each weaving section based on PKJI 2023.

$$Ds = \frac{Q}{C}$$

$$Ds = \frac{3892}{3621}$$

$$Ds = 1,07$$

The degree of saturation was subsequently used to determine the traffic delay of each weaving section based on PKJI 2023. The degree of saturation was then used to determine traffic delay based on PKJI 2023. The delay calculation is presented as follows.

3.3.4. Delay and Queue Analysis

After obtaining the degree of saturation (DS), we calculated the traffic delay and queue probability for each weaving section based on PKJI 2023. These parameters were evaluated to determine the operational conditions of each weaving section and the overall performance of the proposed roundabout.

For $DS \leq 0,60$

$$T_R = 2 + 2,68982Ds - (1 - Ds) \times 2$$

For $DS \geq 0,60$

$$T_R = \frac{1}{0,59186 - 0,52525Ds} - (1 - Ds) \times 2$$

$$T_R = \frac{1}{0,59186 - 0,52525 \cdot 0,07} - (1 - 1,07) \times 2$$

$$T_R = 34$$

Information:

T_R = Average Delay (s/veh)

DS = Degree of Saturation

Q = Traffic Flow (SMP/hour)

C = Intersection Capacity (SMP/hour)

where is the average traffic delay of the weaving section, and is the degree of saturation. The calculated traffic delays for each weaving section were then used to determine the overall traffic delay of the roundabout.

$$T_{LL} = Q \times T_{Ri}$$

$$T_{LL} = 3892 \times 34$$

$$T_{LL} = 132.328$$

Information:

T_{LL} = Average traffic delay of the roundabout (s/SMP)

Q = Traffic Flow (SMP/h)

T_{Ri} = Total traffic flow entering weaving section (SMP/hour)

For the four weaving sections, the following equation was used:

$$T_{LL} = \frac{(q_{U-T} \times T_{RU-T}) + (q_{T-S} \times T_{RT-S}) + (q_{S-B} \times T_{RS-B}) + (q_{B-U} \times T_{RB-U})}{Q_{MASUK}}$$

$$(132.328) + (133.892) + (14.480)$$

$$T_{LL} = \frac{+ (87850)}{7651}$$

$$T_{LL} = 48$$

Information:

T_{LL} :Average delay in the weaving section (s/ veh)

$qu-T$:Traffic flow from approach to approach T (SMP/h)

$qu-S$:Traffic flow from approach U to approach S (SMP/hour)

$qu-B$:Traffic flow from approach S to approach B (SMP/hour)

$qu-U$:Traffic flow from approach B to approach U (SMP/hour)

T_{R-U} :Average delay at approach U (s/ veh)

T_{R-S} :Average delay at approach S (s/ veh)

T_{R-B} :Average delay at approach B (s/veh)

Q_{MASUK} :Total entering traffic flow (SMP/hour)

According to PKJI 2023, the geometric delay (T_G) is set at 4 s/SMP. Therefore, the roundabout delay is calculated as follows:

$$T = T_{LL} + 4$$

$$T = 48 + 4$$

$$T = 52$$

Information:

T : roundabout delay (s/SMP)

T_{LL} : average traffic delay of the roundabout (s/SMP)

where is the roundabout delay (s/SMPS) and T_{LL} is the average traffic delay of the roundabout (s/SMP). The degree of saturation was subsequently used to determine the queue probability for each weaving section based on the PKJI 2023 method. The queue probability is expressed as a range of probabilities representing the likelihood of queue formation.

$$P_{a,lower} = 9,14Ds + 29,967Ds^{4,619}$$

$$P_{a,lower} = 9,14(1,07) + 29,967(1,07)^{4,619}$$

$$P_{a,lower} = 51$$

$$P_{a,upper} = 26,65Ds + 55,55Ds^2 + 108,57Ds^3$$

$$P_{a,upper} = 26,65(1,07) + 55,55(1,07)^2 + 108,57(1,07)^3$$

$$P_{a,upper} = 98$$

Information:

$P_{a,lower}$: lower limit of queue probability (%)

$P_{a,upper}$: upper limit of queue probability (%)

DS : degree of saturation

The calculated degree of saturation, traffic delay, and queue probability were then used to evaluate the roundabout's operational performance under the projected traffic conditions. These performance indicators provide an overview of the level of service and traffic conditions at the weaving sections. The results for each weaving section, together with the overall roundabout performance, are summarized in Table 9.

Table. 10 Performance Analysis Results of the Proposed Roundabout

Movement	Approach Flow Q (SMP/hour)	Degree of Saturation DS	Turning Movement T_{Ri} (s/SMP)	Total Turning Movement (s/hour)	Queue Probability P_a %
U-T	3892	1.07	34	132328	51 – 98
T-S	3938	1.07	34	133892	51 – 98
S-B	3620	0.69	4	14480	12 – 28
B-U	3514	1.05	25	87850	47 – 92
Roundabout Turning Movement, TTL (s/SMP)				48	-
Average Roundabout Turning Movement, T = TTL + 4 (s/SMP)				52	-
Maximum Roundabout Queue Probability, P_a					51 – 98

Table 9 provides an overview of the proposed roundabout's operational performance in the design year. The evaluation of <https://doi.org/10.29165/ajarcde.v10i3.1355>

degree of saturation, traffic delay, and queue probability provides a basis for assessing the roundabout's effectiveness in

accommodating projected traffic demand. These results are subsequently used as a reference for comparison with the signalized intersection alternative, which is discussed in the following section.

3.4. Signalized Intersection Alternative

As the second improvement alternative, the intersection is proposed to be converted into a four-phase signalized intersection to improve traffic control and reduce conflicts between approaches. The design also includes approach widening to accommodate projected traffic demand and improve intersection performance in the design year..

3.4.1. Signalized Intersection Geometric Design

The second improvement alternative involved converting the existing unsignalized intersection into a four-phase signalized intersection to improve traffic control and reduce conflicts between traffic movements. The design was developed based on existing geometric conditions and traffic movements, incorporating the proposed signal configuration to meet operational requirements, as shown in Fig. 4.

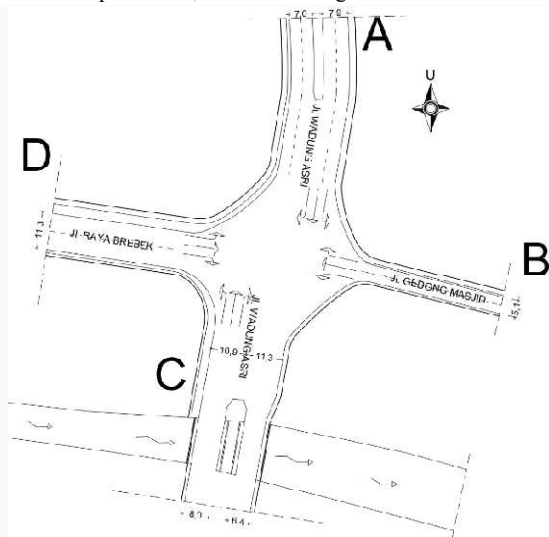


Fig. 4 Existing and Proposed Signalized Intersection Layout

Based on the initial design, geometric widening was applied to approaches requiring additional space to accommodate projected traffic demand and support the operation of the four-phase signal control, as shown in Fig. 5.

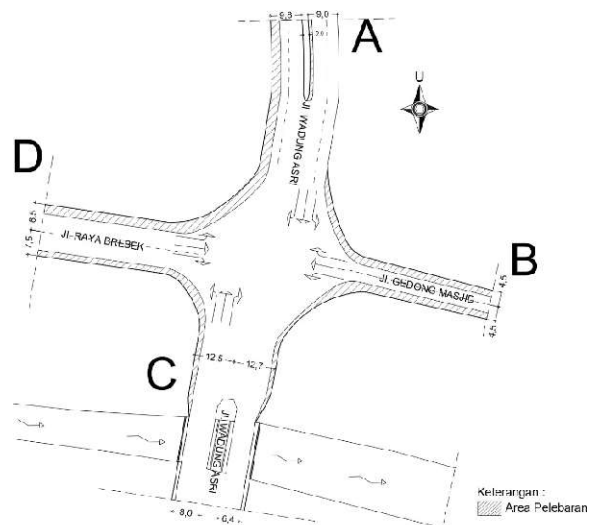


Fig. 5 Proposed Signalized Intersection with Geometric Widening

The proposed geometric widening results in changes to the approach widths of the signalized intersection. A comparison of the existing and proposed geometric conditions is summarized in Table 10.

Table. 11 Geometric Comparison of the Existing and Proposed Signalized Intersection

Approach	Existing (m)	Proposed (m)	Widening (m)	At Stop Line, L_M (m)
North	7,90	9,00	1,10	7,00
South	10,90	12,50	1,60	19,5
East	2,55	4,5	1,95	4,5
West	5,65	8,4	2,75	6,5

Based on the geometric comparison, the proposed widening provides adequate approach widths for the four-phase signalized intersection. The resulting geometry is then used for the subsequent performance analysis.

3.4.2. Signal Timing Design

The projected traffic flow for the design year was subsequently analyzed to determine the distribution of traffic movements at each approach. The traffic volumes were classified according to their respective movements, namely left-turn, through, and right-turn movements, for the North (A), South (B), East (C), and West (D) approaches. This movement distribution forms the basis for determining the traffic flow arrangement and developing the four-phase signal configuration, while also identifying the major traffic movements at each approach. Table 11 presents the classified traffic volumes for each approach and movement.

Traffic flow data obtained from the survey, initially expressed in vehicles per hour (veh/h), were converted into passenger car units per hour (smp/h) using the equivalency values for light vehicles (MP), heavy vehicles (KB), and motorcycles (SM) in accordance with PKJI 2023. The traffic flow was further classified by approach and movement, including left-turn, through, and right-turn movements, with protected and opposed left-turn movements (BKlJT) distinguished based on their operational characteristics and effects on intersection capacity. The resulting adjusted traffic flows were used as input for the

subsequent capacity and performance analysis of the proposed four-phase signalized intersection, as presented in Table 12.

Table. 12 Traffic Flow by Approach and Movement

Approach Code	Movement	Passenger Caars (MP)	Heavy Vehicles (KB)	Motorcycles (SM)
		Veh/h	Veh/h	Veh/h
A (North)	Left Turn	43	11	358
	Through	357	42	2679
	Right Turn	75	19	901
	Total	475	72	3938
B (South)	Left Turn	219	74	1432
	Through	300	59	2102
	Right Turn	29	11	212
	Total	548	144	3746
C (East)	Left Turn	80	11	390
	Through	60	38	337
	Right Turn	32	19	251
	Total	172	68	978
D (West)	Left Turn	198	11	675
	Through	136	13	331
	Right Turn	204	43	1249
	Total	538	67	2255

Table. 13 Traffic Flow by Approach, Vehicle Type, and Movement Type (SMP/h)

Approach Code	Movement	Passenger Cars (MP)		Heavy Vehicles (KB)		Motorcycles (SM)		Total	
		SMP/h		SMP/h		SMP/h		SMP/h	
		Protected	Opposed	Protected	Opposed	Protected	Opposed	Protected	Opposed
A (North)	Left Turn	43	43	14	14	358	54	111	200
	Through	357	357	55	55	2679	402	814	1484
	Right Turn	75	75	25	25	901	135	235	460
	Total	475	475	94	94	3938	591	1160	2144
B (South)	Left Turn	219	219	96	96	1432	215	530	888
	Through	300	300	77	77	2102	315	692	1218
	Right Turn	29	29	14	14	212	32	75	128
	Total	548	548	187	187	3746	562	1297	2234
C (East)	Left Turn	80	80	14	14	390	59	153	250
	Through	60	60	49	49	337	51	160	244
	Right Turn	32	32	25	25	251	38	95	157
	Total	172	172	88	88	978	148	408	651
D (West)	Left Turn	198	198	14	14	675	101	313	482
	Through	136	136	17	17	331	50	203	285
	Right Turn	204	204	56	56	1249	187	447	760
	Total	538	538	87	87	2255	338	963	1527

The basic saturation flow (J_0) was determined based on the effective approach width for each approach. As an example, the calculation is presented for Approach A (North). The calculation was performed using the following equation:

$$J_0 = 600 \times L_M$$

$$J_0 = 600 \times 7,00$$

$$J_0 = 4200 \text{ SMP/hour}$$

Information:

J_0 = Basic saturation flow (SMP/hour);

LE = Effective approach width (m);

600 = Basic saturation flow rate per metre of effective width (SMP/hour/m).

For the adjusted saturation flow (J), the relevant adjustment factors were determined based on the existing intersection

conditions. The adjustment factors used in the analysis are as follows:

- City size adjustment factor (F_{UK}) = 1,00
- Side-friction adjustment factor (F_{HS}) = 0,92
- Parking adjustment factor (F_P) = 1,00

Meanwhile, the right-turn adjustment factor (F_{BKa}) and left-turn adjustment factor (F_{BKi}) were determined based on the traffic flow and movement characteristics of each approach. The adjusted saturation flow (J) was then calculated by incorporating the basic saturation flow and the applicable adjustment factors for city size, side friction, parking, and turning movements, as follows.

$$J = J_0 \times F_{UK} \times F_{HS} \times F_P \times F_{BKi} \times F_{BKa}$$

$$J = 3400 \times 1,00 \times 0,92 \times 1,00 \times 1,0527 \times 0,9847$$

$$J = 4005$$

Information:

J = Adjusted saturation flow (SMP/hour)
 J_0 = Basic saturation flow (SMP/hour)
 F_{UK} = City size adjustment factor
 F_{HS} = Side friction adjustment factor
 F_P = Parking adjustment factor
 F_{BKi} = left-turn adjustment factor
 F_{BKa} : Right-turn adjustment factor.

The all-red time (W_{MS}) was determined based on the distance and speed of vehicles and pedestrians at each approach. The calculated W_{MS} values ranged from 0.32 to 0.96 seconds and were set at 1 second for each phase. With a yellow time of 1 second per phase, the total lost time was 8 seconds per cycle. Given a cycle length of 130 seconds, the effective green time available for the four phases was 122 seconds.

The effective green time was then allocated to the four signal phases based on each phase's critical flow ratio. The green time for each phase was determined using the following equation:

$$W_{Hi} = (s - W_{HH}) \frac{(R_{q/i})_{critical,i}}{\sum (R_{q/j})_{critical}}$$

$$W_{Hi} = (130 - 122) \frac{0,2619}{0,7367}$$

$$W_{Hi} = 34$$

Information:

W_{Hi} = Effective green time for phase (s)
 s = Cycle length (s)
 W_{HH} = Total lost time per cycle (s)
 $(R_{q/i})_{critical,i}$ = Critical flow ratio of phase ;
 $\sum (R_{q/j})_{critical}$ = Sum of critical flow ratios of all phases

With a cycle length of 130 seconds and total lost time of 8 seconds, the available effective green time is 122 seconds. The calculation of saturation flow and green time allocation for each phase is presented in Table 13, with the resulting green time for each phase summarized in the signal timing plan.

Table. 14 Saturation Flow and Green Time Allocation

Approach Code	Effective Width (m)	Basic Saturation Flow (SMP/h)	Adjustment Factor						Adjusted Saturation Flow (SMP/h)	Traffic Flow (SMP/h)	Green Time/Phase
			City Size	Site Friction	Grade	Parking	Right Turn	Left Turn			
	(LE)	(J_0)	(F_{UK})	(F_{HB})	(F_G)	(F_P)	(F_{BKa})	(R_{BKi})	(J)	(q)'	(W_{Hi})
U	8.95	5370	1.00	0.920	1.00	1.00	1.0527	0.9847	5121	1049	33.6232
S	12.20	7320	1.00	0.920	1.00	1.00	1.0150	0.9346	6388	767	19.7152
T	5.10	3060	1.00	0.920	1.00	1.00	1.0605	0.9400	2806	408	23.8754
B	8.30	4980	1.00	0.920	1.00	1.00	1.1833	0.9714	5266	1436	44.7740

Based on the results presented in Table 13, the adjusted saturation flow and green time allocation were obtained for each approach. The available effective green time of 122 seconds was distributed among the four signal phases according to each approach's traffic demand and critical flow ratio. The resulting green times were 33.63 seconds for Approach U, 19.72 seconds for Approach S, 23.88 seconds for Approach T, and 44.77 seconds for Approach

B, with a total effective green time of 122 seconds within the 130-second cycle. The resulting signal timing was then represented through a cycle diagram and a four-phase signal configuration to illustrate the sequence and distribution of green, yellow, and red intervals. The cycle diagram and four-phase signal arrangement are presented in Figures 6 and Figure 7,

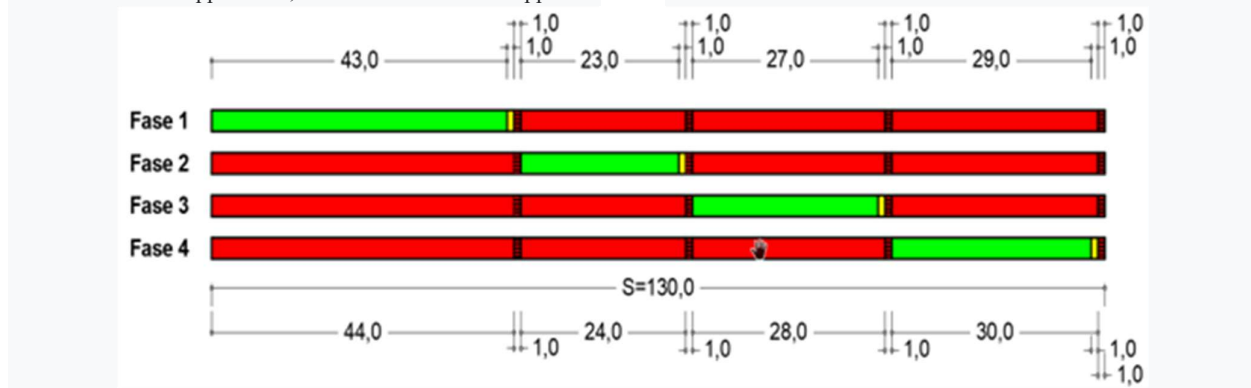


Fig. 6 Signal Cycle Diagram for the Proposed Four-Phase Signalized Intersection

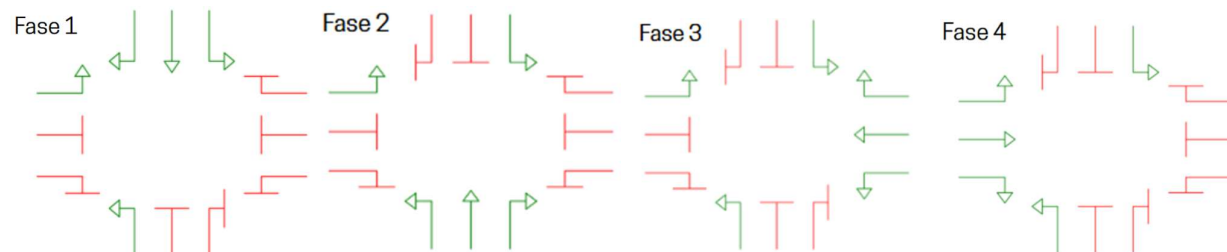


Fig. 7 Four-Phase Signal Configuration for the Proposed Signalized Intersection

3.4.3. Capacity (C)

The capacity of the proposed four-phase signalized intersection was calculated for each approach based on the effective green time and adjusted saturation flow. For illustration purposes, the calculation for Approach A (North) is presented as an example.

$$C = J \times \frac{W_H}{s}$$

$$C = 4005 \times \frac{43}{130}$$

$$C = 1336 \text{ (SMP/hour)}$$

Information:

C = Approach capacity (SMP/hour)
J = Adjusted saturation flow (SMP/hour)
W_H = Effective green time (s)
s = Cycle length (s)

3.4.4. Degree of Saturation (DS)

The degree of saturation for each approach was subsequently determined based on the traffic flow and capacity obtained from the preceding calculation. For illustration purposes, the calculation for Approach A (North) is presented as an example.

$$D_j = \frac{Q}{C}$$

$$D_j = \frac{1049}{1336}$$

$$D_j = 0,79$$

Information:

DS = Degree of saturation;
Q = Traffic flow (SMP/hour)
C = Approach capacity (SMP/hour)

3.4.5. Queue Length (m)

Queue length was determined based on the total number of queued vehicles and the effective approach width at the stop line, using an average queue area of 20 m²/SMP. The total number of queued vehicles (NQ) was determined from NQ₁ and NQ₂ according to PKJI 2023. As an example, the calculation for Approach A (North) is presented below.

NQ₁ calculation:

$$NQ_1 = 0.25s \left[(DS - 1) + \sqrt{(DS - 1)^2 + \frac{8(DS - 0.5)}{s}} \right]$$

$$NQ_1 = 0.25(130) \left[(0.79 - 1) + \sqrt{(0.79 - 1)^2 + \frac{8(0.79 - 0.5)}{130}} \right]$$

$$NQ_1 = 0.580$$

Information:

NQ₁ = Number of vehicles in queue due to uniform arrivals.
S = Signal cycle time (sec)
DS = Degree of saturation

NQ₂ Calculation:

$$NQ_2 = s \times \frac{1 - R_H}{1 - (R_H \times DS)} \times \frac{Q}{3600}$$

$$NQ_2 = 130 \times \frac{1 - 0.3340}{1 - (0.3340 \times 0.79)} \times \frac{1049}{3600}$$

$$NQ_2 = 34.271$$

Information:

NQ₂ = Number of vehicles arriving and stopping during the red phase, (SMP)
s = Signal cycle time, (seconds)
R_H = Green ratio
DS = Degree of saturation
Q = Traffic flow (SMP/hour)
3600 = Conversion factor from hours to seconds

The total number of queued vehicles (NQ) was determined by summing the queue components NQ₁ and NQ₂, as expressed in the following equation:

$$NQ = NQ_1 + NQ_2$$

$$NQ = 0.580 + 34.271$$

$$NQ = 34.851$$

Information:

NQ = Total number of queued vehicles at the approach (SMP)
NQ₁ = Number of queued vehicles resulting from uniform arrivals (SMP)
NQ₂ = Number of vehicles arriving and stopping during the red phase (SMP)

Queue length was determined from the total queued vehicles and effective approach width at the stop line, using an average area of 20 m²/SMP.

$$PA = NQ \times \frac{20}{L_M}$$

$$PA = 34,851 \times \frac{20}{7}$$

$$PA = 99,754$$

Information:

PA : Queue length (m)
NQ : Total number of queued vehicles (SMP)
L_M : Effective approach width at the stop line (m)
20 : average area occupied by one passenger car unit (m²)

The calculated queue length was used to assess queue conditions at the proposed four-phase signalized intersection.

3.4.6. Delay

The performance of each approach was further evaluated based on delay and queue probability. Delay was determined based on traffic flow and degree of saturation, while queue probability was estimated from the resulting values. These parameters were used to assess the operational conditions of the proposed four-phase signalized intersection. The geometric delay was calculated using the following equation, with Approach A (North) shown as an example.

$$T_{LL} = s \left(\frac{0.5(1 - R_H)^2}{1 - R_H \times DS} \right) + \frac{NQ_1 \times 3600}{C}$$

$$T_{LL} = 130 \left(\frac{0.5(1 - 0.3340)^2}{1 - 0.3340 \times 0.79} \right) + \frac{0.580 \times 3600}{C}$$

$$T_{LL} = 40.73$$

Information:

T_{LL} = Average geometric delay (s/SMP);
s = Signal cycle time (seconds)
R_H = Green ratio
DS = Degree of Saturation
NQ₁ = Number of vehicles in queue due to uniform arrival (SMP)
C = Approach capacity (SMP/hour)
3600 = Conversion factor from hours to seconds

The traffic delay (T_{LL}) obtained from the preceding calculation was then used together with the geometric parameters to determine the geometric delay (T_G). The geometric delay was calculated using the following equation, with Approach A (North) presented as an example.

$$T_G = (1 - R_{KH}) \times P_B \times 6 + (R_{KH} \times 4)$$

$$T_G = (1 - 0.828) \times 0.2983 \times 6 + (0.828 \times 4)$$

$$T_G = 3.62$$

Information:

- T_G = Average geometric delay (s/SMP).
 R_{KH} = Proportion of stopped vehicles
 P_B = Proportion of turning vehicles
6 = Geometric delay for vehicles that do not stop (seconds)
4 = Geometric delay for stopped vehicles (seconds)

The resulting traffic delay (T_{LL}) and geometric delay (T_G) were then summed to obtain the average delay (T) for Approach A (North), as expressed in the following equation:

$$T = T_L + T_G$$

$$T = 40.73 + 3.62$$

$$T = 44.35$$

Information:

- T = Average delay (s/SMP)
 T_{LL} = Average traffic delay (s/SMP)

T_G = Average geometric delay (s/SMP)

The average delay obtained for each approach was then used to determine the total average delay at the intersection based on the traffic flow of each approach, as expressed in the following equation:

$$T_{total} = T \times q$$

$$T_{total} = 44.35 \times 1049$$

$$T_{total} = 45.23$$

Information:

- T_{total} : total delay (SMP/s);
 T : average delay (s/SMP);
 q : traffic flow (SMP/h).

The calculated delay values were used to assess the operational performance of the proposed four-phase signalized intersection.

3.4.7. Performance Summary of the Proposed Four-Phase Signalized Intersection

The calculated performance parameters were subsequently summarized to provide an overall assessment of the proposed four-phase signalized intersection under the projected traffic demand and signal timing conditions. The summary includes traffic flow, capacity, degree of saturation, average delay, and queue length for each approach.

Table. 15 Performance Summary of the Proposed Four-Phase Signalized Intersection

Approach	Traffic Flow (q) SMP/h	Capacity (DS) SMP/h	Degree of Saturation (DS)	Queue Length (PA) s/SMP	Average Delay (T) m
U	1049	1336	0,79	99,574	46523
S	767	977	0,79	51,566	43796
T	408	520	0,79	64,351	22991
B	650	828	0,79	70,034	35562
Maximum Queue Length (m)				99,754	-
Total Delay (SMP/s)					148873
Average Intersection Delay (s/SMP)					51,80

The results provide an overall representation of the operational performance of the proposed four-phase signalized intersection and serve as a basis for comparison with the other improvement alternatives.

3.5. Comparative Evaluation of Intersection Improvement Alternatives

The performance of the proposed intersection improvement alternatives was compared to determine the most effective

treatment under the projected traffic conditions. The comparison included the roundabout and four-phase signalised intersection alternatives, both with and without approach widening. The evaluation considered key operational performance indicators, including degree of saturation (DS), average delay, and queue length. The degree of saturation assessed the relationship between traffic demand and available capacity, while delay and queue length described the resulting operational conditions at the intersection. The comparison results for each improvement alternative are summarized in Table 15.

Table. 16 Comparison of Intersection Performance under Different Treatment Alternatives

Parameter	Existing	Design Year (2030)	Roundabout	Signalized Intersection
Traffic flow (Q), SMP/h	3586	4548	4548	2874 (after BKiJT adjustment)
Degree of Saturation (DS)	0.88	1.11	1.07 Critical DS)	0.79
Delay (T), s/SMP	15.16	26.08	52	51.80
Queue Probability (Pa, %)	31.09 – 61.36	49.81 – 99.78	51 - 98	-
Queue Length	-	-	-	99.754

The widened signalized intersection produced a degree of saturation (DS) of 0.79, indicating that the intersection operates below the 0.85 threshold and has better capacity performance than the other alternatives. However, the average delay of 51.80 s/SMP remains relatively high, indicating that improved capacity does not necessarily reduce user delay. Therefore, the signalised intersection with widening is considered the most favourable alternative in terms of capacity and degree of saturation, while the relatively high delay indicates that further operational improvements may still be required to achieve better overall performance.

4. CONCLUSION

1. The existing condition shows a DS of 0.88 and an average delay of 15.16 ss/SMP, indicating that congestion exceeds the performance limit recommended by PKJI 2023 (DS < 0.85)
2. In the design year 2030, the intersection is projected to have a DS of 1,11 and a delay of 26,08 s/SMP, indicating that the DS exceeds the PKJI 2023 performance limit (DS < 0,85)
3. The existing condition shows a DS of 0.88 and an average delay of 15.16 seconds/SMP, indicating that congestion exceeds the performance limit recommended by PKJI 2023 (DS < 0.85).
4. The widened APILL alternative is recommended because its DS meets the PKJI 2023 criteria, while the delay difference between the two alternatives is only 0.20 seconds/SMP.
5. Overall, the signalized intersection provides better performance, as it offers a more effective reduction in the degree of saturation (DS) compared to the roundabout alternative.

REFERENCE

- [1] S. Yuniarti and L. Wananda, "Analisis Kinerja Simpang Tiga Tak Bersinyal (Studi Kasus : Simpang Pall Jl. Raya Jakarta-Bogor-Akses UI)," *Jurnal Teknik Sipil-Arsitektur*, vol. I, no. 24, pp. 1-6, 2025.
- [2] M. R. Ramzy, B. Rahardjo and B. Supriyanto, "Analisis Kinerja Simpang Bersinyal di Kota Malang Menggunakan PKJI 2023 (Studi Kasus: Simpang Dieng Malang)," *Jurnal Inovasi Teknologi dan Edukasi Teknik*, no. 4, 2024.
- [3] S. A. Adha, R. E. Wibisono, M. A. Sabrina and O. E. Putri, "Evaluasi Kinerja Lalu Lintas Simpang Tak Bersinyal Jalan Pulo Wonokromo Kota Surabaya Menggunakan Metode Kapasitas Jalan Indonesia 2023," *Mitrans*, vol. 1, no. 3, pp. 383-391, 2023.
- [4] J. Prananta and Judiono, "Analisa Kinerja Simpang Tak Bersinyal dan Alternatif Penanganannya Studi Kasus: Simpang Tiga Kahuripan, Jalan Jati Raya Kabupaten Sidoarjo," *Portal*, vol. 2, no. 16, pp. 157 - 164, 2024.
- [5] M. Syaifullah, Y. Kadir and F. L. Desei, "Kinerja Simpang Empat Tak Bersinyal Menggunakan Metode PKJI 2023 dan Software VISSIM," *Konstruksia*, vol. 2, no. 15, pp. 147-163, 2024.
- [6] E. Nemers and W. Boy, "Analisis Konfik Lalu Lintas Simpang Tak Bersinyal (Studi Kasus : Simpang 3 Parak Gadang, Kota Padang)," *Rivet*, vol. 2, no. 4, pp. 54-65, 2024.
- [7] D. Khoirotunni'mah, K. H. Putra and T. M. C. Agusdin, "Kinerja Simpang Bersinyal dengan Metode PKJI 2023 dan MKJI 1997 pada Simpang Tiga Jalan Raya Mastrip, Wiyung, Surabaya," *Talenta Sipil*, vol. 1, no. 8, pp. 50-59, 2025.
- [8] A. Maitimu, V. A. Korytelu and H. Tahya, "Analisis Kinerja Simpang Tak Bersinyal Di Ruas Jalan Dr. Malaihollo Kota Ambon," *Jurnal Penelitian Multidisiplin Bangsa*, vol. 7, no. 1, pp. 595-607, 2024.
- [9] Y. F. Pramu, N. Hartatik and D. A. Safitri, "Analisi Kinerja Simpang Bersinyal (Studi Kasus Jl. Manyar Kertoarjo - Jl Kertajaya - Jl. Raya Menur, Kota Surabaya)," *Journal of Scientech Research and Development*, vol. 1, no. 6, pp. 497-504, 2024.
- [10] D. F. F. Desanta, I. Solichin and F. Estikhamah, "Analisis Kinerja Simpang Tidak Bersinyal Jalan Menganti-Jalan Sepat-Jalan Wisma Lidah Kulon Kota Surabaya Menggunakan Metode PKJI 2023," *Agregat*, vol. 2, no. 9, pp. 1109-1116, 2024.
- [11] M. Ridwan, E. Kurniati and Hermansyah, "Analisa Kinerja Bundaran Pada Simpang Cabang Banggo, Kecamatan Manggalewa, Kabupaten Dompu," *J-Central*, vol. 2, no. 2, pp. 6-13, 2024.
- [12] E. P. Sari, Robby and Murniati, "Analisis Volume Lalu Lintas Pada Simpang Bersinyal Studi Kasus Persimpangan Jl. Rajawali – Jl. Hiu Putih Di Kota Palangkaraya," *Basement*, vol. 2, no. 1, pp. 106-113, 2023.
- [13] R. D. Prihantoro, S. Anjarwati and C. A. N. Sari, "Analisis Kinerja Simpang Tak Bersinyal (Studi Kasus Persimpangan Jalan Perintis Kemerdekaan Banyumas Jawa Tengah)," *CIVeng*, vol. 2, no. 6, pp. 91-96, 2025.
- [14] H. E. Prasetyo, A. Setiawan and A. Pradana, "Kinerja Simpang Empat Tak Bersinyal Berdasarkan Derajat Kejenuhan Pada Jalan Raya Mabes Hankam - Jalan Raya Setu, Jakarta Timur," *Konstruksi*, vol. 2, no. 13, pp. 135-145, 2022.
- [15] K. P. U. d. P. R. (PUPR), "Pedoman Kapasitas Jalan Indonesia (PKJI)," Kementerian Pekerjaan Umum dan Perumahan Rakyat (PUPR), Jakarta, 2023.