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Research Article

Microsimulation Analysis of Traffic Operations at Two Diamond Interchange Types

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The operational performance of standard Single Point Urban Interchange (SPUI) and Tight Diamond Interchange (TDI) has already been widely studied. In general, SPUI is more efficient than a TDI in terms of increased capacity and decreased traffic delays even though building a SPUI generally incurs a higher cost. However, due to right-of-way constraints, a standard SPUI may not be implementable at locations with restricted land use; thus, the variations of SPUI are usually considered. Currently, there is no established methodology or guideline available on the performance evaluation of variations of SPUI. This paper aims to investigate the operational efficiency of SPUI with frontage roads (SPUI-F, a variation of SPUI). Based on a field case, the performance of SPUI-F was investigated using microsimulation. An analytical model for capacity estimation, which considered early return and discharge flow rate, was also established and validated based on microsimulation. Multiple traffic scenarios were analyzed and their performance measures were compared against the equivalent TDI design. Simulation results revealed that TDI outperformed SPUI-F in terms of average delay, speed, and queue length, and the proposed analytical model can be used for reliable capacity and delay estimation. The findings from this study can aid the decision-makers choosing an appropriate interchange type for achieving the best benefit-cost ratio.

1. Introduction

Signalized interchanges have been deemed as an efficient and feasible way to connect a freeway to the surface arterials. Currently, the Single Point Urban Interchange (SPUI) and Tight Diamond Interchange (TDI) make up a high proportion of all interchange types in the US due to their stable operational performance, particularly under heavy traffic volume conditions [1]. In real-world condition, to accommodate locations with limited land use space, some variations of diamond interchanges have to be proposed to fit more right of ways [2]. However, geometry variation also leads to the change of conflict movements and thus changes the operational strategy, which may consequently influence the operational efficiency. Among the variations, a frequently used variation has been the Single Point Urban Interchange with Frontage Road (SPUI-F). Nevertheless, the operational efficiency of SPUI-F has not been comprehensively investigated. This is supposed as necessary since a minor change in geometry might cause a significant decrease or increase in its efficiency. Meanwhile, in practice, choosing an appropriate interchange type for a site is critical for improving the efficiency of the transportation system.

Assessing the operational efficiency of different transportation facilities purely based on microscopic simulation may not be convincible. Deploying field case studies can benefit the calibration and validation of microsimulation models and thus provide more realistic results. In Reno, Nevada, a field case of Single Point Urban Interchange with Frontage Road (SPUI-F) existed, which can be used for testing the field performance. With this motivation, this study focuses on the operational efficiency evaluation of SPUI-F and TDI for providing comprehensive recommendations on the selection of interchange types. VISSIM was employed as the microsimulation software to conduct the evaluation analysis. Signal timing data of this interchange was retrieved from

the City's Advanced Traffic Management System (ATMS). Seven different turning volume scenarios combined with a sensitivity analysis were applied, which was supposed to cover the most realistic traffic conditions. At the same time, an analytical model is introduced for estimating capacity with emphasis on the impact of early returns and discharge flow rate, based on which the interchange capacity can be more accurately evaluated. The delay of interchanges can also be realistically adjusted as a result of modification on capacity function.

The rest of the paper is organized as follows: first, a comprehensive literature review on the previous studies; then, description of the case study site and the calibrating the existing models on capacity and delay; after that, evaluation methodology and VISSIM model calibration; finally, results analysis and conclusions of this study.

2. Literature Review

An increasing volume of literature is available on the operational features, applications, and safety concerns of both interchanges. Meanwhile, calibration and validation of microsimulation road network are also constantly growing.

2.1. Operational Features and Applications of SPUI and TDI. The ITE Freeway and Interchange Geometric Design Handbook first officially documented the interchange types of SPUI and TDI [3]. In recent decades, ascribed to the increasing urban traffics, both interchanges have been constantly studied in order to seek the one that has more operational efficiency. In reality, the two interchanges appeared to have different advantages. SPUI was found to have less right of way constraints in contrast to TDI according to conflict points theory [4]. Moreover, its special geometry also enables the dual left-turn operation and thus improves the operational efficiency in heavy left-turn scenarios. Apart from the fewer conflict points, its wide radius [5, 6] can also help to reinforce the operational efficiency by awarding a higher turning speed to vehicles, on which the capacity would be enlarged. Besides those advantages, its disadvantage is also noticeable; SPUI design usually cannot accommodate site with limited space since it requires a longer bridge span [7] for the dedicated left-turn lanes implementation. Another obvious deficient is its large open area which has different entry points which can be unsafe due to the fact that vehicles may misunderstand and violate lane remarks.

The application of the SPUI is currently prevalent but not limited to the US. It also aroused attention from various Asian countries. In Hong Kong, the Kwai Tsing interchange used the design of SPUI [8]. In Indonesia, National Route 1 has the SPUI design implemented on its mainline [8]. In Singapore, the SPUI was implemented in Eunos Flyover [8]. Apart from Asia, SPUI was also widely accepted in Australia [8].

2.2. Operational Features and Applications of TDI. The most significant advantage of TDI is its appearance, since it is similar to two standard intersections; it can greatly reduce the confusions among drivers. Moreover, it does not require additional land resources and a longer bridge span [5] to fulfill the implementation, which may better accommodate urban conditions. However, the design of TDI may not accustom to heavy left-turn conditions according to the previous research. The reason may due to its capacity limitation between its two intersections.

The application of TDI greatly benefits from its multiple control types. Three-phase [9–11] and four-phase control are recommended with different conditions. As the increased percentage of TDI interchanges is deployed in urban areas, four-phase controls are frequently employed because it could enable vehicle make no stops in the middle which catered driver's expectation. The most popular method to achieve this is to use TTI four-phase operation [12–15]. It overlaps the ramp phase and through phase via two dummy phases and the operational efficiency was thus enhanced.

2.3. Studies on Safety. The safety issues were also necessary to be considered. In the view of safety, FHWA report mentioned that the SPUI might be advantageous to TDI due to its fewer conflict points. In other words, the SPUI is expected to cause less safety problem regarding this view. However, Smith's study [16] did not find significant safety performance difference between two interchanges. This conclusion also received the support from field investigation [17]. Another safety research was conducted by Messer et al. [18]; they found that in most cases the red clearance time was insufficient, but no evidence proved that this limitation causes severe safety issues in the real world. Therefore, by now, no solid evidence indicated that one interchange type is over another regarding the view of safety.

2.4. Studies on Microscopic Simulation. Recent years, guidelines published on the microscopic simulation were continuously updated; those methodologies benefited researchers in generating reasonable microscopic simulation models. Park et al. [19] came up with a microscopic simulation model calibration and validation procedure and then tested it on an arterial. Dowling and his fellows [20] were devoted to a systematic level calibration and emphasized the practical operational studies. Park et al. [21] later provided a more formal procedure for calibration of microscopic level models, and in this research, VISSIM was applied. Menneni et al. [22] proposed another research on microsimulation calibration, but in the view using the speed-flow relationship. Its test bed was US-101 and results approved the approach with better performance than capacity view methods. Three years later, Lownes et al. [23] were dedicated to calibrated driver behavior parameters, which aims to provide a better estimation of drivers' influence to capacity.

2.5. Existing Research Shortcoming. The research above already stated the pros and cons of the two interchanges in multiple aspects. However, very few mentioned the change in operational efficiency when SPUI is implemented with two frontage roads. Leisch [24] mentioned that, with the frontage road, the SPUI delay would approximately have an increase of 30%. However, this conclusion has not been updated for several decades. It may differ from today's results due to the fact that traffic volumes and driving patterns have

been changed significantly over the years. Also, there is no current practice of capacity or delay estimation to address the interchange discharge patterns and early return rates. Therefore, this research is devoted to filling the gap by applying a wide-volume range comparison analysis to comprehensively study the operational difference between SPUI-F and TDI.

3. Case Study

As previously mentioned, field data are critical to the model calibration process. The calibration process was mainly based on the comparison of field observed queue lengths and simulated queue lengths. Based on this real-world SPUI-F case, this research collected sample traffic flow and performance data to more accurately develop and calibrate the VISSIM models. For this evaluation, the case study can be even more critical since the current SPUI-F was converted from a TDI, which can serve as a good contrast case as its geometry can be easily accessed by Google Earth's achieved data. This research was conducted based on a real-world variation diamond interchange case in Reno, Nevada (i.e., the I-580/Plumb Ln interchange), which can be used for contrasting the operational performance for SPUI-F and TDI. Prior to 2004, the interchange was a TDI. Due to the annual growth of traffic, the local transportation engineers found that the TDI design cannot accommodate the traffic demands. Meanwhile, the majority of research at that time continuously declared that SPUI outperformed TDI in terms of operational efficiency. Therefore, the TDI design was eventually changed to a SPUI design. However, due to the constraints of the crossroad to the north of Plumb Ln, the through lanes were preserved at the on- and off-ramps. Consequently, the interchange was revised to a SPUI with frontage road (SPUI-F) to accommodate the connection between the crossroad and freeway.

Nevertheless, in reality, it was found that there are a number of issues with the SPUI-F operation. Since the interchange connects the Reno-Tahoe airport and midtown area of Reno, thus traffic volumes are usually heavy at the site. In addition, this interchange is located in the vicinity of the shopping area, indicating that the interchange serves not only the commuting and tourists traffic but also the shopping traffic. The geometric layout of this SPUI-F interchange is distinct from a typical SPUI, as illustrated in Figure 1(a). In addition, Figure 1(b) outlines the scope of the original TDI, which was extracted from the historical document from Google Maps.

To generate reasonable and convincible results, this research used the current I-580 SPUI-F interchange geometry as the basis of evaluation network with the same scale as field use. For TDI road network, it was constructed depending on the same lane configurations of SPUI-F without changing the exterior geometry.

4. Modeling Capacity and Delay

The existing capacity estimation model for signalized interchanges did not fully involve the effects of the discharge flow rate and effective green time. This may not bring significant affection to at-grade intersections (no view blockage), but will significantly impact interchanges. For example, since drivers' view of sight tends to be blocked by the bridge, drivers may not confidently accelerate when the green light is on. In terms of effective green time, for this interchange, traffic flow from the off-ramps might significantly vary from the crossroads; thus the off-ramp may trigger gap out option, resulting in signal early return to coordinate phases. This phenomenon would increase the capacity of specific movements. Therefore, the calibrated models mainly focused on taking account of these two aspects into the estimation of interchange capacity. According to HCM 2010 [25], the capacity of a signalized interchange is summarized as follows:

$$c = N \cdot S \cdot \frac{g}{C} \tag{1}$$

where

N = number of lanes;
S = saturation flow rate;
g = effective green time;
C = cycle length.

The formula above represents a generalized capacity estimation towards signalized intersections and interchanges. However, to the best of the author's knowledge, very few guidelines were published on the selection of the saturation flow rate considering the variations of interchanges. Therefore, to avoid underestimating or overestimating the capacity of an interchange, reasonable saturation flow rates were needed to apply to the capacity estimation. In most circumstances (especially when the initial queue appears), the saturation flow rate can generally be replaced by saturation discharge flow rate. In this regard, the above capacity estimation function can be revised as

$$c = N \cdot S_d \cdot \frac{g}{C} \tag{2}$$

where

S_d = saturation discharge flow rate.

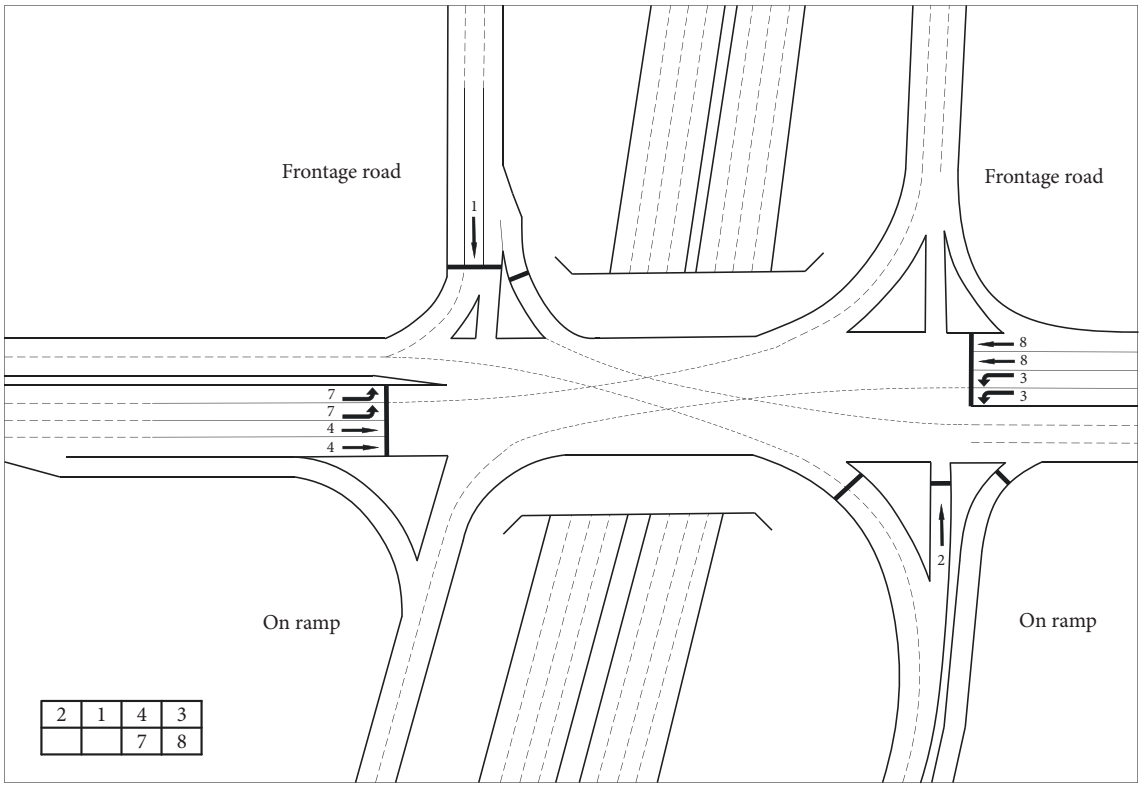
As mentioned before, the actual average green time also influences capacity. In most of the current interchange controls, actuated coordinated strategy is often used. This means that the effective green time may not be fixed. Instead, the noncoordinated phases can gap out; thus the coordinated phases can utilize the time from the gap-out phases. Consequently, the capacity varies differently from fixed time control. In order to address this phenomenon, (2) is written as

$$c = N_i \cdot S_{di} \cdot \frac{g_i'}{C} \tag{3}$$

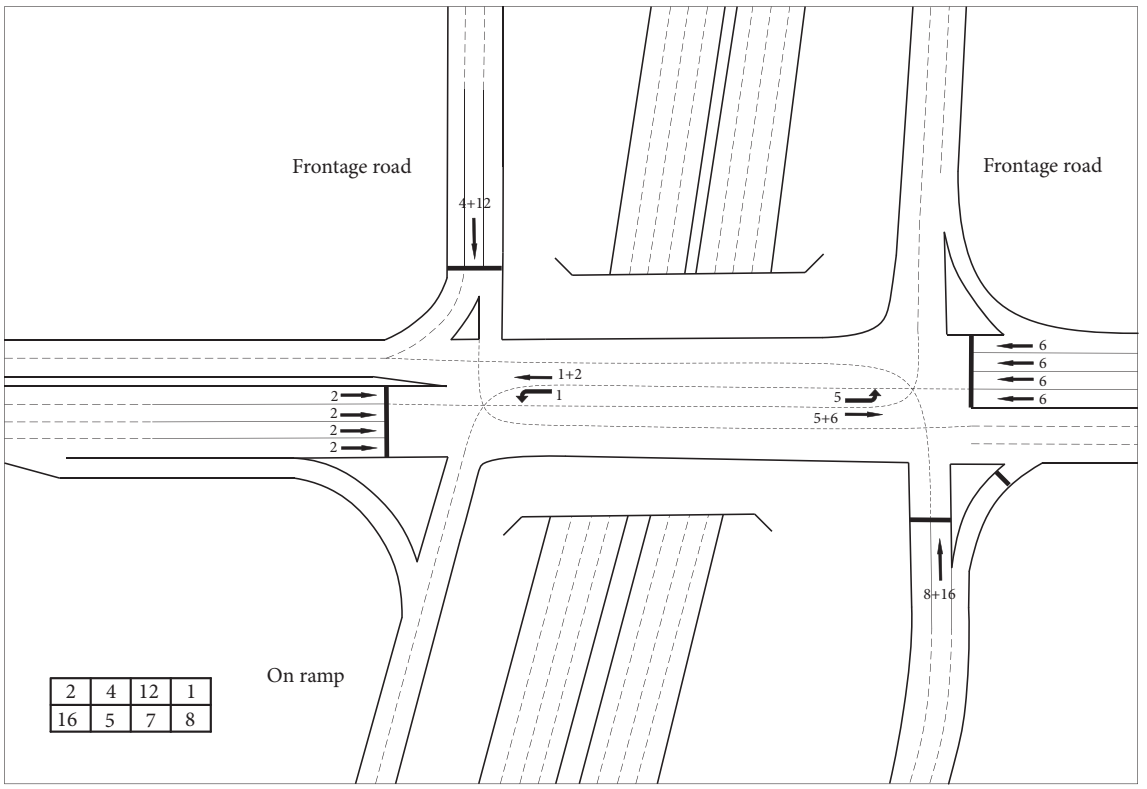
where

N_i = the lane number under phase i control;
S_{di} = the saturation discharge flow rate of phase i;
g_i' = the actual average green time of phase i.

To estimate the average green time, the average gap out and early return indexes need to be taken into account. Average gap out index reduces the designed splits while the early return index promotes the designed splits. Note that



(a) Existing SPUI-F configuration



(b) Previous TDI configuration

Figure 1: Geometric configuration of I-580/Plumb Ln Interchange.

coordinated phases can only have a promotion with the green time. The noncoordinated phases should be considered with both gap out and early return. With this consideration, the following formula is then derived:

$$\sum_{i,j=0}^{n} c_{i,j} = \sum_{i=0}^{n} N_i \cdot S_{di} \cdot \frac{g_{ci} \cdot e_i}{C} + \sum_{j=0}^{n} N_j \cdot S_{dj} \cdot \frac{g_{nj} \cdot e_j \cdot o_j}{C} \quad (4)$$

where

$\sum_{i,j=0}^{n} c_{i,j}$ = the estimated capacity of the whole intersection;

N_i = the lane number of the ith coordinated phase;

g_{ci} = the designed green time of the ith coordinated phase;

e_i = the average early return receive index of ith coordinated phase;

N_j = the lane number of the jth noncoordinated phase;

S_{dj} = the saturation discharge flow rate of the jth noncoordinated phase;

g_{nj} = the designed green time of the jth noncoordinated phase;

e_j = the average early return receive index of the jth noncoordinated phase;

o_j = the average gap out index of the jth noncoordinated phase.

The saturation discharge flow rates for off-ramp left turn movement (phase 1 or 2 in Figure 1(a)) and crossroad left turn movement (phase 3 or 7 in Figure 1(a)) were investigated due to the difference in the vehicle turning radius and view block conditions of two investigated interchange types. For through movements, the saturation discharge flow rate of both interchanges should be the same as a standard intersection. According to the field study, the traffic flow rates for crossroad left-turn movement on SPUI and TDI were found as 1526 veh/h and 1241 veh/h. Rates for SPUI-F were collected on site, while TDI data was collected at a similar site (with similar turning radius and blockage condition). For off-ramp left turn movements, the traffic flow rate for both the interchanges configurations was 1350 veh/h. For early return rate and gap out indexes, these two parameters varied case by case. Therefore, these parameters were calculated based on the historical data obtained from the city's Advanced Transportation Management System (ATMS).

The calibrated capacity can also be used for delay estimation using the updated volume-to-capacity ratio referring to HCM [25]. Regarding this case, with TTI four-phase strategy, the cycle length of TDI could be significantly reduced in contrast to SPUI-F with split phase, which indicates it may have a larger effective green time to cycle length ratio in contrast to SPUI-F. However, for SPUI-F the saturation discharge flow rate of crossroad left turn movement was higher than TDI. Nevertheless, it seems that the credit of saturation discharge flow rate cannot counteract the advantages of TDI. With this consideration, TDI might be more efficient than SPUI-F. To further validate this conclusion, the next section of this paper will employ the microsimulation modeling approach to further investigate the operational efficiency of the two interchange configurations.

5. Microsimulation Modeling

This study employed VISSIM microsimulation for testing the performance of the two interchange alternatives under various traffic demand cases. The experiment design considered different traffic flow scenario, traffic demand levels, and signal timing plans. The operational efficiency was assessed by different Measures of Effectiveness (MOEs), including average delay, average speed, and average queue length.

5.1. Volume Scenario Design. Seven volume scenarios were designed to address different volume patterns. The base scenario was directly derived from the field collected PM peak hour traffic flow. The remaining six scenarios have the different emphasis: left turn (one direction), dual left turn (both directions left turn), through, dual through, ramp, and dual ramp. The details are summarized as follows:

(i) Base Scenario: derived from existing PM peak hour traffic volume.

(ii) Scenario 1 (one approach heavy left turn scenario): designed for evaluation of heavy left turn conditions. For this scenario, the eastbound left turn movement was increased.

(iii) Scenario 2 (two approaches heavy left turn scenario): based on the previous scenario, the left turn volume of westbound was also changed, which aimed to test both directions.

(iv) Scenario 3 (one approach heavy through scenario): designed for evaluating one heavy through movement.

(v) Scenario 4 (two approaches heavy through scenario): the aim of the scenario was similar to the previous one, expect both opposite approaches were tested.

(vi) Scenario 5 (one approach heavy ramp scenario): focused on how the ramp volume variations influence the evaluation results.

(vii) Scenario 6 (two approaches heavy ramps scenario): tested both sides of the ramp volume variations on the evaluation results.

5.2. Volume Sensitivity Design. The sensitivity test is a dimension extension of volume scenario design in this study. Each aforementioned scenario was divided into five cases. Cases were generated based on volume factors (a percentage to the original volume), which varied from 70% to 130% with an increment of 15%. Since these cases were tested for both interchange configurations, a total number of seventy cases were simulated for yielding convincible results.

Table 1 uses the base scenario (for both SPUI-F and TDI) as an example to indicate how the sensitivity analysis was applied. Volume percentage ranged from 70% to 130% of the default volume applied and tested, on the basis of each designed scenario. Therefore, in total 35 cases were applied for each interchange.

Table 1: Base scenario group.

		SBT	SBL	SBR	EBT	EBL	EBR	NBT	NBL	NBR	WBT	WBL	WBR	ToTAL
BASE SCENARIO	70% Volume	77	28	420	175	560	490	21	336	140	210	196	42	2695
	85% Volume	94	34	510	213	680	595	25	408	170	255	238	51	3273
	Default Case	110	40	600	250	800	700	30	480	200	300	280	60	3850
	115% Volume	127	46	690	288	920	805	35	552	230	345	322	69	4429
	130% Volume	143	52	780	325	1040	910	39	624	260	390	364	78	5005

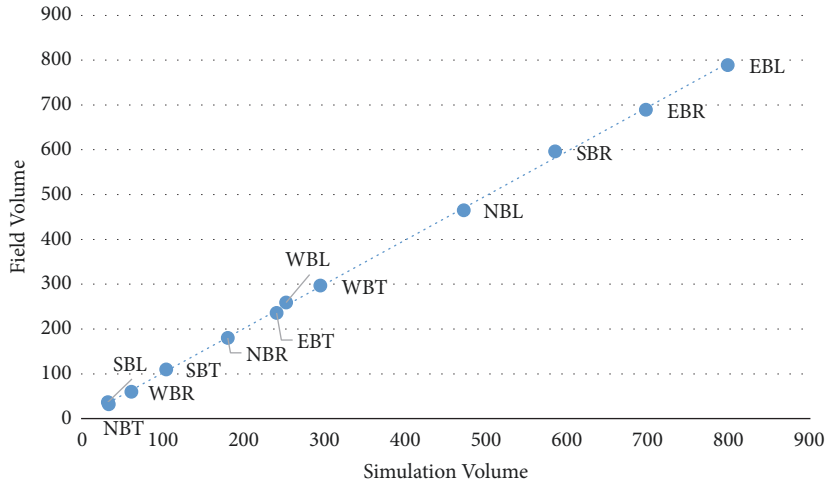


Figure 2: Volume contrast (field vs simulation).

5.3. Signal Operation Setting. Feasible timing plans and reasonable engineering adjustments were also adopted alongside those cases to VISSIM.

Basic timing information, phase splits, and sequences were adjusted for each case. This study did not consider pedestrian constraints for the sake of fully testing the operational efficiency. Parameters like vehicle extension time (passage time), minimum green, yellow, and red clearance were determined separately for two interchanges without further changes when applied to different cases.

In this research, TDI and SPUI-F were also applied to different operational strategies. Three phase operations could not be applied to SPUI-F due to the existence of the two frontage roads. Thus, four-phase control was needed to split the ramp operations. For TDI, since in this case there was not enough queue storage in the middle for vehicles, the TTI four-phase [12–15] was therefore applied to this interchange to ensure a nonstop traffic between the two adjacent intersections.

6. Model Calibration and Validation

Model calibration and validation were conducted on the basis of the PM peak hour volume. In this research, the calibration involved three adjustments: network scale adjustment, signal control strategy adjustment, and driver behavior adjustment [26]. Network scale adjustment was conducted based on the real scale of the interchange. Lane width, segment length, pocket length, and signal head locations were accurately modified based on the field conditions. Signal timing parameters were input which conformed the ATMS [27] information. Truck volumes were also investigated in the calibration process. Based on the observation and video recording, this paper deployed the default truck percentage setting in VISSIM (2%). For model calibration of truck behavior, this paper employed the speed and acceleration data recommended by Yang et al. [28, 29]. For passenger vehicle driver behaviors, apart from vehicle length and standstill distance, the vehicle speed was majorly concerned in this study. To input a reasonable speed, the authors drove a probe vehicle in the flow during the experiment multiple times to obtain the actual travel speed.

The GEH volume validation [30] was used for the model validation. VISSIM produced reasonable results according to Figure 2. The total volume of each movement generally yielded a good match between the simulation and field observation, with an average GEH of 0.33 (good to be accepted due to it being less than 5). The simulation time was set to an hour and divided into 12 intervals (5 min intervals). The maximum queues at the four critical movements were also collected for validation, as illustrated in Figure 3. The Mean Absolute Percentage Error (MAPE) [31] was employed to identify the difference between field observed queue lengths and simulated queue lengths, as demonstrated in Figure 3:

$$MAPE = \frac{1}{n}\sum_{i=1}^{n}\left|\frac{M_m - M_f}{M_f}\right| \quad (5)$$

where

M_m = model calculated MOEs;

M_f = field (simulation) collected MOEs.

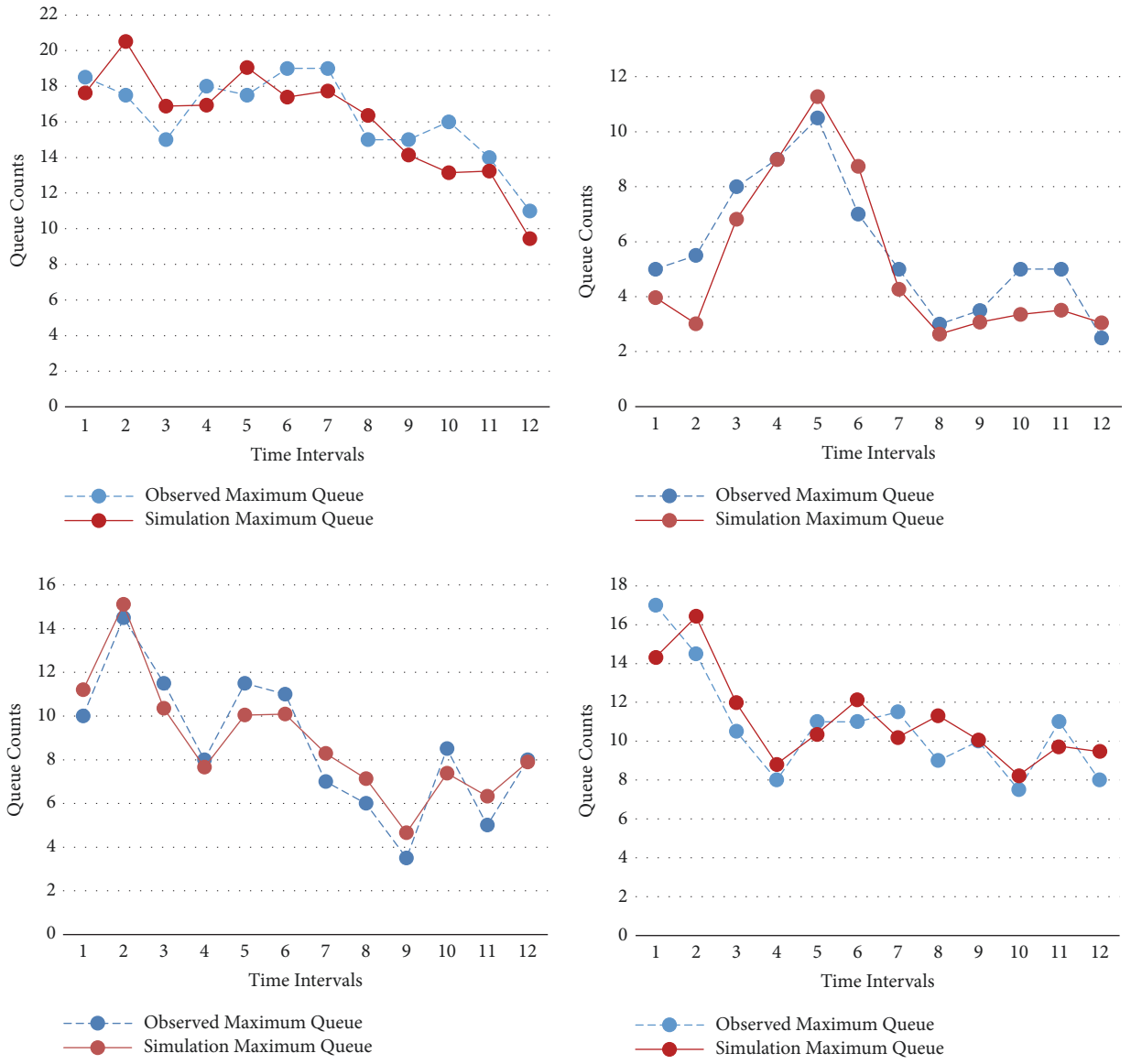


Figure 3: Calibration of VISSIM model (EBL, WBL, SBL, and NBL accordingly).

Results showed that the simulated queue lengths align with field observed queues, with the MAPE value of 0.14 (since queue in number of vehicles is small, one vehicle difference can result in a large MAPE increase; therefore 0.14 here could be accepted), which suggested the network model could represent the actual situation and perform the analysis.

7. Results

Simulation results are illustrated in Figures 4–7. Figure 4 summarizes the average delays for all cases. Average delay kept significantly increasing as the volume level increased for both interchanges. Nevertheless, delay of SPUI-F seemed to have a dramatic growth trend, especially in the cases of +15 and +30 scenarios. In contrast, a stable increase trend was found regarding the delay of TDI. Moreover, the delay of SPUI-F at -30% volume level was even larger than TDI delay at +30% volume level, which implies that TDI is superior to SPUI-F. Therefore, TDI outperformed SPUI-F in view of average delay, which also supported the previous studies concerning the operational difference between SPUI-F and TDI.

Figure 5 illustrates the average speed performance for all cases. With the previous results from average delay, it was not surprising that the average speed of TDI generally performed better than SPUI-F. For reliability, the range of SPUI-F was almost twice larger than TDI in any scenario, which proved that TDI was more reliable. Therefore, the results from average speed reinforced the results from average delay and consistently demonstrated that the performance of TDI was obviously better than SPUI-F.

Figure 6 demonstrates the average queue lengths for all cases. A consistent trend was found for the average delay and speed results. The general trend indicated that SPUI-F was

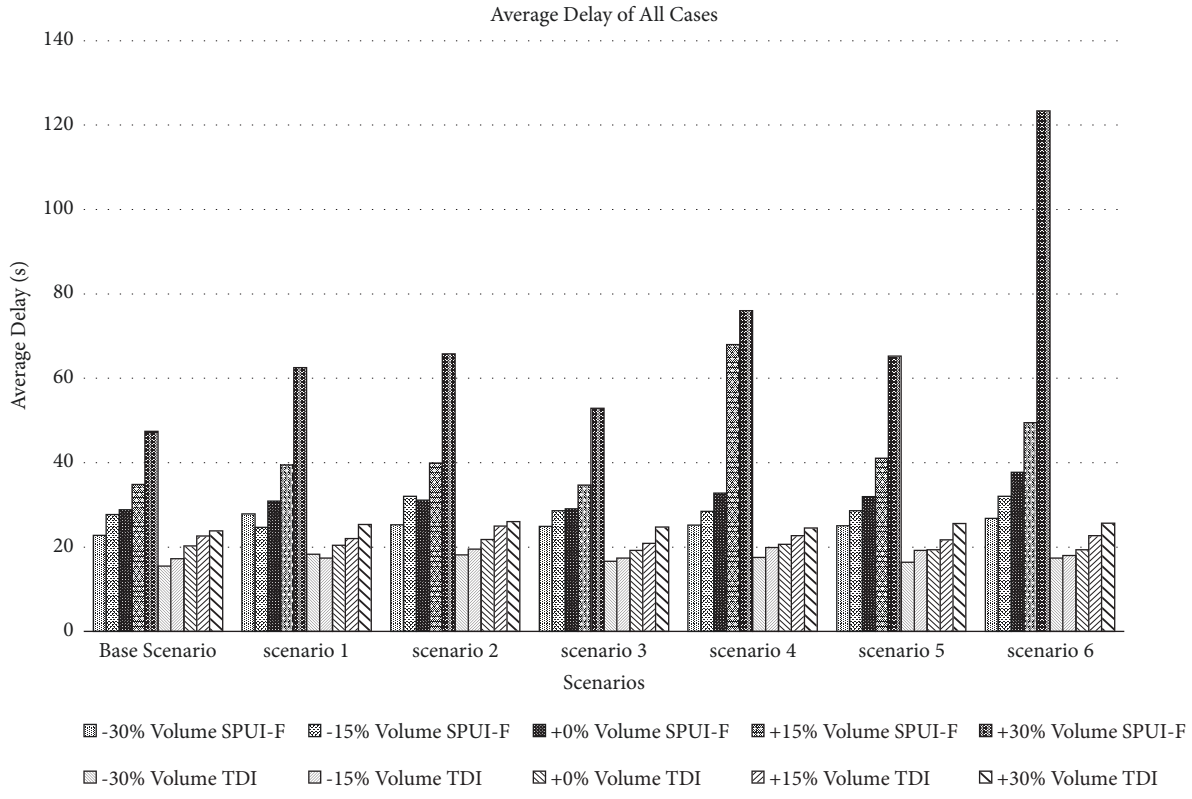


Figure 4: Comparison of average delay under various volume scenarios.

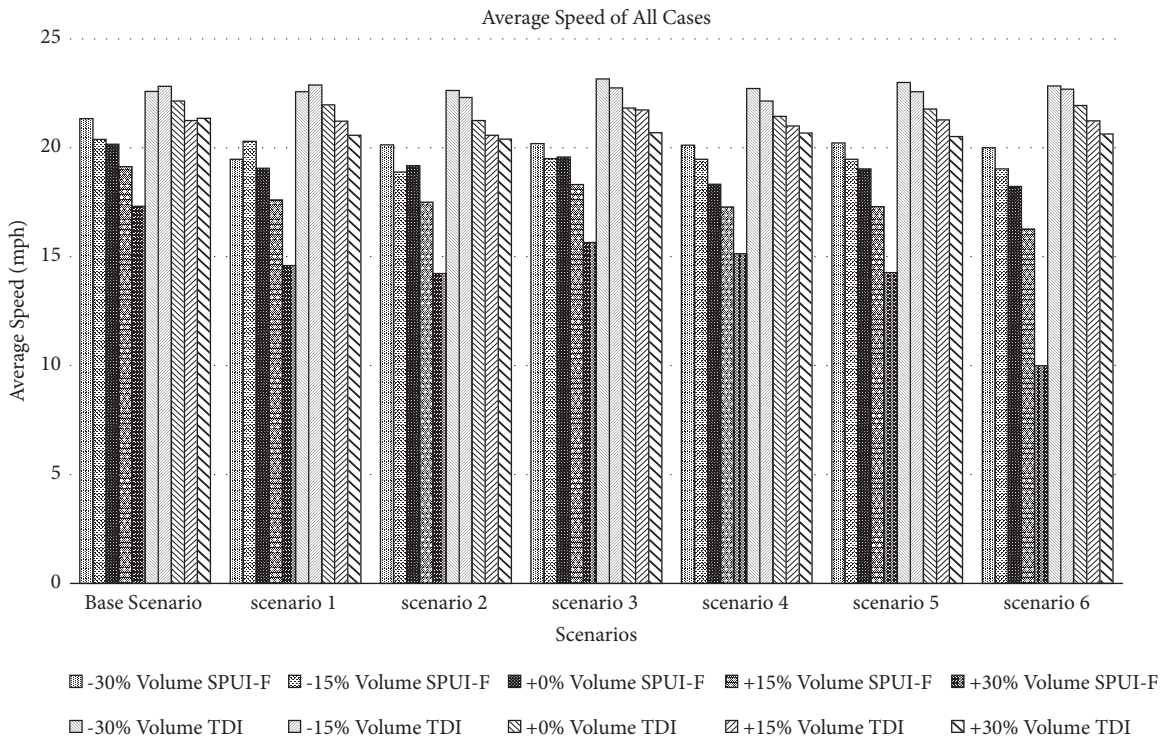


Figure 5: Comparison of average speed under various volume scenarios.

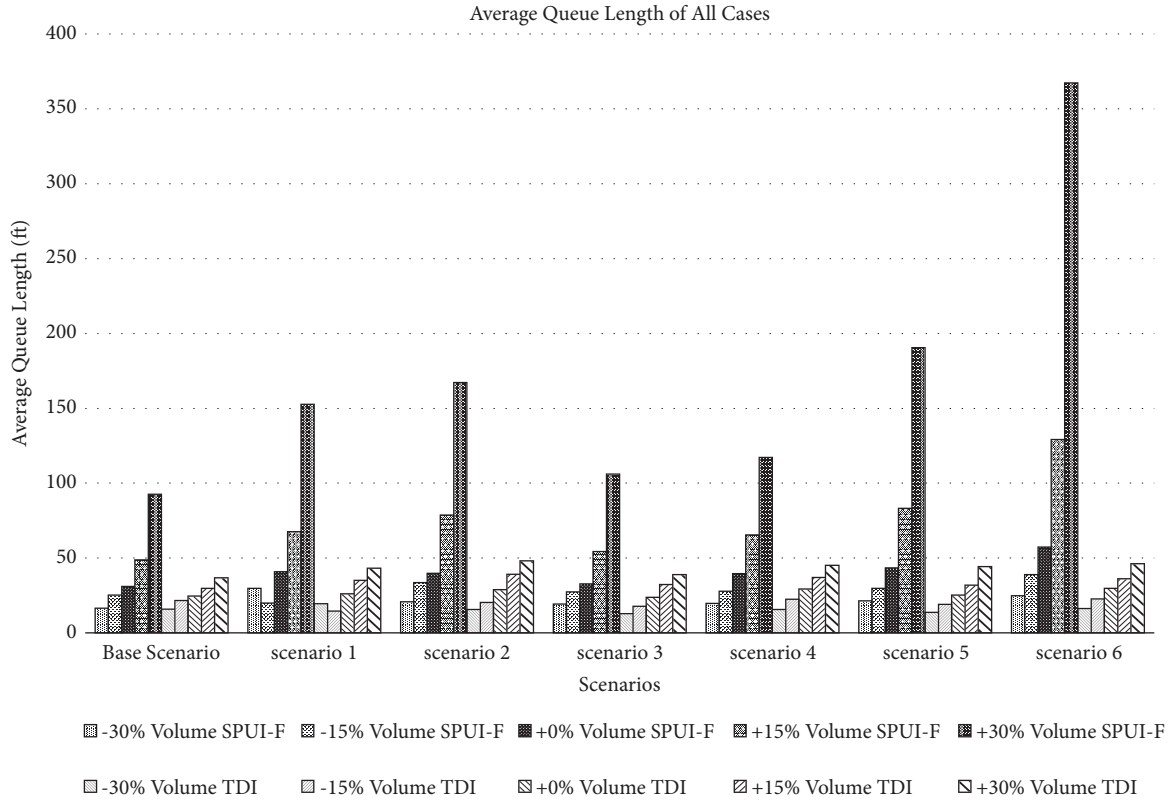


Figure 6: Comparison of average queue length under various volume scenarios.

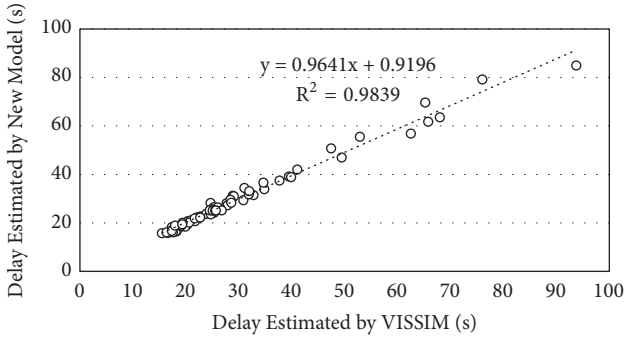


Figure 7: Comparison of VISSIM results and model results on delay.

eager to block more vehicles once the volume increased. For low traffic volume conditions, the difference in average queue length for SPUI-F and TDI was not obvious. However, the situation started to turn down at moderate volume levels and heavy volume levels. Hence, TDI still performed better than SPUI-F in terms of average queue length.

From the comparisions of average delay, average speed, and average queue length under various volume scenarios, as illustrated in Figures 4 to 6, it could be concluded that SPUI-F is not recommended for field implementation. The reason behind the performance drop may be due to the limitation of capacity. Recalling the operation strategies, TDI used the TTI-four-phase; multiple overlaps were used with this special technique, which enhanced the efficiency. On the other hand, SPUI-F also used four-phase operation; however, the strategy for SPUI-F had to be used. Owing to the fact that split control was the only choice for SPUI-F, the operational efficiency dramatically dropped. Therefore, it was not surprising that TDI is more advanced than SPUI-F in all of the designed conditions.

8. Modeling Results against Simulation

Since it is difficult to collect field data for comparison, to test the effectiveness of the proposed model, delays estimated by the proposed model were compared against VISSIM simulation outputs. MAPE was employed to identify the difference between model estimated delay and simulated delay.

Delays for both interchange types were calculated for all the tested cases. It was found that the MAPE results for SPUI-F were 5% and TDI was 3.1%, which validated the accuracy of the model. Figure 7 shows the comparison between the simulation and calculation. The residue square was 0.98, which further illustrated that the calibrated model could be used for delay estimation.

9. Conclusion

This research systematically compared the operational performance between SPUI-F and TDI and developed analytical models for capacity and delay estimation for the two diamond

interchange types. The capacity and delay estimation models were validated using a calibrated VISSIM microsimulation model, which was initially validated by field traffic data.

The results revealed that SPUI with frontage road is less efficient than TDI in operational performance, which is consistent with Leisch's findings [24]. Also, this research found that TDI was more reliable when handling heavy traffic volume scenarios. Conversely, the sensitivity test exposed the shortcomings of SPUI-F that the delay and other MOEs increased considerably faster than TDI. The average queue length performance was quite noticeable since the unexpected steep increase of trends in SPUI-F may result in severe congestion problem. In reality, the spillback might occur under heavy volume conditions. In this study, TDI overperformed SPUI-F; all the MOEs provided consistent results throughout the simulation experiments. Therefore, SPUI-F is not recommended for interchanges with heavy traffic volume. The delay of SPUI-F calculated from the model generally matched the simulation results, which is worth to be used for delay estimation in real-world cases.

The paper still has some limitations regarding the case constraints; this research only adopted the case in Reno, Nevada. Cases in multiple cities can be discovered and contrasted in future research. In addition, the calibrated model did not consider factors such as lane change and pedestrian, but it still provided consistent results with the previous research with more cases being analyzed. Future works can be devoted to evaluate operational efficiency of more geometries variations between SPUI and TDI and utilize the Hardware-in-the-Loop Simulation to generate more accurate simulation results.

Data Availability

The ATMS data supporting this model calibration analysis are from Regional Transportation Commission-Washoe and City of Reno, under license, and so cannot be made freely available. The simulation data are available from the corresponding author upon request.

Conflicts of Interest

The authors declare that they have no conflicts of interest.

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