



Figure 5-4 Stage 1 Road Width

5.3.1.8. Site Stage 2- Residential Vehicle Manoeuvring

The carriageway width in this area is approximately 10.8 m, which is considered sufficient to accommodate both parked construction vehicles and through traffic without introducing significant operational conflicts. An existing footpath is located adjacent to the proposed parking area, with a minimum width of approximately 1.4 m. Given the available separation distance, interaction between pedestrians and construction activities is expected to be minimal. Furthermore, as the surrounding residential lots are not yet fully developed, pedestrian activity is anticipated to remain low during the construction period.



Figure 5-5 Stage 2 Road Width

5.3.1.9. Conclusion

Residential vehicle manoeuvring is not expected to present any operational issues as each lot is provided with individual parking and, there are sufficient manoeuvring space to accommodate turning and reversing movements within the property boundary for both Stage 1 and 2.

5.3.2. Interference with Footpath

There will be no direct interference between the existing footpath and construction activities, as works are largely contained within the development site (Stage 3 and 4).

Construction vehicle movements through Stage 1 and 2 are expected to utilise the available carriageway width, with sufficient separation (approximately 2-3 metres) maintained between live moving vehicles. As a result, interaction between pedestrians and construction activities is expected to be minimal.

Additionally, the surrounding residential lots are not yet fully developed, pedestrian activity is anticipated to remain low during the construction period. Continuous physical barriers or fencing are not considered necessary. Where construction vehicles are required to cross or interact with pedestrian paths, appropriate traffic management measures (such as the use of spotters) will be implemented to ensure safe operation.

5.4 Construction Traffic Management Plan

A construction Traffic Management Plan (CTMP) shall be required that will outline:

- Internal vehicle movement (Stage 4);

- Internal vehicle movement (Stage 3);
- Colorado Road entrance for pedestrian safety, live moving vehicles, on-street park; and
- Pedestrian safety, live moving vehicles, on-street park and construction vehicle parking outside Stage 2.

The CTMP shall detail the following:

- **Preferred access:** The plans that would require that all construction traffic shall turn left or right to enter and exit stage 4 and 3 through Colorado Road.
- **Routeing agreement:** Produce a routeing agreement and consult with the Upper Hutt City Council specifying permit routes vehicle types, times, pavement protection and contact details.
- **Governance and roles:** The CTMP shall define the Nominated Traffic Supervisor (NZGTTM-certified), and NZGTTM-qualified Traffic Marshals and/or spotters for any public interface. Qualifications shall be documented.
- **Booking and staging:** Delivery of plant, equipment, and materials will be managed to minimise disruption to the surrounding road network. Bulk deliveries are expected to occur during the initial mobilisation phase, with ongoing deliveries limited. Where required, delivery timing may be managed to avoid peak traffic periods.
- **Internal vehicle management:** The CTMP will outline how construction vehicle movements will be managed within the site, including internal haul routes, turning areas, and separation between plant, workers, and any public interfaces.
- **Temporary traffic management (public road network):** Given that the majority of construction activities will occur within the site, only limited TTMP measures are anticipated. Where required, these may include advance warning signage and temporary controls at the site access on Colorado Road. All measures will comply with the New Zealand Guide to Temporary Traffic Management (NZGTTM).
- **Pedestrian management (footpaths):** Construction activities are expected to have minimal interaction with existing pedestrian facilities. Where required, temporary measures such as signage, delineation, or spotters will be implemented at key interaction points to ensure pedestrian safety.
- **Monitoring and compliance:** The contractor will monitor construction traffic activities to ensure compliance with the CTMP. This may include periodic checks of access arrangements, vehicle movements, and implementation of corrective actions where required.
- **Worker Parking:** The CTMP will identify areas within the site (e.g. temporary construction compound) for construction worker parking. The CTMP will ensure worker vehicles are accommodated within the development site, and do not impact surrounding residential streets.
- **Induction and site procedure:** All construction workers and drivers will undergo site induction prior to commencing work. This will include information on site access arrangements, internal traffic management procedures, and awareness of any public interfaces within or adjacent to the site.

6. Transport Assessment

6.1 Traffic Model Details

6.1.1. Model Development Assumptions

Model Period Set Up:

- Base year 2025; and
- Future year: 2032.

Defined two weekday periods:

- AM Peak: 7:00 AM-9:00 AM
- PM Peak: 4:00 PM-6:00 PM

User are included:

- Cars, HCVs and HOVs

6.1.2. Model Scenarios

The following modelling scenario were developed to assess the potential impacts of the proposed development.

Table 6-1 Model Scenarios

Scenario	Model Year	Description	Lots	Signal Settings	Road Access
Base Model	2025	Existing	-	Existing signal setting.	-
Do Min	2032	Stage 1, 2	49	Existing signal setting.	Larchmont Gr + Colorado Rd
Option 4	2032	Stage 1, 2 3, and 4 combined.	224	Adjusted signal setting	Larchmont Gr + Colorado Rd

6.1.3. Traffic Model Extent

Figure 6-1 and Figure 6-2 illustrates the model extent used for the Traffic Impact Assessment.



Figure 6-1 Model Area

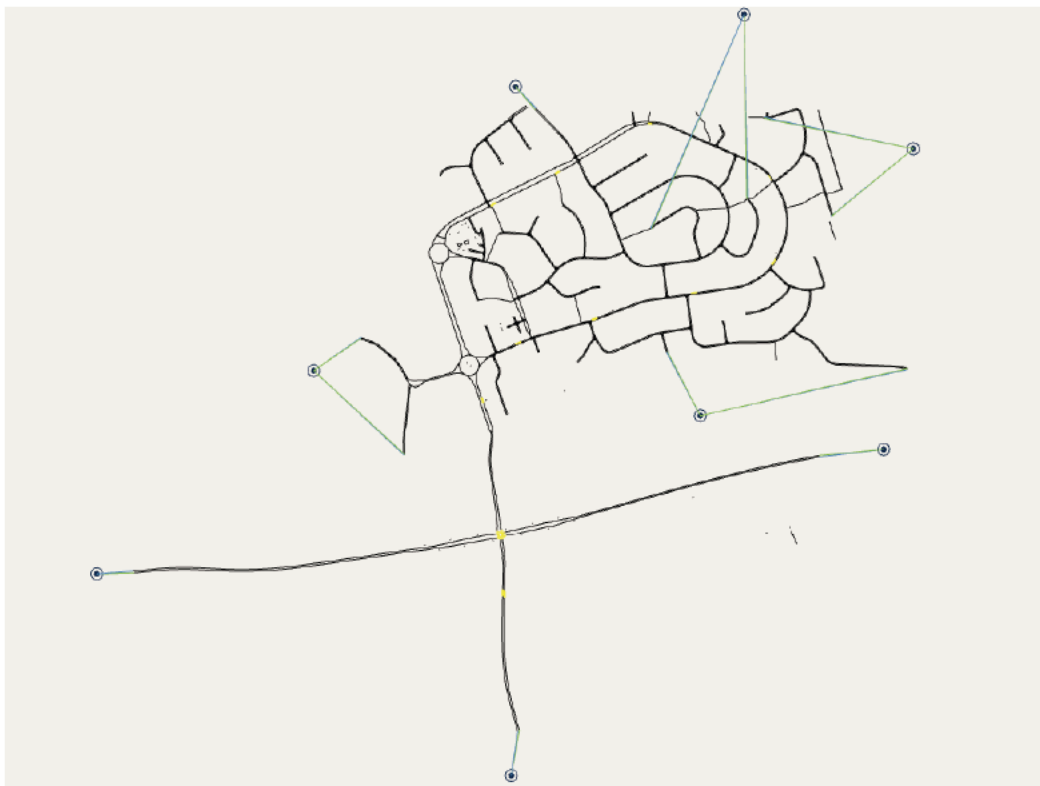


Figure 6-2 Base Model

6.1.4. Trip Generation, Distribution and Mode Split

The **Abley's Tracks Model** and **NZTA Research Report 453** were utilised to understand the existing travel behaviours, specifically in relation to **trip generation**, **trip distribution**, and **mode split**. The outputs from this model were used in the development of a microsimulation model, which was then used to assess the potential traffic impacts of the proposed development on the surrounding road network.

Particular focus was given to the Tōtara Park Road/ SH2 intersection, focusing on key performance indicators such as queue lengths, travel times, and intersection performance under both existing and post-development scenarios.

Item	Assumption
Average household size	2.8 persons/dwelling
Trip legs per person/day	4.4 (all modes)
Car driver mode split	54%
Peak hour factor	10% of daily car driver trips occur in peak hour (1 hr)
Directional splits	AM Peak = 80% outbound / 20% inbound; PM Peak = 60% inbound / 40% outbound
Mode Split	HCV- 15% of total vehicles of SH2 through movements
HCV	5% of total vehicles travelling in and out of Upper Hutt and Tōtara Park
References:	All trip generation and distribution values and methodology are based on: NZTA Research Report 453: Trips and Parking Related to Land Use (2011) :

The trip generation methodology applied in this assessment is consistent with NZTA Research Report 453 – Trips and Parking Related to Land Use (2011). While the trip rate derivation initially referenced average household size, trip legs, and mode share to illustrate the underlying logic, the final trip generation rate adopted in the modelling (0.66 vehicles per dwelling per peak hour) is directly aligned with the vehicle trip rates presented in NZTA RR453.

Accordingly, the car driver mode split was not applied as an additional factor to the NZTA vehicle-based rate. This confirms that the modelling assessment is based on vehicle trips, and the resulting trip generation rate of 0.66 veh/dwelling remains appropriate and consistent with peer expectations.

6.1.5. Level of Service (LoS)

The following Table 6-2 is used to determine the Level of Service (LoS) based on the Highway Capacity Manual (HCM), 2010. The LoS is used to evaluate intersection performance based on average vehicle delay. A delay of **55 seconds or less**, corresponds to the threshold of Level of Service D, is generally considered acceptable. This guideline is applied throughout the assessment to determine whether the operation of the assessed intersection is within acceptable performance threshold.

Table 6-2 Intersection Performance (Level of Service, LoS)

LOS	Signalised Intersection	Unsignalised Intersection
A	<10sec	<10sec
B	10-20 sec	10-15sec
C	20-35sec	15-25sec
D	35-55sec	25-35sec
E	55-80sec	35-50sec
F	>80sec	>50sec

6.1.6. Data Inputs and Comparisons

Only existing available data has been utilised and repurposed, no new data collection is undertaken. The section records the input data obtained and utilised for the model development.

The following has been utilised for model development and calibration:

- Open Street Map Data
- Aerial Photo
- SCATS Data
- TMS Data
- Public Transport Data
- Google Travel Time Data
- Abley Tracks Model

6.1.6.1. Travel Time

The travel time data were obtained from Google Travel Time portal between 12th May 2025 to 16th May 2025 at 30 minutes intervals during the modelled periods (i.e. AM Period 7:00 AM-9:00 AM, PM Period 4:00 PM-6:00 PM). Table 6-3 illustrates the area of the travel time data that was collected and used in the model's development.

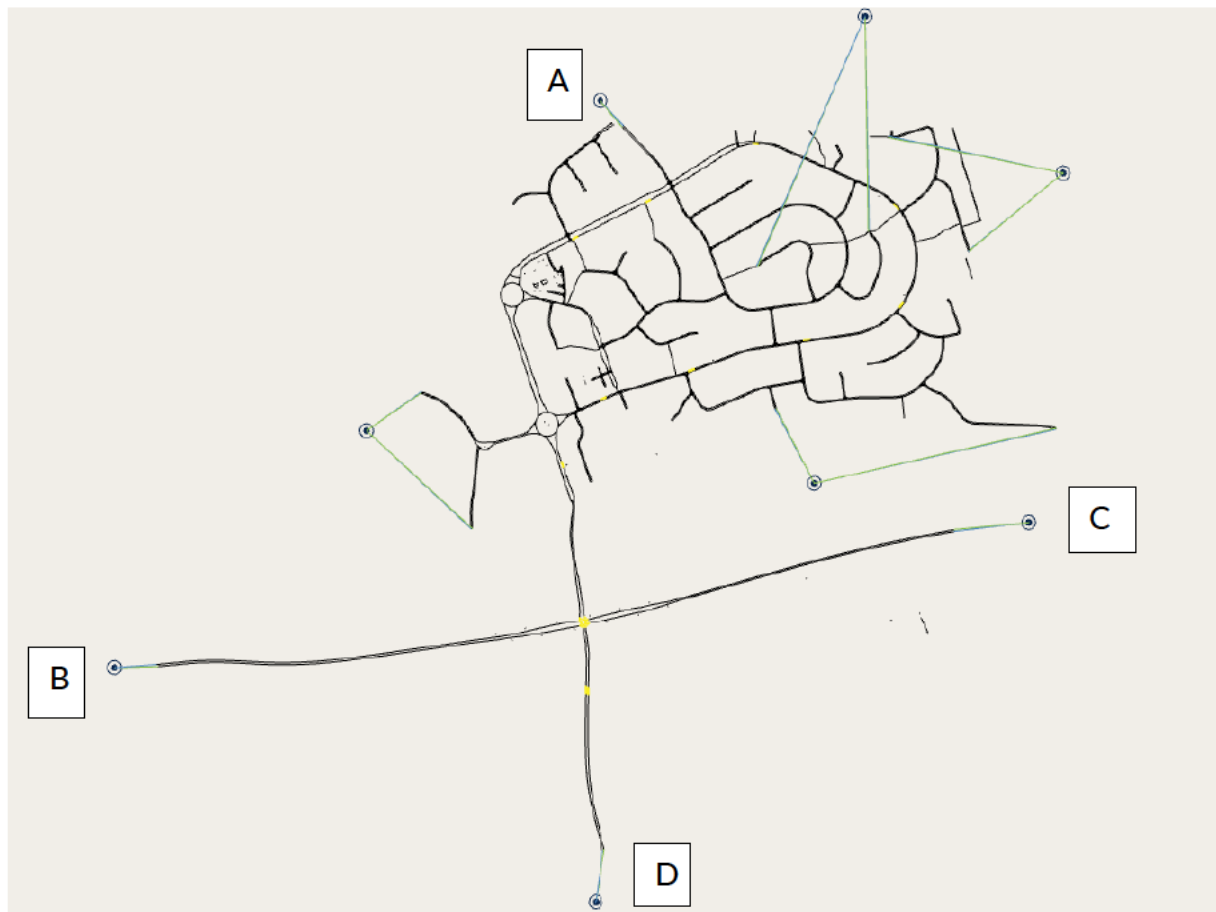


Figure 6-3 Key Route

Table 6-3 Key Route Table

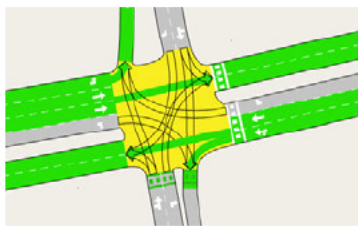
Route	Route ID	Route Name
Route 1	A -> B	Cannon Point to Lower Hutt
Route 2	B -> A	Lower Hutt to Cannon Point
Route 3	C -> A	Upper Hutt to Cannon Point
Route 4	A -> C	Cannon Point to Upper Hutt
Route 5	D -> A	Fergusson Dr to Cannon Point
Route 6	A -> D	Cannon Point to Fergusson Dr
Route 7	B -> C	Lower Hutt to Upper Hutt Via SH2
Route 8	C -> B	Upper Hutt- Lower Hutt Via SH2

6.1.6.2. SCATS Data

For all signalised intersection within and surrounding the proposed model extends are operated by Sydney Coordinated Adaptive Traffic System (SCATS). SCATS would have historical data which it will be extremely useful for the development of the models.

The SCATS traffic volume data and signal timing are obtained from NZTA for dates between 12th May 2025 and 18th May 2025 at the following intersections:

Table 6-4 Tōtara Park Road/ SH2 Intersection SCATS Image

Tōtara Park Rd/ SH2 Intersection			
			
Phase	Phasing	Timings (Sec)	
		AM Peak	PM Peak
A		31	31
Interphase	Interphase	6	6
D		25	19
Interphase	Interphase	6	6
E		17	22

Interphase	Interphase	6	6
G		14	14
Interphase	Interphase	6	6
Cycle Time		111	110

6.2 Base Model 2025 Results

6.2.1. Calibration Results

The base model calibration for the Tōtara Park Road/ SH2 intersection has been conducted to replicate observed traffic conditions accurately, as per the scenario requirements. Calibration involved comparing modelled versus observed link flows for AM and PM peak periods, achieving high correlation levels that exceed NZTA criteria. Key performance indicators, such as GEH statistics, RMSE, and R-squared values, confirm a close match between modelled and observed data, demonstrating the model's reliability for evaluating traffic conditions.

Table 6-5 and Table 6-6 below are the GEH statistics, RMSE and R – Squared values for the section calibration.

Table 6-5 GEH Static

	AM	PM	Target
RMSE	19%	2%	<30%
r ²	0.94	1.00	>0.95

Table 6-6 GEH Summary

GEH SUMMARY						
GEH	AM Count	%	PM Count	%	target %	
0	5	6	75%	8	100%	85%
5	7.5	2	100%	0	100%	90%
7.5	10	0	100%	0	100%	95%
10	12	0	100%	0	100%	\
12	20	0	100%	0	100%	\
Total	8		8			

Overall, the model aligns well with observed data, supporting its use in further analysis and decision-making. The evidence to support the model can be found in the following appendices.

- **Appendix A** - Detailed Section Calibration Data
- **Appendix B** - Turn Calibration Data

6.2.2. Travel Time

Travel times from the model were compared against observed data (Google data), as shown in Table 6-4 and Table 6-5. The results indicate the model closely replicates real world travel conditions.

AM Peak (7:30 - 8:30) (in seconds)

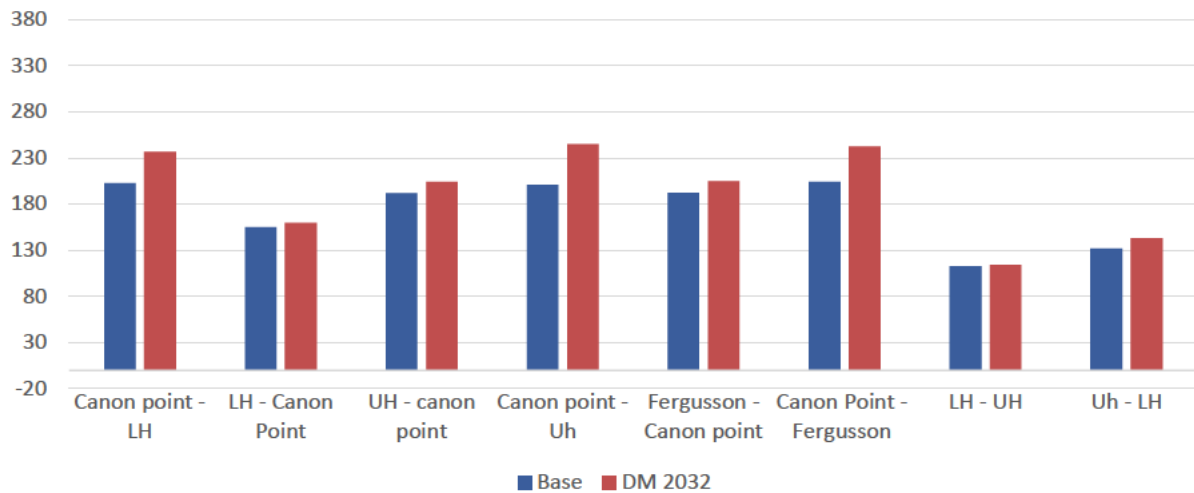


Figure 6-4 Observed vs Modelled Travel Time (AM Peak)

PM Peak (4:30 - 5:30) (in seconds)

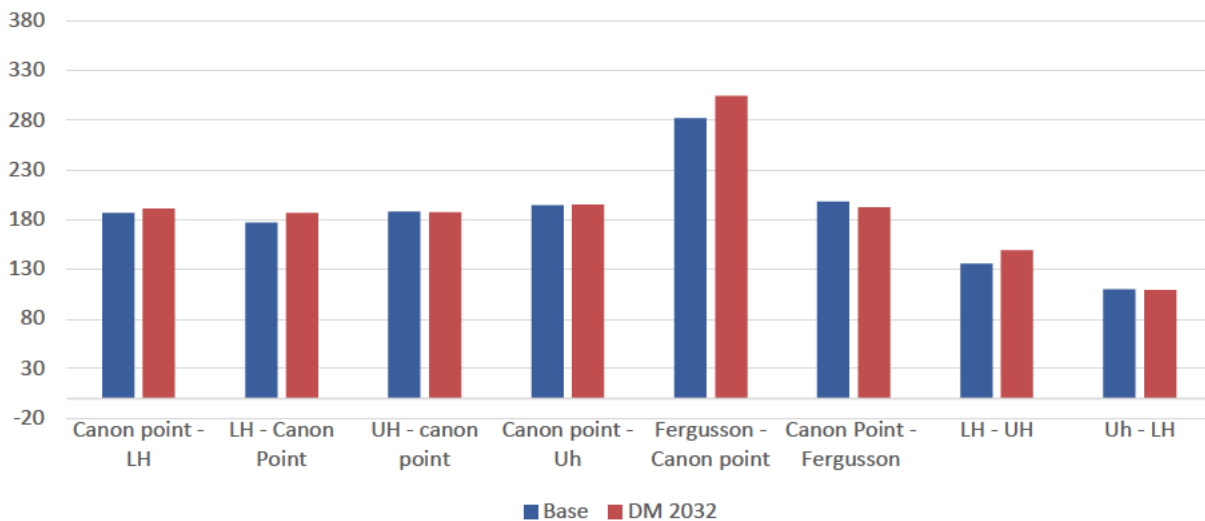


Figure 6-5 Observed vs Modelled Travel Time (PM Peak)

6.2.3. Queue length

The graphs below illustrate the comparison the base model queue lengths. Detailed 15-minute peak queue length data for the modelled results is provided in **Appendix C**.

Table 6-7 Base vs Modelled Queue Length (AM Peak)

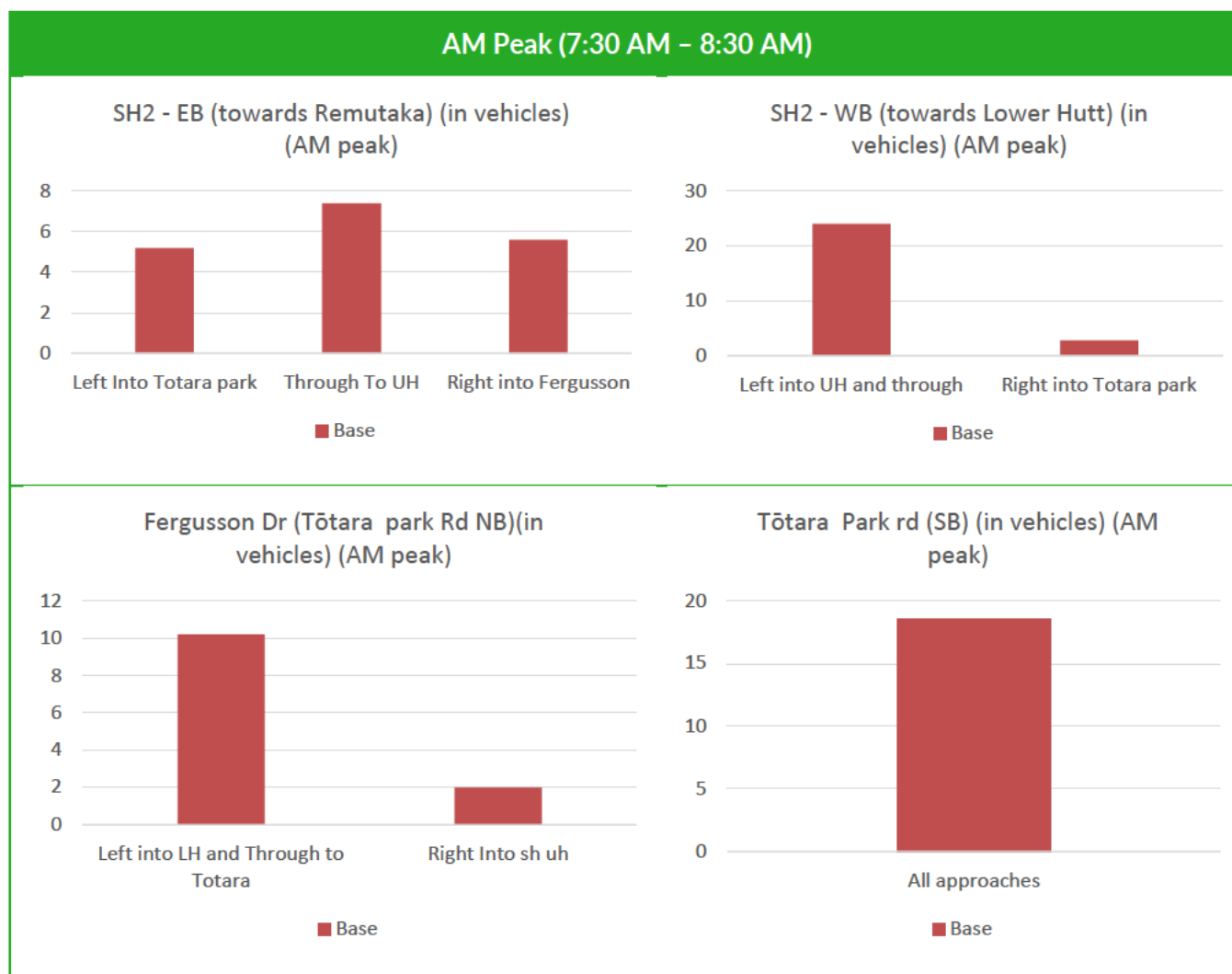
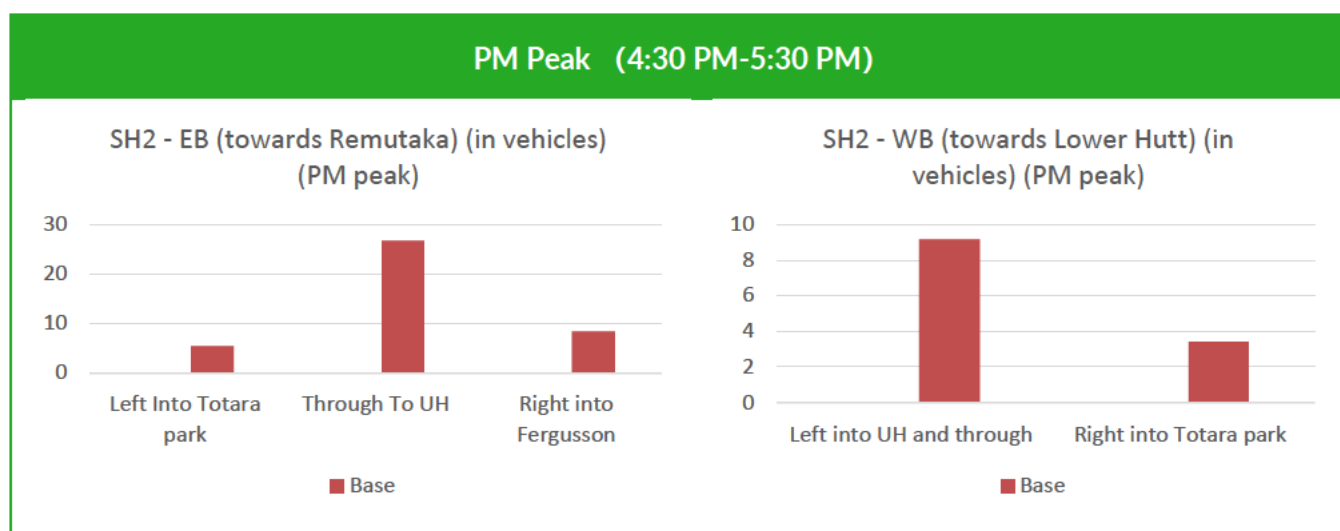
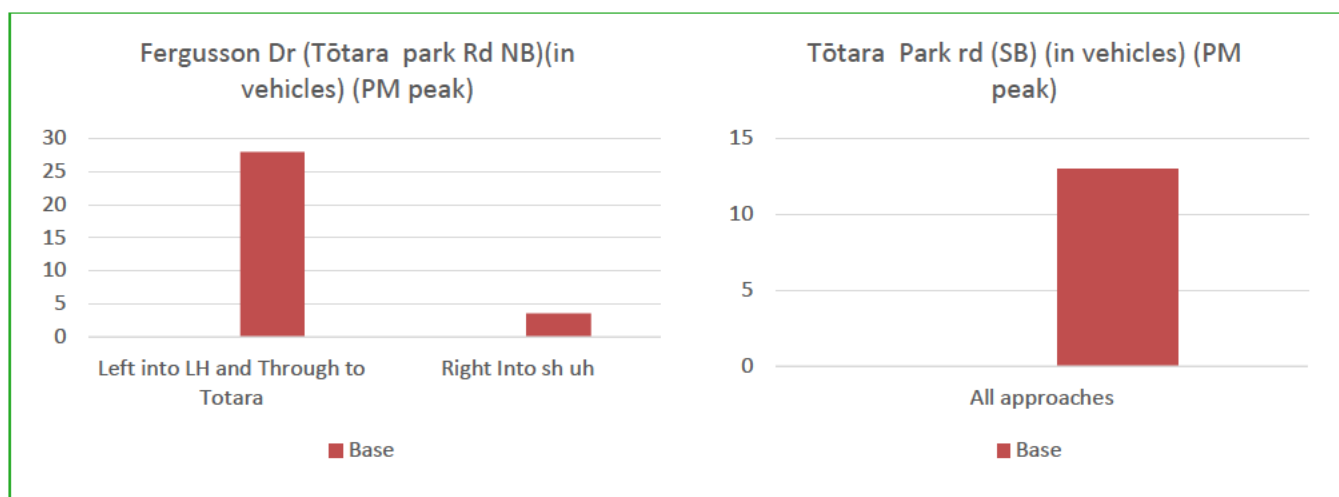


Table 6-8 Base vs Modelled Queue Length (PM Peak)





6.2.4. Turn Performance (LoS)

Table 6-9 and

Table 6-10 below present the base model turn delays and turn LoS ratings for each movement during the AM and PM peak periods. The intersection is currently operating at an average turn delay of 52 seconds (LoS D) during the AM peak and 48 seconds (LoS D) during the PM peak. This result shows that the intersection is already operating near the lower end of LoS D under existing conditions.

Table 6-9 Base Model Turn Delay Time and Turn LoS (AM Peak)

Intersection	Approach	Movement	AM Base								
			Movement			Approach			Intersection		
			Volume	Delay (sec)	LOS	Volume	Delay (sec)	LOS	Volume	Delay (sec)	LOS
SH2/Totara Park	Totara Park North	Left	88	67	E	661	61	E	3507	52	D
		Thru	178	57	E						
		Right	395	62	E						
	SH2 East	Left	100	52	D	1609	55	D			
		Thru	1462	55	D						
		Right	47	56	E						
	Totara Park South	Left	140	53	D	295	52	D			
		Thru	129	52	D						
		Right	26	50	D						
	SH2 West	Left	138	17	B	942	39	D			
		Thru	642	41	D						
		Right	162	53	D						

Table 6-10 Base Model Turn Delay Time and Turn LoS (PM Peak)

Intersection	Approach	Movement	PM Base		
			Movement	Approach	Intersection

			Volume	Delay (sec)	LOS	Volume	Delay (sec)	LOS	Volume	Delay (sec)	LOS
SH2/Totara Park	Totara Park North	Left	198	57	E	439	52	D	4263	48	D
		Thru	60	50	D						
		Right	181	49	D						
	SH2 East	Left	49	35	D	965	39	D			
		Thru	840	39	D						
		Right	76	46	D						
	Totara Park South	Left	375	57	E	703	54	D			
		Thru	217	51	D						
		Right	111	51	D						
	SH2 West	Left	439	26	C	2156	49	D			
		Thru	1466	53	D						
		Right	251	60	E						

6.3 Do Min Scenario 2032

The Do Minimum scenario utilises the same model layout as the Base Model 2025, with the primary difference being the inclusion of trip generation from Stage 1 and Stage 2, which have already been consented. The model scenario includes:

- Existing traffic
- Traffic growth up to 2032
- Traffic from **Stage 1 and Stage 2** of the Tōtara Park subdivision (stages consented and expected to be completed)

Figure 6-6 illustrates the Do Min 2032 Scenario for the proposed development.



Figure 6-6 Do Min 2032 Scenario Model

6.3.2. Do Min 2032 Scenario- Travel Time Results

The travel times under the Do Minimum 2032 scenario were compared with the Base scenario to assess the impact of forecast traffic growth on the network. Figure 6-7 and Figure 6-8 below presents the comparison for both the AM and PM peak periods.

During the AM peak, the following routes are expected to experience an increase in travel times:

- Cannon Point – Lower Hutt: travel time increases from 203.2 to 237.3 seconds (+34.0 seconds)
- Cannon Point – Upper Hutt: travel time increases from 201.0 to 245.2 seconds (+44.2 seconds)
- Cannon Point – Fergusson: travel time increases from 204.6 to 242.5 seconds (+37.9 seconds)
- Upper Hutt – Cannon Point: travel time increases from 191.7 to 204.6 seconds (+12.9 seconds)
- Fergusson – Cannon Point: travel time increases from 192.8 to 205.4 seconds (+12.5 seconds)
- Upper Hutt – Lower Hutt: travel time increases from 132.1 to 143.0 seconds (+10.9 seconds)
- Lower Hutt – Cannon Point: travel time increases from 155.1 to 159.9 seconds (+4.8 seconds)
- Lower Hutt – Upper Hutt: travel time remains relatively unchanged at 112.7 to 114.0 seconds (+1.2 seconds)

During the PM peak, the following routes are expected to experience an increase in travel times:

- Fergusson – Cannon Point: travel time increases significantly from 281.9 to 304.8 seconds (+22.9 seconds)
- Lower Hutt – Upper Hutt: travel time increases from 135.6 to 149.5 seconds (+13.9 seconds)

- Lower Hutt – Cannon Point: travel time increases from 176.8 to 186.6 seconds (+9.8 seconds)
- Cannon Point – Lower Hutt: travel time increases from 186.8 to 191.6 seconds (+4.8 seconds)
- Cannon Point – Upper Hutt: travel time remains relatively unchanged at 194.5 to 195.1 seconds (+0.6 seconds)

Based on the observed results, travel times are expected to experience moderate growth under the Do Minimum 2032 scenario due to forecast traffic growth alone. The increase is most pronounced during the AM peak, with the largest increases recorded on routes originating from Cannon Point. The Fergusson – Cannon Point route shows the most notable increase during the PM peak at approximately 23 seconds. These results establish the baseline growth trend against which the impacts of the proposed development should be assessed.

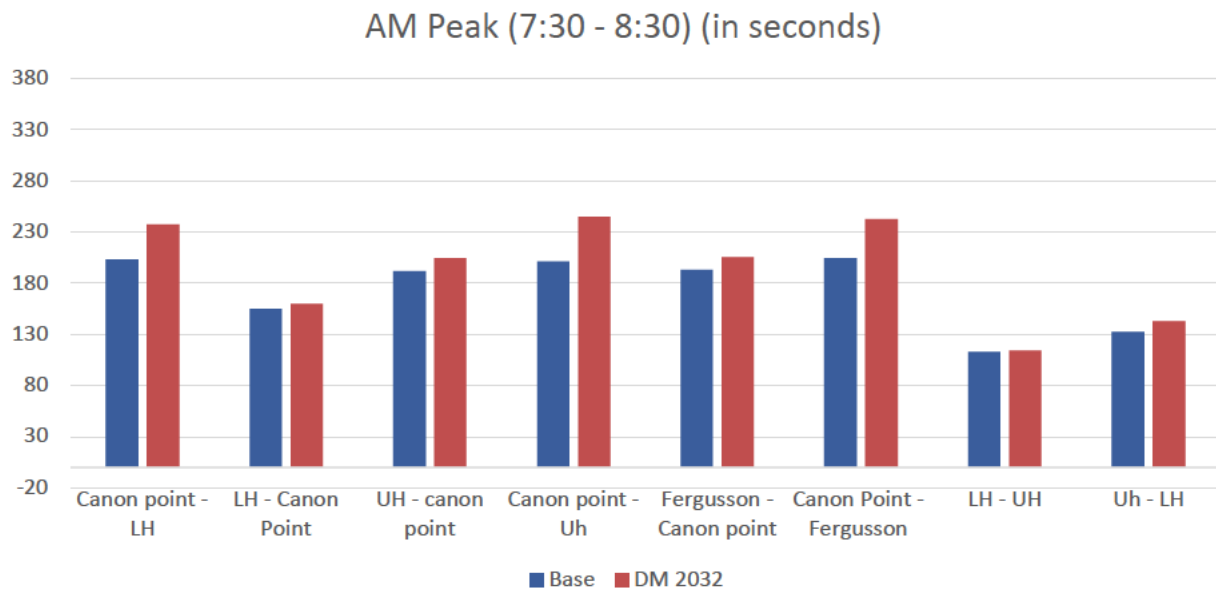


Figure 6-7 Base vs Do Min 2032 Travel Time (AM Peak)

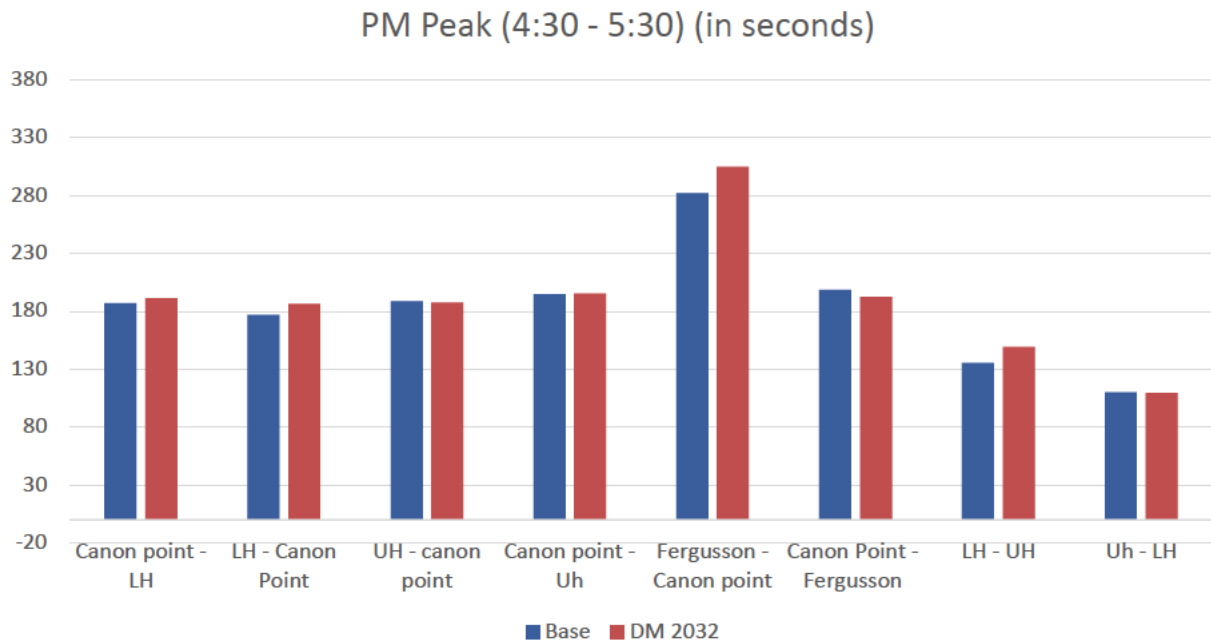


Figure 6-8 Base vs Do Min 2032 Travel Time (PM Peak)

6.3.3. Do Min 2032 Scenario- Queue Length

During the AM peak, it is observed that most approaches show minimal change in queue length between the Base and Do Minimum (2032) scenarios. Except for:

- SH2 - Upper Hutt (Lanes 1 & 2, left into Upper Hutt and through): queue increases from 24 to 35 vehicles (+11 vehicles)
- Tōtara Park (all approaches): queue increases from 19 to 27 vehicles (+8 vehicles)
- Fergusson Drive (Lane 1, left into Lower Hutt and through to Tōtara Park): queue increases from 10 to 12 vehicles (+2 vehicles)

During the PM peak, it is observed that most approaches show minimal change in queue length between the Base and Do Minimum (2032) scenarios. Except for:

- Fergusson Drive (Lane 1, left into Lower Hutt and through to Tōtara Park): queue increases from 28 to 37 vehicles (+9 vehicles)
- SH2 - Lower Hutt (Lanes 2 & 3, through to Upper Hutt): queue increases from 27 to 33 vehicles (+6 vehicles)
- SH2 - Upper Hutt (Lane 3, right into Tōtara Park): queue increases from 3 to 6 vehicles (+3 vehicles)

Overall, the most notable changes under the Do Minimum (2032) scenario are observed on the SH2 - Upper Hutt approach and Tōtara Park during the AM peak, and on the Fergusson Drive (Lane 1) approach during the PM peak. These increases are attributable to forecast traffic growth alone and therefore establishing the baseline queuing conditions against which the impacts of the proposed development should be assessed.

Table 6-11 Base vs DM 2032 Queue Length (AM Peak)

AM Peak (7:30 AM - 8:30 AM)

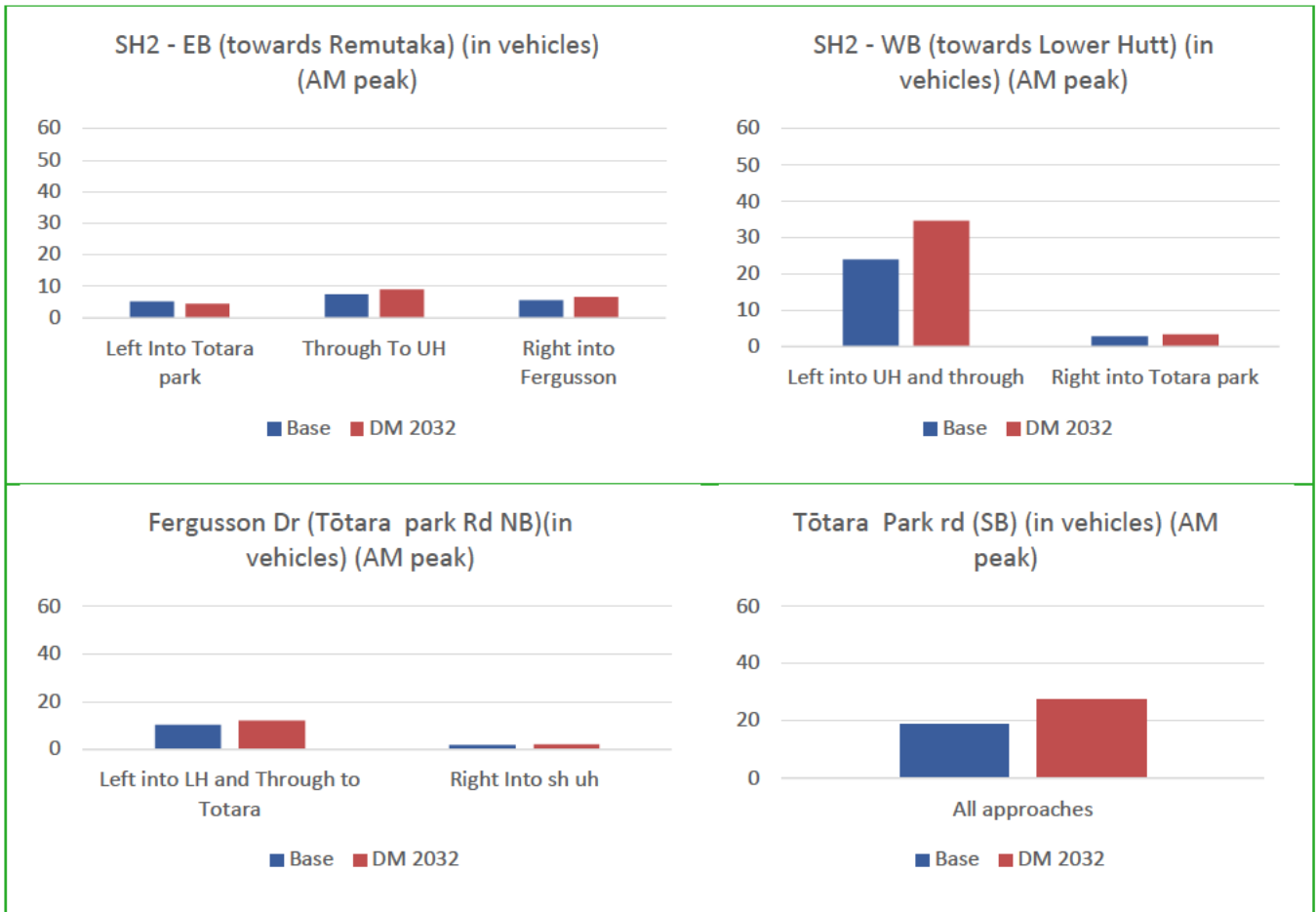
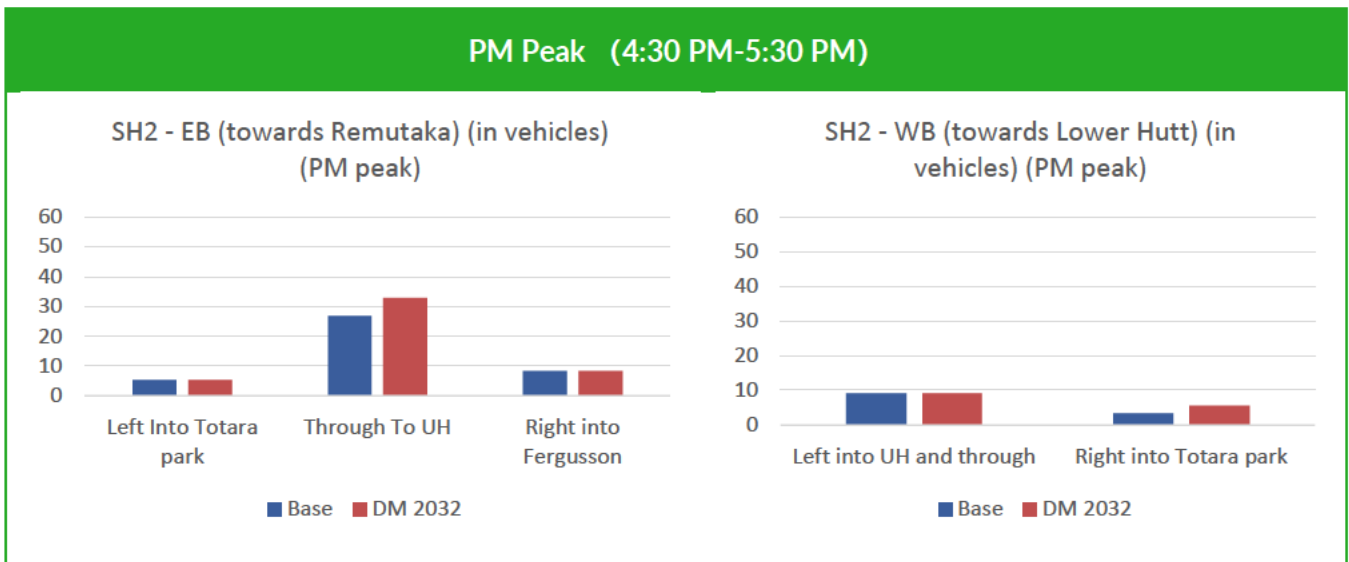
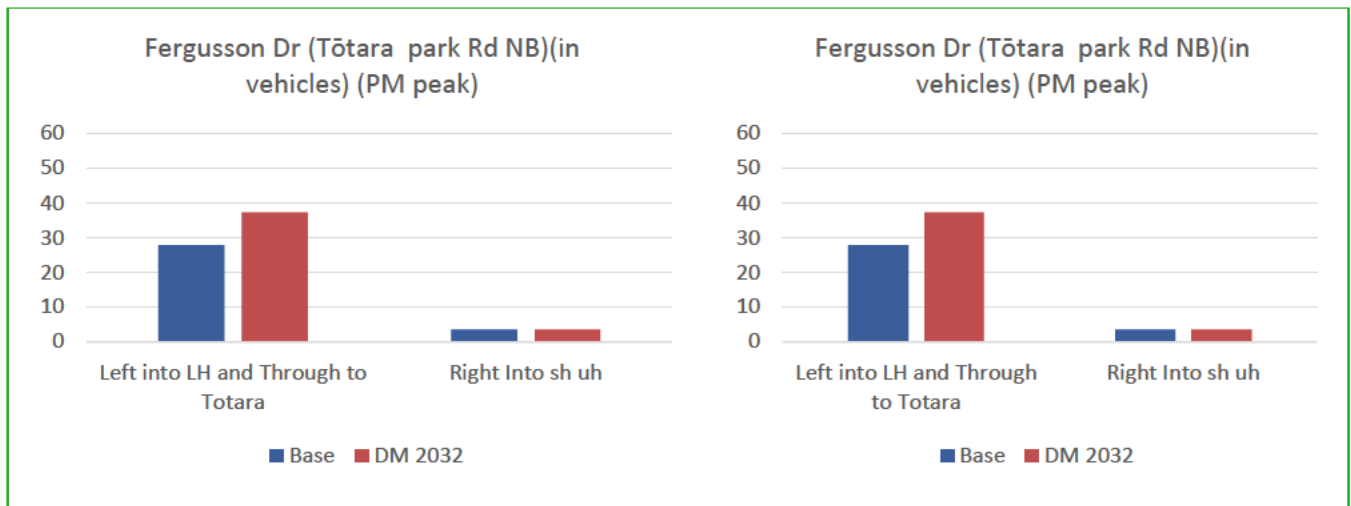


Table 6-12 Base vs DM 2032 Queue Length (PM Peak)





6.3.4. Do Min 2032 Scenario- Turn Performance (LoS) Results

The turn Level of Service (LoS) for the Base scenario and the Do Minimum 2032 scenario were compared to assess the impact of forecast traffic growth on the intersection performance.

During the AM peak, it is observed that the following movements will experience a decrease in Level of Service (LoS):

- **Tōtara Park North - Left:** LoS E to F, delay increases from 67 to 85 seconds (+18 seconds)
- **SH2 East - Thru:** LoS D to E, delay increases from 55 to 63 seconds (+8 seconds)
- **SH2 East - Left:** LoS D to E, delay increases from 52 to 62 seconds (+10 seconds)
- **Tōtara Park South - Thru:** LoS D to E, delay increases from 52 to 60 seconds (+8 seconds)
- **Tōtara Park South - Left:** LoS D to E, delay increases from 53 to 60 seconds (+7 seconds)
- The average delay across all movements increases from 52 seconds (LoS D) to 59 seconds (LoS E) during the AM peak.

During the PM peak, delays remain largely consistent across most turns, with no change in turn Level of Service observed. The average turn delay remains at 48 seconds (LoS D) under both the Base and Do Minimum 2032 scenarios.

Overall, the AM peak results indicate that forecast traffic growth alone is expected to result in a noticeable deterioration in intersection performance, with the average turn Level of Service degrading from D to E. The most significant degradation is observed on the Tōtara Park North - Left, which degrades from LoS E to LoS F. During the PM peak, the intersection performance remains stable with no change in Level of Service across all turns. These results confirm that the intersection is already approaching capacity under forecast growth conditions, independent of the proposed development, and establish the baseline performance against which the impacts of the proposed development should be assessed.

Table 6-13 Do Min 2032 Turn Delay Time and Turn Level of Service (LoS) (AM Peak)

Intersection	Approach	Movement	AM DM								
			Movement			Approach			Intersection		
			Volume	Delay (sec)	LOS	Volume	Delay (sec)	LOS	Volume	Delay (sec)	LOS

SH2/Totara Park	Totara Park North	Left	92	85	F	749	77	E	3728	59	E
		Thru	190	75	E						
		Right	466	76	E						
	SH2 East	Left	102	62	E	1672	63	E			
		Thru	1515	63	E						
		Right	55	61	E						
	Totara Park South	Left	160	60	E	327	60	E			
		Thru	139	60	E						
		Right	28	51	D						
	SH2 West	Left	160	17	B	981	39	D			
		Thru	656	42	D						
		Right	165	50	D						

Table 6-14 Do Min 2032 Turn Delay Time and Turn Level of Service (LoS) (PM Peak)

Intersection	Approach	Movement	PM DM								
			Movement			Approach			Intersection		
			Volume	Delay (sec)	LOS	Volume	Delay (sec)	LOS	Volume	Delay (sec)	LOS
SH2/Totara Park	Totara Park North	Left	224	58	E	482	54	D	4459	48	D
		Thru	67	53	D						
		Right	191	51	D						
	SH2 East	Left	54	37	D	1012	39	D			
		Thru	866	39	D						
		Right	92	46	D						
	Totara Park South	Left	364	60	E	719	56	E			
		Thru	231	53	D						
		Right	123	50	D						
	SH2 West	Left	477	24	C	2246	47	D			
		Thru	1499	53	D						
		Right	270	58	E						

6.4 Option 4 Scenario

6.4.1. Background

Option 4 builds on the Do Minimum 2032 scenario by incorporating forecast traffic growth, along with additional traffic generated by the proposed Stage 3 and Stage 4 development.

The signal settings adopted for this option based on those developed in Option 2, Traffic Impact Assessment Report- Totara Park V2.1, page 27 and 28.

Table below presents the adjusted signal phasing used in this option. Following a discussion with Wellington Transport Operations Centre (WTOC) on 3rd June 2026, WTOC advised that the cycle time should not exceed 150 seconds. As shown in Table 6-15 Table 6-16 the signal setting remain within recommended cycle time.

Table 6-15 Existing Signal & Adjusted Signal Comparison (AM Peak)

Existing AM Peak	Adjusted PM Peak

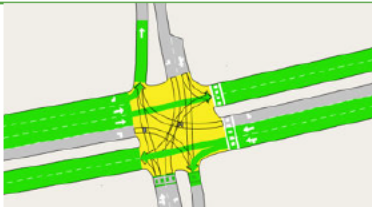
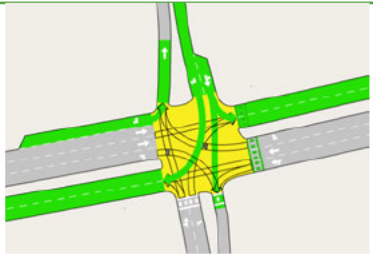
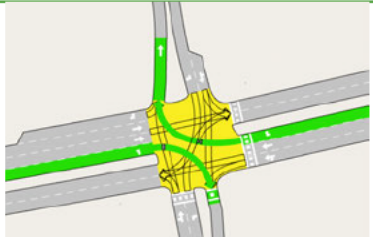
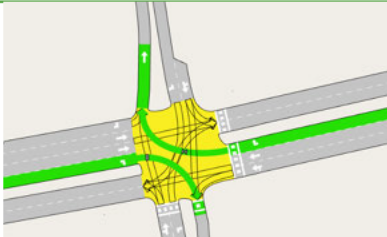
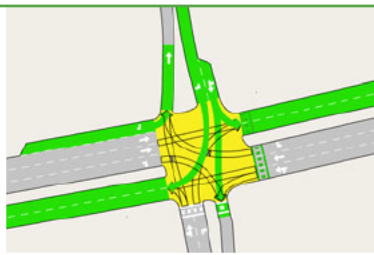
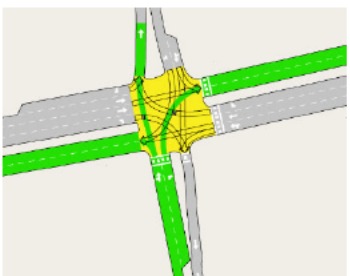
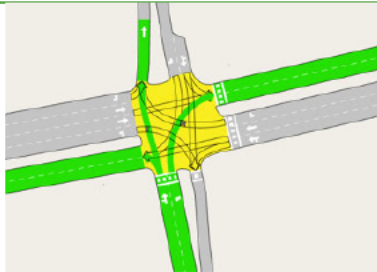
Phase	Phasing	Timing (Sec)	Phasing	Timing (Sec)
A		31		7 (new split phase)
				25 (+1s existing)
Interphase	Interphase	6	Interphase	6
D		25		30 (+5s existing)
Interphase	Interphase	6	Interphase	6
E		17		15 (-2s existing)
Interphase	Interphase	6	Interphase	6
G		14		10 (-4s existing)
Interphase	Interphase	6	-	-
Cycle Time	Total	110	Total	105 (-5 existing)

Table 6-16 Existing Signal & Adjusted Signal Comparison (PM Peak)

Existing PM Peak			Adjusted PM Peak	
Phase	Phasing	Timing (Sec)	Phasing	Timing (Sec)
A		31		15 (new split phase)
				22 (+5s existing)
Interphase	Interphase	6	Interphase	6
D		19		20 (-1s existing)
Interphase	Interphase	6	Interphase	6
E		22		25 (+3 existing)
Interphase	Interphase	6	Interphase	6
G		14		10 (-4s existing)
Interphase	Interphase	6	-	-

Cycle Time

Total

110

Total

110

(0s
existing)

Figure 6-9 below shows the model layout for Option 4:

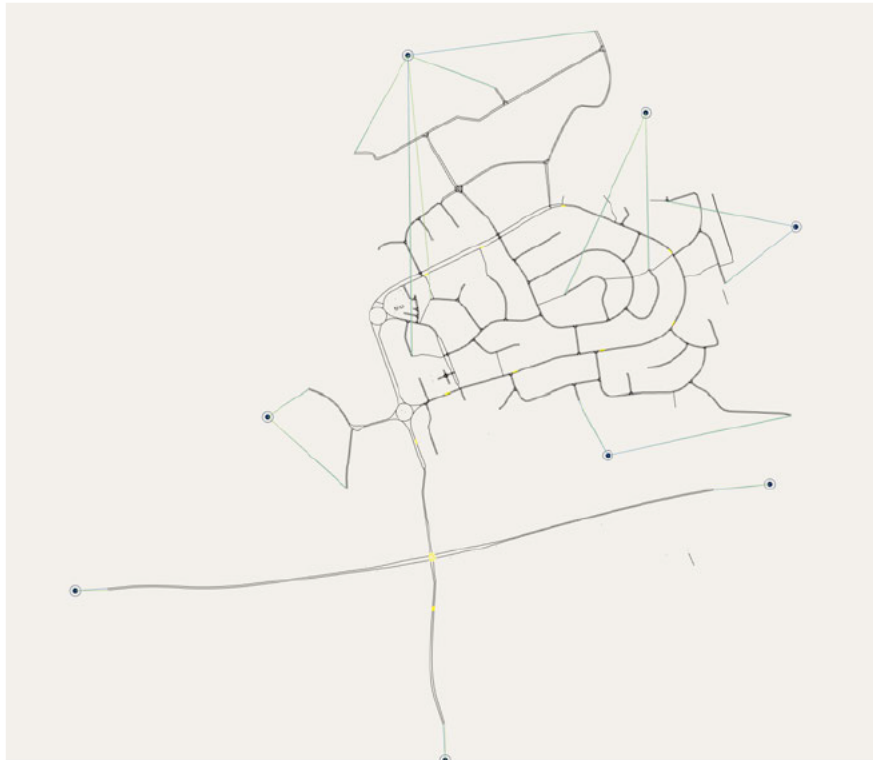


Figure 6-9 Option 4 2032 Scenario Model

6.4.2. Option 4 2032- Travel Time Results

The travel times under Option 4 were compared with the Do Minimum 2032 scenario to assess the impact of Option 4 on the network. Figure 6-10 and Figure 6-11 The figure below presents the comparison for both the AM and PM peak periods.

During the AM peak, the following routes are expected to experience increase in travel times:

- Cannon Point – Lower Hutt: travel time increases from 237.3 to 267.3 seconds (+30.0 seconds)
- Cannon Point – Upper Hutt: travel time increases from 245.2 to 269.6 seconds (+24.4 seconds)
- Cannon Point – Fergusson: travel time increases from 242.5 to 264.2 seconds (+21.7 seconds)

During the PM peak, the following routes are expected to experience increase in travel times:

- Cannon Point – Lower Hutt: travel time increases from 191.6 to 198.8 seconds (+7.2 seconds)
- Upper Hutt – Cannon Point: travel time increases from 187.3 to 204.0 seconds (+16.6 seconds)
- Cannon Point – Upper Hutt: travel time increases from 195.1 to 204.1 seconds (+9.0 seconds)
- Fergusson – Cannon Point: travel time increases significantly from 304.8 to 352.5 seconds (+47.7 seconds)
- Upper Hutt – Lower Hutt: travel time increases from 109.4 to 123.3 seconds (+13.9 seconds)

Based on the observed results, travel time are expected to experience moderate growth with the inclusions of Stages 3 and 4 of the proposed development, with the increase particularly visible

during the PM Peak. It is recommended that the signal phasing should be review and further Table 6-15 and Table 6-16 to accommodate the change traffic patterns and peak demand profiles.

AM Peak (7:30 - 8:30) (in seconds)

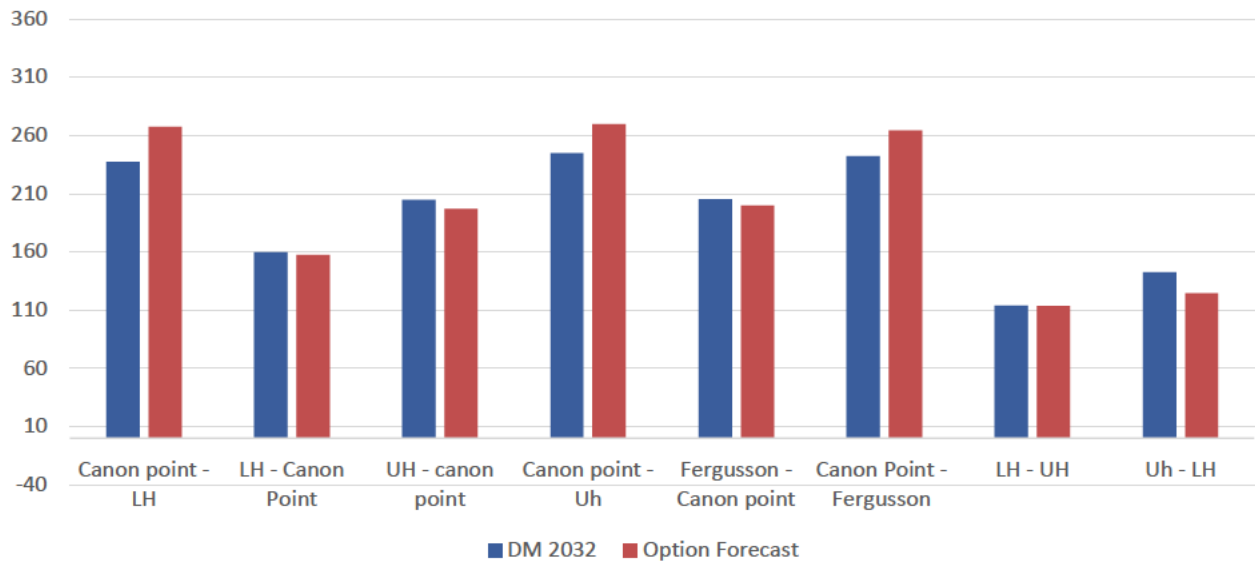


Figure 6-10 Do Min 2032 vs Option 4 Travel Time (AM Peak)

PM Peak (4:30 - 5:30) (in seconds)

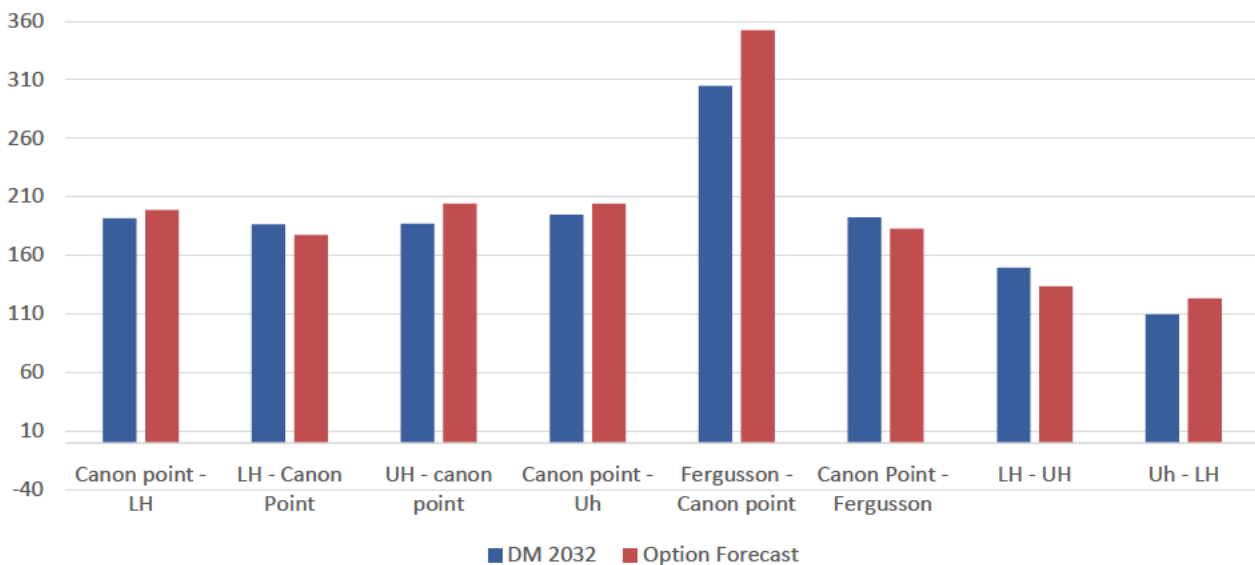


Figure 6-11 Do Min 2032 vs Option 4 Travel Time (PM Peak)

6.4.3. Option 4- Queue Length

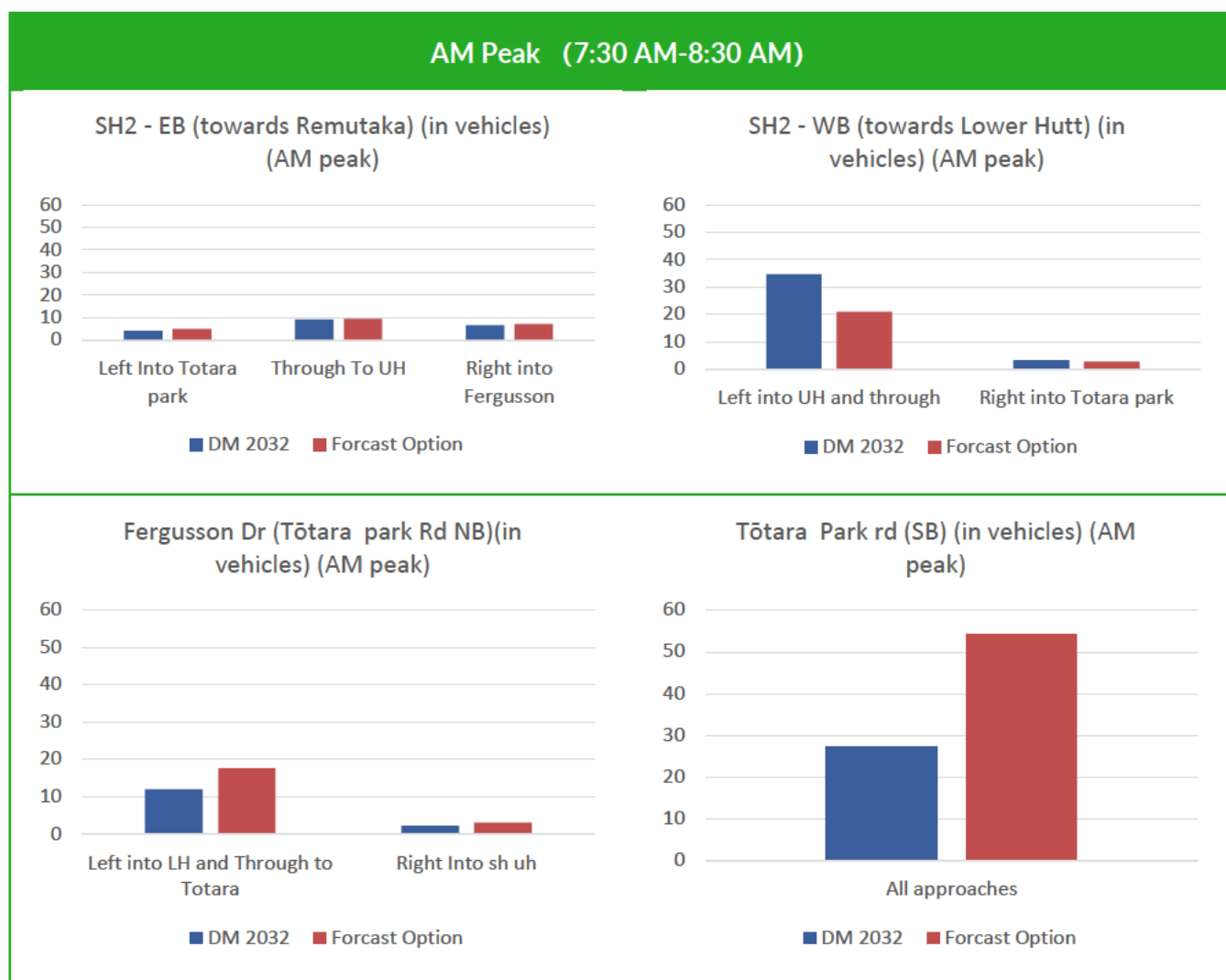
During the AM peak, it is observed that most approaches show minimal change in queue length between the Do Minimum (2032) and Option Forecast scenarios. Except for:

- Fergusson Drive (Lane 1, left into Lower Hutt and through to Tōtara Park): queue increases from 12 to 18 vehicles (+6 vehicles)

- SH2 – Upper Hutt (Lanes 1 & 2, left into Upper Hutt and through): queue decreases from 35 to 21 vehicles (-14 vehicles)
- Tōtara Park (all approaches): queue increases from 27 to 54 vehicles (+27 vehicles)

Overall, the most notable changes are observed on the Fergusson Drive (Lane 1) and Tōtara Park approaches across both peaks. Fergusson Drive (Lane 1) consistently experiences increased queuing under the Option 4 Scenario, while Tōtara Park shows a significant increase during the AM peak. It is recommended that signal phasing be reviewed at this intersection to better accommodate the additional demand generated by the proposed development.

Table 6-17 Do Min 2032 vs Option 4 Queue Length (AM Peak)



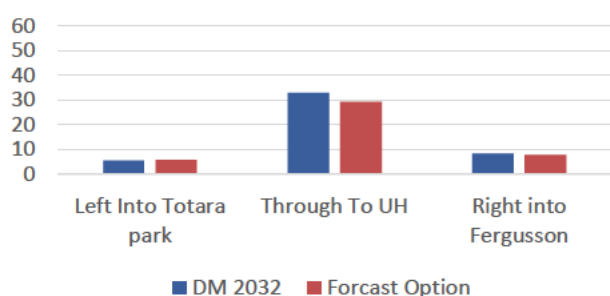
During the PM peak, queue lengths remain largely consistent across most approaches, with only minor variations observed. Except for:

- Fergusson Drive (Lane 1, left into Lower Hutt and through to Tōtara Park): queue increases from 37 to 48 vehicles (+11 vehicles)
- Tōtara Park (all approaches): queue increases from 13 to 18 vehicles (+5 vehicles)

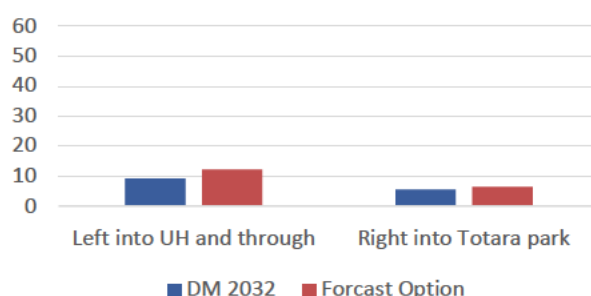
Table 6-18 Do Min 2032 vs Option 4 Queue Length (PM Peak)

PM Peak (4:30 PM-5:30 PM)

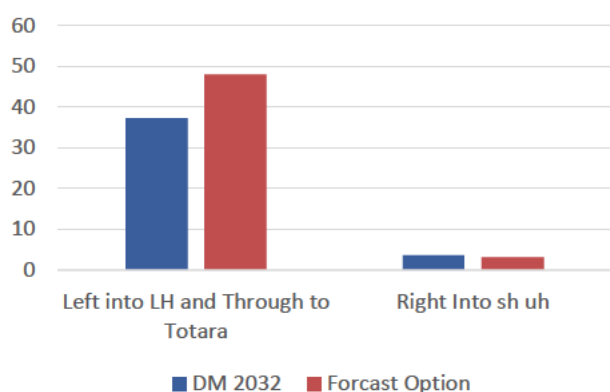
SH2 - EB (towards Remutaka) (in vehicles)
(PM peak)



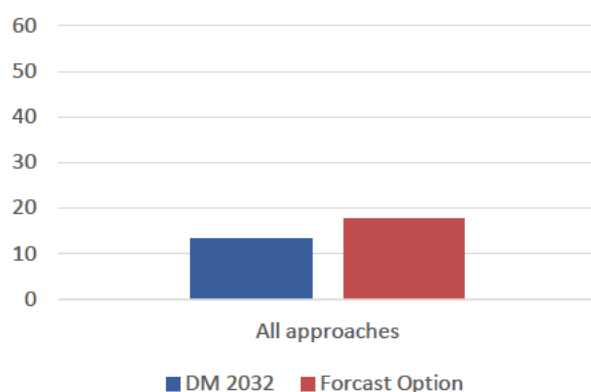
SH2 - WB (towards Lower Hutt) (in vehicles)
(PM peak)



Fergusson Dr (Tōtara park Rd NB)(in vehicles)
(PM peak)



Tōtara Park rd (SB) (in vehicles) (PM peak)



6.4.4. Option 4- Turn Performance (LoS) Results

The turn Level of Service (LoS) for the Do Minimum 2032 scenario and the Option 4 scenario were compared to assess the impact of the proposed Stage 3 and Stage 4 development on intersection performance.

During the AM peak, it is observed that the following movements will experience a decrease in level of service:

- **SH2 West – Right:** LoS D to E, delay increases from 50 to 67 seconds (+17 seconds)
- **Tōtara Park South – Right:** LoS D to E, delay increases from 51 to 67 seconds (+16 seconds)

During the PM peak, it is observed that the following movements will experience a decrease in level of service:

- **Tōtara Park North – Right:** LoS D to E, delay increases from 51 to 58 seconds (+7 seconds)
- **Tōtara Park North – Thru:** LoS D to E, delay increases from 53 to 56 seconds (+3 seconds)
- **SH2 East – Right:** LoS D to E, delay increases from 46 to 58 seconds (+12 seconds)

Overall, at the intersection level the volume-weighted average delay under the Option 4 scenario remains within Level of Service D during both peak periods – 54 seconds (LoS D) in the AM peak and

46 seconds (LoS D) in the PM peak. The proposed development therefore does not result in any deterioration in the overall intersection Level of Service relative to the Do Minimum 2032 scenario. The changes in individual movement LoS noted above reflect delays marginally crossing the threshold between categories on relatively low-volume movements, rather than a material change in overall intersection performance.

The intersection already operates predominantly at LoS D, with several individual movements at LoS E, under the Do Minimum scenario – indicating that the existing performance constraints are inherent to the intersection's geometry and signal configuration rather than being driven by the additional demand from the proposed development. Under the Option 4 scenario the intersection continues to operate at LoS D in both peak periods, and the localised movement-level changes do not alter this conclusion. The intersection is therefore expected to continue operating at an acceptable level under the development scenario.

On this basis, the resulting traffic effects of the proposed development are considered acceptable. However, the Do Minimum results suggest that traffic monitoring should be actively implemented prior to the proposed development are occupied.

Table 6-19 Option 4 Turn Delay results and Turn Level of Service (LoS) (AM Peak)

Intersection	Approach	Movement	AM Option								
			Movement			Approach			Intersection		
			Volume	Delay (sec)	LOS	Volume	Delay (sec)	LOS	Volume	Delay (sec)	LOS
SH2/Totara Park	Totara Park North	Left	123	77	E	991	72	E	4026	54	D
		Thru	267	70	E						
		Right	601	72	E						
	SH2 East	Left	100	47	D	1699	49	D			
		Thru	1534	49	D						
		Right	66	48	D						
	Totara Park South	Left	156	69	E	337	68	E			
		Thru	155	66	E						
		Right	26	67	E						
	SH2 West	Left	174	16	B	999	42	D			
		Thru	661	43	D						
		Right	164	67	E						

Table 6-20 Option 4 Turn Delay results and Turn Level of Service (LoS) (PM Peak)

Intersection	Approach	Movement	PM Option								
			Movement			Approach			Intersection		
			Volume	Delay (sec)	LOS	Volume	Delay (sec)	LOS	Volume	Delay (sec)	LOS
SH2/Totara Park	Totara Park North	Left	222	62	E	496	59	E	4662	46	D
		Thru	73	56	E						
		Right	201	58	E						
	SH2 East	Left	56	46	D	1045	51	D			

		Thru	881	50	D						
		Right	108	58	E						
		Left	382	53	D						
	Totara Park South	Thru	284	49	D	774	51	D			
		Right	108	45	D						
		Left	570	19	B						
	SH2 West	Thru	1498	44	D	2346	39	D			
		Right	278	51	D						

6.5 Vehicle Count Comparison

To further assess the impact of the proposed development, traffic volume data was extracted from the transport model to identify the additional vehicle trips generated by Option 4. The Base 2025, Do Min (2032), and Option 4 (2032) scenarios were reviewed and compared.

As the Base Model has been calibrated and validated, the model outputs are expected to provide a reasonable representation of existing traffic conditions. It should be noted that the results presented below are not a direct comparison with observed traffic count data, as no traffic count information was made available during the preparation of this report.

Accordingly, the results presented are solely model outputs and should be interpreted as an indication of the traffic volumes and vehicle trips generated by the proposed development under Option 4.

Based on the comparison result, the most notable increase in traffic volumes during the AM Peak is observed on the Totara Park Road right-turn movement, which is expected to increase by up to 74 vehicles per hour. The Totara Park Road through movement is also expected to experience a noticeable increase of approximately 46 vehicles per hour.

During the PM peak, the most significant increase is observed on the SH2 left-turn movement, which is expected to increase by up to 53 vehicles per hour.

The model predicted turning volume reflects the trip generation is at a correct level, and the increase in turning demand is not significant at the Totara Park Road/ SH2 intersection.

AM Peak (8am-9am)	Turn	2025 Base (Reference) (Veh/ Hr)	Do Min 2032 (Veh/ Hr)	Option 4 2032 (Veh/ Hr)	Development Trips (Difference) Opt 4 vs DM (Veh/ Hr)
Totara Park Road (North)	left	48	49	69	20
	thru	90	95	141	46
	right	194	246	320	74
SH2 (East)	left	53	53	52	-1
	thru	740	764	789	25
	right	31	38	39	1
Totara Park Road (South)	left	87	97	91	-6
	thru	79	90	97	7
	right	17	19	18	-1
SH2 (West)	left	95	110	122	12
	thru	363	380	386	6
	right	92	90	92	2

Total	1889	2031	2216	185
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PM Peak (5pm-6pm)	Turn	2025 Base (Reference)	Do Min 2032	Option 4 2032	Development Trips (Difference) Opt 4 vs DM
Totara Park Road (North)	left	107	121	121	0
	thru	35	37	39	2
	right	106	109	115	6
SH2 (East)	left	21	27	26	-1
	thru	380	397	396	-1
	right	42	53	64	11
Totara Park Road (South)	left	186	182	197	15
	thru	105	113	142	29
	right	60	69	63	-6
SH2 (West)	left	223	233	286	53
	thru	739	764	755	-9
	right	132	138	145	7
Total		2136	2243	2349	106

6.6 Road User Safety

6.6.1 Safe System Audit

The Safe System approach recognises that while human error is inevitable, the transport system should be designed to ensure that such errors do not result in death or serious injury. The four Safe System principles—safe roads, safe speeds, safe vehicles, and safe road users—have been considered for both the construction and operational phases of the proposed Tōtara Park development.

6.6.1.1 Safe Roads and Roadsides

The surrounding road network (Colorado Road, Tacoma Drive, Hartford Crescent, and Larchmont Grove) operates within a low-volume residential environment, which inherently reduces crash severity risk.

During construction, the temporary construction compound (refer Section 5.2.1) ensures that most construction-related activities are contained within the site boundary. The compound has been designed to accommodate:

- Construction vehicle parking within designated areas
- Container and material storage areas
- Machinery and plant storage
- Defined U-turn and manoeuvring points within Stage 3 and Stage 4
- Internal haul routes separated from public traffic

This arrangement significantly reduces interaction between construction vehicles and the public road network, thereby minimising exposure to crash risk.

6.6.1.2 Safe Speeds

The surrounding network is characterised by a low-speed residential environment (50 km/h), supported by low traffic volumes and local road function.

During construction, speed-related risks are mitigated through:

- Internal site speed controls for construction vehicles
- Driver and operator induction procedures
- Controlled access via Colorado Road
- Use of spotters during high-risk manoeuvres such as reversing and loading

Given the limited construction traffic volumes (approximately 26 vehicle trips per day), the development is not expected to materially influence operating speeds on the surrounding network.

6.6.1.3 Safe Road Users

The surrounding area currently experiences low pedestrian and cyclist activity due to the partially developed nature of the site. As development progresses, pedestrian activity is expected to increase; however, safety is supported by:

- Provision of footpaths within the development
- Adequate carriageway widths (approximately 8.5 m to 10.8 m)
- Low traffic volumes on surrounding roads

During construction, potential conflicts are minimised through:

- Internalisation of construction activities within the site
- Limited interaction between construction vehicles and footpaths
- Use of spotters at key pedestrian interaction points where required

6.6.1.4 Safe Vehicles

Construction activities will involve a range of vehicles and machinery (refer Section 4.4.1), including light vehicles, heavy vehicles, and plant equipment.

These include:

- Light vehicles such as service vehicles
- Heavy vehicles such as dump trucks, concrete trucks, and delivery vehicles
- Plant and machinery including excavators, bulldozers, rollers, loaders, and graders

A key safety outcome is that these vehicles will predominantly operate within the site boundary, with heavy vehicle movements largely limited to the initial mobilisation phase.

Vehicle movements are managed through:

- Internal circulation within the construction compound
- Compliance with the Construction Traffic Management Plan (CTMP)
- Use of trained personnel for traffic management where required

6.6.1.5 Safe System Outcome

The integration of the temporary construction compound, internal vehicle circulation, and controlled access arrangements ensures alignment with Safe System principles.

The development:

- Minimises exposure to high-risk vehicle interactions

- Maintains a low-speed, low-volume road environment
- Effectively manages construction traffic within the site

While a marginal increase in crash risk (approximately 3–5%) is predicted at the SH2 intersection, this is primarily attributable to background traffic growth rather than the development itself.

Overall, the transport network is expected to remain safe and fit for purpose, with no additional Safe System interventions required beyond standard design and operational controls.

6.6.2 Safe Sight Distance

6.6.2.1 Construction Phase Sight Distance

Construction access via Colorado Road has been assessed with consideration of the temporary construction compound (Section 5.2.1), vehicle types (Section 4.4.1), and vehicle tracking (Section 5.3.1).

The construction compound provides sufficient space and layout to ensure safe visibility and manoeuvring, including:

- Adequate internal space for turning and circulation
- Dedicated U-turn areas within Stage 3 and Stage 4
- Separation of parking, storage, and vehicle movement areas
- Forward entry and exit movements for heavy vehicles
- Minimal likelihood of queuing on Colorado Road

Vehicle tracking assessment (refer Section 5.3.1) confirms that:

- Heavy vehicles (including truck and trailer configurations) can safely access and exit the site
- All turning movements can be accommodated within the site boundary
- No reversing onto the public road network is required
- No encroachment into opposing lanes or pedestrian paths is expected

Colorado Road is a low-volume road with good forward visibility and no significant geometric constraints. Accordingly, adequate sight distance is available in both directions for construction vehicle movements.

Where temporary visibility constraints may arise (e.g. parked vehicles or material storage), mitigation measures will include:

- Use of trained spotters
- Temporary signage at access points
- Management of delivery timing to avoid peak periods

6.6.2.2 Operational (Post-Development) Sight Distance

Following completion of the development, all roads and intersections will comply with NZS 4404:2010, which incorporates appropriate sight distance requirements for residential environments.

The following outcomes are achieved:

- Adequate stopping sight distance for a 50 km/h design speed
- Safe intersection sight distance at all internal intersections
- Satisfactory visibility at the Colorado Road connection
- No adverse vertical or horizontal alignment constraints

Given the low traffic volumes on the surrounding road network, the available sight distance is considered appropriate for safe operation.

6.6.2.3 Pedestrian Visibility

Pedestrian visibility is supported by:

- Open residential environment with minimal visual obstructions
- Low traffic volumes allowing safe gap selection
- Defined pedestrian routes within the development

During construction, pedestrian visibility will be maintained through:

- Clear delineation of construction areas
- Avoidance of material storage near access points
- Use of spotters where pedestrian interaction may occur

6.6.2.4 Summary – Sight Distance

The assessment confirms that adequate sight distance is available for both construction and operational phases of the development.

The combination of:

- Internal construction compound layout
- Controlled vehicle circulation
- Low-volume road environment

ensures that visibility-related risks are minimised.

No additional sight distance mitigation measures are required, and the transport network will continue to operate safely.

6.6.3 Crash Record Analysis

The New Zealand Transport Agency (NZTA) Crash Analysis System (CAS) has been reviewed from 2020 to 2025 to identify crash history along some major road corridor, intersection, that are the main access to the new development over the past 5 years. The system shows the following record:

Table 6-21 Crash Record Analysis - Tōtara Park Rd at California Dr

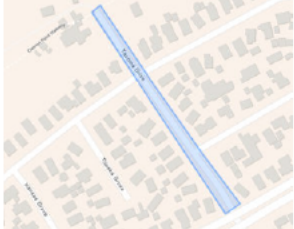
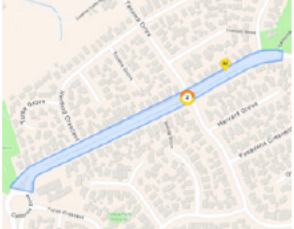
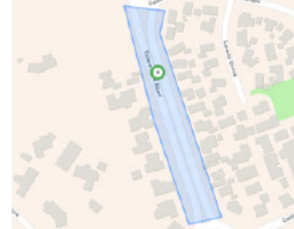
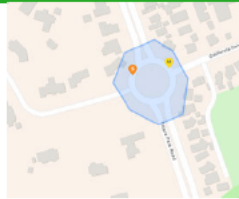
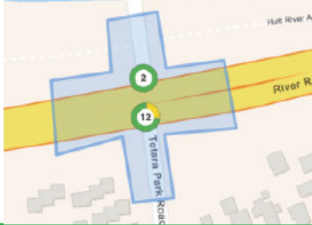
Road Name	Tacoma Drive	California Drive	Tōtara Park Rd North
Severity			
Fatal	0	0	0
Serious	0	1	0
Minor Injury	0	2	0
Non-Injury	0	0	2
Total	0	3	2

Table 6-22 Crash Record Analysis - Tōtara Park Rd at Turon Cres

Road Name	Tōtara Park Rd/ Turon Cres Roundabout	Tōtara Park Rd/California Dr Roundabout	Tōtara Park Rd South
Severity			
Fatal	0	0	0

Serious	0	1	0
Minor Injury	0	1	0
Non-Injury	0	0	1
Total	0	2	1

Table 6-23 Crash Record Analysis – Tōtara Park Rd at SH2

Road Name	Tōtara Park Rd/ SH 2 Intersection	
Severity		
Fatal	0	
Serious	0	
Minor Injury	3	
Non-Injury	11	
Total	14	

6.6.4 Safety Assessment (Crash Model)

A crash model has been completed to assess the effect of the development on the surrounding network; the method is based on the Surrogate Safety Assessment Model and Validation introduced by The Safety Model uses the model output to predict the actual conflict numbers and pattern of the crashes. The crash model shows that it is unlikely that there be a significant change in crash risks associated with the proposed development traffic.

The following logic were applied:

Often roundabouts do not have the roads with the highest or lowest volumes on opposing arms, or if they have 3 arms these are seldom in a 'T' type layout. Therefore, crash rates are calculated for each arm of the roundabout, and the total obtained by adding these together. The typical crash rate (reported injury crashes per approach per year) is calculated using the model:

$$A_T = b_0 \times Q_{\text{approach}}^{b_1}$$

where: Q_{approach} is the 2-way link volume (AADT) on the approach being examined.

b_0 , and b_1 are given in [Table 7-4](#)

This model can be applied for roundabouts with 3, 4 or 5 approaches.

Table 7-4: general urban roundabouts 50–70km/h coefficients (reference 5)

Number of entry lanes per approach	Single		Multiple	
	b_0	b_1	b_0	b_1
Roundabout	4.43×10^{-4}	0.58	7.95×10^{-4}	0.58

For urban intersections on the primary road network (excluding roundabouts), the typical crash rate (reported injury crashes per year) is calculated using:

$$A_T = b_0 \times Q_{\text{major}}^{b_1} \times Q_{\text{minor/side}}^{b_2} \times \prod \text{CMFs}$$

where: Q_{major} is the highest 2-way link volume (AADT) for crossroads and the primary road volume for T-junctions.

$Q_{\text{minor/side}}$ is the lowest of the daily 2-way link volumes (AADT) for crossroads and the side road flow for T-junctions

b_0 , b_1 and b_2 are given in [Table 7-2](#)

CMF is crash modification factors for options

$\prod$ is product of

CRASH ESTIMATION COMPENDIUM | SECOND EDITION, VERSION 3, MAY 2025 // 27

Table 7-2: general crossroad and T-junction urban intersections (50–70km/h) coefficients (reference 21)

Intersection type	b_0	b_1	b_2
Uncontrolled – T	2.08×10^{-3}	0.19	0.36
Priority – cross	1.13×10^{-3}	0.51	0.21
Priority – T	4.68×10^{-5}	0.20	0.76
Traffic signals – cross	2.26×10^{-3}	0.14	0.46
Traffic signals – T	1.21×10^{-1}	0.12	0.04

The following intersection and roundabout were assessed:

- Tōtara Park Rd/SH2 Signalised Intersection
- Tōtara Park Rd/ Turon Cres Roundabout
- Tōtara Park Rd/California Dr Roundabout

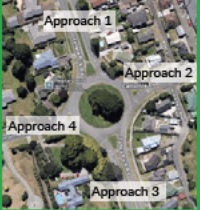
Table 6-24 Tōtara Park/ SH2 Crash Model Result

	Tōtara Park Rd/SH2 Signalised Intersection	Q major	Q minor	b_0	b_1	b_2	Predict Injury Pedestrian
	Base (2025)	19716	5775	0.00226	0.14	0.46	0.48
	Do Min (2032)	20904	5988	0.00226	0.14	0.46	0.50
	Option 4 (2032)	21339	6234	0.00226	0.14	0.46	0.51

Table 6-25 Tōtara Park Rd/ Turon Cres Crash Model Result

	Tōtara Park Rd/ Turon Cres Roundabout	Q Approach 1	Q Approach 2	Q Approach 3	b_0	b_1	Predict Injury Pedestrian
	Base (2025)	2271	0	2271	0.000443	0.58	0.08
	Do Min (2032)	3918	309	4212	0.000443	0.58	0.12
	Option 4 (2032)	4092	327	4416	0.000443	0.58	0.13

Table 6-26 Tōtara Park Rd/ California Dr Crash Model Result

	Tōtara Park Rd/California Dr Roundabout	Q Approach 1	Q Approach 2	Q Approach 3	Q Approach 4	b0	b1	Predict Injury Pedestrian
	Base (2025)	2271	3969	6462	225	0.000443	0.58	0.18
	Do Min (2032)	3054	3936	8388	225	0.000443	0.58	0.19
	Option 4 (2032)	4416	4056	8697	225	0.000443	0.58	0.21

6.7 Transport Network Alignment

The proposed development has been assessed in the context of the existing transport network and relevant strategic planning frameworks. Based on the Upper Hutt City Council RAMM data and One Network Framework (ONF) classification, the surrounding road network is predominantly classified as **Low Volume Roads**, with no National, Regional, Arterial, or Primary Collector routes located within the immediate vicinity of the site.

Given the localised nature of the surrounding network, the development is not expected to impact the strategic function or level of service of the wider transport network. Vehicle access to the development is provided via Colorado Road and the internal road network, which are consistent with the function of local residential roads and are appropriate for the anticipated traffic volumes.

The proposed development does not introduce any changes to the strategic transport network, nor does it require any plan changes, new arterial or collector routes, or significant upgrades to existing infrastructure. The access arrangements and internal road layout are considered to align with the intended function of the surrounding network and are not expected to adversely affect network performance.

In terms of active modes, the development does not create or rely on any strategic walking or cycling routes. There are no identified deficiencies or missing links in the walking or cycling network within the immediate area. The surrounding environment is characterised by a low-speed, low-volume road network where pedestrians and cyclists can be safely accommodated.

Overall, the proposed development is consistent with the existing transport network hierarchy and function and is not expected to result in adverse effects on the performance or strategic role of the surrounding transport network.

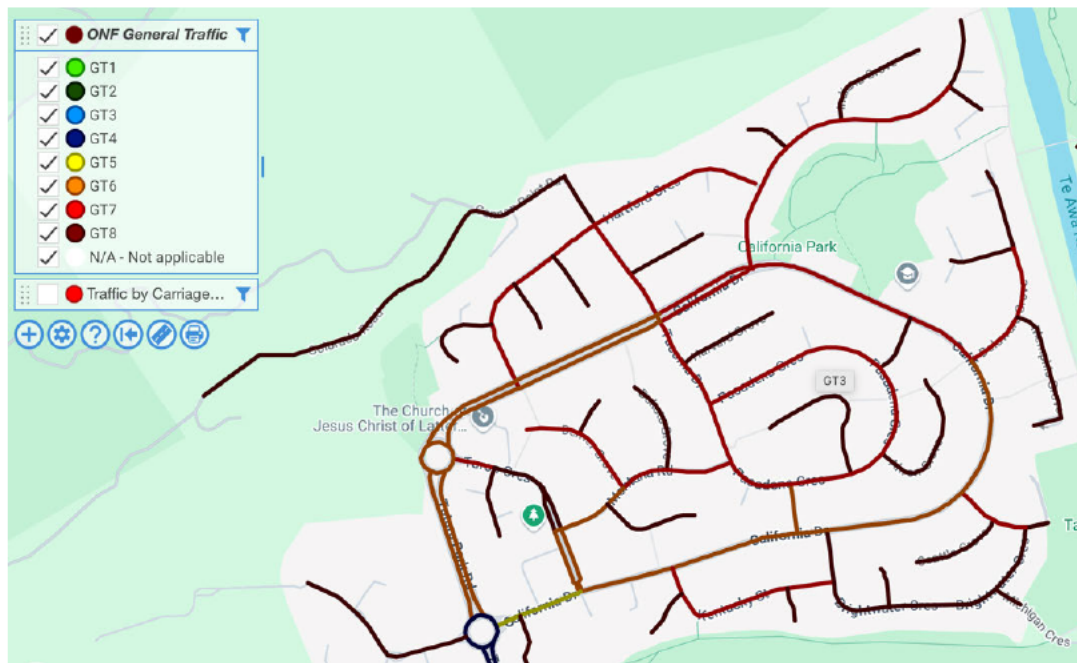


Figure 6-12: One Network Framework (ONF) Map

6.8 Climate Adaptation Effects

Environmental Assessment was conducted made in reference to the Vehicle Emission Prediction Model by The NZ Transport Agency. The model predicts emissions from vehicles in the New Zealand fleet under typical road, traffic and operating conditions.

1.1 VEPM

To assess the air quality effects of road projects, or changes in vehicle technology or fleet characteristics it is necessary to estimate emissions from motor vehicles. This can be achieved using emission factors.

Emissions factors are the quantity of pollutants emitted per kilometre driven. Vehicle emissions are primarily dependent on the vehicle type and fuel. Vehicle emissions are also dependent on the driving conditions. For example, emissions are different for any given vehicle under acceleration or deceleration, at different speeds and engine loads.

VEPM is an average speed model which predicts emission factors for the New Zealand fleet under typical road, traffic and operating conditions. Average speed models are based on the fact that the average emissions factor for a pollutant and vehicle type/technology varies as a function of the average speed during a trip.

VEPM calculates New Zealand fleet weighted emission factors for:

- Exhaust emission factors for harmful pollutants:
 - Particulate matter ($PM_{2.5}^3$)
 - Nitrogen oxides (NO_x)
 - Nitrogen dioxide (NO_2)
 - Volatile organic compounds (VOC)
 - Carbon monoxide (CO)
- Exhaust emission factors for greenhouse gases:
 - Carbon dioxide (CO_2)
 - Methane (CH_4)
 - Nitrous oxide (N_2O)
 - Carbon dioxide equivalent (CO_2e), which is calculated from CO_2 , CH_4 and N_2O factors
- Brake, tyre and sealed road wear factors for particulate matter smaller than $10\ \mu m$ (PM_{10}) or smaller than $2.5\ \mu m$ ($PM_{2.5}$)

1.2 Purpose of VEPM

VEPM has been developed to quantify vehicle emissions and predict how these are likely to change over time. The model can estimate the effect that new technology and improved fuel will have on emissions from New Zealand's vehicle fleet. An important feature of the model is the ability to estimate changes to vehicle emissions in future years (from 2001 to 2055).

VEPM provides estimates that are suitable for air quality assessments, greenhouse gas assessments and emissions inventories. VEPM has been successfully used around New Zealand to estimate vehicle emissions in air quality and greenhouse gas assessments for road projects. The model provides vehicle emission factors, which are used in conjunction with traffic models and air dispersion models to predict air pollutant concentrations downwind of the road.

VEPM emission factors are used in NZTA project evaluation tools and methods including, for example the monetised benefits and costs manual (MBCM) and the Air quality screening model⁵, which is a simple online tool to assist with undertaking preliminary or screening assessments. VEPM is also used in more detailed assessments. A schematic of the detailed assessment process using VEPM is shown in Figure 1. The NZTA guide to assessing air quality impacts from state highway projects⁶ provides specific guidance

⁴ For example, the developers of COPERT used a similar process to develop emission factors for Cyprus where the fleet is dominated by Japanese used imports.

⁵ [Air quality screening model \(nzta.govt.nz\)](https://www.nzta.govt.nz/air-quality/screening-model)

⁶ [Air quality assessment guide - October 2019 \(nzta.govt.nz\)](https://www.nzta.govt.nz/air-quality/assessment-guide)

The following is the emission result across multiple scenarios:

- Base Model (2025)
- Do Min (2032)
- Option 4 (2032)

Base (2025)	Days / year	Emissions (g)	Unit Cost (\$/tonne)	Cost (\$/year)
PM _{2.5} ,	240	474.69	\$853,824	\$97,273
NOx	240	8766.94	\$865,797	\$1,821,694
CO	240	9457.27	\$5	\$11
Volatile organic compounds	240	682.37	\$1,545	\$253
Total				\$1,919,231

Do Min (2032)	Days / year	Emissions (g)	Unit Cost (\$/tonne)	Cost (\$/year)
PM _{2.5} ,	240	373.22	\$853,824	\$76,479
NOx	240	5854.46	\$865,797	\$1,216,505
CO	240	5417.81	\$5	\$6
Volatile organic compounds	240	285.41	\$1,545	\$106
Total				\$1,293,096

Option 4 Forecast (2032)	Days / year	Emissions (g)	Unit Cost (\$/tonne)	Cost (\$/year)
PM _{2.5} ,	240	407.53	\$853,824	\$83,509
NOx	240	6277.92	\$865,797	\$1,304,498
CO	240	5841.89	\$5	\$7
Volatile organic compounds	240	303.61	\$1,545	\$113
Total				\$1,388,126

Based on the VEPM modelling results, the Base Model (2025) is estimated to have a total emission cost of approximately \$1.92M per year.

The Do Minimum (2032) scenario builds on the same base model, including Stages 1 and 2 of the development, and increased traffic demand results in a reduction in total emission cost, with costs decreasing to around \$1.29M per year. This indicates a short-term increase in emissions compared to the base case.

The reduction in emission costs observed between the Base 2025 and Do Minimum 2032 scenarios is primarily driven by the increased adoption of electric and low-emission vehicles in the fleet, as projected in the VEPM 7.2 default fleet profile. This is most directly reflected in the CO₂ emission factor, resulting in other exhaust pollutants (PM_{2.5}, NOx, CO, and VOC) also benefiting from the fleet transition.

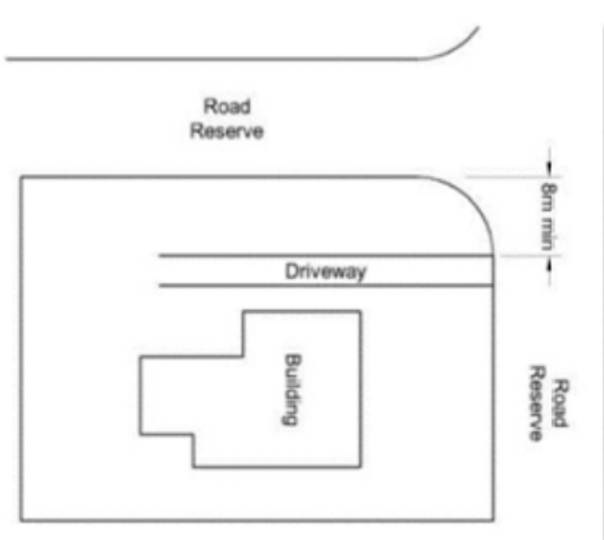
In the future scenario (Option 4, 2032) with the proposed Stages 3 and 4 of the development, the total emission is expected to increase slightly to 1.39M per year.

The \$0.1M increase between the Do Min 2032 and Option 4 2032 scenario is considered relatively modest and broadly proportionate to the scale of growth. Mitigation such as cycle parking, rain gardens, and planting could also be considered to help offset the development's environmental effects.

7. District Plan Considerations

The proposed development is located within Upper Hutt, therefore is required to meet the provisions set out in Part 2 'District-Wide Matters – 'Transport and Parking' of the District Plan. Table 7-1 lists the relevant rules and whether the proposed subdivision complies with the District Plan Requirements.

Table 7-1 District Plan Compliance Summary

TP-S1	Proposed	Compliance
All accessways and manoeuvring areas shall be formed and surfaced in accordance with the Code of Practice for Civil Engineering Works. Exemption – the requirement for accessways serving sites solely occupied by unstaffed utilities shall be that the accessway shall be surfaced with permanent all-weather surfacing for a minimum length of 5m from the edge of the road carriageway seal.	Ample space for on-site manoeuvring and will be formed in concrete	Complies.
Sites shall have practical vehicle access to car parking and loading spaces (where provided or required), in accordance with the Code of Practice for Civil Engineering Works. This requirement does not apply to sites solely occupied by unstaffed utilities, provided that vehicles associated with utilities shall not obstruct the footpath or create a traffic hazard on the road.	Vehicle access is acceptable and will be in accordance with UHCC CoP	
Vehicular access to a corner allotment shall be located no closer than 8m from the street corner. Where a site is located on an intersection of a primary or secondary arterial traffic route (as identified in the Transport and Parking (TP) Chapter) the siting of the vehicular access shall be located as far as practicable from the corner of the street. The 8m setback shall be measured from where the two front boundaries of the site (refer to the definition of a corner allotment) join, or in accordance with the diagram below. 	N/A	
Where a corner allotment is located at an intersection of a national primary or secondary arterial traffic route, as identified in TP-SCHED 1- Roding Hierarchy, no building, fence or other structure		Not Applicable