October 1, 2021

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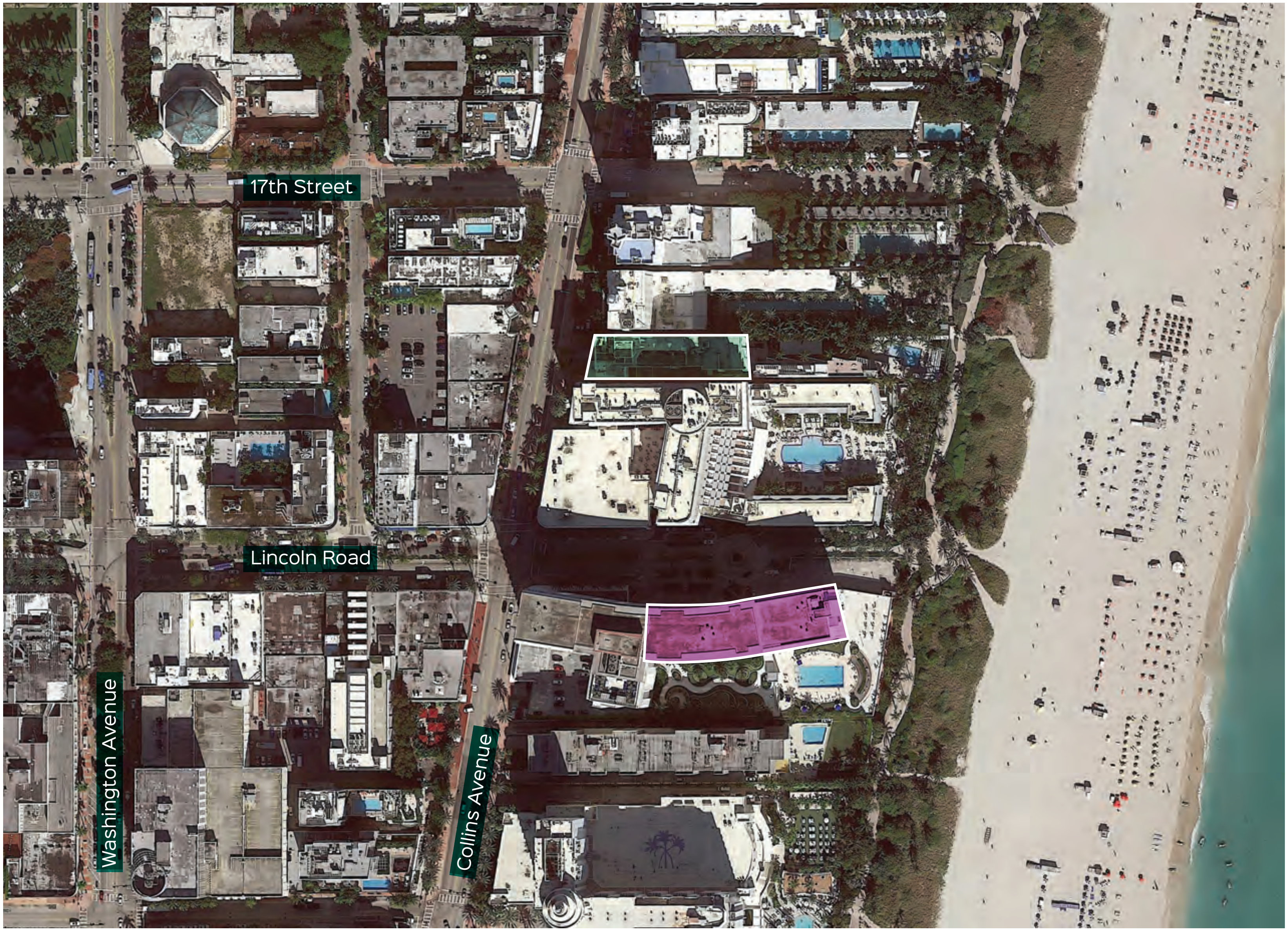
RE: Sagamore Hotel Conditional Use Permit Traffic Analysis - #21215

Dear Firat,

The Sagamore Hotel is located at 1671 Collins Avenue in Miami Beach, Florida. The hotel is seeking a Conditional Use Permit (CUP) for events in the existing restaurant and pool area. The traffic caused by the events will utilize the existing valet drop-off / pick-up service area located along Collins Avenue at the entrance to the Sagamore Hotel. The valet will park within an existing parking garage located at 100 Lincoln Road. (See Exhibit 1 for the location of the hotel and valet parking garage).

The purpose of this traffic statement is to conduct a trip generation analysis and valet queuing analysis for the proposed events, and an intersection capacity analysis for the Collins Avenue / Lincoln Road intersection. At the request of the City, a valet queuing analysis was performed for two different types of events that may be hosted by the Sagamore Hotel; a sit-down event that can accommodate 142 seats and a 582-person event (maximum occupancy of the event space). See Attachment A for the site plan with the proposed event space and maximum occupancy. Trip generation analyses were performed to estimate the trips generated by the possible events. Inbound and outbound queuing analyses were performed for each event to anticipated the expected vehicle queues and valet demand for each event. At the request of the City, an intersection capacity analysis for the Collins Avenue / Lincoln Road intersection was performed for a maximum occupancy event.



Project Location

Valet Parking

Exhibit 1

Location Map

NORTH
MAP NOT TO SCALE

Sit-down Event Trip Generation

A trip generation analysis was conducted for the proposed sit-down events hosted by the Sagamore Hotel event space. The project trip generation was based on the rates/equations published by the Institute of Transportation Engineers (ITE) *Trip Generation Manual*, 10th Edition. Land Use 931, Quality Restaurant was used in the analysis. A 20% reduction for other modes of transportation was applied at the request of the City. Trip generation calculations were performed for a typical weekday daily, AM and PM peak hours of the adjacent street and were compared to the trips generated by the existing restaurant within the Hotel. Trip generation for the proposed and existing restaurant are summarized in Exhibit 2. Support documentation is provided in Attachment B.

Exhibit 2: Sit-Down Event Trip Generation

Proposed ITE Land Use Designation[1]	Number of Units	Daily Vehicle Trips	AM Peak Hour Vehicle Trips			PM Peak Hour Vehicle Trips		
			In	Out	Total	In	Out	Total
Quality Restaurant *Land Use Code: 931*	434 Seats	1,246	4	4	8	81	40	121
Total Gross Trips		**1,246**	**4**	**4**	**8**	**81**	**40**	**121**
Internalization with Existing Hotel (AM,PM)[2]	(0, 8.2%)		0	0	0	-4	-3	-7
Other Modes of Transportation[3]		20%	-1	-1	-2	-15	-7	-22
Quality Restaurant Pass-by (PM)[4]		34%	0	0	0	-16	-16	-32
Net Proposed Trips			**3**	**3**	**6**	**46**	**14**	**60**

Existing ITE Land Use Designation[1]	Number of Units	Daily Vehicle Trips	AM Peak Hour Vehicle Trips			PM Peak Hour Vehicle Trips		
			In	Out	Total	In	Out	Total
Quality Restaurant *Land Use Code: 931*	142 Seats	106	1	1	2	27	13	40
Total Gross Trips		**106**	**1**	**1**	**2**	**27**	**13**	**40**
Internalization with Existing Hotel (AM,PM)[2]	(0, 4.5%)		0	0	0	-1	-1	-2
Other Modes of Transportation[3]		20%	0	0	0	-5	-2	-7
Quality Restaurant Pass-by (PM)[4]		34%	0	0	0	-5	-5	-10
Net Existing Trips			**1**	**1**	**2**	**16**	**5**	**21**

Exhibit 2 (Continued): Trip Generation

Net New External Trips	Daily Vehicle Trips	AM Peak Hour Vehicle Trips			PM Peak Hour Vehicle Trips		
		In	Out	Total	In	Out	Total
Proposed	**1,246**	**3**	**3**	**6**	**46**	**14**	**60**
Existing	**106**	**1**	**1**	**2**	**16**	**5**	**21**
Difference	**1,140**	**2**	**2**	**4**	**30**	**9**	**39**

[1] Based on ITE Trip Generation Manual, 10th Edition.

[2] Iternalization rate with the Existing 100 room Sagamore Hotel (see attachment B)

[3]Based on data provided by the City

[4]Based on ITE Trip Generation Handbook, 3rd Ed.

The results of the analysis show that the sit-down events hosted by the Sagamore Hotel will generate an increase of 1,140 daily vehicle trips and 4 and 39 vehicle trips during morning and afternoon peak hours respectively.

Sit-Down Event Valet Queuing Analysis

As previously stated, the events will utilize the existing Sagamore Hotel drop-off / pick-up area at the entrance of the Sagamore Hotel lobby; accessed via a one-way Semi-circular driveway along Collins Avenue. The area will be utilized as both the valet pick-up / drop-off area and the rideshare pick-up / drop-off area. The valet parking will be located within a parking garage located on 100 Lincoln Road. Inbound / outbound queuing analyses were performed for the valet station.

As the drop-off / pick-up area will be utilized as both the rideshare and valet pick-up / drop-off area inbound / outbound queuing analyses were performed for the rideshare vehicles and the valet station utilizing the pick-up / drop-off area to determine if the combined queues will spill back onto Collins Avenue.

The queuing analysis for the proposed valet drop-off / pick-up area was performed based on the methodology outlined in the *Institute of Transportation Engineers (ITE) Transportation and Land Development*. The analysis was performed to determine the number of valet parking attendants required during the peak hour so that the queue does not extend past the valet storage area (95%

confidence level analysis). The potential queues were calculated based on the peak hour traffic published by the Institute of Transportation Engineers (ITE) trip generation rates and/or equations.

A trip generation was performed to calculate the rideshare demand and the demand at the valet station for a sit-down event during the morning and afternoon peak hours. A 20% reduction for other modes of transportation and a 44% rideshare reduction were applied at the request of the City. The proposed trip generation is summarized in Exhibit 3. Trip generation documentation is available in Attachment B.

Exhibit 3: Sit-Down Event Valet Trip Generation

ITE Land Use Designation[1]	Number of Units	Daily Vehicle Trips	AM Peak Hour Vehicle Trips			PM Peak Hour Vehicle Trips		
			In	Out	Total	In	Out	Total
Quality Restaurant *Land Use Code: 931*	434 Seats	1,246	4	4	8	81	40	121
Total Gross Trips		**1,246**	**4**	**4**	**8**	**81**	**40**	**121**
Internalization with Existing Hotel (AM,PM)[2]		(0, 8.2%)	0	0	0	-4	-3	-7
Other Modes of Transportation[3]		20%	-1	-1	-2	-15	-7	-22
Net Proposed Trips			**3**	**3**	**6**	**62**	**30**	**92**
Rideshare Reduction[4]		44%	-1	-1	-2	-27	-13	-40
Trips @ Valet Station			**2**	**2**	**4**	**35**	**17**	**52**

[1] Based on ITE Trip Generation Manual, 10th Edition.

[2] Iternalization rate with the Existing 100 room Sagamore Hotel (see attachment B)

[3] Based on data provided by the City

[4] Rideshare reduction provided by the City

The results of the trip generation show that the critical peak hour for the rideshare demand and valet parking is the PM peak hour with a total of 40 trips using rideshare services and 52 vehicle trips (in/out) utilizing the valet services.

A queuing analysis for the proposed drop-off / pick-up area was performed to determine the number of valet parking attendants required during the peak hour so that the queue does not extend past the valet storage area (95% confidence level analysis). The queuing analysis was performed based on the methodology outlined in the *Institute of Transportation Engineers (ITE) Transportation and Land Development*. The potential queues were calculated based on the peak hour traffic published by the Institute of Transportation Engineers (ITE) trip generation rates and/or equations (see Exhibit 3).

The queuing analysis used the single-channel waiting line model with Poisson arrivals and exponential service times. The analysis is based on the coefficient of utilization (ρ) which is the ratio of the average arrival rate of vehicles to the average service rate.

$$\rho = \frac{Average\ Demand\ Rate}{Average\ Sevice\ Rate}$$

The average service rate corresponds to the time it will take a passenger to enter / exit the rideshare vehicle or the time it will take a valet parking attendant to park or retrieve a vehicle. If the coefficient of utilization is greater than 1, then the calculation will yield an infinite queue length.

The required queue storage (M) is determined using the following equation:

$$M = \left[\frac{\ln P(x > M) - \ln Q_M}{\ln \rho}\right] - 1$$

In this equation, $P(x > M)$ is set at 5% to yield a 95% confidence that the queue will not back-up onto the adjacent street.

A rideshare processing rate of 30 seconds per vehicle was provided by the City. The valet processing rates were calculated by adding the time it will take a valet attendant to process the vehicles (**processing time**), the time it will take the attendant to circulate to the parking space (**driving time)**, The time it will take the valet to enter / exit the parking garage's mechanical arm gate (**mechanical arm gate lift time)**, the time it will take the attendant to park or retrieve a vehicle (**park processing time**), and the time it will take the attendant to walk to/from the parking area (**walking time**).

A processing time of 60 seconds per vehicle was used in the analysis. This information was provided by the City of Miami Beach. The driving time for the valet attendant was calculated on a conservative speed of 15 mph, and the walking time for the valet attendant was calculated on a jogging speed of 5 ft / sec (provided by the City). Since the processing time for the valet parking differs for the inbound / outbound parking, a weighted average was taken of the inbound / outbound valet processing time. The weighted average was based on the inbound / outbound trip distribution, which is 67% inbound parking and 33% outbound parking. The processing rate for the valet drop-off / pick-up during the PM peak hour can be seen in Exhibit 5.

Exhibit 5: Valet Station Processing Rate (PM Peak Hour)
Valet Drop-off / Pick-up

<u>**Valet Time (Inbound)**</u>

***Processing time*:**	60 sec / 60 sec / 1 min = ***1.00 min***
***Driving time*:**	2,270 ft * 1 mile / 5,280 ft * 1hr / 15 miles * 60 min / hr = ***1.72 min***
Mechanical arm gate lift time:	4.25 sec / arm 60 sec / 1 min = ***0.7 min***
***Park Processing Time*:**	= ***0.15 min***
Walking time:	1,210 ft / 5 ft / sec / 60 sec / min = ***4.03 min***
Total	**= <u>6.97 min</u>**

<u>**Valet Time (Outbound)**</u>

***Processing time*:**	60 sec / 60 sec / 1 min = ***1.00 min***
***Driving time*:**	1,200 ft * 1 mile / 5,280 ft * 1hr / 15 miles * 60 min / hr = ***0.91 min***
Mechanical arm gate lift time:	4.25 sec / arm 60 sec / 1 min = ***0.7 min***
***Park Processing Time*:**	= ***0.15 min***
Walking time:	1,210 ft / 5 ft / sec / 60 sec / min = ***4.03 min***
Total	**= <u>6.16 min</u>**

<u>**Weighted Valet Time**</u>

67% Inbound:	0.67*6.97 min = ***4.67 min***
33% Outbound:	0.33*6.16 min = ***2.03 min***
Total	**= <u>6.70 min</u>**

As the valet pick-up / drop-off area will serve both the rideshare and valet trips to the site; a weighted average was taken of the valet / rideshare processing times. The weighted average was based on the valet / rideshare trip distribution, which is 56% valet and 44% rideshare. Exhibit 6 shows the weighted processing rate at the valet station.

Exhibit 6: Valet Station Processing Rate – Weighted Valet parking / Rideshare Dropoff

Weighted Valet Time	
56% Valet Parking	0.56 * 6.70 min = ***3.76 min***
44% Rideshare:	0.44 * 0.5 min = ***0.22 min***
Total	= **3.98 min**

An iterative approach was used to determine the minimum number of valet attendants required to serve the entering and exiting vehicles during the PM peak hour (critical hour) that will ensure that the average queue at the valet station will not extend past the pick-up / drop-off storage area. Exhibit 7 shows the calculations for the inbound / outbound valet drop-off / pick-up area during the AM peak hour.

Exhibit 7: Valet Station Queuing Calculations

Q = Processing Rate = $\frac{60\ min/hr}{3.98\ min/process} = 15.09\ process/hr$

q = Demand Rate = $92\,\frac{veh}{hr}$

N = Service Positions = 9 Attendants

ρ = Utilization factor = $\frac{q}{(NQ)} = \frac{92\ veh/hr}{9 \times 15.09\ process/hr} = 0.6773$

Q_m = Table Value = 0.2177

M = queue length which is exceeded 5% of the time [P(x>M)]

$$M = \frac{\ln P(x>M) - \ln(Q_m)}{\ln(\rho)} - 1 = \frac{\ln(0.05) - \ln(0.2177)}{\ln(0.6773)} - 1 = 2.76,\ say\ 3 \text{ Vehicles on queue}$$

The results of the analysis show that a total of 9 valet attendants would be able to handle the demand during the PM peak hour at the valet station with an average queue of approximately three vehicle or less. Based on the site plan, the Sagamore Hotel driveway has approximately 100 feet of storage. This distance is enough to accommodate the three-vehicle queue produced by the rideshare and valet drop-off / pick-up operations.

Maximum Occupancy Event Trip Generation

A trip generation was conducted utilizing the maximum occupancy allowed within the proposed Sagamore Hotel event space. Trip generation for the project was based on the maximum number of attendees (582 persons) allowed within the event space per the fire department. In order to quantify the vehicle trips, percentages and rates were applied based on data provided by the City of Miami Beach and engineering judgement. Percentages and rates include maximum occupancy allowed by the fire department (Shown in Attachment A), percent of attendees that are internal (hotel guests), vehicle occupancy, percent of trips arriving during the peak hour, percent of trips arriving by transit, and percent of attendees arriving through rideshare vehicle. The calculations for the events assumed that not all of the attendees would arrive simultaneously. Exhibit 8 shows the calculations and trip generation for the proposed event space. The results of the analysis show that the maximum occupancy events hosted by the Sagamore Hotel will generate 72 vehicle trips and 56 rideshare vehicle trips during the arrival and dismissal of the event.

Exhibit 8: Maximum Occupancy Event Trip Generation

Proposed Event Trip Generation			
Calculations	**Event Space**	**Percentages / Rates Applied**	
Number of Attendees	582	Maximum Expected Event Occupancy	
Attendees - Internal	48	8.2%	Internal[1]
Attendees - External	534	91.8%	External
External Vehicle trips	267	2	Persons/Vehicle[2]
Peak Hour Trips	160	60%	Arrive/Depart during the peak hour
Alternative Transport Trips	32	20%	of Peak Hour Trips[2]
Arriving by vehicles Trips	128	80%	of Peak Hour Trips
Total Peak Hour Vehicle Trip	128		
Rideshare Trips	56	44%	Rideshare reduction[2]
Total Valet Trips	72		

[1]Based on ITE internalization rates between the hotel and restaurant approved by Miami Beach

[2]Based on Information Provided by Miami Beach

Maximum Occupancy Event Queuing Analysis

As previously stated, the existing Sagamore Hotel entrance drop-off / pick-up area will be utilized as both the valet and the rideshare pick-up / drop-off area for the events. Event arrival and departure queuing analyses were performed for the maximum occupancy event to ensure that the combined queues from the valet station and the rideshare vehicles utilizing the pick-up / drop-off area will not spill back onto Collins Avenue.

A queuing analysis was performed based on the methodology outlined in the *Institute of Transportation Engineers (ITE) Transportation and Land Development*. The queuing analysis was performed to determine the queue generated by the arrival / departure of the vehicles to/from the event. The queuing analysis was performed based on the calculated trip generation shown in Exhibit 8.

The queuing analysis used the single-channel waiting line model with Poisson arrivals and exponential service times. The analysis is based on the coefficient of utilization (ρ) which is the ratio of the average arrival rate of vehicles to the average service rate.

$$\rho = \frac{Average\ Demand\ Rate}{Average\ Sevice\ Rate}$$

The average service rate corresponds to the time it will take a passenger to enter / exit the rideshare vehicle or the time it will take a valet parking attendant to park or retrieve a vehicle. If the coefficient of utilization is greater than 1, then the calculation will yield an infinite queue length.

The required queue storage (M) is determined using the following equation:

$$M = \left[\frac{\ln P(x > M) - \ln Q_M}{\ln \rho}\right] - 1$$

In this equation, *P(x > M)* is set at 5% to yield a 95% confidence that the queue will not back-up onto the adjacent street.

A rideshare processing rate of 30 seconds per vehicle was provided by the City. The processing rates for the valet parking were calculated by adding the time it will take a valet attendant to process the vehicles (**processing time**), the time it will take the attendant to circulate to the parking space (**driving time)**, The time it will take the valet to enter / exit the parking garage’s mechanical arm gate (**mechanical arm gate lift time)**, the time it will take the attendant to park or retrieve a vehicle (**park processing time**), and the time it will take the attendant to walk to/from the parking area (**walking time**).

A processing time of 60 seconds per vehicle was used in the analysis. This information was provided by the City of Miami Beach. The driving time for the valet attendant was calculated on a conservative speed of 15 mph, and the walking time for the valet attendant was calculated on a jogging speed of 5 ft / sec (provided by the City). The processing rate for the valet station for the start of the event (inbound trips) can be seen in Exhibit 9. The processing rate for the valet station for the end of the event (outbound trips) can be seen in Exhibit 10.

Exhibit 9: Inbound Valet Station Processing Rate
Valet Drop-off / Pick-up

Event Arrival Valet Time (Inbound)

***Processing time*:**	60 sec / 60 sec / 1 min = ***1.00 min***
***Driving time*:**	2,270 ft * 1 mile / 5,280 ft * 1hr / 15 miles * 60 min / hr = ***1.72 min***
Mechanical arm gate lift time:	4.25 sec / arm 60 sec / 1 min = ***0.7 min***
***Park Processing Time*:**	= ***0.15 min***
Walking time:	1,210 ft / 5 ft / sec / 60 sec / min = ***4.03 min***
Total	**= 6.97 min**

Exhibit 10: Outbound Valet Station Processing Rate
Valet Drop-off / Pick-up

Event Departure Valet Time (Outbound)

***Processing time*:**	60 sec / 60 sec / 1 min = ***1.00 min***
***Driving time*:**	1,200 ft * 1 mile / 5,280 ft * 1hr / 15 miles * 60 min / hr = ***0.91 min***
Mechanical arm gate lift time:	4.25 sec / arm 60 sec / 1 min = ***0.7 min***
***Park Processing Time*:**	= ***0.15 min***
Walking time:	1,210 ft / 5 ft / sec / 60 sec / min = ***4.03 min***
Total	**= 6.16 min**

As the valet pick-up / drop-off area will serve both the rideshare and valet trips to the site; a weighted average was taken of the valet / rideshare processing times. The weighted average was based on the valet / rideshare trip distribution, which is 56% valet and 44% rideshare. Exhibit 11 shows the weighted processing rate at the valet station during the peak hours of the event arrival and departure.

Exhibit 11: Valet Station Processing Rate – Weighted Valet parking / Rideshare Dropoff

<u>**Arrival Weighted Valet Time**</u>

56% Valet Parking	0.56 * 6.97 min = ***3.91 min***
44% Rideshare:	0.44 * 0.5 min = ***0.22 min***
Total	**= <u>4.13 min</u>**

<u>**Departure Weighted Valet Time**</u>

56% Valet Parking	0.56 * 6.16 min = ***3.45 min***
44% Rideshare:	0.44 * 0.5 min = ***0.22 min***
Total	**= <u>3.67 min</u>**

An iterative approach was used to determine the minimum number of valet attendants required during the peak hours of the arrival and departure of event to serve the entering / exiting vehicles that will ensure that the average queue within the pick-up / drop-off area will not extend past the stacking area and spill onto the roadway. Exhibit 12 shows the calculations for the trips to the drop-off / pick-up area during the peak arrival hour of the event. Exhibit 13 shows the calculations for the trips to the drop-off / pick-up area during the peak departure hour of the event.

Exhibit 12: Valet Station Queuing Calculations (Event Arrival)

Q = Processing Rate = $\frac{60\ min/hr}{4.13\ min/process} = 14.54\ process/hr$

q = Demand Rate = $128\frac{veh}{hr}$

N = Service Positions = 13 Attendants

ρ = Utilization factor = $\frac{q}{(NQ)} = \frac{128\ veh/hr}{13 \times 14.54\ process/hr} = 0.6770$

Q_m = Table Value = 0.1442

M = queue length which is exceeded 5% of the time [P(x>M)]

$$M = \frac{\ln P(x>M) - \ln(Q_m)}{\ln(\rho)} - 1 = \frac{\ln(0.05) - \ln(0.6770)}{\ln(0.6770)} - 1 = 1.72,\ say\ 2 \text{ Vehicles on queue}$$

The results of the analysis show that a total of 13 valet attendants would be able to handle the demand at the start of the event at the valet station with an average queue of approximately two vehicles or less. This queue, combined with the rideshare queue creates a total queue of three (3) vehicles within the drop-off / pick-up area. Based on the site plan, the Sagamore Hotel driveway has approximately 100 feet of storage. This distance is enough to accommodate the three-vehicle queue produced by the rideshare and valet drop-off / pick-up operations.

Exhibit 13: Valet Station Queuing Calculations (Event Departure)

Q = Processing Rate = $\frac{60\ min/hr}{3.67\ min/process} = 16.34\ process/hr$

q = Demand Rate = $128\frac{veh}{hr}$

N = Service Positions = 12 Attendants

ρ = Utilization factor = $\frac{q}{(NQ)} = \frac{128\ veh/hr}{12 \times 16.34\ process/hr} = 0.6527$

Q_m = Table Value = 0.1322

M = queue length which is exceeded 5% of the time [P(x>M)]

$M = \frac{\ln P(x>M) - \ln(Q_m)}{\ln(\rho)} - 1 = \frac{\ln(0.05) - \ln(0.6527)}{\ln(0.1322)} - 1 = 1.27,\ say\ 2$ Vehicles on queue

The results of the analysis show that a total of 13 and 12 valet attendants would be able to handle the demand at the drop-off / pick-up area during the respective arrival and departure hours for a maximum occupancy event with an average queue of approximately two vehicles or less. Based on the site plan, the Sagamore Hotel driveway has approximately 100 feet of storage. This distance is enough to accommodate a maximum vehicle queue of four vehicles. Therefore, the event traffic at the pick-up / drop-off area will not spill back onto the roadway.

It should be noted that the queuing analysis considers the worst-case scenario during the peak hours to make sure that the queue never spills onto the public right-of-way or interferes with site operations. Once operational, the development can assess the actual need for valet attendants at different times of the day. Furthermore, the project is considering the implementation of a digital valet drop-off / recall system to decrease the valet drop-off / pick-up times.

Intersection Capacity Analysis Methodology

The intersection analysis was conducted for the AM and PM peak hours of a typical Saturday, and was based on the typical requirements for the City of Miami Beach. The methodology used in the analysis is outlined below:

- Traffic Counts – Turning movement counts were obtained from the City of Miami Beach. The counts were adjusted to reflect average annual daily traffic conditions using the latest weekly volume adjustment factors obtained from FDOT.
- Signal Location and Timing – Existing signal phasing and timing for the signalized intersection were obtained from Miami-Dade County (see Attachment C).
- Background Traffic – Available Florida Department of Transportation (FDOT) and Miami-Dade County (MDC) counts were consulted to determine a growth factor consistent with historical annual growth in the area. As the growth rate for the area was negative, the counts were not reduced and were considered as the future with project condition at the advice of the City.
- Future Transportation Projects – The 2021 TIP and the 2045 LRTP were reviewed and considered in the analysis at project build-out. No capacity projects were found at the intersection (See Attachment C).
- Analysis – Intersection analysis was done using the Synchro software based on Highway Capacity Manual (HCM 6th Edition).

Traffic Data Collection

Signal timing data was obtained from Miami-Dade County for the analyzed signalized intersection in this study. This information was used for the signal phasing and timing required for the intersection capacity analysis. A field survey was conducted to obtain the lane configurations used in the intersection analysis. Exhibit 15 shows the existing lane configurations.

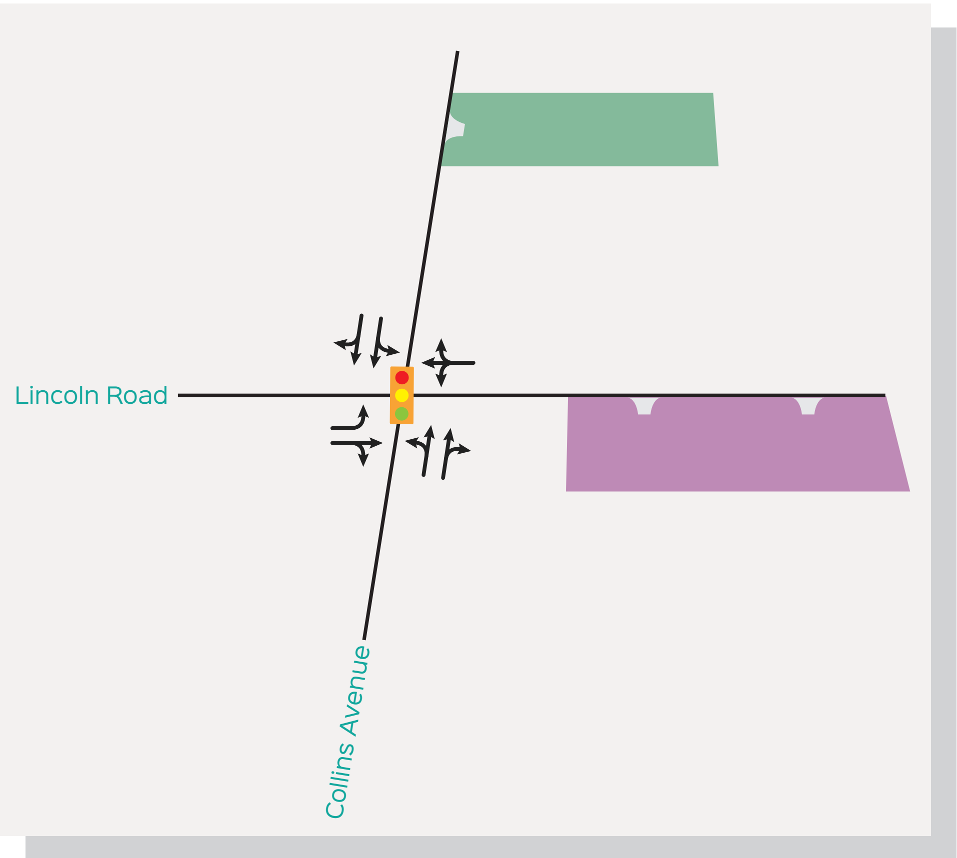
Vehicle turning movement counts were collected on June 6, 2019 at the Collins Avenue / Lincoln Road intersection during the midday / afternoon (10 AM–4 PM) and evening (9 PM-11 PM) for a typical Saturday. The counts were adjusted to reflect average annual daily traffic conditions using the latest weekly volume adjustment factors obtained from FDOT. A weekly volume adjustment factor of 1.04 (Miami-Dade County North) corresponding to the dates of the counts was used. Traffic counts are provided in Attachment C. Traffic volumes used in the analyses are also shown in Exhibit 15.

Existing Conditions Intersection Capacity Analysis

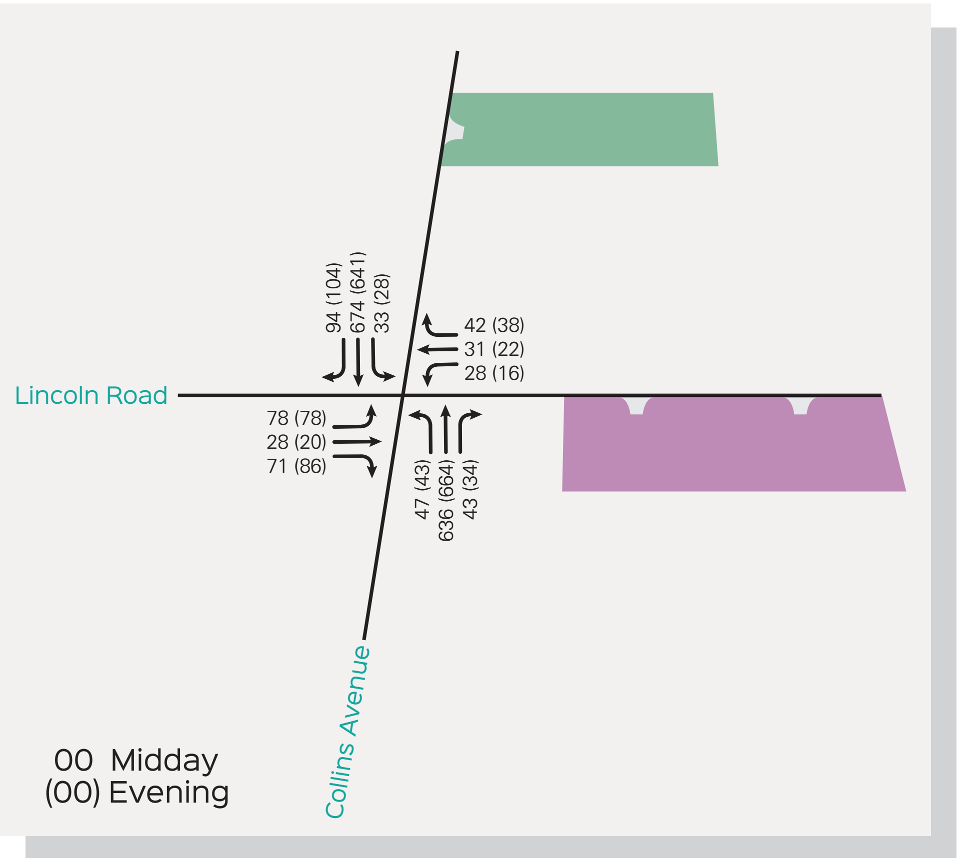
The Synchro software was used to perform intersection capacity analysis. **Synchro** is a macroscopic analysis and optimization software application that implements the Intersection Capacity Utilization method for determining intersection capacity. Synchro also supports the Highway Capacity Manual's methodology for signalized intersections. Exhibit 14 shows the resulting Level of Service (LOS) for the Saturday midday and evening peak hour conditions. The analysis shows that the Collins Avenue / Lincoln Drive intersection currently operates at LOS D during the midday and evening peak hours. Capacity worksheets are included in Attachment D.

Exhibit 14: Existing Conditions Intersection Capacity Analysis
Saturday Midday and Evening Peak Hour Conditions

Intersection	Signalized/ Un-signalized	Direction	Midday Peak		Evening Peak	
			LOS	Delay	LOS	Delay
Collins Avenue / Lincoln Road	S	NB	C	32.2	C	30.0
		SB	C	31.6	C	29.5
		EB	F	121.0	F	154.1
		WB	E+19	95.1	E+2	81.5
		Overall	***D***	***44.2***	***D***	***44.9***



Existing Lane Configurations



Existing Traffic Volumes

Project Location

Valet Parking

Exhibit 15

Existing Lane Configurations & Traffic Volumes

NORTH
MAP NOT TO SCALE

Trip Distribution and Trip Assignment

As seen in the above sections, trip generation analyses were conducted for a sit-down event and maximum occupancy events. The maximum occupancy events are projected to generate the most trips for events held at the Sagamore Hotel. Exhibit 16 shows the calculations and trip generation summary for a maximum occupancy event.

Exhibit 16: Maximum Occupancy Event Trip Generation

Proposed Event Trip Generation			
Calculations	**Event Space**	**Percentages / Rates Applied**	
Number of Attendees	582	Maximum Expected Event Occupancy	
Attendees - Internal	48	8.2%	Internal[1]
Attendees - External	534	91.8%	External
External Vehicle trips	267	2	Persons/Vehicle[2]
Peak Hour Trips	160	60%	Arrive/Depart during the peak hour
Alternative Transport Trips	32	20%	of Peak Hour Trips[2]
Arriving by vehicles Trips	128	80%	of Peak Hour Trips
Total Peak Hour Vehicle Trip	128		
Rideshare Trips	56	44%	Rideshare reduction[2]
Total Valet Trips	72		

[1]Based on ITE internalization rates between the hotel and restaurant approved by Miami Beach

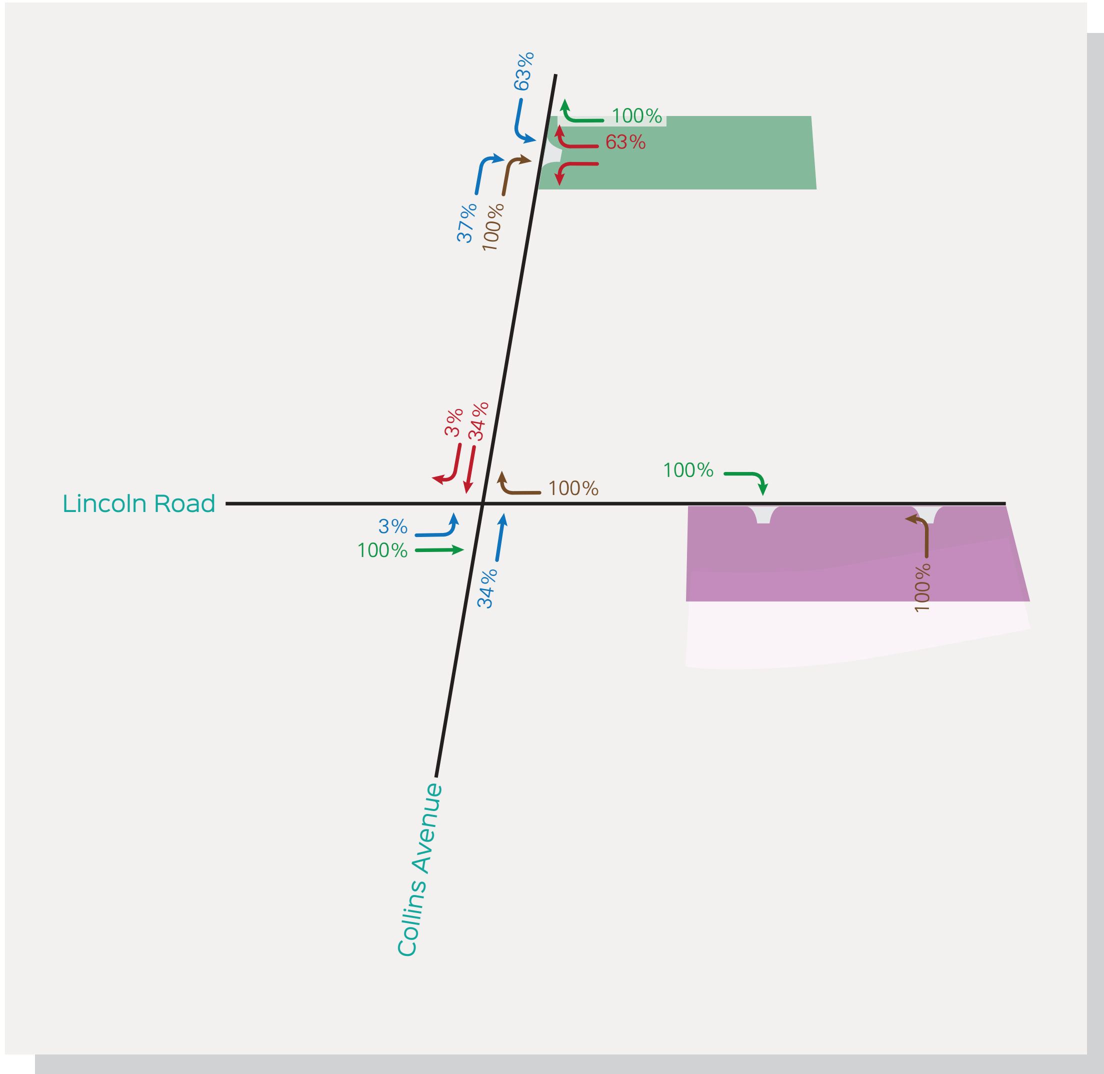
[2]Based on Information Provided by Miami Beach

Project traffic was distributed and assigned to the study area using the Cardinal Distribution for TAZ 644, shown in Exhibit 17. The Cardinal Distribution gives a generalized distribution of trips from a TAZ to other parts of Miami-Dade County. The TAZ can be summarized as 33% to the north, 16% to the south, 0% to the east, and 51% to the west. For estimating the trip distribution for the project location, consideration was given to conditions such as the roadway network accessed by the project, driveway placement and land uses, roadways available to travel in the desired direction, and attractiveness of traveling on a specific roadway. Exhibit 18 shows the project vehicular trip distribution to/from the site and the valet distribution to/from the parking garage located at 100 Lincoln Road. The traffic patterns for the event traffic will differ between the event arrival and departure. Exhibit 19 shows the event trip assignment and valet trip assignment at the Collins Avenue / Lincoln Road intersection for the event arrival and departure.

The project trip assignments and existing traffic were combined to obtain future with event traffic volumes at the analyzed intersection; as the event arrival and event departure affect the traffic differently. Exhibit 20 shows the event arrival and departure traffic volumes for the midday peak hour. The event arrival and departure traffic volumes for the evening peak hour are shown in Exhibit 21.

Exhibit 17: Cardinal Distribution
TAZ 644

DIRECTION	2015	2045	2023
NNE	14.8%	12.1%	14.1%
ENE	0.0%	0.0%	0.0%
ESE	0.0%	0.0%	0.0%
SSE	0.0%	0.0%	0.0%
SSW	16.5%	13.9%	15.8%
WSW	30.4%	34.5%	31.5%
WNW	19.0%	20.3%	19.3%
NNW	19.4%	19.2%	19.3%

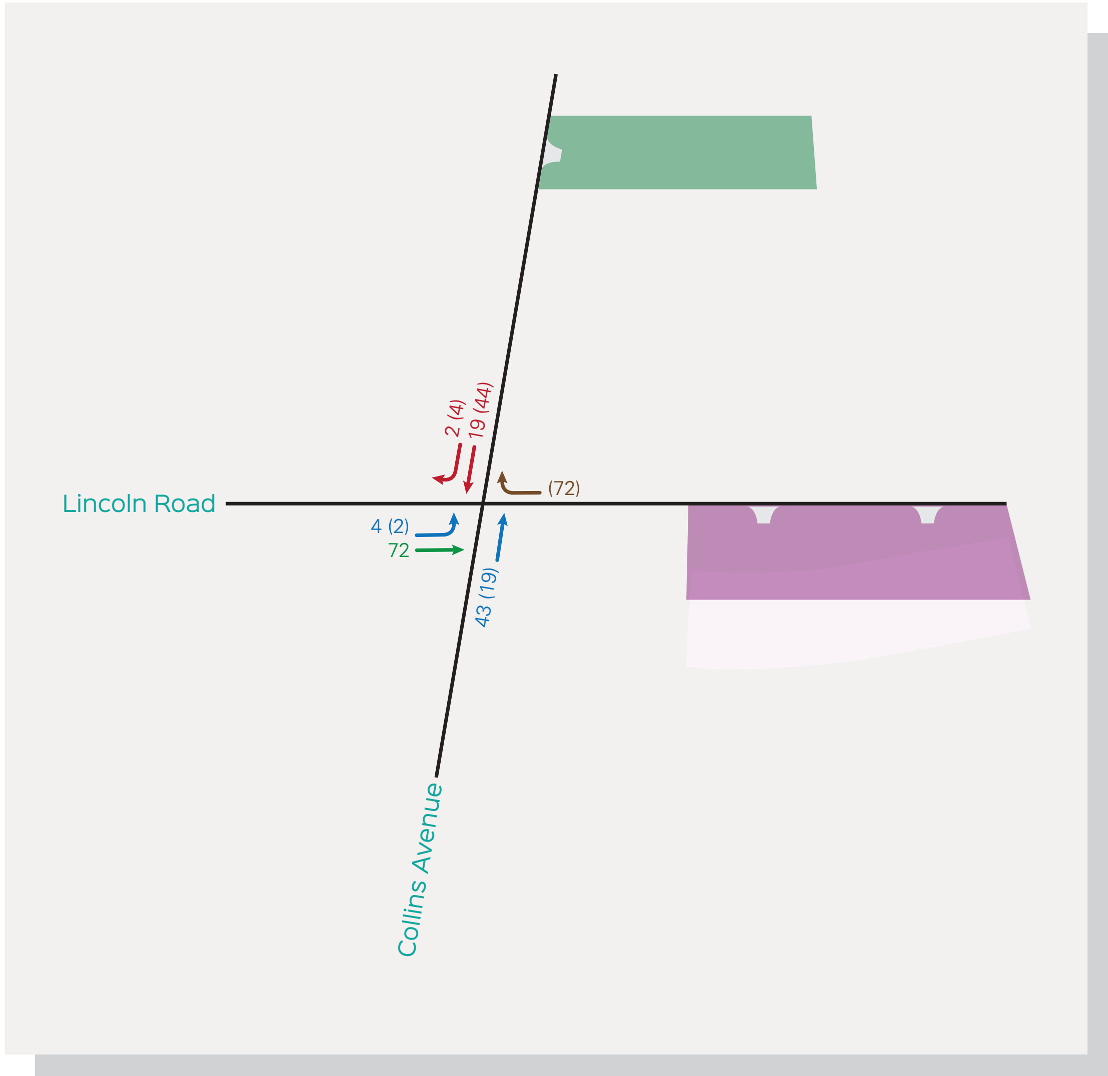


Project Location
Valet Parking

Inbound
Outbound
Valet Dropoff
Valet Pick-Up

Exhibit 18
Event Trip Distribution

NORTH
MAP NOT TO SCALE



00 Event Arrival
(00) Event Departure

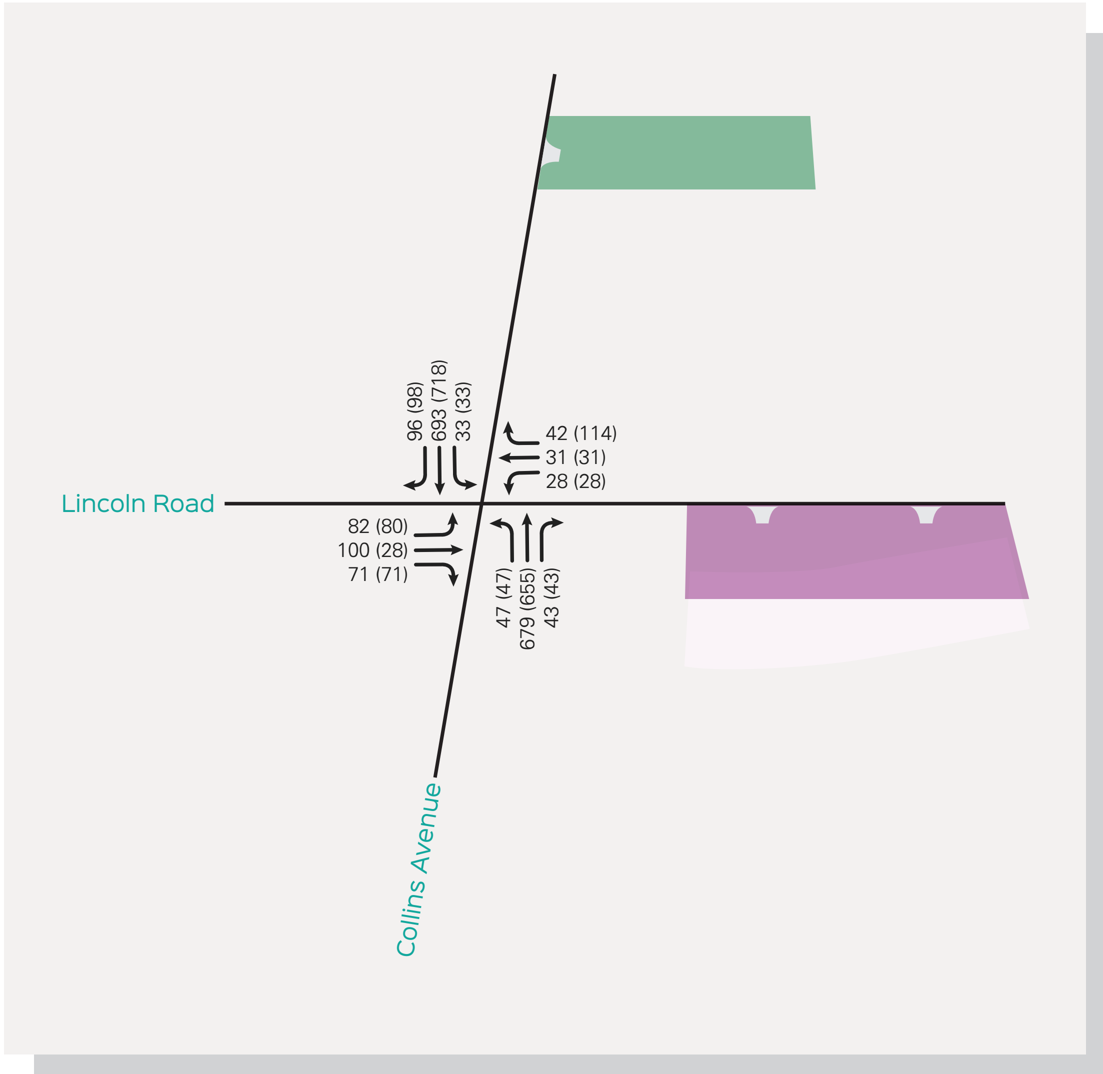
Project Location
Valet Parking

Inbound
Outbound
Valet Dropoff
Valet Pick-Up

NORTH
MAP NOT TO SCALE

Exhibit 19

Event Trip Assignment



00 Event Arrival
(00) Event Departure

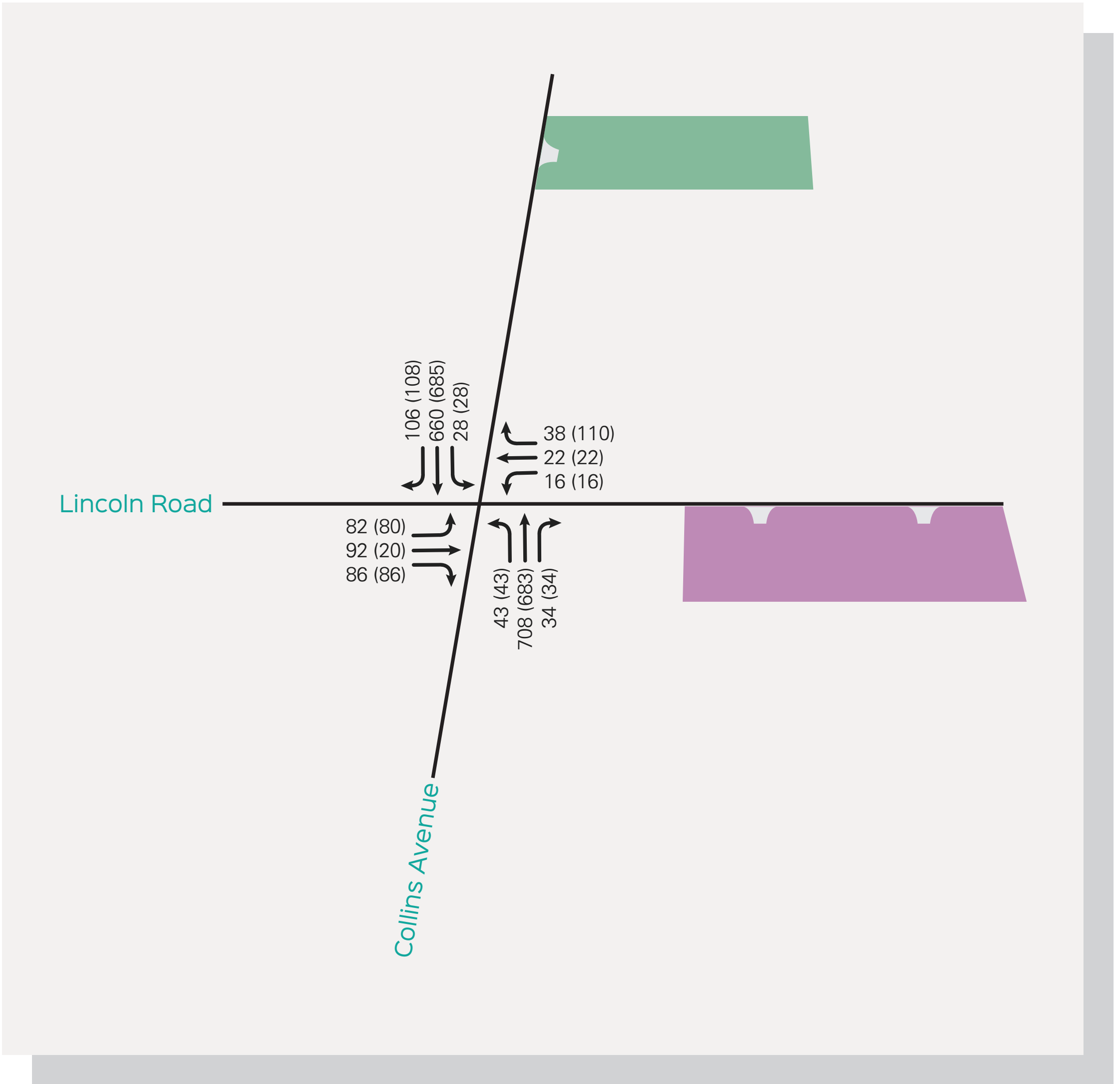
Project Location

Valet Parking

Exhibit 20

Future With Midday Event Traffic Volumes

NORTH
MAP NOT TO SCALE



00 Event Arrival
(00) Event Departure

Project Location

Valet Parking

Exhibit 21

Future With Evening Event Traffic Volumes

NORTH
MAP NOT TO SCALE

Future with Maximum Occupancy Event Conditions Intersection Capacity Analysis

The intersection of Collins Avenue and Lincoln Road was analyzed for future with maximum occupancy event conditions for the midday and evening peak hours of a typical saturday. As the venue is seeking a conditional use permit for future unplanned events at the Sagamore Hotel, the intersection was analyzed as if the event began or ended during the midday and evening peak hours of a typical Saturday. Exhibit 22 shows the resulting LOS for the arrival (event start) and departure (event end) conditions for the midday and evening peak hours of a typical Saturday. The analysis shows that the intersection is projected to continue to operate at LOS D during the midday and evening peak hours during the event arrival and departure. Capacity worksheets are included in Attachment D.

Exhibit 22: Future with Maximum Occupancy Event Conditions Intersection Capacity Analysis

Saturday Midday and Evening Peak Hour Conditions

Intersection	Signalized/ Un-signalized	Direction	Midday Peak		Evening Peak	
			LOS	Delay	LOS	Delay
Collins Avenue / Lincoln Road (Arrival)	S	NB	D	36.8	D	36.9
		SB	C	34.8	C	34.8
		EB	E+44	115.3	E+44	115.5
		WB	E+23	98.6	E	67.0
		Overall	***D***	***49.4***	***D***	***47.9***
Collins Avenue / Lincoln Road (Departure)	S	NB	D	46.4	D	39.4
		SB	D	43.8	D	37.8
		EB	E	70.1	E+11	88.6
		WB	F	121.0	F	139.9
		Overall	***D***	***54.1***	***D***	***51.2***

Conclusions

A trip generation analysis was conducted for the proposed sit-down and maximum occupancy events hosted by the Sagamore Hotel event space. The results of the analysis show that the proposed sit-down events hosted by the Sagamore Hotel will generate an increase of 4 and 39 vehicle trips during morning and afternoon peak hours, respectively. The results of the maximum occupancy trip generation show that the events hosted by the Sagamore Hotel will generate a maximum of 72 vehicle trips and 56 rideshare vehicle trips during the peak hour of the arrival and dismissal periods for an event.

Rideshare and valet queuing analyses were performed to ensure that the drop-off / pick-up area for the Sagamore Hotel can accommodate the vehicle stacking during events without spill-back onto Collins Avenue. These analyses were performed for three different scenarios:

- A sit-down event during the PM peak hour (worst case scenario for a sit-down event)
- The arrival peak hour for a maximum occupancy event (critical inbound scenario)
- The departure peak hour for a maximum occupancy event (critical outbound scenario)

The results of the queuing analysis for a sit-down event show that a total of 9 valet attendants would be able to handle the rideshare and valet parking demand at the drop-off / pick-up area with an average queue of three vehicles or less. The results of the queuing analysis for the arrival peak hour of a maximum occupancy event show that 13 valet attendants could handle the rideshare and valet parking demand with a queue of approximately two-vehicles or less. The results of the queuing analysis for the departure peak hour of a maximum occupancy event show that a total of 12 valet attendants are needed to handle the demand at the drop-off / pick-up area with an average queue of two vehicles or less.

The Sagamore Hotel has approximately 100 feet of stacking space. Thus, the pick-up / drop-off can accommodate a four-vehicle queue. Therefore, the projected queues produced by the events will be accommodated within the Sagamore Hotel stacking area without spill-back onto Collins Avenue. Furthermore, at the request of the City, the project is considering the implementation of a digital valet drop-off / recall system to decrease the valet drop-off / pick-up times.

The results of the analysis for existing and future with a maximum occupancy event conditions shows that the Collins Avenue / Lincoln Road intersection currently operates and will continue to operate at a LOS D during the midday and evening peak hours of a typical Saturday. Therefore, the addition of the event traffic will not adversely impact the adjacent roadway network.

We stand ready to provide any support needed for this project. Should you have any questions or comments, please call me at (305) 447-0900.

Sincerely,

Juan Espinosa, PE
Vice-President – Transportation

JUAN B. ESPINOSA-LEAL, PE
LICENSE
No. 49512
STATE OF
FLORIDA
PROFESSIONAL ENGINEER

Digitally signed by Juan Espinosa
Reason: I am the author of this document
Date: 2021-10-01 17:02:52

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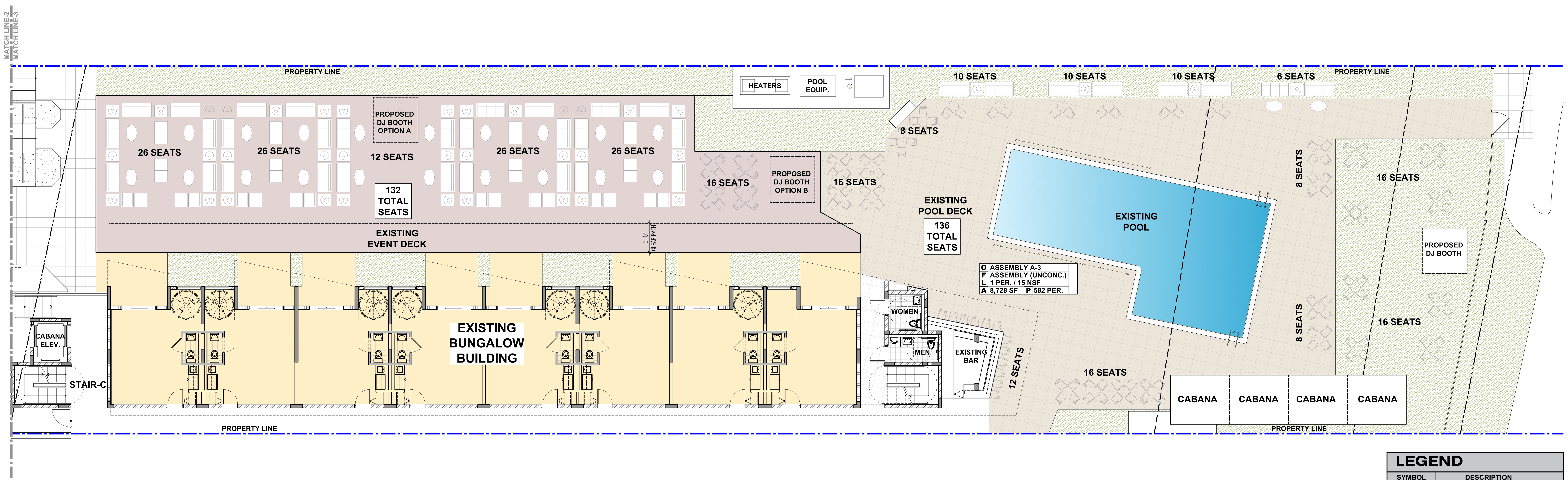
Attachment A

Site Plan



1 FURNITURE LAYOUT - SEATING COUNT PARTIAL OVERALL EXISTING LEVEL 1

TOTAL OCCUPANCY CAPACITY = 582 PERSON

SCALE: 3/32" = 1'-0"

N

LEGEND

SYMBOL	DESCRIPTION
	EXISTING BUILDING FOOTPRINT
	EXISTING PATIO DECK
	EXISTING POOL DECK
	EXISTING GREEN AREAS

DRB FIRST SUBMITTAL SET. August 9th., 2021 - NOT FOR CONSTRUCTION

PROJECT:

SAGAMORE

1671 Collins Avenue. Miami Beach, FL 33139

PROJECT OWNER :

EBJ / Sagamore, LLC
104 West 40th Street, 14th Floor
New York, NY 10018

ARCHITECT OF RECORD:

R+N Architecture
420 E. Palmetto Park Road. Boca Raton, Florida 33432
Office: 786.762.2679 C: 305.282.0005
Email: vh.rodriguez@rna-archint.com
www.rda@rna-archint.com / AA26002510

INTERIOR DESIGNER:

MEP+FP CONSULTANT:

STRUCTURAL ENGINEER:

KEY PLAN

SIGNATURE / DATE / SEAL

NOT VALID IF MISSING SIGNATURE

STATE OF FLORIDA VICTOR HUGO RODRIGUEZ AR94965 REGISTERED ARCHITECT

Victor H. Rodriguez,
Registered Architect
State of Florida # AR0094965

DRB FIRST SUBMITAL POOL DECK C.U.P.

Issue	Issue Date / For
1	08.09.2021 / DRB First Submittal

DDCI Project #: 2021.050
Drawn by: VHR
Approved by: VHR

SHEET INDEX

- Seating Layout Options

SCALE : 3/32" = 1'-0"

SHEET NO.

A-6

ITEM #	MULTIFAMILY - COMMERCIAL - ZONING DATA SHEET				
	Zoning Information				
1	Address:	1671 Collins Avenue. Miami Beach, FL 33139			
2	Board and file numbers :				
3	Folio number(s):	02-3234-019-0530			
4	Year constructed:	1948	Zoning District:	RM-3	
5	Based Flood Elevation:	8 FT	Grade value in NGVD:	9.5 FT (AT LOT MIDPOINT)	
6	Adjusted grade (Flood+Grade/2):	N/A	Lot Area:	44,847.75 SF	
7	Lot width:	76.09 FEET	Lot Depth:	598.58 FEET	
8	Minimum Unit Size	200 SF	Average Unit Size	N/A	
9	Existing use:	HOTEL	Proposed use:	HOTEL	
		Maximum	Existing	Proposed	Deficiencies
10	Height	200 FT	83.35' NGVD	NO CHANGE	N/A
11	Number of Stories	22 STORIES	6 STORIES	NO CHANGE	N/A
12	**FAR (LOT AREA = 44,847.75) X 2.0**	89,648 SF	87,499 SF	87,993 SF	N/A
13	Gross square footage				
14	Square Footage by use	N/A			N/A
15	Number of units Residential	N/A	N/A	N/A	N/A
16	Number of units Hotel	N/A	**93**	**146**	N/A
17	Number of seats	N/A	N/A	N/A	N/A
18	Occupancy load	N/A	N/A	N/A	N/A
	Setbacks	Required	Existing	Proposed	Deficiencies
	Subterranean:				
19	Front Setback:	N/A	N/A	N/A	N/A
20	Side Setback:	N/A	N/A	N/A	N/A
21	Side Setback:	N/A	N/A	N/A	N/A
22	Side Setback facing street:	N/A	N/A	N/A	N/A
23	Rear Setback:	N/A	N/A	N/A	N/A
	At Grade Parking:				
24	Front Setback:	N/A	N/A	N/A	N/A
25	Side Setback:	N/A	N/A	N/A	N/A
26	Side Setback:	N/A	N/A	N/A	N/A
27	Side Setback facing street:	N/A	N/A	N/A	N/A
28	Rear Setback:	N/A	N/A	N/A	N/A
	Pedestal:				
29	Front Setback (WEST):	20'-0"	46.32 FT	NO CHANGE	N/A
30	Side Setback (NORTH):	7.5 FT	5.0 FT	NO CHANGE	N/A
31	Side Setback (SOUTH):	7.5 FT	5.0 FT	NO CHANGE	N/A
32	Side Setback facing street:	N/A	N/A	N/A	N/A
33	Rear Setback (EAST):	119.6	43.2 FT	NO CHANGE	N/A
	Tower:				
34	Front Setback:	N/A	N/A	N/A	N/A
35	Side Setback:	N/A	N/A	N/A	N/A
37	Side Setback facing street:	N/A	N/A	N/A	N/A
38	Rear Setback:	N/A	N/A	N/A	N/A

1 ZONING DATA CHART
NTS

	Parking	Required	Existing	Proposed	Deficiencies
39	Parking district	1	1	1	
40	Total # of parking spaces		0	0	
41	# of parking spaces per use (Provide a separate chart for a breakdown calculation)	4	0	0	4
42	# of parking spaces per level (Provide a separate chart for a breakdown calculation)	N/A			
43	Parking Space Dimensions	N/A			
44	Parking Space configuration (45o,60o,90o,Parallel)	N/A			
45	ADA Spaces	N/A			
46	Tandem Spaces	N/A			
47	Drive aisle width	N/A			
48	Valet drop off and pick up				
49	Loading zones and Trash collection areas				
50	Bicycle parking, location and Number of racks				
	Restaurants, Cafes, Bars, Lounges, Nightclubs	Required	Existing	Proposed	Deficiencies
51	Type of use	N/A	RESTAURANT / BAR	RESTAURANT / BAR	
52	Number of seats located outside on private property	N/A	REST & BAR 30 SEATS	BAR / 292 SEATS	
53	Number of seats inside	N/A	REST & BAR 112 SEATS	0 SEATS	
54	Total number of seats	N/A	REST & BAR 142 SEATS	REST & BAR 292 SEATS	
55	Total number of seats per venue (Provide a separate chart for a breakdown calculation)		(SEE CHART THIS SHEET)		
56	Total occupant content	N/A	783 P	434 P	
57	Occupant content per venue (Provide a separate chart for a breakdown calculation)		(SEE CHART THIS SHEET)		
58	Proposed hours of operation	Indoor Restaurant and Bar (8 00 AM to 5 00 AM)			
59	Is this an NIE? (Neighboot Impact stablishment, see CMB 141-1361)		NO		
60	Is dancing and/or entertainment proposed ? (see CMB 141-1361)		NO		
61	Is this a contributing building?		YES		
62	Located within a Local Historic District?		YES		
Notes:					
If not applicable write N/A					
All other data information may be required and presented like the above format.					

PARKING CALCULATION					
USE	DESCRIPTION	RATE	REQURED SPACES *		PROPOSED
EXISTING HOTEL ROOMS (HISTORICAL BUILDING)	N/A	N/A	0	0 P.S.	0 P.S.
EXISTING HOTEL ROOMS (BUNGALOW BUILDING)	N/A	N/A	0	0 P.S.	0 P.S.
EXISTING RESTAURANT /BAR (WITHING HISTORIC BUILDING)	N/A	N/A	0	0 P.S.	0 P.S.
			TOTAL	0	0 P.S.
	* SPACES REQUIRED PER CITY OF MIAMI BEACH FL CODE OF ORDINANCE / ARTICLE IV. SEC 130.33 - OFF STREET PARKING REQUIREMENTS FOR PARKING DISTRICT No. 1				

LOADING PARKING CALCULATION					
USE	DESCRIPTION	RATE	REQURED SPACES *		PROPOSED
EXISTING RESTAURANT / MEETING ROOM	N/A	N/A	0	0 P.S.	0 P.S.
			TOTAL	0 P.S.	0 P.S.
	* SPACES REQUIRED PER CITY OF MIAMI BEACH FL CODE OF ORDINANCE / ARTICLE IV. SEC 130.101 - OFF STREET PARKING REQUIREMENTS FOR LOADING				

2 PARKING CALCULATIONS
NTS

OCCUPANCY COUNT (RESTAURANT, BAR & LOUNGE / NEW OUTDOOR VENUE)					
USE	DESCRIPTION	RATE	OCCUPANT LOAD	SEAT COUNT	
EXISTING LOUNGE	837 SF	1 PERSON PER 15 SF	56 PERSON	12 SEATS	EXISTING
EXISTING RESTAURANT (INDOOR)	1,633 SF	1 PERSON PER 15 SF	108 PERSON	100 SEATS	EXISTING
EXISTING RESTAURANT OUTDOOR	553 SF	1 PERSON PER 15 SF	37 PERSON	30 SEATS	EXISTING
NEW EXTERIOR VENUE	8,728 SF	1 PERSON PER 15 SF	582 PERSON	292 SEATS	NEW
		TOTAL	783 PERSON	434 SEATS	

3 OCCUPANCY LOADS & SEATING COUNT
NTS

DRB FINAL SUBMITTAL SET. August 30th., 2021 - NOT FOR CONSTRUCTION

PROJECT:
SAGAMORE
1671 Collins Avenue. Miami Beach, FL 33139

PROJECT OWNER :
EBJ / Sagamore, LLC
104 West 40th Street, 14th Floor
New York, NY 10018

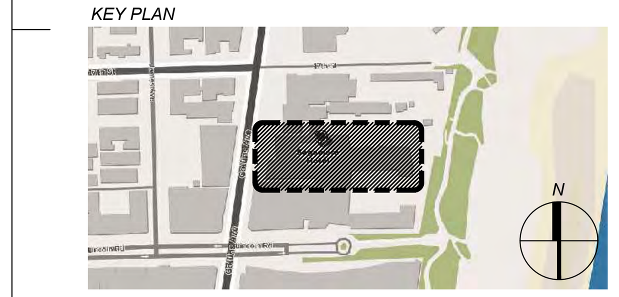
ARCHITECT OF RECORD:
R+N

R+N Architecture
420 E. Palmetto Park Road. Boca Raton, Florida 33432
Office: 786.762.2679 C: 305.282.0005
Email: vh.rodriguez@rna-archint.com
www.rda@rna-archint.com / AA26002510

INTERIOR DESIGNER:

MEP+FP CONSULTANT:

STRUCTURAL ENGINEER:

KEY PLAN


SIGNATURE / DATE / SEAL

NOT VALID IF MISSING SIGNATURE

STATE OF FLORIDA VICTOR HUGO RODRIGUEZ AR94965 REGISTERED ARCHITECT

Victor H. Rodriguez,
Registered Architect
State of Florida # AR0094965

DRB FINAL SUBMITAL POOL DECK C.U.P.

Issue	Issue Date / For
1	**08.09.2021 / DRB First Submittal**
2	**08.30.2021 / DRB Final Submittal**

DDCI Project #: **2021.050**
Drawn by: **VHR**
Approved by: **VHR**

SHEET INDEX
- Project Zoning Data

SCALE : **As Indicated**

SHEET NO.
A-2

Attachment B

Trip Generation Documentation

Scenario - 1

Scenario Name: Existing restaurant
User Group:
Dev. phase: 1
No. of Years to Project Traffic : 0
Analyst Note:
Warning: The time periods among the land uses do not appear to match.

VEHICLE TRIPS BEFORE REDUCTION

Land Use & Data Source	Location	IV	Size	Time Period	Method Rate/Equation	Entry Split%	Exit Split%	Total
931 - Quality Restaurant Data Source: Trip Gen Manual, 10th Ed	General Urban/Suburban	Seats	142	Weekday	Best Fit (LIN) T = 3.90(X) - 447.07	53 50%	53 50%	106
931(1) - Quality Restaurant Data Source: Trip Gen Manual, 10th Ed	General Urban/Suburban	Seats	142	Weekday, Peak Hour of Adjacent Street Traffic,	Average 0.02	1 50%	1 50%	2
931(2) - Quality Restaurant Data Source: Trip Gen Manual, 10th Ed	General Urban/Suburban	Seats	142	Weekday, Peak Hour of Adjacent Street Traffic,	Average 0.28	27 67%	13 33%	40

Scenario - 2

Scenario Name: Proposed restaurant
User Group:
Dev. phase: 1
No. of Years to Project Traffic : 0
Analyst Note:
Warning: The time periods among the land uses do not appear to match.

VEHICLE TRIPS BEFORE REDUCTION

Land Use & Data Source	Location	IV	Size	Time Period	Method Rate/Equation	Entry Split%	Exit Split%	Total
931 - Quality Restaurant Data Source: Trip Gen Manual, 10th Ed	General Urban/Suburban	Seats	434	Weekday	Best Fit (LIN) T = 3.90(X) - 447.07	623 50%	623 50%	1246
931(1) - Quality Restaurant Data Source: Trip Gen Manual, 10th Ed	General Urban/Suburban	Seats	434	Weekday, Peak Hour of Adjacent Street Traffic,	Average 0.02	4 50%	4 50%	8
931(2) - Quality Restaurant Data Source: Trip Gen Manual, 10th Ed	General Urban/Suburban	Seats	434	Weekday, Peak Hour of Adjacent Street Traffic,	Average 0.28	81 67%	40 33%	121

AM Peak Hour Trip Generation and Internalization

The Sagmore Weekday Existing

Hotel Land Use 310 100 Rooms		Quality Reastaurant Land Use 931 142 Seats		
In	Out	In	Out	
26	**18**	**1**	**1**	46 ITE Trips

UNBALANCED INTERNALIZATION

	9%		*6%*	
	2	υ	*0*	
4%				*0%*
1		υ		*0*

Hotel		Quality Reastaurant		
In	Out	In	Out	
26	**18**	**1**	**1**	46 Vehicle Trips

BALANCED INTERNALIZATION

	0	*0*	
0			*0*

Hotel In	Hotel Out	Quality Reastaurant In	Quality Reastaurant Out	Total		
0	0	0	0	0		Internal
26	18	1	1	46		External Trips
	0.0%		*0.0%*		*0.0%*	% Internal
-5	-4	0	0	-9	20.0%	Transit/Pedestrian
21	14	1	1	37		
0	0			0	0%	Passby
		0	0	0	0%	Passby
21	**14**	**1**	**1**	**37**		**Net New External Trips**
-9	-6	0	0	-15	44.0%	Valet reduction
21	**14**	**1**	**1**	**37**		**Valet Station trips**

PM Peak Hour Trip Generation and Internalization

The Sagmore Weekday Existing

Hotel Land Use 310 100 Rooms			Quality Reastaurant Land Use 931 142 Seats		
In	**Out**		**In**	**Out**	
25	**24**		**27**	**13**	89 ITE Trips
UNBALANCED INTERNALIZATION					
	68%		*5%*		
	16	1	*1*		
71%				*7%*	
18		1		*1*	
Hotel			**Quality Reastaurant**		
In	**Out**		**In**	**Out**	
25	**24**		**27**	**13**	89 Vehicle Trips
BALANCED INTERNALIZATION					
	-1		*-1*		
-1				*-1*	
-1	-1		-1	-1	-4 Internal
24	23		26	12	85 External Trips
	4.1%			*5.0%*	*4.5%* % Internal
-5	-5		-5	-2	-17 20.0% Transit/Pedestrian
19	18		21	10	68
0	0				0 0% Passby
			-5	-5	-10 34% Passby
19	**18**		**16**	**5**	**58 Net New External Trips**
					Trips @ Driveway
-8	-8		-7	-2	-25 44.0% Valet reduction
19	**18**		**11**	**0**	**48 Valet Station trips**

AM Peak Hour Trip Generation and Internalization

The Sagmore Weekday Proposed

Hotel Land Use 310 100 Rooms		Quality Reastaurant Land Use 931 434 Seats		
In	Out	In	Out	
26	**18**	**4**	**4**	52 ITE Trips

UNBALANCED INTERNALIZATION

	9%		*6%*		
	2	*0*	*0*		
4%				*0%*	
1		*0*		*0*	

Hotel		Quality Reastaurant		
In	Out	In	Out	
26	**18**	**4**	**4**	52 Vehicle Trips

BALANCED INTERNALIZATION

	0	*0*	
0			*0*

0	0	0	0	0		Internal
26	18	4	4	52		External Trips
	0.0%		*0.0%*		*0.0%*	% Internal
-5	-4	-1	-1	-11	20.0%	Transit/Pedestrian
21	14	3	3	41		
0	0			0	0%	Passby
		0	0	0	0%	Passby
21	**14**	**3**	**3**	**41**		**Net New External Trips**
-9	-6	-1	-1	-17	44.0%	Valet reduction
21	**14**	**3**	**3**	**41**		**Valet Station trips**

PM Peak Hour Trip Generation and Internalization

The Sagmore Weekday Proposed

Hotel Land Use 310 100 Rooms			Quality Reastaurant Land Use 931 434 Seats		
In	**Out**		**In**	**Out**	
25	**24**		**81**	**40**	170 ITE Trips
UNBALANCED INTERNALIZATION					
	68%		*5%*		
	16	**4**	*4*		
71%				*7%*	
18		**3**		*3*	
Hotel			**Quality Reastaurant**		
In	**Out**		**In**	**Out**	
25	**24**		**81**	**40**	170 Vehicle Trips
BALANCED INTERNALIZATION					
	-4		*-4*		
-3				*-3*	
-3	-4		-4	-3	-14 Internal
22	20		77	37	156 External Trips
	14.3%			*5.8%*	*8.2%* % Internal
-4	-4		-15	-7	-30 20.0% Transit/Pedestrian
18	16		62	30	126
0	0				0 0% Passby
			-16	-16	-32 34% Passby
18	**16**		**46**	**14**	**94 Net New External Trips**
					Trips @ Driveway
-8	-7		-20	-6	-41 44.0% Valet reduction
18	**16**		**30**	**-2**	**62 Valet Station trips**

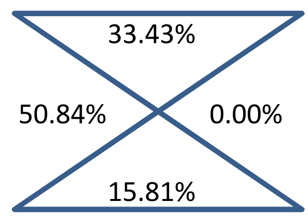
Cardinal Distribution **21215**

Sagamore Hotel CUP

TAZ	644

DIRECTION	2015	2045	2023
NNE	14.8%	12.1%	14.1%
ENE	0.0%	0.0%	0.0%
ESE	0.0%	0.0%	0.0%
SSE	0.0%	0.0%	0.0%
SSW	16.5%	13.9%	15.8%
WSW	30.4%	34.5%	31.5%
WNW	19.0%	20.3%	19.3%
NNW	19.4%	19.2%	19.3%

38.69%	14.08%
47.30%	0.00%



Miami-Dade 2015 Base Year Direction Trip Distribution Summary											
TAZ of Origin		Trips / Percent	Cardinal Directions								Total Trips
County TAZ	Regional TAZ		NNE	ENE	ESE	SSE	SSW	WSW	WNW	NNW	
625	3525	Trips	610	160	-	557	431	1,317	679	1,035	4,961
625	3525	Percent	12.7	3.3	-	11.6	9.0	27.5	14.2	21.6	
626	3526	Trips	122	-	-	-	2,090	2,277	1,198	2,942	9,399
626	3526	Percent	1.4	-	-	-	24.2	26.4	13.9	34.1	
627	3527	Trips	279	-	-	-	2,051	2,578	845	1,965	8,061
627	3527	Percent	3.6	-	-	-	26.6	33.4	11.0	25.5	
628	3528	Trips	298	-	49	79	984	902	332	679	3,579
628	3528	Percent	9.0	-	1.5	2.4	29.6	27.2	10.0	20.5	
629	3529	Trips	1,374	549	344	1,656	1,708	3,707	1,668	2,101	14,261
629	3529	Percent	10.5	4.2	2.6	12.6	13.0	28.3	12.7	16.0	
630	3530	Trips	952	-	210	347	1,696	2,375	794	1,114	8,135
630	3530	Percent	12.7	-	2.8	4.6	22.7	31.7	10.6	14.9	
631	3531	Trips	255	-	-	-	1,215	1,471	440	1,030	4,651
631	3531	Percent	5.8	-	-	-	27.6	33.4	10.0	23.4	
632	3532	Trips	309	-	-	-	1,242	1,751	750	635	4,880
632	3532	Percent	6.6	-	-	-	26.5	37.4	16.0	13.5	
633	3533	Trips	310	-	-	-	1,181	1,428	750	730	4,590
633	3533	Percent	7.0	-	-	-	26.9	32.5	17.1	16.6	
634	3534	Trips	1,502	112	240	837	1,718	1,928	976	1,727	9,998
634	3534	Percent	16.6	1.2	2.7	9.3	19.0	21.3	10.8	19.1	
635	3535	Trips	779	-	-	-	2,021	1,994	952	1,411	8,010
635	3535	Percent	10.9	-	-	-	28.2	27.9	13.3	19.7	
636	3536	Trips	1,041	-	-	686	1,152	2,072	911	1,071	7,384
636	3536	Percent	15.0	-	-	9.9	16.6	29.9	13.1	15.4	
637	3537	Trips	323	31	87	217	126	601	303	290	1,987
637	3537	Percent	16.4	1.6	4.4	11.0	6.4	30.4	15.3	14.7	
638	3538	Trips	152	35	87	86	114	218	162	126	999
638	3538	Percent	15.5	3.6	8.9	8.7	11.6	22.3	16.5	12.9	
639	3539	Trips	825	281	277	1,089	131	1,364	796	599	5,721
639	3539	Percent	15.4	5.2	5.2	20.3	2.4	25.4	14.9	11.2	
640	3540	Trips	344	247	868	104	43	685	405	274	3,053
640	3540	Percent	11.6	8.3	29.2	3.5	1.5	23.1	13.6	9.2	
641	3541	Trips	1,051	1,714	291	723	309	1,572	1,188	916	8,356
641	3541	Percent	13.5	22.1	3.7	9.3	4.0	20.3	15.3	11.8	
642	3542	Trips	1,849	1,404	115	1,263	457	2,697	1,962	1,518	12,299
642	3542	Percent	16.4	12.5	1.0	11.2	4.1	23.9	17.4	13.5	
643	3543	Trips	1,747	551	-	965	479	2,595	1,554	1,715	10,383
643	3543	Percent	18.2	5.7	-	10.1	5.0	27.0	16.2	17.9	
644	3544	Trips	2,022	-	-	-	2,250	4,141	2,585	2,646	15,224
644	3544	Percent	14.8	-	-	-	16.5	30.4	19.0	19.4	
645	3545	Trips	1,268	-	-	-	907	1,498	1,720	1,351	7,018
645	3545	Percent	18.8	-	-	-	13.5	22.2	25.5	20.0	
646	3546	Trips	986	-	156	520	250	1,081	1,094	1,181	5,470
646	3546	Percent	18.7	-	3.0	9.9	4.7	20.5	20.8	22.4	
647	3547	Trips	350	103	114	165	66	354	359	408	1,979
647	3547	Percent	18.2	5.4	5.9	8.6	3.5	18.5	18.7	21.2	
648	3548	Trips	1,027	434	254	401	48	903	1,001	514	4,747
648	3548	Percent	22.4	9.5	5.5	8.8	1.0	19.7	21.9	11.2	
649	3549	Trips	754	192	184	230	41	612	743	427	3,320
649	3549	Percent	23.7	6.0	5.8	7.2	1.3	19.2	23.3	13.4	
650	3550	Trips	45	80	104	0	14	155	304	133	850
650	3550	Percent	5.4	9.6	12.4	0.0	1.6	18.5	36.5	16.0	

Miami-Dade 2045 Cost Feasible Plan Direction Trip Distribution Summary											
TAZ of Origin		Trips / Percent	Cardinal Directions								Total Trips
County TAZ	Regional TAZ		NNE	ENE	ESE	SSE	SSW	WSW	WNW	NNW	
625	3525	Trips	515	114	-	541	802	1,791	829	1,096	5,972
625	3525	Percent	9.1	2.0	-	9.5	14.1	31.5	14.6	19.3	
626	3526	Trips	66	-	-	-	2,417	3,260	1,417	2,993	11,237
626	3526	Percent	0.7	-	-	-	23.8	32.1	14.0	29.5	
627	3527	Trips	174	-	-	-	2,276	3,212	1,138	1,885	9,055
627	3527	Percent	2.0	-	-	-	26.2	37.0	13.1	21.7	
628	3528	Trips	238	-	23	101	1,053	1,266	390	660	4,028
628	3528	Percent	6.4	-	0.6	2.7	28.2	33.9	10.5	17.7	
629	3529	Trips	1,686	621	373	1,692	1,801	6,032	2,362	2,490	18,425
629	3529	Percent	9.9	3.6	2.2	9.9	10.6	35.4	13.9	14.6	
630	3530	Trips	888	-	326	303	1,717	3,876	1,515	1,553	11,277
630	3530	Percent	8.7	-	3.2	3.0	16.9	38.1	14.9	15.3	
631	3531	Trips	296	-	-	-	1,351	2,360	838	1,324	6,591
631	3531	Percent	4.8	-	-	-	21.9	38.3	13.6	21.5	
632	3532	Trips	343	-	-	-	1,500	2,647	1,390	1,098	7,499
632	3532	Percent	4.9	-	-	-	21.5	37.9	19.9	15.7	
633	3533	Trips	368	-	-	-	1,052	1,986	859	841	5,391
633	3533	Percent	7.2	-	-	-	20.6	38.9	16.8	16.5	
634	3534	Trips	1,404	80	149	773	1,637	2,733	1,332	1,712	10,593
634	3534	Percent	14.3	0.8	1.5	7.9	16.7	27.8	13.6	17.4	
635	3535	Trips	566	-	-	-	1,311	2,266	1,228	1,254	7,246
635	3535	Percent	8.5	-	-	-	19.8	34.2	18.5	18.9	
636	3536	Trips	1,066	-	-	607	978	3,045	1,398	1,193	8,805
636	3536	Percent	12.9	-	-	7.3	11.8	36.8	16.9	14.4	
637	3537	Trips	468	44	144	315	198	868	501	309	2,865
637	3537	Percent	16.5	1.6	5.1	11.1	6.9	30.5	17.6	10.9	
638	3538	Trips	127	33	78	94	79	401	285	185	1,342
638	3538	Percent	9.9	2.6	6.1	7.3	6.2	31.3	22.2	14.5	
639	3539	Trips	944	303	253	1,068	176	2,395	1,085	905	7,569
639	3539	Percent	13.2	4.3	3.6	15.0	2.5	33.6	15.2	12.7	
640	3540	Trips	119	74	216	10	30	177	136	147	1,166
640	3540	Percent	13.1	8.2	23.7	1.1	3.4	19.4	14.9	16.2	
641	3541	Trips	1,145	1,056	206	569	242	2,378	1,724	1,142	9,066
641	3541	Percent	13.5	12.5	2.4	6.7	2.9	28.1	20.4	13.5	
642	3542	Trips	1,701	1,196	113	964	433	3,470	2,140	1,631	12,324
642	3542	Percent	14.6	10.3	1.0	8.3	3.7	29.8	18.4	14.0	
643	3543	Trips	1,884	580	-	1,133	631	3,768	2,190	2,157	13,183
643	3543	Percent	15.3	4.7	-	9.2	5.1	30.5	17.7	17.5	
644	3544	Trips	1,948	-	-	-	2,227	5,534	3,264	3,082	17,780
644	3544	Percent	12.1	-	-	-	13.9	34.5	20.3	19.2	
645	3545	Trips	1,314	-	-	-	844	1,661	2,170	1,703	8,075
645	3545	Percent	17.1	-	-	-	11.0	21.6	28.2	22.1	
646	3546	Trips	1,025	-	125	496	263	1,741	1,656	1,299	6,976
646	3546	Percent	15.5	-	1.9	7.5	4.0	26.4	25.1	19.7	
647	3547	Trips	296	122	96	109	79	582	661	405	2,490
647	3547	Percent	12.6	5.2	4.1	4.6	3.4	24.8	28.1	17.3	
648	3548	Trips	943	278	128	313	73	1,525	1,351	576	5,397
648	3548	Percent	18.2	5.4	2.5	6.0	1.4	29.4	26.0	11.1	
649	3549	Trips	643	120	121	216	43	873	952	508	3,661
649	3549	Percent	18.5	3.4	3.5	6.2	1.3	25.1	27.4	14.6	
650	3550	Trips	60	71	65	8	14	279	312	136	969
650	3550	Percent	6.4	7.5	6.9	0.9	1.5	29.5	33.0	14.4	

Attachment C

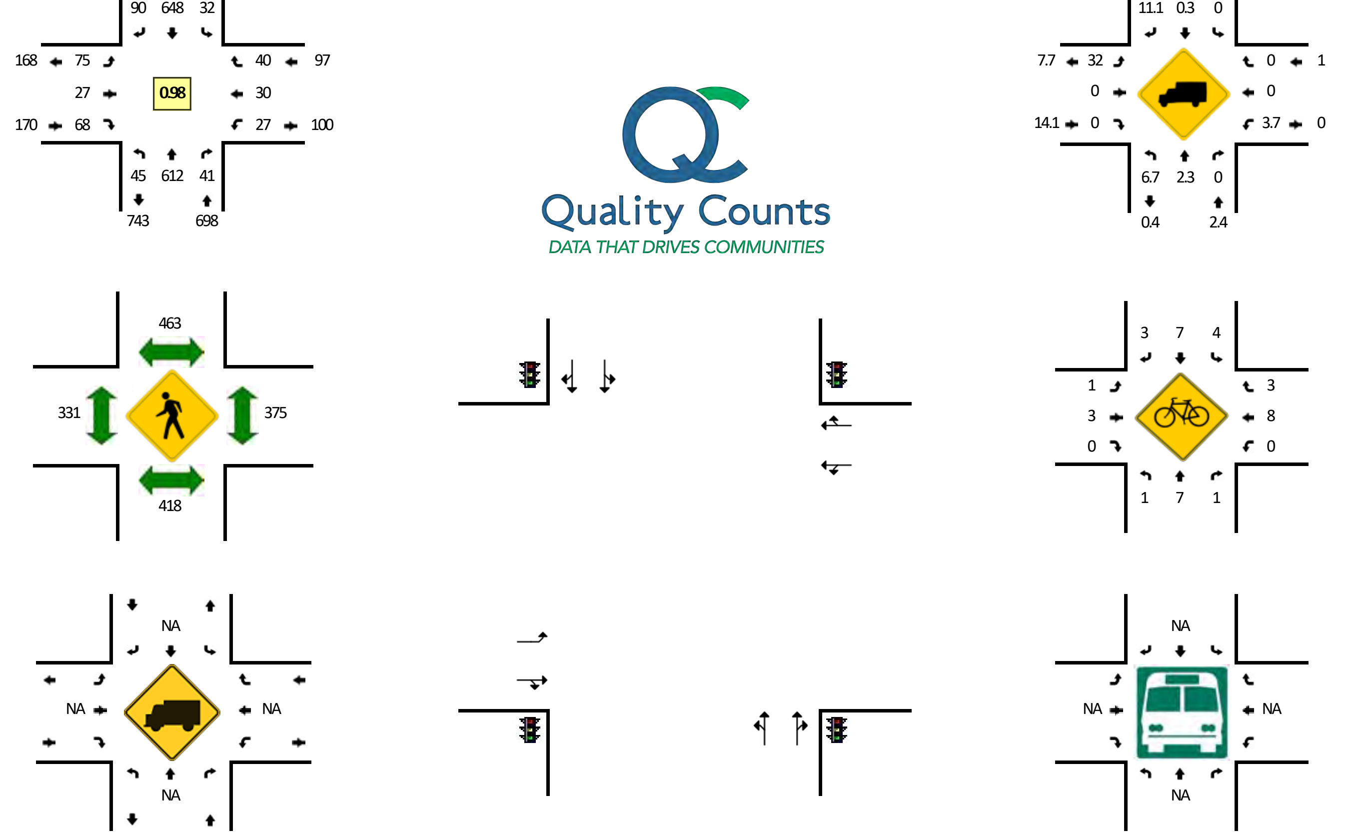
Data Collection

Traffic Counts

Type of peak hour being reported: Intersection Peak — Method for determining peak hour: Total Entering Volume

LOCATION: Collins Ave -- Lincoln Rd — **QC JOB #:** 15001103
CITY/STATE: Miami Beach, FL — **DATE:** Sat, Jun 8 2019

Peak-Hour: 1:45 PM -- 2:45 PM
Peak 15-Min: 1:45 PM -- 2:00 PM



15-Min Count Period Beginning At	Collins Ave (Northbound) Left	Thru	Right	U	Collins Ave (Southbound) Left	Thru	Right	U	Lincoln Rd (Eastbound) Left	Thru	Right	U	Lincoln Rd (Westbound) Left	Thru	Right	U	Total	Hourly Totals
10:00 AM	8	115	7	0	11	128	15	0	15	5	9	0	3	8	9	0	333	
10:15 AM	14	98	5	0	5	137	20	0	13	4	16	0	6	3	10	0	331	
10:30 AM	14	118	9	0	9	154	18	0	17	2	15	1	3	2	16	0	378	
10:45 AM	12	135	10	0	7	133	12	0	11	6	14	1	8	7	9	0	365	1407
11:00 AM	11	123	7	0	3	121	14	0	25	8	19	0	5	10	5	0	351	1425
11:15 AM	14	117	10	0	4	143	24	0	17	3	18	0	5	2	6	0	363	1457
11:30 AM	5	136	12	0	3	130	24	0	18	2	13	1	4	11	14	0	373	1452
11:45 AM	20	152	7	0	6	139	23	0	10	3	23	1	1	11	7	0	403	1490
12:00 PM	10	166	11	0	6	140	24	0	17	6	12	0	6	11	12	0	421	1560
12:15 PM	14	146	9	0	7	136	25	1	16	8	16	4	8	14	11	0	415	1612
12:30 PM	5	141	8	0	10	164	20	0	27	7	11	1	6	11	10	0	421	1660
12:45 PM	21	133	7	0	13	151	18	1	20	3	20	1	5	8	13	0	414	1671
1:00 PM	12	133	9	1	11	143	14	0	17	3	18	1	6	5	10	0	383	1633
1:15 PM	13	138	10	0	11	161	26	0	15	5	17	4	8	8	6	1	423	1641
1:30 PM	7	151	9	0	11	146	20	0	17	8	22	0	8	9	9	1	418	1638
1:45 PM	9	161	7	0	7	170	26	0	20	5	19	1	4	7	8	0	444	1668
2:00 PM	14	166	13	0	10	149	20	0	17	8	15	0	7	12	9	0	440	1725
2:15 PM	11	144	11	0	9	148	22	0	20	5	13	1	8	6	11	0	409	1711
2:30 PM	11	141	10	0	6	181	22	0	15	9	21	1	8	5	12	0	442	1735
2:45 PM	22	149	11	0	9	144	20	1	23	5	13	0	3	9	5	0	414	1705
3:00 PM	16	122	7	0	13	157	24	0	18	9	10	1	12	14	13	1	417	1682
3:15 PM	18	131	15	0	6	128	18	0	19	3	12	3	13	8	8	0	382	1655
3:30 PM	17	163	10	0	10	156	22	0	16	2	13	0	5	12	13	0	439	1652
3:45 PM	16	116	15	0	7	159	14	0	15	7	14	3	6	9	12	0	393	1631

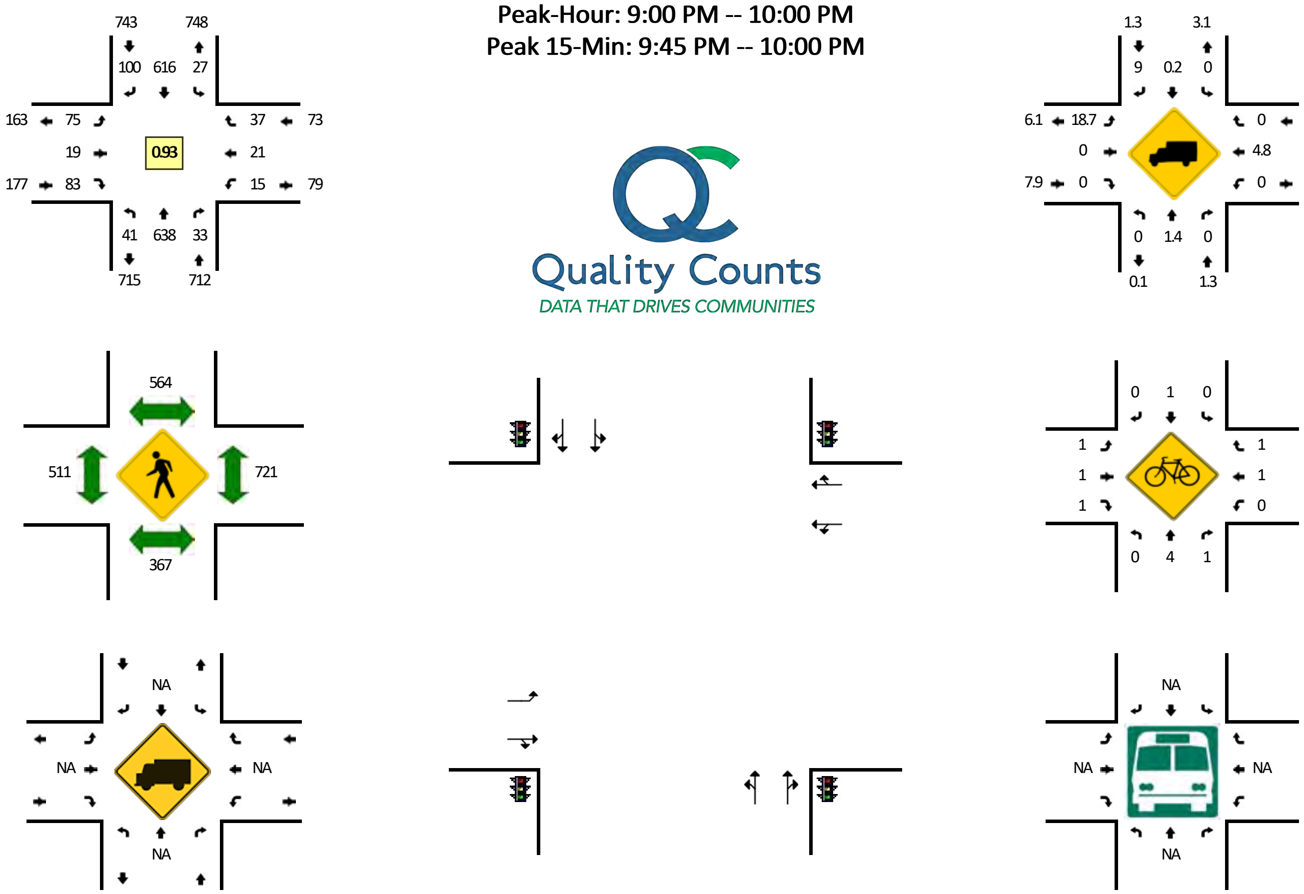
Peak 15-Min Flowrates	Northbound Left	Thru	Right	U	Southbound Left	Thru	Right	U	Eastbound Left	Thru	Right	U	Westbound Left	Thru	Right	U	Total
All Vehicles	36	644	28	0	28	680	104	0	80	20	76	4	16	28	32	0	1776
Heavy Trucks	0	8	0		0	0	12		36	0	0		0	0	0		56
Pedestrians		456				456				348				408			1668
Bicycles	1	2	0		0	0	1		0	0	0		0	0	0		4
Railroad																	
Stopped Buses																	

Comments:

LOCATION: Collins Ave -- Lincoln Rd
CITY/STATE: Miami Beach, FL

QC JOB #: 15001104
DATE: Sat, Jun 8 2019

Peak-Hour: 9:00 PM -- 10:00 PM
Peak 15-Min: 9:45 PM -- 10:00 PM



15-Min Count Period Beginning At	Collins Ave (Northbound)				Collins Ave (Southbound)				Lincoln Rd (Eastbound)				Lincoln Rd (Westbound)				Total	Hourly Totals
	Left	Thru	Right	U	Left	Thru	Right	U	Left	Thru	Right	U	Left	Thru	Right	U		
9:00 PM	9	171	7	0	4	168	36	0	21	6	17	0	5	5	8	0	457	
9:15 PM	10	136	11	0	10	149	23	0	14	5	19	0	5	8	8	0	398	
9:30 PM	13	163	6	0	9	166	24	0	20	4	17	1	2	5	12	0	442	
9:45 PM	8	168	9	1	4	133	17	0	18	4	30	1	3	3	9	0	408	1705
10:00 PM	12	168	5	0	8	145	30	0	22	2	25	1	4	5	13	0	440	1688
10:15 PM	9	143	5	1	3	153	26	0	23	5	25	1	0	4	7	0	405	1695
10:30 PM	8	148	12	0	1	163	35	0	25	8	20	1	2	6	8	0	437	1690
10:45 PM	7	153	9	0	4	127	37	0	14	8	15	0	5	4	9	1	393	1675

Peak 15-Min Flowrates	Northbound				Southbound				Eastbound				Westbound				Total
	Left	Thru	Right	U	Left	Thru	Right	U	Left	Thru	Right	U	Left	Thru	Right	U	
All Vehicles	32	672	36	4	16	532	68	0	72	16	120	4	12	12	36	0	1632
Heavy Trucks	0	8	0		0	0	4		12	0	0		0	4	0		28
Pedestrians		456				520				452				640			2068
Bicycles	0	3	0		0	0	0		0	0	1		0	1	0		5
Railroad																	
Stopped Buses																	

Comments:

Signal Timing

TOD Schedule Report

for 2664: Collins Av&Lincoln Rd

Print Date: 5/8/2020

Print Time: 2:05 AM

Asset	Intersection	TOD Schedule	Op Mode	Plan #	Cycle	Offset	TOD Setting	Active PhaseBank	Active Maximum
2664	Collins Av&Lincoln Rd	DOW-6		[07] NOON/LUNCH	120	0	N/A	1	Max 2

Splits

PH 1	PH 2	PH 3	PH 4	PH 5	PH 6	PH 7	PH 8
-	NBT	-	EBT	-	SBT	-	WBT
0	66	0	42	0	66	0	42

Active Phase Bank: Phase Bank 1

Phase	Walk Phase Bank 1 - 2 - 3	Don't Walk 1 - 2 - 3	Min Initial 1 - 2 - 3	Veh Ext 1 - 2 - 3	Max Limit 1 - 2 - 3	Max 2 1 - 2 - 3	Yellow	Red
1 -	0 - 0 - 0	0 - 0 - 0	0 - 0 - 0	0 - 0 - 0	0 - 0 - 0	0 - 0 - 0	0	0
2 NBT	0 - 5 - 5	0 - 24 - 24	16 - 7 - 7	1 - 1 - 1	35 - 35 - 35	0 - 35 - 31	4	2.5
3 -	0 - 0 - 0	0 - 0 - 0	0 - 0 - 0	0 - 0 - 0	0 - 0 - 0	0 - 0 - 0	0	0
4 EBT	0 - 5 - 5	0 - 17 - 17	7 - 7 - 7	2.5 - 2.5 - 2.5	22 - 30 - 29	50 - 40 - 32	4	2.2
5 -	0 - 0 - 0	0 - 0 - 0	0 - 0 - 0	0 - 0 - 0	0 - 0 - 0	0 - 0 - 0	0	0
6 SBT	0 - 5 - 5	0 - 24 - 24	16 - 7 - 7	1 - 1 - 1	35 - 35 - 35	0 - 35 - 31	4	2.5
7 -	0 - 0 - 0	0 - 0 - 0	0 - 0 - 0	0 - 0 - 0	0 - 0 - 0	0 - 0 - 0	0	0
8 WBT	0 - 5 - 5	0 - 17 - 17	7 - 7 - 7	2.5 - 2.5 - 2.5	22 - 30 - 29	50 - 40 - 32	4	2.2

Last In Service Date: unknown

Permitted Phases

	12345678
Default	-234-6-8
External Permit 0	-234-6-8
External Permit 1	-234-6-8
External Permit 2	-234-6-8

TOD Schedule Report

for 2664: Collins Av&Lincoln Rd

Print Date: 5/8/2020

Print Time: 2:05 AM

Current TOD Schedule	Plan	Cycle	Green Time 1 -	2 NBT	3 -	4 EBT	5 -	6 SBT	7 -	8 WBT	Ring Offset	Offset
	1	100	0	51	0	37	0	51	0	37	0	74
	2	100	0	51	0	37	0	51	0	37	0	68
	3	100	0	51	0	37	0	51	0	37	0	20
	4	100	0	51	0	37	0	51	0	37	0	57
	5	110	0	59	0	39	0	59	0	39	0	8
	6	130	0	76	0	42	0	76	0	42	0	0
	7	120	0	66	0	42	0	66	0	42	0	0
	8	130	0	78	0	40	0	78	0	40	0	81
	11	90	0	45	0	33	0	45	0	33	0	27
	12	90	0	49	0	29	0	49	0	29	0	42
	13	90	0	45	0	33	0	45	0	33	0	42
	14	120	0	69	0	39	0	69	0	39	0	76
	15	120	0	74	0	34	0	74	0	34	0	70
	20	100	0	44	0	44	0	44	0	44	0	53
	22	100	0	51	0	37	0	51	0	37	0	81
	25	140	0	83	0	45	0	83	0	45	0	81

Local TOD Schedule

Time	Plan	DOW
0000	1	Su M T W Th
0000	7	F S
0300	4	Su
0300	3	M T W Th F S
0700	Free	Su M T W Th F S
0930	2	Su M T W Th
1000	5	Su F S
1500	Free	M T W Th
1500	8	Su F S
1500	8	Su F S
1800	20	M T W Th F
2200	6	M T W Th F

Current Time of Day Function

Time	Function	Settings *	Day of Week
0000	TOD OUTPUTS	8-------	SuM T W ThF S
0000	TOD LOCAL MULTIFU	----4---	SuM T W ThF S
0000	PED RECALL	8---4---	ThF S
0200	PED RECALL	--------	ThF S
0300	TOD OUTPUTS	87--4---	SuM T W ThF S
0500	TOD LOCAL MULTIFU	--------	SuM T W ThF S
0530	PED RECALL	8---4---	M T W ThF
0600	TOD OUTPUTS	87----2-	SuM T W ThF S
0700	TOD OUTPUTS	--------	SuM T W ThF S
0930	TOD OUTPUTS	-7----2-	SuM T W ThF S
1500	TOD OUTPUTS	--------	SuM T W ThF S
2200	TOD OUTPUTS	8-------	SuM T W ThF S

Local Time of Day Function

Time	Function	Settings *	Day of Week
0000	TOD OUTPUTS	8-------	SuM T W ThF S
0000	TOD LOCAL MULTIFUNCT	----4---	SuM T W ThF S
0000	PED RECALL	8---4---	ThF S
0000	PED RECALL	--------	SuM T W
0200	PED RECALL	--------	ThF S
0300	TOD OUTPUTS	87--4---	SuM T W ThF S
0500	PED RECALL	8---4---	Su S
0500	TOD LOCAL MULTIFUNCT	--------	SuM T W ThF S
0530	PED RECALL	8---4---	M T W ThF
0600	TOD OUTPUTS	87----2-	SuM T W ThF S
0700	TOD OUTPUTS	--------	SuM T W ThF S
0930	TOD OUTPUTS	-7----2-	SuM T W ThF S
1500	TOD OUTPUTS	--------	SuM T W ThF S
2200	TOD OUTPUTS	8-------	SuM T W ThF S

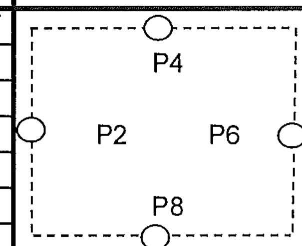
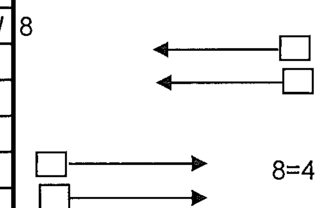
*** Settings**

Blank - FREE - Phase Bank 1, Max 1
Blank - Plan - Phase Bank 1, Max 2
1 - Phase Bank 2, Max 1
2 - Phase Bank 2, Max 2
3 - Phase Bank 3, Max 1
4 - Phase Bank 3, Max 2
5 - EXTERNAL PERMIT 1
6 - EXTERNAL PERMIT 2
7 - X-PED OMIT
8 - TBA

No Calendar Defined/Enabled

SIGNAL OPERATING PLAN

N

	Direction	SB	NB	WB	EB	Ped Heads				
Timing Phases	Head No.	6	2	8	4	P6	P2	P8	P4	Movements/Display/Actuation
	Dwell									
	Clear to									
	Dwell									
	Clear to									
(2+6) N/SB Collins Av (RECALL)	Dwell	G	G	R	R	DW	DW	DW	DW	
	Clear to 4+8	Y	Y	R	R	DW	DW	DW	DW	
X-PED ACTUATED	Dwell	R	R	R	R	W/F	W/F	W/F	W/F	
	Clear to 4+8	R	R	R	R	DW	DW	DW	DW	
(4+8) E/WB 22 Street (ACTUATED)	Dwell	R	R	G	G	DW	DW	DW	DW	
	Clear to 2+6	R	R	Y	Y	DW	DW	DW	DW	
	Dwell									
	Clear to									
Flashing Operation		FY	FY	FR	FR					Page 1 of 1

PEDS HAVE A DUAL MODE OPERARTION:
CONCURRENT OR EXCLUSIVE, SEE TIMING.
SHOWN HERE IS THE EXCLUSIVE ONLY

Miami-Dade County Public Works Department

Drawn	Date	
WILLIAM RIVERA PAZ	9/10/2014	COLLINS AV & LINCOLN RD

Checked	Date	Placed in Service		Phasing No.	Asset Number
H. HERNANDEZ	9/10/14	Date 12/01/2014	By UND	4	2664

Blank12

(E)

PREPARE TO STOP

W3-1b
24"x48"
700-89-143 2 EA
[ACTIVATED BLANK OUT SIGN]

(X)

COLLINS AV
AIA

D3 (6'x2')
TO REMAIN

(Y)

LINCOLN RD

D3 (6'x2')
TO REMAIN

2 4 6 8
R Y G
3-SECT., 1-WAY
EXISTING TO REMAIN

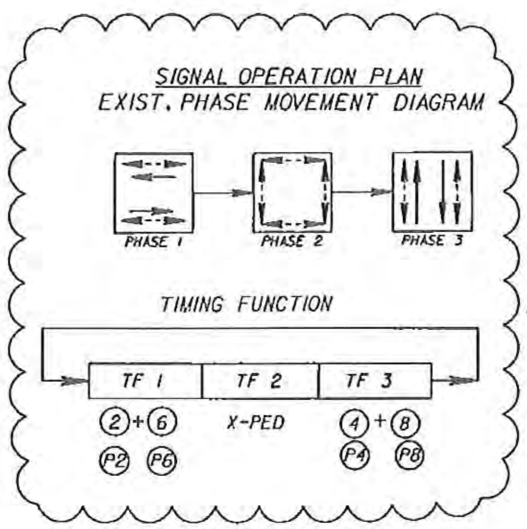
1/6
R Y Y G G
5-SECT., 1-WAY
EXISTING TO BE REMOVED

6
R Y G
3-SECT., 1-WAY
1 AS
650- 1-311

CONTROLLER OPERATION

MAJOR STREET: COLLINS AVENUE
MINOR STREET: LINCOLN ROAD
SOP AS SHOWN

SIGNAL OPERATION PLAN
EXIST. PHASE MOVEMENT DIAGRAM

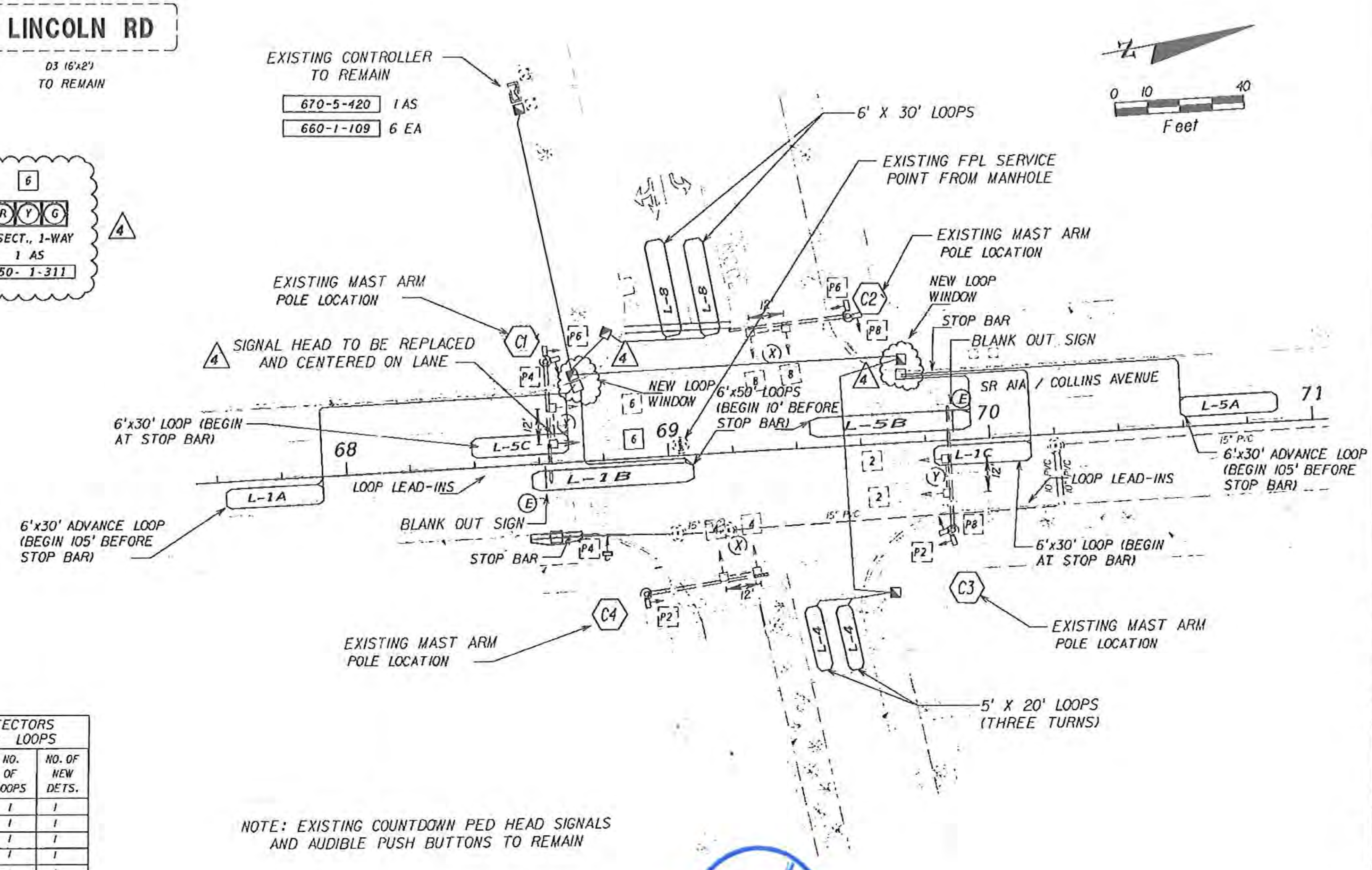


GENERAL NOTES:

1. SIGNAL TIMING TO BE PROVIDED BY MIAMI-DADE COUNTY SIGNAL DIVISION.
2. THE CONTRACTOR SHALL INSTALL CONDUITS TO AVOID ANY DAMAGE TO EXISTING LOOPS AND THEIR OPERATION. ANY DAMAGE TO EXISTING LOOPS AND THEIR OPERATION DURING CONSTRCTION SHALL BE AT THE CONTRACTOR EXPENSE AND REPLACED IN ACCORDANCE WITH LATEST FDOT AND MIAMI DADE TRAFFIC SIGNAL AND SIGN DIVISION STANDARDS.
3. SIDEWALK FLAG REPLACEMENT DUE TO PULL BOX INSTALLATION SHALL BE INCLUDED IN THE COST OF PULL BOX PAY ITEM 635-1-11

DETECTORS FOR LOOPS

LOOP	NO. OF LOOPS	NO. OF NEW DETS.
L-1A	1	1
L-1B	1	1
L-1C	1	1
L-5A	1	1
L-5B	1	1
L-5C	1	1



NOTE: EXISTING COUNTDOWN PED HEAD SIGNALS AND AUDIBLE PUSH BUTTONS TO REMAIN

BLANK-OUT SIGN AND DETECTION NOTES:

1. BLANK-OUT SIGN WILL ONLY TURN ON DURING THE GREEN AND YELLOW INTERVALS.
2. THE "PREPARE TO STOP" BLANK-OUT SIGN WILL REMAIN OFF UNTIL A VEHICLE IS DETECTED FOR MORE THAN 4 SECONDS BY EITHER L-1B OR L-5B.
3. WHEN VEHICLE IS DETECTED FOR MORE THAN 4 SECONDS BY LOOPS L-1A, L-1B AND L-1C OR BY L-5A, L-5B AND L-5C, THE "PREPARE TO STOP" BLANK-OUT SIGN WILL TURN OFF.

CONDUITS AND PULL BOXES:
630-1-13 400 FT
635-1-11 3 EA

LOOP ASSEMBLY:
660-2-101 6 AS

UNDER POWER CORP.
7900 NW 60 Street
Doral, FL 33166
1/20/15

REMOVAL ITEMS:
690-100 1 PI

SR-AIA /COLLINS AVENUE AND LINCOLN RD
INTERSECTION ID # 2664

NOTICE: THE OFFICIAL RECORD OF THIS SHEET IS THE ELECTRONIC FILE SIGNED AND SEALED UNDER RULE 61G15-23.003, F.A.C.

REVISIONS

DATE	DESCRIPTION	DATE	DESCRIPTION
1/15/2013	1 CHANGE VIDEO DETECTION SYSTEM TO LOOP DETECTOR SYSTEM	12/5/2013	4 W3-1b SIGN SIZE MODIFIED. RELOCATE LOOP WINDOWS. SOP MODIFIED; SIGNAL HEAD 6 ADDED. EXISTING LOOPS ADDED

FDOT DISTRICT SIX
VINOD TULI, P.E.
1000 N.W. 111TH AVENUE
MIAMI, FLORIDA 33172
P.E. NO. 44916

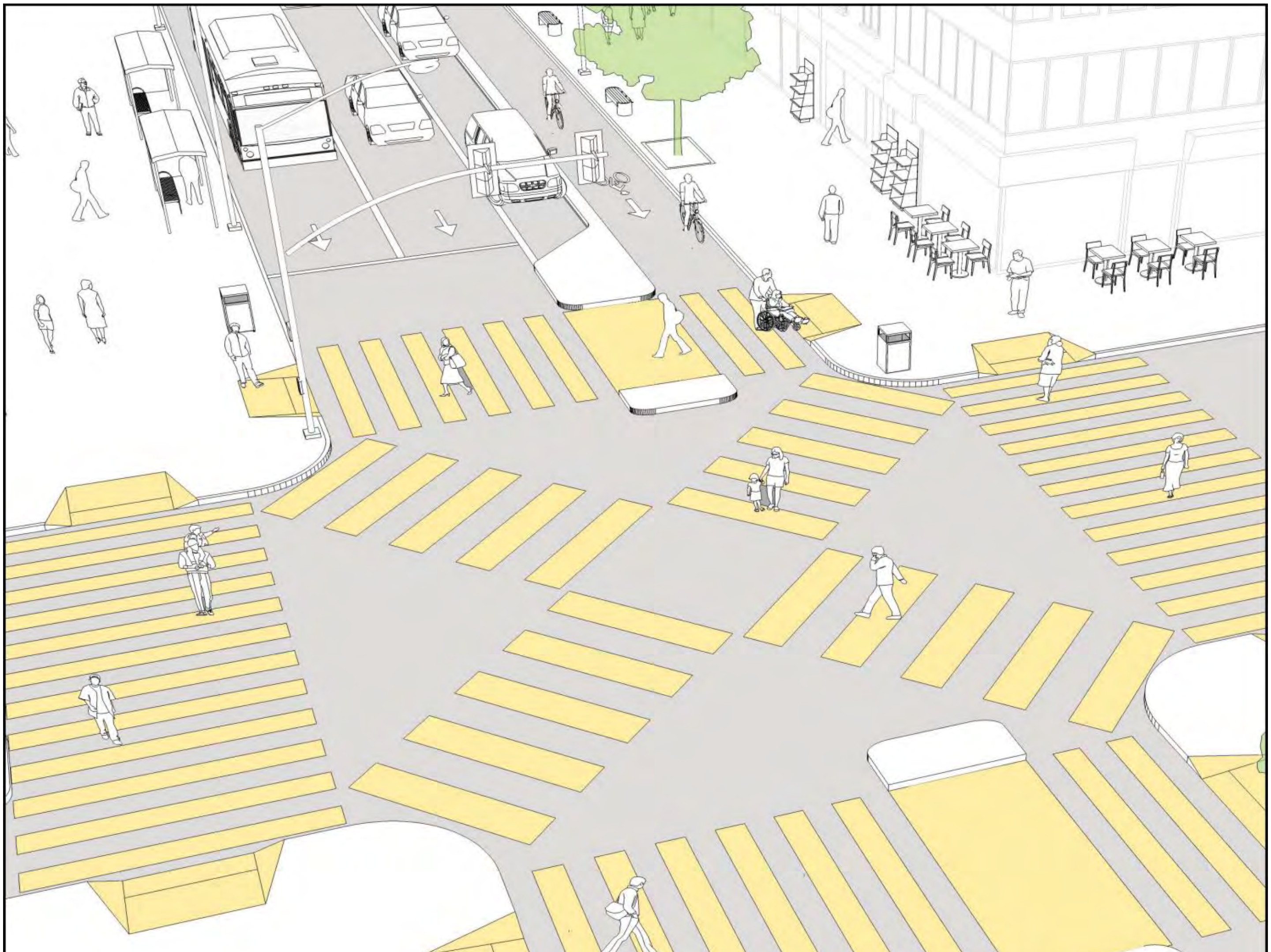
STATE OF FLORIDA
DEPARTMENT OF TRANSPORTATION

ROAD NO.	COUNTY	FINANCIAL PROJECT ID
AIA	MIAMI-DADE	250236-3-52-01

SIGNALIZATION PLAN

SHEET NO.
T-18

KNAPTLG 1/16/2015 3:52:04 PM c:\e\Projects\25023615201\signals\PLANSG12Rev4.dgn



COLLINS AVENUE AND LINCOLN ROAD PEDESTRIAN SCRAMBLE

Technical Memorandum

City of Miami Beach Transportation and Mobility Department

a scramble crossing was installed in December 2020, at the intersection of NE 1st Avenue and NE 2nd Street, in Downtown Miami.

A pedestrian scramble is an exclusive pedestrian phase (all motor vehicles must stop) which also allows pedestrians to cross the intersection in all directions including diagonally. According to the National Association of City Transportation Officials, some of the benefits of pedestrian scramble include:

- increases pedestrian visibility
- reduces conflicts between vehicles and pedestrians
- reduces pedestrian crossing time and exposure
- reduces the buffer zone between vehicles and pedestrians

Pedestrian scrambles have been estimated to have a Crash Reduction Factor of 35%.

According to a study presented in the *Transportation Research Record: Journal of the Transportation Research Board,* No. 847, 1982, entitled "Effect of Pedestrian Signals and Signal Timing on Pedestrian Accidents" (Zeeger, Opiela, and Cynecki), exclusive pedestrian signal phases, which include pedestrian scrambles, produced significantly fewer pedestrian crashes as compared to locations with concurrent pedestrian timing. It was noted that exclusive pedestrian phases were most effective at locations with pedestrian volumes of more than 1,200 people per day. It is important to note that *the study intersection has more than 2,300 pedestrian crosings in a single peak hour*.

While this study serves as more of a proof of concept rather than a definitive design proposal, there are basic physical elements which would be required at any pedestrian scramble. Below is a list of basic these elements:

- Additional pedestrian signal heads for the diagonal crossings,
- Additional curb cuts for the diagonal crossings,
- Diagonal crossing markings or high-emphasis crosswalk markings,
- Diagonal crossing signage.

Operationally, a pedestrian scramble would simply operate as an exclusive pedestrian signal phase.

Project Area/Existing Conditions

The intersection of Collins Avenue and Lincoln Road is one of the most active intersections in the City in terms of its multi-modal demand. The intersection serves as a central link between the Beachwalk and the Lincoln Road Mall in the east-west direction. In the north-south direction, the intersection is one of the main hubs for tourists visiting the Collins Avenue commercial corridor. Further, the intersection is highly served by transit. The principal transit hub is City of Miami Beach is just over 200 feet west of this intersection.

Geometry and Land-Use

- Northbound (SR A1A/Collins Avenue): this approach consists of one (1) shared through/left turn lane and one (1) shared through/right turn lane.
- Southbound (SR A1A/Collins Avenue): this approach consists of one (1) shared through/left turn lane and one shared through/right turn lane.
- Eastbound (Lincoln Road): this approach consists of one (1) left-turn lane and one (1) shared through/right turn lane

▪ Westbound (Lincoln Road): this approach consists of a single lane for all movements.

Collins Avenue is classified as an urban arterial (SR A1A) under the jurisdiction of the Florida Department of Transportation (FDOT) District 6. Annual Average Daily Traffic along this section of Collins Avenue is 27,500 vehicles. Land-use throughout this section of the corridor is commercial and consists of retail, restaurant, and hotel uses. Lincoln Road is classified as an urban collector. Similar to Collins Avenue, Lincoln Road is a commercial corridor with high pedestrian and transit activity. While Collins Avenue has no on-street parking along this section, Lincoln Road has some on street parking on the north and south side of the street.

Figure 1. Project Area

Pedestrian Facilities and Signalization

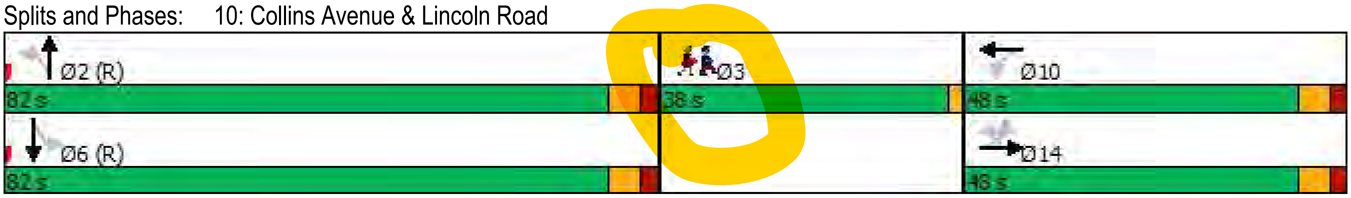
The intersection of Collins Avenue and Lincoln Road is a high pedestrian activity. Currently, the intersection is used by more than 2000 pedestrians during the peak hour. The intersection is currently equipped with crosswalks on all approaches and wide sidewalks (>8 feet). The crosswalk on the west leg of the intersection is overlayed with black and white pavers. All other crosswalks are parallel white lines. All corners are equipped with standard ADA ramps.

Given the high pedestrian activity, the intersection is currently programed for a recall exclusive pedestrian phase between 7AM and 9:30AM and between 3PM and 3AM on weekdays and weekends. Outside of these hours, the pedestrian movements operate as concurrent phases. Given the high number of tourists using this intersection, the pedestrian movements are programmed to come on automatically all day except between 3AM and 7AM.

Phase Number	2	3	6	10	14
Movement	NBTL	Ped	SBTL	WBTL	EBTL
Lead/Lag					
Lead-Lag Optimize					
Recall Mode	C-Max	Ped	C-Max	None	None
Maximum Split (s)	82	38	82	48	48
Maximum Split (%)	48.8%	22.6%	48.8%	28.6%	28.6%
Minimum Split (s)	22.5	38	22.5	13.2	13.2
Yellow Time (s)	4	2	4	4	4
All-Red Time (s)	2.5	0	2.5	2.2	2.2
Minimum Initial (s)	16	1	16	7	7
Vehicle Extension (s)	3	3	3	3	3
Minimum Gap (s)	3	3	3	3	3
Time Before Reduce (s)	0	0	0	0	0
Time To Reduce (s)	0	0	0	0	0
Walk Time (s)		4			
Flash Dont Walk (s)		32			
Dual Entry	Yes	No	Yes	Yes	Yes
Inhibit Max	Yes	Yes	Yes	Yes	Yes
Start Time (s)	0	82	0	120	120
End Time (s)	82	120	82	0	0
Yield/Force Off (s)	75.5	118	75.5	161.8	161.8
Yield/Force Off 170(s)	75.5	86	75.5	161.8	161.8
Local Start Time (s)	0	82	0	120	120
Local Yield (s)	75.5	118	75.5	161.8	161.8
Local Yield 170(s)	75.5	86	75.5	161.8	161.8

Intersection Summary	
Cycle Length	168
Control Type	Actuated-Coordinated
Natural Cycle	110
Offset: 0 (0%), Referenced to phase 2:NBTL and 6:SBTL, Start of Green	

Splits and Phases: 10: Collins Avenue & Lincoln Road



TIP / LRTP

Project Details - NW00153

Field Name	Field Value
LRTP Project Code	NW00153
Facility	Lincoln Road
Limit From	Beachwalk
Limit To	SR A1A / Collins Ave
Description	On-Road Bicycle Facility Improvement
LRTP Year	2045
Project Type	Bicycle/Pedestrian Improvements
Agency Name	Miami-Dade Dept. of Transportation and Public Works
Purpose	
Last Approved Date	
Last Approved User Name	
Last Amended Date	
Last Amended User Name	
Project Costs Funded	$7.337M
Total Capital Cost	$3.579M

Priority Data

	P1 2020-2025(Y-O-E$)	P2 2026-2030(Y-O-E$)	P3 2031-2035(Y-O-E$)	P4 2036-2045(Y-O-E$)
Preliminary Engineering	$M	$M	$M	$0.753M
Right of Way	$M	$M	$M	$M
Construction	$M	$M	$M	$6.584M
Operations and Maintenance	$M	$M	$M	$M
Capital	$M	$M	$M	$M

Attachment D

Intersection Capacity Analysis

Existing

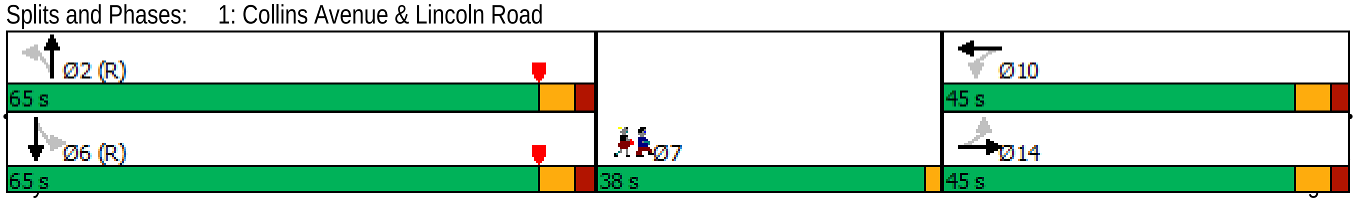
Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (vph)	78	28	71	28	31	42	47	636	43	33	674	94
Future Volume (vph)	78	28	71	28	31	42	47	636	43	33	674	94
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Total Lost time (s)	9.2	9.2			9.2			9.5			9.5	
Lane Util. Factor	1.00	1.00			1.00			0.95			0.95	
Frpb, ped/bikes	1.00	0.69			0.81			0.98			0.97	
Flpb, ped/bikes	0.65	1.00			0.90			1.00			0.99	
Frt	1.00	0.89			0.94			0.99			0.98	
Flt Protected	0.95	1.00			0.99			1.00			1.00	
Satd. Flow (prot)	798	955			1036			3051			3022	
Flt Permitted	0.68	1.00			0.87			0.78			0.88	
Satd. Flow (perm)	571	955			915			2375			2656	
Peak-hour factor, PHF	0.98	0.98	0.98	0.98	0.98	0.98	0.98	0.98	0.98	0.98	0.98	0.98
Adj. Flow (vph)	80	29	72	29	32	43	48	649	44	34	688	96
RTOR Reduction (vph)	0	0	0	0	0	0	0	0	0	0	0	0
Lane Group Flow (vph)	80	101	0	0	104	0	0	741	0	0	818	0
Confl. Peds. (#/hr)	472		426	426		472	338		383	383		338
Heavy Vehicles (%)	32%	0%	0%	4%	0%	0%	7%	2%	0%	0%	1%	11%
Parking (#/hr)		0	0		0	0						
Turn Type	Perm	NA		Perm	NA		Perm	NA		Perm	NA	
Protected Phases		14			10			2			6	
Permitted Phases	14			10			2			6		
Actuated Green, G (s)	23.5	23.5			23.5			73.8			73.8	
Effective Green, g (s)	20.5	20.5			20.5			70.8			70.8	
Actuated g/C Ratio	0.14	0.14			0.14			0.48			0.48	
Clearance Time (s)	6.2	6.2			6.2			6.5			6.5	
Vehicle Extension (s)	2.5	2.5			2.5			1.0			1.0	
Lane Grp Cap (vph)	79	132			126			1136			1270	
v/s Ratio Prot		0.11										
v/s Ratio Perm	c0.14				0.11			c0.31			0.31	
v/c Ratio	1.01	0.77			0.83			0.65			0.64	
Uniform Delay, d1	63.8	61.4			62.0			29.3			29.1	
Progression Factor	1.00	1.00			1.00			1.00			1.00	
Incremental Delay, d2	104.8	21.9			33.1			2.9			2.5	
Delay (s)	168.5	83.4			95.1			32.2			31.6	
Level of Service	F	F			F			C			C	
Approach Delay (s)		121.0			95.1			32.2			31.6	
Approach LOS		F			F			C			C	

Intersection Summary			
HCM 2000 Control Delay	44.2	HCM 2000 Level of Service	D
HCM 2000 Volume to Capacity ratio	0.53		
Actuated Cycle Length (s)	148.0	Sum of lost time (s)	20.7
Intersection Capacity Utilization	89.9%	ICU Level of Service	E
Analysis Period (min)	15		
c Critical Lane Group			

Lane Group	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (vph)	78	28	71	28	31	42	47	636	43	33	674	94
Future Volume (vph)	78	28	71	28	31	42	47	636	43	33	674	94
Confl. Peds. (#/hr)	472		426	426		472	338		383	383		338
Peak Hour Factor	0.98	0.98	0.98	0.98	0.98	0.98	0.98	0.98	0.98	0.98	0.98	0.98
Heavy Vehicles (%)	32%	0%	0%	4%	0%	0%	7%	2%	0%	0%	1%	11%
Parking (#/hr)		0	0		0	0						
Shared Lane Traffic (%)												
Lane Group Flow (vph)	80	101	0	0	104	0	0	741	0	0	818	0
Turn Type	Perm	NA		Perm	NA		Perm	NA		Perm	NA	
Protected Phases		14			10			2			6	
Permitted Phases	14			10			2			6		
Detector Phase	14	14		10	10		2	2		6	6	
Switch Phase												
Minimum Initial (s)	7.0	7.0		7.0	7.0		7.0	7.0		7.0	7.0	
Minimum Split (s)	13.2	13.2		13.2	13.2		22.5	22.5		22.5	22.5	
Total Split (s)	45.0	45.0		45.0	45.0		65.0	65.0		65.0	65.0	
Total Split (%)	30.4%	30.4%		30.4%	30.4%		43.9%	43.9%		43.9%	43.9%	
Yellow Time (s)	4.0	4.0		4.0	4.0		4.0	4.0		4.0	4.0	
All-Red Time (s)	2.2	2.2		2.2	2.2		2.5	2.5		2.5	2.5	
Lost Time Adjust (s)	3.0	3.0			3.0			3.0			3.0	
Total Lost Time (s)	9.2	9.2			9.2			9.5			9.5	
Lead/Lag												
Lead-Lag Optimize?												
Recall Mode	None	None		None	None		C-Max	C-Max		C-Max	C-Max	
v/c Ratio	0.95	0.71			0.78			0.66			0.65	
Control Delay	145.5	84.4			93.5			34.7			34.0	
Queue Delay	0.0	0.0			0.0			0.4			0.5	
Total Delay	145.5	84.4			93.5			35.1			34.6	
Queue Length 50th (ft)	78	95			98			281			308	
Queue Length 95th (ft)	134	148			153			420			452	
Internal Link Dist (ft)		137			302			499			519	
Turn Bay Length (ft)												
Base Capacity (vph)	149	249			235			1129			1260	
Starvation Cap Reductn	0	0			0			94			144	
Spillback Cap Reductn	0	0			0			0			0	
Storage Cap Reductn	0	0			0			0			0	
Reduced v/c Ratio	0.54	0.41			0.44			0.72			0.73	

Intersection Summary
Cycle Length: 148
Actuated Cycle Length: 148
Offset: 8 (5%), Referenced to phase 2:NBTL and 6:SBTL, Start of Yellow
Natural Cycle: 110
Control Type: Actuated-Coordinated

Splits and Phases: 1: Collins Avenue & Lincoln Road



Lane Group	Ø7
Lane Configurations	
Traffic Volume (vph)	
Future Volume (vph)	
Confl. Peds. (#/hr)	
Peak Hour Factor	
Heavy Vehicles (%)	
Parking (#/hr)	
Shared Lane Traffic (%)	
Lane Group Flow (vph)	
Turn Type	
Protected Phases	7
Permitted Phases	
Detector Phase	
Switch Phase	
Minimum Initial (s)	1.0
Minimum Split (s)	38.0
Total Split (s)	38.0
Total Split (%)	26%
Yellow Time (s)	2.0
All-Red Time (s)	0.0
Lost Time Adjust (s)	
Total Lost Time (s)	
Lead/Lag	
Lead-Lag Optimize?	
Recall Mode	Ped
v/c Ratio	
Control Delay	
Queue Delay	
Total Delay	
Queue Length 50th (ft)	
Queue Length 95th (ft)	
Internal Link Dist (ft)	
Turn Bay Length (ft)	
Base Capacity (vph)	
Starvation Cap Reductn	
Spillback Cap Reductn	
Storage Cap Reductn	
Reduced v/c Ratio	

Intersection Summary

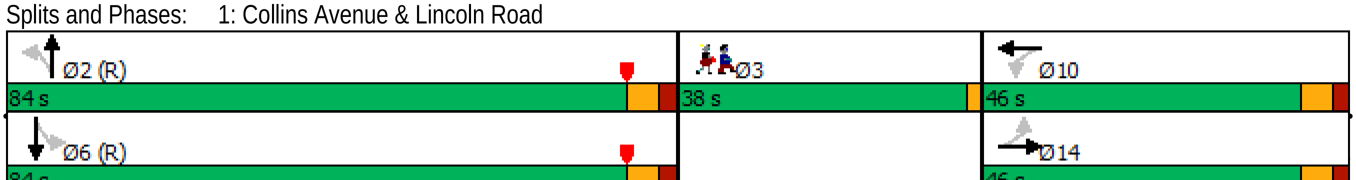
Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (vph)	78	20	86	16	22	38	43	664	34	28	641	104
Future Volume (vph)	78	20	86	16	22	38	43	664	34	28	641	104
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Total Lost time (s)	9.2	9.2			9.2			9.5			9.5	
Lane Util. Factor	1.00	1.00			1.00			0.95			0.95	
Frpb, ped/bikes	1.00	0.67			0.75			0.97			0.96	
Flpb, ped/bikes	0.58	1.00			0.93			1.00			0.99	
Frt	1.00	0.88			0.93			0.99			0.98	
Flt Protected	0.95	1.00			0.99			1.00			1.00	
Satd. Flow (prot)	797	906			978			3092			2969	
Flt Permitted	0.71	1.00			0.91			0.79			0.88	
Satd. Flow (perm)	600	906			900			2465			2624	
Peak-hour factor, PHF	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93
Adj. Flow (vph)	84	22	92	17	24	41	46	714	37	30	689	112
RTOR Reduction (vph)	0	0	0	0	0	0	0	0	0	0	0	0
Lane Group Flow (vph)	84	114	0	0	82	0	0	797	0	0	831	0
Confl. Peds. (#/hr)	575		374	374		575	521		735	735		521
Heavy Vehicles (%)	19%	0%	0%	0%	5%	0%	0%	1%	0%	0%	1%	9%
Parking (#/hr)		0	0		0	0						
Turn Type	Perm	NA		Perm	NA		Perm	NA		Perm	NA	
Protected Phases		14			10			2			6	
Permitted Phases	14			10			2			6		
Actuated Green, G (s)	25.8	25.8			25.8			91.5			91.5	
Effective Green, g (s)	22.8	22.8			22.8			88.5			88.5	
Actuated g/C Ratio	0.14	0.14			0.14			0.53			0.53	
Clearance Time (s)	6.2	6.2			6.2			6.5			6.5	
Vehicle Extension (s)	2.5	2.5			2.5			1.0			1.0	
Lane Grp Cap (vph)	81	122			122			1298			1382	
v/s Ratio Prot		0.13										
v/s Ratio Perm	c0.14				0.09			c0.32			0.32	
v/c Ratio	1.04	0.93			0.67			0.61			0.60	
Uniform Delay, d1	72.6	71.9			69.0			27.8			27.5	
Progression Factor	1.00	1.00			1.00			1.00			1.00	
Incremental Delay, d2	110.5	60.9			12.4			2.2			1.9	
Delay (s)	183.1	132.8			81.5			30.0			29.5	
Level of Service	F	F			F			C			C	
Approach Delay (s)		154.1			81.5			30.0			29.5	
Approach LOS		F			F			C			C	

Intersection Summary			
HCM 2000 Control Delay	44.9	HCM 2000 Level of Service	D
HCM 2000 Volume to Capacity ratio	0.53		
Actuated Cycle Length (s)	168.0	Sum of lost time (s)	20.7
Intersection Capacity Utilization	89.3%	ICU Level of Service	E
Analysis Period (min)	15		
c Critical Lane Group			

Lane Group	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (vph)	78	20	86	16	22	38	43	664	34	28	641	104
Future Volume (vph)	78	20	86	16	22	38	43	664	34	28	641	104
Confl. Peds. (#/hr)	575		374	374		575	521		735	735		521
Peak Hour Factor	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93
Heavy Vehicles (%)	19%	0%	0%	0%	5%	0%	0%	1%	0%	0%	1%	9%
Parking (#/hr)		0	0		0	0						
Shared Lane Traffic (%)												
Lane Group Flow (vph)	84	114	0	0	82	0	0	797	0	0	831	0
Turn Type	Perm	NA		Perm	NA		Perm	NA		Perm	NA	
Protected Phases		14			10			2			6	
Permitted Phases	14			10			2			6		
Detector Phase	14	14		10	10		2	2		6	6	
Switch Phase												
Minimum Initial (s)	7.0	7.0		7.0	7.0		16.0	16.0		16.0	16.0	
Minimum Split (s)	13.2	13.2		13.2	13.2		22.5	22.5		22.5	22.5	
Total Split (s)	46.0	46.0		46.0	46.0		84.0	84.0		84.0	84.0	
Total Split (%)	27.4%	27.4%		27.4%	27.4%		50.0%	50.0%		50.0%	50.0%	
Yellow Time (s)	4.0	4.0		4.0	4.0		4.0	4.0		4.0	4.0	
All-Red Time (s)	2.2	2.2		2.2	2.2		2.5	2.5		2.5	2.5	
Lost Time Adjust (s)	3.0	3.0			3.0			3.0			3.0	
Total Lost Time (s)	9.2	9.2			9.2			9.5			9.5	
Lead/Lag												
Lead-Lag Optimize?												
Recall Mode	None	None		None	None		C-Max	C-Max		C-Max	C-Max	
v/c Ratio	0.93	0.87			0.63			0.61			0.60	
Control Delay	146.7	118.7			87.0			32.0			31.4	
Queue Delay	0.0	0.0			0.0			1.2			1.1	
Total Delay	146.7	118.7			87.0			33.2			32.5	
Queue Length 50th (ft)	93	125			87			315			326	
Queue Length 95th (ft)	154	189			139			455			465	
Internal Link Dist (ft)		137			302			499			519	
Turn Bay Length (ft)												
Base Capacity (vph)	146	213			211			1297			1380	
Starvation Cap Reductn	0	0			0			279			306	
Spillback Cap Reductn	0	0			0			0			0	
Storage Cap Reductn	0	0			0			0			0	
Reduced v/c Ratio	0.58	0.54			0.39			0.78			0.77	

Intersection Summary
Cycle Length: 168
Actuated Cycle Length: 168
Offset: 81 (48%), Referenced to phase 2:NBTL and 6:SBTL, Start of Yellow
Natural Cycle: 110
Control Type: Actuated-Coordinated

Splits and Phases: 1: Collins Avenue & Lincoln Road



Lane Group	Ø3
Lane Configurations	
Traffic Volume (vph)	
Future Volume (vph)	
Confl. Peds. (#/hr)	
Peak Hour Factor	
Heavy Vehicles (%)	
Parking (#/hr)	
Shared Lane Traffic (%)	
Lane Group Flow (vph)	
Turn Type	
Protected Phases	3
Permitted Phases	
Detector Phase	
Switch Phase	
Minimum Initial (s)	1.0
Minimum Split (s)	38.0
Total Split (s)	38.0
Total Split (%)	23%
Yellow Time (s)	2.0
All-Red Time (s)	0.0
Lost Time Adjust (s)	
Total Lost Time (s)	
Lead/Lag	
Lead-Lag Optimize?	
Recall Mode	Ped
v/c Ratio	
Control Delay	
Queue Delay	
Total Delay	
Queue Length 50th (ft)	
Queue Length 95th (ft)	
Internal Link Dist (ft)	
Turn Bay Length (ft)	
Base Capacity (vph)	
Starvation Cap Reductn	
Spillback Cap Reductn	
Storage Cap Reductn	
Reduced v/c Ratio	

Intersection Summary

Midday Events

HCM Signalized Intersection Capacity Analysis
1: Collins Avenue & Lincoln Road

Midday Event Traffic (Arrival)
09/29/2021

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (vph)	82	100	71	28	31	42	47	679	43	33	693	96
Future Volume (vph)	82	100	71	28	31	42	47	679	43	33	693	96
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Total Lost time (s)	9.2	9.2			9.2			9.5			9.5	
Lane Util. Factor	1.00	1.00			1.00			0.95			0.95	
Frpb, ped/bikes	1.00	0.83			0.82			0.98			0.97	
Flpb, ped/bikes	0.66	1.00			0.92			1.00			0.99	
Frt	1.00	0.94			0.94			0.99			0.98	
Flt Protected	0.95	1.00			0.99			1.00			1.00	
Satd. Flow (prot)	814	1200			1074			3056			3021	
Flt Permitted	0.69	1.00			0.73			0.76			0.87	
Satd. Flow (perm)	588	1200			792			2337			2633	
Peak-hour factor, PHF	0.98	0.98	0.98	0.98	0.98	0.98	0.98	0.98	0.98	0.98	0.98	0.98
Adj. Flow (vph)	84	102	72	29	32	43	48	693	44	34	707	98
RTOR Reduction (vph)	0	0	0	0	0	0	0	0	0	0	0	0
Lane Group Flow (vph)	84	174	0	0	104	0	0	785	0	0	839	0
Confl. Peds. (#/hr)	472		426	426		472	338		383	383		338
Heavy Vehicles (%)	32%	0%	0%	4%	0%	0%	7%	2%	0%	0%	1%	11%
Parking (#/hr)		0	0		0	0						
Turn Type	Perm	NA		Perm	NA		Perm	NA		Perm	NA	
Protected Phases		14			10			2			6	
Permitted Phases	14			10			2			6		
Actuated Green, G (s)	26.1	26.1			26.1			71.2			71.2	
Effective Green, g (s)	23.1	23.1			23.1			68.2			68.2	
Actuated g/C Ratio	0.16	0.16			0.16			0.46			0.46	
Clearance Time (s)	6.2	6.2			6.2			6.5			6.5	
Vehicle Extension (s)	2.5	2.5			2.5			1.0			1.0	
Lane Grp Cap (vph)	91	187			123			1076			1213	
v/s Ratio Prot		c0.15										
v/s Ratio Perm	0.14				0.13			c0.34			0.32	
v/c Ratio	0.92	0.93			0.85			0.73			0.69	
Uniform Delay, d1	61.6	61.7			60.7			32.4			31.6	
Progression Factor	1.00	1.00			1.00			1.00			1.00	
Incremental Delay, d2	69.1	46.2			37.9			4.4			3.3	
Delay (s)	130.7	107.9			98.6			36.8			34.8	
Level of Service	F	F			F			D			C	
Approach Delay (s)		115.3			98.6			36.8			34.8	
Approach LOS		F			F			D			C	

Intersection Summary			
HCM 2000 Control Delay	49.4	HCM 2000 Level of Service	D
HCM 2000 Volume to Capacity ratio	0.56		
Actuated Cycle Length (s)	148.0	Sum of lost time (s)	20.7
Intersection Capacity Utilization	107.4%	ICU Level of Service	G
Analysis Period (min)	15		
c Critical Lane Group			

Lane Group	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (vph)	82	100	71	28	31	42	47	679	43	33	693	96
Future Volume (vph)	82	100	71	28	31	42	47	679	43	33	693	96
Confl. Peds. (#/hr)	472		426	426		472	338		383	383		338
Peak Hour Factor	0.98	0.98	0.98	0.98	0.98	0.98	0.98	0.98	0.98	0.98	0.98	0.98
Heavy Vehicles (%)	32%	0%	0%	4%	0%	0%	7%	2%	0%	0%	1%	11%
Parking (#/hr)		0	0		0	0						
Shared Lane Traffic (%)												
Lane Group Flow (vph)	84	174	0	0	104	0	0	785	0	0	839	0
Turn Type	Perm	NA		Perm	NA		Perm	NA		Perm	NA	
Protected Phases		14			10			2			6	
Permitted Phases	14			10			2			6		
Detector Phase	14	14		10	10		2	2		6	6	
Switch Phase												
Minimum Initial (s)	7.0	7.0		7.0	7.0		7.0	7.0		7.0	7.0	
Minimum Split (s)	13.2	13.2		13.2	13.2		22.5	22.5		22.5	22.5	
Total Split (s)	45.0	45.0		45.0	45.0		65.0	65.0		65.0	65.0	
Total Split (%)	30.4%	30.4%		30.4%	30.4%		43.9%	43.9%		43.9%	43.9%	
Yellow Time (s)	4.0	4.0		4.0	4.0		4.0	4.0		4.0	4.0	
All-Red Time (s)	2.2	2.2		2.2	2.2		2.5	2.5		2.5	2.5	
Lost Time Adjust (s)	3.0	3.0			3.0			3.0			3.0	
Total Lost Time (s)	9.2	9.2			9.2			9.5			9.5	
Lead/Lag												
Lead-Lag Optimize?												
Recall Mode	None	None		None	None		C-Max	C-Max		C-Max	C-Max	
v/c Ratio	0.87	0.91			0.81			0.73			0.70	
Control Delay	120.2	104.1			99.4			38.9			37.0	
Queue Delay	0.0	0.0			0.0			0.0			0.6	
Total Delay	120.2	104.1			99.4			38.9			37.6	
Queue Length 50th (ft)	80	168			98			318			332	
Queue Length 95th (ft)	138	238			158			#487			476	
Internal Link Dist (ft)		137			302			499			519	
Turn Bay Length (ft)												
Base Capacity (vph)	150	298			199			1070			1204	
Starvation Cap Reductn	0	0			0			0			114	
Spillback Cap Reductn	0	0			0			0			0	
Storage Cap Reductn	0	0			0			0			0	
Reduced v/c Ratio	0.56	0.58			0.52			0.73			0.77	

Intersection Summary

Cycle Length: 148

Actuated Cycle Length: 148

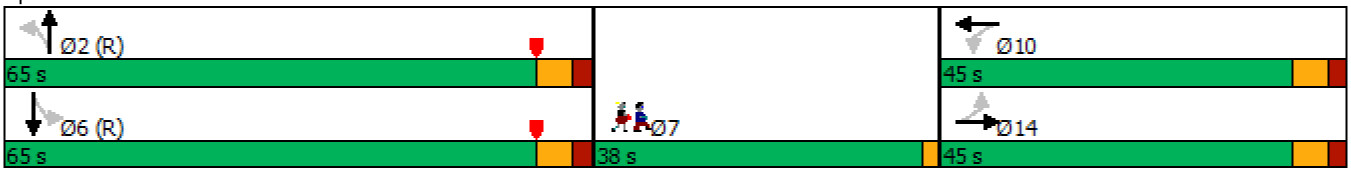
Offset: 8 (5%), Referenced to phase 2:NBTL and 6:SBTL, Start of Yellow

Natural Cycle: 110

Control Type: Actuated-Coordinated

95th percentile volume exceeds capacity, queue may be longer.
Queue shown is maximum after two cycles.

Splits and Phases: 1: Collins Avenue & Lincoln Road



Lane Group	Ø7
Lane Configurations	
Traffic Volume (vph)	
Future Volume (vph)	
Confl. Peds. (#/hr)	
Peak Hour Factor	
Heavy Vehicles (%)	
Parking (#/hr)	
Shared Lane Traffic (%)	
Lane Group Flow (vph)	
Turn Type	
Protected Phases	7
Permitted Phases	
Detector Phase	
Switch Phase	
Minimum Initial (s)	1.0
Minimum Split (s)	38.0
Total Split (s)	38.0
Total Split (%)	26%
Yellow Time (s)	2.0
All-Red Time (s)	0.0
Lost Time Adjust (s)	
Total Lost Time (s)	
Lead/Lag	
Lead-Lag Optimize?	
Recall Mode	Ped
v/c Ratio	
Control Delay	
Queue Delay	
Total Delay	
Queue Length 50th (ft)	
Queue Length 95th (ft)	
Internal Link Dist (ft)	
Turn Bay Length (ft)	
Base Capacity (vph)	
Starvation Cap Reductn	
Spillback Cap Reductn	
Storage Cap Reductn	
Reduced v/c Ratio	

Intersection Summary

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (vph)	80	28	71	28	31	114	47	655	43	33	718	98
Future Volume (vph)	80	28	71	28	31	114	47	655	43	33	718	98
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Total Lost time (s)	9.2	9.2			9.2			9.5			9.5	
Lane Util. Factor	1.00	1.00			1.00			0.95			0.95	
Frpb, ped/bikes	1.00	0.73			0.75			0.97			0.97	
Flpb, ped/bikes	0.74	1.00			0.95			1.00			0.99	
Frt	1.00	0.89			0.91			0.99			0.98	
Flt Protected	0.95	1.00			0.99			1.00			1.00	
Satd. Flow (prot)	916	1008			977			3045			3014	
Flt Permitted	0.55	1.00			0.93			0.72			0.85	
Satd. Flow (perm)	526	1008			914			2199			2579	
Peak-hour factor, PHF	0.98	0.98	0.98	0.98	0.98	0.98	0.98	0.98	0.98	0.98	0.98	0.98
Adj. Flow (vph)	82	29	72	29	32	116	48	668	44	34	733	100
RTOR Reduction (vph)	0	0	0	0	0	0	0	0	0	0	0	0
Lane Group Flow (vph)	82	101	0	0	177	0	0	760	0	0	867	0
Confl. Peds. (#/hr)	472		426	426		472	338		383	383		338
Heavy Vehicles (%)	32%	0%	0%	4%	0%	0%	7%	2%	0%	0%	1%	11%
Parking (#/hr)		0	0		0	0						
Turn Type	Perm	NA		Perm	NA		Perm	NA		Perm	NA	
Protected Phases		14			10			2			6	
Permitted Phases	14			10			2			6		
Actuated Green, G (s)	32.2	32.2			32.2			65.1			65.1	
Effective Green, g (s)	29.2	29.2			29.2			62.1			62.1	
Actuated g/C Ratio	0.20	0.20			0.20			0.42			0.42	
Clearance Time (s)	6.2	6.2			6.2			6.5			6.5	
Vehicle Extension (s)	2.5	2.5			2.5			1.0			1.0	
Lane Grp Cap (vph)	103	198			180			922			1082	
v/s Ratio Prot		0.10										
v/s Ratio Perm	0.16				c0.19			c0.35			0.34	
v/c Ratio	0.80	0.51			0.98			0.82			0.80	
Uniform Delay, d1	56.6	53.0			59.2			38.1			37.6	
Progression Factor	1.00	1.00			1.00			1.00			1.00	
Incremental Delay, d2	32.5	1.7			61.8			8.3			6.3	
Delay (s)	89.0	54.7			121.0			46.4			43.8	
Level of Service	F	D			F			D			D	
Approach Delay (s)		70.1			121.0			46.4			43.8	
Approach LOS		E			F			D			D	

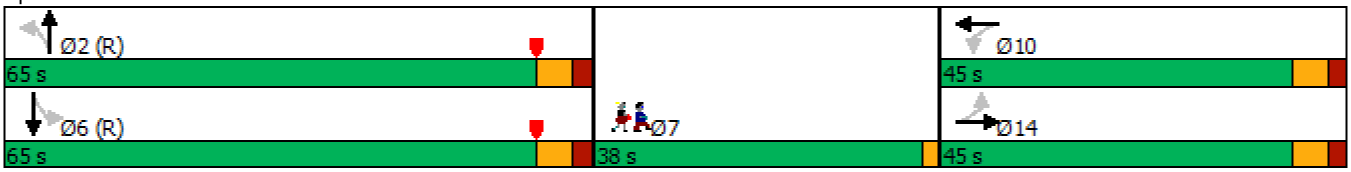
Intersection Summary			
HCM 2000 Control Delay	54.1	HCM 2000 Level of Service	D
HCM 2000 Volume to Capacity ratio	0.63		
Actuated Cycle Length (s)	148.0	Sum of lost time (s)	20.7
Intersection Capacity Utilization	99.2%	ICU Level of Service	F
Analysis Period (min)	15		
c Critical Lane Group			

Lane Group	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (vph)	80	28	71	28	31	114	47	655	43	33	718	98
Future Volume (vph)	80	28	71	28	31	114	47	655	43	33	718	98
Confl. Peds. (#/hr)	472		426	426		472	338		383	383		338
Peak Hour Factor	0.98	0.98	0.98	0.98	0.98	0.98	0.98	0.98	0.98	0.98	0.98	0.98
Heavy Vehicles (%)	32%	0%	0%	4%	0%	0%	7%	2%	0%	0%	1%	11%
Parking (#/hr)		0	0		0	0						
Shared Lane Traffic (%)												
Lane Group Flow (vph)	82	101	0	0	177	0	0	760	0	0	867	0
Turn Type	Perm	NA		Perm	NA		Perm	NA		Perm	NA	
Protected Phases		14			10			2			6	
Permitted Phases	14			10			2			6		
Detector Phase	14	14		10	10		2	2		6	6	
Switch Phase												
Minimum Initial (s)	7.0	7.0		7.0	7.0		7.0	7.0		7.0	7.0	
Minimum Split (s)	13.2	13.2		13.2	13.2		22.5	22.5		22.5	22.5	
Total Split (s)	45.0	45.0		45.0	45.0		65.0	65.0		65.0	65.0	
Total Split (%)	30.4%	30.4%		30.4%	30.4%		43.9%	43.9%		43.9%	43.9%	
Yellow Time (s)	4.0	4.0		4.0	4.0		4.0	4.0		4.0	4.0	
All-Red Time (s)	2.2	2.2		2.2	2.2		2.5	2.5		2.5	2.5	
Lost Time Adjust (s)	3.0	3.0			3.0			3.0			3.0	
Total Lost Time (s)	9.2	9.2			9.2			9.5			9.5	
Lead/Lag												
Lead-Lag Optimize?												
Recall Mode	None	None		None	None		C-Max	C-Max		C-Max	C-Max	
v/c Ratio	0.79	0.50			0.96			0.83			0.80	
Control Delay	99.6	60.0			114.3			48.2			45.6	
Queue Delay	0.0	0.0			0.0			0.0			0.4	
Total Delay	99.6	60.0			114.3			48.2			46.1	
Queue Length 50th (ft)	75	87			168			346			390	
Queue Length 95th (ft)	#152	145			#287			#500			#540	
Internal Link Dist (ft)		137			302			499			519	
Turn Bay Length (ft)												
Base Capacity (vph)	128	249			226			920			1079	
Starvation Cap Reductn	0	0			0			0			34	
Spillback Cap Reductn	0	0			0			0			0	
Storage Cap Reductn	0	0			0			0			0	
Reduced v/c Ratio	0.64	0.41			0.78			0.83			0.83	

Intersection Summary

Cycle Length: 148
Actuated Cycle Length: 148
Offset: 8 (5%), Referenced to phase 2:NBTL and 6:SBTL, Start of Yellow
Natural Cycle: 130
Control Type: Actuated-Coordinated
95th percentile volume exceeds capacity, queue may be longer.
Queue shown is maximum after two cycles.

Splits and Phases: 1: Collins Avenue & Lincoln Road



Lane Group	Ø7
Lane Configurations	
Traffic Volume (vph)	
Future Volume (vph)	
Confl. Peds. (#/hr)	
Peak Hour Factor	
Heavy Vehicles (%)	
Parking (#/hr)	
Shared Lane Traffic (%)	
Lane Group Flow (vph)	
Turn Type	
Protected Phases	7
Permitted Phases	
Detector Phase	
Switch Phase	
Minimum Initial (s)	1.0
Minimum Split (s)	38.0
Total Split (s)	38.0
Total Split (%)	26%
Yellow Time (s)	2.0
All-Red Time (s)	0.0
Lost Time Adjust (s)	
Total Lost Time (s)	
Lead/Lag	
Lead-Lag Optimize?	
Recall Mode	Ped
v/c Ratio	
Control Delay	
Queue Delay	
Total Delay	
Queue Length 50th (ft)	
Queue Length 95th (ft)	
Internal Link Dist (ft)	
Turn Bay Length (ft)	
Base Capacity (vph)	
Starvation Cap Reductn	
Spillback Cap Reductn	
Storage Cap Reductn	
Reduced v/c Ratio	

Intersection Summary

Evening Events

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (vph)	82	92	86	16	22	38	43	708	34	28	660	106
Future Volume (vph)	82	92	86	16	22	38	43	708	34	28	660	106
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Total Lost time (s)	9.2	9.2			9.2			9.5			9.5	
Lane Util. Factor	1.00	1.00			1.00			0.95			0.95	
Frpb, ped/bikes	1.00	0.82			0.78			0.98			0.96	
Flpb, ped/bikes	0.62	1.00			0.95			1.00			0.99	
Frt	1.00	0.93			0.93			0.99			0.98	
Flt Protected	0.95	1.00			0.99			1.00			1.00	
Satd. Flow (prot)	845	1169			1031			3098			2971	
Flt Permitted	0.72	1.00			0.85			0.77			0.88	
Satd. Flow (perm)	643	1169			888			2405			2613	
Peak-hour factor, PHF	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93
Adj. Flow (vph)	88	99	92	17	24	41	46	761	37	30	710	114
RTOR Reduction (vph)	0	0	0	0	0	0	0	0	0	0	0	0
Lane Group Flow (vph)	88	191	0	0	82	0	0	844	0	0	854	0
Confl. Peds. (#/hr)	575		374	374		575	521		735	735		521
Heavy Vehicles (%)	19%	0%	0%	0%	5%	0%	0%	1%	0%	0%	1%	9%
Parking (#/hr)		0	0		0	0						
Turn Type	Perm	NA		Perm	NA		Perm	NA		Perm	NA	
Protected Phases		14			10			2			6	
Permitted Phases	14			10			2			6		
Actuated Green, G (s)	31.5	31.5			31.5			85.8			85.8	
Effective Green, g (s)	28.5	28.5			28.5			82.8			82.8	
Actuated g/C Ratio	0.17	0.17			0.17			0.49			0.49	
Clearance Time (s)	6.2	6.2			6.2			6.5			6.5	
Vehicle Extension (s)	2.5	2.5			2.5			1.0			1.0	
Lane Grp Cap (vph)	109	198			150			1185			1287	
v/s Ratio Prot		c0.16										
v/s Ratio Perm	0.14				0.09			c0.35			0.33	
v/c Ratio	0.81	0.96			0.55			0.71			0.66	
Uniform Delay, d1	67.1	69.2			63.8			33.3			32.1	
Progression Factor	1.00	1.00			1.00			1.00			1.00	
Incremental Delay, d2	33.1	53.3			3.2			3.7			2.7	
Delay (s)	100.2	122.6			67.0			36.9			34.8	
Level of Service	F	F			E			D			C	
Approach Delay (s)		115.5			67.0			36.9			34.8	
Approach LOS		F			E			D			C	

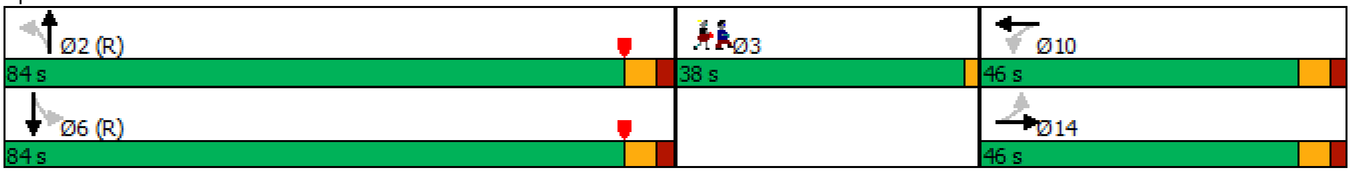
Intersection Summary			
HCM 2000 Control Delay	47.9	HCM 2000 Level of Service	D
HCM 2000 Volume to Capacity ratio	0.59		
Actuated Cycle Length (s)	168.0	Sum of lost time (s)	20.7
Intersection Capacity Utilization	100.2%	ICU Level of Service	G
Analysis Period (min)	15		
c Critical Lane Group			

Lane Group	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (vph)	82	92	86	16	22	38	43	708	34	28	660	106
Future Volume (vph)	82	92	86	16	22	38	43	708	34	28	660	106
Confl. Peds. (#/hr)	575		374	374		575	521		735	735		521
Peak Hour Factor	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93
Heavy Vehicles (%)	19%	0%	0%	0%	5%	0%	0%	1%	0%	0%	1%	9%
Parking (#/hr)		0	0		0	0						
Shared Lane Traffic (%)												
Lane Group Flow (vph)	88	191	0	0	82	0	0	844	0	0	854	0
Turn Type	Perm	NA		Perm	NA		Perm	NA		Perm	NA	
Protected Phases		14			10			2			6	
Permitted Phases	14			10			2			6		
Detector Phase	14	14		10	10		2	2		6	6	
Switch Phase												
Minimum Initial (s)	7.0	7.0		7.0	7.0		16.0	16.0		16.0	16.0	
Minimum Split (s)	13.2	13.2		13.2	13.2		22.5	22.5		22.5	22.5	
Total Split (s)	46.0	46.0		46.0	46.0		84.0	84.0		84.0	84.0	
Total Split (%)	27.4%	27.4%		27.4%	27.4%		50.0%	50.0%		50.0%	50.0%	
Yellow Time (s)	4.0	4.0		4.0	4.0		4.0	4.0		4.0	4.0	
All-Red Time (s)	2.2	2.2		2.2	2.2		2.5	2.5		2.5	2.5	
Lost Time Adjust (s)	3.0	3.0			3.0			3.0			3.0	
Total Lost Time (s)	9.2	9.2			9.2			9.5			9.5	
Lead/Lag												
Lead-Lag Optimize?												
Recall Mode	None	None		None	None		C-Max	C-Max		C-Max	C-Max	
v/c Ratio	0.77	0.95			0.53			0.71			0.66	
Control Delay	104.0	119.6			74.5			38.8			36.5	
Queue Delay	0.0	0.0			0.0			2.1			1.5	
Total Delay	104.0	119.6			74.5			40.9			38.0	
Queue Length 50th (ft)	93	210			83			380			371	
Queue Length 95th (ft)	158	#309			139			511			493	
Internal Link Dist (ft)		137			302			499			519	
Turn Bay Length (ft)												
Base Capacity (vph)	148	260			200			1184			1286	
Starvation Cap Reductn	0	0			0			203			247	
Spillback Cap Reductn	0	0			0			0			0	
Storage Cap Reductn	0	0			0			0			0	
Reduced v/c Ratio	0.59	0.73			0.41			0.86			0.82	

Intersection Summary

Cycle Length: 168
Actuated Cycle Length: 168
Offset: 81 (48%), Referenced to phase 2:NBTL and 6:SBTL, Start of Yellow
Natural Cycle: 120
Control Type: Actuated-Coordinated
95th percentile volume exceeds capacity, queue may be longer.
Queue shown is maximum after two cycles.

Splits and Phases: 1: Collins Avenue & Lincoln Road



Lane Group	Ø3
Lane Configurations	
Traffic Volume (vph)	
Future Volume (vph)	
Confl. Peds. (#/hr)	
Peak Hour Factor	
Heavy Vehicles (%)	
Parking (#/hr)	
Shared Lane Traffic (%)	
Lane Group Flow (vph)	
Turn Type	
Protected Phases	3
Permitted Phases	
Detector Phase	
Switch Phase	
Minimum Initial (s)	1.0
Minimum Split (s)	38.0
Total Split (s)	38.0
Total Split (%)	23%
Yellow Time (s)	2.0
All-Red Time (s)	0.0
Lost Time Adjust (s)	
Total Lost Time (s)	
Lead/Lag	
Lead-Lag Optimize?	
Recall Mode	Ped
v/c Ratio	
Control Delay	
Queue Delay	
Total Delay	
Queue Length 50th (ft)	
Queue Length 95th (ft)	
Internal Link Dist (ft)	
Turn Bay Length (ft)	
Base Capacity (vph)	
Starvation Cap Reductn	
Spillback Cap Reductn	
Storage Cap Reductn	
Reduced v/c Ratio	

Intersection Summary

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (vph)	80	20	86	16	22	110	43	683	34	28	685	108
Future Volume (vph)	80	20	86	16	22	110	43	683	34	28	685	108
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900	1900
Total Lost time (s)	9.2	9.2			9.2			9.5			9.5	
Lane Util. Factor	1.00	1.00			1.00			0.95			0.95	
Frpb, ped/bikes	1.00	0.70			0.68			0.97			0.96	
Flpb, ped/bikes	0.71	1.00			0.97			1.00			0.99	
Frt	1.00	0.88			0.90			0.99			0.98	
Flt Protected	0.95	1.00			0.99			1.00			1.00	
Satd. Flow (prot)	966	952			899			3094			2973	
Flt Permitted	0.54	1.00			0.96			0.75			0.88	
Satd. Flow (perm)	550	952			865			2335			2623	
Peak-hour factor, PHF	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93
Adj. Flow (vph)	86	22	92	17	24	118	46	734	37	30	737	116
RTOR Reduction (vph)	0	0	0	0	0	0	0	0	0	0	0	0
Lane Group Flow (vph)	86	114	0	0	159	0	0	817	0	0	883	0
Confl. Peds. (#/hr)	575		374	374		575	521		735	735		521
Heavy Vehicles (%)	19%	0%	0%	0%	5%	0%	0%	1%	0%	0%	1%	9%
Parking (#/hr)		0	0		0	0						
Turn Type	Perm	NA		Perm	NA		Perm	NA		Perm	NA	
Protected Phases		14			10			2			6	
Permitted Phases	14			10			2			6		
Actuated Green, G (s)	33.9	33.9			33.9			83.4			83.4	
Effective Green, g (s)	30.9	30.9			30.9			80.4			80.4	
Actuated g/C Ratio	0.18	0.18			0.18			0.48			0.48	
Clearance Time (s)	6.2	6.2			6.2			6.5			6.5	
Vehicle Extension (s)	2.5	2.5			2.5			1.0			1.0	
Lane Grp Cap (vph)	101	175			159			1117			1255	
v/s Ratio Prot		0.12										
v/s Ratio Perm	0.16				c0.18			c0.35			0.34	
v/c Ratio	0.85	0.65			1.00			0.73			0.70	
Uniform Delay, d1	66.3	63.6			68.5			35.1			34.4	
Progression Factor	1.00	1.00			1.00			1.00			1.00	
Incremental Delay, d2	45.6	7.5			71.4			4.2			3.3	
Delay (s)	111.9	71.1			139.9			39.4			37.8	
Level of Service	F	E			F			D			D	
Approach Delay (s)		88.6			139.9			39.4			37.8	
Approach LOS		F			F			D			D	

Intersection Summary			
HCM 2000 Control Delay	51.2	HCM 2000 Level of Service	D
HCM 2000 Volume to Capacity ratio	0.61		
Actuated Cycle Length (s)	168.0	Sum of lost time (s)	20.7
Intersection Capacity Utilization	102.4%	ICU Level of Service	G
Analysis Period (min)	15		
c Critical Lane Group			

Lane Group	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (vph)	80	20	86	16	22	110	43	683	34	28	685	108
Future Volume (vph)	80	20	86	16	22	110	43	683	34	28	685	108
Confl. Peds. (#/hr)	575		374	374		575	521		735	735		521
Peak Hour Factor	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93	0.93
Heavy Vehicles (%)	19%	0%	0%	0%	5%	0%	0%	1%	0%	0%	1%	9%
Parking (#/hr)		0	0		0	0						
Shared Lane Traffic (%)												
Lane Group Flow (vph)	86	114	0	0	159	0	0	817	0	0	883	0
Turn Type	Perm	NA		Perm	NA		Perm	NA		Perm	NA	
Protected Phases		14			10			2			6	
Permitted Phases	14			10			2			6		
Detector Phase	14	14		10	10		2	2		6	6	
Switch Phase												
Minimum Initial (s)	7.0	7.0		7.0	7.0		16.0	16.0		16.0	16.0	
Minimum Split (s)	13.2	13.2		13.2	13.2		22.5	22.5		22.5	22.5	
Total Split (s)	46.0	46.0		46.0	46.0		84.0	84.0		84.0	84.0	
Total Split (%)	27.4%	27.4%		27.4%	27.4%		50.0%	50.0%		50.0%	50.0%	
Yellow Time (s)	4.0	4.0		4.0	4.0		4.0	4.0		4.0	4.0	
All-Red Time (s)	2.2	2.2		2.2	2.2		2.5	2.5		2.5	2.5	
Lost Time Adjust (s)	3.0	3.0			3.0			3.0			3.0	
Total Lost Time (s)	9.2	9.2			9.2			9.5			9.5	
Lead/Lag												
Lead-Lag Optimize?												
Recall Mode	None	None		None	None		C-Max	C-Max		C-Max	C-Max	
v/c Ratio	0.93	0.64			0.97			0.73			0.70	
Control Delay	141.5	78.6			128.3			41.2			39.4	
Queue Delay	0.0	0.0			0.0			1.8			1.9	
Total Delay	141.5	78.6			128.3			43.0			41.3	
Queue Length 50th (ft)	93	115			173			387			410	
Queue Length 95th (ft)	#195	187			#301			496			515	
Internal Link Dist (ft)		137			302			499			519	
Turn Bay Length (ft)												
Base Capacity (vph)	110	213			196			1116			1254	
Starvation Cap Reductn	0	0			0			157			218	
Spillback Cap Reductn	0	0			0			0			0	
Storage Cap Reductn	0	0			0			0			0	
Reduced v/c Ratio	0.78	0.54			0.81			0.85			0.85	

Intersection Summary

Cycle Length: 168

Actuated Cycle Length: 168

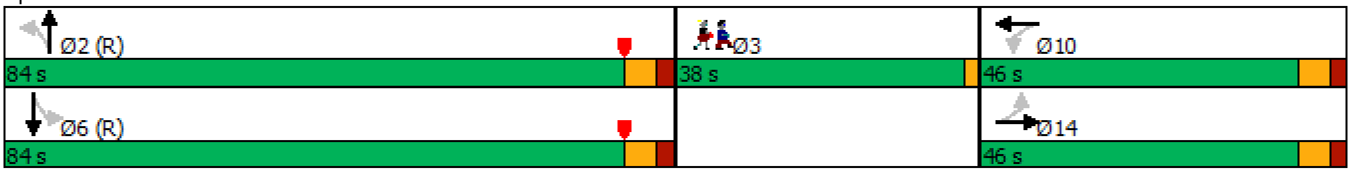
Offset: 81 (48%), Referenced to phase 2:NBTL and 6:SBTL, Start of Yellow

Natural Cycle: 130

Control Type: Actuated-Coordinated

\# 95th percentile volume exceeds capacity, queue may be longer.
Queue shown is maximum after two cycles.

Splits and Phases: 1: Collins Avenue & Lincoln Road



Lane Group	Ø3
Lane Configurations	
Traffic Volume (vph)	
Future Volume (vph)	
Confl. Peds. (#/hr)	
Peak Hour Factor	
Heavy Vehicles (%)	
Parking (#/hr)	
Shared Lane Traffic (%)	
Lane Group Flow (vph)	
Turn Type	
Protected Phases	3
Permitted Phases	
Detector Phase	
Switch Phase	
Minimum Initial (s)	1.0
Minimum Split (s)	38.0
Total Split (s)	38.0
Total Split (%)	23%
Yellow Time (s)	2.0
All-Red Time (s)	0.0
Lost Time Adjust (s)	
Total Lost Time (s)	
Lead/Lag	
Lead-Lag Optimize?	
Recall Mode	Ped
v/c Ratio	
Control Delay	
Queue Delay	
Total Delay	
Queue Length 50th (ft)	
Queue Length 95th (ft)	
Internal Link Dist (ft)	
Turn Bay Length (ft)	
Base Capacity (vph)	
Starvation Cap Reductn	
Spillback Cap Reductn	
Storage Cap Reductn	
Reduced v/c Ratio	

Intersection Summary