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December 30, 2020

VIA EMAIL
ken@hixsnedeker.com

Mr. Kenneth Fioretti
Hix Snedeker Companies
805 Trione Street
Daphne, AL 36526

Re: Dollar General – U.S. Route 9W
SEQRA # 20-161
Town of Marlborough, Ulster County, New York
MC Project No. 20006148A

Dear Mr. Fioretti:

As requested, we have completed our traffic evaluation for the proposed 9,100 square foot Dollar General store, which is proposed on a parcel of land located in the Town of Marlborough (see Figure No. 1), New York approximately 400± feet north of the Mahoney Road and U.S. Route 9W intersection. This evaluation is based on the requirements of the New York State Department of Transportation (NYSDOT) and the following sections describe of the tasks undertaken.

1. Description of Existing Roadway Conditions
   U.S. Route 9W along the frontage of the site consists of one travel lane in each direction plus approximately 8-foot wide paved shoulders on either side. The roadway varies in terms of sections that are curbed at developed driveways and no curbs in other areas. Note that opposite the site there is a small commercial development and the access will have to be coordinated with the proposed Dollar General driveways. This section of U.S. Route 9W has an unposted speed limit of 55 MPH and along the frontage of the site there is an existing passing zone.

   As indicated on the site plan, the sight distances in the vicinity of the site and at the access, satisfy NYSDOT design criteria. Also note that to the north of the site is a property that is currently occupied by an industrial use. The property to the south has an office and related commercial use. Utility poles are located along the frontage of the site. The project is proposed to be served by a single driveway consisting of one entering and one exiting lane.

2. 2020 Existing Traffic Volumes (Figure No.2)
   Turning movement traffic counts were conducted in the vicinity of the site and at the intersection of U.S. Route 9W and Mahoney Road in order to identify existing traffic conditions. The traffic volumes were collected for the PM Peak Highway Hour. The traffic volumes collected on December 8, 2020. These counts were compared with historical data, including data collected in association with other projects along the corridor as well as data

published by NYSDOT. Note that the roadway has a current ADT of approximately 15,000 vehicles per day with peak hour two-directional volumes in the order of 1,500 vehicles per hour during peak periods of the day. The existing traffic volumes included adjustments based on historical data to account for any variations in traffic due to the Covid-19 Pandemic at the time of the recent traffic counts. The traffic volumes reflecting the peak hour conditions are shown on Figure No. 2.

3. Year 2022_No-Build Traffic Volumes (Figure No. 3)
   The 2020 Existing Traffic Volumes were projected to a future design year to represent conditions without the proposed variety store, the Existing Traffic Volumes were increased by a growth factor of 1% per year. Also, traffic from known significant developments, including those in the adjacent Town of Lloyd, were also accounted for in the traffic projections. The resulting Year 2022 No-Build Traffic Volumes are shown on Figure No. 3.

4. Site Generated Traffic Volumes (Table No. 1)
   Estimates of the amount of traffic to be generated by the proposed 9,100 square foot building during each of the peak hours were developed based on information published by the Institute of Transportation Engineers (ITE) as contained in the report entitled "Trip Generation", 10th Edition, 2017, based on Land Use Category – 814 Variety Store. This category includes this type of retail use and based on the data, would generate approximately 32 entering and 30 exiting vehicles during the peak one-hour period. Based on the traffic flows along U.S. Route 9W on a daily basis, the expected number of "new" trips to the roadway system would be even less. Based on ITE data for a use such as this, it can be expected that 25% of the vehicle trips would not be new to the system. Table No. 1 summarizes the trip generation rates and corresponding site generated traffic volumes for the Weekday Peak AM and Weekday Peak PM Hours. Note that the PM Peak Hour is the critical time period in terms of operations and that period is analyzed herein.

5. Arrival/Departure Distribution
   An arrival and departure distribution was established for the site driveway based upon a review of the traffic flows along U.S. Route 9W.

6. 2022 Build Conditions Traffic Volumes (Figures No. 4 and 5)
   The site generated traffic volumes shown in Table No. 1 were assigned to the driveway based on the expected arrival and departure distributions referenced above. The resulting site generated traffic volumes are shown on Figures No. 4. The site generated traffic volumes were then added to the Year 2022 No-Build Traffic Volumes to obtain the Year 2022 Build Traffic Volumes for the driveway intersection. The resulting Year 2022 Build Traffic Volumes are shown on Figure No. 5 for the critical PM Peak Highway Hour.

7. Description of Analysis Procedures
   It was necessary to perform capacity analyses in order to determine existing and future traffic operating conditions at the study area intersections. The following is a brief description of the analysis method utilized in this report:

- Signalized Intersection Capacity Analysis
  The capacity analysis for a signalized intersection was performed in accordance with the procedures described in the *Highway Capacity Manual, 6th Edition,* published by the Transportation Research Board. The terminology used in identifying traffic flow conditions is Levels of Service. A Level of Service "A" represents the best condition and a Level of Service "F" represents the worst condition. A Level of Service "C" is generally used as a design standard while a Level of Service "D" is acceptable during peak periods. A Level of Service "E" represents an operation near capacity. In order to identify an intersection's Level of Service, the average amount of vehicle delay is computed for each approach to the intersection as well as for the overall intersection.

- Unsignalized Intersection Capacity Analysis
  The unsignalized intersection capacity analysis method utilized in this report was also performed in accordance with the procedures described in the *Highway Capacity Manual, 6th Edition.* The procedure is based on total elapsed time from when a vehicle stops at the end of the queue until the vehicle departs from the stop line. The average total delay for any particular critical movement is a function of the service rate or capacity of the approach and the degree of saturation. In order to identify the Level of Service, the average amount of vehicle delay is computed for each critical movement to the intersection.

Additional information concerning signalized and unsignalized Levels of Service can be found in Appendix "C" of this report.

8. Results of Capacity Analysis (Table No. 2)
   Capacity analyses which take into consideration appropriate truck percentages, pedestrian activity, roadway grades and other factors were performed at the study area intersections utilizing the procedures described above to determine the Levels of Service and average vehicle delays.

   Table No. 2 summarizes the results of the capacity analysis for the 2022 Build Conditions. Appendix "D" contains copies of the capacity analysis which also indicate the existing geometrics (including lane widths) and other characteristics for the driveway intersection.

9. Accident Evaluation (Table A)
   All available accident data was obtained from NYSDOT for this section of U.S. Route 9W from Mahoney Road north for 1,000 feet for the latest available three-year period. A summary of those accidents is contained in Table A. A review of the limited number accidents indicates a few right angle and rear-end accidents over this period.

10. Left Turn Warrant Analysis

    Based on the American Association of State Highway and Transportation Officials (AASHTO) guidelines described in the 2018 report entitled, *A Policy Geometric Design of Highways and Streets*, an evaluation of the potential need for a left turn lane at the site driveway was completed utilizing the Build Traffic Volumes. Note that this section of U.S. Route 9W does not currently have turning lanes at the various existing commercial and industrial driveways along this section of roadway. However, based on the heavy through traffic volumes along the corridor and the anticipated left turn volume at the driveway, this would satisfy NYSDOT guidelines for the provision of a left turn lane (see attached figure). As noted in the AASHTO manual, consideration of the other parameters and in reviewing the volume-based guidelines presented, indicates a situation where "left turn lanes may be desirable but not necessarily situations where it is definitely needed and/or practical based on current constraints and cost consideration." Based on a review of the accident history as described above, there does not appear to be a major crash history at other existing driveways, which should also be taken into consideration. This will have to be discussed with NYSDOT in more detail as part of the Highway Work Permit (HWP) process.

11. Consideration of Route 9W Corridor Management Plan

    Appendix F of the Route 9W Corridor Management Plan addresses various access management recommendations for various sections of the roadway corridor, including closure of some existing driveways for parcels that have multiple curb cuts, adding landscape along frontage, accommodating pedestrians, and provide access from a rear road or adjacent parcel. The parcel to the north has an existing driveway and the use and layout does not accommodate a connection. The site has no rear frontage to a rear road. Also, the property to the south has an existing curb cut on Route 9W but it is in close proximity to the Mahoney Road intersection. The addition of additional traffic though the parking area of that site and adding more turning movements in proximity of the Mahoney Road intersection is not desirable from a traffic flow standpoint. Thus, the individual driveway to the Dollar General at a location with good sight distances and further away from the Mahoney Road intersection will function more efficiently and does not conflict with other existing driveways.

Very truly yours,
MASER CONSULTING CONNECTICUT, P.C.

Philip J. Grealy, Ph.D., P.E.
Principal/Department Manager

PJG/ces
Enclosures
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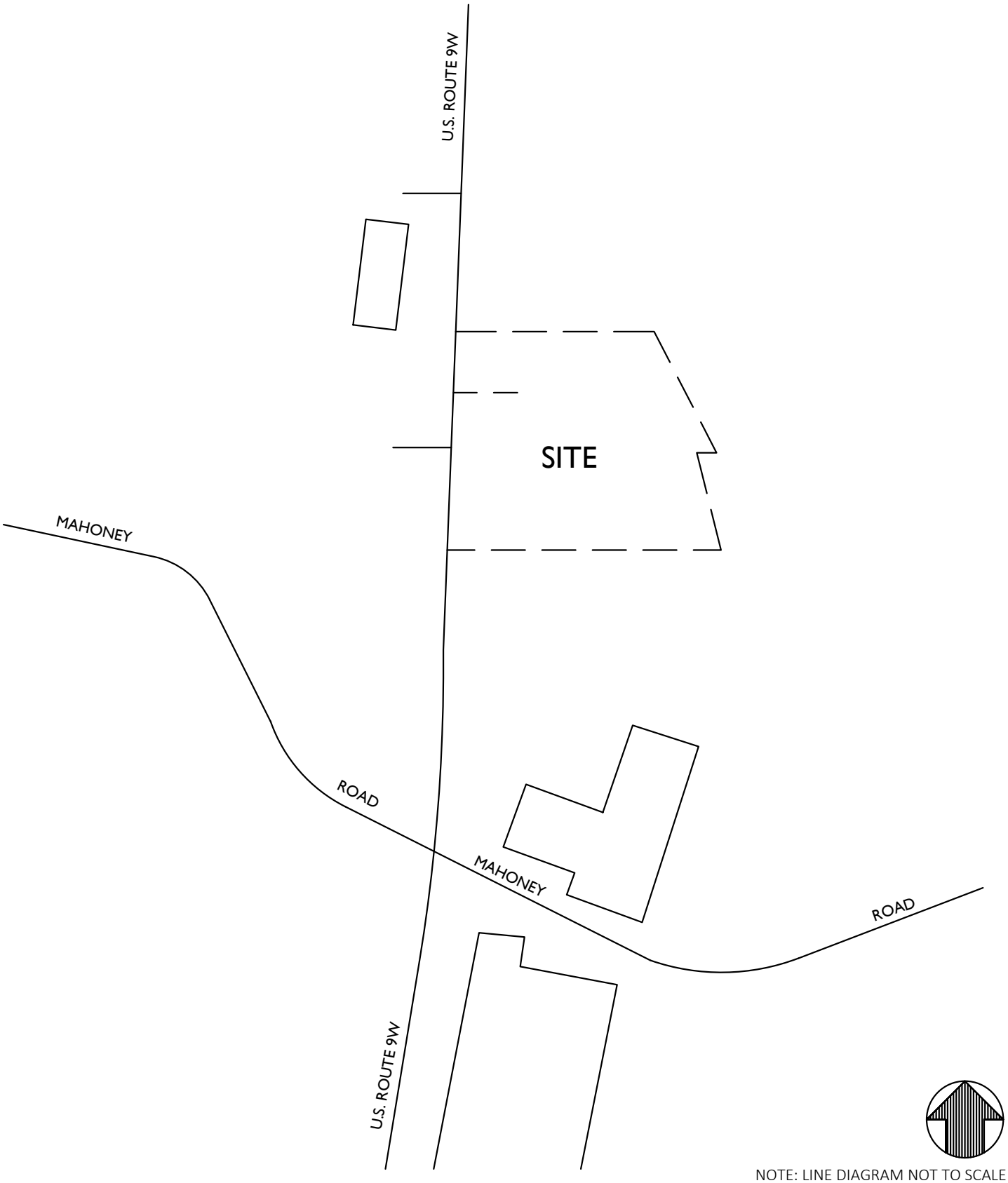
PROPOSED DOLLAR GENERAL

APPENDIX A

FIGURES



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AS SHOWN	12/8/20	R.H.	P.J.G.

PROJECT NUMBER:	DRAWING NAME:
20006148A	201208RH_FIGURE

SHEET TITLE: SITE LOCATION MAP

SHEET NUMBER: 1

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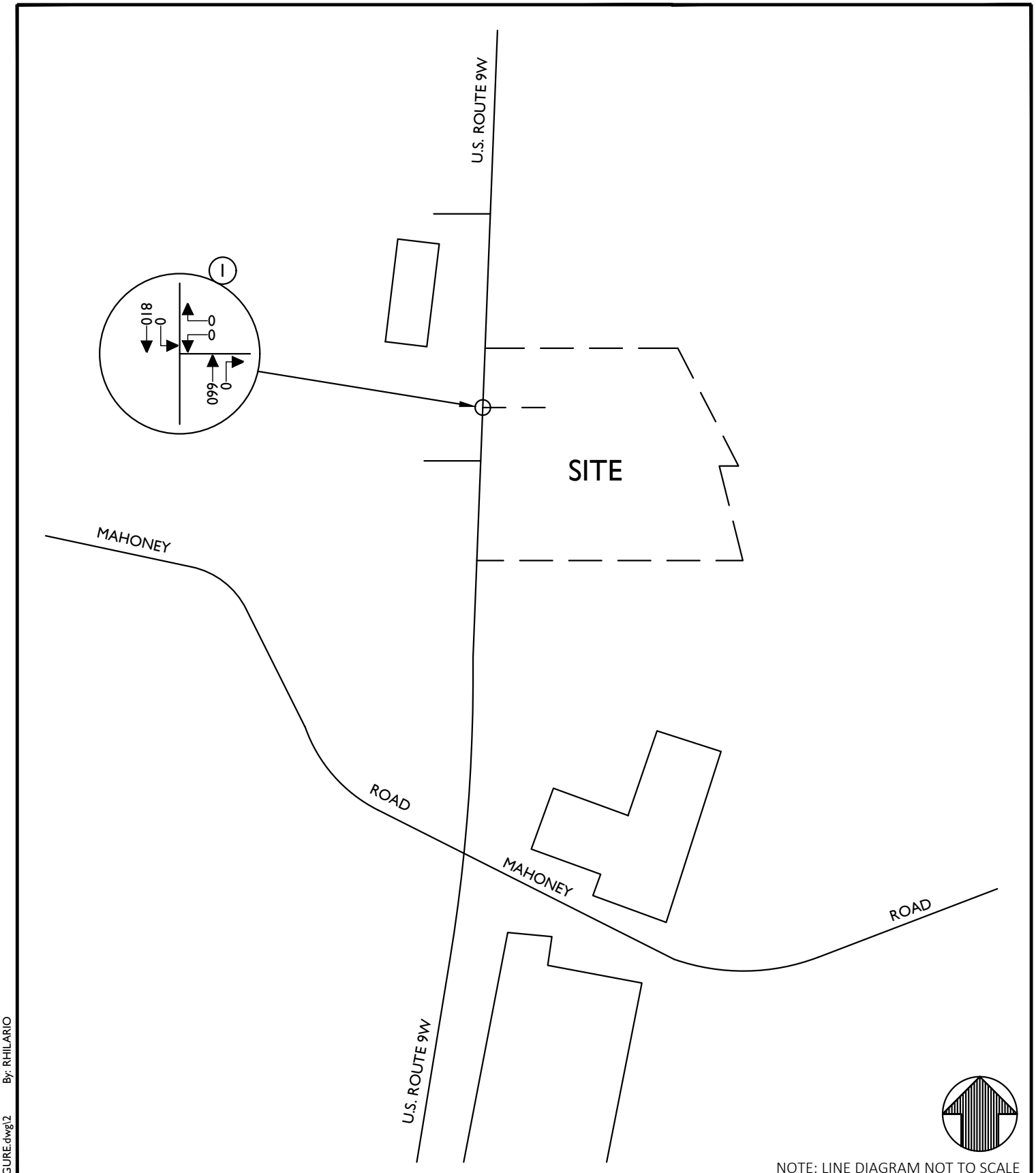

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PROJECT NUMBER:	DRAWING NAME:
20006148A	201208RH_FIGURE

SHEET TITLE:
2020 EXISTING TRAFFIC VOLUMES
WEEKDAY PEAK PM HOUR

SHEET NUMBER:
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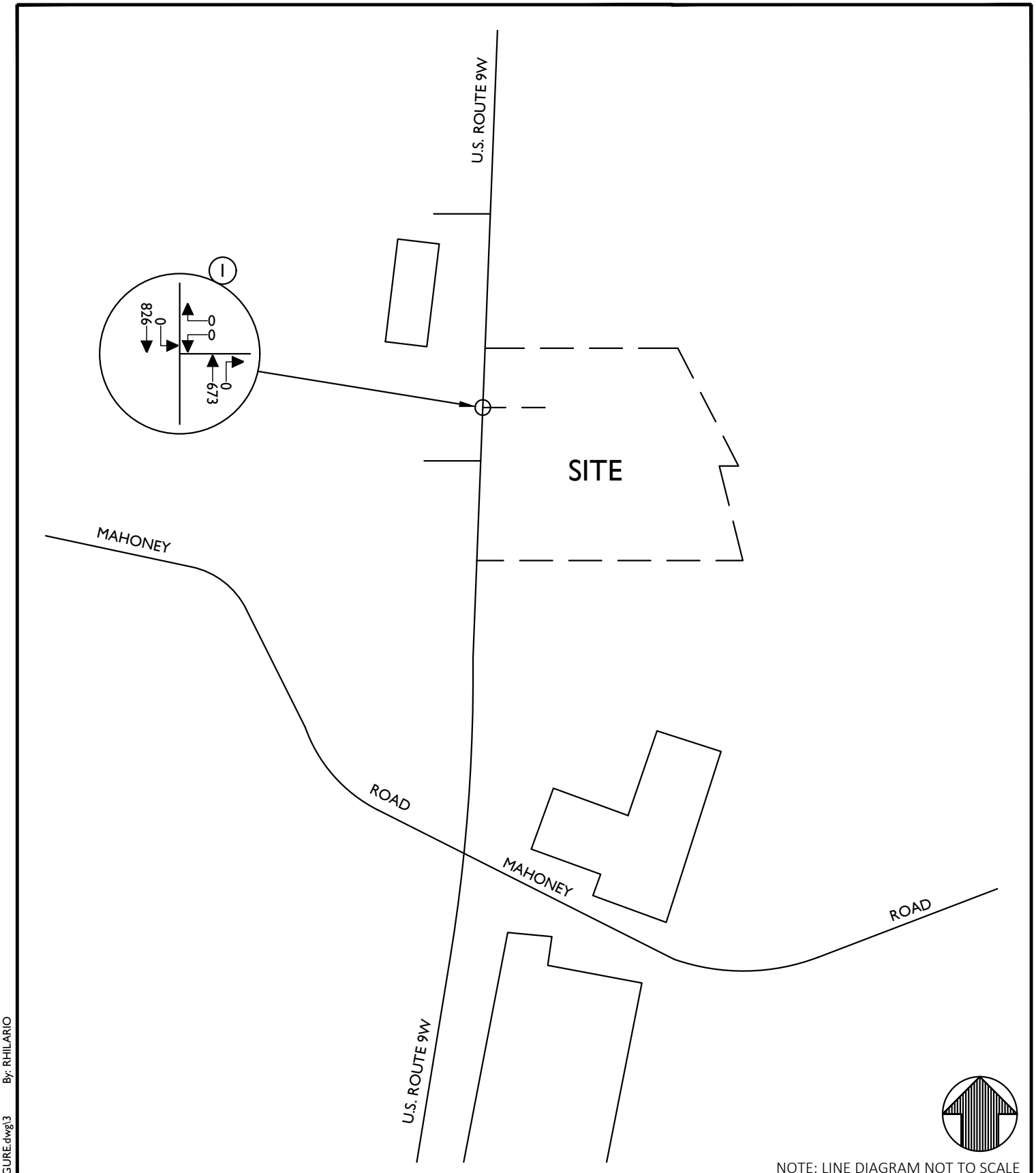

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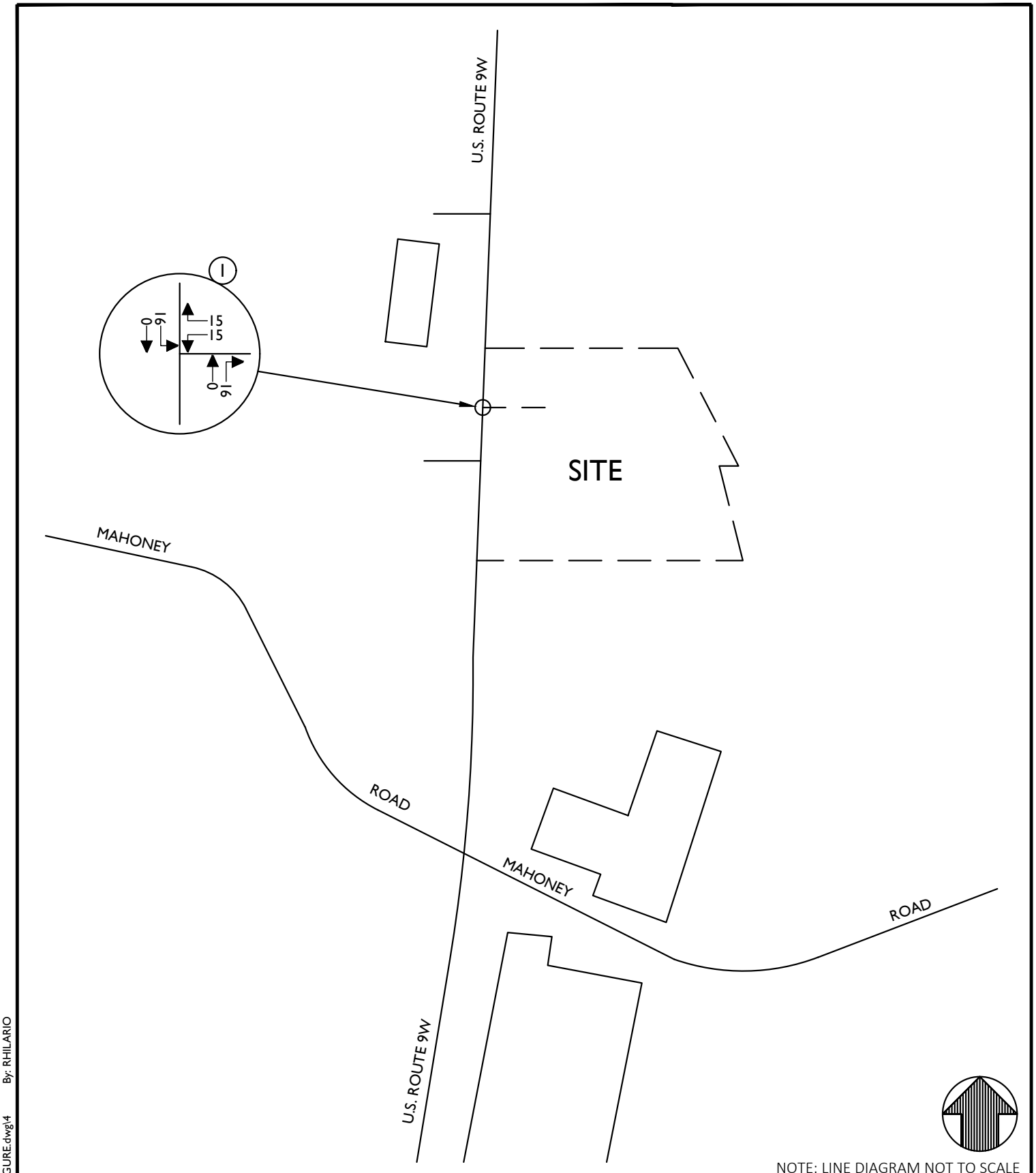
PROJECT NUMBER:	DRAWING NAME:
20006148A	201208RH_FIGURE

SHEET TITLE:
2022 NO-BUILD TRAFFIC VOLUMES
WEEKDAY PEAK PM HOUR

SHEET NUMBER:
3



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PROJECT NUMBER:	DRAWING NAME:
20006148A	201208RH_FIGURE

SHEET TITLE:
SITE GENERATED
TRAFFIC VOLUMES
WEEKDAY PEAK PM HOUR

SHEET NUMBER:
4

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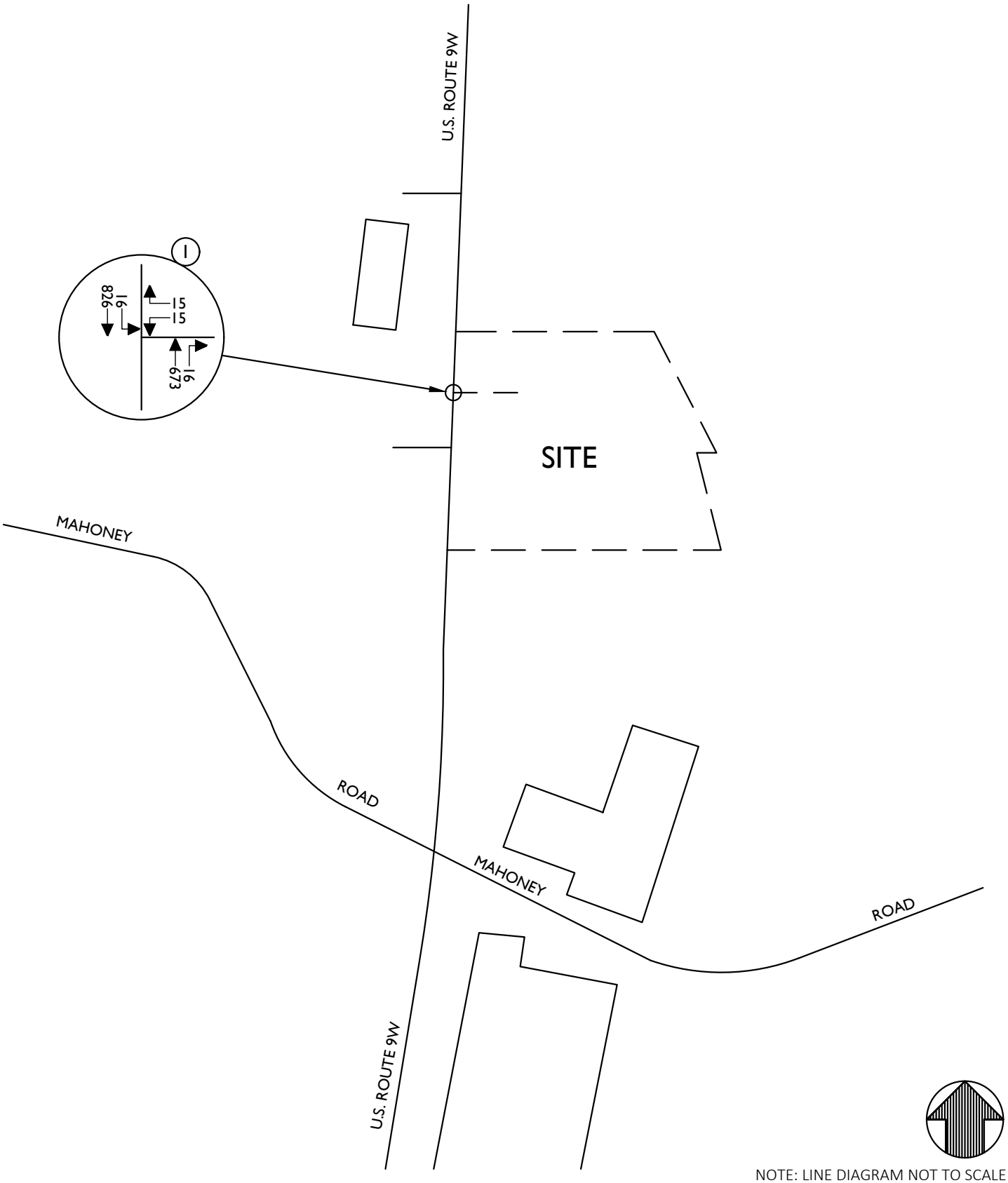

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PROJECT NUMBER:	DRAWING NAME:
20006148A	201208RH_FIGURE

SHEET TITLE:
2022 BUILD TRAFFIC VOLUMES
WEEKDAY PEAK PM HOUR

SHEET NUMBER:
5

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PROPOSED DOLLAR GENERAL

APPENDIX B

TABLES

TABLE NO. 1

HOURLY TRIP GENERATION RATES (HTGR) AND ANTICIPATED SITE GENERATED TRAFFIC VOLUMES

DOLLAR GENERAL MILTON, NEW YORK	ENTRY		EXIT	
	HTGR[1]	VOLUME	HTGR[1]	VOLUME
VARIETY STORE (9,100 SQ. FT.)				
PEAK AM HOUR	1.76	16	1.43	13
PEAK PM HOUR	3.52	32	3.30	30

NOTES:

1) THE HOURLY TRIP GENERATION RATES (HTGR) ARE BASED ON DATA PUBLISHED BY THE INSTITUTE OF TRANSPORTATION ENGINEERS (ITE) AS CONTAINED IN THE TRIP GENERATION HANDBOOK, 10TH EDITION, 2017. ITE LAND USE CODE - 814 - VARIETY STORE.

TABLE NO. 2

LEVEL OF SERVICE SUMMARY TABLE

				2020 BUILD		
				V/C	LOS	DELAY
1	U.S. ROUTE 9W & SITE ACCESS DRIVEWAY	**UNSIGNALIZED**				
		WB	LR	0.22	D	34.5
		SB	LT	0.02	A	9.3

NOTES:

1) THE ABOVE REPRESENTS THE LEVEL OF SERVICE AND VEHICLE DELAY IN SECONDS, C [16.2], FOR EACH KEY APPROACH OF THE UNSIGNALIZED INTERSECTIONS AS WELL AS FOR EACH APPROACH AND THE OVERALL INTERSECTION FOR THE SIGNALIZED INTERSECTIONS. SEE APPENDIX "C" FOR A DESCRIPTION OF THE LEVELS OF SERVICE.

TABLE A

ACCIDENT SUMMARY - TOWN ACCIDENT DATA
VARIOUS INTERSECTIONS IN THE TOWN OF MARLBOROUGH

Node/Link	Location	Mile Marker	Date	Time	Traffic Control	Accident Class	# of Vehicles Injuries	Light Condition	Road Condition	Weather	Manner of Collision	Apparent Contributing Factors
MAHONEY RD	MAHONEY RD		08/03/20	09:03am	STOP SIGN	PDO	2-0	DAYLIGHT	DRY	CLEAR	REAR END	DRIVER INATTENTION
			04/25/18	03:22pm	NONE	N/R	2-0	DAYLIGHT	WET	RAIN	RIGHT ANGLE	BACKING UNSAFELY
ROUTE 9W	AT INT. W/ MAHONEY RD	9W86031059	08/25/20	08:41pm	NONE	PDO & I	2-2	DARK-ROAD UNLIGHTED	DRY	CLEAR	RIGHT ANGLE	FAILURE TO YIELD RIGHT OF WAY
ROUTE 9W	AT INT. W/ MAHONEY RD	9W86031059	10/31/19	04:19pm	STOP SIGN	PDO & I	2-1	DAYLIGHT	WET	RAIN	RN (AGAINST OTI	FAILURE TO YIELD RIGHT OF WAY
ROUTE 9W	ROUTE 9W	9W86031059	05/04/19	01:38pm	NONE	PDO	2-0	DAYLIGHT	DRY	CLOUDY	OVERTAKING	NOT APPLICABLE
ROUTE 9W	AT INT. W/ MAHONEY RD	9W86031059	01/16/19	10:15am	STOP SIGN	PDO	2-0	DAYLIGHT	DRY	CLOUDY	OTHER	NOT APPLICABLE
ROUTE 9W	AT INT. W/ MAHONEY RD	9W86031059	11/24/18	12:22pm	STOP SIGN	PDO & I	2-3	DAYLIGHT	DRY	CLOUDY	RIGHT ANGLE	FAILURE TO YIELD RIGHT OF WAY
ROUTE 9W	AT INT. W/ MAHONEY RD	9W86031059	07/23/18	05:25pm	NONE	PDO & I	2-2	DAYLIGHT	WET	RAIN	REAR END	PAVEMENT SLIPPERY
ROUTE 9W	AT INT. W/ MAHONEY RD	9W86031059	03/05/17	05:40pm	STOP SIGN	N/R	1-0	DAYLIGHT	DRY	CLEAR	OTHER	REACTION TO OTHER UNINVOLVED VEHICL
ROUTE 9W	ROUTE 9W	9W86031059	05/23/16	08:46pm	NONE	PDO & I	2-2	DARK-ROAD LIGHTED	DRY	CLOUDY	RN (AGAINST OTI	FAILURE TO YIELD RIGHT OF WAY
ROUTE 9W	ROUTE 9W	9W86031060	12/13/17	07:42am	NONE	PDO	2-0	DARK-ROAD LIGHTED	DRY	CLEAR	SIDESWIPE	UNSAFE LANE CHANGE
ROUTE 9W	ROUTE 9W	9W86031060	03/21/17	03:04pm	NONE	PDO	2-0	DAYLIGHT	DRY	CLEAR	REAR END	FOLLOWING TOO CLOSELY

NYSDOT QRA ACCIDENT SEVERITY SUMMARY

Print Date 12/10/2020 **Print Time** 2:53:17PM

Query Number/Name	Query Type	Query Sub Type	Accident Date Range
60478 17783	AttributeQuery	None	1/1/2019 12:00:00AM To 12/9/2020 12:00:00AM

Case Year	Injury	Fatality	Property Damage	Non-Reportables	Totals
2019	1	0	2	0	3

Case Year	Injury	Fatality	Property Damage	Non-Reportables	Totals
2020	1	0	1	0	2

Grand Total:	2	0	3	0

NYSDOT QRA ACCIDENT SEVERITY SUMMARY

Print Date 12/7/2020 **Print Time** 2:28:32PM

Query Number/Name	Query Type	Query Sub Type	Accident Date Range
60397 17762	AttributeQuery	None	1/1/2015 12:00:00AM To 12/31/2018 12:00:00AM

Case Year	Injury	Fatality	Property Damage	Non-Reportables	Totals
2016	1	0	0	0	1

Case Year	Injury	Fatality	Property Damage	Non-Reportables	Totals
2017	0	0	2	1	3

Case Year	Injury	Fatality	Property Damage	Non-Reportables	Totals
2018	2	0	0	1	3

Grand Total:	3	0	2	2	

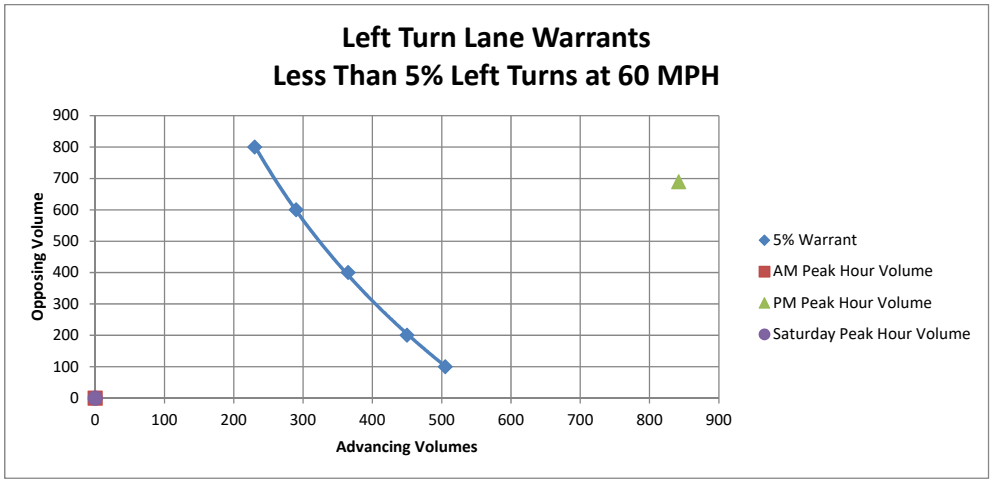
(2022 BUILD) LEFT TURN LANE WARRANTS FOR 60 MPH OPERATING SPEED

.

	Eastbound			Westbound Site Access			Northbound U.S. Route 9W			Southbound U.S. Route 9W		
	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right
AM Peak Hour	-	-	-	-	-	-	-	-	-	-	-	-
PM Peak Hour	-	-	-	15	-	15	-	673	16	16	826	-
Saturday Peak Hour	-	-	-	-	-	-	-	-	-	-	-	-

Southbound Left Turn Calcs			
Opposing	Advancing	Left Turns	% Left Turns
0	0	-	#VALUE!
689	842	16	2%
0	0	-	#VALUE!

0-5% Left Turns	
Opposing	Advancing
#VALUE!	#VALUE!
689	842
#VALUE!	#VALUE!



AASHTO Left Turn Warrants - 60 MPH				
	Advancing			
Opposing	5%	10%	20%	30%
800	230	170	125	115
600	290	210	160	140
400	365	270	200	175
200	450	330	250	215
100	505	370	275	240

PROPOSED DOLLAR GENERAL

APPENDIX C

LEVEL OF SERVICE STANDARDS

LEVEL OF SERVICE STANDARDS

LEVEL OF SERVICE FOR SIGNALIZED INTERSECTIONS

Level of Service (LOS) can be characterized for the entire intersection, each intersection approach, and each lane group. Control delay alone is used to characterize LOS for the entire intersection or an approach. Control delay and volume-to-capacity (v/c) ratio are used to characterize LOS for a lane group. Delay quantifies the increase in travel time due to traffic signal control. It is also a measure of driver discomfort and fuel consumption. The volume-to-capacity ratio quantifies the degree to which a phase's capacity is utilized by a lane group.

LOS A describes operations with a control delay of 10 s/veh or less and a volume-to-capacity ratio no greater than 1.0. This level is typically assigned when the volume-to-capacity ratio is low and either progression is exceptionally favorable or the cycle length is very short. If it is due to favorable progression, most vehicles arrive during the green indication and travel through the intersection without stopping.

LOS B describes operations with control delay between 10 and 20 s/veh and a volume-to-capacity ratio no greater than 1.0. This level is typically assigned when the volume-to-capacity ratio is low and either progression is highly favorable or the cycle length is short. More vehicles stop than with LOS A.

LOS C describes operations with control delay between 20 and 35 s/veh and a volume-to-capacity ratio no greater than 1.0. This level is typically assigned when progression is favorable or the cycle length is moderate.

LOS D describes operations with control delay between 35 and 55 s/veh and a volume-to-capacity ratio no greater than 1.0. This level is typically assigned when the volume-to-capacity ratio is high and either progression is ineffective or the cycle length is long.

LOS E describes operations with control delay between 55 and 80 s/veh and a volume-to-capacity ratio no greater than 1.0. This level is typically assigned when the volume-to-capacity ratio is high, progression is unfavorable, and the cycle length is long.

LOS F describes operations with control delay exceeding 80 s/veh or a volume-to-capacity ratio greater than 1.0. This level is typically assigned when the volume-to-capacity ratio is very high, progression is very poor, and the cycle length is long.

A lane group can incur a delay less than 80 s/veh when the volume-to-capacity ratio exceeds 1.0. This condition typically occurs when the cycle length is short, the signal progression is favorable, or both. As a result, both the delay and volume-to-capacity ratio are considered when lane group LOS is established. A ratio of 1.0 or more indicates that cycle capacity is fully utilized and represents failure from a capacity perspective (just as delay in excess of 80 s/veh represents failure from a delay perspective).

The Level of Service Criteria for signalized intersections are given in Exhibit 19-8 from the *Highway Capacity Manual, 6th Edition* published by the Transportation Research Board.

Exhibit 19-8

	LOS by Volume-to-Capacity Ratio	
Control Delay (s/veh)	**v/c ≤1.0**	**v/c >1.0**
≤10	A	F
>10-20	B	F
>20-35	C	F
>35-55	D	F
>55-80	E	F
>80	F	F

For approach-based and intersection wide assessments, LOS is defined solely by control delay.

LEVEL OF SERVICE CRITERIA
FOR TWO-WAY STOP-CONTROLLED (TWSC) UNSIGNALIZED INTERSECTIONS

Level of Service (LOS) for a two-way stop-controlled (TWSC) intersection is determined by the computed or measured control delay. For motor vehicles, LOS is determined for each minor-street movement (or shared movement) as well as major-street left turns. LOS is not defined for the intersection as a whole or for major-street approaches.

The Level of Service Criteria for TWSC unsignalized intersections are given in Exhibit 20-2 from the *Highway Capacity Manual, 6th Edition* published by the Transportation Research Board.

Exhibit 20-2

	LOS by Volume-to-Capacity Ratio	
Control Delay (s/veh)	**v/c ≤1.0**	**v/c >1.0**
0-10	A	F
>10-15	B	F
>15-25	C	F
>25-35	D	F
>35-50	E	F
>50	F	F

The LOS criteria apply to each lane on a given approach and to each approach on the minor street.
LOS is not calculated for major-street approaches or for the intersection as a whole.

As Exhibit 20-2 notes, LOS F is assigned to the movement if the volume-to-capacity ratio for the movement exceeds 1.0, regardless of the control delay.

The Level of Service Criteria for unsignalized intersections are somewhat different from the criteria for signalized intersections.

LEVEL OF SERVICE CRITERIA
FOR ALL-WAY STOP-CONTROLLED (AWSC) UNSIGNALIZED INTERSECTIONS

The Levels of Service (LOS) for all-way stop-controlled (AWSC) intersections are given in Exhibit 21-8. As the exhibit notes, LOS F is assigned if the volume-to-capacity (v/c) ratio of a lane exceeds 1.0, regardless of the control delay. For assessment of LOS at the approach and intersection levels, LOS is based solely on control delay.

The Level of Service Criteria for AWSC unsignalized intersections are given in Exhibit 21-8 from the *Highway Capacity Manual, 6th Edition* published by the Transportation Research Board.

Exhibit 21-8

	LOS by Volume-to-Capacity Ratio	
Control Delay (s/veh)	**v/c ≤1.0**	**v/c >1.0**
0-10	A	F
>10-15	B	F
>15-25	C	F
>25-35	D	F
>35-50	E	F
>50	F	F

For approaches and intersection wide assessment, LOS is defined solely by control delay.

PROPOSED DOLLAR GENERAL

APPENDIX D

CAPACITY ANALYSIS

Lane Group	WBL	WBR	NBT	NBR	SBL	SBT
Lane Configurations						
Traffic Volume (vph)	15	15	673	16	16	826
Future Volume (vph)	15	15	673	16	16	826
Ideal Flow (vphpl)	1900	1900	1900	1900	1900	1900
Grade (%)	0%		-1%			0%
Lane Util. Factor	1.00	1.00	1.00	1.00	1.00	1.00
Frt	0.932		0.997			
Flt Protected	0.976					0.999
Satd. Flow (prot)	1694	0	1814	0	0	1809
Flt Permitted	0.976					0.999
Satd. Flow (perm)	1694	0	1814	0	0	1809
Link Speed (mph)	30		30			30
Link Distance (ft)	187		328			169
Travel Time (s)	4.3		7.5			3.8
Peak Hour Factor	0.90	0.90	0.90	0.90	0.90	0.90
Heavy Vehicles (%)	2%	2%	5%	2%	2%	5%
Adj. Flow (vph)	17	17	748	18	18	918
Shared Lane Traffic (%)						
Lane Group Flow (vph)	34	0	766	0	0	936
Enter Blocked Intersection	No	No	No	No	No	No
Lane Alignment	Left	Right	Left	Right	Left	Left
Median Width(ft)	12		0			0
Link Offset(ft)	0		0			0
Crosswalk Width(ft)	16		16			16
Two way Left Turn Lane						
Headway Factor	1.00	1.00	0.99	0.99	1.00	1.00
Turning Speed (mph)	15	9		9	15	
Sign Control	Stop		Free			Free

Intersection Summary	
Area Type:	Other
Control Type: Unsignalized	

Intersection	
Int Delay, s/veh	0.8

Movement	WBL	WBR	NBT	NBR	SBL	SBT
Lane Configurations						
Traffic Vol, veh/h	15	15	673	16	16	826
Future Vol, veh/h	15	15	673	16	16	826
Conflicting Peds, #/hr	0	0	0	0	0	0
Sign Control	Stop	Stop	Free	Free	Free	Free
RT Channelized	-	None	-	None	-	None
Storage Length	0	-	-	-	-	-
Veh in Median Storage, #	0	-	0	-	-	0
Grade, %	0	-	-1	-	-	0
Peak Hour Factor	90	90	90	90	90	90
Heavy Vehicles, %	2	2	5	2	2	5
Mvmt Flow	17	17	748	18	18	918

Major/Minor	Minor1		Major1		Major2	
Conflicting Flow All	1711	757	0	0	766	0
Stage 1	757	-	-	-	-	-
Stage 2	954	-	-	-	-	-
Critical Hdwy	6.42	6.22	-	-	4.12	-
Critical Hdwy Stg 1	5.42	-	-	-	-	-
Critical Hdwy Stg 2	5.42	-	-	-	-	-
Follow-up Hdwy	3.518	3.318	-	-	2.218	-
Pot Cap-1 Maneuver	100	408	-	-	847	-
Stage 1	463	-	-	-	-	-
Stage 2	374	-	-	-	-	-
Platoon blocked, %			-	-		-
Mov Cap-1 Maneuver	96	408	-	-	847	-
Mov Cap-2 Maneuver	96	-	-	-	-	-
Stage 1	443	-	-	-	-	-
Stage 2	374	-	-	-	-	-

Approach	WB	NB	SB
HCM Control Delay, s	34.5	0	0.2
HCM LOS	D		

Minor Lane/Major Mvmt	NBT	NBR	WBLn1	SBL	SBT
Capacity (veh/h)	-	-	155	847	-
HCM Lane V/C Ratio	-	-	0.215	0.021	-
HCM Control Delay (s)	-	-	34.5	9.3	0
HCM Lane LOS	-	-	D	A	A
HCM 95th %tile Q(veh)	-	-	0.8	0.1	-