July 18, 2022

Mr. Britton Jones
Flowerdale Commons 2021, LP
1723-B University Avenue, Suite 292
Oxford, MS 38655

RE: Traffic Impact Analysis for Flowerdale Commons

Dear Mr. Jones:

Please accept this letter as a third independent analysis of the impact of traffic to be potentially generated by the proposed Flowerdale Commons multifamily development ("Development") on Colonial Estates Road. The first Traffic Impact Analysis was performed by Engineering Services Solutions, Inc. in May 2022, and a second analysis was provided by W. L. Burle Engineers, P.A. in June 2022. Both studies demonstrated that the proposed development would have no significant impact on the quality and level of service along Colonial Estates Road and that the geometry and construction of Colonial Estates Road complies with both City Code and industry standards.

My review and analysis confirms the findings of the prior two studies. The impact produced by the Development will be small, and the roadway has more than sufficient capacity to support this Development and also future growth which may occur in this area. Further, Colonial Estates Road at the location of the Development is straight and flat and, thus, has excellent sight lines and safety characteristics.

Further, although not noted in the other analyses, Colonial Estates Road at this location, as compared to many other roadways in the City of Tupelo on which development has and continues to occur, has lower congestion and higher visibility and safety characteristics.

1. Background

I am a registered Professional Engineer (PE) in five states including Mississippi, and I am a registered Professional Traffic Operations Engineer (PTOE). I have over twenty years of traffic engineering experience performing work all over the southeastern United States for various departments of transportation, municipalities, and private sector clients. My career began in 2000 with the City of Jackson, Mississippi as the Assistant and then the City Traffic Engineer. In 2006, I became the Senior Traffic Engineer for Thompson Engineering in their Ridgeland, Mississippi office. In 2018, I opened my own firm specializing in traffic engineering services, and I currently serve as the County Engineer for Madison County among other clients.

2. Analysis

The first step in an analysis is to determine how many trips the proposed development will generate. This was determined as prescribed by Section 12.5.5(3) of the *City of Tupelo*

Development Code by using the Institute of Transportation Engineers, *Trip Generation, 10th Edition*. Using Land Use 220, Multi-family Housing (Low Rise) from this manual, it was determined that the Development will generate an additional 21 vehicle trips in the AM Peak Hour and 26 vehicle trips in the PM Peak Hour.

The Tupelo Code at Section 12.5.1 states that a Traffic Impact Analysis is not even required for a site plan unless the potential trip generation is at least 130 trips per hour, and these findings show that the Development will only produce 20% of that figure to even trigger a TIA. Thus, by City Code, the study is complete at this point.

However, as requested, we continued beyond the requirements of the Code with an analysis to determine what effect the additional trips will have on the adjacent street network. The levels of service (LOS) for Colonial Estates Road, both pre-development and post-development, were determined using the methodology for a Class 2 highway as outlined in Chapter 15 of the *Highway Capacity Manual, 2010 Edition (HCM)*. The LOS for the proposed intersections of the north driveway and the south driveway with Colonial Estates Road were determined using *Synchro 10*, a software package that uses the methodology for a two-way stop-controlled intersection (one way in our case) as outlined in Chapter 19 of the HCM.

The results from the above analysis are consistent with the prior two analyses, and these results verify that there is a significant amount of additional capacity available along Colonial Estates Road during both the AM and PM peak hours after the proposed development is constructed.

3. Roadway Geometry

In the letter provided to me from Ms. Jenny Savely on behalf of the City of Tupelo Planning Committee, the Planning Committee states that the application was recommended for denial due to concerns for dangerous street designs and concerns for the safety of passengers along Colonial Estates Road where the development's ingress and egress is provided.

While the analysis below often goes beyond the City of Tupelo's own Code, I can speak competently regarding Mississippi and industry standards for these roadway elements.

There are two proposed driveways for this development, a northern driveway and a southern driveway. The near side of the northern driveway is located roughly 150 feet from a BNSF railroad track. The near sides of the two proposed driveways are roughly 650 feet apart. The near side of the southern driveway is located 230 feet from the next driveway on the west side which is south of the bridge.

The ingress and egress locations relative to other roadway features are well within the 100 foot minimum distance between driveways as stated in the MDOT *Access Management Manual*.

Additionally, Table 3-1 of the AASHTO *A Policy on the Geometric Design of Highways and Streets* states the minimum safe stopping sight distance (SSD) for a 30 MPH traveled roadway is 112 feet with that of a 70 MPH travelled roadway being 301 feet. At the Development, visibility for the southern driveway is 700 feet to the south and over 900 feet to the north. The northern

driveway is over 1000 feet to the south and the intersection of Colonial Estates Road at McCullough Blvd is visible at roughly 550 feet to the north.

Thus, both driveways have visibility of traffic far beyond the established minimums and represent excellent sight conditions for the existing roadway and the Development.

Finally, the travel lanes on Colonial Estates Road are 10 feet wide, and this complies with traffic safety standards for a roadway of this type. The National Association of City Transportation Officials *Urban Street Design Guide* states that "Lane widths of 10 feet are appropriate in urban areas and have a positive impact on a street's safety without impacting traffic operations." Wider lanes are needed when there are commercial trucks and buses regularly using the roadway, which is not the case on Colonial Estates Road.

Shoulders and clear zones are important factors in the safety or a roadway, and they are objectively defined in Mississippi. Table 12-2-C of the MDOT *Roadway Design Manual* shows a minimum safety slope of 3:1 for roads using 3R Criteria. This roadway in the area of the proposed development has safety slopes below 3:1 (flatter and thus superior) and should a vehicle run off the road past the safety shoulder, the areas adjacent to the roadway are flat agricultural land. Further, except for one fire hydrant and five power poles there are no obstructions in the clear zones in the area of development.

I noted that additional residential development has occurred along Colonial Estates Road, including in areas with poorer visibility, but it was also appropriate for the City to approve these developments with regard to Colonial Estates Road's roadway type, construction and geometry.

4. Conclusion

The post-construction conditions in the area are quantifiable, as are the characteristics of Colonial Estates Road. They are all well within acceptable ranges for safety, and the Development will have a very small impact on the existing conditions. The roadway itself meets and exceeds the minimum requirements for design typically used in the design of roads in Mississippi, and, in many cases, it has excellent safety characteristics. When compared to other constricted and congested roadways within the City Limits of Tupelo on which development has and continues to occur, the Development, as noted by the City's Zoning Map, is appropriate for this location.

Sincerely,

Tim Bryan, P.E., PTOE
Principal Engineer

Enclosures

Existing AM Peak Hour Traffic

Time Period	Colonial Estates Road Northbound Approach				Colonial Estates Road Southbound Approach				N/A Eastbound Approach				N/A Westbound Approach			
	Left	Thru	Right	Total	Left	Thru	Right	Total	Left	Thru	Right	Total	Left	Thru	Right	Total
7:00 AM - 7:15 AM	0	22	0	22	0	32	0	32								
7:15 AM - 7:30 AM	0	33	0	33	0	33	0	33								
7:30 AM - 7:45 AM	0	29	0	29	0	45	0	45								
7:45 AM - 8:00 AM	0	41	0	41	0	57	0	57								
8:00 AM - 8:15 AM	0	24	0	24	0	39	0	39								
8:15 AM - 8:30 AM	0	16	0	16	0	41	0	41								
Peak Hour Totals	0	127	0	127	0	174	0	174	0	0	0	0	0	0	0	0
PHF				0.77				0.76								
Directional Split				42%				58%								

Existing PM Peak Hour Traffic

Time Period	Colonial Estates Road Northbound Approach				Colonial Estates Road Southbound Approach				N/A Eastbound Approach				N/A Westbound Approach			
	Left	Thru	Right	Total	Left	Thru	Right	Total	Left	Thru	Right	Total	Left	Thru	Right	Total
4:15 PM - 4:30 PM	0	31	0	31	0	33	0	33								
4:30 PM - 4:45 PM	0	58	0	58	0	32	0	32								
4:45 PM - 5:00 PM	0	52	0	52	0	39	0	39								
5:00 PM - 5:15 PM	0	81	0	81	0	49	0	49								
5:15 PM - 5:30 PM	0	39	0	39	0	41	0	41								
5:30 PM - 5:45 PM	0	51	0	51	0	30	0	30								
Peak Hour Totals	0	230	0	230	0	161	0	161	0	0	0	0	0	0	0	0
PHF				0.71				0.82								
Directional Split				59%				41%								

Site Generated Traffic Per Driveway

ITE Trip Generation Land Use **220** **Multi-Family Housing (Low Rise)**

AM Peak Hour						PM Peak Hour					
No. of Units	46	Average Rate			0.46	No. of Units	46	Average Rate			0.56
Directional Distribution	23%	Enter		77%	Exit	Directional Distribution	63%	Enter		37%	Exit
Total Veh. Trip Ends	21	Trips				Total Veh. Trip Ends	26	Trips			
Vehicle Trip	5	Enter		16	Exit	Vehicle Trip	16	Enter		10	Exit
SB RT (Enter)	3		WB RT (Exit)		9	SB RT (Enter)	7		WB RT (Exit)		4
NB LT (Enter)	2		WB LT (Exit)		7	NB LT (Enter)	10		WB LT (Exit)		6

Buildout Peak Hour Traffic

Time Period	Colonial Estates Road Northbound Approach				Colonial Estates Road Southbound Approach				Driveway Eastbound Approach				N/A Westbound Approach			
	Left	Thru	Right	Total	Left	Thru	Right	Total	Left	Thru	Right	Total	Left	Thru	Right	Total
Northern Driveway																
AM Peak Hour	2	134	0	136	0	174	3	177	7	0	9	16				
PM Peak Hour	10	236	0	246	0	161	7	168	6	0	4	10				
Southern Driveway																
AM Peak Hour	2	127	0	129	0	183	3	186	7	0	9	16				
PM Peak Hour	10	230	0	240	0	165	7	172	6	0	4	10				

CLASS TWO HIGHWAY LEVEL OF SERVICE
Highway Capacity Manual 2010, Chapter 15

PROJECT INFORMATION

Project Name	Flowerdale Commons Apartments	Analysis Description	Existing Conditions
Project Location	Tupelo, Mississippi		AM Peak Hr
Highway Name	Colonial Estates Road	Analysis Period	7:15 AM - 8:15 AM

INPUT DATA

Volume	301	veh/hr	Directional Split	42% /	58%
Percent Trucks	0%		Lane Width	10	ft
Percent RV's	0%		Shoulder Width	2	ft
Peak Hour Factor	0.77		BFFS	30	mph
Access Points/Mile	20		*Posted Speed Limit*		
Terrain	Rolling		No Passing Zone	100%	

LOS CALCULATIONS

Estimate FFS

f_{LS} = 3.7 mph f_A = 5 mph *Exhibits 15-7 and 15-8*

FFS= BFFS- f_{LS} - f_A = 21 mph *Equation 15-2*

Demand Adjustment for PTSF

$V_{42\%} = V_1$ = Volume * Directional Split = 126 veh/hr NB

$V_{58\%} = V_2$ = Volume * Directional Split = 175 veh/hr SB

P_{T1} = 0 E_{T1} = 1.87 P_{R1} = 0 E_{R1} = 1 *Exhibit 15-18*

P_{T2} = 0 E_{T2} = 1.83 P_{R2} = 0 E_{R2} = 1

$f_{HV, PTSF1} = 1 / (1 + P_T (E_T - 1) + P_R (E_R - 1)$ = 1

$f_{HV, PTSF2} = 1 / (1 + P_T (E_T - 1) + P_R (E_R - 1)$ = 1

$f_{g, PTSF1}$ = 0.75 $f_{g, PTSF2}$ = 0.78 $f_{HV, PTSF1}$ = 1 $f_{HV, PTSF2}$ = 1 *Exhibit 15-16*

$v_{i, PTSF1} = V_i / (PHF * f_{g, PTSF} * f_{HV, PTSF})$ = 210 pc/hr *Equation 15-7*

$v_{i, PTSF2} = V_i / (PHF * f_{g, PTSF} * f_{HV, PTSF})$ = 290.7 pc/hr

Estimate PTSF

Direction 1 a= -0.00259 b = 0.904 $f_{np, PTSF}$ = 38.13 *Exhibits 15-20 and 15-21*

Direction 2 a= -0.00217 b = 0.925

$BPTSF_1 = 100 (1 - \exp(a * v^b))$ = 27.8 % *Equation 15-10*

$BPTSF_2 = 100 (1 - \exp(a * v^b))$ = 26.3 %

$PTSF_1 = BPTSF_d + f_{np, PTSF} (v_{d,PTSF} / (v_{d,PTSF} + v_{o,PTSF})$ 43.8 % *Equation 15-9*

$PTSF_2 = BPTSF_d + f_{np, PTSF} (v_{d,PTSF} / (v_{d,PTSF} + v_{o,PTSF})$ 48.5 %

Determine LOS

Northbound Level of Service B

Southbound Level of Service B

Exhibit 15-3

LOS	Class II HWY PTSF (%)
A	≤ 40
B	> 40-55
C	> 55-70
D	> 70-85
E	> 85

CLASS TWO HIGHWAY LEVEL OF SERVICE
Highway Capacity Manual 2010, Chapter 15

PROJECT INFORMATION

Project Name	Flowerdale Commons Apartments	Analysis Description	Existing Conditions
Project Location	Tupelo, Mississippi		PM Peak Hr
Highway Name	Colonial Estates Road	Analysis Period	4:30 PM - 5:30 PM

INPUT DATA

Volume	391	veh/hr	Directional Split	59% /	41%
Percent Trucks	0%		Lane Width	10	ft
Percent RV's	0%		Shoulder Width	2	ft
Peak Hour Factor	0.77		BFFS	30	mph
Access Points/Mile	20		*Posted Speed Limit*		
Terrain	Rolling		No Passing Zone	100%	

LOS CALCULATIONS

Estimate FFS

f_{LS} = 3.7 mph f_A = 5 mph *Exhibits 15-7 and 15-8*

FFS= BFFS- f_{LS} - f_A = 21 mph *Equation 15-2*

Demand Adjustment for PTSF

$V_{42\%} = V_1$ = Volume * Directional Split = 231 veh/hr NB

$V_{58\%} = V_2$ = Volume * Directional Split = 160 veh/hr SB

P_{T1} = 0 E_{T1} = 1.77 P_{R1} = 0 E_{R1} = 1 *Exhibit 15-18*

P_{T2} = 0 E_{T2} = 1.84 P_{R2} = 0 E_{R2} = 1

$f_{HV, PTSF1} = 1 / (1 + P_T (E_T - 1) + P_R (E_R - 1)$ = 1

$f_{HV, PTSF2} = 1 / (1 + P_T (E_T - 1) + P_R (E_R - 1)$ = 1

$f_{g, PTSF1}$ = 0.82 $f_{g, PTSF2}$ = 0.77 $f_{HV, PTSF1}$ = 1 $f_{HV, PTSF2}$ = 1 *Exhibit 15-16*

$v_{i, PTSF1} = V_i / (PHF * f_{g, PTSF} * f_{HV, PTSF})$ = 389 pc/hr *Equation 15-7*

$v_{i, PTSF2} = V_i / (PHF * f_{g, PTSF} * f_{HV, PTSF})$ = 270.4 pc/hr

Estimate PTSF

Direction 1 a= -0.0014 b = 0.973 $f_{np, PTSF}$ = 53.85 *Exhibits 15-20 and 15-21*

Direction 2 a= -0.0015 b = 0.966

$BPTSF_1 = 100 (1 - \exp(a * v^b))$ = 37.1 % *Equation 15-10*

$BPTSF_2 = 100 (1 - \exp(a * v^b))$ = 37.9 %

$PTSF_1 = BPTSF_d + f_{np, PTSF} (v_{d,PTSF} / (v_{d,PTSF} + v_{o,PTSF})$ 68.88 % *Equation 15-9*

$PTSF_2 = BPTSF_d + f_{np, PTSF} (v_{d,PTSF} / (v_{d,PTSF} + v_{o,PTSF})$ 59.98 %

Determine LOS

Northbound Level of Service C

Southbound Level of Service C

LOS	Class II HWY PTSF (%)
A	≤ 40
B	> 40-55
C	> 55-70
D	> 70-85
E	> 85

Exhibit 15-3

CLASS TWO HIGHWAY LEVEL OF SERVICE
Highway Capacity Manual 2010, Chapter 15

PROJECT INFORMATION

Project Name	Flowerdale Commons Apartments	Analysis Description	Buildout Conditions
Project Location	Tupelo, Mississippi		AM Peak Hr
Highway Name	Colonial Estates Road	Analysis Period	7:15 AM - 8:15 AM

INPUT DATA

Volume	322	veh/hr	Directional Split	42% /	58%
Percent Trucks	0%		Lane Width	10	ft
Percent RV's	0%		Shoulder Width	2	ft
Peak Hour Factor	0.77		BFFS	30	mph
Access Points/Mile	20		*Posted Speed Limit*		
Terrain	Rolling		No Passing Zone	100%	

LOS CALCULATIONS

Estimate FFS

f_{LS} = 3.7 mph f_A = 5 mph *Exhibits 15-7 and 15-8*

FFS= BFFS- f_{LS} - f_A = 21 mph *Equation 15-2*

Demand Adjustment for PTSF

$V_{42\%} = V_1$ = Volume * Directional Split = 135 veh/hr NB

$V_{58\%} = V_2$ = Volume * Directional Split = 187 veh/hr SB

P_{T1} = 0 E_{T1} = 1.87 P_{R1} = 0 E_{R1} = 1 *Exhibit 15-18*

P_{T2} = 0 E_{T2} = 1.81 P_{R2} = 0 E_{R2} = 1

$f_{HV, PTSF1} = 1 / (1 + P_T (E_T - 1) + P_R (E_R - 1) =$ 1

$f_{HV, PTSF2} = 1 / (1 + P_T (E_T - 1) + P_R (E_R - 1) =$ 1

$f_{g, PTSF1}$ = 0.75 $f_{g, PTSF2}$ = 0.79 $f_{HV, PTSF1}$ = 1 $f_{HV, PTSF2}$ = 1 *Exhibit 15-16*

$v_{i, PTSF1} = V_i / (PHF * f_{g, PTSF} * f_{HV, PTSF}) =$ 222 pc/hr *Equation 15-7*

$v_{i, PTSF2} = V_i / (PHF * f_{g, PTSF} * f_{HV, PTSF}) =$ 307 pc/hr

Estimate PTSF

Direction 1 a= -0.00149 b = 0.968 $f_{np, PTSF}$ = 55.2 *Exhibits 15-20 and 15-21*

Direction 2 a= -0.00183 b = 0.946

$BPTSF_1 = 100 (1 - \exp(a * v^b)) =$ 24.3 % *Equation 15-10*

$BPTSF_2 = 100 (1 - \exp(a * v^b)) =$ 26.2 %

$PTSF_1 = BPTSF_d + f_{np, PTSF} (v_{d,PTSF} / (v_{d,PTSF} + v_{o,PTSF})$ 47.5 % *Equation 15-9*

$PTSF_2 = BPTSF_d + f_{np, PTSF} (v_{d,PTSF} / (v_{d,PTSF} + v_{o,PTSF})$ 58.2 %

Determine LOS

Northbound Level of Service: B

Southbound Level of Service: C

Exhibit 15-3

LOS	Class II HWY PTSF (%)
A	≤ 40
B	> 40-55
C	> 55-70
D	> 70-85
E	> 85

CLASS TWO HIGHWAY LEVEL OF SERVICE
Highway Capacity Manual 2010, Chapter 15

PROJECT INFORMATION

Project Name	Flowerdale Commons Apartments	Analysis Description	Buildout Conditions
Project Location	Tupelo, Mississippi		PM Peak Hr
Highway Name	Colonial Estates Road	Analysis Period	4:30 PM - 5:30 PM

INPUT DATA

Volume	322	veh/hr	Directional Split	59% /	41%
Percent Trucks	0%		Lane Width	10	ft
Percent RV's	0%		Shoulder Width	2	ft
Peak Hour Factor	0.77		BFFS	30	mph
Access Points/Mile	20		*Posted Speed Limit*		
Terrain	Rolling		No Passing Zone	100%	

LOS CALCULATIONS

Estimate FFS

f_{LS} = 3.7 mph f_A = 5 mph *Exhibits 15-7 and 15-8*

FFS= BFFS- f_{LS} - f_A = 21 mph *Equation 15-2*

Demand Adjustment for PTSF

$V_{42\%} = V_1$ = Volume * Directional Split = 190 veh/hr NB

$V_{58\%} = V_2$ = Volume * Directional Split = 132 veh/hr SB

P_{T1} = 0 E_{T1} = 1.81 P_{R1} = 0 E_{R1} = 1 *Exhibit 15-18*

P_{T2} = 0 E_{T2} = 1.87 P_{R2} = 0 E_{R2} = 1

$f_{HV, PTSF1} = 1 / (1 + P_T (E_T - 1) + P_R (E_R - 1)$ = 1

$f_{HV, PTSF2} = 1 / (1 + P_T (E_T - 1) + P_R (E_R - 1)$ = 1

$f_{g, PTSF1}$ = 0.79 $f_{g, PTSF2}$ = 0.75 $f_{HV, PTSF1}$ = 1 $f_{HV, PTSF2}$ = 1 *Exhibit 15-16*

$v_{i, PTSF1} = V_i / (PHF * f_{g, PTSF} * f_{HV, PTSF})$ = 329 pc/hr *Equation 15-7*

$v_{i, PTSF2} = V_i / (PHF * f_{g, PTSF} * f_{HV, PTSF})$ = 229 pc/hr

Estimate PTSF

Direction 1 a= -0.0019 b = 0.941 $f_{np, PTSF}$ = 54.96 *Exhibits 15-20 and 15-21*

Direction 2 a= -0.0015 b = 0.966

$BPTSF_1 = 100 (1 - \exp(a * v^b))$ = 35.9 % *Equation 15-10*

$BPTSF_2 = 100 (1 - \exp(a * v^b))$ = 33.3 %

$PTSF_1 = BPTSF_d + f_{np, PTSF} (v_{d,PTSF} / (v_{d,PTSF} + v_{o,PTSF})$ 68.3 % *Equation 15-9*

$PTSF_2 = BPTSF_d + f_{np, PTSF} (v_{d,PTSF} / (v_{d,PTSF} + v_{o,PTSF})$ 55.8 %

Determine LOS

Northbound Level of Service C

Southbound Level of Service C

Exhibit 15-3

LOS	Class II HWY PTSF (%)
A	≤ 40
B	> 40-55
C	> 55-70
D	> 70-85
E	> 85

Intersection	
Int Delay, s/veh	0.5

Movement	EBL	EBR	NBL	NBT	SBT	SBR
Lane Configurations						
Traffic Vol, veh/h	7	9	2	127	183	3
Future Vol, veh/h	7	9	2	127	183	3
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	-	-	0	0	-
Peak Hour Factor	92	92	77	77	76	76
Heavy Vehicles, %	0	0	0	0	0	0
Mvmt Flow	8	10	3	165	241	4

Major/Minor	Minor2		Major1		Major2	
Conflicting Flow All	414	243	245	0	-	0
Stage 1	243	-	-	-	-	-
Stage 2	171	-	-	-	-	-
Critical Hdwy	6.4	6.2	4.1	-	-	-
Critical Hdwy Stg 1	5.4	-	-	-	-	-
Critical Hdwy Stg 2	5.4	-	-	-	-	-
Follow-up Hdwy	3.5	3.3	2.2	-	-	-
Pot Cap-1 Maneuver	599	801	1333	-	-	-
Stage 1	802	-	-	-	-	-
Stage 2	864	-	-	-	-	-
Platoon blocked, %				-	-	-
Mov Cap-1 Maneuver	598	801	1333	-	-	-
Mov Cap-2 Maneuver	598	-	-	-	-	-
Stage 1	800	-	-	-	-	-
Stage 2	864	-	-	-	-	-

Approach	EB	NB	SB
HCM Control Delay, s	10.3	0.1	0
HCM LOS	B		

Minor Lane/Major Mvmt	NBL	NBT	EBLn1	SBT	SBR
Capacity (veh/h)	1333	-	697	-	-
HCM Lane V/C Ratio	0.002	-	0.025	-	-
HCM Control Delay (s)	7.7	0	10.3	-	-
HCM Lane LOS	A	A	B	-	-
HCM 95th %tile Q(veh)	0	-	0.1	-	-

Intersection	
Int Delay, s/veh	0.5

Movement	EBL	EBR	NBL	NBT	SBT	SBR
Lane Configurations						
Traffic Vol, veh/h	7	9	2	134	174	3
Future Vol, veh/h	7	9	2	134	174	3
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	-	-	0	0	-
Peak Hour Factor	92	92	71	71	82	82
Heavy Vehicles, %	0	0	0	0	0	0
Mvmt Flow	8	10	3	189	212	4

Major/Minor	Minor2		Major1		Major2	
Conflicting Flow All	409	214	216	0	-	0
Stage 1	214	-	-	-	-	-
Stage 2	195	-	-	-	-	-
Critical Hdwy	6.4	6.2	4.1	-	-	-
Critical Hdwy Stg 1	5.4	-	-	-	-	-
Critical Hdwy Stg 2	5.4	-	-	-	-	-
Follow-up Hdwy	3.5	3.3	2.2	-	-	-
Pot Cap-1 Maneuver	602	831	1366	-	-	-
Stage 1	826	-	-	-	-	-
Stage 2	843	-	-	-	-	-
Platoon blocked, %				-	-	-
Mov Cap-1 Maneuver	601	831	1366	-	-	-
Mov Cap-2 Maneuver	601	-	-	-	-	-
Stage 1	824	-	-	-	-	-
Stage 2	843	-	-	-	-	-

Approach	EB	NB	SB
HCM Control Delay, s	10.2	0.1	0
HCM LOS	B		

Minor Lane/Major Mvmt	NBL	NBT	EBLn1	SBT	SBR
Capacity (veh/h)	1366	-	712	-	-
HCM Lane V/C Ratio	0.002	-	0.024	-	-
HCM Control Delay (s)	7.6	0	10.2	-	-
HCM Lane LOS	A	A	B	-	-
HCM 95th %tile Q(veh)	0	-	0.1	-	-

Intersection	
Int Delay, s/veh	0.4

Movement	EBL	EBR	NBL	NBT	SBT	SBR
Lane Configurations						
Traffic Vol, veh/h	6	4	10	230	165	7
Future Vol, veh/h	6	4	10	230	165	7
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	-	-	0	0	-
Peak Hour Factor	92	92	77	77	76	76
Heavy Vehicles, %	0	0	0	0	0	0
Mvmt Flow	7	4	13	299	217	9

Major/Minor	Minor2		Major1		Major2	
Conflicting Flow All	547	222	226	0	-	0
Stage 1	222	-	-	-	-	-
Stage 2	325	-	-	-	-	-
Critical Hdwy	6.4	6.2	4.1	-	-	-
Critical Hdwy Stg 1	5.4	-	-	-	-	-
Critical Hdwy Stg 2	5.4	-	-	-	-	-
Follow-up Hdwy	3.5	3.3	2.2	-	-	-
Pot Cap-1 Maneuver	502	823	1354	-	-	-
Stage 1	820	-	-	-	-	-
Stage 2	737	-	-	-	-	-
Platoon blocked, %				-	-	-
Mov Cap-1 Maneuver	496	823	1354	-	-	-
Mov Cap-2 Maneuver	496	-	-	-	-	-
Stage 1	810	-	-	-	-	-
Stage 2	737	-	-	-	-	-

Approach	EB	NB	SB
HCM Control Delay, s	11.2	0.3	0
HCM LOS	B		

Minor Lane/Major Mvmt	NBL	NBT	EBLn1	SBT	SBR
Capacity (veh/h)	1354	-	590	-	-
HCM Lane V/C Ratio	0.01	-	0.018	-	-
HCM Control Delay (s)	7.7	0	11.2	-	-
HCM Lane LOS	A	A	B	-	-
HCM 95th %tile Q(veh)	0	-	0.1	-	-

Intersection	
Int Delay, s/veh	0.4

Movement	EBL	EBR	NBL	NBT	SBT	SBR
Lane Configurations						
Traffic Vol, veh/h	6	4	10	236	161	7
Future Vol, veh/h	6	4	10	236	161	7
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	-	-	0	0	-
Peak Hour Factor	92	92	71	71	82	82
Heavy Vehicles, %	0	0	0	0	0	0
Mvmt Flow	7	4	14	332	196	9

Major/Minor	Minor2		Major1		Major2	
Conflicting Flow All	561	201	205	0	-	0
Stage 1	201	-	-	-	-	-
Stage 2	360	-	-	-	-	-
Critical Hdwy	6.4	6.2	4.1	-	-	-
Critical Hdwy Stg 1	5.4	-	-	-	-	-
Critical Hdwy Stg 2	5.4	-	-	-	-	-
Follow-up Hdwy	3.5	3.3	2.2	-	-	-
Pot Cap-1 Maneuver	492	845	1378	-	-	-
Stage 1	838	-	-	-	-	-
Stage 2	710	-	-	-	-	-
Platoon blocked, %				-	-	-
Mov Cap-1 Maneuver	486	845	1378	-	-	-
Mov Cap-2 Maneuver	486	-	-	-	-	-
Stage 1	828	-	-	-	-	-
Stage 2	710	-	-	-	-	-

Approach	EB	NB	SB
HCM Control Delay, s	11.3	0.3	0
HCM LOS	B		

Minor Lane/Major Mvmt	NBL	NBT	EBLn1	SBT	SBR
Capacity (veh/h)	1378	-	586	-	-
HCM Lane V/C Ratio	0.01	-	0.019	-	-
HCM Control Delay (s)	7.6	0	11.3	-	-
HCM Lane LOS	A	A	B	-	-
HCM 95th %tile Q(veh)	0	-	0.1	-	-

TYPE 3 – CONVENTIONAL HIGHWAYS

Signalized Intersection Spacing

For Type 3 – Conventional Highways, the minimum spacing for signalized intersections will be 1/3 mile for urban areas and 2/3 mile for rural areas. Distances between signals should not vary by more than 10% in order that good progression of traffic may be maintained in both directions.

Driveway Spacing - General

The minimum spacing for driveways will be as shown in Table 6 and as further described in the following sections. Spacing between driveways is measured from near edge to near edge of adjacent driveways as shown in Figure 1.

Table 6
Minimum Connection Spacing for Driveways on Type 3 Highways

			POSTED SPEED	SPACING DISTANCE
COMMERCIAL DRIVEWAY	A	>50 PEAK HOUR TRIPS AND ≥2000 AADT	≤30mph	185’
			35mph	245’
			40mph	300’
			45mph	350’
			≥50mph	425’
	B	≤50 PEAK HOUR TRIPS OR <2000 AADT	NA	100’
NON-COMMERCIAL DRIVEWAY	C	≥2000 AADT	NA	50’
	D	<2000 AADT	NA	25’

NOTE: The above criteria are also summarized in Appendix 2.

The criteria in Table 6 apply to individual sites with multiple driveways to a highway, and also govern the allowable spacing between driveways located on adjacent properties. Exceptions to the minimum connection spacing may be approved for the following conditions if it is determined that MDOT does not want to purchase the right-of-way.

1. As a result of an MDOT action such as construction modifications the property would become land-locked.
2. Exception is necessary in order to replace reasonable access that may be lost due to MDOT highway reconstruction or modification.

Table 3-1. Stopping Sight Distance on Level Roadways

Metric					U.S. Customary				
Design Speed (km/h)	Brake Reaction Distance (m)	Braking Distance on Level (m)	Stopping Sight Distance		Design Speed (mph)	Brake Reaction Distance (ft)	Braking Distance on Level (ft)	Stopping Sight Distance	
			Calculated (m)	Design (m)				Calculated (ft)	Design (ft)
20	13.9	4.6	18.5	20	15	55.1	21.6	76.7	80
30	20.9	10.3	31.2	35	20	73.5	38.4	111.9	115
40	27.8	18.4	46.2	50	25	91.9	60.0	151.9	155
50	34.8	28.7	63.5	65	30	110.3	86.4	196.7	200
60	41.7	41.3	83.0	85	35	128.6	117.6	246.2	250
70	48.7	56.2	104.9	105	40	147.0	153.6	300.6	305
80	55.6	73.4	129.0	130	45	165.4	194.4	359.8	360
90	62.6	92.9	155.5	160	50	183.8	240.0	423.8	425
100	69.5	114.7	184.2	185	55	202.1	290.3	492.4	495
110	76.5	138.8	215.3	220	60	220.5	345.5	566.0	570
120	83.4	165.2	248.6	250	65	238.9	405.5	644.4	645
130	90.4	193.8	284.2	285	70	257.3	470.3	727.6	730
					75	275.6	539.9	815.5	820
					80	294.0	614.3	908.3	910

Note: Brake reaction distance predicated on a time of 2.5 s; deceleration rate of 3.4 m/s^2 [11.2 ft/s^2] used to determine calculated sight distance.

Design Values

The stopping sight distance is the sum of the distance traversed during the brake reaction time and the distance to brake the vehicle to a stop. The computed distances for various speeds at the assumed conditions on level roadways are shown in Table 3-1 and were developed from the following equation:

Metric	U.S. Customary
$SSD = 0.278Vt + 0.039\frac{V^2}{a}$	$SSD = 1.47Vt + 1.075\frac{V^2}{a}$ (3-2)
where:	where:
SSD = stopping sight distance, m	SSD = stopping sight distance, ft
V = design speed, km/h	V = design speed, mph
t = brake reaction time, 2.5 s	t = brake reaction time, 2.5 s
a = deceleration rate, m/s^2	a = deceleration rate, ft/s^2

Stopping sight distances exceeding those shown in Table 3-1 should be used as the basis for design wherever practical. Use of longer stopping sight distances increases the margin for error for all drivers and, in particular, for those who operate at or near the design speed during wet pavement conditions. New pavements should have initially, and should retain, friction coefficients consistent with the deceleration rates used to develop Table 3-1.

EXISTING

Lane widths of 10 feet are appropriate in urban areas and have a positive impact on a street's safety without impacting traffic operations.

DISCUSSION

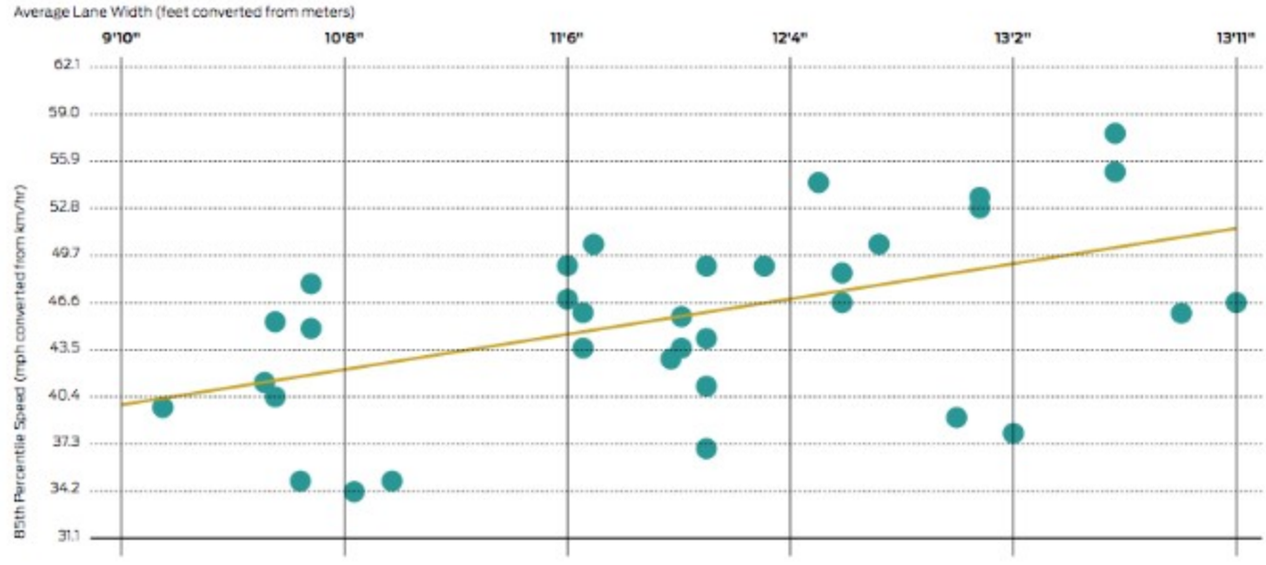
Travel lanes are striped to define the intended path of travel for vehicles along a corridor. Historically, wider travel lanes (11–13 feet) have been favored to create a more forgiving buffer to drivers, especially in high-speed environments where narrow lanes may feel uncomfortable or increase potential for side-swipe collisions.

Lane widths less than 12 feet have also historically been assumed to decrease traffic flow and capacity, a claim new research refutes.[1]

+ More Info

The relationships between lane widths and vehicle speed is complicated by many factors, including time of day, the amount of traffic present, and even the age of the driver. Narrower streets help promote slower driving speeds which, in turn, reduce the severity of crashes. Narrower streets have other benefits as well, including reduced crossing distances, shorter signal cycles, less stormwater, and less construction material to build.

Wider travel lanes are correlated with higher vehicle speeds.



"As the width of the lane increased, the speed on the roadway increased... When lane widths are 1 m (3.3 ft) greater, speeds are predicted to be 15 km/h (9.4 mph) faster."

Chart source: Fitzpatrick, Kay, Paul Carlson, Marcus Brewer, and Mark Wooldridge. 2000. "Design Factors That Affect Driver Speed on Suburban Streets." *Transportation Research Record* 1751: 18–25.

Wider travel lanes are correlated with higher vehicle speeds.

For multi-lane roadways where transit or freight vehicles are present and require a wider travel lane, the wider lane should be the outside lane (curbside or next to parking). Inside lanes should continue to be designed at the minimum possible width. Major truck or transit routes through urban areas may require the use of wider lane widths.

Lane widths of 10 feet are appropriate in urban areas and have a positive impact on a street's safety without impacting traffic operations. For designated truck or transit routes, one travel lane of 11 feet may be used in each direction. In select cases, narrower travel lanes (9–9.5 feet) can be effective as through lanes in conjunction with a turn lane.[2]

Table 12-2-C
GEOMETRIC DESIGN CRITERIA FOR RURAL ARTERIALS (2-LANE)
(3R Criteria)

	DESIGN ELEMENT			Manual Section	Design ADT			
					0 – 2000		Over 2000	
					T < 10%	T ≥ 10%	T < 10%	T ≥ 10%
Design Controls	Design Year			12-2.02	Desirable: 10 Years Minimum: Current			
	*Design Speed **(2)**			12-2.02	≤ 55 mph			
	Control of Access			11-1.05	Control by Regulation (Type 3)			
	Level of Service Threshold			12-2.02	Desirable: B Minimum: D			
Cross-Section Elements	*Travel Lane Width			12-2.03	Des.: 12 ft Min.: 11 ft		12 ft	
	Shoulder Width	*Usable		12-2.03	3 ft		5 ft	6 ft
		Paved **(3)**			2 ft			
	*Cross Slope	Travel Lane **(4)**		2-8.03 12-2.04	2%			
		Shoulder			See Note **(5)**			
	Auxiliary Lanes	Lane Width		2-8.03 12-2.06	Desirable: 12 ft Minimum: 11 ft			
		Usable Shoulder Width			3 ft			
	Reconstructed/ Rehabilitated Bridges	*Design Loading Structural Capacity		12-2.03	See Note **(6)**			
		Minimum Width **(6)**			Des.: 30 ft Min.: 28 ft		34 ft	36 ft
	Existing Bridges to Remain in Place	*Design Loading Structural Capacity		12-2.03	See Note **(7)**			
		Minimum Width **(7)**			Traveled Way Width + 2 ft		Traveled Way Width + 4 ft	
	Desirable Right of Way Border Width (beyond toe/top of fill/cut slope)			2-8.03 11-1.01	15 ft - 20 ft			
	Roadside Clear Zone	Guardrail		12-2.03 12-2.08	Usable Shoulder Width			
		Obstruction			See Note **(8)**			
	Slope Schedule **(9)**	Cut	Foreslope (within clear zone)	2-8.03 12-2.08	4:1			
			Depth of Ditch		3 ft			
			Backslope		3:1			
		Fill	Safety Slope (within clear zone)		Desirable: 6:1 Maximum: 3:1			
			Fill Slope (outside clear zone)		Desirable: 3:1 Maximum: 2:1			
Alignment Elements	**DESIGN SPEED**				**45 mph**	**50 mph**	**55 mph**	
	*Stopping Sight Distance			12-2.05	See Section 12-2.05			
	Intersection Sight Distance **(10)**			6-6.0	500 ft	555 ft	610 ft	
	*Superelevation Rate			2-8.02 12-2.04	See Section 12-2:04 e_{max} = 10%			
	*Minimum Horizontal Curve Radius			2-8.02 12-2.04	See Section 12-2:04			
	*Maximum Grades **(11)**	Level		2-8.02 12-2.05	Existing			
		Rolling						
	Minimum Grades			12-2.05	See Note **(12)**			
	Vertical Curve (K-values)	*Crest		2-8.02 12-2.05	See Section 12-2.05			
		Sag						
	*Vertical Clearance (arterial under) **(13)**	Reconstructed/ Rehabilitated Bridges		2-8.02	Desirable: 17 ft Minimum: 16 ft			
		Existing Bridges			Desirable: 16 ft Minimum: 14.5 ft			
		Sign Truss/ Pedestrian Bridge			19 ft			
	Vertical Clearance (arterial over railroad) **(14)**			2-8.02	Desirable: 25 ft Minimum: 23.5 ft			

*For application of controlling design criteria, see Section 2-9.02.