

MEMORANDUM

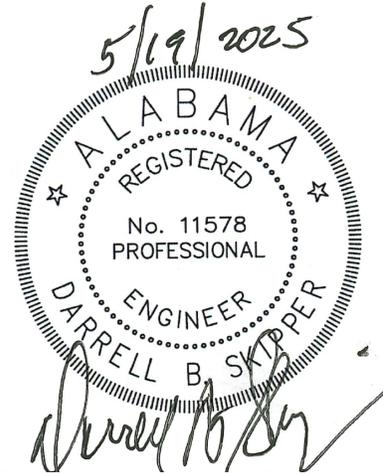
To: Matthew Johnson, P.E., ALDOT Area Traffic Engineer
Michael Johnson, P.E., Madison City Engineer
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cc: Jeff Mullins, P.E., Mullins, LLC.

Date: May 19, 2025

Subject: Jack Clift Boulevard at U.S. Highway 72 Intersection Modification
Madison County, Alabama



During a Team's meeting conducted on April 22nd, 2025 we committed to provide a technical memorandum to each of you which summarized the implications of modifying the traffic signal phasing and timings for the intersection of U.S. Highway 72 at Jack Clift Boulevard in Madison County, Alabama. The intent of the subsequent materials and a concept drawing of the traffic control modifications recommended at the intersection of U.S. Highway 72 and Jack Clift Boulevard are presented for review and concurrence by the Alabama Department of Transportation (ALDOT) and the City of Madison.

Purpose and Need for Intersection Modification: Traffic volumes during peak hour operations on U.S. Highway 72 have increased in recent years for the morning and afternoon peak conditions. Additionally, traffic volumes on U.S. Highway 72 on Saturdays during peak hours for retail trips have also increased to a point that require modifications that reallocate traffic signal phasing and timings.

Introduction

The purpose of this memorandum is to document the findings of analysis undertaken to determine the impact from restricting the northbound and southbound through movements at the intersection of U.S. Highway 72 at Jack Clift Boulevard in Madison County, Alabama. The primary objectives to be met by this memorandum are as follows:

- To obtain traffic count data for the study intersections; and
- Conduct Peak Hour Capacity Analysis at the study intersections both with and without the northbound and southbound through movements in place at the intersection of U.S. Highway 72 at Jack Clift Boulevard.

Study Intersection and Background Information

The study intersection being analyzed is the intersection of U.S. Highway 72 at Jack Clift Boulevard. Approximately 325' north of the study intersection, Jack Clift Boulevard intersects John Henry Way in Madison, Alabama. In addition to the intersection of U.S. Highway 72 at Jack Clift Boulevard, the intersection of Jack Clift Boulevard at John Henry Way is being analyzed due to their shared proximity.

Existing traffic counts at the study intersections were conducted on April 26, 2025. The Saturday peak hour of traffic was chosen for analysis purposes due to the surrounding land uses. A discount club (ITE Land Use 857) was recently constructed along Jack Clift Boulevard and is likely a main contributor to the traffic turning onto and off of Jack Clift Boulevard. **Figure 1** illustrates the peak hour traffic counts recorded and analyzed.

The main study intersection of U.S. Highway 72 at Jack Clift Boulevard currently has zero movement restrictions; however, the purpose of this report is to analyze the benefits provided by restricting the northbound and southbound through movements on Jack Clift Boulevard.

Restricting the northbound and southbound through movements at the intersection of U.S. Highway 72 at Jack Clift Boulevard would cause traffic to be re-routed through adjacent accesses to each shopping center. **Figure 2** illustrates the resulting peak hour traffic volumes at the intersection of U.S. Highway 72 at Jack Clift Boulevard.

Figure 1 – Existing Peak Hour Traffic Counts

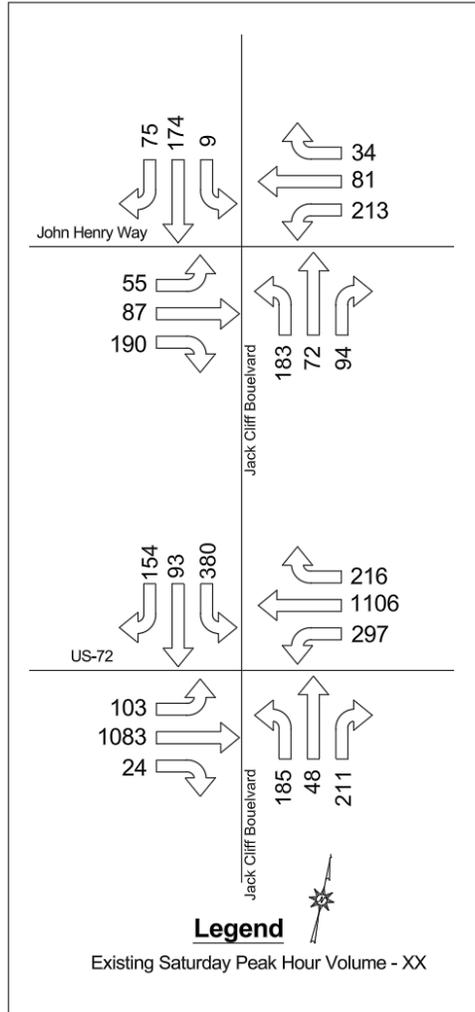
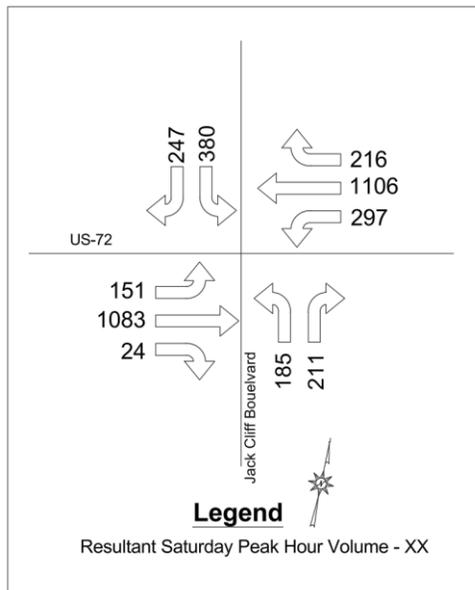


Figure 2 – Resultant Peak Hour Traffic Volumes



Capacity Analysis

Saturday peak hour capacity analyses were conducted for the study intersections both with and without the proposed movement restrictions in place.

Capacity analyses were conducted using methods as outlined in the *Highway Capacity Manual*. According to methods of analysis, intersection capacity is expressed as levels of service, ranging from “A” (best) to “F” (worst). In general, a level of service (LOS) “C” is considered desirable, while a level of service “D” is considered acceptable during peak hours of traffic flow. The results of the capacity analyses for are illustrated in **Table 1**, and capacity printouts are provided as an attachment.

Table 1 –Study Intersection Capacity Analysis

Intersection		Approach	LOS	Delay	95th Queue
US-72 at Jack Clift Blvd (signalized)	With NB and SB Throughs	NB	F	128.1	243
		SB	E	59.8	293
		EB	C	34.8	556
		WB	D	35.3	526
		Overall	D	50.4	N/A
US-72 at Jack Clift Blvd (signalized)	Without NB and SB Throughs	NB	E	61.1	234
		SB	E	58.4	205
		EB	C	20	435
		WB	C	22.3	391
		Overall	C	27.9	N/A
John Henry Way at Jack Clift Blvd (all- way stop)		NB	C	19.7	80
		SB	C	22.8	95
		EB	F	60.8	262.5
		WB	E	44.2	197.5

The capacity analyses indicate that the approaches to the study intersections currently operate with mostly acceptable levels of service. The Jack Clift Boulevard approaches to U.S. Highway 72 currently experiences below acceptable levels of service. The Jack Clift Boulevard approaches to U.S. Highway 72 currently operate with poor levels of service due to the intersection being coordinated with other intersections along U.S. Highway 72 and having a relatively long cycle length. The John Henry Way approaches to Jack Clift Boulevard currently experiences below acceptable levels of service.

Restricting the northbound and southbound through movements at the intersection of U.S. Highway 72 at Jack Clift Boulevard reduces delay and queueing for all approaches to the intersection. Due to the coordinated signal system present along U.S. Highway 72, there will continue to be increased delay on the Jack Clift Boulevard approaches to U.S. Highway 72.

The capacity printouts that illustrate the results of the analyses for future traffic conditions are provided as **Attachment A**.

Conclusions

After analyzing the intersection of U.S. Highway 72 at Jack Clift Boulevard as previously outlined, it was determined that the proposed movement restrictions on the northbound and southbound through movements would yield improved levels of service, reduced delays, and reduced 95th percentile queues for all approaches. Additionally, analysis shows that the current traffic volumes at the intersection of Jack Clift Boulevard at John Henry Way do not queue back onto U.S. Highway 72.

Attachment A – Capacity Analysis

John Henry Way at Jack Clift Blvd
Saturday Peak

4-26-25 Traffic Volumes

Intersection	
Intersection Delay, s/veh	37.4
Intersection LOS	E

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations		↕			↕	↗	↖	↖	↗		↕	↗
Traffic Vol, veh/h	55	87	190	213	81	34	183	72	94	9	174	75
Future Vol, veh/h	55	87	190	213	81	34	183	72	94	9	174	75
Peak Hour Factor	0.87	0.87	0.87	0.93	0.93	0.93	0.92	0.92	0.92	0.82	0.82	0.82
Heavy Vehicles, %	2	2	2	2	2	2	2	2	2	2	2	2
Mvmt Flow	63	100	218	229	87	37	199	78	102	11	212	91
Number of Lanes	0	1	0	0	1	1	1	1	1	0	1	1

Approach	EB	WB	NB	SB
Opposing Approach	WB	EB	SB	NB
Opposing Lanes	2	1	2	3
Conflicting Approach Left	SB	NB	EB	WB
Conflicting Lanes Left	2	3	1	2
Conflicting Approach Right	NB	SB	WB	EB
Conflicting Lanes Right	3	2	2	1
HCM Control Delay, s/veh	60.8	44.2	19.7	22.8
HCM LOS	F	E	C	C

Lane	NBLn1	NBLn2	NBLn3	EBLn1	WBLn1	WBLn2	SBLn1	SBLn2
Vol Left, %	100%	0%	0%	17%	72%	0%	5%	0%
Vol Thru, %	0%	100%	0%	26%	28%	0%	95%	0%
Vol Right, %	0%	0%	100%	57%	0%	100%	0%	100%
Sign Control	Stop							
Traffic Vol by Lane	183	72	94	332	294	34	183	75
LT Vol	183	0	0	55	213	0	9	0
Through Vol	0	72	0	87	81	0	174	0
RT Vol	0	0	94	190	0	34	0	75
Lane Flow Rate	199	78	102	382	316	37	223	91
Geometry Grp	8	8	8	8	8	8	8	8
Degree of Util (X)	0.551	0.205	0.247	0.94	0.847	0.087	0.603	0.228
Departure Headway (Hd)	9.97	9.448	8.716	8.867	9.649	8.547	9.722	8.956
Convergence, Y/N	Yes							
Cap	362	380	412	412	376	419	370	401
Service Time	7.737	7.214	6.482	6.567	7.413	6.311	7.49	6.724
HCM Lane V/C Ratio	0.55	0.205	0.248	0.927	0.84	0.088	0.603	0.227
HCM Control Delay, s/veh	24.4	14.7	14.3	60.8	47.9	12.1	26.3	14.4
HCM Lane LOS	C	B	B	F	E	B	D	B
HCM 95th-tile Q	3.2	0.8	1	10.5	7.9	0.3	3.8	0.9

US-72 at Jack Cliff Blvd
Saturday Peak

4-26-25 Traffic Volumes
With NB/SB Throughs

Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	103	1083	24	297	1106	216	185	48	211	380	93	154
Future Volume (veh/h)	103	1083	24	297	1106	216	185	48	211	380	93	154
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Lane Width Adj.	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Ped-Bike Adj(A_pbT)	1.00		1.00	1.00		1.00	1.00		1.00	1.00		1.00
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870	1870
Adj Flow Rate, veh/h	108	1140	25	309	1152	0	199	52	227	388	95	0
Peak Hour Factor	0.95	0.95	0.95	0.96	0.96	0.96	0.93	0.93	0.93	0.98	0.98	0.98
Percent Heavy Veh, %	2	2	2	2	2	2	2	2	2	2	2	2
Cap, veh/h	158	1583	706	361	1792		226	44	194	452	288	
Arrive On Green	0.05	0.45	0.45	0.10	0.50	0.00	0.13	0.15	0.15	0.13	0.15	0.00
Sat Flow, veh/h	3456	3554	1585	3456	3554	1585	1781	304	1327	3456	1870	0
Grp Volume(v), veh/h	108	1140	25	309	1152	0	199	0	279	388	95	0
Grp Sat Flow(s),veh/h/ln	1728	1777	1585	1728	1777	1585	1781	0	1631	1728	1870	0
Q Serve(g_s), s	4.0	34.0	1.2	11.4	30.9	0.0	14.3	0.0	19.0	14.3	5.9	0.0
Cycle Q Clear(g_c), s	4.0	34.0	1.2	11.4	30.9	0.0	14.3	0.0	19.0	14.3	5.9	0.0
Prop In Lane	1.00		1.00	1.00		1.00	1.00		0.81	1.00		0.00
Lane Grp Cap(c), veh/h	158	1583	706	361	1792		226	0	238	452	288	
V/C Ratio(X)	0.68	0.72	0.04	0.86	0.64		0.88	0.00	1.17	0.86	0.33	
Avail Cap(c_a), veh/h	266	1583	706	399	1792		315	0	238	651	302	
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Upstream Filter(I)	1.00	1.00	1.00	1.00	1.00	0.00	1.00	0.00	1.00	1.00	1.00	0.00
Uniform Delay (d), s/veh	61.1	29.4	20.3	57.3	23.6	0.0	55.8	0.0	55.5	55.3	49.0	0.0
Incr Delay (d2), s/veh	3.8	2.9	0.1	15.0	1.8	0.0	17.1	0.0	112.0	7.0	0.5	0.0
Initial Q Delay(d3), s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
%ile BackOfQ(50%),veh/ln	1.8	14.1	0.5	5.6	12.3	0.0	7.5	0.0	15.2	6.7	2.8	0.0
Unsig. Movement Delay, s/veh												
LnGrp Delay(d), s/veh	64.9	32.3	20.4	72.3	25.4	0.0	72.8	0.0	167.5	62.3	49.5	0.0
LnGrp LOS	E	C	C	E	C		E		F	E	D	
Approach Vol, veh/h		1273			1461			478			483	
Approach Delay, s/veh		34.8			35.3			128.1			59.8	
Approach LOS		C			D			F			E	
Timer - Assigned Phs	1	2	3	4	5	6	7	8				
Phs Duration (G+Y+Rc), s	10.9	72.5	22.5	24.0	18.6	64.9	21.5	25.0				
Change Period (Y+Rc), s	5.0	7.0	5.5	5.0	5.0	7.0	5.0	5.0				
Max Green Setting (Gmax), s	10.0	54.0	24.5	19.0	15.0	49.0	23.0	21.0				
Max Q Clear Time (g_c+I1), s	6.0	32.9	16.3	21.0	13.4	36.0	16.3	7.9				
Green Ext Time (p_c), s	0.1	12.7	0.7	0.0	0.1	8.9	0.2	0.3				
Intersection Summary												
HCM 7th Control Delay, s/veh			50.4									
HCM 7th LOS			D									
Notes												
Unsignalized Delay for [WBR, SBR] is excluded from calculations of the approach delay and intersection delay.												

US-72 at Jack Cliff Blvd
Saturday Peak

4-26-25 Traffic Volumes
Without NB/SB Throughs

												
Movement	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	NBR	SBL	SBT	SBR
Lane Configurations												
Traffic Volume (veh/h)	151	1083	24	297	1106	216	185	0	211	380	0	247
Future Volume (veh/h)	151	1083	24	297	1106	216	185	0	211	380	0	247
Initial Q (Qb), veh	0	0	0	0	0	0	0	0	0	0	0	0
Lane Width Adj.	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Ped-Bike Adj(A_pbT)	1.00		1.00	1.00		1.00	1.00		1.00	1.00		1.00
Parking Bus, Adj	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Work Zone On Approach		No			No			No			No	
Adj Sat Flow, veh/h/ln	1870	1870	1870	1870	1870	1870	1870	0	1870	1870	0	1870
Adj Flow Rate, veh/h	159	1140	25	309	1152	0	199	0	0	388	0	0
Peak Hour Factor	0.95	0.95	0.95	0.96	0.96	0.96	0.93	0.93	0.93	0.98	0.98	0.98
Percent Heavy Veh, %	2	2	2	2	2	2	2	0	2	2	0	2
Cap, veh/h	216	2226	993	368	2382		236	0		458	0	
Arrive On Green	0.06	0.63	0.63	0.11	0.67	0.00	0.13	0.00	0.00	0.13	0.00	0.00
Sat Flow, veh/h	3456	3554	1585	3456	3554	1585	1781	199		3456	388	
Grp Volume(v), veh/h	159	1140	25	309	1152	0	199	61.1		388	58.4	
Grp Sat Flow(s),veh/h/ln	1728	1777	1585	1728	1777	1585	1781	E		1728	E	
Q Serve(g_s), s	5.9	22.9	0.8	11.4	20.6	0.0	14.2			14.3		
Cycle Q Clear(g_c), s	5.9	22.9	0.8	11.4	20.6	0.0	14.2			14.3		
Prop In Lane	1.00		1.00	1.00		1.00	1.00			1.00		
Lane Grp Cap(c), veh/h	216	2226	993	368	2382		236			458		
V/C Ratio(X)	0.74	0.51	0.03	0.84	0.48		0.84			0.85		
Avail Cap(c_a), veh/h	532	2226	993	532	2382		473			917		
HCM Platoon Ratio	1.00	1.00	1.00	1.00	1.00	1.00	1.00			1.00		
Upstream Filter(I)	1.00	1.00	1.00	1.00	1.00	0.00	1.00			1.00		
Uniform Delay (d), s/veh	59.9	13.4	9.2	57.0	10.4	0.0	55.1			55.1		
Incr Delay (d2), s/veh	3.7	0.8	0.0	7.0	0.7	0.0	6.0			3.3		
Initial Q Delay(d3), s/veh	0.0	0.0	0.0	0.0	0.0	0.0	0.0			0.0		
%ile BackOfQ(50%),veh/ln	2.6	8.2	0.3	5.1	7.0	0.0	6.8			6.4		
Unsig. Movement Delay, s/veh												
LnGrp Delay(d), s/veh	63.6	14.2	9.3	64.0	11.2	0.0	61.1			58.4		
LnGrp LOS	E	B	A	E	B		E			E		
Approach Vol, veh/h		1324			1461							
Approach Delay, s/veh		20.0			22.3							
Approach LOS		C			C							
Timer - Assigned Phs	1	2	3		5	6	7					
Phs Duration (G+Y+Rc), s	13.1	94.1	22.7		18.8	88.4	22.7					
Change Period (Y+Rc), s	5.0	7.0	5.5		5.0	7.0	5.5					
Max Green Setting (Gmax), s	20.0	58.0	34.5		20.0	58.0	34.5					
Max Q Clear Time (g_c+I1), s	7.9	22.6	16.3		13.4	24.9	16.2					
Green Ext Time (p_c), s	0.3	17.3	1.0		0.4	16.7	0.4					
Intersection Summary												
HCM 7th Control Delay, s/veh			27.9									
HCM 7th LOS			C									
Notes												
Unsignalized Delay for [NBR, WBR, SBR] is excluded from calculations of the approach delay and intersection delay.												

US-72 at Jack Clift Blvd
Saturday Peak

4-26-25 Traffic Volumes
With NB/SB Throughs

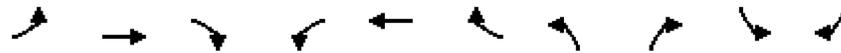
										
Lane Group	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBT	SBL	SBT
Lane Group Flow (vph)	108	1140	25	309	1152	225	199	279	388	252
v/c Ratio	0.47	0.75	0.03	0.79	0.69	0.26	0.78	0.81	0.76	0.86
Control Delay (s/veh)	65.1	36.9	0.1	71.3	30.8	4.6	74.2	44.4	62.7	68.9
Queue Delay	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total Delay (s/veh)	65.1	36.9	0.1	71.3	30.8	4.6	74.2	44.4	62.7	68.9
Queue Length 50th (ft)	45	450	0	130	408	6	163	112	163	163
Queue Length 95th (ft)	76	556	0	#195	526	57	242	#243	210	#293
Internal Link Dist (ft)		481			579			280		266
Turn Bay Length (ft)	275		250	250		275			165	
Base Capacity (vph)	264	1513	735	405	1681	862	313	371	646	321
Starvation Cap Reductn	0	0	0	0	0	0	0	0	0	0
Spillback Cap Reductn	0	0	0	0	0	0	0	0	0	0
Storage Cap Reductn	0	0	0	0	0	0	0	0	0	0
Reduced v/c Ratio	0.41	0.75	0.03	0.76	0.69	0.26	0.64	0.75	0.60	0.79

Intersection Summary

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

US-72 at Jack Cliff Blvd
Saturday Peak

4-26-25 Traffic Volumes
Without NB/SB Throughs



Lane Group	EBL	EBT	EBR	WBL	WBT	WBR	NBL	NBR	SBL	SBR
Lane Group Flow (vph)	159	1140	25	309	1152	225	199	227	388	252
v/c Ratio	0.55	0.55	0.03	0.71	0.52	0.21	0.73	0.53	0.73	0.16
Control Delay (s/veh)	64.3	19.1	0.0	63.7	15.4	2.2	67.3	11.9	60.4	0.2
Queue Delay	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Total Delay (s/veh)	64.3	19.1	0.0	63.7	15.4	2.2	67.3	11.9	60.4	0.2
Queue Length 50th (ft)	67	292	0	130	262	1	163	9	163	0
Queue Length 95th (ft)	102	435	0	174	391	37	234	79	205	0
Internal Link Dist (ft)		481			579					
Turn Bay Length (ft)	275		250	250		275		100	165	
Base Capacity (vph)	528	2066	950	532	2218	1075	469	578	911	1583
Starvation Cap Reductn	0	0	0	0	0	0	0	0	0	0
Spillback Cap Reductn	0	0	0	0	0	0	0	0	0	0
Storage Cap Reductn	0	0	0	0	0	0	0	0	0	0
Reduced v/c Ratio	0.30	0.55	0.03	0.58	0.52	0.21	0.42	0.39	0.43	0.16

Intersection Summary