US 12 MADISON BELTLINE YAHARA BRIDGE WORK ZONE CAPACITY

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INTRODUCTION

The intent of this study is to summarize the work zone capacity estimation for a work zone located in Wisconsin. This report analyzes a specific work zone to both define the capacity of the location and help redefine the parameters that affect the work zone capacity calculation procedure found in the Wisconsin Department of Transportation's (WisDOT) *Facilities Development Manual* (FDM).

Work Zone Safety and Mobility Rule

In 2004, the FHWA disseminated the Work Zone Safety and Mobility Rule to update "Traffic Safety in Highway and Street Work Zones" (23 CRF 630 Subpart J) (1). All state and local governments receiving federal-aid highway funding were required to adhere to the rule as of October 12, 2007. The purpose of the update was to address the country's changing highway travel environment, which features more traffic, more congestion, greater safety issues, and more work zones.

WisDOT Facilities Development Manual

WisDOT's FDM establishes procedures for implementing the FHWA Work Zone Safety and Mobility Rule (2). The FDM Work Zone Policy Statement indicates work zone delays less than 15 minutes are considered minimal and acceptable. Projects with expected delays exceeding 15 minutes need to incorporate additional traffic control strategies to minimize potential delays. For freeway and expressway projects, this delay calculation can be challenging since the work zone capacity varies due to a number of factors. Currently, the equation used to calculate work zone capacity is based off Equation 22-2 of the 2000 HCM (3) with an additional adjustment factor for the shoulder width. The equation is:

$$C_a = (1600 + I - R) f_{HV} f_{LS} N$$

Where,

C_a = adjusted mainline capacity (vehicles per hour [veh/h]);
I = adjustment factor for type, intensity, and location of the work activity (ranges from –160 to +160 passenger cars per lane per hour [pc/l/hr]);
R = adjustment for ramps;
f_{HV} = adjustment for heavy vehicles as defined in 2000 HCM Equation 22-1;
f_{LS} = adjustment for lane/shoulder widths; and
N = number of lanes open through the short-term work zone.

** Assumes starting capacity of 1,600 veh/h. Use 1,550 or 1,750 where appropriate.*

2010 Highway Capacity Manual

The HCM is the primary document used to calculate roadway capacity and to establish a level of service (LOS) (4). WisDOT desired to review and update the FDM procedure when the 2010 HCM was released; however, the 2010 HCM did not include additional details on how to determine capacity in a long term work zone. The 2010 HCM reported wide variances among the long-term work zone capacity studies completed. With the change in the 2010 HCM, WisDOT still desired to review their current procedure for determining work zone capacity. This study is one in a series intended to determine the capacity of work zones based on the factors included in the FDM. Additional consideration is given to other factors that may affect the capacity of a work zone.

PROJECT SUMMARY

This report summarizes the work zone data collection for WisDOT Project I.D. 1203-04-61, Sauk City – Cambridge, US 12, Dane County. US 12/18 is a 6-lane freeway that loops around the south side of Madison, WI. A location map is shown in **Figure 1**. The project extended 0.49 miles and included bridge rehabilitation. The project was constructed from May 6, 2013 to August 16, 2013. The existing facility is a six lane divided freeway with 12-foot lanes and 6-foot left shoulders and 8-foot right shoulders. The speed limit throughout the project location is 55 mph. The 2011 AADT was 118,500 vpd with 7 percent trucks and a directional traffic split of 51 percent/49 percent.

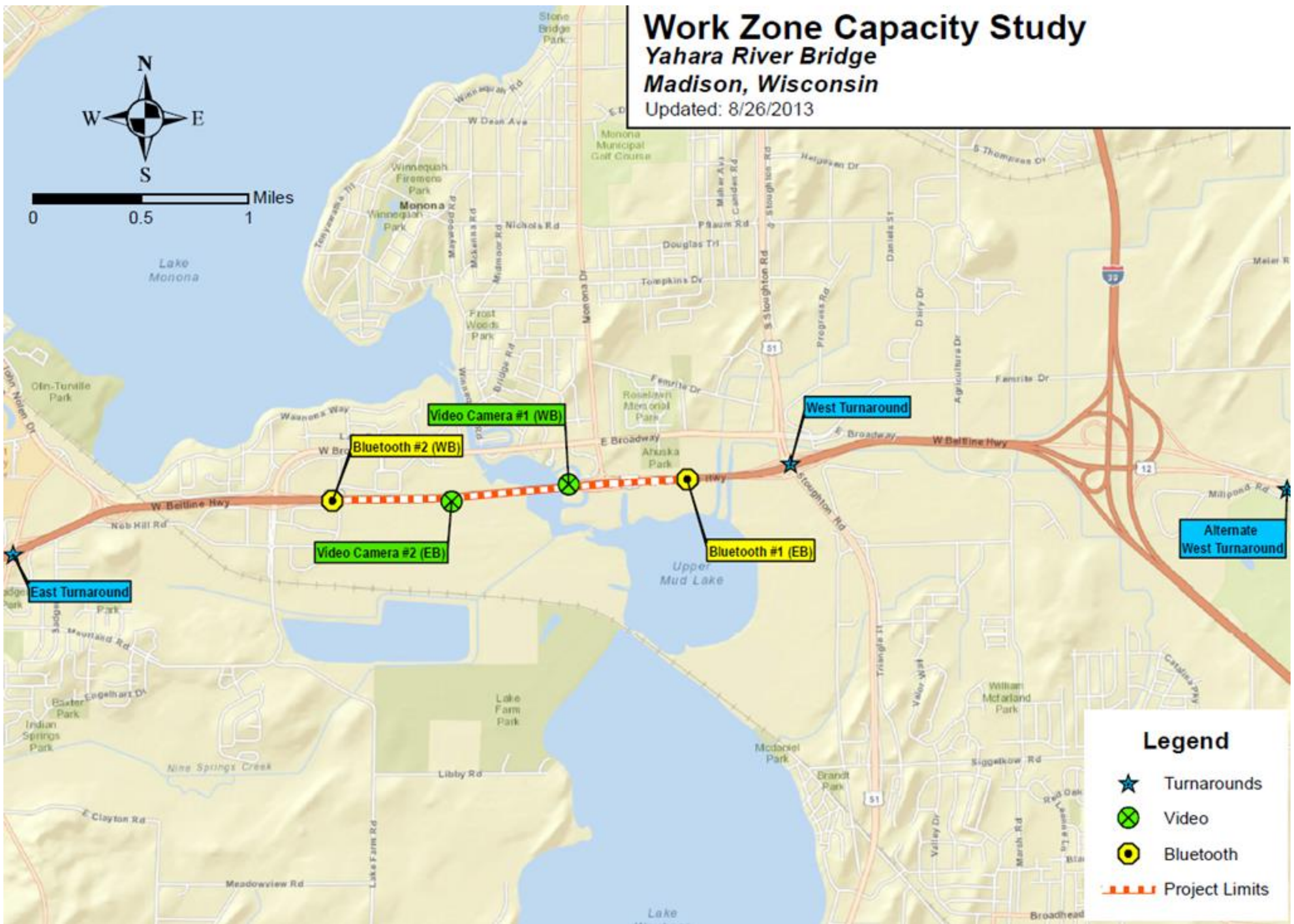

FIGURE 1 Project Location Map.

Traffic Control Staging

The Yahara River bridge project utilized a split contraflow lane method to maintain three lanes of traffic in each direction. Traffic utilized 11-foot lanes, with 1-foot shy distance to the concrete barriers on each shoulder. The temporary concrete barriers included a protective screen to shield construction workers from roadway debris and to obstruct the view of passing drivers. Various entrance and exit ramps near the project site were closed during construction. Traffic diversion to a parallel route, Broadway Avenue, was also expected due to the ramp closures and the expected mainline capacity reduction.

TMP Conclusions

The TMP used the methodology in the 2000 HCM for short term work zones to determine the capacity. The base capacity was assumed to be 1,600 passenger cars per hour per lane (pcphpl). The adjustment for heavy vehicles was based on a passenger car equivalent factor of 1.77 and 7 percent heavy vehicles. The resulting heavy vehicle adjustment factor was calculated as 0.95. No capacity adjustments were used for the presence of ramps in proximity to the work zone or the intensity of work activity. A 3 percent reduction was estimated to account for the 11-foot wide travel lanes. No shoulder width adjustment factor was applied. The resulting estimated 3-lane short-term work zone capacity was 1,475 vphpl for a total of 4,425 vph. The TMP concluded there would be less than two miles of queuing that would last 1 to 2 hours during both the AM and PM peak hours.

METHODOLOGY

The goal was to determine the capacity of the Yahara River bridge work zone. Data was collected from Monday, July 15, 2013 to Wednesday, July 31, 2013 during stages 3B and 3A. In these stages, WB traffic was shifted north onto the shoulder, and EB traffic became split with a contraflow configuration. The vehicles in the median EB lane crossed over to the WB side and were separated from the WB traffic with temporary concrete barrier. The two remaining EB lanes were kept on the EB side, but shifted to the outside in Stage 3B and to the inside in Stage 3A. The dual lane EB traffic was separated from the work zone with temporary concrete barrier with a protective screen. See **Figure 2** for staging typical sections. All lanes were 11-feet wide, with a 1-foot shy distance to the concrete barrier. For Stage 3A, the EB entrance ramp from South Towne Drive, the EB exit ramp to Monona Drive, and the WB entrance ramp from Monona Drive were closed.

Data was collected for both EB and WB traffic using Bluetooth sensors for travel time and using video cameras for volume data. Bluetooth data was collected 24/7, and video data was collected from 6 AM to 8 PM every day. Field travel time runs were completed to validate the Bluetooth data and to conduct work zone observations. In addition, WisDOT collects volume, speed, and occupancy data at selected locations throughout the state, which is retrievable from the Volume, Speed, and Occupancy (V-SPOC) data base. V-SPOC data was retrieved for the locations nearest this work zone and compared to the collected volume and speed data.

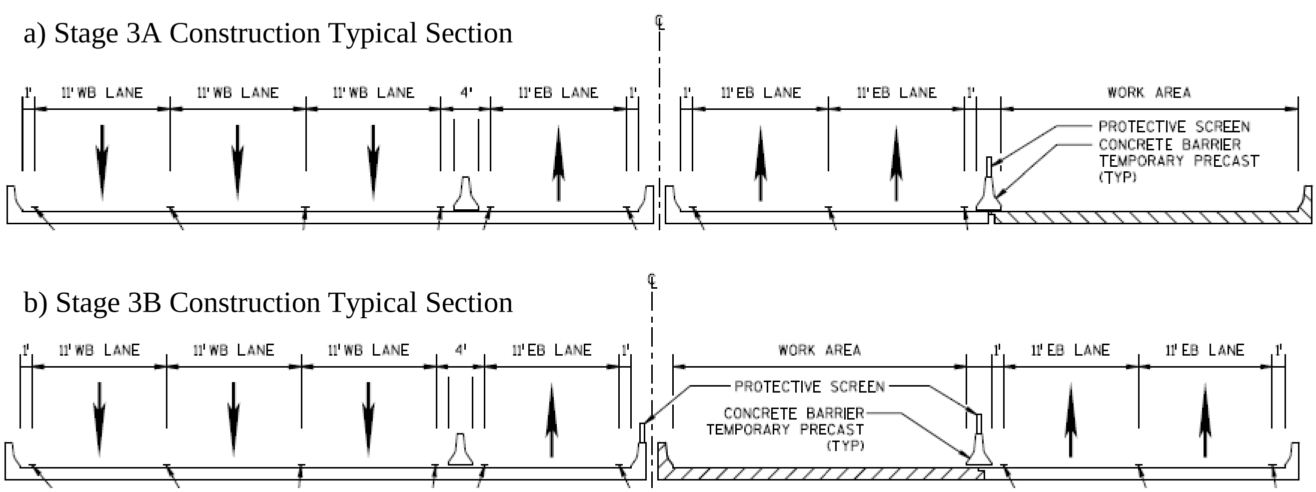

FIGURE 2 Construction Typical Sections.

Bluetooth

Bluetooth sensors were placed outside the construction zone in the median to determine the travel time of vehicles traveling through the work zone. The travel time data was used to look for time periods of congestion, which were characterized by increased travel times. The east Bluetooth sensor location was 2,100 feet east of the Monona Drive overpass. The west Bluetooth sensor location was 1,400 feet east of the South Towne Drive overpass. The distance between the detectors was 1.65 miles.

The Bluetooth detectors included a GPS sensor, plus a battery for 15 continuous days of recording. A software program was provided with the equipment to analyze the data collected by comparing MAC addresses at the two sites and match them given criteria provided, such as the minimum and maximum travel time. The data was downloaded into an Excel file, and each day was evaluated to remove any outliers. Outliers were most likely from construction vehicles or any vehicle that made multiple trips through the project area within a single period.

Bluetooth data was collected from Monday, July 15, 2013 to Wednesday, July 31, 2013. The 15-minute Bluetooth sample size was compared to the actual 15-minute volume, to determine the Bluetooth detection rates. The statistics are shown in **Table 1.**

TABLE 1 Bluetooth Detection Rates.

	WB	EB	EB Adjusted*
Average Detection Rate	3.8 percent	2.9 percent	2.9 percent
Standard Deviation	1.1 percent	1.2 percent	0.8 percent
Minimum	1.2 percent	0.7 percent	0.7 percent
Maximum	6.2 percent	11.3 percent*	5.0 percent
* *The highest detection rate occurred on July 25 at noon, when the two inside EB lanes were closed due to an incident. The EB Adjusted column excluded the outliers created from this incident*.			

Video

Weather-proof video cameras were placed in the construction zone on the outside shoulders to collect volume data. Video data was collected from 6 AM to 8 PM every day from Tuesday, July 16, 2013 thru Wednesday, July 31, 2013. The time periods selected for video processing were determined by looking at the travel time and calculated speed data. Two full days of data were processed to see how the volumes changed throughout the day. In addition, data from the AM and PM peak hours was processed.

Video was processed off site, and volumes were provided for WB and EB traffic. For WB traffic, the volume data that was provided grouped the three lanes together, and for EB traffic, lane by lane volumes were provided to determine volume differences between the single and dual lanes. Volumes were provided for cars, medium trucks, and heavy trucks. After receiving the traffic volumes, total traffic and vehicle class percentages were calculated.

WisDOT Data

V-SPOC data was retrieved for 5-minute and 15-minute intervals from July 15 to July 31 at two nearby detection locations: the Stoughton Road interchange (approximately 1.2 miles east of the project), and the John Nolen Drive interchange (about 1.6 miles west of the project).

WisDOT recorded video at three locations; two cameras were permanent surveillance installations (at Monona Drive and Stoughton Road), while the third camera (at South Towne Drive) was installed temporarily for this construction project. Video from the cameras was used to observe the overall work zone operations and to confirm any incidents that occurred.

Data Validation

Bluetooth travel time data was validated using sample physical travel time runs and V-SPOC data. When completing travel time runs, Bluetooth-enabled devices in the drivers' personal vehicles were set to discoverable mode so that the device's MAC address could be captured by the Bluetooth detectors. The travel time runs were completed in laps. For each route, the vehicle's travel lane, begin time, and elapsed travel time were recorded at the Bluetooth sensors. The travel time data was entered into an Excel file, and the travel speed, based on a distance of 1.7 miles, was calculated.

Since the MAC addresses of the drivers' devices were known, the MAC addresses were found in the Bluetooth travel time database to compare the field measured travel time to the Bluetooth measured travel time of the exact same trips. Overall, the difference between the measured travel time and the Bluetooth travel time was typically within 0.065 minutes, or 3.9 seconds.

There appeared to be a slight tendency to obtain more data from vehicles traveling in the inside travel lanes. In total, 27 of the 38 travel time runs were picked up by the Bluetooth sensors. For the inside lane, the travel time was recorded 79 percent of the time, for the middle lane, it was recorded 75 percent of the time, and for the outside lane, it was recorded 58 percent of the time. With the Bluetooth detectors located in the median, there was a slight tendency to obtain more data from the inside lanes.

The calculated speed was also compared to the V-SPOC speed data. **Figure 3** shows speed comparisons for EB and WB traffic. Both figures show the Bluetooth data is comparable to the data retrieved from V-SPOC. The largest discrepancies in speed occurred when traffic became congested and speeds decreased.

a) WB Speed Data b) EB Speed Data

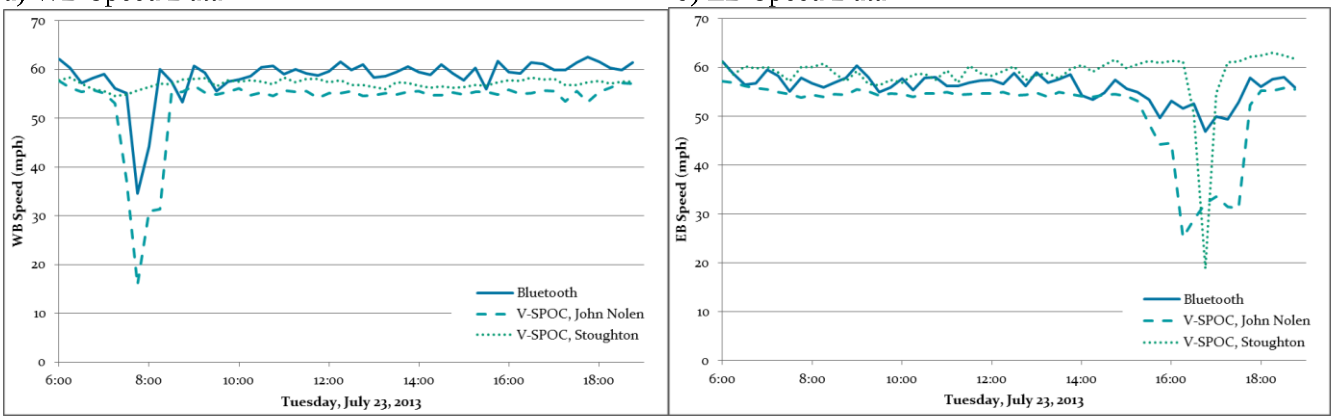

FIGURE 3 Comparison of Bluetooth and V-SPOC Speed Data.

Video camera volume data was validated by manually counting traffic for 15-minute intervals on the video captured for July 25. The video processing company claims an accuracy rate of 95 percent, which was verified from the hand counts shown. For the six 15-minute time periods manually counted, the largest percent error was 1.33 percent.

RESULTS

The speed data and volume data were combined in three data sets: (1) EB 1-lane, (2) EB 2-lane, and (3) WB 3-lane. For each data set, the following were included: 1) Date and time (in 15-minute intervals); 2) 15-minute volume; 3) Medium, heavy, and total truck percent; 4) Average speed; and 5) Bluetooth sample size (15-minute intervals). From speed and volume data, the flow rate, in both vphpl and pcphpl, and density were calculated. The maximum flow rates (capacity) for each data set were identified, speed-flow-density curves were plotted, and other general observations were made.

Capacity

The capacity of each lane configuration was determined by looking at the maximum 15-minute flow rate, the top 10 flow rates, and the top 20 flow rates. The results are shown in **Table 2,** and a sample of 15-minute data is shown in **Table 3**. Since the EB 1-lane and EB 2-lane reached their respective top 20 flow rates in different time periods, the EB Combined category is not an average of the EB 1-lane and EB 2-lane groups. In **Table 3**, the density values shaded gray represent a LOS E (density > 35 pcpmpl) and the non-shaded values represent LOS D. The peak flow rates occurred during time intervals within the AM and PM peak hours; however, it cannot be guaranteed that capacity was reached. For example, for WB traffic, peak flow rates occurred on July 24, 25, and 29 from 6:30 AM to 7:30 AM. Since the flow rate was maintained over a sustained time period, it suggests that the values in **Table 2** do represent capacity. In addition, the density varied from 29 vph per mile to 39 vph per mile, which represents a roadway with a LOS of D or E (4).

TABLE 2 Work Zone Capacity.

Lane Configuration	Maximum			Average of top 10			Average of top 20		
	Flow (vphpl)	Truck Percent	Speed (mph)	Flow (vphpl)	Truck Percent	Speed (mph)	Flow (vphpl)	Truck Percent	Speed (mph)
EB 2-lanes	1838	4.68%	50	1775	5.44%	52	1747	5.30%	53
EB 1-lane	2098	0.39%	58	1925	1.38%	53	1887	1.24%	53
EB Combined	1867	3.43%	53	1812	4.05%	53	1782	3.79%	53
WB 3-lanes	1984	5.31%	56	1905	5.68%	58	1853	5.73%	59

TABLE 3 Sample Data.

	Rank	Date	Time	15-min Volume (vph)	Total Truck %	Average Speed (mph)	Flow (vph)	Flow (vphpl)	Flow (pcphpl)	Density (pc per mile per lane)
Westbound 3-Lane	1	7/24	7:15	1440	5.83%	53.2	5760	1920	1976	37.1
	2	7/30	7:15	1488	5.31%	56.4	5952	1984	2037	36.1
	3	7/23	7:15	1455	5.64%	56.1	5820	1940	1995	35.6
	4	7/24	7:00	1426	6.45%	57.6	5704	1901	1963	34.1
	5	7/23	7:30	1354	5.76%	55.1	5416	1805	1857	33.7
	6	7/25	7:15	1446	5.05%	59.3	5784	1928	1977	33.3
	7	7/29	7:15	1452	4.55%	59.4	5808	1936	1980	33.3
	8	7/30	6:30	1385	6.06%	58.1	5540	1847	1903	32.7
	9	7/23	7:00	1405	5.55%	59.0	5620	1873	1925	32.6
	10	7/29	7:30	1317	6.45%	56.2	5268	1756	1813	32.3
Eastbound 3-Lane	1	7/29	17:15	1380	3.26%	49.9	5520	1840	1870	37.5
	2	7/29	16:15	1372	4.81%	51.3	5488	1829	1873	36.5
	3	7/26	17:00	1311	4.27%	49.1	5244	1748	1785	36.4
	4	7/29	17:00	1323	3.40%	49.9	5292	1764	1794	36.0
	5	7/24	16:30	1400	3.43%	53.2	5600	1867	1899	35.7
	6	7/30	16:30	1322	3.93%	50.5	5288	1763	1797	35.6
	7	7/29	16:30	1360	4.41%	52.6	5440	1813	1853	35.2
	8	7/24	16:45	1317	2.51%	50.6	5268	1756	1778	35.1
	9	7/24	16:15	1348	3.93%	52.2	5392	1797	1833	35.1
	10	7/23	16:30	1352	3.62%	52.6	5408	1803	1835	34.9
Eastbound 1-Lane	1	7/23	16:45	456	0.88%	46.9	1824	1824	1832	39.1
	2	7/26	17:00	465	0.65%	49.1	1860	1860	1866	38.0
	3	7/29	16:15	481	2.29%	51.3	1924	1924	1946	37.9
	4	7/30	16:30	472	2.54%	50.5	1888	1888	1912	37.9
	5	7/29	17:00	468	0.64%	49.9	1872	1872	1878	37.7
	6	7/24	16:15	484	1.86%	52.2	1936	1936	1954	37.4
	7	7/23	16:15	480	1.04%	51.7	1920	1920	1930	37.3
	8	7/29	17:15	461	0.43%	49.9	1844	1844	1848	37.0
	9	7/24	16:45	466	0.86%	50.6	1864	1864	1872	37.0
	10	7/24	16:30	485	1.65%	53.2	1940	1940	1956	36.8
Eastbound 2-Lane	1	7/29	17:15	919	4.68%	49.9	3676	1838	1881	37.7
	2	7/23	17:15	886	4.63%	49.3	3544	1772	1813	36.7
	3	7/29	16:15	891	6.17%	51.3	3564	1782	1837	35.8
	4	7/24	16:30	915	4.37%	53.2	3660	1830	1870	35.2
	5	7/29	17:00	855	4.91%	49.9	3420	1710	1752	35.1
	6	7/24	15:45	869	6.56%	51.4	3476	1738	1795	35.0
	7	7/29	16:30	893	5.82%	52.6	3572	1786	1838	34.9
	8	7/24	17:15	851	4.23%	50.0	3404	1702	1738	34.7
	9	7/24	16:45	851	3.41%	50.6	3404	1702	1731	34.2
	10	7/23	16:00	884	5.32%	53.2	3536	1768	1815	34.1

As shown, the EB single lane obtained the highest flow rate, and the EB dual lane obtained the lowest flow rate. This is most likely explained by no impacts from lane changes and no passing available in the EB single lane; furthermore, trucks tended to avoid the single lane, probably due to the reduced width of 13-feet between concrete barriers.

Speed, Flow, and Density Curves

Speed, flow, and density curves were plotted for the three lane configurations in this work zone, as shown in **Figure 4**. Although this work zone was expected to be over capacity frequently, there were very few time periods that experienced substantial delays. In turn, during the oversaturated time periods, there were few data points in the oversaturated portions of **Figure 4**. For the EB dual lane, the flow rate observed was typically between 1,000 vphpl and 1,800 vphpl. For the WB 3-lane configuration the flow rate observed was typically between 1,000 vphpl and 2,000 vphpl. The WB 3-lane configuration tended to have higher speeds than the EB configuration, as shown in **Figure 4**.

On the speed-flow curve, the EB 1-lane configuration had the both lowest and highest flow rates. This is further demonstrated in **Figure 5**, which compares flow rates of the EB lanes. The EB single lane was utilized less during non-peak hours, creating much lower flow rates. However, when volumes began to increase, volume in the single lane increased until the single lane had flow rates higher than the dual EB lanes.

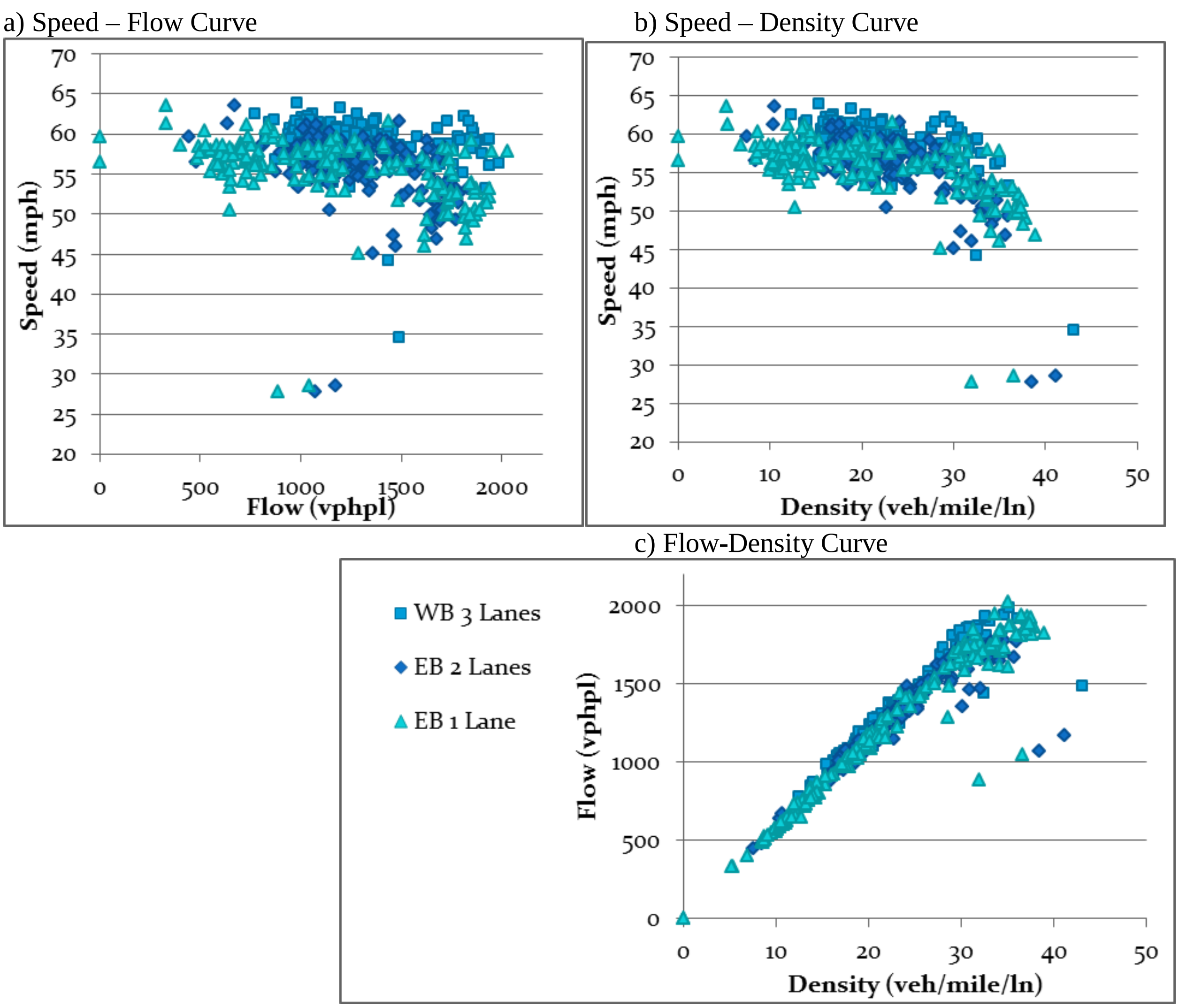

FIGURE 4 Speed-Flow-Density Curves.

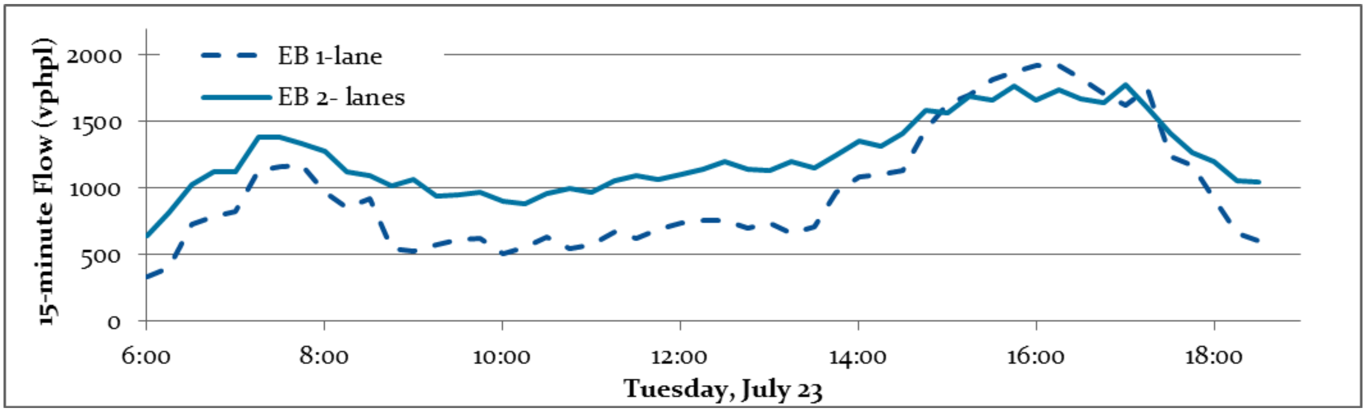

FIGURE 5 EB Flow Rates over Time.

Truck Percentage near Capacity

The truck percentage for each 15 minute period was also plotted against the flow rate, as shown in **Figure 6**. The truck percentage used for pavement design and the f_{HV} factor was 7 percent. As shown, when more vehicles used the facility, the percentage of trucks on the roadway decreased. This suggests operators of heavy vehicles avoided traveling through the work zone during peak times. In addition, heavy vehicles tended to avoid the single EB lane. The design truck percentage was slightly higher than the actual truck percentages observed during the time periods near capacity.

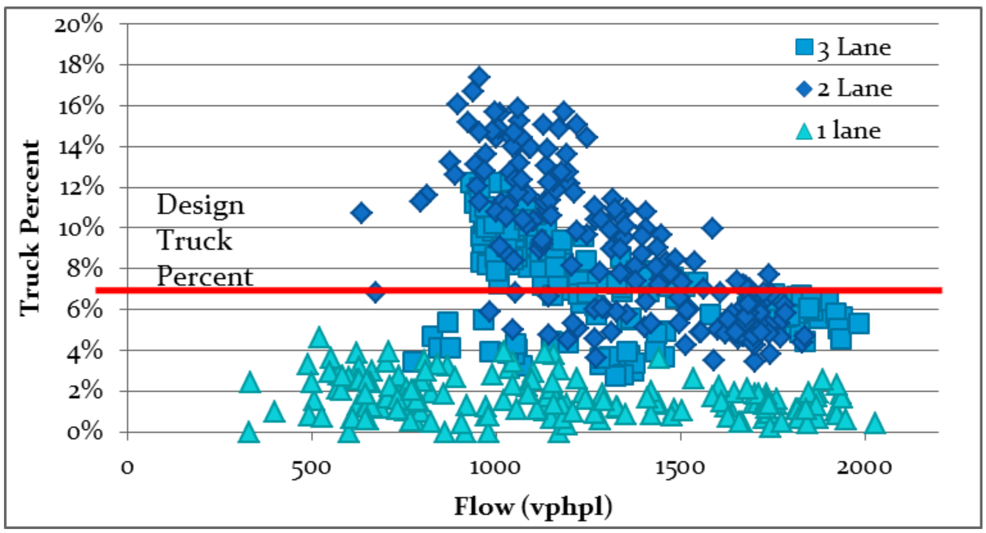

FIGURE 6 Truck Percent Observed.

CONCLUSIONS

This research shows the FDM work zone capacity policy should be updated as the observed flow rates near capacity were greater than the expected capacity calculated in the TMP. Potential reasons for the additional capacity observed could be: (1) an urban area with aggressive driving, (2) length of the work zone, and (3) entrance and exit ramps closures. Using the current FDM work zone capacity equation, it is unlikely to predict capacities as high as those that were actually observed.

For future work, 1,900 vphpl is recommended to be the capacity for a 3-lane configuration with reduced lane and shoulder width. For a split flow configuration, an additional reduction of 100 vphpl is suggested, producing a total capacity of 1,800 vphpl.

REFERENCES

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