Bismarck Mandan 2021 TRAVEL DEMAND MODEL UPDATE

REPORT

To the Bismarck Mandan MPO

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1. INTRODUCTION

The Bismarck Mandan Metropolitan Planning Organization (B-M MPO) Travel Demand Model (TDM) undergoes a comprehensive update every five years to incorporate the latest ground truths/data and advancements in transportation modeling techniques. The latest update incorporates data from the base year 2021 and partially from 2022, necessitated by the unique traffic patterns resulting from the COVID-19 pandemic, which rendered the 2020 data unreliable for traffic forecasting. The model adheres to a four-step TDM framework, comprising trip generation, trip distribution, modal split, and trip assignment. The update procedure includes meticulous calibration of the model's input parameters and validation against actual data. This calibration is iterative, as depicted in Figure 1.

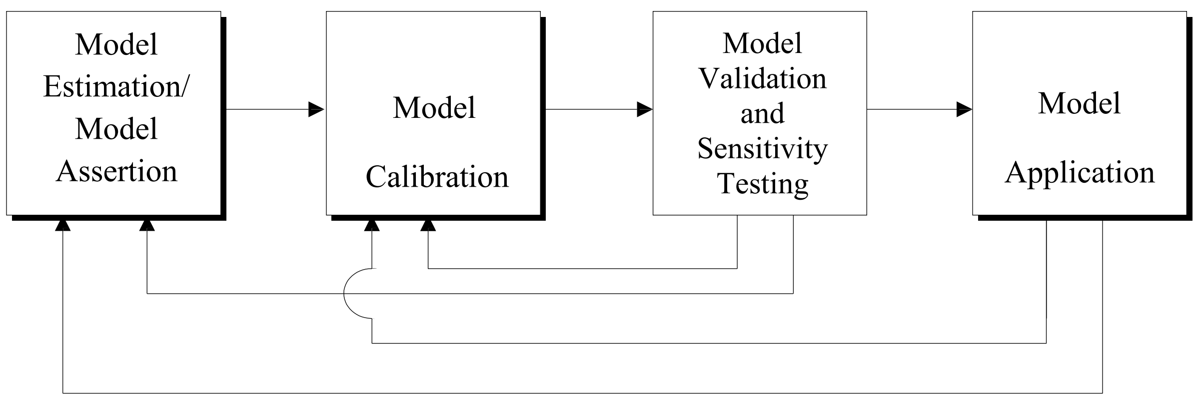

Figure 1 B-M TDM Calibration Flow Chart

The rest of this document describes the model update process including the data, methods, and models that were used to update the model. Chapter 2 discusses the improvements made to the 2021 TDM; Chapter 3 discusses the capacity calculation methodology; Chapter 4 discusses the input data used in the model; Chapter 5 summarizes the trip generation models and methods; Chapter 6 discusses the trip distribution step; Chapter 7 discusses the trip assignment step; Chapter 8 discusses the model calibration, validation, and output.

2. IMPROVEMENTS TO THE 2021 TDM

The 2021 base year model received numerous updates to incorporate the latest data and methodologies, along with enhancements in modeling software and the integration of long-haul freight movements. The updates for the 2021 model leveraged several new data sources:

1. Origin Destination Data obtained from Streetlight, which offers detailed insights into travel patterns.

2. Traffic analysis tool data, providing comprehensive traffic flow and usage statistics.

3. Traffic Analysis Zone (TAZ) and socioeconomic data supplied by the MPO, are crucial for understanding demographic and geographic influences on travel demand.

4. Traffic count data is sourced from the NDDOT website's interactive map, offering real-time and historical traffic volume information.

These updates ensure that the model remains cutting-edge, reflecting the latest trends and data in transportation modeling.

2.1.Origin Destination Data Obtained from Streetlight

Origin-destination (OD) data, is a critical component for TDMs, and this data was obtained from Streetlight Data. Streetlight uses anonymized cellular data from millions of individuals nationwide to craft comprehensive mobility patterns for road users. Their online platform offers a suite of analytical tools, enabling users to generate estimates on Annual Average Daily Traffic (AADT), Vehicle Miles Traveled (VMT), turning movement counts, OD data, trip speed, demographic insights, travel modes, trip attributes (such as travel time and length), and the proportion of trip purposes (Home-Based Work [HBW], Non-Home-Based [NHB], and Home-Based Other [HBO]) for varying time frames from 15 minutes up to a year. Several Datasets were obtained from Streetlight's platform and used to calibrate the model including:

1. Three matrices each for weekdays, weekends, and all days were estimated showing OD trips separately for HBW, HBO, and NHB purposes.
2. Hourly OD trips were estimated for each of the HBW, HBO, and NHB purposes, which were further divided into 15-minute time spans to identify the peak hour.
3. Daily trips were divided into four time periods to differentiate between peak-hour trips and off-peak-hours trips. 7 AM to 8 AM was selected as AM peak hours, 3 PM to 6 PM was selected as PM peak hours, a time span between 9 AM and 3 PM was selected as AM off-peak hours and the time between 6 PM and 7 AM is selected as PM off-peak hours.
4. Trips were estimated for the months of March 2022 and April 2022, this is because all the traffic data used for calibration and comparison of TDM was also from 2022.
5. The data was estimated separately for private vehicles and trucks. Further, long-distance OD trips were also estimated to reflect internal-external, external-internal, and external-external trips. These trips were processed separately for HBW, HBO, and NHB in the case of private vehicles, and in case of fright traffic, trips were analyzed as NHB trips.

2.1.1. Internal-Internal OD Trip Summary

Table 1 shows the distribution of trip purposes by time of day within the Bismarck Mandan Metropolitan Planning Organization (B-M MPO) Traffic Analysis Zones (TAZs) as obtained from Streetlight. It details Home-Based Work (HBW), Home-Based Other (HBO), and Non-Home-Based (NHB) trips during different time segments: AM Peak, AM Off-Peak, PM Peak, and Night.

For HBW trips, the distribution is as follows: 19.57% in the AM Peak, 27.15% during the AM Off-Peak, 29.39% in the PM Peak, and 23.88% during Night. HBO trips show a significant skew towards Night trips (38.35%), with descending proportions in PM Peak (28.76%), AM Off-Peak (28.39%), and AM Peak (4.49%). This pattern aligns with typical non-work-related travel behaviors, where fewer trips originate in the morning peak. NHB trips are predominantly during the AM Off-Peak (45.75%), with subsequent frequencies in the PM Peak (30.36%), Night (20.52%), and AM Peak (3.36%). The "% of Overall" column indicates the percentage of trips within the B-M MPO area, segmented by purpose: 25% for HBW, 38% for HBO, and 37% for NHB, reflecting their share of total internal-internal trips.

Table 1 Summary of Internal-Internal OD Data from Streetlight Analysis

Bismarck-Mandan MPO TAZ OD Trips

Purpose	7-8 AM	8 AM-3 PM	3-6 PM	Night	Total	% of Overall
HBW	29,631	41,096	44,493	36,166	151,386	24.76%
HBO	10,558	66,617	67,511	89,989	234,675	38.39%
NHB	7,569	103,018	68,371	46,206	225,164	36.83%
Total	47,758	210,731	180,375	172,361	611,225	100%

Proportions by Trip Purpose and Time of Day, B-M MPO TAZ Only

Purpose	7-8 AM	8 AM-3 PM	3-6 PM	Night	Total	% of Overall
HBW	19.57%	27.15%	29.39%	23.88%	100%	25%
HBO	4.49%	28.39%	28.76%	38.35%	100%	38%
NHB	3.36%	45.75%	30.36%	20.52%	100%	37%

NCHRP 718 Time-of-day Distributions by Purpose

Purpose	7-8 AM	8 AM-3 PM	3-6 PM	Night	Total
HBW	14.4%	25%	28%	32.6%	100%
HBO	5.5%	36.2%	24.5%	28.8%	100%
NHB	4.9%	53.9%	25.00%	16.2%	100%

2.1.2. Internal-External/External-Internal Origin-Destination Data

Table 2 shows details of the External-Internal (EI) and Internal-External (IE) trip data for the Bismarck-Mandan MPO area, highlighting the proportions of these trips concerning the total trips for each purpose (HBW, HBO, NHB) and time period. These figures offer insight into the through-trip travel patterns within the study area.

2.1.2.1. Analysis of EI Trips:

- **HBW Trips**: EI trips account for 17.9% of total HBW trips, with the highest proportion (5.5%) during the 7-8 AM time slot, which aligns with typical morning commute hours.

- **HBO Trips**: EI trips represent a significant 35.9% of HBO trips, peaking during the 8 AM to 3 PM period (14.5%), possibly reflecting errands or other non-work-related activities.

- **NHB Trips**: For NHB trips, EI trips are the most prevalent, comprising 46.2% of total NHB trips, with the highest proportion (24.3%) from 8 AM to 3 PM, indicating substantial midday travel.

The overall data show that EI trips are most common in the midday hours (8 AM to 3 PM), suggesting a significant amount of travel into the MPO area during these hours, likely for various purposes including work, errands, and other activities.

2.1.2.2. Analysis of IE Trips:

- **HBW Trips**: IE trips make up 13.2% of HBW trips, with the largest share (4.3%) in the 3-6 PM window, aligning with typical evening commuting times.

- **HBO Trips**: HBO trips see the largest IE proportion at 40.1%, with the highest percentage (15.8%) during nighttime, which might indicate returning home after various personal activities.

- **NHB Trips**: Similar to EI, NHB trips have a high IE trip proportion (46.7%), with a significant amount during the 7 AM to 3 PM period (12.6%), suggesting outward travel from the MPO area for various non-home-based activities.

The data indicates that IE trips are more evenly distributed throughout the day, with a notable number of trips occurring in the morning and early afternoon hours.

2.1.2.3. Conclusions on IE-EI Trips

- The high percentage of NHB trips in both EI and IE categories suggests a dynamic interplay of travel purposes that extend beyond typical work commutes, likely influenced by the area's economic, social, and geographical characteristics.

- The substantial percentage of HBO and NHB trips in EI and IE categories during non-peak hours reflects the diversity of travel reasons, including leisure, errands, and other non-work-related activities.

Analysis of IE Trips:

Table 2 EI and IE Trips from OD Data for the B-M MPO Area

EI Trips Total					
Purpose	**7-8 AM**	**8 AM-3 PM**	**3-6 PM**	**Night**	**Total**
HBW	1,254	720	1,030	1,083	4,087
HBO	391	3,322	2,192	2,290	8,195
NHB	367	5,551	2,899	1,751	10,568
Total	2,012	9,593	6,121	5,124	22,850
Percentage of EI Trips to Total Trips for B-M Area					
Purpose	**7-8 AM**	**8 AM-3 PM**	**3-6 PM**	**Night**	**Total**
HBW	5.5%	3.2%	4.5%	4.7%	17.9%
HBO	1.7%	14.5%	9.6%	10.1%	35.9%
NHB	1.6%	24.3%	12.7%	7.6%	46.2%
Total	8.8%	42%	26.8%	22.4%	100%

IE Trips Total					
Purpose	**6-7 AM**	**7 AM-3 PM**	**3-6 PM**	**Night**	**Total**
HBW	397	668	891	792	2,748
HBO	280	2,628	2,142	3,285	8,335
NHB	381	4,889	2,338	2,093	9,701
Total	1,058	8,185	5,371	6,170	20,784
Percentage of IE Trips to Total Trips for BM Area					
Purpose	**6-7 AM**	**7 AM-3 PM**	**3-6 PM**	**Night**	**Total**
HBW	1.9%	3.2%	4.3%	3.8%	13.2%
HBO	1.3%	12.6%	10.3%	15.8%	40.1%
NHB	1.8%	23.6%	11.3%	10.0%	46.7%
Total	5.0%	39.4%	25.9%	29.6%	100%

2.1.3. External-External OD Data

Table 3 shows presents the External-External (EE) Origin-Destination (OD) data derived from Streetlight, highlighting the through trips that traverse the B-M MPO area without any stops. This data was sourced from Streetlight, a robust web platform designed for extensive data analysis in transportation planning. The approach to estimating EE OD pairs traversing the B-M MPO area involved a methodical process, which included:

1. Identifying and selecting all OD pairs that are external to the B-M MPO's internal OD Traffic Analysis Zones (TAZs), focusing on the remaining 13 external OD TAZs.

2. Uploading these external zones into the Streetlight platform for further analysis.

3. Conducting a comprehensive Origin-Destination Analysis, utilizing the external zones as the starting and ending points. The analysis covered all vehicle types and was conducted over a recent time frame from March 1, 2022, to April 30, 2022. The analysis was restricted to weekdays (Monday to Thursday) and segmented into hourly intervals to capture detailed travel patterns. Additional parameters, including Trip and Traveler Attributes, were factored into the analysis, yielding a dataset that includes vehicle trip volumes and the distribution of trips across different purposes (HBW, HBO, and NHB).

The findings from Table 3 reveal that EE trips account for approximately 0.03% of the total combined EE and EI/IE trips within the B-M MPO area, a figure considerably lower than the usual 10-12% typically observed in through-trip analyses. The distribution of EE trips varies by time of day and purpose:

- During the Night period, EE trips peak at 0.03% of the total, likely due to the inclusive early morning and late evening hours, suggesting that travelers passing through the area prefer these less congested times.
- HBW trips predominantly occur at night (0.005%), with minimal activity during the AM Peak and Off-Peak periods, indicating that work-related through trips are more frequent during early and late hours.
- HBO trips follow a similar pattern to HBW, with the highest occurrence at night (0.0039%) and the lowest in the AM Peak and Off-Peak periods.
- NHB trips exhibit the most substantial variation, with the highest night-time activity (0.008%) and significant movement during the PM Peak period (0.007%).

Table 3 EE Trips from Streetlight OD Data

EE Trips Passing through BM MPO					
Purpose	**9-11 AM**	**11 AM-1 PM**	**1-5 PM**	**Night**	**Total**
HBW	1	1	14	20	36
HBO	25	16	42	105	188
NHB	151	138	318	364	971
Total	177	155	374	489	1195
Percentage of EE Trips Passing through BM MPO					
Purpose	**9-11 AM**	**11 AM-1 PM**	**1-5 PM**	**Night**	**Total**
HBW	2.8%	2.8%	38.9%	55.5%	100%
HBO	13.3%	8.5%	22.3%	55.9%	100%
NHB	15.6%	14.2%	32.7%	37.5%	100%
Total	14.8%	13.0%	31.3%	40.9%	100%

Percentage of EE Trips to Total EE/EI Trips					
Purpose	**9-11 AM**	**11 AM-1 PM**	**1-5 PM**	**Night**	**Total**
HBW	0%	0%	0.0003%	0.0005%	0.0008%
HBO	0.0006%	0.0004%	0.0009%	0.002%	0.0039%
NHB	0.003%	0.003%	0.007%	0.008%	0.021%
Total	0.004%	0.004%	0.008%	0.01%	0.03%

2.2. Summary of OD Data

The Origin-Destination (OD) data presented, sourced from Streetlight, offers a detailed examination of travel patterns within the Bismarck-Mandan MPO area, enhancing the calibration and validation of the Travel Demand Model (TDM). This data provides insights into various trip types and purposes, including External-Internal (EI), Internal-External (IE), and External-External (EE) trips, across different time periods and for distinct trip purposes (HBW, HBO, NHB). By leveraging anonymized cellular data, Streetlight enables the estimation of crucial metrics such as AADT, VMT, and trip attributes, facilitating a more nuanced understanding of mobility patterns. This comprehensive dataset is instrumental in refining the TDM, ensuring it accurately reflects current and evolving transportation dynamics in the Bismarck-Mandan area, ultimately aiding in more informed transportation planning and policy decision-making.

3. CAPACITY CALCULATIONS

Capacities are pivotal in the operation of Travel Demand Models (TDM), serving as key indicators for measuring the Level of Service (LOS) and playing a crucial role in the traffic assignment phase. Traffic assignment in TDMs is influenced by the saturation levels (Volume to Capacity ratio) on each link, guiding the redistribution of traffic as saturation increases. The Transportation Research Board (2010) defines capacity as the maximum sustainable hourly flow rate at which persons or vehicles can traverse a specific point or section under prevailing conditions. However, the notion of capacity in traffic engineering, as outlined by NCHRP 716, differs slightly, often aligning with the volume at LOS E, whereas traditional travel models used capacity to denote the volume at LOS C.

Link capacities are generally determined by the number of lanes, but the actual capacity per lane can vary based on several factors such as lane and shoulder widths, peak-hour factors, transit stops, truck percentages, median treatments, access control, intersection control types, provision of turning lanes, and signal timing at intersections. Some methodologies integrate link and node capacities to offer a more nuanced view of each link, considering the attributes of the approaching intersection.

To refine the model's capacity calculations, a comprehensive literature review was conducted, examining capacity calculation methodologies across various MPOs, including those in Lincoln-NE, Des Moines Area-IA, and larger regions like Atlanta-GA and Dallas-Fort Worth-TX. This review aimed to gather insights and assumptions from these models, guided by population thresholds as suggested by NCHRP 716. Table 4 summarizes the literature review used in different MPO planning models for capacity calculations. It showcases the diverse approaches to capacity calculations in TDMs across different MPOs, providing a foundation for updating the capacity analysis in the B-M MPO's TDM.

Table 4 Summary of Capacity Calculations for MPO Planning Models

Lincoln MPO-NE, 2006	For the Lincoln MPO model, capacity at Level of Service (LOS) C was used as the threshold capacity. Highway Capacity Manual (HCM) 2000 procedures were used for estimating the capacity for each combination of functional class and area type. First, peak hour lane capacity was calculated after the effects of percent green time and peak hour factor. Second, the 24-hour lane capacity was calculated using peak hour lane capacity and percent of traffic in the peak hour. Finally, the threshold capacity at LOS C was assumed to be 75% of the 24-hour lane capacity. Reference: LIMA & Associates, 2006 http://www.princeton.edu/~alaink/Orf467F12/LincolnTravelDemandModel.pdf
VDOT, 2014	For all model regions, it is an acceptable practice and recommended practice to use the most recent version Highway Capacity Manual (HCM) as the basis for roadway capacities. It is not acceptable to use older versions of the HCM or arbitrary figures for roadway capacities. Based on functional class and land use/area type Tabulation process Reference: http://www.virginiadot.org/projects/resources/vtm/vtm_policy_manual.pdf
ODOT, 1995	The procedure used to estimate free flow speed and capacity is a detailed methodology that utilizes the

	maximum amount of information from the network and "connects" this data with information from the Highway Capacity Manual. http://www.oregon.gov/ODOT/TD/TP/docs/reports/guidex.pdf
Memphis MPO-TN	Hourly capacities were developed for the Memphis model in order to use collected street data. This provides the most accurate representation of actual capacity (levels of service A through E) on an individual link. These capacities — detailed in the Technical Memorandum #8(b) – Capacity Development — are implemented using an equation which takes into account functional classification, speed limit, lanes, signal density, median treatment, area type, average lane width, and average shoulder width. The capacity equations are built into the model process as a TransCAD lookup table, so modifications to network attributes automatically update the capacity in subsequent runs Since the model is based on four multi-hour time periods, a conversion factor must be used to create a time period capacity for each of the four time periods. The capacity factors below are based on hourly traffic count data and the Memphis household travel survey http://www.memphismpo.org/sites/default/files/public/documents/lrtp/appendix-g-travel-demand-model.pdf
GDOT, 2013	Facility type and area type are used in combination to determine free-flow speeds and capacities. Link capacities for the model network are obtained from a lookup table of per-lane hourly capacities based on facility type and area type. The final link capacity is calculated by multiplying the hourly capacity per lane by the number of lanes, which is automatically added to the links during the model application. http://www.dot.ga.gov/BuildSmart/Programs/Documents/TravelDemandModel/GDOT%20Model%20Users%20Gude_050813.pdf
MassDOT, 2013	The coding of the EMME/2 highway network basically follows the hierarchy of the functional classification system. Expressways, other than those passing through denser urban areas, are generally coded for 60 mph speeds and hourly capacity per lane of 1,950. Higher-level arterials are coded for speeds ranging from 45 to 50 mph and corresponding capacities of 1,050 to 1,100. Lower-level arterials and major collectors range from 35 mph to 40 mph, with capacities of 950 to 1,000. Minor collectors and local streets that are not in urban centers range from 23 mph to 30 mph, with capacity generally at 800. Streets in urban centers can have substantially lower speeds and capacities. https://www.massdot.state.ma.us/theurbanring/downloads/CTPS_Travel_Demand_Modeling_Methodology.pdf
Syracuse Metropolitan Transportation Council, NY, 2012	The speed and capacity values are stored in lookup tables and automatically imported to the network each time the model runs. The main benefits of importing these data from a lookup table, as opposed to maintaining an explicit speed and capacity for every link within the highway network, are that the user has less data to manage and can easily quote values. However, there are some links in the SMTC network that warrant special attention because their actual speed or capacity is substantially different from what the lookup tables say. Therefore, the SMTC model also supports the ability to code a speed or capacity for each link by entering a value into the “TOTAL_HCAP_FIXED” or “SPEED_FIXED” fields on the network http://www.thei81challenge.org/cm/ResourceFiles/resources/SMTC%20Model%20Version%203.023%20Documentation.pdf
Atlanta Regional Commission (ARC), GA, 2011	By area type and facility type Tabulation method 20 facility type and 7 area type Total link capacity (1Hr- LOS E) http://www.atlantaregional.com/transportation/travel-demand-model
Capital Area MPO (CAMPO)-MO, 2013	The model computes link capacities at run time. Capacities are initially based on functional class and number of lanes, adjusted based on directionality, median type, and roadway slope. Capacity is expressed in terms of vehicles per day for each link by direction. http://www.jeffersoncitymo.gov/11Jan2013CAMPOTDMDocumentation.pdf
Champaign-Urbana Urbanized Area Transportation Study (CUUATS), IL	The daily capacity for each link in the Champaign County model network was calculated based on its facility type and area type. If a Two-Way Left Turn Lane (TWLTL) was present, the link capacity was increased by 30%. The lookup table was included in the model script to uniformly assign the

	capacity on the model network. The centroid connectors have high capacity and very low speed (15mph).
Chattanooga-Hamilton County Regional Planning Agency, TN, 2013	Using the collected street data, the proposed capacity calculation for the Chattanooga model will be implemented using an equation that takes into account data such as functional classification, speed limit, lanes, median treatment, area type, average lane width, and average shoulder width. Traffic signal delays and the impact of steep grades may also be considered. The equations were originally developed using the Highway Capacity Manual (HCM) and analysis performed by the Indiana Department of Transportation in 1997 for the Indiana State Highway Congestion Analysis Plan. KHA successfully applied this method in other urban area models, in conjunction with analysis performed using North Carolina DOT's Level of Service (LOS) software. http://www.chcrpa.org/2040RTP/2040RTP_Draft_Plan/Volume_III_Travel_Demand_Model.pdf
Dallas-Fort Worth (DF): North Central Texas COG, TX, 2009	Hourly Capacity Per Lane (Divided or One-Way Roads) – The hourly capacity per lane for divided roads is given by area type and functional class. AMFactor, PMFactor, OPFactor – These factors are used in the conversion of capacity from hourly to time period. Factors are defined by functional class 1-8 http://www.nctcog.org/trans/modeling/documentation/DFWRTMModelDescription.pdf
San Diego Association of Governments, CA, 2011	Two capacities are calculated for each direction of a highway link: 1. Intersection and mid-link Hourly basis Time category Factored Future ramp metering improved the capacity growth by 10 percent. See the equations http://www.sandag.org/uploads/publicationid/publicationid_1624_13779.pdf
Chicago Metropolitan Agency for Planning, IL, 2014	Zonal capacity system Capacity represented within the link travel time function is approximately the service volume at the level of service C. It is calculated as 75 percent of the level of service E time link capacity. Note that link capacity is calculated by multiplying the hourly lane capacity by the number of lanes and the number of hours in the assignment time period
Omaha-Council Bluffs Metropolitan Area Planning Agency (MAPA), NE, 2010	The daily capacity is based on the hourly ultimate capacity, that is the point at which the Level of Service (LOS) changes from an "E" to an "F" as defined by the Highway Capacity Manual. To support the daily model, the hourly capacity is multiplied by a factor of 10, which represents a typical ratio of peak hour to daily traffic. Capacity varies by functional class, presence of turn lanes, the number of lanes, and whether the road is divided or undivided. The capacities are based on those used in Des Moines, Iowa. The capacities vary by side friction to take into account differences in driveway density. MAPA is currently comparing the capacities with other sources such as the capacity tables developed by the Florida DOT. The model does not include intersection delay separately from link delay. MAPA has attempted to represent intersection delay using downward adjustments to free-flow speeds https://www.fhwa.dot.gov/planning/tmip/resources/peer_review_program/mapa/mapa_report.pdf
Des Moines Area MPO, IA, 2006	Daily directional capacity of a link Divided or undivided Number of lanes Access condition Facility coding http://www.ctre.iastate.edu/educweb/ce451/LABS/Lab%2012/DSM_Documentation.pdf
KYOVA Interstate Planning Commission, WV, 2013	Capacity based on area and functional class Tabulation and look-up method http://www.kyovaipc.org/2040MTP/documents/KYOVA2040_ModelDocumentation_121213_withFigures.pdf
Knoxville Regional Transportation Planning Organization, TN, 2010	Peak hour capacities of the roadway network were estimated using Highway Capacity Manual 2000 procedures, which results in much more precise estimates of capacity verses traditional methods used in models that entail using a lookup table based on functional class and area type. http://www.knoxtrans.org/plans/mobilityplan/cndetern.pdf

Tulare County Association of Governments, CA, 2021	Link capacity is defined as the number of vehicles that can pass a point on a roadway at free-flow speed in an hour. One important reason for using link capacity as a model input is for congestion impact; which can be estimated as the additional vehicle -hours of delay based on the 2000 Highway Capacity Manual (2000 HCM). The capacity assumption used in the TCAG model of each road segment in the network is based on the terrain, facility type, and area type, which is consistent with the methodology suggested in the 2000 HCM http://www.arb.ca.gov/cc/sb375/tcag_scs_staff_report_final.pdf

Figure 3 shows the comparison of the capacity calculations from the 2021 base year planning model of the Bismarck Mandan MPO against those from various other MPOs. Notably, freeway capacities align closely with those from the 2021 Bismarck Mandan base model. However, ramp capacities in other MPO areas tend to be lower than those in the 2021 Bismarck Mandan model. When examining major arterials, minor arterials, collectors, and local roadways, it is evident that the capacity calculations are generally higher in the compared MPOs. This is primarily attributed to these MPOs utilizing Level of Service E for their capacity calculations, which typically yields higher capacity values. The comparison provides a perspective on how different MPOs approach capacity calculations within their traffic models, which is essential for traffic management and planning.

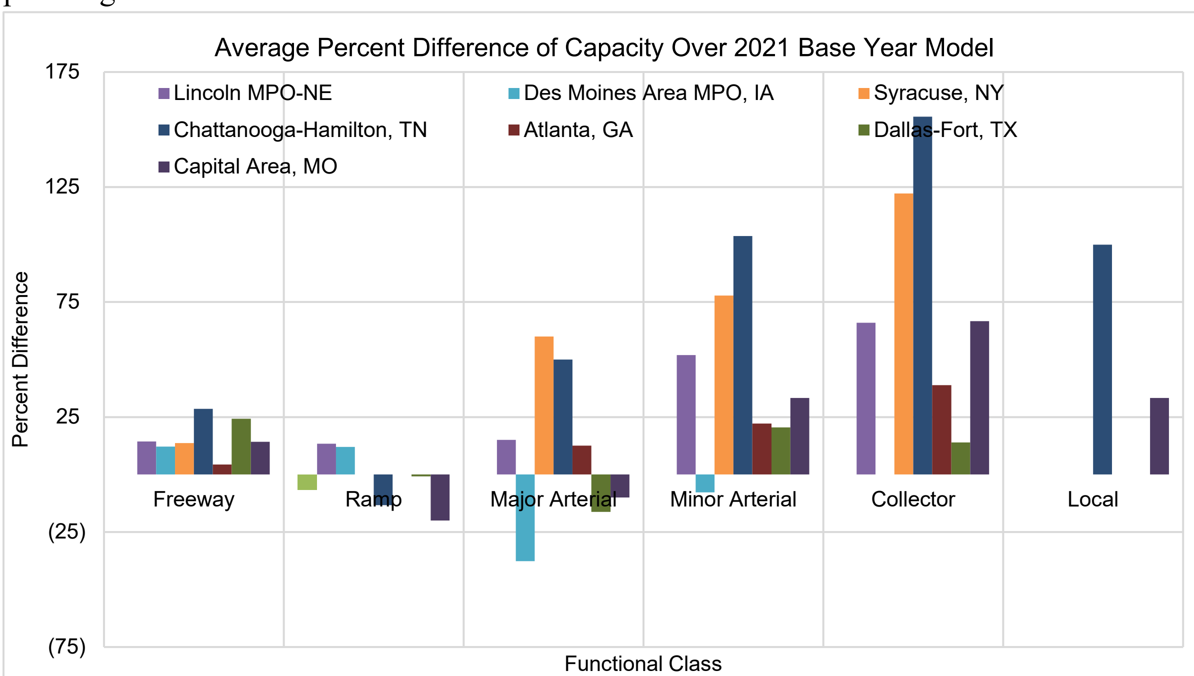

Figure 2 Capacity Comparisons to Bismarck Mandan MPO 2021 Base Year Model

For the 2021 base year model, network-wide capacities were updated to reflect the most recent and updated Highway Capacity Manual HCM 6th Edition and capacities estimated in other recent literature. The calculation of capacities took into account several variables including the functional classification, the number of through links, the number of turn lanes, the location of the intersection (rural, urban, CBD, suburban), the intersection control, and effective green ratios, heavy vehicle adjustment factors and the speeds. The next subsections discuss the capacity calculations for different types of intersections.

3.1. Capacity Calculations for Signalized intersections

For signalized intersections, a step-by-step procedure was used to estimate the capacities.

3.1.1. Step 1: Develop Lane Groups for each Link

The first step defined the lane groups for each link. For the 2021 network, lane groups are defined by the Attribute Linkgrp1. Table 5 shows the codes for each link group. The lane group describes the geometry at the B-node of each link including the number of through lanes, the number of right turn lanes, and the number of left turn lanes. The first Number in the linkgroup1 category shows the number of through lanes while the second number represents the number of turn lanes for either right or left turns as shown in Table 5. For example, if Linkgroup1 for a link was 20, it meant that the link had two through lanes with no turn lanes. Similarly, if the Linkgroup1 code was 35, it means the link had three through lanes, with two right-turn lanes.

Table 5 Lane Group Classification (Linkgroup 1)

Code	Lane Group Description
N0	N through lanes and no turn lane
N1	N through lanes and single exclusive left turn lane
N2	N through lanes and two exclusive left turn lanes
N3	N through lanes and continuous exclusive left turn lane from intersection to intersection
N4	N through lanes and single exclusive right turn lane
N5	N through lanes and two exclusive right turn lanes
N6	N through lanes and continuous exclusive right turn lane from intersection to intersection
N7	N through lanes, single exclusive left turn lane, and single exclusive right turn lane
N8	N through lanes, two exclusive left turn lanes, and a single exclusive right turn lane
N9	N through lanes, two exclusive right turn lanes, and a single exclusive left turn lane

3.1.2. Step 2: Determining saturation flow rate (S_i) for each lane group:

Step 2 included determining the saturation flow rate *(S_i)* for each Lanegroup using Equation 1. It is important to note that not all the parameters in Equation 1 were used for the model. Some of the parameters like the lane width and approach grades are not used in calculating the saturation flow rate. If the data is however available, say for a subarea study, these parameters can potentially be used to estimate capacities. The parameters were developed from different sources including Highway Performance Monitoring System (HPMS) and HCM6.

Equation 1

$$\mathbf{S_i = S_0 \times N \times f_W \times f_{HV} \times f_g \times f_p \times f_{bb} \times f_a \times f_{LU} \times f_{LT} \times f_{RT} \times f_{Lpb} \times f_{Rpb} \times PHF}$$

Where:

S_i	=	Saturation flow rate for subject lanegroup, expressed as a total for all lanes in lane group (vph)
S_O	=	Base saturation flow rate per lane (pcphpln)
N	=	Number of lanes in lane group
f_W	=	Adjustment factor for lane width
f_{HV}	=	Adjustment factor for heavy vehicles in the traffic stream
f_g	=	Adjustment factor for approach grade
f_p	=	Adjustment factor for the existence of a parking lane and parking activity adjacent to lane group
f_{bb}	=	Adjustment factor for blocking effect of local buses that stop within the intersection area
f_a	=	Adjustment factor for area type
f_{LU}	=	Adjustment factor for lane utilization
f_{LT}	=	Adjustment factor for left turns in lane group
f_{RT}	=	Adjustment factor for right turns in lane group
f_{Lpb}	=	Pedestrian-bicycle adjustment factor for left turn movements
f_{Rpb}	=	Pedestrian-bicycle adjustment factor for right turn movements
PHF	=	Peak Hour Factor

The formulas for calculating the parameters in Equation 1 from the HPBS are shown next:

1. Base Saturation Flow Rate, So

Following the HPMS procedure, the base saturation flow rate was set at 1,900 passenger cars per hour per lane (pcphpl).

2. Adjustment Factor for Lane Width, f_W

Using HPMS lane adjustment factors directly, Equation 2 was used to calculate the adjustment for lane widths,

Equation 2

$$\mathbf{f_W = 1 + \frac{(W-12)}{30}}$$

Where:

W = Lane width, minimum of 8ft and maximum of 16ft.

3. Heavy Vehicle Adjustment Factor, f_{HV}

Equation 3 was used to calculate the heavy vehicle adjustment factor.

Equation 3

$$f_{HV} = \frac{100}{100 + HV(E_T - 1)}$$

Where:

HV = percent heavy vehicles

E_T = 2.0 passenger car equivalents

4. Adjustment for Grade, f_g

Due to a lack of grade information on urban minor arterials and collectors, HPMS uses f_g as 1.0.

5. Adjustment for Parking, f_p

For parking adjustment, Equation 4 is used to calculate the capacity adjustment.

Equation 4

$$f_p = \frac{N - 0.1 - \frac{18N_m}{3{,}600}}{N}$$

Where:

f_p = Parking adjustment factor

N = Number of lanes in a group

N_m = Number of parking maneuvers per hour (6 for two-way streets with parking on one side, 12 for two-way streets with parking on both sides or one-way streets with parking on one side, 24 for one-way streets with parking on both sides)

If no parking space or parking data is available, then f_p is set equal to 1.0.

6. Adjustment for Bus Blockage, f_{bb}

Due to the non-availability of bus route data, f_{bb} is set to 1.0. Also, the default values of f_{bb} used in HCM 2000 for bus routes are close to one.

7. Type of Area Adjustment, f_a

According to HCM 6, f_a is set to 0.9 for CBDs and 1 elsewhere.

8. Lane Utilization Adjustment, f_{LU}

A lane utilization adjustment factor of 1.0 was used for the model.

9. Adjustment for Left Turns, f_{LT}

An adjustment factor of 0.95 is used for left turn movements to estimate the capacities in this study.

10. Adjustment for Right Turns, f_{RT}

For right-turn movements, the adjustment factor of 0.85 was used for the model.

11. Adjustment for Pedestrian-Bicycle Blockage on Left Turns, f_{Lpb}

The adjustment factor for pedestrian-bicycle blockage is set to 1.0 in the HPMS procedure due to the non-availability of extensive inputs.

12. Adjustment for Pedestrian-Bicycle Blockage on Right-Turns, f_{Rpb}

Similarly, the adjustment factor for pedestrian-bicycle blockage for right turns is also set to 1.

13. Peak Hour Factor (PHF)

The default values of 0.92 and 0.88 are set for urban and rural sections respectively.

14. Effective Green Ratios (g_i/C) for Lane Groups

A g_i/C value of 0.45 is used for principal and minor arterials while 0.40 is used for collectors. These values were default values suggested in HPMS. The values were evaluated based on signal timing data provided by the MPO and were found to be reasonable.

3.1.3. Step 3: Approach Capacity Calculation

After estimating the saturation flow rate for each lane group, the approach capacity for each link at the B end node of the link is calculated. This calculation is done by incorporating adjustment factors using the effective green ratio as shown in Equation 5.

Equation 5

$$C_{SI} = \sum_i S_i \times \frac{g_i}{C}$$

Where C_{SI} is signalized intersection approach capacity,

S_i represents the saturation flow rate for lane group i and

$\frac{g_i}{C}$ represents effective green ratio for lane group i.

3.2. Capacities for Stop Control Intersections

The calculation for capacities for links that have stop controls at the B-node end also follows a series of steps as described next.

3.2.1. Step 1: Calculate the Potential Capacity for each Turning Movement

The potential capacity for each turning movement uses the conflicting flow rate, the critical gap, the number of lanes, the follow-up time for each movement, and percent heavy vehicles as input parameters. Equation 6 is used to calculate the potential capacity for stop-controlled intersections for movements that are not shared.

Equation 6

$$C_{p,x} = CV_{c,x} \times \frac{e^{-V_{c,x} \times t_{c,x}/3600}}{1 - e^{-V_{c,x} \times t_{f,x}/3600}}$$

Where:

$C_{p,x}$	=	Potential Capacity of movement x (vph)
$CV_{c,x}$	=	Conflicting flow rate for each movement x (vph)
$t_{c,x}$	=	Critical gap (seconds) for each movement x $= t_{c,base} + (P_{HV} * t_{c,HV})$
$t_{c,base}$	=	Default values from Table 6
$t_{c,HV}$	=	1.0 for one or two-through-lane roads 2.0 otherwise
P_{HV}	=	Percent of heavy vehicles in traffic stream, peak period, expressed as decimal
$t_{f,x}$	=	Follow-up time (seconds) for each movement x $= t_{f,base} + (P_{HV} * t_{f,HV})$
$t_{f,HV}$	=	0.9 for one or two through-lane roads 1.0 otherwise

Tables 6 and 7 show the default values that were used for calculating the potential capacities for stop-controlled intersections in the model.

Table 6 Default values for calculating potential capacities (Cp,x) of stop sign-controlled highways

Vehicle Movement (x)	Base Critical Gap, $t_{c,base}$	Follow-up Time, $t_{f,base}$
Right Turns	6.2	3.3
Through	6.5	4.0
Left Turns	7.1	3.5

Table 7 Default Values for Conflicting Flow Rates

Functional Class	Conflicting Flow Rate, $CV_{c,x}$
Rural Principal Arterials	**100**
Rural Minor Arterials	**150**
Other Rural	**200**
Urban Principal Arterials	**250**
Urban Minor Arterials	**500**
Other Urban	**750**

3.2.2. Step 2: Determine Potential Approach Capacity for Shared Lanes

For stop-controlled intersections with shared turning lanes, Equation 7 was used to determine each approach's capacity. If turn lanes are not shared, step 2 is skipped.

Equation 7

$$C_{p,SH} = \frac{\sum_x V_x}{\sum_x \left(\frac{V_x}{C_{p,x}}\right)}$$

Where,

$C_{p,SH}$	=	Potential capacity of the shared lane (vph)
V_x	=	Flow rate of the x movement in the shared lane (vph)
$C_{p,x}$	=	Potential capacity of x movement in the shared lane (vph)

3.2.3. Step 3: Calculate Approach Capacity for each Lane Group Type

Table 8 shows the different equations that are used to calculate the approach capacity for each lane group as described previously for stop-controlled intersections.

Table 8. Stop Sign Control Intersection Capacity Equations for Different Lane Groups

1	**All Movements from Shared Lane**	$C_A = N_T \times C_{p,SH}$
2	**Shared LT + T lane; exclusive RT lane**	$C_A = N_T \times C_{p,SH(LT+T)} + N_{RT} + C_{p,RT}$
3	**Shared RT + T lane; exclusive LT lane**	$C_A = N_T \times C_{p,SH(RT+T)} + N_{LT} + C_{p,LT}$
4	**Exclusive lanes for all movements**	$C_A = N_{LT} \times C_{p,LT} + N_T \times C_{p,T} + N_{RT} \times C_{p,RT}$
5	**Consider only through volumes**	$C_A = N_T \times C_{p,T}$

Where:

N_T	=	Number of peak through lanes; 1 for rural highways with two through lanes, 2 for rural highways with three through lanes
N_{LT}	=	Number of left turn lanes
N_{RT}	=	Number of right turn lanes
$C_{p,SH}$	=	Potential capacity of shared lane (vph)
$C_{p,T}$	=	Potential capacity for through movement (vph)
$C_{p,RT}$	=	Potential capacity for right turn movement (vph)
$C_{p,LT}$	=	Potential capacity for left turn movement (vph)

3.3. Freeway Capacity

For freeways, the following steps detail the equations and procedures used to calculate their capacities.

3.3.1. Step 1: Calculate Free Flow Speed

Equation 8 is used to calculate free-flow speeds. The equation utilizes the base free flow speed which is calculated using an algorithm that incorporates real-time travel time data, lane width, right shoulder, number of lanes, and interchange density adjustments.

Equation 8

$$FFS = BFFS - f_{LW} - f_{LC} - f_N - f_{ID}$$

Where:

BFFS	=	Base free flow speed
f_{LW}	=	Adjustment factor for lane width
f_{LC}	=	Adjustment factor for right shoulder lateral clearance
f_N	=	Adjustment factor for number of lanes
f_{ID}	=	Adjustment factor for interchange density

Table 9 shows the adjustment factors for lane width. This value is zero for 12ft wide lanes. However, if different widths exist, the values should be adjusted accordingly.

Table 9 Adjustment Factors Lane Width

Lane Width	Reduction in FFS (mph, f_{LW})
12 Ft	0.0
11 Ft	1.9
<= 10 ft	6.6

Table 10 shows the adjustment factors for right shoulder clearance. The model assumed a right shoulder clearance of greater than 6 ft. Adjustments should be made accordingly if these are different. For studies used to evaluate the construction/reconstruction impacts on freeways, this parameter will be critical in determining the reduced capacity if shoulders are closed or reduced.

Table 10 Right Shoulder Clearance Adjustment Factor

Right Shoulder Width (Ft)	Reduction in FFS (mph, f_{LC})			
	Lanes in one direction			
	2	3	4	>=5
>=6	0.0	0.0	0.0	0.0
5	0.6	0.4	0.2	0.1
4	1.2	0.8	0.4	0.2
3	1.8	1.2	0.6	0.3
2	2.4	1.6	0.8	0.4
1	3.0	2.0	1.0	0.5
0	3.6	2.4	1.2	0.6

Table 11 shows the adjustments used for interchange densities. The distance between two nodes connecting the interchanges is used to calculate the interchange density. The values for small urban areas are used in the model. For the model, all interchange densities were greater than 1 mile. This parameter becomes important when new interchanges that increase interchange densities are being considered as they will potentially reduce freeway capacities.

Table 11 Adjustments for Interchange Density

Area Size	Interchange Density	Interchange Adj. Factor, (f_{ID})
Small Urban	0.70	1.0
Small Urbanized	0.76	1.3
Large Urbanized	0.83	1.7
Small Urban	0.83	1.7
Small Urbanized	0.88	1.9
Large Urbanized	0.91	2.1

Table 12 details the adjustment factors used for adjusting freeway capacities based on the number of lanes.

Table 12 Adjustments for Number of Lanes

No of Lanes (One direction; Urban only)	Reduction in FFS (mph, f_N)
>=5	0.0
4	1.5
3	3.0
2	4.5

3.3.2. Step 2: Calculate Base Freeway Capacity

The base freeway capacity is calculated using Equation 9 for freeways with speeds less than 70mph and freeways with speeds greater than 70mph.

Equation 9

$$BaseCap = 1{,}700 + 10FFS; for\ FFS \leq 70\ mph$$

$$BaseCap = 2{,}400 \qquad ; for\ FFS > 70\ mph$$

3.4. Ramp Capacity Calculations

The following steps were used to calculate ramp capacities:

3.4.1. Step 1: Calculate Free Flow Speed

Using Equation 10, the free flow speed for ramps was calculated as follows

Equation 10: Ramp Capacity Equation

$$S_{fo} = 25.6 + 0.47 * S_{pl}$$

Where S_{fo} = base free-flow speed (BFSS); and

S_{pl}= posted speed limit

3.4.2. Step 2: Calculate Maximum Saturation Flow Capacity

The Chattanooga-Hamilton model was used to develop Equation 11 to calculate ramp capacities as follows:

Equation 11: Maximum Saturation Flow Capacity

$SF= C *N* (v/c)_I * PHF$

Where SBMaximum service flow rate;

C ideal capacity based on S_{fo};

N represents lumber of lanes;

(v/c) is rate of service flow for levels of service D or E. v/c=0.88 at LOS D, 1 at LOS E; and

PHF represents the peak hour factor.

Table 26 and Table 27 Appendix 1 shows sample Capacity calculations that are used in the model for signalized intersections.

4. MODEL INPUT DATA

The main data inputs for the travel demand model consist of transportation network details and socioeconomic datasets. These are developed in a concerted effort by the MPO staff and the Advanced Transportation Analysis Center (ATAC). The ensuing discussion elaborates on these datasets.t.

4.1.Transportation Network Data

The transportation network forms a vital abstract model of the actual transportation infrastructure, encapsulating crucial supply-side data. Maintained within a GIS geodatabase, it encompasses four primary feature classes: links, which depict the roadways; nodes, representing the intersections; centroids, denoting trip origins/destinations within the Transportation Analysis Zones (TAZs); and external centroids, marking the external trip entry points.

This network has been jointly updated by ATAC and the MPO to mirror the conditions of the base year 2021, ensuring an accurate representation of the transportation system's current state. Critical attributes of the network utilized in the modeling process include the geometric configurations of the network (such as the number of lanes, including turn lanes), posted and optimal free-flow speeds, functional classifications, the length of links, Average Daily Traffic (ADT) for both passenger and freight traffic, the type of area in which a link is located, and the nature of intersection controls. These attributes are integral to the model as they directly influence the simulation of traffic flow and behavior. Distribution of Modeled Network by Functional Classifications. Table 13 shows the percentage of centerline miles by functional class.

Table 13 Centerline Miles Distribution by Functional Classification

Functional Class	Centerline Miles	Percentage
Interstate	35.32261	9.6%
Major	76.7199	21.0%
Minors	131.20811	35.8%
Collectors	121.0004	33.1%
Locals	1.80722	0.5%

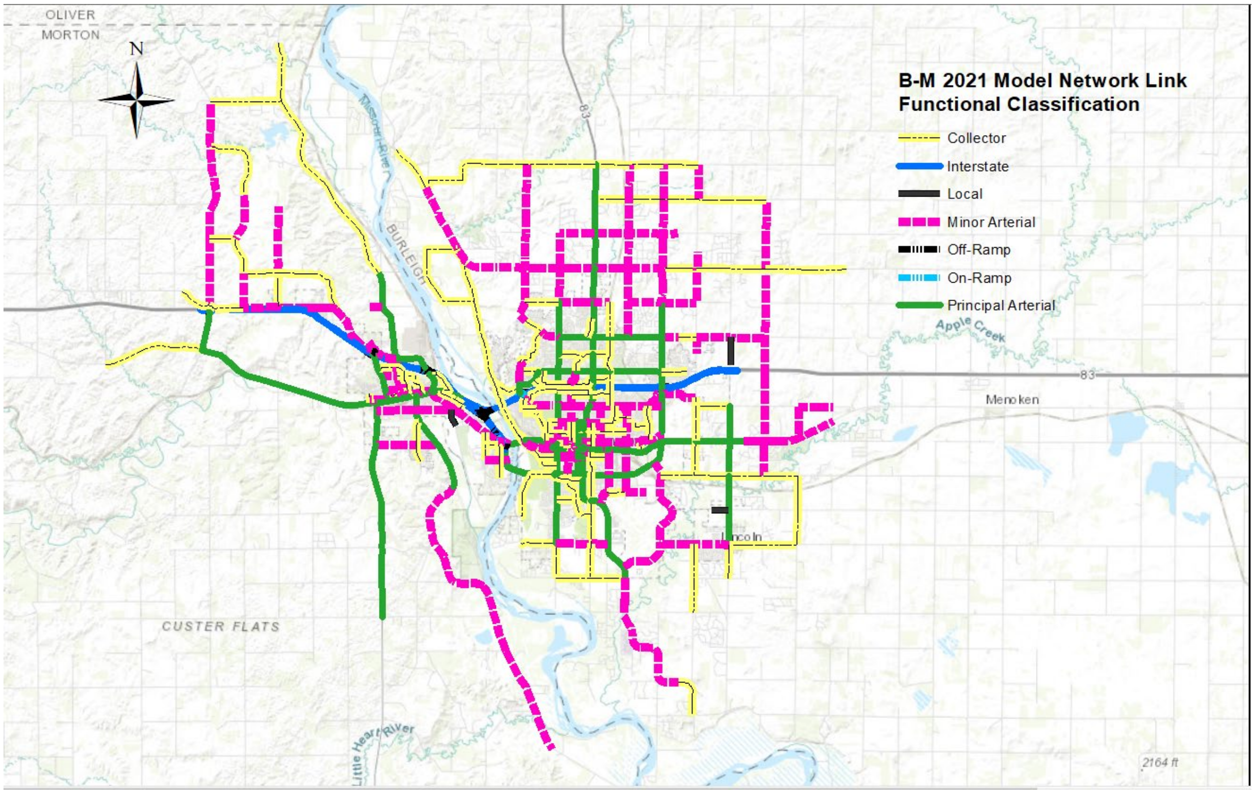

Figure 3 2021 Model Network

Figure 4 shows the modeled network distribution by functional class. The network does not show the centroid connectors.

4.2.Socioeconomic Data

Socioeconomic data are used to generate the total number of trips produced and attracted by each TAZ in the TDM. The TAZ geographies and the socioeconomic data included within each TAZ were developed by a collaborative effort between MPO staff and the ATAC. The socioeconomic data that was used in the model is described next.

4.2.1. TAZ Geography files:

A total of 408 internal TAZs were used for the 2021 model. Only one TAZ (TAZ 44) was modified (split or merged). TAZ 44 was a TAZ that was East of State Street from 43^{rd} Ave NE to 57^{th} Ave NE and included both Walmart and Costco. This TAZ was split into three TAZs (444,407 and 408).

4.2.2. Socioeconomic Data TAZ Attributes

The socioeconomic data within the TAZ contained the following fields

4.2.2.1. Number of Persons per household in each TAZ according to the following categories (attributes)

1. # of one person households
2. # of two person households
3. # of three person households
4. # of four person households
5. # of five person households
6. > # five person households
7. Total number of households

1. # of Grade school age children
2. # of Middle age school children
3. # of High school age children
4. # of College age (18-23)

4.2.2.3. Employment data (# for each TAZ)

For the employment data, the categories listed below were grouped into retail, service, and other jobs for trip generation. The data was provided by SRF and was vetted by HDR Inc. The data underwent several iterations to get it into the final dataset that was used for the model.

1. Commercial Jobs
2. Industrial Jobs
3. Commercial Office Jobs
4. Comm. Shopping Jobs
5. Other Jobs

4.2.2.4. Enplanements

8. Yearly enplanements for the Bismarck Airport for 2021 (259,734)

4.2.2.5. Special generators

9. Special generator TAZS (wholesale distributors (Walmart and Super Target, large retail stores, and Malls).

4.2.2.6. ADT at external locations

Used as estimates of trips that have at least one trip end outside of the MPO area.

5. TRIP GENERATION

Trip generation is the initial phase of the Travel Demand Model (TDM), which assesses the volume of trips both originating and destined for each Transportation Analysis Zone (TAZ). Utilizing the socioeconomic data detailed in Chapter 4 alongside established regression parameters, the model estimates trips generated and attracted per TAZ. Typically, trips produced are associated with household attributes within a TAZ, while trips attracted correlate with the employment factors of that TAZ. A notable enhancement in this iteration of the model is the incorporation of long-haul freight movements, which adds depth to the trip generation analysis. The next sections describe in detail, the different trip generation procedures that were used and their results.

5.1.Internal-Internal Passenger Vehicle Trip Productions and Attractions

The internal-internal passenger vehicle trip generations (II Trips) encapsulate those passenger vehicle trips that both start and finish within the MPO's purview. These trips are categorized into six principal purposes: Home-Based Work (HBW), Home-Based Shop (HB-Shop), Home-Based Other (HBO), Home-Based School K-12 (HBSchool K-12), Home-Based University (HBU), and Non-Home Based (NHB).

5.1.1. Trip Productions

Table 14 shows the trip generation equations that were used to develop the II trip production tables. The numbers in bold show the actual regression parameters used while the number underneath each one shows the p-value for each of the regression equations. The model parameters were developed from a household travel survey that was done in the Fargo-Moorhead area. These parameters are the starting equations that were used, the final equations were adjusted during the calibration process to reflect different area types and to match the observed traffic counts in the trip assignment step.

Table 14 Internal-Internal Passenger Trip Generation Equations

Persons per Household					
Purpose	**1**	**2**	**3**	**4+**	**Overall**
HBW	**1.049**	**1.665**	**2.624**	**2.457**	**2.21**
	14.9	*19.82*	*13.61*	*17.15*	*30.45*
HBSH	**1.127**	**2.092**	**3.424**	**3.424**	**2.86**
	5.03	11.52	5.70	6.65	14.23
HBO	**1.322**	**2.465**	**2.390**	**4.665**	**3.08**
	11.9	*21.04*	*9.64*	*9.74*	*20.81*
NHB	**2.006**	**2.421**	**2.961**	**3.329**	**3.04**
	11.44	*17.78*	*7.39*	*10.1*	*22.49*

5.1.2. Trip Attractions

Trip attractions represent the number of trips attracted to each zone typically based on employment and the size of the school for school trips. Table 15 shows the trip attraction rates derived from NCHRP 718 employed in developing the trip attraction tables. While the socioeconomic data indicated diverse employment categories, for modeling simplicity, these were consolidated into broader groups as depicted in Table 15.

Table 15 Trip Attraction Rates

Purpose	Retail	Service	Other
HBW	1.284	1.284	1.284
HBO	1.1	1.5	0.2
NHB	2.1	1.4	0.5

Table 16 shows the school trip attraction rates that were used for the model. These trip rates were obtained from the ITE Trip Generation Manual.

Table 16 School Trip Attraction Rates

School	Rate
Elementary	1.85
Middle	1.85
High	1.96

6. TRIP DISTRIBUTION

The trip distribution step is an essential component in the progression of the Travel Demand Model (TDM), where it allocates the trip productions and attractions, identified during the trip generation phase, between various Origin-Destination (OD) pairs. Utilizing the gravity model, this step disperses trips according to the volume of trip productions and attractions, a friction factor (F), and a scaling factor (K).

The friction factor is critical to the gravity model as it represents the travel impedance, or resistance, between zonal pairs and is inversely related to measures such as distance, time, or cost. This factor plays a pivotal role in modeling as it influences the likelihood of travel between zones based on the "cost" of travel, whether in terms of actual monetary cost, travel time, or distance.

The K factor acts as an adjustment tool within the model calibration process. It is applied to modify the volume of traffic traversing the network's segments, effectively scaling the distributed trips to match observed traffic counts. By fine-tuning the K factor, the model can more accurately reflect real-world traffic patterns, ensuring the distribution of trips within the model aligns with the empirical data. This process of calibration and validation is crucial to ensure the model's outputs are reliable and can be used to inform transportation planning and policy decisions. Equation 12 shows the gravity model formulation used in the model.

Equation 12 Gravity Model Used for Trip Distribution

$$T_{ij} = P_i \frac{K_{ij} A_j F_{ij}}{\sum K_{ij} A_j F_{ij}}$$

T_{ij} = Number of trips assigned between Zones i and

j;

P_i = Number of Productions in Zone i;

A_j = Number of Attractions in Zone

j; F_{ij} = Friction Factor; and

K_{ij} = Scaling factor used in calibration to influence specific ij pairs

The trip distribution phase in Travel Demand Models (TDM) typically produces an Origin-Destination (OD) matrix that outlines the starting point and endpoint for each trip within the study area. This process employs the outputs from the trip generation step-namely the number of trips produced and attracted by purpose in each zone. It also uses a measure of travel impedance (often travel time) between zonal pairs, in conjunction with a set of socio-economic and area characteristic variables, often referred to as the "K-factor."

The K-factor is an adjustment variable used to account for the influence of factors beyond the standard travel impedance within the gravity model. This includes socio-economic variables and other area-specific characteristics that might affect trip distribution patterns. OD data serves as a foundational element for deriving K-factor matrices which are then integrated into the gravity model, particularly for the distribution of External-Internal (EI) and Internal-External (IE) trips.

To refine the K-factors, external trips were consolidated into four principal external "super zones" based on their geographic origination relative to the MPO area. For instance, all trips originating north of the MPO were grouped into a single "super TAZ." The trip proportions from each internal 2021 OD TAZ to this "super TAZ" were calculated and applied as the K-factor in the trip distribution process. This approach enhances the model's efficiency in distributing trips across the network.

For the distribution of External-External (EE) trips, the OD data were leveraged to develop K-factors in a method akin to that of EI/IE trips, facilitating their inclusion in the EE trip distribution segment of the TDM.

Regarding the distribution of K-12 school trips, specific school zones were designated for public schools in Bismarck Mandan, with the gravity model being utilized to distribute trips to private schools. This dual approach ensures that the model accurately reflects the unique travel patterns associated with school trips within the region.

7. 1. TRIP ASSIGNMENT

The trip assignment is the final computational step in the four-step travel demand modeling process. This phase involves determining the specific routes that trips will take across the transportation network from origin to destination. For the purposes of the model, trip assignments are differentiated across three temporal matrices: the morning peak (AM peak), evening peak (PM peak), and off-peak periods.

The model employs the user equilibrium method for traffic assignment. This method posits that each traveler chooses their route based on the least cost to themselves, which is often synonymous with the least travel time. However, this individual optimization does not account for the overall efficiency of the network; users operate independently without consideration for the collective travel time.

In contrast, a system-equilibrium approach would entail travelers selecting routes in a cooperative manner that benefits the entire system, thereby minimizing the average travel time or cost for all users. Such a method assumes a level of coordination among users to achieve an optimized network.

8. VALIDATION AND CALIBRATION

Model calibration and validation are integral to the efficacy and accuracy of a Travel Demand Model (TDM). Calibration is the process of fine-tuning the model's input parameters to ensure that the model's outputs closely match observed data from the real world. This is often done for a specific base year and encompasses adjustments to trip generation rates, node delays, free flow speeds, K factors, friction factors, and other relevant parameters. The goal of calibration is to tailor the model so that it not only replicates known traffic patterns but also provides a reliable framework for predicting future conditions.

Validation, on the other hand, follows calibration and is a process to confirm that the calibrated model can accurately forecast travel behavior and traffic flows under different conditions. It typically involves comparing the model's outputs to an independent set of traffic data not previously used in calibration. Successful validation indicates that the model can be a reliable tool for predicting traffic responses to changes in the network, such as new infrastructure, policy changes, or fluctuations in demand.

Figure 6 illustrates the calibration and validation flow chart, which outlines the iterative nature of these processes. This iterative process is critical as it allows for continuous refinement of the model parameters. The aim is to reach a level of confidence where the model's simulated outputs have an acceptable level of agreement with observed traffic data, which is often quantified through statistical measures such as the root mean square error (RMSE), the coefficient of determination (R^2), or other goodness-of-fit indicators.

Calibration and validation are not one-time processes but rather ongoing requirements as new data become available, the transportation network changes or the model is applied to forecast future scenarios. This ensures that the model remains accurate over time and can adapt to the evolving patterns of travel behavior and network usage.

The next sections describe the different model parameters that were used for model calibration and validation.

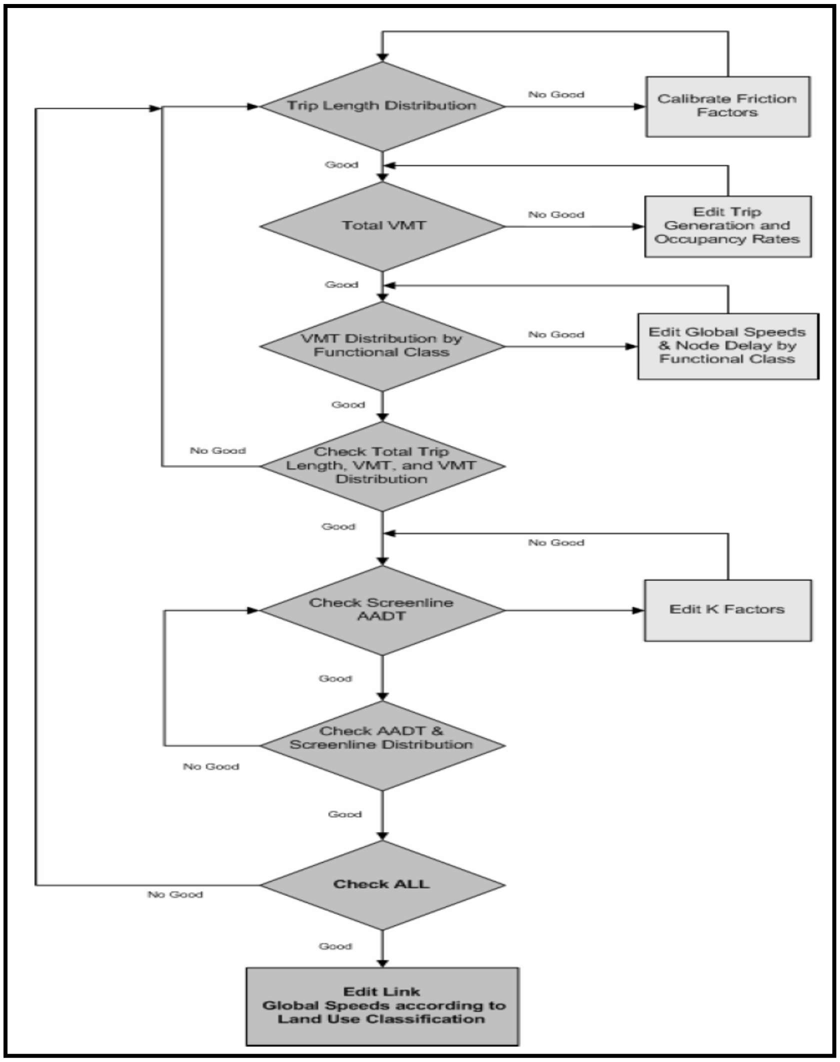

Figure 4 Calibration Flow Chart

8.1.Trip Length Frequency Calibration and Validation

Trip length frequency distributions are fundamental in understanding the behavioral aspects of travel demand, as they reflect the tendency of travelers to undertake trips of various distances. These distributions are often visualized as curves, with the steepness indicating how sensitive travelers are to travel time for a particular trip purpose. For instance, steep curves signify a high sensitivity, implying that travelers are less likely to engage in longer trips, or will do so only if travel times are relatively short.

Calibrating friction factors is a crucial step in tailoring the trip length frequency to align with observed data. Friction factors, pivotal in the gravity model, essentially weigh the attractiveness of a destination against the cost of getting there, with the cost usually being measured in terms of travel time or distance. When the observed trip lengths are matched, it suggests that the model can accurately reflect how real-world factors such as distance and time influence trip-making behavior.

The gamma function is a flexible tool for shaping friction factors because it can be adjusted to produce a variety of curve forms to match the observed trip length frequencies. By manipulating the parameters of the gamma function, modelers can fine-tune the sensitivity of the gravity model to distance or travel time, ensuring that the modeled trip distribution realistically reflects the observed patterns of travel within the region. This level of detail in the modeling process ensures that the resulting TDM can reliably forecast travel behaviors and inform transportation planning and infrastructure investment decisions.

The gamma function was used to develop the friction factor for this model and are shown in Figure 7.

Equation 13 Friction Factor Equation

$$\mathbf{F_{ij}^{p} = a * t_{ij}^{b} * exp(c * t_{ij})}$$

Where,

F_{ij}^{p} = Friction factor for purpose p (HBW, HBO, NHB)

t_{ij}^{b} = travel impedance between zone i and j,

a, b and c are gamma function scaling factors.

The friction factors were calibrated by adjusting the b and c parameters until the desirable trip length frequency distribution for Home Based Work Travel times were reached. Observed trip length frequency data for the home-based work trips were obtained from Streetlight. Only trips up to 40 minutes were considered with the assumption that 40 minutes was the highest possible travel time between any two points within the metro area. However, Streetlight data shows that about 45% of the trips fall within 2 minutes which is not feasible. Therefore, the trips that fall within the trip length 1-4 minutes are deleted. After those trips were deleted, the average trips were closer to the model trip length frequency. Deletion of trips less than 5 minutes is justified because the American Community Survey (ACS) report about travel time to work in the United States (2019) calculates the travel time to work at a 4 minutes interval. Also, most of the trips percentages fall within probably 10-14 minutes in the census block data. The average trip length for the observed data was calculated as 9.93 compared to the average trip length of 10.498 produced by the model for HBW trips. The desired average trip lengths for HBO and NHB trips were 100% and 104% of the average trip length for HBW trips. The average trip length for the models HBO and NHB trips were 10.0 and 10.05 minutes respectively.

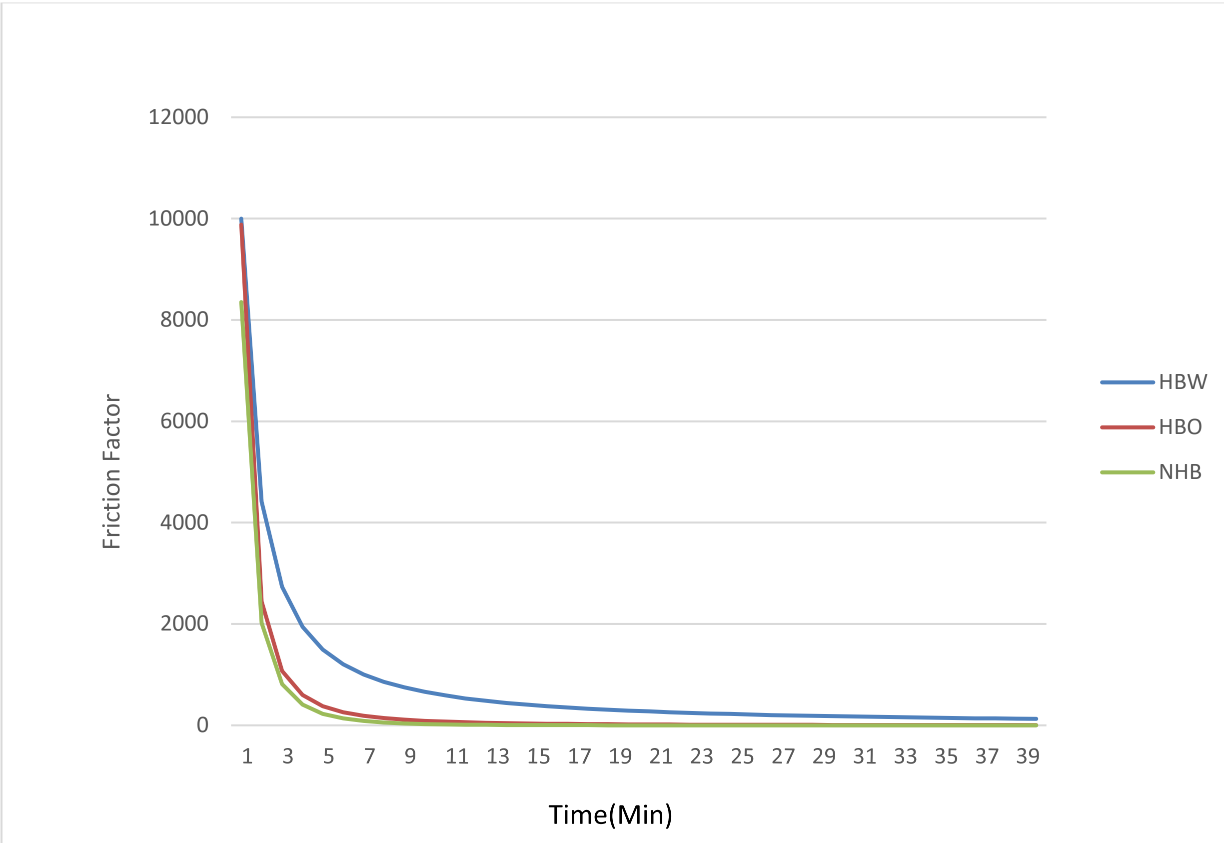

Figure 5 Friction Factors

Figure 8 shows the comparison between observed and modeled trip length frequencies across total Home-Based Work (HBW), Home-Based Other (HBO), and Non-Home Based (NHB) trips. The close alignment between the observed and modeled data suggests that the model has been calibrated effectively.

The two curves in the graph present a similar shape and follow the same trend across the range of trip lengths, which indicates that the model accurately captures the real-world distribution of trip lengths. The peak of both curves occurs at the shorter trip lengths, with a rapid decline as trip length increases. This is typical of trip length frequency distributions, as shorter trips are usually more common.

A small difference at the beginning of the trip lengths suggests some variation in trip initiation or recording, but the overall correspondence between the two curves in the majority of the trip length bands underscores a well-calibrated model. Such congruence demonstrates that the model can reliably replicate actual travel patterns, which is essential for predicting future travel behaviors under various scenarios and for assessing the impacts of transportation planning decisions.

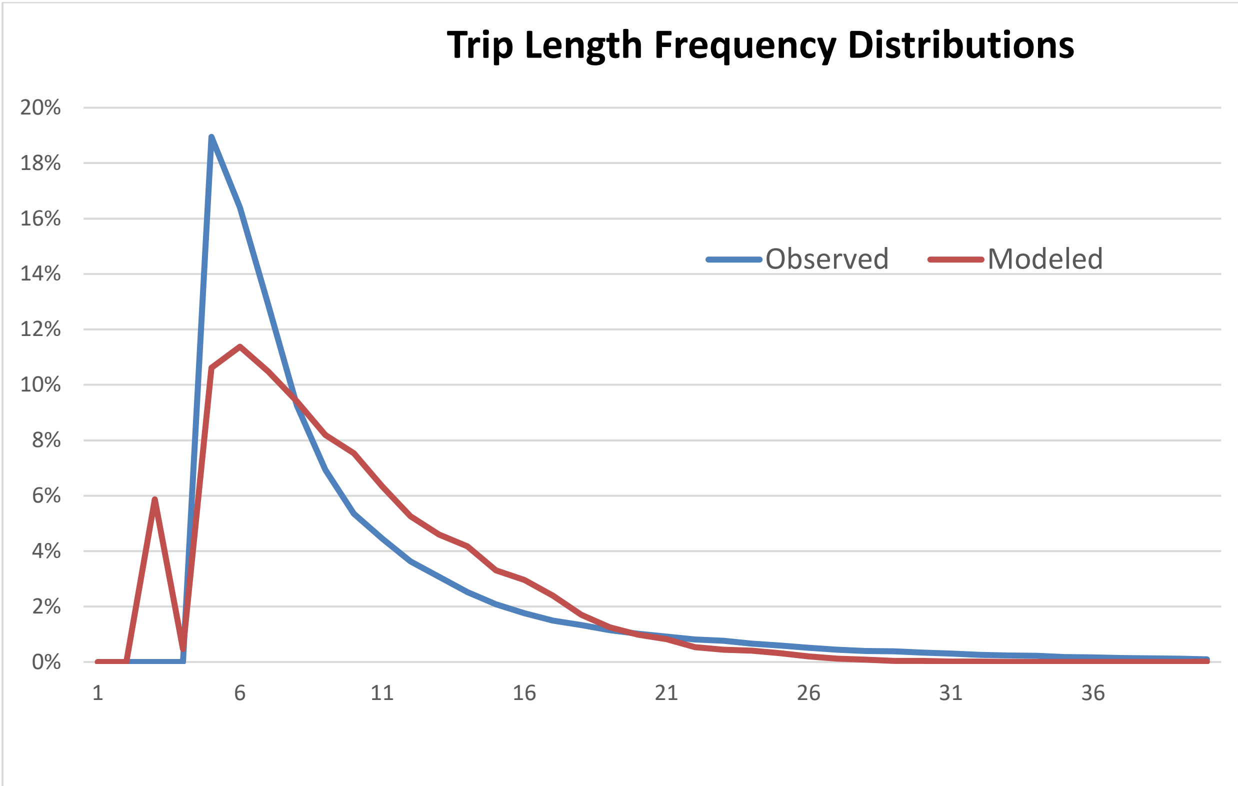

Figure 6 Comparison of Observed to Model Trip Length Frequency

8.2.Vehicle Miles Traveled (VMT) Calibration and Validation

The modeled VMT is calculated based on the number of trips generated by the model and their respective trip lengths. An accurate VMT estimate is essential, as it provides a measure of the travel demand's reasonableness and can have significant implications for planning and policy decisions related to transportation. To calibrate VMT, the total modeled VMT for the entire model area is initially adjusted. This is done by comparing it against observed VMTs, which are calculated by multiplying the Average Daily Traffic (ADT) counts by the lengths of the links. Discrepancies between the modeled and observed VMTs prompt adjustments in trip generation rates and vehicle occupancy figures until the two values converge. Adjusting the trip generation and occupancy rates changes the total number of trips that are generated within the transportation model. This in turn increases or decreases the total number of vehicle miles traveled.

Once the total VMT was reasonable, ATAC checked the VMT distribution across various functional classes of roadways is examined. This distribution offers insights into the model's assignment accuracy and can reveal whether the model is effectively replicating real-world speed, capacity, and assignment behaviors. If the VMT distribution by functional class diverges from expected patterns, modifications to the model, such as adjusting global speeds by facility type, may be necessary.

Table 17 shows the comparison between the observed and modeled VMTs, showcasing how they align by functional class as a percentage of the total VMT. A well-calibrated model is indicated by minor VMT differences—such as the less than 5.1% variance observed for interstates—and a distribution that mirrors the observed data. An overall deviation of 0.3% suggests the model is robust in mimicking VMT by functional class.

Table 17 Modeled VMTs compared to Observed VMTs by Volume Range

Functional Class	Observed VMT	Modeled VMT	% Diff
Interstate	340,737	350,320	3%
Major	544264	585191	7%
Minors	239,948	221,695	-8%
Collectors	131,353	134,397	2%
Total	1,256,302	1,291,603	3%

8.3.Modeled ADT Comparison to Observed ADT

Comparing the modeled ADTs to the Observed ADTs is the ultimate test of how well the model can replicate ground truths. The MPO provided traffic counts for several links that were compared to the Model ADTs. Two comparisons are made, one for the different functionally classifications and one by volume ranges.

Table 18 shows the comparison of the modeled and observed ADTs by functional classification. Overall, the model performs reasonably replicating over 73% of observed counts.

Table 18 Comparison of Modeled and Observed ADTS by Functional Classification

Functional Classification	Below Criteria	Within Criteria	Above Criteria	Total	%age Within	RMSE %
Interstates	1	11	1	12	84.62%	22.50%
Major Arterials	33	147	28	208	70.67%	41.16%
Minor Arterial	29	160	53	242	66.12%	141.18%
Collectors	21	114	24	159	71.70%	100%
Locals	0	1	0	0	100%	0%
Total	84	433	106	621		
Percent	13.53%	69.73%	17.07%			

Table 19 shows the comparison of modeled and Observed ADTs by volume range. The FHWA criterion sets limits to the deviations between observed and modeled ADTs. Overall the model meets all deviation criterion for all the volume ranges and replicates 70% of the observed traffic.

Table 19 Comparison of Modeled and Observed ADT by Volume Range

ADT Range	#Above	#Within	#Below	%Within	RMSE
ADT >25,000	2	10	1	76.92%	0.1283
25,000 TO 10,000	16	115	28	0.7233	0.2396
10,000 TO 5,000	30	116	33	0.6480	0.3469
5,000 TO 2,500	38	84	22	0.5833	0.6326
2,500 TO 1,000	13	81	0	0.8617	1.0439
ADT<1000	7	27	0	0.7941	28.2414
Total	106	433	84	70%	

8.4.Scatter Plots, R Squares of Model and Observed Traffic

Scatter plots of the modeled traffic volumes against the observed traffic volumes are a good indicator of the model's fit. Figure 9 shows the scatter plot of modeled traffic volumes versus observed counts. The scatter plot suggests that the amount of error in the modeled volumes is proportional to the observed traffic count which is an indication of a good fit between the model and the observed traffic counts.

The R-square (coefficient of determination) is the proportion of the variance in a dependent variable that is attributable to the variance of the independent variable. They typically measure the strength of the relationships between the assigned volumes and the traffic counts. It measures the amount of variation in traffic counts explained by the model. The modeled R-square of 0.85 shows a strong linear relationship between modeled and observed traffic counts.

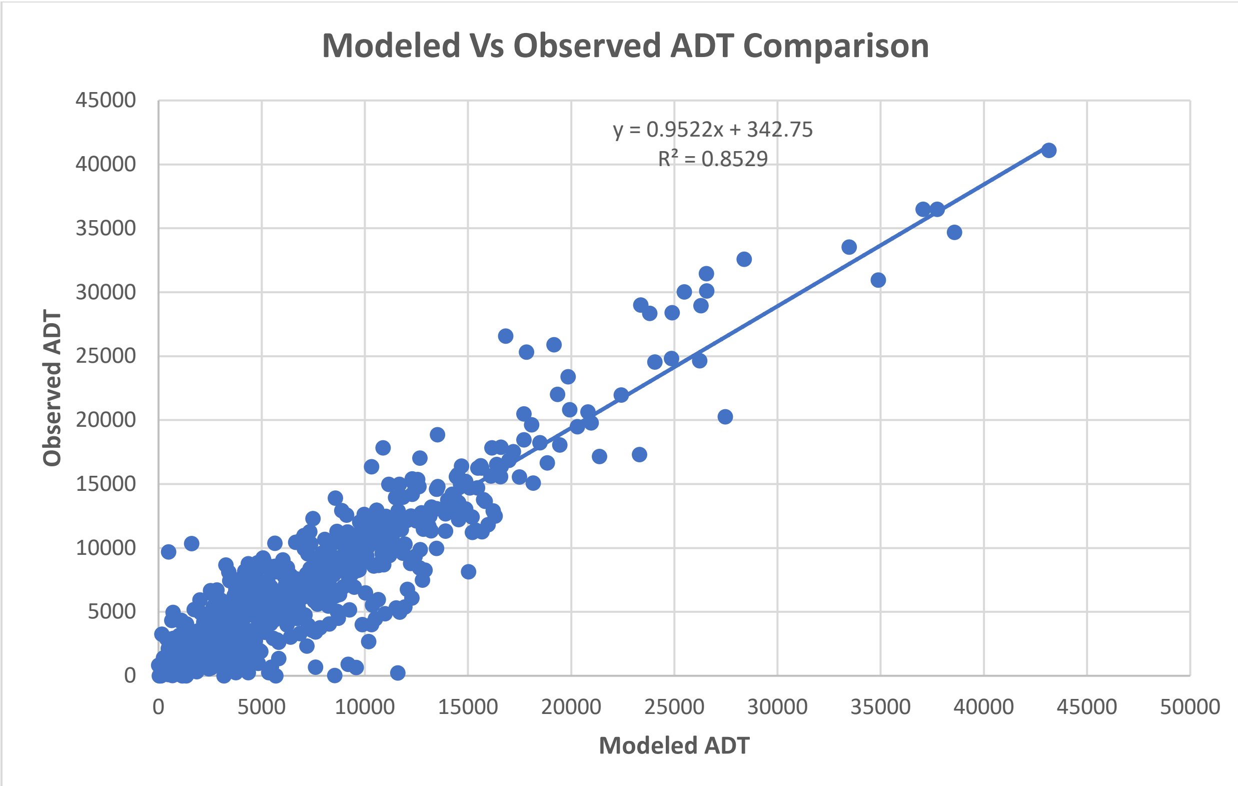

Figure 7 Scatter Plot of Modeled and Observed ADTS

9. CONCLUSIONS

In conclusion, the travel demand modeling process for the Bismarck Mandan Metropolitan Planning Organization (B-M MPO) has been conducted with a meticulous approach, adhering to the current state-of-the-art methodologies within the field. This comprehensive process encompassed the essential steps of trip generation, trip distribution, modal split, and trip assignment, each carefully executed to build a model that reflects the intricate dynamics of the region's transportation system.

The trip generation phase accurately estimated the number of trips starting and ending in each TAZ, employing advanced regression techniques and the inclusion of long-haul freight movements—a notable enhancement. The trip distribution step, underpinned by the gravity model, effectively assigned these trips between OD pairs, with friction factors finely tuned to match observed trip length frequencies.

During the trip assignment phase, the user equilibrium method was adeptly applied to simulate realistic routing choices based on individual travel times, thus providing a robust estimate of traffic flows for different periods of the day. The model's performance was particularly exemplified by its close replication of observed VMT distributions and trip length frequencies, indicating a high degree of accuracy in capturing travel patterns.

Throughout the calibration and validation processes, a series of iterative adjustments ensured that the model's output aligned with real-world data, reflecting the true state of travel in the B-M MPO area. The calibration honed in on key parameters such as trip generation rates and VMT, while validation processes confirmed the model's predictive reliability across various traffic scenarios.

The culmination of this rigorous modeling effort is a robust tool, now finely tuned and validated, that stands as a testament to the application of contemporary best practices in travel demand forecasting. This model is poised to inform strategic decision-making, support sustainable transportation initiatives, and guide the development of infrastructure that meets the current and future needs of the Bismarck Mandan Regional MPO area.

10.APPENDIX

Table 20 Calculated Capacities for Signalized Intersections for Different Functional Classifications

Lane Group	Number of Through Lanes (N)	Number of Left Turn Lanes	Number of Right Turn Lanes	Total Number of Through Lanes	Type of Arterial	Area Type	Area Type Adjustment Factor (f_a)	Base Saturation Flow Rate (S_o)	Heavy Vehicle Adjustment Factor (f_{HV})	Saturation Flow Rate for Through Lanes (S)	Total Saturation Flow Rate	Effective Green Ratio (g_i/C)	Intersection Approach Hourly Capacity (C_A)	Intersection Daily Approach Capacity
N0	1	0	0	1	Principal	Urban	0.9	1900	0.90	1416	1416	0.55	779	7,787
	1	0	0			Rural	1	1900	0.90	1505	1505	0.55	828	8,276
	1	0	0		Minor	Urban	0.9	1900	0.90	1416	1416	0.45	637	6,371
	1	0	0			Rural	1	1900	0.90	1505	1505	0.45	677	6,772
	1	0	0		Collector	Urban	0.9	1900	0.99	1308	1308	0.4	523	5,233
	1	0	0			Rural	1	1900	0.99	1390	1390	0.4	556	5,562
	2	0	0	2	Principal	Urban	0.9	1900	0.90	2832	2832	0.55	1557	15,575
	2	0	0			Rural	1	1900	0.90	3010	3010	0.55	1655	16,553
	2	0	0		Minor	Urban	0.9	1900	0.90	2832	2832	0.45	1274	12,743
	2	0	0			Rural	1	1900	0.90	3010	3010	0.45	1354	13,543
	2	0	0		Collector	Urban	0.9	1900	0.99	2866	2866	0.4	1146	11,463
	2	0	0			Rural	1	1900	0.99	3046	3046	0.4	1218	12,183
	3	0	0	3	Principal	Urban	0.9	1900	0.90	4248	4248	0.55	2336	23,362
	3	0	0			Rural	1	1900	0.90	4514	4514	0.55	2483	24,829
	3	0	0		Minor	Urban	0.9	1900	0.90	4248	4248	0.45	1911	19,114
	3	0	0			Rural	1	1900	0.90	4514	4514	0.45	2031	20,315

Lane Group	Number of Through Lanes (N)	Number of Left Turn Lanes	Number of Right Turn Lanes	Total Number of Through Lanes	Type of Arterial	Area Type	Area Type Adjustment Factor (f_a)	Base Saturation Flow Rate (S_o)	Heavy Vehicle Adjustment Factor (f_{HV})	Saturation Flow Rate for Through Lanes (S)	Total Saturation Flow Rate	Effective Green Ratio (g_i/C)	Intersection Approach Hourly Capacity (C_A)	Intersection Daily Approach Capacity
	3	0	0		**Collector**	**Urban**	**0.9**	**1900**	0.99	4439	4439	**0.4**	1776	**17,755**
	3	0	0			**Rural**	**1**	**1900**	0.99	4718	4718	**0.4**	1887	**18,870**
N1	1	1	0	2	Principal	Urban	0.9	1900	0.90	1416	1841	0.55	1012	**10,124**
	1	1	0			Rural	1	1900	0.90	1505	1956	0.55	1076	**10,759**
	1	1	0		Minor	Urban	0.9	1900	0.90	1416	1841	0.45	828	**8,283**
	1	1	0			Rural	1	1900	0.90	1505	1956	0.45	880	**8,803**
	1	1	0		**Collector**	**Urban**	**0.9**	**1900**	0.99	1433	1863	**0.4**	745	**7,451**
	1	1	0			**Rural**	**1**	**1900**	0.99	1523	1980	**0.4**	792	**7,919**
	2	1	0	3	Principal	Urban	0.9	1900	0.90	2832	3257	0.55	1791	**17,911**
	2	1	0			Rural	1	1900	0.90	3010	3461	0.55	1904	**19,036**
	2	1	0		Minor	Urban	0.9	1900	0.90	2832	3257	0.45	1465	**14,654**
	2	1	0			Rural	1	1900	0.90	3010	3461	0.45	1557	**15,575**
	2	1	0		**Collector**	**Urban**	**0.9**	**1900**	0.99	2959	3403	**0.4**	1361	**13,612**
	2	1	0			**Rural**	**1**	**1900**	0.99	3145	3617	**0.4**	1447	**14,467**
	3	1	0	4	Principal	Urban	0.9	1900	0.90	4248	4672	0.55	2570	**25,698**
	3	1	0			Rural	1	1900	0.90	4514	4966	0.55	2731	**27,312**
	3	1	0		Minor	Urban	0.9	1900	0.90	4248	4672	0.45	2103	**21,026**
	3	1	0			Rural	1	1900	0.90	4514	4966	0.45	2235	**22,346**
	3	1	0		**Collector**	**Urban**	**0.9**	**1900**	0.99	4486	4934	**0.4**	1974	**19,736**
	3	1	0			**Rural**	**1**	**1900**	0.99	4767	5244	**0.4**	2098	**20,976**

Lane Group	**Number of Through Lanes (N)**	**Number of Left Turn Lanes**	**Number of Right Turn Lanes**	**Total Number of Through Lanes**	**Type of Arterial**	**Area Type**	**Area Type Adjustment Factor (f_a)**	**Base Saturation Flow Rate (S_o)**	**Heavy Vehicle Adjustment Factor (f_{HV})**	**Saturation Flow Rate for Through Lanes (S)**	**Total Saturation Flow Rate**	**Effective Green Ratio (g_i/C)**	**Intersection Approach Hourly Capacity (C_A)**	**Intersection Daily Approach Capacity**
N2	1	2	0	3	Principal	Urban	0.9	1900	0.90	1416	2265	0.55	1246	**12,460**
	1	2	0			Rural	1	1900	0.90	1505	2408	0.55	1324	**13,242**
	1	2	0		Minor	Urban	0.9	1900	0.90	1416	2265	0.45	1019	**10,194**
	1	2	0			Rural	1	1900	0.90	1505	2408	0.45	1083	**10,835**
	1	2	0		**Collector**	**Urban**	**0.9**	**1900**	0.99	1480	2367	**0.4**	947	**9,469**
	1	2	0			**Rural**	**1**	**1900**	0.99	1573	2516	**0.4**	1006	**10,064**
	2	2	0	4	Principal	Urban	0.9	1900	0.90	2832	3681	0.55	2025	**20,247**
	2	2	0			Rural	1	1900	0.90	3010	3912	0.55	2152	**21,519**
	2	2	0		Minor	Urban	0.9	1900	0.90	2832	3681	0.45	1657	**16,566**
	2	2	0			Rural	1	1900	0.90	3010	3912	0.45	1761	**17,606**
	2	2	0		**Collector**	**Urban**	**0.9**	**1900**	0.99	2990	3887	**0.4**	1555	**15,550**
	2	2	0			**Rural**	**1**	**1900**	0.99	3178	4132	**0.4**	1653	**16,526**
	3	2	0	5	Principal	Urban	0.9	1900	0.90	4248	5097	0.55	2803	**28,034**
	3	2	0			Rural	1	1900	0.90	4514	5417	0.55	2980	**29,795**
	3	2	0		Minor	Urban	0.9	1900	0.90	4248	5097	0.45	2294	**22,937**
	3	2	0			Rural	1	1900	0.90	4514	5417	0.45	2438	**24,378**
	3	2	0		**Collector**	**Urban**	**0.9**	**1900**	0.99	4532	5439	**0.4**	2175	**21,755**
	3	2	0			**Rural**	**1**	**1900**	0.99	4817	5780	**0.4**	2312	**23,121**
N3	1	1	0	2	Principal	Urban	0.9	1900	0.90	1416	1841	0.55	1012	**10,124**
	1	1	0			Rural	1	1900	0.90	1505	1956	0.55	1076	**10,759**

Lane Group	Number of Through Lanes (N)	Number of Left Turn Lanes	Number of Right Turn Lanes	Total Number of Through Lanes	Type of Arterial	Area Type	Area Type Adjustment Factor (f_a)	Base Saturation Flow Rate (S_o)	Heavy Vehicle Adjustment Factor (f_{HV})	Saturation Flow Rate for Through Lanes (S)	Total Saturation Flow Rate	Effective Green Ratio (g_i/C)	Intersection Approach Hourly Capacity (C_A)	Intersection Daily Approach Capacity
	1	1	0		Minor	Urban	0.9	1900	0.90	1416	1841	0.45	828	**8,283**
	1	1	0			Rural	1	1900	0.90	1505	1956	0.45	880	**8,803**
	1	1	0		**Collector**	**Urban**	**0.9**	**1900**	0.99	1433	1863	**0.4**	745	**7,451**
	1	1	0			**Rural**	**1**	**1900**	0.99	1523	1980	**0.4**	792	**7,919**
	2	1	0	3	Principal	Urban	0.9	1900	0.90	2832	3257	0.55	1791	**17,911**
	2	1	0			Rural	1	1900	0.90	3010	3461	0.55	1904	**19,036**
	2	1	0		Minor	Urban	0.9	1900	0.90	2832	3257	0.45	1465	**14,654**
	2	1	0			Rural	1	1900	0.90	3010	3461	0.45	1557	**15,575**
	2	1	0		**Collector**	**Urban**	**0.9**	**1900**	0.99	2959	3403	**0.4**	1361	**13,612**
	2	1	0			**Rural**	**1**	**1900**	0.99	3145	3617	**0.4**	1447	**14,467**
	3	1	0	4	Principal	Urban	0.9	1900	0.90	4248	4672	0.55	2570	**25,698**
	3	1	0			Rural	1	1900	0.90	4514	4966	0.55	2731	**27,312**
	3	1	0		Minor	Urban	0.9	1900	0.90	4248	4672	0.45	2103	**21,026**
	3	1	0			Rural	1	1900	0.90	4514	4966	0.45	2235	**22,346**
	3	1	0		**Collector**	**Urban**	**0.9**	**1900**	0.99	4486	4934	**0.4**	1974	**19,736**
	3	1	0			**Rural**	**1**	**1900**	0.99	4767	5244	**0.4**	2098	**20,976**
N4	1	0	1	2	Principal	Urban	0.9	1900	0.90	1416	1557	0.55	857	**8,566**
	1	0	1			Rural	1	1900	0.90	1505	1655	0.55	910	**9,104**
	1	0	1		Minor	Urban	0.9	1900	0.90	1416	1557	0.45	701	**7,009**
	1	0	1			Rural	1	1900	0.90	1505	1655	0.45	745	**7,449**

Lane Group	Number of Through Lanes (N)	Number of Left Turn Lanes	Number of Right Turn Lanes	Total Number of Through Lanes	Type of Arterial	Area Type	Area Type Adjustment Factor (f_a)	Base Saturation Flow Rate (S_o)	Heavy Vehicle Adjustment Factor (f_{HV})	Saturation Flow Rate for Through Lanes (S)	Total Saturation Flow Rate	Effective Green Ratio (g_i/C)	Intersection Approach Hourly Capacity (C_A)	Intersection Daily Approach Capacity
	1	0	1		**Collector**	**Urban**	**0.9**	**1900**	0.99	1433	1576	**0.4**	630	**6,305**
	1	0	1			**Rural**	**1**	**1900**	0.99	1523	1675	**0.4**	670	**6,701**
	2	0	1	3	Principal	Urban	0.9	1900	0.90	2832	2973	0.55	1635	**16,353**
	2	0	1			Rural	1	1900	0.90	3010	3160	0.55	1738	**17,380**
	2	0	1		Minor	Urban	0.9	1900	0.90	2832	2973	0.45	1338	**13,380**
	2	0	1			Rural	1	1900	0.90	3010	3160	0.45	1422	**14,220**
	2	0	1		**Collector**	**Urban**	**0.9**	**1900**	0.99	2959	3107	**0.4**	1243	**12,429**
	2	0	1			**Rural**	**1**	**1900**	0.99	3145	3302	**0.4**	1321	**13,209**
	3	0	1	4	Principal	Urban	0.9	1900	0.90	4248	4389	0.55	2414	**24,141**
	3	0	1			Rural	1	1900	0.90	4514	4665	0.55	2566	**25,657**
	3	0	1		Minor	Urban	0.9	1900	0.90	4248	4389	0.45	1975	**19,752**
	3	0	1			Rural	1	1900	0.90	4514	4665	0.45	2099	**20,992**
	3	0	1		**Collector**	**Urban**	**0.9**	**1900**	0.99	4486	4635	**0.4**	1854	**18,540**
	3	0	1			**Rural**	**1**	**1900**	0.99	4767	4926	**0.4**	1970	**19,704**
N5	1	0	2	3	Principal	Urban	0.9	1900	0.90	1416	1699	0.55	934	**9,345**
	1	0	2			Rural	1	1900	0.90	1505	1806	0.55	993	**9,932**
	1	0	2		Minor	Urban	0.9	1900	0.90	1416	1699	0.45	765	**7,646**
	1	0	2			Rural	1	1900	0.90	1505	1806	0.45	813	**8,126**
	1	0	2		**Collector**	**Urban**	**0.9**	**1900**	0.99	1480	1776	**0.4**	710	**7,102**
	1	0	2			**Rural**	**1**	**1900**	0.99	1573	1887	**0.4**	755	**7,548**

Lane Group	Number of Through Lanes (N)	Number of Left Turn Lanes	Number of Right Turn Lanes	Total Number of Through Lanes	Type of Arterial	Area Type	Area Type Adjustment Factor (f_a)	Base Saturation Flow Rate (S_o)	Heavy Vehicle Adjustment Factor (f_{HV})	Saturation Flow Rate for Through Lanes (S)	Total Saturation Flow Rate	Effective Green Ratio (g_i/C)	Intersection Approach Hourly Capacity (C_A)	Intersection Daily Approach Capacity
	2	0	2	4	Principal	Urban	0.9	1900	0.90	2832	3115	0.55	1713	**17,132**
	2	0	2			Rural	1	1900	0.90	3010	3311	0.55	1821	**18,208**
	2	0	2		Minor	Urban	0.9	1900	0.90	2832	3115	0.45	1402	**14,017**
	2	0	2			Rural	1	1900	0.90	3010	3311	0.45	1490	**14,898**
	2	0	2		**Collector**	**Urban**	**0.9**	**1900**	0.99	2990	3289	**0.4**	1316	**13,157**
	2	0	2			**Rural**	**1**	**1900**	0.99	3178	3496	**0.4**	1398	**13,984**
	3	0	2	5	Principal	Urban	0.9	1900	0.90	4248	4531	0.55	2492	**24,919**
	3	0	2			Rural	1	1900	0.90	4514	4815	0.55	2648	**26,484**
	3	0	2		Minor	Urban	0.9	1900	0.90	4248	4531	0.45	2039	**20,389**
	3	0	2			Rural	1	1900	0.90	4514	4815	0.45	2167	**21,669**
	3	0	2		**Collector**	**Urban**	**0.9**	**1900**	0.99	4532	4834	**0.4**	1934	**19,338**
	3	0	2			**Rural**	**1**	**1900**	0.99	4817	5138	**0.4**	2055	**20,552**
N6	1	0	1	2	Principal	Urban	0.9	1900	0.90	1416	1557	0.55	857	**8,566**
	1	0	1			Rural	1	1900	0.90	1505	1655	0.55	910	**9,104**
	1	0	1		Minor	Urban	0.9	1900	0.90	1416	1557	0.45	701	**7,009**
	1	0	1			Rural	1	1900	0.90	1505	1655	0.45	745	**7,449**
	1	0	1		**Collector**	**Urban**	**0.9**	**1900**	0.99	1433	1576	**0.4**	630	**6,305**
	1	0	1			**Rural**	**1**	**1900**	0.99	1523	1675	**0.4**	670	**6,701**
	2	0	1	3	Principal	Urban	0.9	1900	0.90	2832	2973	0.55	1635	**16,353**
	2	0	1			Rural	1	1900	0.90	3010	3160	0.55	1738	**17,380**

Lane Group	Number of Through Lanes (N)	Number of Left Turn Lanes	Number of Right Turn Lanes	Total Number of Through Lanes	Type of Arterial	Area Type	Area Type Adjustment Factor (f_a)	Base Saturation Flow Rate (S_o)	Heavy Vehicle Adjustment Factor (f_{HV})	Saturation Flow Rate for Through Lanes (S)	Total Saturation Flow Rate	Effective Green Ratio (g_i/C)	Intersection Approach Hourly Capacity (C_A)	Intersection Daily Approach Capacity
	2	0	1		Minor	Urban	0.9	1900	0.90	2832	2973	0.45	1338	**13,380**
	2	0	1			Rural	1	1900	0.90	3010	3160	0.45	1422	**14,220**
	2	0	1		**Collector**	**Urban**	**0.9**	**1900**	0.99	2959	3107	**0.4**	1243	**12,429**
	2	0	1			**Rural**	**1**	**1900**	0.99	3145	3302	**0.4**	1321	**13,209**
	3	0	1	4	Principal	Urban	0.9	1900	0.90	4248	4389	0.55	2414	**24,141**
	3	0	1			Rural	1	1900	0.90	4514	4665	0.55	2566	**25,657**
	3	0	1		Minor	Urban	0.9	1900	0.90	4248	4389	0.45	1975	**19,752**
	3	0	1			Rural	1	1900	0.90	4514	4665	0.45	2099	**20,992**
	3	0	1		**Collector**	**Urban**	**0.9**	**1900**	0.99	4486	4635	**0.4**	1854	**18,540**
	3	0	1			**Rural**	**1**	**1900**	0.99	4767	4926	**0.4**	1970	**19,704**
N7	1	1	1	3	Principal	Urban	0.9	1900	0.90	1416	1982	0.55	1090	**10,902**
	1	1	1			Rural	1	1900	0.90	1505	2107	0.55	1159	**11,587**
	1	1	1		Minor	Urban	0.9	1900	0.90	1416	1982	0.45	892	**8,920**
	1	1	1			Rural	1	1900	0.90	1505	2107	0.45	948	**9,480**
	1	1	1		**Collector**	**Urban**	**0.9**	**1900**	0.99	1480	2071	**0.4**	829	**8,286**
	1	1	1			**Rural**	**1**	**1900**	0.99	1573	2202	**0.4**	881	**8,806**
	2	1	1	4	Principal	Urban	0.9	1900	0.90	2832	3398	0.55	1869	**18,690**
	2	1	1			Rural	1	1900	0.90	3010	3612	0.55	1986	**19,863**
	2	1	1		Minor	Urban	0.9	1900	0.90	2832	3398	0.45	1529	**15,292**
	2	1	1			Rural	1	1900	0.90	3010	3612	0.45	1625	**16,252**

Lane Group	Number of Through Lanes (N)	Number of Left Turn Lanes	Number of Right Turn Lanes	Total Number of Through Lanes	Type of Arterial	Area Type	Area Type Adjustment Factor (f_a)	Base Saturation Flow Rate (S_o)	Heavy Vehicle Adjustment Factor (f_{HV})	Saturation Flow Rate for Through Lanes (S)	Total Saturation Flow Rate	Effective Green Ratio (g_i/C)	Intersection Approach Hourly Capacity (C_A)	Intersection Daily Approach Capacity
	2	1	1		**Collector**	**Urban**	**0.9**	**1900**	0.99	2990	3588	**0.4**	1435	**14,354**
	2	1	1			**Rural**	**1**	**1900**	0.99	3178	3814	**0.4**	1526	**15,255**
	3	1	1	5	Principal	Urban	0.9	1900	0.90	4248	4814	0.55	2648	**26,477**
	3	1	1			Rural	1	1900	0.90	4514	5116	0.55	2814	**28,140**
	3	1	1		Minor	Urban	0.9	1900	0.90	4248	4814	0.45	2166	**21,663**
	3	1	1			Rural	1	1900	0.90	4514	5116	0.45	2302	**23,023**
	3	1	1		**Collector**	**Urban**	**0.9**	**1900**	0.99	4532	5137	**0.4**	2055	**20,546**
	3	1	1			**Rural**	**1**	**1900**	0.99	4817	5459	**0.4**	2184	**21,836**
N8	1	2	1	4	Principal	Urban	0.9	1900	0.90	1416	2407	0.55	1324	**13,238**
	1	2	1			Rural	1	1900	0.90	1505	2558	0.55	1407	**14,070**
	1	2	1		Minor	Urban	0.9	1900	0.90	1416	2407	0.45	1083	**10,831**
	1	2	1			Rural	1	1900	0.90	1505	2558	0.45	1151	**11,512**
	1	2	1		**Collector**	**Urban**	**0.9**	**1900**	0.99	1495	2542	**0.4**	1017	**10,167**
	1	2	1			**Rural**	**1**	**1900**	0.99	1589	2701	**0.4**	1081	**10,806**
	2	2	1	5	Principal	Urban	0.9	1900	0.90	2832	3823	0.55	2103	**21,026**
	2	2	1			Rural	1	1900	0.90	3010	4063	0.55	2235	**22,346**
	2	2	1		Minor	Urban	0.9	1900	0.90	2832	3823	0.45	1720	**17,203**
	2	2	1			Rural	1	1900	0.90	3010	4063	0.45	1828	**18,283**
	2	2	1		**Collector**	**Urban**	**0.9**	**1900**	0.99	3021	4079	**0.4**	1632	**16,316**
	2	2	1			**Rural**	**1**	**1900**	0.99	3211	4335	**0.4**	1734	**17,341**

Lane Group	**Number of Through Lanes (N)**	**Number of Left Turn Lanes**	**Number of Right Turn Lanes**	**Total Number of Through Lanes**	**Type of Arterial**	**Area Type**	**Area Type Adjustment Factor (f_a)**	**Base Saturation Flow Rate (S_o)**	**Heavy Vehicle Adjustment Factor (f_{HV})**	**Saturation Flow Rate for Through Lanes (S)**	**Total Saturation Flow Rate**	**Effective Green Ratio (g_i/C)**	**Intersection Approach Hourly Capacity (C_A)**	**Intersection Daily Approach Capacity**
	3	2	1	6	Principal	Urban	0.9	1900	0.90	4248	5239	0.55	2881	**28,813**
	3	2	1			Rural	1	1900	0.90	4514	5568	0.55	3062	**30,623**
	3	2	1		Minor	Urban	0.9	1900	0.90	4248	5239	0.45	2357	**23,574**
	3	2	1			Rural	1	1900	0.90	4514	5568	0.45	2505	**25,055**
	3	2	1		**Collector**	**Urban**	**0.9**	**1900**	0.99	4532	5590	**0.4**	2236	**22,359**
	3	2	1			**Rural**	**1**	**1900**	0.99	4817	5941	**0.4**	2376	**23,763**
N9	1	1	2	4	Principal	Urban	0.9	1900	0.90	1416	2124	0.55	1168	**11,681**
	1	1	2			Rural	1	1900	0.90	1505	2257	0.55	1241	**12,415**
	1	1	2		Minor	Urban	0.9	1900	0.90	1416	2124	0.45	956	**9,557**
	1	1	2			Rural	1	1900	0.90	1505	2257	0.45	1016	**10,157**
	1	1	2		**Collector**	**Urban**	**0.9**	**1900**	0.99	1495	2243	**0.4**	897	**8,971**
	1	1	2			**Rural**	**1**	**1900**	0.99	1589	2384	**0.4**	953	**9,534**
	2	1	2	5	Principal	Urban	0.9	1900	0.90	2832	3540	0.55	1947	**19,468**
	2	1	2			Rural	1	1900	0.90	3010	3762	0.55	2069	**20,691**
	2	1	2		Minor	Urban	0.9	1900	0.90	2832	3540	0.45	1593	**15,929**
	2	1	2			Rural	1	1900	0.90	3010	3762	0.45	1693	**16,929**
	2	1	2		**Collector**	**Urban**	**0.9**	**1900**	0.99	3021	3777	**0.4**	1511	**15,107**
	2	1	2			**Rural**	**1**	**1900**	0.99	3211	4014	**0.4**	1606	**16,056**
	3	1	2	6	Principal	Urban	0.9	1900	0.90	4248	4956	0.55	2726	**27,256**
	3	1	2			Rural	1	1900	0.90	4514	5267	0.55	2897	**28,967**

Lane Group	Number of Through Lanes (N)	Number of Left Turn Lanes	Number of Right Turn Lanes	Total Number of Through Lanes	Type of Arterial	Area Type	Area Type Adjustment Factor (f_a)	Base Saturation Flow Rate (S_o)	Heavy Vehicle Adjustment Factor (f_{HV})	Saturation Flow Rate for Through Lanes (S)	Total Saturation Flow Rate	Effective Green Ratio (g_i/C)	Intersection Approach Hourly Capacity (C_A)	Intersection Daily Approach Capacity
	3	1	2		Minor	Urban	0.9	1900	0.90	4248	4956	0.45	2230	**22,300**
	3	1	2			Rural	1	1900	0.90	4514	5267	0.45	2370	**23,701**
	3	1	2		**Collector**	**Urban**	**0.9**	**1900**	0.99	4532	5288	**0.4**	2115	**21,150**
	3	1	2			**Rural**	**1**	**1900**	0.99	4817	5620	**0.4**	2248	**22,479**

Table 21 Calculated Capacities for Ramps

	Speed	Ideal Capacity (Ex 13-10)	Speed Adjustment	V/C	PHF	Capacity	Daily Capacity
Urban	>50	2,100	1.00	0.9	0.800	1,512	15,120
	>40-50	2,100	0.95	0.9	0.800	1,443	14,433
	>30-40	2,100	0.91	0.9	0.800	1,375	13,745
	>=20-30	2,100	0.86	0.9	0.800	1,306	13,058
	<20	2,100	0.82	0.9	0.800	1,237	12,371
Rural	>50	2,200	1.00	0.9	0.868	1,719	17,186
	>40-50	2,200	0.95	0.9	0.868	1,641	16,405
	>30-40	2,200	0.91	0.9	0.868	1,562	15,622
	>=20-30	2,200	0.86	0.9	0.868	1,484	14,843
	<20	2,200	0.82	0.9	0.868	1,406	14,062