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## $9^{\text {th }}$ St. $/ 57^{\text {th }}$ St. Corridor Analysis

## Final Report

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## BACKGROUND

The southwest portion of the Fargo-Moorhead metropolitan area has experienced significant growth over the past several years. As a result, traffic congestion has also developed within this area, as well as along corridors accessing other portions of the metropolitan area. Sheyenne St. (County Highway 17) serves as the major north/south arterial for West Fargo, ND, having an average daily traffic (ADT) as high as 14,450 vehicles per day (NDDOT, 2007). Sheyenne St. is the only arterial providing direct access from the southwest portion of West Fargo, ND, to Interstate 94 (I-94) and the northern part of the city. The next closest alternative route is $45^{t h}$ St. in Fargo, ND, which is two miles to the east. Over the past several years, the Sheyenne St. and I-94 Interchange has been experiencing increased traffic congestion during the peak-hour periods.

To help alleviate the congestion at the Sheyenne St. and I-94 Interchange and provide a more convenient alternative for the deployments between Sheyenne St. and $45^{\text {th }}$ St., an interchange at $9^{\text {th }} \mathrm{St} . / 57^{\text {th }}$ St. will be constructed in 2009. Originally, this interchange design only consisted of an overpass; however, it will now include the overpass and interchange ramps. With the ramp inclusions, the traffic volume and resulting congestion at the Sheyenne St. interchanges could be significantly reduced at least in the short to medium term. In addition, the $45^{\text {th }} \mathrm{St}$. and I-94 Interchange should experience less traffic due to the additional interchange.

The $9^{\text {th }}$ St. $/ 57^{\text {th }}$ St. corridor will serve as a north/south arterial roadway. A challenge facing transportation agencies relates to providing adequate capacity to reasonably accommodate the projected, long-term traffic volume while having access to limited funds. This study will provide insight in selecting an appropriate roadway design.

## OBJECTIVES

The main objective of this study is to determine if a four-lane roadway will provide adequate capacity for the $9^{\text {th }}$ St. $/ 57^{\text {th }}$ St. corridor between I-94 and $52^{\text {nd }}$ Ave. S. In addition, the level of congestion along $9^{\text {th }}$ St. $/ 57^{\text {th }}$ St. will be evaluated at the intersections of $23^{\text {rd }}$ Ave. S. and $32^{\text {nd }}$ Ave. S.

## METHODOLY

This study will use both planning and operational level analysis tools to analyze the $9^{\text {th }}$ St. $/ 57^{\text {th }}$ St. corridor. On the planning level, the Fargo-Moorhead Metro Council of Government's (Metro COG) travel demand model will be used to provide the estimated 2035 PM peak-hour traffic. The traffic volume from the travel demand model will take into account for the projected socio-economic data (employment and households) of the metropolitan area. CORSIM, a microscopic simulation program, will be used to evaluate the corridor operations based on the projected traffic volume, network geometry, and traffic control. Measures of effectiveness (MOE) will be compared among the scenarios, and include network delay time, intersection delay time, and maximum queue length.

## Analysis Scenarios

This study will have four analysis scenarios for estimating the projected PM peak-hour conditions. Since the main objective of this study is to determine if a four-lane facility will provide enough capacity during the peak-hour conditions, various traffic levels will be evaluated. Since it is difficult to estimate the level of development 25+ years into the future, various traffic levels will be evaluated. In addition to the modeled PM peak-hour traffic, growth factors of $1.33,1.67$, and 2.0 will be used. Although, these growth factors provide significant increases to the modeled traffic, it was desired to determine the oversaturated conditions of the four-lane facility. The four analysis scenarios used for this study are as follows:

- 2035 PM Peak - Base Case: Projected 2035 PM traffic using a 4-lane facility with turning lanes ( $9^{\text {th }} \mathrm{St} . / 57^{\text {th }}$ St. between I-94 and $52^{\text {nd }}$ Ave. S.) and incorporating optimized traffic signal plans.
- 2035 PM Peak - 1.33 Growth Factor: Increasing the projected 2035 PM traffic by 33 percent using a 4-lane facility with turning lanes ( $9^{\text {th }} \mathrm{St} . / 57^{\text {th }}$ St. between I-94 and $52^{\text {nd }}$ Ave. S.) and incorporating optimized traffic signal plans.
- 2035 PM Peak - 1.67 Growth Factor: Increasing the projected 2035 PM traffic by 67 percent using a 4-lane facility with turning lanes ( $9^{\text {th }}$ St. $/ 57^{\text {th }}$ St. between I-94 and $52^{\text {nd }}$ Ave. S.) and incorporating optimized traffic signal plans.
- 2035 PM Peak - 2.00 Growth Factor: Increasing the projected 2035 PM traffic by 100 percent using a 4-lane facility with turning lanes ( $9^{\text {th }} \mathrm{St} . / 57^{\text {th }}$ St. between I-94 and $52^{\text {nd }}$ Ave. S.) and incorporating optimized traffic signal plans.


## Network Geometry

The analysis network spans over three miles and includes eight intersections (Figure 1). Other intersections may exist along this corridor, but this analysis will capture all of the presumed signalized intersections. This analysis will incorporate a six-lane facility between the I-94 North Ramp and $23^{\text {rd }}$ Ave. S. and a four-lane facility south of $23^{\text {rd }}$ Ave. S. The I-94 Interchange will consist of a diamond interchange with a loop ramp in the southwest quadrant (southbound to eastbound movement). In addition, turn lanes will be available at all of the intersections, ranging from 200 feet on the side-street approaches to 350 feet on major-street approaches. The corridor will be modeled using a speed limit of 35 mph .


Figure 1. $9^{\text {th }} \mathrm{St} . / 57^{\text {th }}$ St. corridor (4-lane facility with turn lanes)

## Traffic Volume

This study will analyze the projected 2035 PM peak-hour traffic (which accounts for approximately 8.5\% of the ADT) and variations of this traffic using growth factors of $1.33,1.67$, and 2.00 . The 2035 average vehicles per day for this corridor decreases from north to south and ranges from 11,000 to 26,000 vehicles per day (shown in Appendix A). The PM peak-hour turning moment volumes along with the boundary approach volumes are shown in Figure 2. The traffic volume for the growth factor scenarios would equate to taking the volumes from the base case and multiplying them by the respective growth factor. Using a percentage of the ADT of $8.5 \%$ to account for the base case PM peak hour, the growth factors of $1.33,1.67$, and 2.00 , would provide ADT percentages of $11.3,14.7$, and 17 , respectively.


Figure 2. Projected 2035 PM peak-hour traffic volume

## Traffic Control

The analysis network consists of eight signalized intersections having signal phasing ranging from two phases to eight phases. The I-94 South Ramp is the only intersection with two-phase control since it does not have any left-turn phases. In addition, this intersection incorporates a free movement for the traffic making a southbound right-turn movement, which would be using the southwest loop ramp. All of the remaining intersections incorporate north and south left-turn phases, while the intersections of $32^{\text {nd }}$ Ave. S. and $52^{\text {nd }}$ Ave. S. also incorporate east and west left-turn phases. All of the left-turn phases will operate under protected/permitted control.

The Synchro traffic signal analysis program will be used to develop optimized timing plans for each traffic volume scenario. An actuated-coordinated system will be incorporated for the five signals between the I94 North Ramp and $32^{\text {nd }}$ Ave. S. The remaining signals will operate under actuated-uncoordinated control.

## TRAFFIC SIMULATION

Traffic simulation models allow practitioners to evaluate different scenarios prior to field implementation by replicating current and proposed traffic volume, network geometry, and traffic control devices. This study will use CORSIM (TSIS 6.0), which is a microscopic, stochastic, traffic simulation model, developed for the Federal Highway Administration. CORSIM provides numerical and visual output to assess the operational conditions of a transportation network, such as queue lengths and delay time.

The input parameters for CORSIM include roadway geometry, turning movement counts, and traffic control. The peak-hour traffic will consist of 4, 15-minute time periods. To provide a degree of peak flow within the peak hour, a peak-hour factor (PHF) will be incorporated. A PHF of .85 will be used for the second 15 -minute period. The other three time periods will use an anti-PHF to simulate the remaining peak traffic within the hour.

Each scenario will have a seed time of 10 minutes followed by a 60-minute simulation. The seed time loads vehicles into the network while not producing simulation output. In addition, each scenario will be simulated 30 times to normalize the results.

## SIMULATION RESULTS

The simulation analysis compared MOE for the total network, arterial, and individual intersections. The network and arterial output comparisons are beneficial for determining trends and indentifying system degradation at a larger level. The intersection output provides operational aspects at specific locations, which can assist in analyzing capacity at an approach or movement level.

## Network/Arterial Output

As expected, system performance decreases as traffic volume increases. Network delay time increased $50 \%, 114 \%$, and $232 \%$, for traffic growth factors of $33 \%, 67 \%$, and $100 \%$, respectively (Table 1). A significant delay increase was observed between traffic growth factors of $67 \%$ and $100 \%$. This could be explained by the fact that the arterial reaches over-saturated conditions (volume is greater than capacity) between these traffic scenarios. This trend is also observed for the arterial delay and arterial speed percent differences.

Table 1. Total Delay Time and Speed Comparisons

| 2035 Peak Hour <br> Traffic Growth | Network Total Delay |  | 9th St./57th St. Total Delay |  | 9th St./57th St. Speed |  |
| :---: | :---: | :---: | :---: | :---: | :---: | :---: |
|  | Hours | \% Change | Hours | \% Change | mph | \% Change |
| Base Case | 61.4 | - | 34.7 | - | 27.0 | - |
| $33 \%$ | 92.3 | 50 | 56.0 | 61 | 25.8 | -4 |
| $67 \%$ | 131.3 | 114 | 84.7 | 144 | 24.5 | -9 |
| $100 \%$ | 204.2 | 232 | 138.3 | 298 | 22.3 | -18 |

$9^{\text {th }}$ St. $/ 57^{\text {th }}$ St. $\& 23^{\text {rd }}$ Ave. S. Output
Although the intersection of $9^{\text {th }}$ St. $/ 57^{\text {th }}$ St. \& $23^{\text {rd }}$ Ave. S. does not consist of two intersecting arterials, the project team was interested in determining the operational performance of this intersection. In addition, insight into the southbound left-turn movement's queue length was desired for determining an appropriate turn-bay length. The intersection control delay for the base traffic, $33 \%$ traffic growth, $67 \%$ traffic growth, and $100 \%$ traffic growth, was $9.4,11.7,14.0$, and 27.1 seconds per vehicle, respectively (Table 2). It should be noted that the control delay time reported in CORSIM will not correlate to those using the Highway Capacity Manual (HCM) methodology. The HCM methodology uses a peak-hour factor (PHF), which provides a worst-case scenario for computing delay time, while the CORSIM output is based on the projected peak-hour volume, which is not factored.

Since this signalized intersection operated as actuated-coordinated, the north/south approaches should typically have less delay time compared to the east/west approaches since more vehicles traverse on these approaches. However, the $100 \%$ traffic growth scenario reported higher delay time for the north/south approaches due to cycle/phase failures (not all of the demand was served).

The maximum queue length data can provide insight into oversaturated conditions and guidance into adequate turn-bay lengths. The northbound through lanes experience the largest queue length values ranging from 162 feet to 853 feet (Table 3). The traffic volume for the $100 \%$ traffic growth scenario is large enough that not all of the vehicles at the intersection can be served during the cycle length, creating a substantial queue length for the peak 15 -minute period. This condition also impacts the southbound left-turn movement (vehicles can only turn during the protected left-turn phase).

Queue lengths for the southbound left-turn lane ranged from 80 feet to 242 feet. Although the base case and $33 \%$ traffic growth case have queue lengths of approximately 100 feet, turn bays of this length are not recommended on arterials. Since the through lanes have queue lengths of 98 feet to 279 feet, short turn-bay lengths may starve the left-turn traffic (these vehicles may not be able to get into the left-turn lane since they are queued in the through lane). To eliminate this occurrence during the simulation scenarios, 350 foot left-turn lanes were used.

Table 2. Control Delay for the Intersection of $9^{\text {th }}$ St. $/ 57^{\text {th }}$ St. \& $23^{\text {rd }}$ Ave. S.

| 2035 Peak Hour Traffic Growth | $9^{\text {th }}$ St. $/ 57^{\text {th }}$ St. \& $\mathbf{2 3}^{\text {rd }}$ Ave. S. Control Delay (sec/veh) |  |  |  |  |  |  |  |  |  |  |  |  |  |
| :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: |
|  | EB Approach |  |  | WB Approach |  |  | NB Approach |  |  | SB Approach |  |  | Intersection | \% Change |
|  | L | T | R | L | T | R | L | T | R | L | T | R |  |  |
| 0\% | 38.4 | 35.8 | 8.7 | 37.7 | 35.4 | 9.1 | 0.0 | 7.6 | 3.6 | 8.9 | 4.3 | 3.8 | 9.4 | - |
| 33\% | 37.7 | 31.5 | 9.8 | 36.1 | 32.0 | 13.3 | 0.0 | 12.5 | 4.3 | 10.9 | 5.6 | 3.7 | 11.7 | 25 |
| 67\% | 37.4 | 31.0 | 13.0 | 34.9 | 32.5 | 16.9 | 0.0 | 14.8 | 4.8 | 17.0 | 8.3 | 4.4 | 14.0 | 49 |
| 100\% | 41.7 | 34.3 | 18.3 | 39.1 | 35.0 | 23.3 | 0.0 | 42.5 | 21.4 | 41.1 | 11.1 | 5.6 | 27.1 | 189 |

Table 3. Maximum Queue Length for the Intersection of $9^{\text {th }}$ St. $/ 57^{\text {th }}$ St. \& $23^{\text {rd }}$ Ave. S.

| 2035 Peak Hour Traffic Growth | $9^{\text {th }}$ St. $/ 57^{\text {th }}$ St. $\& 23^{\text {rd }}$ Ave. S. <br> Max Queue Length (feet*) |  |  |  |  |  |  |  |  |  |  |  |  |
| :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: |
|  | EB Approach |  | WB Approach |  |  | NB Approach |  |  |  | SB Approach |  |  |  |
|  | L-1 | T-1 | L-1 | T-1 | R-1 | L-1 | T-1 | T-2 | R-1 | L-1 | T-1 | T-2 | R-1 |
| 0\% | 133 | 30 | 108 | 45 | 78 | 0 | 200 | 162 | 23 | 80 | 110 | 98 | 71 |
| 33\% | 153 | 35 | 125 | 51 | 119 | 0 | 310 | 266 | 32 | 101 | 186 | 176 | 71 |
| 67\% | 180 | 41 | 155 | 64 | 162 | 0 | 368 | 307 | 37 | 161 | 237 | 228 | 93 |
| 100\% | 199 | 54 | 178 | 90 | 193 | 0 | 853 | 773 | 51 | 242 | 279 | 245 | 137 |

* Calculated by multiplying the number of queued vehicles times the effective vehicle length.

$9^{\text {th }}$ St. $/ 57^{\text {th }}$ St. $\& 32^{\text {nd }}$ Ave. S. Output

The intersection of $9^{\text {th }}$ St. $/ 57^{\text {th }}$ St. \& $32^{\text {nd }}$ Ave. S. will consist of two intersecting arterials. Therefore, the traffic volume at this intersection could be substantial. Although the modeled ADT along this corridor decreases from north to south, some turning movement volumes at this intersection may be significant. Analyzing the eastbound left-turn movement's queue length was requested for selecting a turn-bay length. The intersection control delay for the base case, 33\% traffic growth, $67 \%$ traffic growth, and 100\% traffic growth, was 17.8, 19.3, 20.4, and 26.0 seconds per vehicle, respectively (Table 4).

Similar to $9^{\text {th }}$ St. $/ 57^{\text {th }}$ St. \& $23^{\text {rd }}$ Ave. S., the $9^{\text {th }}$ St. $/ 57^{\text {th }}$ St. \& $32^{\text {nd }}$ Ave. S. intersection operated as actuated-uncoordinated. As a result, the north/south approaches (which are the coordinated approaches) typically have less delay time per vehicle compared to the east/west approaches since more vehicles traverse on these approaches. To better manage the delay time and queue length of the eastbound leftturn movement, double turn lanes were used.

The eastbound left-turn movement reported the largest queue length of this intersection. The combined queue length for the eastbound left-turn movement ranged from 193 feet to 340 feet (Table 5). Queue lengths for the southbound and northbound left-turn movements ranged from 38 feet to 98 feet for all four traffic scenarios. As previously discussed, short turn bays are not recommended on arterials.

Table 4. Control Delay for the Intersection of $9^{\text {th }}$ St. $/ 57^{\text {th }}$ St. \& $32^{\text {nd }}$ Ave. S.

| 2035 Peak Hour Traffic Growth | $\begin{gathered} 9^{\text {th }} \text { St. } 157^{\text {th }} \text { St. \& } 32^{\text {nd }} \text { Ave. S. } \\ \text { Control Delay (sec/veh) } \end{gathered}$ |  |  |  |  |  |  |  |  |  |  |  |  |  |
| :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: |
|  | EB Approach |  |  | WB Approach |  |  | NB Approach |  |  | SB Approach |  |  | Intersection | \% Change |
|  | L | T | R | L | T | R | L | T | R | L | T | R |  |  |
| 0\% | 27.2 | 29.6 | 17.7 | 26.6 | 35.5 | 6.1 | 11.4 | 12.1 | 4.0 | 10.9 | 11.3 | 12.0 | 17.8 | - |
| 33\% | 26.1 | 27.3 | 19.0 | 24.3 | 35.2 | 7.7 | 14.1 | 14.4 | 4.2 | 13.5 | 14.8 | 16.2 | 19.3 | 8 |
| 67\% | 23.9 | 25.3 | 19.0 | 22.3 | 33.8 | 9.8 | 17.4 | 19.9 | 4.7 | 12.9 | 16.3 | 15.8 | 20.4 | 14 |
| 100\% | 29.6 | 30.5 | 23.2 | 27.0 | 41.2 | 13.6 | 24.3 | 23.8 | 4.5 | 20.0 | 22.1 | 24.8 | 26.0 | 46 |

Table 5. Maximum Queue Length for the Intersection of $9^{\text {th }}$ St. $/ 57^{\text {th }}$ St. \& $32^{\text {nd }}$ Ave. S.

| 2035 Peak Hour Traffic Growth | 9th St./57th St. \& 32nd Ave. S. Max Queue Length (feet*) |  |  |  |  |  |  |  |  |  |  |  |  |  |  |
| :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: | :---: |
|  | EB Approach |  |  |  | WB Approach |  |  |  | NB Approach |  |  |  | SB Approach |  |  |
|  | L-1 | L-2 | T-1 | T-2 | L-1 | T-1 | T-2 | R-1 | L-1 | T-1 | T-2 | R-1 | L-1 | T-1 | T-2 |
| 0\% | 102 | 91 | 89 | 72 | 57 | 112 | 118 | 65 | 38 | 132 | 128 | 17 | 54 | 157 | 150 |
| 33\% | 123 | 110 | 98 | 82 | 70 | 133 | 142 | 90 | 50 | 182 | 179 | 18 | 74 | 235 | 239 |
| 67\% | 134 | 123 | 110 | 91 | 80 | 156 | 161 | 113 | 56 | 237 | 239 | 19 | 76 | 288 | 305 |
| 100\% | 173 | 167 | 136 | 126 | 92 | 205 | 213 | 165 | 68 | 326 | 319 | 20 | 98 | 351 | 379 |

* Calculated by multiplying the number of queued vehicles times the effective vehicle length.


## Average Daily Traffic Analysis

The Highway Capacity Manual (HCM) provides six levels of service (LOS) thresholds to evaluate the operation of an arterial roadway (Table 6). A LOS D is typically used as a minimum threshold for designing a future urban arterial. Using the HCM methodology, a four-lane arterial with turning lanes could sustain a LOS D having traffic ranging from 24,000 to 30,000 vehicles per day. The 2035 forecasted traffic for the $9^{\text {th }}$ St. $/ 57^{\text {th }}$ St. corridor ranges from 11,000 (near $52^{\text {nd }}$ Ave. S.) to 26,000 (near I$94)$ vehicles per day.

Table 6. Level of Service Descriptions for Urban Streets

| Level of Service | Description |
| :---: | :--- |
| LOS A | Free-flow operations. Vehicles are completely unimpeded to maneuver within the <br> traffic stream. |
| LOS B | Reasonably unimpeded operations. Vehicles are slightly restricted to maneuver <br> within the traffic stream. |
| LOS C | Stable operations; however, ability to maneuver and change lanes in midblock <br> locations may be more restricted. |
| LOS D | Borders on a range in which small increases in flow may cause substantial increases <br> in delay and decreases in travel speed. |
| LOS E | Characterized by significant delays and low average travel speed. |
| LOS F | Characterized by extremely low speeds. Intersection congestion is likely at critical <br> signalized locations, with high delays, high volumes, and extensive queuing. |

Source: Highway Capacity Manual 2000 (Chapter 10: Urban Street Concepts)
For comparison purposes, the south side of $25^{\text {th }}$ St. near $17^{\text {th }}$ Ave. S. encounters ADT of around 26,000 vehicles per day (while school is in session). This roadway includes four through lanes and a two-way left-turn lane. Based on past experience, this facility (arterial) typically operates within a LOS D during the peak-hour periods.

## SUMMARYIRECOMMENDATIONS

This study analyzed the future $9^{\text {th }}$ St. $/ 57^{\text {th }}$ St. corridor using the forecasted 2035 PM peak-hour traffic, roadway geometry, and traffic control devices. This study used traffic simulation and the HCM methodology to evaluate the corridor's performance under various levels of traffic.

The simulation analysis observed the performance of the network, arterial, and critical intersections. As expected, system performance decreases as traffic volume increases. Significant delay increases were observed between traffic growth factors of $67 \%$ and $100 \%$ due to the over-saturated conditions caused by these scenarios. This trend is also observed for the arterial delay and arterial speed percent differences.

The intersection of $9^{\text {th }}$ St. $/ 57^{\text {th }}$ St. \& $23^{\text {rd }}$ Ave. S. typically operated uncongested for the four traffic scenarios. However, the northbound approach experienced significant queues due to cycle failures under the $100 \%$ traffic growth scenario. Queue lengths for the southbound left-turn lane ranged from 80 feet to 242 feet.

The intersection of $9^{\text {th }}$ St. $/ 57^{\text {th }}$ St. \& $32^{\text {nd }}$ Ave. S. also operated uncongested for all four traffic scenarios. Queue lengths for the northbound and southbound left-turn lanes ranged from 38 feet to 68 feet and 54 feet to 98 feet, respectively. The combined queue length for the eastbound left-turn movement ranged from 193 feet to 340 feet.

Although the northbound/southbound left-turn queue lengths were typically less than 100 feet for the base case and $33 \%$ traffic growth scenarios, implementing turn-bay lanes of this length are not recommended, especially on arterials. Many agencies have policies that set the minimum length of storage lengths, which range from 150 feet to 200 feet (not including the taper length).

Based on the projected 2035 ADT and peak-hour traffic, a four-lane facility with turning lanes should provide enough capacity for the $9^{\text {th }} \mathrm{St} . / 57^{\text {th }}$ St. corridor. The ADT of 26,000 vehicles per day should be accommodated by this facility if proper access management policies are followed, as well as implementing adequate traffic signal timing and phasing.

Appendix A: $9^{\text {th }}$ St. $/ 57^{\text {th }}$ St. 2035 ADT (Page 1 of 2 is provided on the following page)


