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# Two-Lane Highway Analysis Methodology Enhancements Considering Commercial Trucks

**Final Report**

by

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## Abstract

Two-lane highways are critical components of the national highway system and are continuing to see increased truck traffic along with all other components of the highway network. The presence of heavy vehicles creates more pressure on two-lane highway facility operations due to their large size and poorer performance capabilities relative to passenger cars, especially given the much more limited passing opportunities available on two-lane highways versus multilane facilities. Consequently, it is essential to have analysis methods for two-lane highways that are sensitive to the unique characteristics of commercial trucks.

The objective of this research was to improve the state-of-the-art for accounting for the impact of heavy vehicles (i.e., commercial trucks) on two-lane highway operations, particularly as it relates to the current two-lane highway analysis methodology of the Highway Capacity Manual. This was accomplished primarily by building on the work that was done for NCHRP Project 17-65. For this project, this objective was addressed in four different areas: 1) capacity on non-passing lane upgrade segments when trucks are present in traffic stream, 2) effective length of passing lane considering upgrades and trucks, 3) guidance for climbing lane design (and return to level grade conditions), and 4) passing lane diverge/merge configurations.

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# 1. Introduction

## 1.1. Background

Two-lane highways are critical components of the national highway system and are continuing to see increased truck traffic along with all other components of the highway network. The presence of heavy vehicles creates more pressure on two-lane highway facility operations due to their large size and poorer performance capabilities relative to passenger cars, especially given the much more limited passing opportunities available on two-lane highways versus multilane facilities. Consequently, it is essential to have analysis methods for two-lane highways that are sensitive to the unique characteristics of commercial trucks.

A significant revision to the Highway Capacity Manual (HCM) two-lane highway analysis methodology was completed in 2018, as part of NCHRP Project 17-65, "Improved Analysis of Two-Lane Highway Capacity and Operational Performance" (Washburn et al., 2018). This revised analysis methodology is now incorporated in the 7$^{th}$ edition of the HCM (Transportation Research Board, 2022). The updated analysis methodology in the HCM addresses several limitations and gaps that were present in the previous two-lane highway analysis methodology of the HCM. However, there are still several key areas in the updated analysis methodology where further investigation is warranted:

- Capacity on non-passing lane upgrade segments when trucks are present in traffic stream
  - Field data collected as part of NCHRP Project 17-65 did not yield enough very high flow rate conditions to make meaningful insights into the concept of capacity. Capacity was investigated, through simulation, for passing lane segments (which was constrained by the downstream merging operations). However, capacity, for which trucks can have a significant influence, was not examined for non-passing lane segments. This issue needs further examination, particularly for non-level terrain.
- Effective length of passing lane
  - In NCHRP Project 17-65, quantification of the effective length of a passing lane (i.e., distance downstream of the passing lane for which the improvements to the performance measures extend) was only determined for level terrain. On non-level terrain, commercial trucks can have a significant impact on this distance. Additional quantitative guidance is needed for the effective length of passing lanes on non-level terrain when trucks are present in the traffic stream.
- Guidance for climbing lane design (length and return to level grade conditions)
  - The merging behavior of trucks at the end of a passing lane segment can be even more problematic on upgrades (i.e., a climbing lane), as the speed differential between the trucks and passenger cars can be quite significant. The AASHTO Green Book recommends that a passing lane on a grade be continued onto a relatively level segment of roadway until the truck speeds are at a minimum of 40 mi/h and within 10 mi/h of the average passenger car speed. Some quantitative guidance on expected lengths of passing lane needed to achieve smooth reintegration of trucks to the regular lane, based on overall flow rate, grade %, and truck %, is needed.
- Passing lane performance for various diverge/merge rules
  - Many passing lane configurations require "slower drivers keep right", which usually entails that the slower vehicles move over to the added lane and merge back to the regular lane downstream before the added lane ends. Since commercial trucks are usually slower vehicles, their merging from the added lane to the regular lane at the

lane drop area can cause disruptive turbulence at the merge point when traffic flows are moderately high and higher. Some alternative passing lane designs are starting to appear, such as slower vehicles moving right at the start of the passing lane segment, but faster vehicles having to merge at the end of the passing lane, and '2+1' type of configurations where the fasters vehicles need to change lanes at both the start and end of the passing lane segment. The relative impacts to the traffic stream performance due to these different designs needs to be better understood, particularly for traffic streams with non-trivial percentages of commercial trucks.

## 1.2. Objective and Tasks

The objective of this research was to improve the state-of-the-art for accounting for the impact of heavy vehicles (i.e., commercial trucks) on two-lane highway operations, particularly as it relates to the current two-lane highway analysis methodology of the Highway Capacity Manual. This was accomplished primarily by building on the work that was done for NCHRP Project 17-65 (Washburn et al., 2018). For this project, we addressed this objective in four different areas, as follows.

1. Capacity on non-passing lane upgrade segments when trucks are present in traffic stream
2. Effective length of passing lane considering upgrades and trucks
3. Guidance for climbing lane design (and return to level grade conditions)
4. Passing lane diverge/merge configurations

## 1.3. Simulation Tool

Given the wide range of two-lane highway geometric and traffic conditions tested in this research, as well as the available project budget, microsimulation was used to generate the analysis data for all of the tasks. The selected simulation tool was SwashSim, because of its specific modeling strengths with respect to two-lane highways and heavy vehicles. SwashSim has the ability to model a wide range of two-lane highway configurations and operational scenarios. SwashSim performs detailed modeling of vehicle and powertrain characteristics, which provides for more realistic acceleration performance modeling. This is particularly important when modeling heavy vehicles, climbing lanes, and upgrades. Please refer to Ozkul and Washburn (2015) for more information about the calibration of truck characteristics (size, weight, and performance) used within SwashSim. SwashSim has been calibrated and validated through several previous projects, such as NCHRP Project 17-65. For more information about SwashSim, please refer to the SwashSim documentation (Washburn, 2023).

## 1.4. Document Organization

The remainder of this document consists of separate chapters for each task of the research objective.

# 2. Capacity Estimation

## 2.1. Background

Capacity, or the volume-to-capacity ratio, is not a commonly utilized performance measure for two-lane highways, at least for evaluating operations under regular conditions. However, capacity values can still be important for planning and design purposes—for example, evacuation route planning, special event (e.g., regional festival) planning, and diversion of traffic from another route due to construction or closure.

Capacity values for two-lane highways provided in the Highway Capacity Manual (HCM) have historically been based on a very limited set of field measurements. Finding two-lane highways with very high flow rates, at least for some part of the day, has proven to be very challenging. Often, two-lane highways are upgraded to multilane facilities well before capacity flow levels are reached, due to the very poor operational quality that can occur at volume-to-capacity ratios as low as 0.5.

The various calculations in the two-lane highway analysis methodology of the HCM 7th edition, developed from NCHRP Project 17-65, are generally more sensitive to the impact of heavy vehicles on operations than the previous analysis methodology. This latest methodology also provided additional guidance on capacity values for passing lane segments, which consider truck percentage and roadway grade. However, the historical guidance in the HCM for the base (or ideal) capacity of non-passing lane segments was left unchanged—1700 pc/h/ln. This value is for non-passing lane segments, a passenger-car only traffic stream, and level terrain. Additionally, a distinction between the capacities of a Passing Zone and a Passing Constrained segment is not made, as the empirical evidence is currently too limited in this regard.

The objective of this research was to develop estimates of capacity for non-passing lane segments of two-lane highways, considering heavy vehicle percentage and upgrades.

## 2.2. Literature Review

### *Empirical Studies*

Some empirical studies have examined two-lane highway capacity, for example (Polus et al., 1991; Rozic, 1992; Harwood et al., 1999; Luttinen, 2001; Brilon and Weiser, 2006), but the literature is generally sparse with such studies. Many of the studies are 25 or more years older, corresponding to vehicle fleets with significantly different vehicle sizes and/or performance characteristics than current vehicle fleets. Commercial truck performance, especially, has improved significantly over the past 20 years. Some studies are from non-U.S. locations, where vehicle fleets and/or local driving behavior may result in different operating characteristics and capacities than observed in the U.S. Not surprisingly, the reported measured directional maximum flow rates vary widely, ranging from approximately 1000 veh/h to 1800 veh/h. Reported values are not always based on the same aggregation time interval, although most researchers do provide values based on a 15-min aggregation interval. Because the studied field sites usually did not consist of a passenger-car only traffic stream, some researchers acknowledge that actual capacity values for an ‘ideal’ traffic stream could be as high as 2000 pc/h (e.g., Rozic, 1992).

### *Simulation Studies*

Two-lane highway capacity has also been examined by some researchers using simulation, for example (Stock and May, 1976; Brilon and Weiser, 2006; Kim and Elefteriadou, 2010). Again, some studies are dated and/or based on non-U.S. vehicle and driver behavior conditions. The study by Kim and Elefteriadou (2010), which is a relatively more recent study and based on U.S. conditions found base (i.e., ideal conditions) capacity values to range from 1,835 to 2,141 pc/h/ln as a function of the free-flow speed.

### *Capacity Values in the HCM*

The base (or ideal) capacity values for a two-lane highway provided in the first six editions of the Highway Capacity Manual are as follows:

- First: 2000 pc/h, for both travel directions combined (Bureau of Public Roads, 1950)
- Second: 2000 pc/h, for both travel directions combined (Highway Research Board, 1965)
- Third: 2800 pc/h, for both travel directions combined; 2000 pc/h for one direction if there is zero flow rate in the opposing direction (Transportation Research Board, 1985)
- Fourth/Fifth/Sixth: 1700 pc/h for one direction, regardless of directional split, but a maximum of 3200 pc/h for both travel directions combined (Transportation Research Board, 2000/2010/2016)

The capacity values provided in the 4$^{th}$, 5$^{th}$, and 6$^{th}$ editions of the HCM were based on the results of NCHRP Project 3-55(3) (Harwood et al., 1999). That study did obtain data from numerous field sites, but only a couple of them experienced high flow rates. One site (Madera-Olsen Rd., Simi Valley, California) was observed with a peak 15-min flow rate in one direction of 1839 veh/h. Another site (Hwy. 4, Contra Costa County, California) was observed with a peak 15-min flow rate in one direction of 1920 veh/h. The study authors also state "Note that the volumes in the table are in veh/h and, thus, have not been adjusted for any heavy vehicles that might be present in the traffic stream. Flow rates adjusted to incorporate the passenger-car equivalents (PCEs) of heavy vehicles would necessarily be higher than those shown in the table." The study authors also state "A review of directional flow rates on higher-volume two-lane highways led to a choice of 1,700 veh/h as the capacity for one direction of travel by itself. While directional flow rates above 1,700 pc/h were observed in some cases, these did not appear sufficient to justify the value of 2,000 pc/h used in the 1985 HCM." The capacity value of 2000 pc/h in the 1985 HCM assumed a directional split of 100/0; that is, traffic is only traveling in one direction. However, the authors state that "capacity is much less influenced by directional split than was suggested by the 1985 HCM". Thus, the researchers specified the two-way flow rate to be 3200 pc/h, which is almost twice the directional capacity value, but still accounts for a small influence due to directional split. The Madera-Olsen Rd. site had two maximum two-way flow rates exceeding 3000 veh/h (3107 and 3027) and the Hwy. 4 site had a maximum two-way flow rate of 3350 veh/h. The recommended base capacity value of 1700 pc/h appears to have been a conservative estimate by the research team, especially since the potential presence of heavy vehicles in the field measured flow rates was not explicitly considered.

As mentioned in the background section, the revised analysis methodology now incorporated in the 7$^{th}$ edition of the HCM retained the base capacity value of 1700 pc/h/ln of the previous HCM edition, with two minor revisions. First, the units were changed from pc/h/ln to veh/h/ln. Second, the two-way capacity is simply double the one-way capacity; that is, 3400 veh/h/ln. Data were collected from numerous field sites as part of NCHRP Project 17-65; however, only two sites recorded relatively high flow rates. One site was in North Carolina and had maximum directional flow rates of approximately 1600 veh/h, based on a 5-min aggregation level. The other site was in California and had maximum directional flow rates of approximately 1400 veh/h, based on a 60-

min aggregation level. Both sites had heavy vehicle percentages on the order of 5-10% during peak flow rate times. With the limited sample of sites with relatively high flow rates, the research team for NCHRP Project 17-65 decided that there was insufficient data to recommend any change to the base capacity value of the previous HCM edition.

### *Heavy Vehicles and Capacity in the HCM*

The previous section mentioned the concept of passenger-car equivalents (PCEs), which was utilized in the two-lane highway analysis methodology of the last several editions of the HCM to account for the impact of large trucks on operations relative to passenger cars. The most recent update to the HCM methodology for two-lane highways, in the 7$^{th}$ edition of the HCM, however, eliminated the use of PCEs and now directly uses the percentage of heavy vehicles in the traffic stream. Thus, the traffic stream units are veh/h instead of pc/h. The reasons for this change were to 1) improve the sensitivity of the analysis results to the effects of heavy vehicles, and 2) improve the understanding and intuitiveness of the analysis process (Washburn et al., 2018). With respect to the first item, the use of PCE values does not result in a different speed-flow curve—one which likely would bend towards lower average speeds—than the curve for a passenger-car (pc) only stream. Instead, it just moves the analysis flow rate to a higher value on the same speed-flow curve. In the case of two-lane highways, which often have more varying terrain than other roadway facilities, using a higher analysis flow rate on a speed-flow curve more appropriate for pc-only traffic streams will likely result in an underestimate of the average speed of the traffic stream when there is a non-negligible percentage of heavy vehicles. With respect to the second item, the PCE values varied by performance measure, which meant that two different sets of analysis flow rates had to be computed in some cases, depending on the highway classification. This was a frequent source of confusion for users. The PCE values also did not vary by heavy vehicle percentage. The heavy vehicle adjustment factor, a function of PCE values, was also used to adjust the base directional capacity of 1700 pc/h to a value that accounts for the impacts of the prevailing heavy vehicle composition in the traffic stream. The resulting capacity value, in units of veh/h, would then be reduced from the 1700 pc/h value.

In the latest HCM two-lane highway analysis methodology, heavy vehicles are considered to be large trucks, buses, and recreational vehicles that fall within FHWA classes 4-10. Vehicles with FHWA class 11-13 are rare for two-lane highway facilities and therefore were excluded from development of the analysis process; however, these larger vehicles can still be included within the analysis when present. To accommodate for vertical and horizontal alignment, a classification scheme that takes heavy vehicles into consideration has been implemented. For the vertical class scheme, it includes five levels which are based on reductions in heavy vehicle free flow speed. Each level is a function of segment length and upgrade and downgrade percentages, where speed reduction in heavy vehicles is accounted for. Upgrade classification accounts for speed reduction due to acceleration capability and downgrade classification accounts for speed reduction due to downshifting to avoid the runaway-truck phenomenon. The horizontal alignment classifications account for heavy-vehicle free flow speed reductions as a function of curve radius and super elevation.

As previously mentioned, the latest HCM analysis methodology retains the historical guidance in past HCM editions for the capacity of non-passing lane segments. This is because the field data obtained and analyzed as part of NCHRP Project 17-65 did not offer enough conclusive evidence to justify revising the capacity value. Capacity values were developed for passing lane segments as a function of grade and heavy vehicle percentage as part of NCHRP Project 17-65 and are included in the latest HCM analysis methodology.

## 2.3. Research Approach

As mentioned in the introduction, simulation was used for this task, as well as all other tasks in this project. Also as mentioned in the introduction, the simulation tool, SwashSim, used in this project went through an extensive calibration and validation process for two-lane highway modeling as part of NCHRP Project 17-65 (see the NCHRP Project 17-65 final report for the details on this effort). However, this calibration effort was specific to the field data collected for the project, which did not include capacity-level flow rates in the research team's estimation. Thus, for this task, we decided to initially use the same SwashSim simulation parameter settings as were used in the NCHRP Project 17-65. We felt that base (i.e., level grade, 0% heavy vehicles) capacity flow rates on the order of 2000 veh/h/ln, or slightly higher would be reasonable. If higher values were obtained, we would adjust appropriate model parameters to reduce the base capacity flow rate to approximately 2000 veh/h/ln. Reasonable arguments could be made for using a value between 1700 and 2000, and this issue will be discussed further in the results section.

Again, the sample size of high flow rate field measurements from two-lane highways is quite limited, especially over the last 20 years. The lack of data in this respect leaves considerable uncertainty about the true value of capacity (base and non-base conditions), at least for any high level of statistical confidence. The relatively recent efforts on estimating capacity for freeways based on the concept that capacity is a stochastic value (i.e., distributed according to a probability distribution, such as the Weibull distribution) should also be noted. One variant of this approach also relies on the argument that capacity values cannot be estimated strictly from the highest flow rates measured at a site—that there must also be a certain percentage of breakdown events (see Modi et al., 2014 for a summary of this approach, specifically the section on the Product Limit Method). In other words, if breakdown events do not occur, one cannot be sure that higher flow rates might be attainable than those currently occurring. This issue is generally more complicated for two-lane highways. Freeway breakdowns, often the result of a geometric bottleneck situation, result in stop-and-go traffic conditions and are relatively easy to identify. Two-lane highway bottlenecks, on the other hand, are usually the result of a relatively slow platoon leader; thus, it is a moving queue situation that is more challenging to identify than freeway breakdown events. Nonetheless, without the application of an approach to the determination of two-lane highway capacity that is consistent with the more recent methods for freeway capacity determination, it is possible that the highest flow rates that have been measured at two-lane highways are still lower than the highest flow rates that could be attained.

Of course, the base capacity value for a two-lane highway should be less than that for a multilane highway (2300 pc/h/ln for highest FFS). How much reduction from the multilane highway value is reasonable given the greater inefficiency of gap utilization on two-lane highways? Again, a value of 2000 veh/h/ln seemed like a reasonable starting point.

The simulation experimental design is described in the remainder of this section.

### *Experimental Design*

#### Network Configuration

A 10-mile and 13-mile test facility, with passing not allowed in the opposing direction throughout the network, was used for simulation. Each network included a lead-up segment, a 2-mile graded segment that served as the test segment, and an exit segment. The 10-mile facility included a lead-up segment of 5 miles, which was used for upgrades of 0%-4%. The 13-mile facility included a lead-up segment of 8 miles, which was used for upgrades of 6% and 8%. This extra network lead-up distance was used to accommodate the additional queue build up caused by heavy vehicles reaching crawl speeds on the steep grades. Detectors were placed at the beginning and end of the 2-mile test segment as shown in Figure 2-1. The capacity was measured from the detectors at the end of the test segment. The free flow speed for the entire network was consistent across all simulations at 65 mi/h.

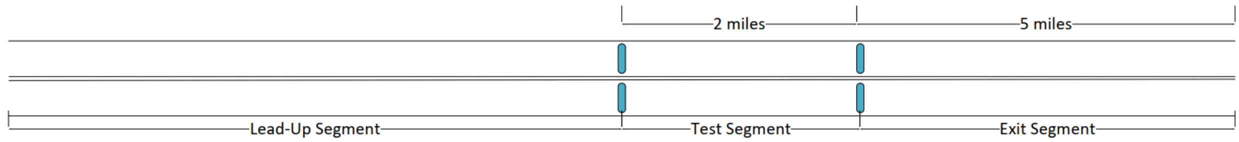


Figure 2-1. Network Configuration

## Traffic Data Settings

The input flow rates for each scenario were set such that they were higher than the realized capacity values, but not so high as to potentially cause complications with vehicle entries into the network or result in excessive simulation run time. The opposing flow rate did not factor into the analysis because passing in the oncoming lane was not allowed; thus, it was set to a small value of 100 veh/h for all runs. Vehicle fleet type percentage inputs were specified according to Table 2-1.

Table 2-1. Heavy Vehicle Composition Percentage

| HV% | Small Auto (%) | Large Auto (%) | Small Truck (%) | Large Truck (%) |
|---|---|---|---|---|
| 0% | 60 | 40 | 0 | 0 |
| 5% | 50 | 45 | 3 | 2 |
| 10% | 54 | 36 | 6 | 4 |
| 15% | 52 | 33 | 9 | 6 |
| 20% | 48 | 32 | 12 | 8 |
| 40% | 36 | 24 | 24 | 16 |
| 60% | 24 | 16 | 36 | 24 |
| 80% | 12 | 8 | 48 | 32 |
| 100% | 0 | 0 | 60 | 40 |

## Variable Settings

A total of 270 simulation runs were completed for the analysis—9 levels of heavy vehicle percentage times 5 levels of grade percentage (Table 2-2) and 6 replications of each heavy vehicle-grade combination. The number of replications was based on the sample size calculation for a 90% confidence level, a standard deviation of capacity values that ranged from approximately 25-40 veh/h, and a desired flow rate error of ± 30vehicles.

Table 2-2. Input Settings

| Variable | Settings |
|---|---|
| % HV[1] | 0, 5, 10, 15, 20, 40, 60, 80, 100 |
| % Grade | 0, 2, 4, 6, 8 |
| FFS (mi/h) | 65 |

[1] The fleet type was split 60/40 for small and large trucks, 60/40 for small and large automobiles, as shown in Table 2-1. These values are based on results from NCHRP Project 17-65.

The simulation time used for each run was 75 minutes, which included 15 minutes of network initialization time.

## 2.4. Analysis Results

The average capacity flow rates and average speeds, measured at the end of the test segment in the analysis direction, are shown in Table 2-3. Each value corresponds to the average of the six replications for each HV%-Grade% combination. The flow rates were then rounded to the nearest 10 veh/h.

Table 2-3. Observed Average Capacities and Speeds [veh/h (mi/h)]

| **HV%** | **Grade (%)** | | | | |
|---|---|---|---|---|---|
| | **0** | **2** | **4** | **6** | **8** |
| 0 | 2000 (58.4) | 2000 (58.4) | 2000 (58.4) | 2000 (58.3) | 2000 (58.4) |
| 5 | 1990 (58.2) | 1990 (58.2) | 1990 (47.3) | 1990 (43.3) | 1940 (37.0) |
| 10 | 1980 (58.1) | 1970 (58.1) | 1970 (44.9) | 1970 (39.0) | 1890 (32.5) |
| 15 | 1960 (58.0) | 1960 (58.0) | 1960 (44.1) | 1960 (37.4) | 1860 (31.2) |
| 20 | 1950 (57.9) | 1950 (57.8) | 1950 (43.7) | 1870 (36.3) | 1810 (29.6) |
| 40 | 1830 (57.8) | 1830 (57.8) | 1830 (43.2) | 1670 (34.8) | 1650 (28.1) |
| 60 | 1660 (57.7) | 1660 (57.7) | 1660 (42.9) | 1570 (34.2) | 1540 (27.4) |
| 80 | 1520 (57.7) | 1520 (57.7) | 1520 (42.8) | 1480 (33.9) | 1430 (27.1) |
| 100 | 1380 (57.7) | 1380 (57.6) | 1380 (42.7) | 1380 (33.7) | 1340 (26.8) |

The capacity and speed values decrease with increasing grade and heavy vehicles, in an expected manner. This can be seen in Figure 2-2 and Figure 2-3 where capacity and speed were plotted as a function of grade and heavy vehicle percentage. The presence of large trucks on steeper grades significantly reduced speeds. Lower grades of 0% and 2% did not show a large change in speed, but upon reaching 4% and steeper grades a larger reduction in speed becomes apparent.

If trucks are not present in the traffic stream, capacity is not affected, as passenger vehicles are able to maintain their desired speed even on relatively steep grades. Reduction in speed is only due to traffic conditions. Thus, capacity is impacted by the interaction of grade and heavy vehicles and not due to grade alone.

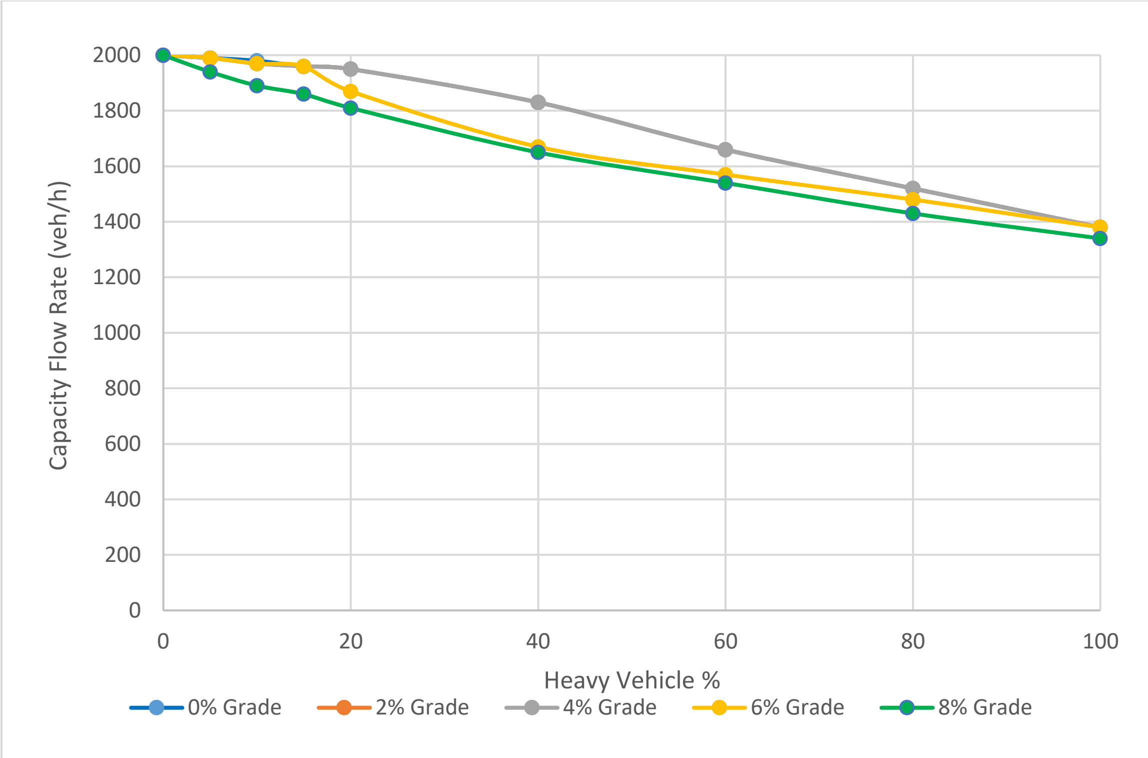

Figure 2-2. Capacity as a function of %HV and Grade

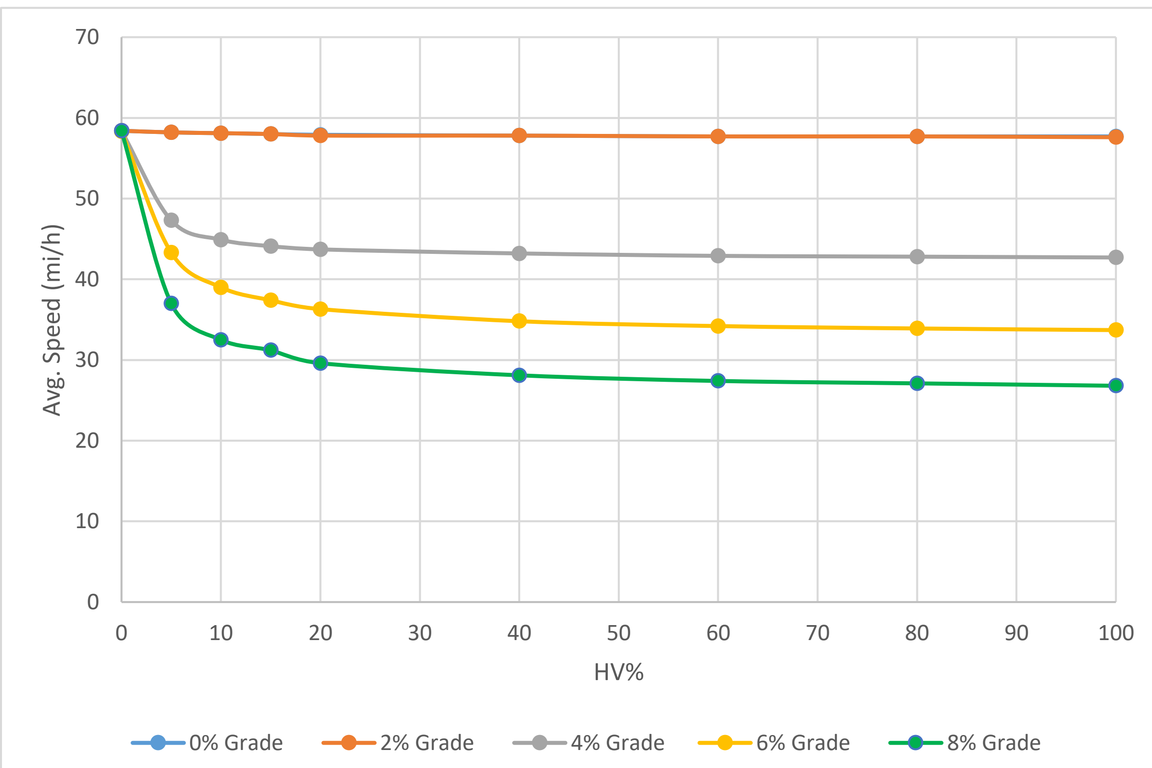

Figure 2-3. Average speed as a function of %HV and Grade

Visual observations of many of the simulation animations showed reasonable traffic stream behavior. Thus, the various SwashSim model parameter settings (e.g., car-following) were not modified from the values used for NCHRP Project 17-65.

Regression analysis was performed to develop an equation for capacity estimation as a function of grade and heavy vehicle percentages. A linear model, shown in Eq. 2-1, was used for simplicity and because there was a negligible difference in goodness-of-fit between a linear and non-linear formulation. The intercept value for this equation was constrained to 2000, to be consistent with the identified capacity value for 0% heavy vehicles and level grade; that is, base capacity.

$$Capacity = 2000 - 5.5168(HVPct) - 0.1761(HVGradePct) \quad [2\text{-}1]$$

where,

*Capacity* = maximum sustainable flow rate on segment, by direction, for the given heavy vehicle and grade conditions (veh/h);
*HVPct* = percentage of heavy vehicles in traffic stream; and
*HVGradePct* = percentage of heavy vehicles in traffic stream multiplied by segment incline percentage (for downgrades, use a value of 0).

The adjusted $R^2$ for the model is 0.99 and the two independent variables were significant at well over the 99% confidence level (t-statistic ≥ 2.326), using a one-tailed t-test. The very high $R^2$ value is expected because the variance from each HV%-Grade% combination has been removed by using the average value of the six replications. Further, there are likely some other factors that may affect capacity in the field that are not fully accounted for in the simulation. Thus, Eq. 2-1 is a quasi-deterministic relationship, but was developed to provide a convenient alternative to looking up values in Table 2-3.

Again, the validity of a base capacity value of 2000 veh/h/ln can be reasonably debated. But just as important as the base value are the capacities for non-base conditions (i.e., HV% > 0 and/or grade% > 0). If an agency believes that the base capacity value should be different than 2000 veh/h, based on some local flow rate measurements, Eqs. 2-2 and 2-3 can be used in lieu of Eq. 2-1.

$$AdjFact = 1 - 0.2758(HVProp) - 0.8805(HVProp \times GradeProp) \quad [2\text{-}2]$$

where,

*AdjFact* = adjustment factor to adjust capacity value from Eq. 2-1 for base capacity other than 2000 veh/h/ln (unitless);
*HVProp* = proportion of heavy vehicles in traffic stream; and
*GradeProp* = segment incline proportion (for downgrades, use a value of 0).

$$Capacity = BaseCapacity \times AdjFact \quad [2\text{-}3]$$

where,

*BaseCapacity* = maximum sustainable flow rate on segment, by direction, for level grade and 0% heavy vehicles (veh/h);

Eq. 2-2 was developed based on the capacity values in Table 2-3 relative to the base capacity value of 2000 veh/h/ln. That is, the adjustment factor values indicate the proportion of capacity relative to base capacity for heavy vehicle and/or grade percentages greater than zero. While these adjustment factor (proportion) values would be slightly different if the base capacity and capacities for non-base conditions obtained from the simulation results were different, the error introduced by using the adjustment factor values based on a base capacity of 2000 veh/h/ln is minimal, assuming base capacity values in the range of 1700-2000 veh/h/ln.

*Example Calculations*

Assume an agency is evaluating an existing two-lane highway route for a potential evacuation event. They are examining upgrade segments that may be potential bottleneck locations. One segment, an upgrade long and steep enough (6% incline) to result in very low speed operations for commercial trucks is expected to include 20% trucks in the traffic stream. The agency wants to estimate the capacity for this segment, but believes the base capacity is approximately 1900 veh/h/ln based on local data. Since the agency is using a base capacity other than 2000 veh/h/ln, Eqs. 2-2 and 2-3 are instead of Eq. 2-1.

The adjustment value is calculated per Eq. 2-2:

$$AdjFact = 1 - 0.2758(0.2) - 0.8805(0.2 \times 0.06) = 0.934$$

The adjusted capacity is then calculated with Eq. 2-3:

$$Capacity = 1900 \times 0.934 = 1745 \text{ (rounded) veh/h/ln}$$

If the agency assumed base capacity was 2000 veh/h/ln, then the capacity for the non-base conditions could be calculated from just Eq. 2-1:

$$Capacity = 2000 - 5.5168(20) - 0.1761(20 \times 6) = 1868.5 \text{ veh/h/ln}$$

Note that 1868.5÷2000 = 0.934, the adjustment factor value that was applied to the calculations assuming a base capacity of 1900 veh/h/ln.

## 2.5. Conclusions and Recommendations

This study developed capacity estimates for non-passing lane segments of two-lane highways, as a function of grade and heavy vehicles. Specific values of capacity are provided for varying levels of upgrade and heavy vehicle percentages, as well as an estimation equation. While there is currently no consensus for the most appropriate value for capacity under level grade and all passenger vehicle conditions, this study also provides a mechanism to adjust capacity values for assumed base capacity values ranging from 1700-2000 veh/h/ln.

This study did not consider passing in the oncoming lane. There is currently no consensus on whether passing in the oncoming lane affects capacity, but any potential effect is theorized to be relatively small. This is because high flow rates in the analysis direction have limited gaps available for passing vehicles to merge into and opposing flow further limits passing opportunities. Nonetheless, it is desirable for future research to examine this issue. This study also used only one length of upgrade segment, but one still long enough to induce crawl speeds for trucks for the steeper grades. Consideration of additional grade lengths is also desirable for future research.

More generally, more research on high flow rate two-lane highway sites in the field is desired. Realistically, there are likely some factors that influence the capacity flow rate phenomenon in the field that are not represented in the simulation.

# 3. Effective Length of Passing Lane

## 3.1. Background

The presence of heavy vehicles on two-lane highways can significantly reduce the speed of the traffic stream when no passing lanes are provided. This effect intensifies with the introduction of grades as passenger cars will catch up to slower heavy vehicles, forcing them to reduce their speeds. Heavy vehicles also impede passing opportunities on two-lane highways as they obstruct the view of passenger car drivers and make it more difficult to determine acceptable gaps in the opposing traffic stream. To combat these shortcomings, passing lanes can be implemented to present safe passing opportunities that positively affect operational performance some distance downstream. The HCM refers to this distance as the ‘Effective Length’, which is defined as the distance from the start of the passing lane to a point downstream where the performance returns to approximately the same level as it was immediately upstream of the start of the passing lane.

The research for this task looked to fill in the gaps of the approaches discussed in the next section with respect to geometric design and various heavy vehicle percentages. More specifically, it considers the percentage of followers entering a passing lane segment explicitly, rather than the ‘surrogate’ measure of percent no-passing zones. Furthermore, passing lane grade and a varying percentage of heavy vehicles are considered.

## 3.2. Literature Review

Prior research has been done to evaluate the effectiveness of passing lanes on two-lane highways, with the earliest dating back to 1985. Harwood and St. John (1985) used traffic speed, percentage of platooned vehicles (vehicles with headways of 4 seconds or less) and passing rate to evaluate the operational improvements. The results showed that the amount of improvement was a function of the level of platooning upstream of the passing lane as well as the length of the passing lane. The benefits were measured in PTSF (Percent Time Spent Following) and lasted several miles downstream.

Harwood and St. John (1986) used the TWOPAS simulation program to evaluate the operational improvements of passing lanes on two-lane highways. Percent time delay was used to evaluate improvements and showed that implementation of passing lanes had minimal effect on vehicle speeds, but drastic effect on the level of platooning. Effective length varied depending on passing lane length, traffic flow and composition, and downstream passing opportunities. The study found no consistent trend between the effects of traffic composition on effective length, but it was suggested that other geometric and traffic control features such as narrow lanes, grades, and no-passing zones may reduce the effective length of passing lanes (published version).

In 1991, May (1991) used the TRARR simulation program to investigate traffic performance and design of passing lanes with field data from five sites in California being utilized for calibration. Various lengths of passing lanes were used in the simulation to determine the effect on traffic performance and the number of passes, reduction in percent time delay, and estimated annual travel times were used as measurements. Vehicles with headways less than 5 seconds were considered to be delayed. The results showed that hourly flowrate and percent heavy vehicles were found to affect the number of passes, percent time delay, and mean travel speed.

Potts and Harwood (2004) utilized TWOPAS simulation to study the operational benefits of field data from 28 passing lanes on two-lane highways in Missouri. This study compared the difference in two-lane highways with and without passing and showed that passing lanes could reduce PTSF from 10-31 percent (published version).

Al-Kaisy and Freedman (2010) investigated the operational improvements downstream for passing lanes on passing lane sites in Montana using automatic traffic recorders. In terms of percent followers, the results determined that effective length of passing at this site may extend more than 10 miles downstream.

### *NCHRP Project 3-55*

The two-lane highway analysis methodology in the HCM 5th and 6th (ver. 6.0) Editions were based on the results of NCHRP Project 3-55, "Capacity and Quality of Service of Two-Lane Highways" (Harwood et al., 1999). In this work, the effective length of passing lanes was based on two performance measures: Average Travel Speed (ATS) and Percent Time Spent Following (PTSF). For ATS, the effective length is 1.7 miles, regardless of directional flow rate. For PTSF, the effective length varies with directional flow rate, ranging from 13% for the lowest flow rate to 3.6% for the highest flow rate. See Table 3-1 for the full set of effective length values.

Table 3-1. HCM 2016 (ver. 6.0) Effective Length of Passing Lane

| **Directional Demand Flow Rate, $v_d$ (pc/h)** | **Downstream Length of Roadway Affected, $L_{de}$ (mi)** | |
|---|---|---|
| | **PTSF** | **ATS** |
| ≤200 | 13.0 | 1.7 |
| 300 | 11.6 | 1.7 |
| 400 | 8.1 | 1.7 |
| 500 | 7.3 | 1.7 |
| 600 | 6.5 | 1.7 |
| 700 | 5.7 | 1.7 |
| 800 | 5.0 | 1.7 |
| 900 | 4.3 | 1.7 |
| ≥1,000 | 3.6 | 1.7 |

Source: HCM 6th Edition (version 6.0) Exhibit 15-23 ("Downstream Length of Roadway Affected by Passing Lanes on Directional Segments in Level and Rolling Terrain").

### *NCHRP Project 17-65*

As part of NCHRP Project 17-65, "Improved Analysis of Two-Lane Highway Capacity and Operational Performance" (Washburn et al., 2018), two approaches to estimating passing lane effective lane were investigated.

The first approach, published in a separate paper (Jafari et al., 2020), investigated passing lane effective length using microscopic simulation. The simulation was calibrated and validated using field data from sites in Oregon. The following performance measures were used: percent followers (PF), follower density (FD), and average travel speed (ATS). The effective length of a passing lane was determined when the platooning level, as defined by follower density, reached some maximum value downstream and remained at or near that maximum density. In this research follower density is the density multiplied by percent followers, where percent followers was found as the percentage of vehicles with headways of 2.5 seconds or less.

Study results showed that both traffic level and percent no-passing have a considerable effect on the effective length of the passing lane, as seen in Eq. 3-1. This was determined through regression analysis.

$$Effective\ Length = 22.53 \times \exp(-0.0014\ \times FlowRate) - 0.023\ \times \%NP \quad [3\text{-}1]$$

where,

*EffLength* = distance from the start of passing lane to a point downstream where the performance returns to the same level as immediately upstream of the passing lane (mi);

*FlowRate* = flow rate entering the passing lane (veh/h); and

*%NP* = percentage of no-passing zones upstream of the start of the passing lane. This value should correspond to a sufficient distance upstream of the passing lane such that the level of platooning is considered bot be in a quasi-equilibrium state.

Moreover, this study revealed that operational benefits of passing lanes generally last for a significant distance downstream of the passing lane and ranged from 3 miles to 20 miles depending on traffic levels and percentage of no-passing zones. The effective lengths found are relatively longer than the values provided in Table 3-1.

The results of this first approach helped address the need for accurate methods for the design of passing lanes but did not consider some variables that might affect the effective length of passing lanes such as grade and heavy vehicle percentage. Furthermore, the percent no-passing variable is only an approximate surrogate indicator of platooning level upstream of the passing lane. For example, a highway with 50% no passing zones could consist of different configurations, such as:

1) A 5-mile passing zone followed by a 5 mile no passing zone immediately upstream of the passing lane,
2) A series of 1-mile zones alternating between passing and no passing for 10 miles immediately upstream of the passing lane.

Both cases would be considered 50% no-passing but would have significantly different platooning levels. A better indicator of the level of vehicle platooning is the actual percentage of followers. Thus, the second approach used percent followers immediately upstream of the passing lane as an independent variable in the analysis process. Again, the effective length is defined as the distance downstream from the start of the passing lane to the point where operational performance returns to approximately the same level just prior to the start of the passing lane. For the purposes of this approach, that point is identified as of one of the following:

- The percentage improvement to the percent followers becomes zero, or
- Follower density is at least 95% of the level entering the passing lane.

Whichever of these two distances is shorter is taken as the effective length. The second approach resulted in the following equations.

$$\%Improve_{PF} = \max(0, 27 - 8.75 \times \ln[\max(0.1, DownstreamDistance)] + 0.1 \times \max[0, PF - 30] + 3.5 \times \ln[\max(0.3, PassLaneLength)] - 0.01 \times FlowRate) \quad [3\text{-}2]$$

$$\%Improve_{S} = \max(0, 3 - 0.8\ \times DownstreamDistance + 0.1 \times \max[0, PF - 30] + 0.75 \times PassLaneLength\ - 0.005 \times FlowRate) \quad [3\text{-}3]$$

$$FD_{adj} = \frac{PF}{100} \times \left(1 - \frac{\%Improve_{PF}}{100}\right) \times \frac{FlowRate}{S \times \left(1 + \frac{\%Improve_{S}}{100}\right)} \quad [3\text{-}4]$$

where:

$\%Improve_{PF}$ = % improvement to percent followers on a segment downstream of a passing lane segment;

$\%Improve_S$ = % improvement to the average speed on a segment downstream of a passing lane segment;

$FD_{adj}$ = adjusted follower density on a segment downstream of a passing lane segment (followers/mi);

*DownstreamDistance* = distance downstream from the start of the passing lane segment (mi);

*PassLaneLength* = length of passing lane segment (mi);

$S$ = average speed in the analysis direction for the analysis segment (mi/h);

$PF$ = percent followers (%), determined as follows;

- When calculating passing lane effective length or downstream segment $\%Improve_{PF}$ or $\%Improve_S$, use $PF$ entering the passing lane segment (i.e., $PF$ estimated at the end of the segment just upstream of the passing lane segment, and
- When calculating $FD_{adj}$ downstream of the passing lane, use $PF$ for the analysis segment, and

*FlowRate* = demand flow rate (veh/h), determined as follows;

- When calculating passing lane effective length, use the flow rate entering the passing lane segment, and
- When calculating downstream segment $\%Improve_{PF}$, $\%Improve_S$, or $FD_{adj}$, use the flow rate for the analysis segment.

These equations are used to both determine the passing lane's effective length and the improvement in performance measures in downstream segments within the effective length. If the subject analysis segment is located within the effective length of the upstream passing lane, these equations are applied to determine the improvement in follower density, due to the upstream passing lane, for the subject segment. Only the effect of the closest upstream passing lane segment is considered when determining the improvement to follower density. In other words, the analysis of segments downstream of a passing lane "resets" with each new passing lane. These equations were ultimately incorporated into HCM versions 6.1 and later (Chapter 15, Eqs. 15-36 – 15-38).

While the second approach improved upon the first approach by explicitly considering the percentage of followers immediately upstream of the passing lane, as well as the length of the passing lane, it still did not consider important variables such as segment grade and percentage of heavy vehicles.

## 3.3. Research Approach

The simulation experimental design is described in this section.

### *Experimental Design*

#### Network Configurations

The initial network configuration was guided by the work performed for the NCHRP Project 17-65, discussed in the previous section. This configuration was designed to take grade, length of passing lane, and percent no passing into consideration.

The upstream section of the networks consisted of 5.5 miles of highway divided into 11 half-mile links. This setup was done to facilitate different percent no passing configurations prior to the passing lane. The link immediately prior to the passing lane link included a detector at the downstream end that was used to measure the percent followers entering the passing lane link. The next link was the passing lane, which was either 1, 2, or 3 miles in length. Two detector stations were contained in the passing lane link, at the midpoint and endpoint. The passing lane segment was followed by two 6-mile link and one 1.5-mile link. The two 6-mile links contained detectors at approximately 0.3-mile increments (resulting in 20 detectors per link) to monitor the percent follower and follower density to determine the effective length of the passing lane. Preliminary testing indicated that the effective length would not extend more than 12 miles downstream of the length of the passing lane for the range of inputs in the experimental design. With the passing lane length varying from 1-3 miles, the total network length varied from 20-22 miles. The non-passing lane links were always level grade.

Due to the stochastic nature of microscopic simulation, it is not possible to control percent follower levels with any level of precision several miles downstream of the network entry point. Thus, three general levels of platooning were targeted: low, medium, and high. These levels of platooning were affected by the configuration of passing-allowed or not allowed in the links upstream of the passing lane. The 'low level' percent follower networks allowed passing throughout all upstream links, the 'medium level' percent follower networks allowed passing on 45% (5 of 11) of upstream links, and the 'high level' percent follower networks did not allow passing on any upstream links. The preliminary runs showed that continuous passing opportunities presented the best results, so the 45% passing allowed configuration consisted of passing allowed on the five middle links (4-8) of the 11 links upstream of the passing lane.

In addition to the link passing configuration upstream of the passing lane, the percent followers entering the passing lane is a function of the flow rate and heavy vehicle percentage. For any given network and traffic condition, the average value of percent followers differed by approximately 20-30% between the three levels (e.g., 25% for low, 50% for medium, 75% for high).

The experimental design settings for the passing lane are shown in Table 3-2. A total of 27 ($3 \times 3 \times 3$) networks were used for this experimental design.

Table 3-2. Passing Lane Link Experimental Design Settings

| Variables | Values |
|---|---|
| Passing Lane Grade (%) | 0, 4, 8 |
| Passing Lane Length (mi) | 1, 2, 3 |
| Level of Platooning Entering Passing Lane | Low, Medium, High |

#### Traffic Data Settings

The experimental design settings for the traffic characteristics are shown in Table 3-3.

Table 3-3. Traffic Characteristics Experimental Design Settings

| Variables | Values |
|---|---|
| Flow Rate (veh/h) | 300, 900, 1500 |
| Heavy Vehicle Percentage (%)[1] | 0, 6, 12 |

[1] The fleet type was split 60/40 for small and large trucks, 60/40 for small and large autos

Given the disruptive operational impacts that opposing traffic can have with upstream passing maneuvers, opposing traffic was not used as an input and was set to zero. The input free-flow speed was consistent at 60 mi/h.

### Simulation Settings

A total of 243 simulation scenarios were completed for the analysis—27 networks × 3 flow rates × 3 heavy truck percentages. Each scenario consisted of the average results from six replications. The simulation time used for each run was 90 minutes, which included 30 minutes of network initialization time.

## *Data Analysis*

The simulation results used for this analysis were obtained from the 'Detector Aggregate Measures Data' file. This file contains a variety of aggregated measurements of all individual detector actions for all detectors contained within the network (see the SwashSim documentation for more information about this output file). These values, primarily flow rates, heavy vehicle percentages, percent followers, and follower density, were combined with the corresponding scenario input values (e.g., passing lane length and grade) and analyzed with the R statistical software package (R Core Team, 2022).

### Process to estimate effective length

The basic process to determine the effective length of the passing lane from the simulation results is as follows:

For each scenario (the averages across six replications)

- Identify the detector at the end of the link preceding the passing lane link and store the percent followers and follower density values
- Compare the percent followers value at each detector within the passing lane and downstream links to the percent followers value from the previous step. The detectors are checked in order from upstream to downstream.
- The first detector to have a percent followers value equal to or higher than that for the detector just prior to the start of the passing lane link is considered to be at the end of the effective passing lane length.
- The effective length is calculated as the distance from the start of the passing lane to the location of the detector identified in the previous step.

## 3.4. Analysis Results

A summary of the effective length results from the 243 simulation runs is given in Table 3-4.

Table 3-4. Summary of effective length results from simulation experimental design

| Effective Length (mi) | Passing Lane Length (mi) | | |
|---|---|---|---|
| | 1 | 2 | 3 |
| Minimum | 1.3 | 2.8 | 4.5 |
| Average | 3.7 | 5.1 | 6.3 |
| Maximum | 6.2 | 7.5 | 9.0 |

The results from the experimental design were used to develop a model, via regression analysis, to estimate the effective length of a passing lane (see Eq. 3-5).

$$EffLength = -5.457 - 0.3146 \times FlowRateHundreds + 0.001751 \times NumTrucks + 1.306 \times PassLaneLength + 0.0007 \times (PassLaneGrade \times NumTrucks) + 0.1984 \times (\%Followers \times LowFollowers) + 0.1390 \times (\%Followers \times HighFollowers) \quad [3\text{-}5]$$

where:

*EffLength* = distance from the start of passing lane to a point downstream where the performance returns to the same level as immediately upstream of the passing lane (mi);
*FlowRateHundreds* = flow rate entering the passing lane, divided by 100 (veh/h/100),
*NumTrucks* = proportion of trucks entering the passing lane (decimal) × flow rate entering the passing lane (veh/h);
*PassLaneLength* = length of the passing lane (mi);
*PassLaneGrade* = incline of the passing lane (%);
*%Followers* = percent followers in the analysis direction (%);
*LowFollowers* = 1 if % followers entering the passing lane < 60, 0 otherwise; and
*HighFollowers* = 1 if % followers entering the passing lane ≥ 60, 0 otherwise.

The goodness-of-fit, adjusted R-squared, for the model is 0.8959. Some non-linear model forms were also examined, but none provided a better overall model fit than the simpler linear models. All model variables are significant at the 95% confidence level (t-statistic ≥ 1.645), using a one-tailed t-test. All of the model variables also have logical coefficient signs. The percent of followers entering the passing lane exhibited more of a bimodal effect on the effective length, rather than a linear continuous effect; thus, the last two terms in the model rather than just a simple '%followers' variable.

*Example Calculations*

Assume the following input values for a passing lane and the traffic stream entering the passing lane:

- Passing lane length: 2 miles
- Passing lane grade: 4%
- Flow rate: 900 veh/h
- Truck percentage: 6%
- Follower percentage: 68%

Calculate the effective length for these conditions.

The number of trucks = 900 × 6/100 = 54

$$EffLength = -5.457 - 0.3146 \times \frac{900}{100} + 0.001751 \times 54 + 1.306 \times 2 + 0.0007 \times (4 \times 54) + 0.1984 \times (68 \times 0) + 0.1390 \times (68 \times 1) = 4.0 \text{ mi}$$

### 3.5. Conclusions and Recommendations

The two-lane highway analysis methodology of the HCM (7th edition, Chapter 15) includes calculations for estimating the effective length of a passing lane. However, those calculations do not account for the grade of the passing lane or the percentage of trucks in the traffic stream (see Eqs. 3-2 – 3-4).

It is recommended that additional research be performed to determine how to integrate the results of this task with the current calculations for passing lane effective length in the HCM.

# 4. Climbing Lanes

## 4.1. Background

Climbing lane sections are similar to passing lane sections in that they provide an added lane that allows faster vehicles to pass slower vehicles without using the oncoming lane. Both climbing and passing lanes also serve to break up platoons. However, the considerations for implementing a climbing lane are distinctly different from the considerations for adding a passing lane. As their name implies, climbing lanes are implemented on upgrades and are intended to allow slower-moving trucks to move out of the way of faster vehicles on the grade, as the speed differential between passenger vehicles and large trucks can be substantial when the grade exceeds 2% for any significant distance.

AASHTO, in its "Green Book" document, provides the following criteria for when a climbing lane should be considered (AASHTO, 2018)[1]:

- Upgrade traffic flow rate exceeds 200 veh/h;
- Upgrade truck flow rate exceeds 20 veh/h; and
- One or more of the following conditions exists:
    - A 10 mi/h or greater speed reduction is expected for a typical heavy truck;
    - LOS E or F exists on the grade; or
    - A reduction of two or more levels of service is experienced when moving from the approach segment to the grade.

The Green Book recommends that a climbing lane should start on an upgrade no later than the location when the difference between overall average travel speed (ATS) and truck ATS is greater than 10 mi/h. The Green Book also recommends extending the climbing lane beyond the upgrade for a distance that allows trucks to accelerate to a minimum speed of 40 mi/h and within 10 mi/h of the passenger car speed.

The focus in this task was on truck speed-distance relationships for upgrades and downstream of upgrades. Obtaining detailed truck speed trajectory data from roadway sensors or field studies can be very challenging. Consequently, the application of pre-developed truck speed-distance curves is often the most practical method for evaluating the speed reduction aspect of the climbing lane criteria, as well as the needed acceleration lane distance downstream of an upgrade where a climbing lane is implemented.

The Green Book provides some material with respect to truck speed-distance relationships under deceleration and acceleration scenarios (see Section 3.4.2). However, this material is largely based on truck performance studies that are 25 or more years old. This task sought to align the truck performance curves, deceleration and acceleration, with more recent studies on this topic (e.g., see Ozkul and Washburn, 2015; Washburn et al., 2018). Those studies have also served as references for some of the truck performance updates to the Highway Capacity Manual (HCM). The results from this task can supplement the existing material in the Green Book.

The AASHTO climbing lane criteria also include LOS criteria. The HCM traffic operations analysis procedure (Chapter 15) can be applied for this purpose. However, another focus of this task was to provide additional information on expected performance measure differences for a

[1] Discussion in Green Book 7th edition starts at p. 3-137, Section 3.4.3.

climbing lane versus no climbing lane on an upgrade. This information can be used in an initial preliminary engineering effort to assess the general operational benefits that might be achieved by adding an additional lane to an upgrade section of roadway. If these preliminary results suggest that a climbing lane is worth considering further, a more detailed operational analysis can be conducted with the HCM analysis methodology and/or simulation.

## 4.2. Literature Review

This section reproduces material that is contained in Chapter 15, Appendix A, of the 7th edition of the HCM with respect to truck speed reductions on upgrades.

### *Truck Speed Reductions on Upgrades*

#### Base Method

The truck speed–distance curves in Figure 4-1 through Figure 4-3 can be used to determine speed reductions on different lengths of grade for three truck types (single-unit, intermediate semitrailer, and interstate semitrailer). No set of curves are provided for double semitrailer trucks, because field data indicated that this truck type was not prevalent on two-lane highways.

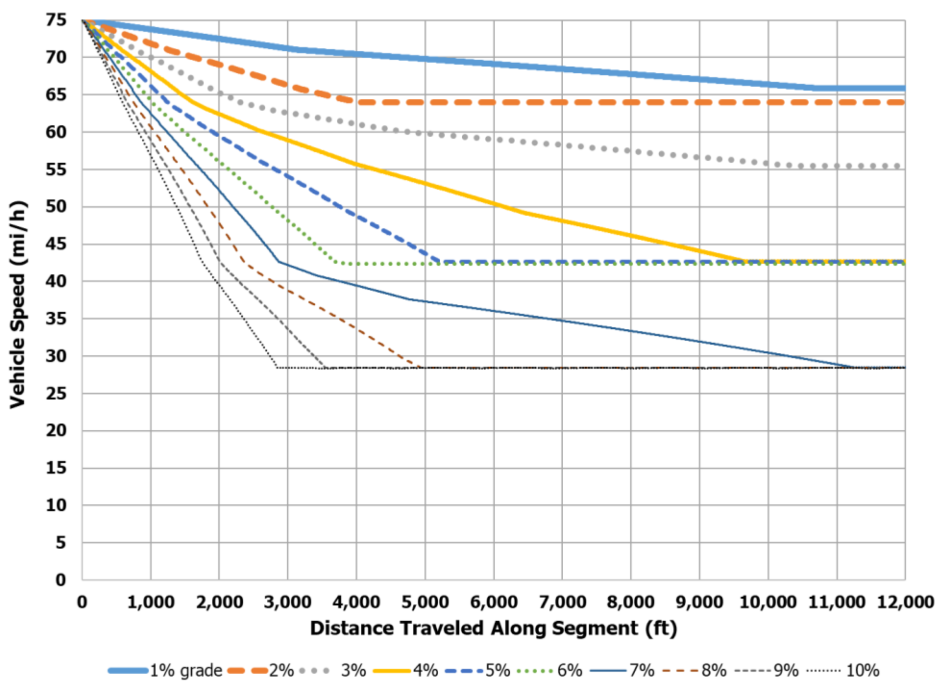

Figure 4-1. Upgrade Speed–Distance Curves for Single-Unit Trucks (HCM Exhibit 15-A3)

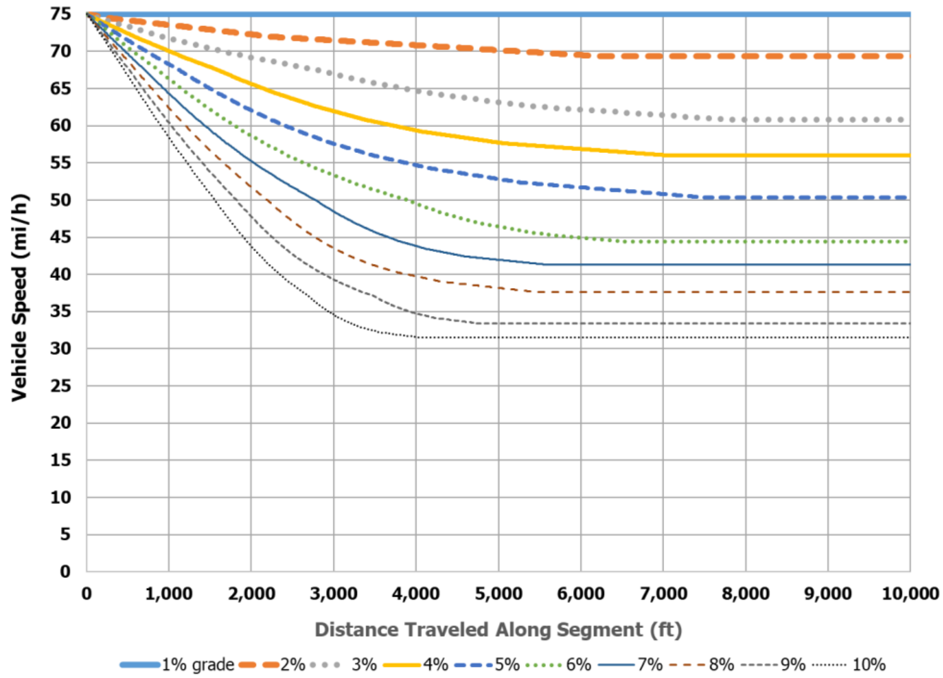


Figure 4-2. Upgrade Speed–Distance Curves for Intermediate Semitrailer Trucks (HCM Exhibit 15-A4)

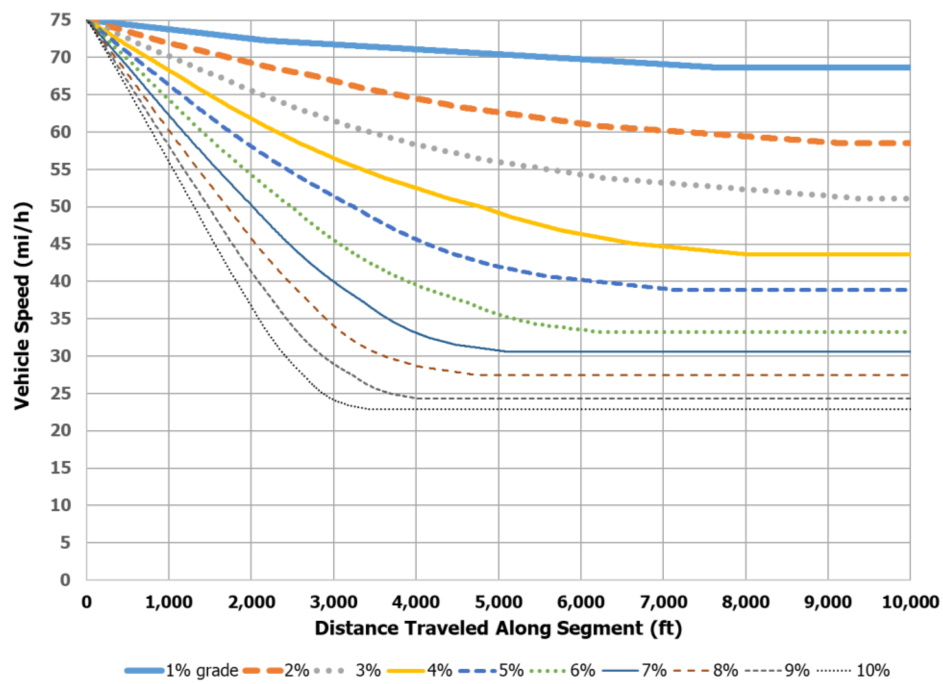


Figure 4-3. Upgrade Speed–Distance Curves for Interstate Semitrailer Trucks (HCM Exhibit 15-A5)

Alternatively, Eq. 4-1 (HCM Eq. 15-A1) can be used to estimate the speed reduction.

$$V = 75 + a \times L + b \times L^2 + c \times L^3 \quad [4\text{-}1]$$

where:

$V$ = speed of heavy vehicle at the end of the upgrade segment (mi/h);

$L$ = length of the upgrade segment (mi); and

$a$, $b$, $c$ = model coefficients (decimal), from Table 4-1, Table 4-2, or Table 4-3 for single-unit, intermediate semitrailer, and interstate semitrailer trucks, respectively.

Table 4-1. Upgrade Speed Model Coefficients for Single-Unit Trucks (HCM Exhibit 15-A6)

| Grade Slope (%) | *a* | *b* | *c* |
|---|---|---|---|
| 1 | −7.99117 | 3.34943 | −0.80873 |
| 2 | −16.79550 | 1.90540 | 1.36780 |
| 3 | −32.09620 | 21.98800 | −5.51770 |
| 4 | −39.03610 | 21.53390 | −5.45420 |
| 5 | −52.54130 | 37.09590 | −17.43770 |
| 6 | −61.54480 | 38.29370 | −22.79690 |
| 7 | −80.51610 | 54.45520 | −12.78160 |
| 8 | −88.40130 | 47.70330 | −5.71440 |
| 9 | −97.19730 | 41.85210 | 0.00000 |
| 10 | −93.95550 | −33.73320 | 93.20230 |

Table 4-2. Upgrade Speed Model Coefficients for Intermediate Semitrailer Trucks (HCM Exhibit 15-A7)

| Grade Slope (%) | *a* | *b* | *c* |
|---|---|---|---|
| 1 | 0.00000 | 0.00000 | 0.00000 |
| 2 | −9.11990 | 6.63672 | −2.51232 |
| 3 | −17.52110 | 5.44550 | 0.00000 |
| 4 | −29.10240 | 11.41810 | 0.00000 |
| 5 | −42.79200 | 24.99010 | −4.85490 |
| 6 | −52.06060 | 26.76310 | −3.74860 |
| 7 | −63.70110 | 30.18420 | 0.00000 |
| 8 | −77.24510 | 40.32630 | 0.00000 |
| 9 | −89.75260 | 48.34020 | 0.00000 |
| 10 | −90.21160 | 1.41830 | 56.44760 |

Table 4-3. Upgrade Speed Model Coefficients for Interstate Semitrailer Trucks (HCM Exhibit 15-A8)

| Grade Slope (%) | *a* | *b* | *c* |
|---|---|---|---|
| 1 | −7.92121 | 4.78662 | −1.63570 |
| 2 | −16.71740 | 3.63040 | 0.37130 |
| 3 | −29.79650 | 11.81370 | −1.39070 |
| 4 | −39.51320 | 13.24520 | −0.52500 |
| 5 | −49.57050 | 11.49140 | 4.32190 |
| 6 | −60.94040 | 12.96240 | 7.63790 |
| 7 | −66.62850 | −9.65440 | 32.62600 |
| 8 | −75.89060 | −24.93370 | 57.74360 |
| 9 | −82.36480 | −55.27030 | 101.05490 |
| 10 | −85.01500 | −114.73900 | 188.34900 |

### *Adjustment for Upgrades with Initial Speeds Less Than 75 mi/h*

The upgrade speed versus distance functions were developed assuming an initial speed of 75 mi/h. These functions can be modified to produce upgrade speeds for different initial speeds. Rather than creating new functions with different initial speeds, the length of the upgrade segment used in the equation can be extended to account for the difference in the initial speed.

For example, assume that an interstate semitrailer truck is approaching a 0.5-mi, 6 percent upgrade at an initial speed of 60 mi/h. Based on the speed versus distance function for a 6 percent grade and initial speed of 75 mi/h (shown in Figure 4-3), the vehicle does not reach a speed of 60 mi/h until it has traveled a distance of 1,426 ft (0.27 mi). To account for the 15 mi/h difference in initial speed, this distance can be added to the original segment length of 0.5 mi. Therefore, the adjusted upgrade segment length is 0.77 mi. This segment length is input into the speed versus distance function for a 6 percent grade (Eq. 4-1), which returns a final speed on the upgrade of 40 mi/h. Had the segment length not been adjusted, the final speed would have equaled 50 mi/h. Therefore, the 15 mi/h difference in initial speed created a 10 mi/h difference in the final speed on the upgrade.

Table 4-4 through Table 4-6 provide additional upgrade segment lengths for various combinations of initial speed and grade slope, for single-unit, intermediate semitrailer, and interstate semitrailer trucks, respectively. "N/A" in the exhibits indicates a situation where the truck initial speed is less than the truck minimum speed for a particular upgrade. Eq. 4-1 only models the non-constant portion of a speed-distance curve that is prior to the constant minimum speed portion. Table 4-7 provides the segment lengths at which minimum speed is reached if the truck enters the segment with an initial speed of 75 mi/h. Table 4-7 can be used to check if a truck would reach its minimum speed for a given adjusted segment length. If the adjusted segment length exceeds the value in Table 4-7, the minimum speed can be used directly rather than using Eq. 4-1.

Table 4-4. Additional Upgrade Segment Lengths (mi) for Different Initial Speeds of Single-Unit Trucks (HCM Exhibit 15-A9)

| Initial Speed (mi/h) | Grade Slope (%) | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 |
| 75 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| 70 | 0.89 | 0.32 | 0.18 | 0.14 | 0.11 | 0.09 | 0.07 | 0.06 | 0.06 | 0.06 |
| 65 | N/A | 0.68 | 0.42 | 0.31 | 0.23 | 0.19 | 0.14 | 0.13 | 0.11 | 0.11 |
| 60 | N/A | N/A | 0.89 | 0.51 | 0.37 | 0.29 | 0.22 | 0.19 | 0.17 | 0.16 |
| 55 | N/A | N/A | N/A | 0.79 | 0.53 | 0.41 | 0.31 | 0.27 | 0.23 | 0.21 |
| 50 | N/A | N/A | N/A | 1.18 | 0.72 | 0.53 | 0.42 | 0.35 | 0.30 | 0.26 |
| 45 | N/A | N/A | N/A | 1.63 | 0.91 | 0.65 | 0.56 | 0.44 | 0.37 | 0.32 |
| 40 | N/A | N/A | N/A | N/A | N/A | N/A | 0.75 | 0.55 | 0.45 | 0.38 |
| 35 | N/A | N/A | N/A | N/A | N/A | N/A | 1.15 | 0.69 | 0.54 | 0.45 |
| 30 | N/A | N/A | N/A | N/A | N/A | N/A | 1.98 | 0.90 | 0.64 | 0.53 |
| 25 | N/A | N/A | N/A | N/A | N/A | N/A | N/A | N/A | N/A | N/A |

Note: N/A: not applicable; the initial speed is less than the truck minimum speed. Use Table 4-7 instead.

Table 4-5. Additional Upgrade Segment Lengths (mi) for Different Initial Speeds of Intermediate Semitrailer Trucks (HCM Exhibit 15-A10)

| Initial Speed (mi/h) | Grade Slope (%) | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 |
| 75 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| 70 | N/A | 1.01 | 0.32 | 0.19 | 0.13 | 0.11 | 0.09 | 0.07 | 0.06 | 0.06 |
| 65 | N/A | N/A | 0.75 | 0.41 | 0.28 | 0.22 | 0.18 | 0.14 | 0.12 | 0.12 |
| 60 | N/A | N/A | N/A | 0.72 | 0.47 | 0.35 | 0.28 | 0.22 | 0.19 | 0.17 |
| 55 | N/A | N/A | N/A | N/A | 0.75 | 0.51 | 0.39 | 0.31 | 0.26 | 0.24 |
| 50 | N/A | N/A | N/A | N/A | N/A | 0.72 | 0.53 | 0.42 | 0.35 | 0.30 |
| 45 | N/A | N/A | N/A | N/A | N/A | 1.12 | 0.71 | 0.55 | 0.44 | 0.37 |
| 40 | N/A | N/A | N/A | N/A | N/A | N/A | N/A | 0.74 | 0.56 | 0.45 |
| 35 | N/A | N/A | N/A | N/A | N/A | N/A | N/A | N/A | 0.75 | 0.56 |
| 30 | N/A | N/A | N/A | N/A | N/A | N/A | N/A | N/A | N/A | N/A |
| 25 | N/A | N/A | N/A | N/A | N/A | N/A | N/A | N/A | N/A | N/A |

Note: N/A: not applicable; the initial speed is less than the truck minimum speed. Use Table 4-7 instead.

Table 4-6. Additional Upgrade Segment Lengths (mi) for Different Initial Speeds of Interstate Semitrailer Trucks (HCM Exhibit 15-A11)

| Initial Speed (mi/h) | Grade Slope (%) | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 |
| 75 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 | 0.00 |
| 70 | 1.08 | 0.33 | 0.19 | 0.14 | 0.11 | 0.09 | 0.08 | 0.07 | 0.06 | 0.06 |
| 65 | N/A | 0.72 | 0.40 | 0.28 | 0.22 | 0.18 | 0.15 | 0.13 | 0.12 | 0.11 |
| 60 | N/A | 1.35 | 0.67 | 0.45 | 0.34 | 0.27 | 0.23 | 0.20 | 0.17 | 0.16 |
| 55 | N/A | N/A | 1.07 | 0.65 | 0.47 | 0.37 | 0.31 | 0.26 | 0.23 | 0.20 |
| 50 | N/A | N/A | N/A | 0.89 | 0.62 | 0.48 | 0.39 | 0.33 | 0.28 | 0.25 |
| 45 | N/A | N/A | N/A | 1.29 | 0.80 | 0.60 | 0.47 | 0.40 | 0.34 | 0.30 |
| 40 | N/A | N/A | N/A | N/A | 1.12 | 0.75 | 0.57 | 0.47 | 0.40 | 0.35 |
| 35 | N/A | N/A | N/A | N/A | N/A | 0.98 | 0.70 | 0.56 | 0.47 | 0.40 |
| 30 | N/A | N/A | N/A | N/A | N/A | N/A | N/A | 0.69 | 0.55 | 0.46 |
| 25 | N/A | N/A | N/A | N/A | N/A | N/A | N/A | N/A | 0.70 | 0.55 |
| 20 | N/A | N/A | N/A | N/A | N/A | N/A | N/A | N/A | N/A | N/A |

Note: N/A: not applicable; the initial speed is less than the truck minimum speed. Use Table 4-7 instead.

Table 4-7. Minimum Speeds and Corresponding Segment Lengths for Different Truck Types (HCM Exhibit 15-A12)

| .Grade Slope (%) | Single-Unit Trucks | | Intermediate Semitrailer Trucks | | Interstate Semitrailer Trucks | |
|---|---|---|---|---|---|---|
| | Segment Length (mi) | Minimum Speed (mi/h) | Segment Length (mi) | Minimum Speed (mi/h) | Segment Length (mi) | Minimum Speed (mi/h) |
| 1 | 2.03 | 65.82 | N/A | N/A | 1.43 | 68.68 |
| 2 | 0.76 | 63.94 | 1.17 | 69.39 | 1.77 | 58.84 |
| 3 | 1.91 | 55.46 | 1.57 | 60.91 | 1.85 | 51.50 |
| 4 | 1.81 | 42.55 | 1.25 | 56.46 | 1.57 | 43.58 |
| 5 | 0.99 | 42.42 | 1.58 | 50.62 | 1.25 | 39.43 |
| 6 | 0.72 | 42.03 | 1.24 | 44.45 | 1.16 | 33.67 |
| 7 | 2.07 | 28.30 | 1.05 | 41.39 | 0.93 | 30.93 |
| 8 | 1.02 | 28.40 | 0.95 | 38.01 | 0.82 | 27.84 |
| 9 | 0.68 | 28.26 | 0.90 | 33.38 | 0.73 | 24.73 |
| 10 | 0.56 | 28.17 | 0.72 | 31.85 | 0.64 | 22.97 |

Note: N/A: not applicable; truck minimum speed is greater than 75 mi/h.

AASHTO indicates that climbing lanes should be extended beyond the crest of the curve for a distance that allows a truck to accelerate to a speed that is (*a*) within 10 mi/h of the passenger vehicle speed and (*b*) at least 40 mi/h (AASHTO, 2018).

## 4.3. Research Approach

The simulations executed for this task were designed to achieve several objectives:

- Outline an approach that can be used to determine truck deceleration along an upgrade, using relatively recent truck performance data consistent with that from the HCM. This approach is intended to supplement the material contained in the AASHTO Green Book.
- Outline an approach that can be used to determine the acceleration distance, on a level grade, needed downstream from a climbing lane for commercial trucks to accelerate to within 10 mi/h of the passenger car average speed.
- Develop models that estimate the expected difference in average speed, percent followers, and follower density for a one-lane versus two-lane (i.e., climbing lane) upgrade link configuration, as measured between the start and end of the upgrade link.

As such, several different simulation experiments were designed and executed. The first question was answered using two experiments: one that determined the deceleration of heavy vehicles on upgrades and one that determined the acceleration of heavy vehicles on level terrain after exiting an upgrade.

### Truck Deceleration Along a Climbing Lane

For this experiment, the network configuration consisted of three 10,000 ft links. The first and last links were level grade with free flow speeds set to 65 mi/h. The middle link varied in grade from 2% to 8%, at 1% increments. Preliminary testing had determined that a 10,000 ft long upgrade link was more than sufficient for trucks to reach their lowest speed on the upgrade, regardless of speed entering the upgrade. For the 8% upgrade, this speed was approximately 27 mi/h, which corresponds to what is commonly referred to as ‘crawl speed’.

The entering flow rate specified for the test direction was 100 veh/h, limited to vehicles belonging to the ‘Large Truck’ fleet type. This fleet type includes both intermediate and interstate semitrailer vehicles. Only interstate semitrailer vehicles were used in the analysis because they have a worse power-to-weight ratio than intermediate semitrailer vehicles. Furthermore, any trucks that were identified as being in a ‘follower’, rather than ‘leader’ platoon status were excluded from the analysis. The analysis truck sample size for each upstream speed ranged from approximately 55 to 65 vehicles, resulting from a 1-hour simulation period.

The results from the deceleration test provide the lengths at which trucks reach their minimum speed (see Figure 4-4).

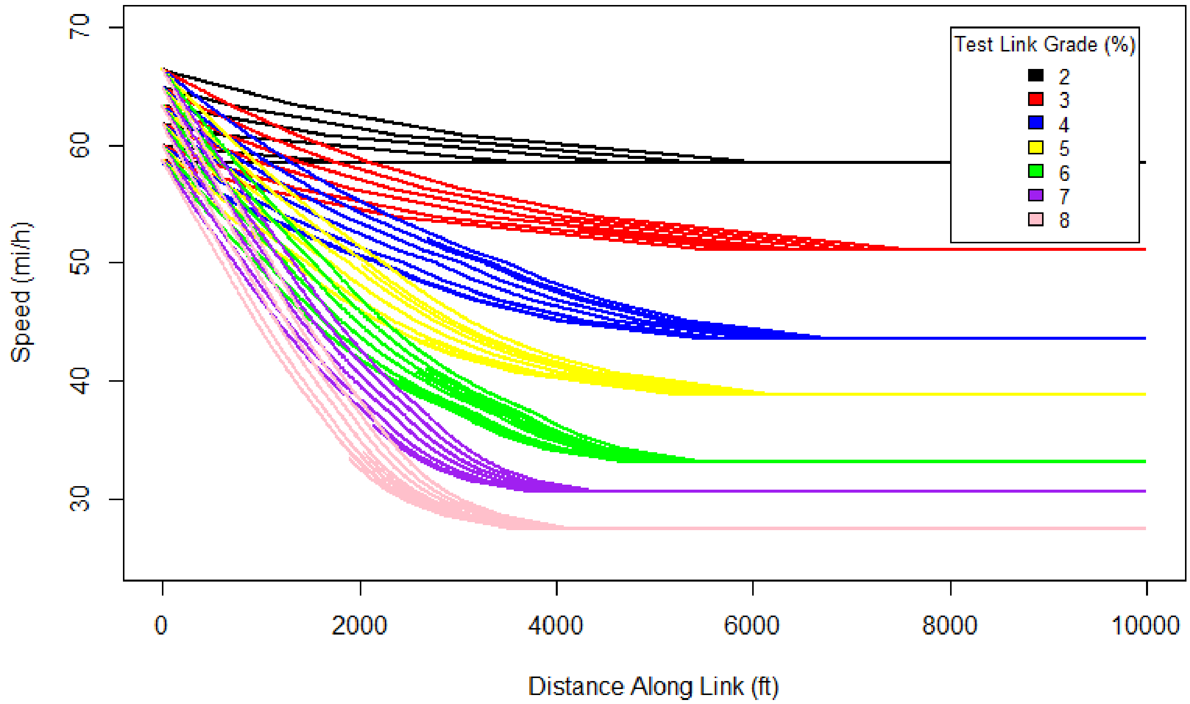


Figure 4-4. Truck Average Speed Versus Distance Traveled on an Upgrade

The spread of starting speeds for each set of upgrade curves correspond to the variation of desired speeds based on driver type (ranging from 3-8 by default for commercial trucks) around the average desired speed. The default values used in SwashSim are shown in Table 4-8.

Table 4-8. SwashSim Default Desired Speed Multipliers by Driver Type

| Driver Type | Desired Speed Multiplier |
|---|---|
| 3 | 0.950 |
| 4 | 0.970 |
| 5 | 1.000 |
| 6 | 1.025 |
| 7 | 1.050 |
| 8 | 1.075 |

Additionally, the average desired speed proportion varies by vehicle type. It is 1.0 for passenger vehicles, 0.98 for single unit trucks, 0.95 for intermediate semitrailers, and 0.95 for interstate semitrailers. For example, the desired speed for an interstate semitrailer with a driver type of 3 entering a link with an FFS of 65 mi/h is:

$$65 \text{ mi/h} \times 0.95 \text{ (for interstate semitrailer)} \times 0.95 \text{ (for driver type 3)} = 58.7 \text{ mi/h}$$

Likewise, the desired speed for an interstate semitrailer with a driver type of 8 entering a link with an FFS of 65 mi/h is:

$65 \times 0.95 \times 1.075 = 66.4$ mi/h

As expected, the curves in Figure 4-4 are generally consistent with those in Figure 4-3. It should be noted that the starting speed for the curves in Figure 4-3 is 75 mi/h versus 65 mi/h for the curves in Figure 4-4. Further, the curves of Figure 4-3 show just a single line for each upgrade percent, as that line represents the average speed across all driver types.

*Implementation of Results*

Figure 4-3 can be used to determine the distance traveled along an upgrade when an interstate semitrailer's speed drops to 10 mi/h below its starting speed when entering the upgrade. For example, for a starting speed of 65 mi/h and an upgrade of 6%, the truck speed would reduce to 55 mi/h after approximately 960 ft (see Figure 4-5).

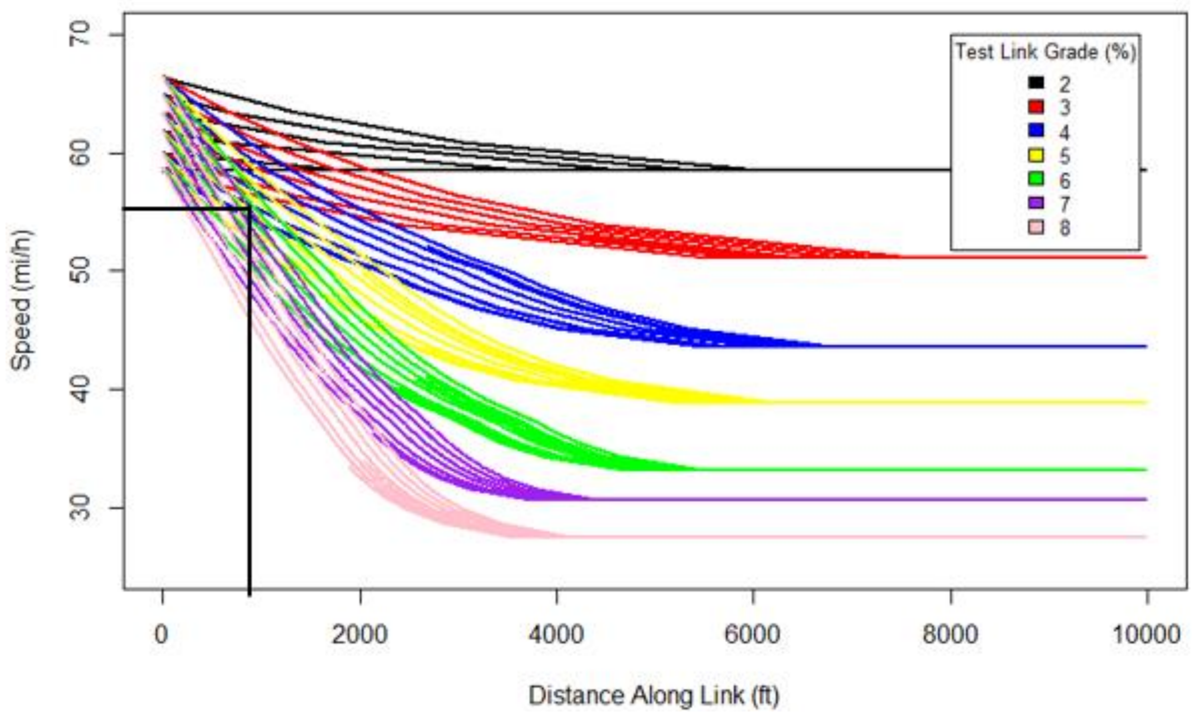


Figure 4-5. Truck Speed Reduction of 10 mi/h on 6% Upgrade

In this example, the 55 mi/h speed was referenced to approximately the midpoint of the 6% (green) speed lines. Different lines can be referenced as desired to account for more or less conservative driver types.

Given the consistency between the curves in Figure 4-4 and Figure 4-3, the calculation approach presented in the literature review section can be used in lieu of Figure 4-4, assuming results for the 'average' truck driver are acceptable. For example, assume the analyst wants to determine the average interstate semitrailer speed at the end of a 4000-ft long 6% upgrade. Following the guidance to adjust the values from Figure 4-3 for a starting speed of 65 mi/h rather than 75 mi/h, Figure 4-3 yields a distance of 950 ft (see Figure 4-6).

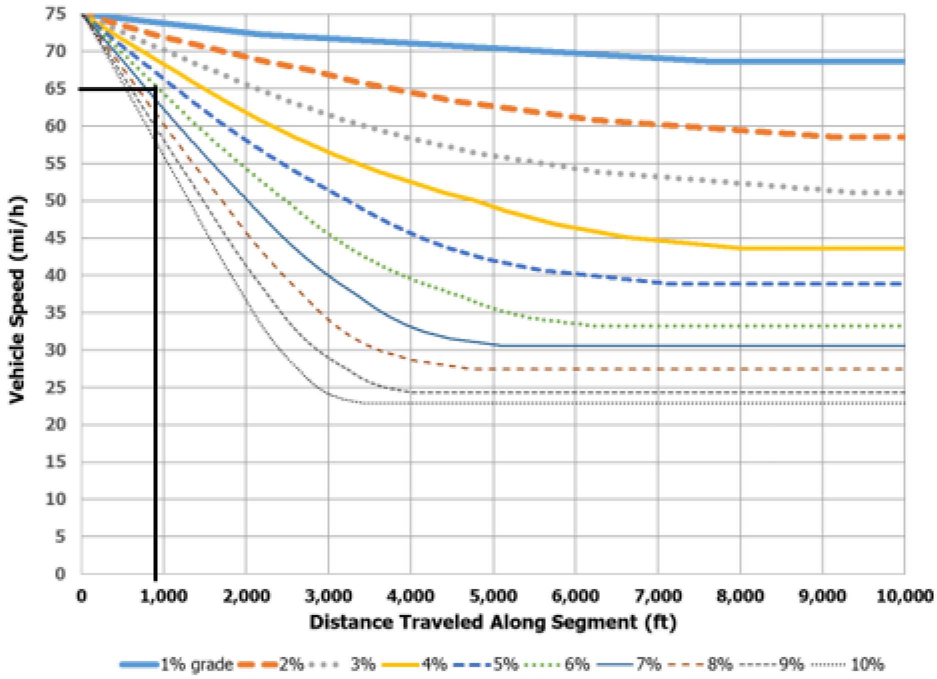


Figure 4-6. Distance Adjustment for Starting Speed of 65 mi/h on 6% Upgrade from Figure 4-3

These distance adjustment values have been tabulated for initial speeds from 20-75 mi/h in increments of 5 mi/h for upgrades of 1-10% in 1% increments. These values are presented in Table 4-4 through Table 4-6 for the three different truck types. For this example (interstate semitrailer, 6% upgrade, 65 mi/h starting speed), Table 4-6 provides an adjustment distance of 0.18 mi (950 ft), consistent with the value read from Figure 4-6.

Eq. 4-1 can be applied to determine the average interstate semitrailer speed at the end of the 4000-ft long grade as follows:

Add grade adjustment distance to actual grade distance: 950 + 4000 = 4950 ft (0.9375 mi)
Obtain equation coefficients from Table 4-3 for a 6% upgrade:
*a*: −60.94040, *b*: 12.96240, *c*: 7.63790
Substitute values into Eq. 4-1 and calculate velocity:

$$V = 75 - 60.94040 \times 0.7576 + 12.96240 \times 0.7576^2 + 7.63790 \times 0.7576^3 = 35.6 \text{ mi/h}$$

This value is consistent with value determined from Figure 4-3 (see Figure 4-7).

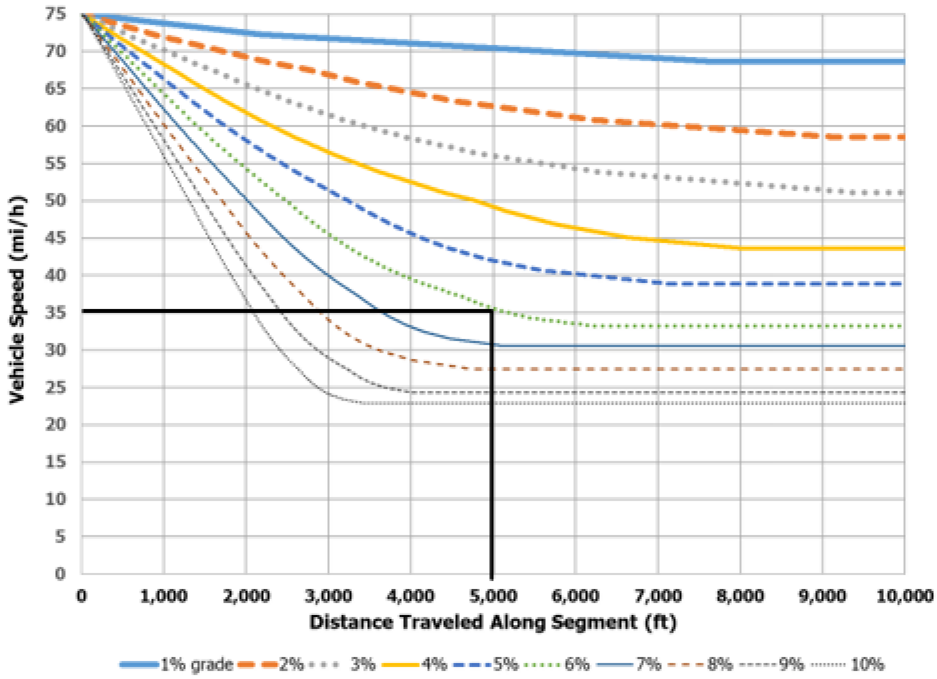


Figure 4-7. Interstate Semitrailer Speed at End of 4000-ft 6% Upgrade with Initial Speed of 65 mi/h

### Truck Acceleration Downstream of Climbing Lane

This experiment used the same general network configuration as for the upgrade deceleration experiment; that is, three 10,000 ft links. For this experiment, however, the middle link grade was set to level and its free flow speed was varied from 25-65 mi/h at 5 mi/h intervals. This range of starting speeds covers the range of speeds expected at the end of the upgrades tested in the previous experiment.

Figure 4-8 shows the resulting acceleration curves for interstate semitrailer trucks accelerating on a level grade starting at speeds from 25-60 mi/h, in increments of 5 mi/h, to a speed of 65 mi/h.

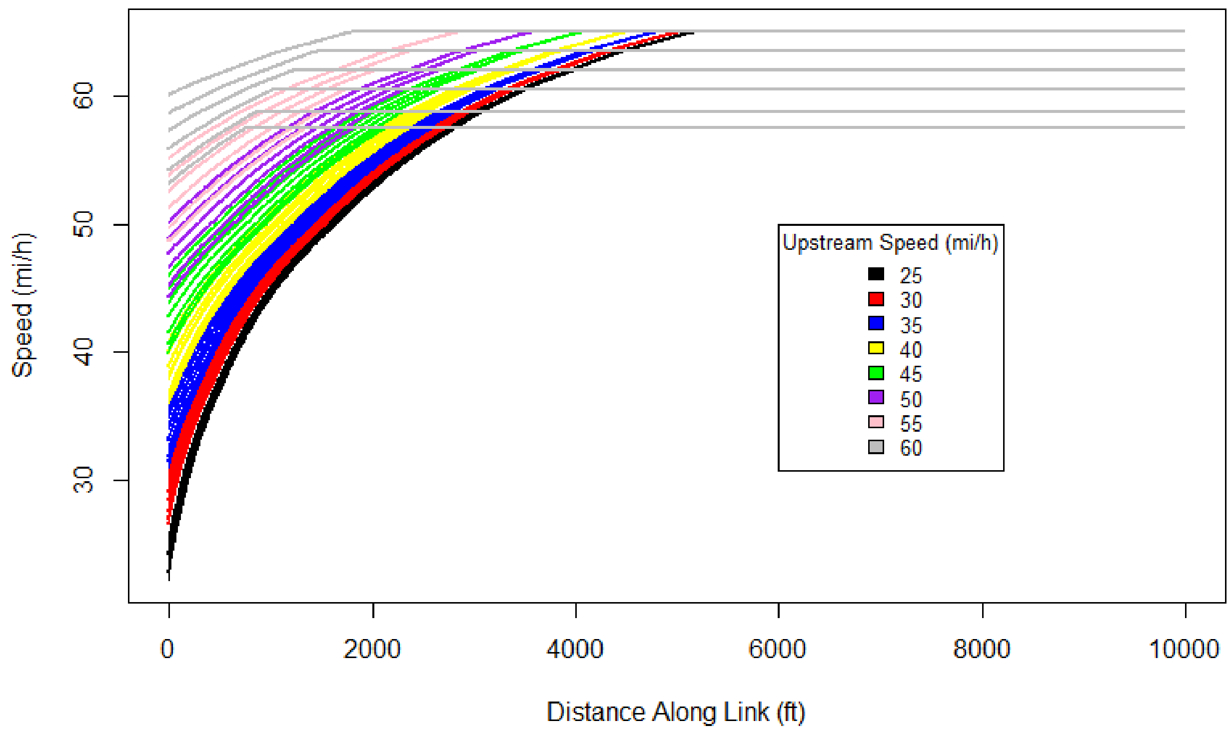


Figure 4-8. Interstate Semitrailer Speed Versus Distance Traveled on Level Grade when Accelerating from Given Upstream Speed to 65 mi/h

As was the case for the deceleration tests, there are multiple lines for each upstream speed, one for each driver type. The entering flow rate specified for the test direction was 100 veh/h, limited to vehicles belonging to the 'Large Truck' fleet type. This fleet type includes both intermediate and interstate semitrailer vehicles. Only interstate semitrailer vehicles were used in the analysis because they have a worse power-to-weight ratio than intermediate semi-trailer vehicles. Furthermore, any trucks that were identified as being in a 'follower', rather than 'leader' platoon status were excluded from the analysis. The analysis truck sample size for each upstream speed ranged from approximately 55 to 65 vehicles, resulting from a 1-hour simulation period.

The simulation run results were processed to identify the point at which the most conservative driver reached a speed ≤ 10 mi/h of the FFS of the downstream acceleration link (the third link). The most conservative driver (i.e., driver type 3) was used since that vehicle requires the longest distance to accelerate given its lowest desired acceleration rate. Three downstream free-flow speeds were considered: 45, 55, and 65 mi/h. For these speeds, the target truck speed was 35, 45, and 55 mi/h, respectively.

Table 4-9 gives the resulting distances necessary for interstate semitrailer vehicles to accelerate from a given upstream speed (assumed to the be the lowest speed on an upgrade) to within 10 mi/h of the downstream free flow speed on level grade.

Table 4-9: Acceleration Lane Lengths by Entry Free Flow Speed

| Upstream Avg. Starting Speed (mi/h)[1] | Length of Acceleration Lane (ft)[2, 3] | | |
|---|---|---|---|
| | 65 mi/h Downstream Speed | 55 mi/h Downstream Speed | 45 mi/h Downstream Speed |
| 25 | 2365 | 1075 | 405 |
| 30 | 2260 | 970 | 300 |
| 35 | 2120 | 830 | 160 |
| 40 | 1910 | 615 | - |
| 45 | 1645 | 355 | - |
| 50 | 1360 | 80 | - |
| 55 | 1190 | - | - |
| 60 | 310 | - | - |

[1] For an upstream starting speed in between the values listed, a linear interpolation of the acceleration lane length is appropriate.

[2] For a downstream speed in between the values listed, a linear interpolation of the acceleration lane length is appropriate.

[3] It is recommended that a minimum acceleration lane distance of 1/8 mi (660 ft) be used to allow sufficient time for a truck driver to evaluate gaps for merging into the regular lane.

*Implementation of Results*

Use the method outlined in the previous section (Truck Deceleration Along a Climbing Lane) to determine the truck minimum speed along the upgrade. Then use that speed along with the target average traffic stream speed downstream of the upgrade with Table 4-9 to determine the necessary length of acceleration lane.

Note that these acceleration lane values apply only to level or negative grades (in which case the distances are conservative). It was beyond the scope of this study to examine positive grades for the roadway downstream of the climbing lane.

## Performance Measure Improvement from Climbing Lane

The purpose of this experiment was to quantify the impact of a climbing lane on traffic stream performance measure values. Two different network configurations were used in this experiment:

1) A climbing lane (2 directional lanes along the upgrade, along with an acceleration lane on the level grade portion immediately downstream of the climbing lane, with acceleration distances set according to the previous experiment.
2) A single directional lane on the upgrade (i.e., no climbing lane), in which trucks would create platooning under moderate to heavy traffic volumes. Because there is no climbing lane, this configuration does not include an acceleration lane downstream of the upgrade.

The network link configuration consisted of the following:

- 10 links, 0.5 miles in length, 0% grade, 1 lane in analysis direction
- 1 upgrade link (varying length and grade, as discussed below)
    - Network Configuration 1: 2 lanes in analysis direction
    - Network Configuration 2: 1 lane in analysis direction
- 1 link, 0% grade
    - Network Configuration 1: 2 lanes in analysis direction, varying length as discussed below
    - Network Configuration 2: 1 lane in analysis direction, used the same length as for Network Configuration 1 just for consistency in overall network length, even though there is no acceleration lane in this configuration
- 1 downstream link, 0.5 mi, 0% grade, 1 lane in analysis direction

As seen in Figure 4-4, a 2% upgrade has minimal impact on truck speeds. Thus, this grade was excluded from the full experimental design. In order to keep the number of experimental scenarios to a reasonable number, upgrades of 3-8% were paired into three groups (3-4%, 5-6%, 7-8%) and assigned four climbing lane lengths separated by equal intervals. The final length for each grade group is a distance long enough for trucks to reach crawl speed. The upgrade lengths are shown in Table 4-11.

The process by which the acceleration lane distances were set is described for the 5-6% grade group, as follows. Using Figure 4-4 and the midpoint of the 6% grade speed-distance curves (green lines) for each distance, the corresponding truck speed was identified (see Figure 4-9). The corresponding acceleration lane lengths for the identified speeds were obtained from Table 4-9. These acceleration lane lengths are shown in Table 4-11.

Table 4-10: Climbing Lane Network Settings

| Variables | Settings |
|---|---|
| Grade (%) | 4, 6, 8 |
| Lengths of Climbing and Acceleration Lanes | Described in following table |

Table 4-11: Climbing Lane Length by Grade and Corresponding Acceleration Lane Length

| Grade (%) | Lengths of Climbing Lane and (Acceleration Lane) (ft) |
|---|---|
| 3-4 | 2000 (1360), 4000 (1645), 6000 (1910), 8000 (1910) |
| 5-6 | 1500 (1645), 3000 (2120), 4500 (2260), 6000 (2260) |
| 7-8 | 1125 (1645), 2250 (2260), 3375 (2365), 4500 (2365) |

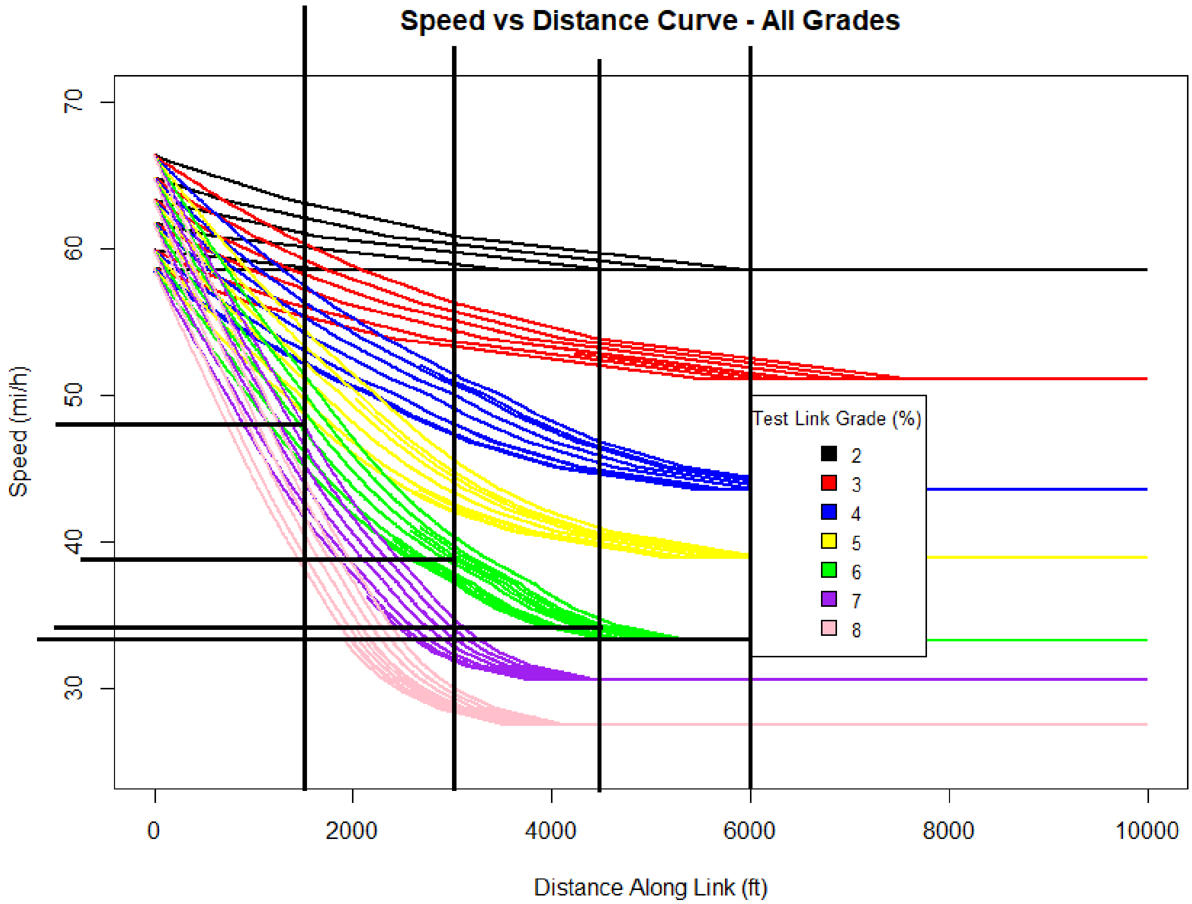


Figure 4-9. Interstate semitrailer speeds at specified distance on 6% upgrade

The experimental design settings for the traffic characteristics are shown in Table 4-12. Given the disruptive operational impacts that opposing traffic can have with upstream passing maneuvers, opposing traffic was limited to 100 veh/h with a fixed heavy vehicle percentage of 10%. The input free-flow speed was fixed at 60 mi/h.

Table 4-12: Climbing Lane: Traffic Characteristics Experimental Design Settings

| Variables | Settings |
|---|---|
| Flow Rate (veh/h) | 200, 600, 1000 |
| Heavy Vehicle Percentage (%)[1] | 5, 10, 15 |
| Level of Platooning Entering Upgrade Link | Low, High |

[1] Heavy vehicle split is 60% single unit truck (SUT) / 40% tractor + semi-trailer

For the ‘low’ platooning level, the full distance (10 links, 5.0 miles) preceding the upgrade link was set to ‘passing allowed’ in the oncoming lane. For the ‘high’ platooning level, the full distance preceding the climbing lane was set to ‘passing prohibited’ in the oncoming lane.

The various experimental design settings resulted in a total of 216 simulation scenarios used for the analysis, the product of the following:

- 3 climbing lane upgrade percentages
- 4 climbing lane upgrade lengths
- 2 platooning levels entering the climbing lane
- 3 flow rates
- 3 heavy vehicle percentages

Six replications were run for each scenario, with the average values of the six sets of results for each scenario used in the analysis. The simulation time used for each run was 90 minutes, which included 30 minutes of network initialization time.

For this analysis, traffic flow measurements were obtained from detectors placed immediately upstream of the start of the upgrade link and at the end of the upgrade link. Specifically, the performance measures of follower density, average speed, and percent followers were examined. As a reminder, follower density is the primary measure used for two-lane highway level of service and is defined in the 7th edition of the Highway Capacity Manual (HCM) by Eq. 15-35, as follows:

$$FD = \frac{PF}{100} \times \frac{v_d}{S} \qquad [4\text{-}2]$$

where:

$PF$ = percent followers in the analysis direction (%),
$v_d$ = analysis direction flow rate (veh/h), and
$S$ = average speed in the analysis direction (mi/h).

The results from the experimental design were used to develop models, via regression analysis, to estimate the expected change in each of the above listed performance measures for a one-lane versus two-lane (i.e., climbing lane) upgrade link. The equations presented here are intended to provide a quick estimate of how much improvement in traffic flow performance can be expected from adding a second lane to an upgrade segment of roadway. Note that only relative values of performance are estimated by the equations, not absolute values. If the analyst is interested in absolute values of performance measures for different upgrade configurations, the full HCM analysis methodology can be applied for this purpose. The estimated models are given in Eqs. 4-3 – 4-5.

$$\Delta\, FollowerDensity = \mathrm{Min}(0{,}5.950 - 0.5099 \times GradePct - 0.0004384 \times GradeLength - 0.0112 \times FlowRate - 0.01463 \times (FlowRate \times PropTrucks) + 0.6407 \times LowFollowers + 0.4874 \times MedFollowers) \qquad [4\text{-}3]$$

$$\Delta\, AvgSpeed = \mathrm{Max}(0, -7.366 + 1.242 \times GradePct + 0.0008116 \times GradeLength + 0.001686 \times FlowRate + 0.02936 \times (FlowRate \times PropTrucks) - 1.636 \times LowFollowers - 0.513 \times MedFollowers) \qquad [4\text{-}4]$$

$$\Delta\, \%Followers = \mathrm{Min}(0, -19.784 - 0.6391 \times GradePct - 0.001141 \times GradeLength - 0.007267 \times FlowRate + 7.173 \times LowFollower + 3.353 \times MedFollowers) \qquad [4\text{-}5]$$

where:

$\Delta$*FollowerDensity* = expected improvement in follower density between start and end of upgrade link for two-lane versus one-lane configuration (followers/mi);
$\Delta$*AverageSpeed* = expected improvement in average speed between start and end of upgrade link for two-lane versus one-lane configuration (mi/h);
$\Delta$*%Followers* = expected improvement in percentage followers between start and end of upgrade link for two-lane versus one-lane configuration (%);
*GradePct* = incline of the grade (%);
*GradeLength* = length of the upgrade (ft);
*FlowRate* = flow rate entering upgrade (veh/h);
*PropTrucks* = proportion of trucks entering upgrade (decimal);
*LowFollowers* = 1 if % followers entering the upgrade < 30, 0 otherwise; and
*MedFollowers* = 1 if % followers entering the upgrade ≥ 30 and < 60, 0 otherwise.

The goodness-of-fit, adjusted R-squared, for each of the models is 0.9348, 0.7958, and 0.7273, respectively. Some non-linear model forms were also examined, but none provided a better overall model fit than the simpler linear models. All model variables are significant at the 90% confidence level (t-statistic ≥ 1.282), with most significant at the 99% confidence level (t-statistic ≥ 2.326), using a one-tailed t-test. The models include the same variables, expect for $\Delta$*%Followers*, which does not include a term for number of large trucks (flow rate × proportion of trucks) as this term was insignificant for this model. All of the model variables also have logical coefficient signs. Note that the *LowFollowers* and *MedFollowers* coefficient values, and signs, are relative to a high followers input value (i.e., percentage of followers entering the upgrade link ≥ 60%). This is illustrated in the following example calculations. Note that for the two-lane configuration, follower density and average speed values at the end of the upgrade were calculated as the volume-weighted average of both lanes.

*Example Calculations*

Assume an existing single-lane upgrade along a section of two-lane highway is 3000 feet long and has an incline of 6%. Also assume during the peak hour that the flow rate entering the upgrade is 600 veh/h, the proportion of large trucks is 0.10, and the percentage of followers is 68%. An analyst would like to determine what level of improvement would be expected to the follower density, average speed, and percent followers measures at the end of the upgrade if a second lane is added to the analysis direction on the upgrade link, relative to the single lane upgrade configuration. Using Eqs. 4-3 – 4-5 give the following results.

$$\begin{aligned}\Delta\, FollowerDensity &\\ &= \mathrm{Min}(0, 5.950 - 0.5099 \times 6 - 0.0004384 \times 3000 - 0.0112 \times 600 \\ &\quad - 0.01463 \times (600 \times 0.10) + 0.6407 \times 0 + 0.4874 \times 0) = -6.0\end{aligned}$$

$$\begin{aligned}\Delta\, AvgSpeed &= \mathrm{Max}(0, -7.366 + 1.242 \times 6 + 0.0008116 \times 3000 + 0.001686 \times 600 \\ &\quad + 0.02936 \times (600 \times 0.10) - 1.636 \times 0 - 0.513 \times 0) = 5.3\end{aligned}$$

$$\Delta\,\%Followers = \mathrm{Min}(0, -19.784 - 0.6391 \times 6 - 0.001141 \times 3000 - 0.007267 \times 600 + 7.173 \times 0 + 3.353 \times 0) = -31.4$$

Since the percentage of followers entering the upgrade is greater than 60% (a high-followers condition), the equations use zero for both the *LowFollowers* and *MedFollowers* values. For these input values, it is expected that adding a second lane to the upgrade would reduce the follower density by 6.0 followers/mi, increase the average speed by 5.3 mi/h, and decrease the percentage of followers by 31.4% at the end of the upgrade relative to the single-lane upgrade configuration. Thus, for this set of input conditions, adding a climbing lane to the upgrade link is likely to provide significant performance benefits. Again, if absolutely values of the performance measures are desired, the analyst should consult the full two-lane highway analysis methodology of the HCM (Chapter 15). Also recall that the free-flow speed for this experimental design was fixed at 60 mi/h.

Note that these models have a minimum or maximum value of zero constraint. Due to the nature of regression analysis for a data set with a semi-limited range of values, it is possible to obtain illogical values for some combination of input values to the models. For example, for a low flow rate and low proportion of trucks, the difference in performance measures for a single lane versus two-lane upgrade are expected to be extremely small. Thus, it may be possible to obtain a value close to zero, but positive for the difference in follower density and negative for the difference in average speed, for such a situation. Thus, the delta follower density model results are constrained to a minimum value of zero, as adding a second lane to the upgrade should never result in a worse follower density than for the single upgrade lane configuration. The same applies for the percentage of followers. For average speed, adding a second upgrade lane should increase, or least not reduce, the average speed. Thus, the delta average speed model is constrained to maximum value of zero.

# 5. Passing Lane Merge/Diverge Configurations

## 5.1. Background

The focus of this task is to determine whether one or more of the merge configurations for a passing lane segment shown in Figure 5-1 have a statistically significantly different impact on downstream operations than the other merge configurations. The focus is not on precisely quantifying the impact, from a performance measure perspective, of the different merge configurations, although some discussion will still be provided in this regard.

The merge configurations are defined as follows:

A. Slower vehicles diverge into added lane and merge into regular lane before lane drop
B. Faster vehicles diverge into added lane and merge into regular lane before lane drop
C. Slower vehicles diverge into added lane and faster vehicles merge into regular lane before lane drop
D. Faster vehicles diverge into added lane and slower vehicles merge into regular lane before lane drop

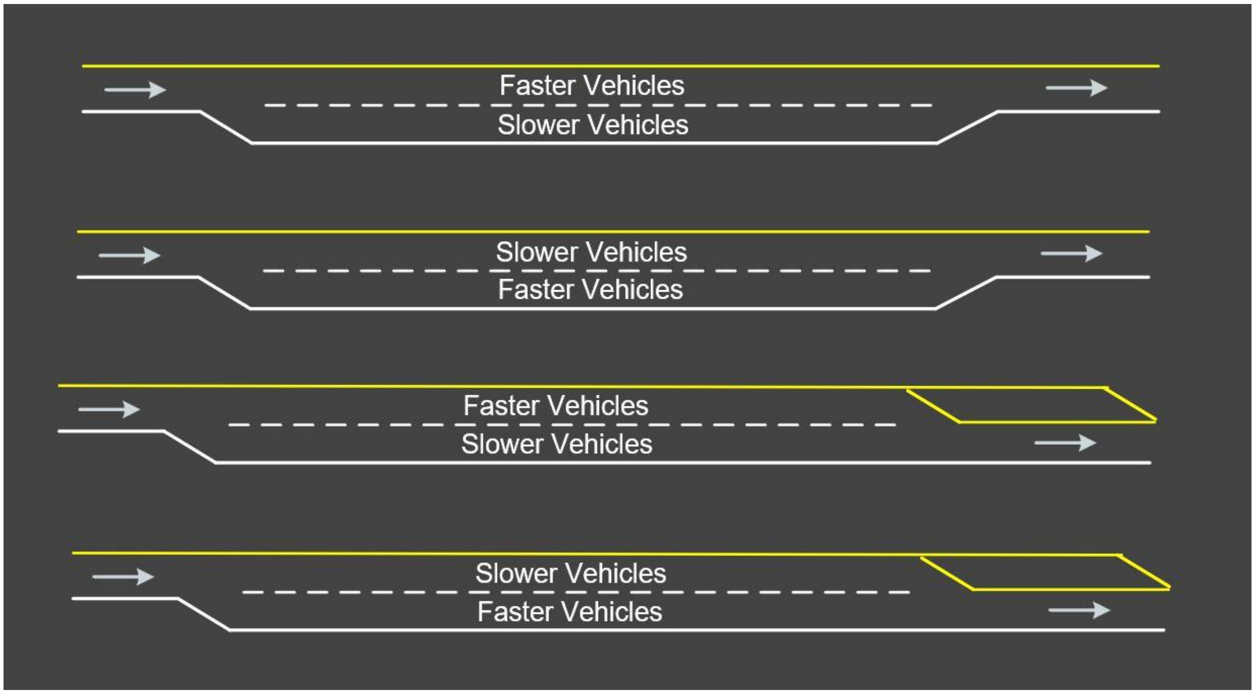


Figure 5-1. Merge Configurations A, B, C, D (from top to bottom)

Standard drawings/typical sections can be found for one or more these merge configuration designs in different state department of transportation design manuals. However, the selected design to implement, at least region-wide if not state-wide, appears to often be a matter of preference. In some instances, the selected design may be a function of issues such as drainage cross-slope needs or sight distance, but the research team was not aware at the time of this study of any quantitative operations guidelines for choosing a specific merge configuration design. The primary roadway design reference document, the AASHTO Green Book (AASHTO, 2018), discusses passing lanes in Chapter 3 (Section 3.4.4.1). While it provides quantitative guidance for the length of lane transition tapers, it does not address specific merge configuration designs.

## 5.2. Research Approach

The simulation experimental design is described in this section.

### *Experimental Design*

#### Network Configuration

The overall network link configuration consisted of the following:

- 10 links, 0.5 miles in length, 0% grade, 1 lane in analysis direction
- 1 passing lane link, 0% grade, varying length and merge configuration (as shown in Table 5-1)
- 2 links, 0.5 miles in length, 0% grade, 1 lane in analysis direction

Table 5-1: Passing Lane Configuration Settings

| Variables | Settings |
| --- | --- |
| Grade (%) | 0 |
| Passing Lane Length (mi) | 1, 2, 3 |
| Merge Configuration | A, B, C, D |

The various merge configurations for the passing lane link were setup in SwashSim as shown in Figure 5-2 through Figure 5-4.

The start of the passing lane link for all merge configurations is set up as shown in Figure 5-2. For merge configuration A, the outer lane (lane ID 1) is specified as the lane for slower vehicles to move into. For merge configuration B, the outer lane is specified as the lane for faster vehicles to move into.

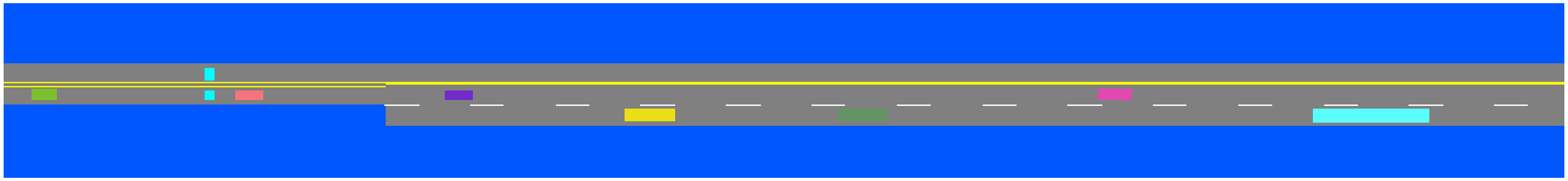

Figure 5-2. Upstream End of Passing Lane Link for all Merge Configurations

The end of the passing lane link for merge configurations A and B is set up as shown in Figure 5-3. The respective lane assignments for slower and faster vehicles are as described for Figure 5-2.

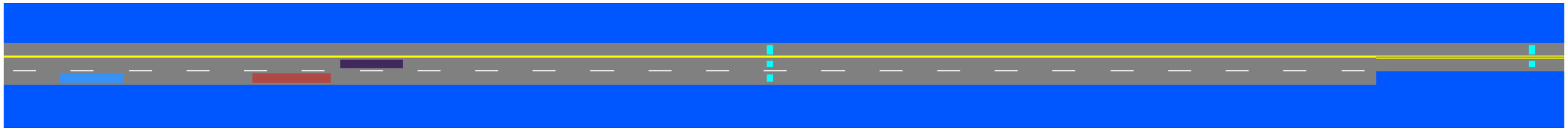

Figure 5-3. Downstream End of Passing Lane Link for Merge Configurations A and B

The end of the passing lane link for merge configurations C and D is set up as shown in Figure 5-4. For merge configuration C, the outer lane (lane ID 1) is specified as the lane for slower vehicles to move into. For merge configuration D, the outer lane is specified as the lane for faster vehicles to move into.

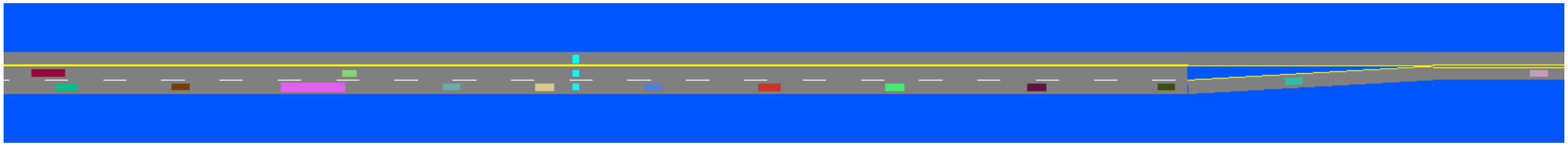

Figure 5-4. Downstream End of Passing Lane Link for Merge Configurations C and D

### Traffic Data Settings

The experimental design settings for the traffic characteristics are shown in Table 5-2.

Table 5-2: Traffic Data Settings

| Variables | Settings |
|---|---|
| Flow Rate (veh/h) | 300, 900, 1500 |
| Heavy Vehicle Percentage (%)[1] | 5, 10, 15 |
| Level of Platooning Entering Passing Lane | Low, High |

[1] Heavy vehicle split is 60% single unit truck (SUT)/ 40% tractor + semi-trailer

### Simulation Settings Summary

The various experimental design settings resulted in a total of 216 simulation scenarios used for the analysis, the product of the following:

- 4 passing lane merge configurations
- 3 passing lane lengths
- 2 platooning levels entering the passing lane
- 3 flow rates
- 3 heavy vehicle percentages

Six replications were run for each scenario, with the average values of the six sets of results for each scenario used in the analysis. The simulation time used for each run was 90 minutes, which included 30 minutes of network initialization time.

For this analysis, traffic flow measurements were obtained from detectors placed immediately upstream of the start of the passing lane link and immediately downstream of the end of the passing lane link.

## 5.3. Analysis Results

The difference in average speed, percent followers, and follower density between the start and end of the passing lane link was calculated for each of the 216 scenarios. The general descriptive statistics for these performance measures are given in Table 5-3. Note that the difference values are calculated as the downstream value (i.e., value just past the point where the two lanes for the passing lane link have merged back into a single lane) minus the upstream value (i.e., the value just before an additional lane is added for the passing lane link). Passing lane links are generally effective at dispersing platoons; thus, it is expected that the % followers value should almost always be lower after the passing lane link than before it. The values in Table 5-3 are consistent with this expectation. Average speed, however, does not necessarily follow a consistent trend between upstream and downstream of the passing lane link. While the average speed within the passing lane link (i.e., 2 lanes in the same direction) generally improves, but often by only a moderate amount, the average speed just downstream of the merge point can be heavily influenced by the "friction" of the merging operations near the point of the lane drop. This friction can be quite significant when the flow rate is high. Follower density values are a function of average speed and % followers (as well as flow rate).

Table 5-3. Descriptive Statistics for Performance Measures of Interest

| Statistic | Δ Avg. Speed (mi/h) | Δ % Followers (%) | Δ Follower Density (followers/mi) |
|---|---|---|---|
| Count | 216 | 216 | 216 |
| Mean | –0.799 | –9.73 | –0.735 |
| Median | 0.216 | –9.02 | –0.742 |
| Std. Dev. | 3.12 | 6.65 | 6.65 |
| Minimum | –15.3 | –29.0 | –3.82 |
| Maximum | 3.62 | 1.29 | 6.78 |

With the primary focus being on whether the difference between performance measure values immediately upstream and downstream of the passing lane link is influenced by the downstream merge configuration, an analysis of variance (ANOVA) was conducted. First, boxplots and tables of standard descriptive statistics are presented in Figure 5-5 through Figure 5-7 and Table 5-4 through Table 5-6.

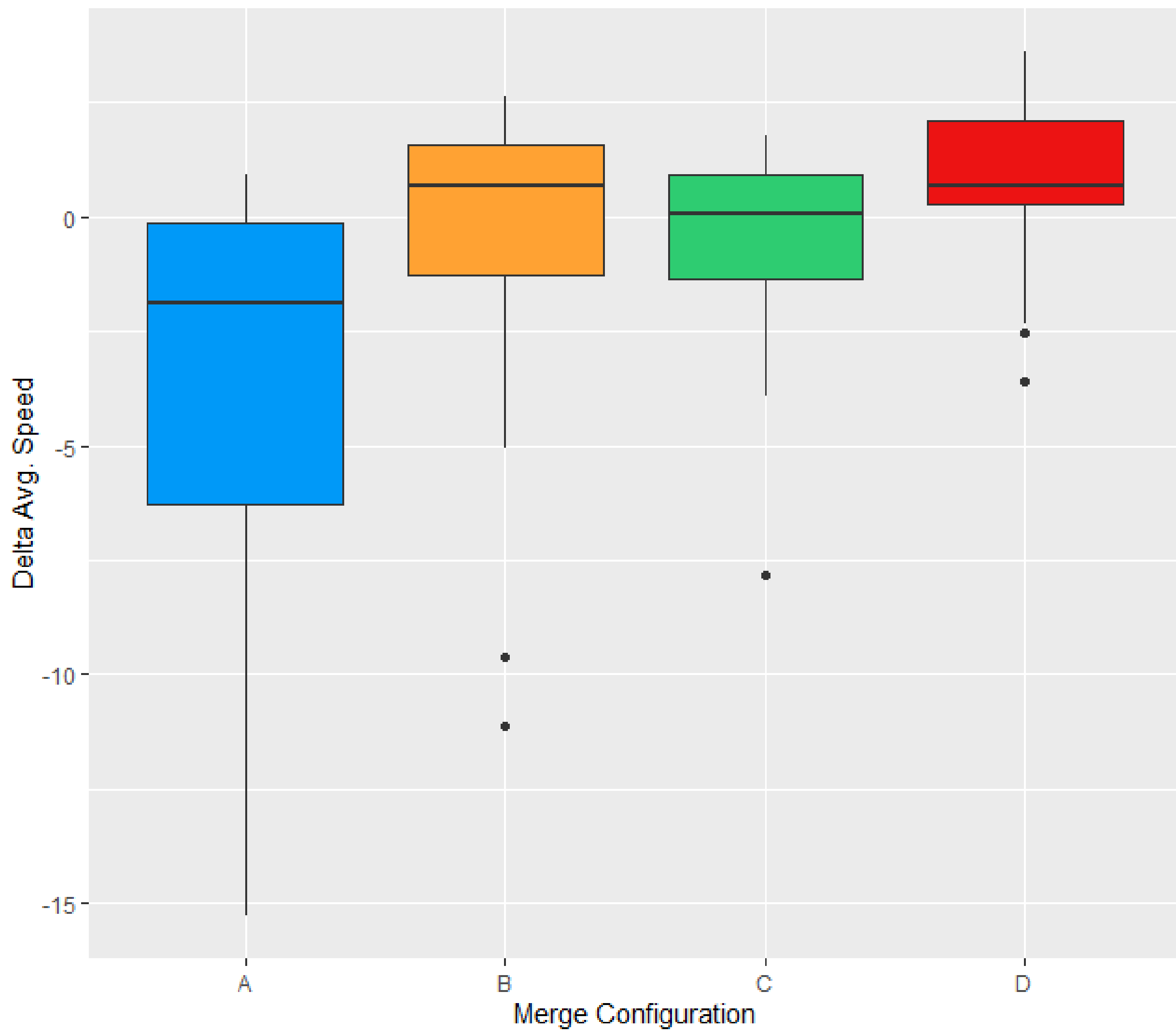


Figure 5-5. Difference in Upstream and Downstream Average Speed by Merge Configuration

Table 5-4. Descriptive Statistics for Difference in Upstream and Downstream Average Speed by Merge Configuration

| Statistic | Merge Configuration | | | |
|---|---|---|---|---|
| | A | B | C | D |
| Count | 54 | 54 | 54 | 54 |
| Mean | –3.30 | –0.291 | –0.442 | 0.837 |
| Median | –1.88 | 0.701 | 0.053 | 0.666 |
| Std. Dev. | 4.01 | 2.85 | 1.83 | 1.57 |
| Minimum | –15.30 | –11.10 | –7.81 | –3.58 |
| Maximum | 0.92 | 2.65 | 1.77 | 3.62 |

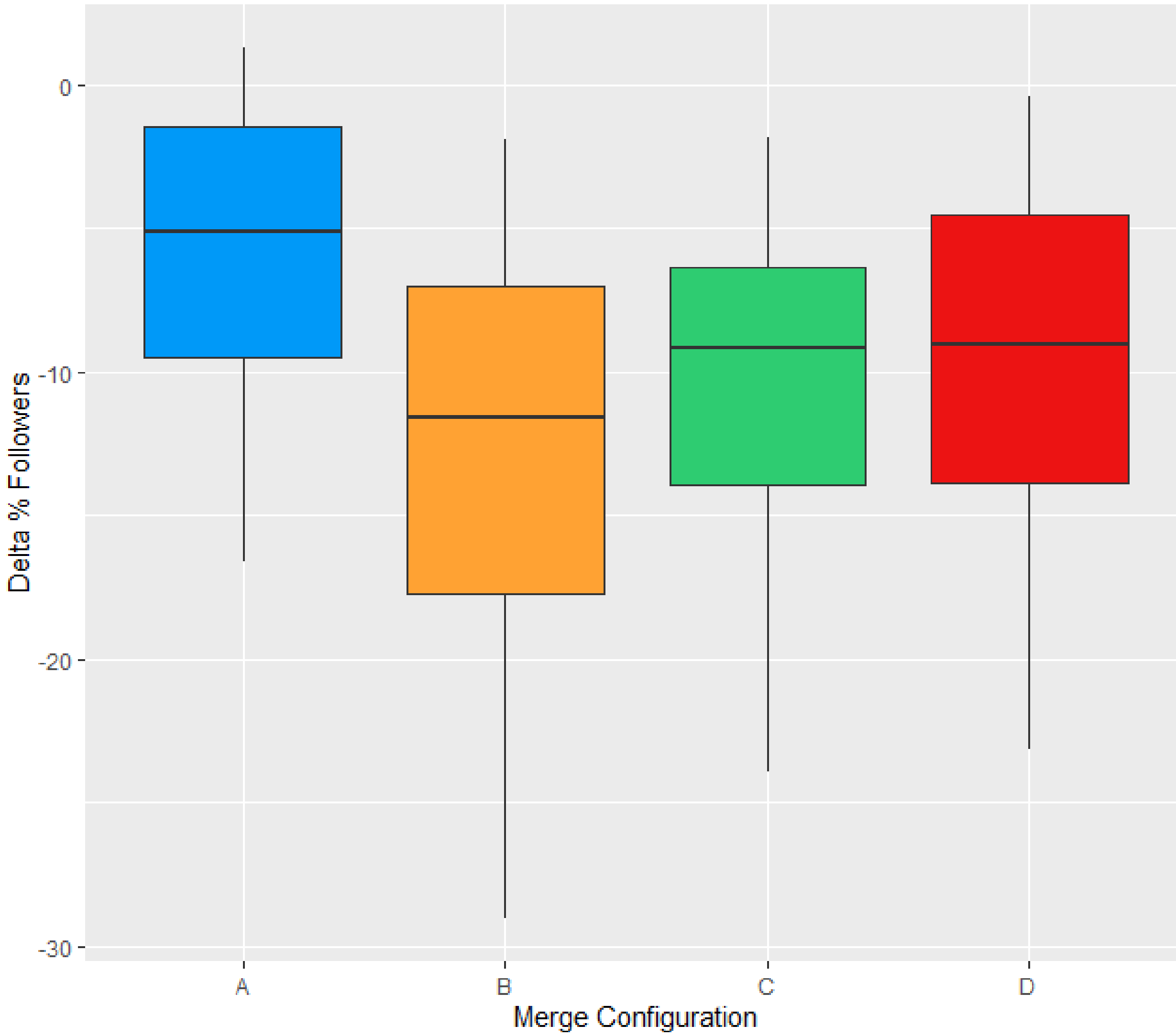

Figure 5-6. Difference in Upstream and Downstream % Followers by Merge Configuration

Table 5-5. Descriptive Statistics for Difference in Upstream and Downstream % Followers by Merge Configuration

| Statistic | Merge Configuration | | | |
|---|---|---|---|---|
| | **A** | **B** | **C** | **D** |
| Count | 54 | 54 | 54 | 54 |
| Mean | –5.87 | –12.9 | –10.5 | –9.68 |
| Median | –5.08 | –11.6 | –9.2 | –9.02 |
| Std. Dev. | 5.13 | 7.31 | 5.88 | 6.24 |
| Minimum | –16.6 | –29.0 | –23.9 | –23.2 |
| Maximum | 1.29 | –1.92 | –1.87 | –0.434 |

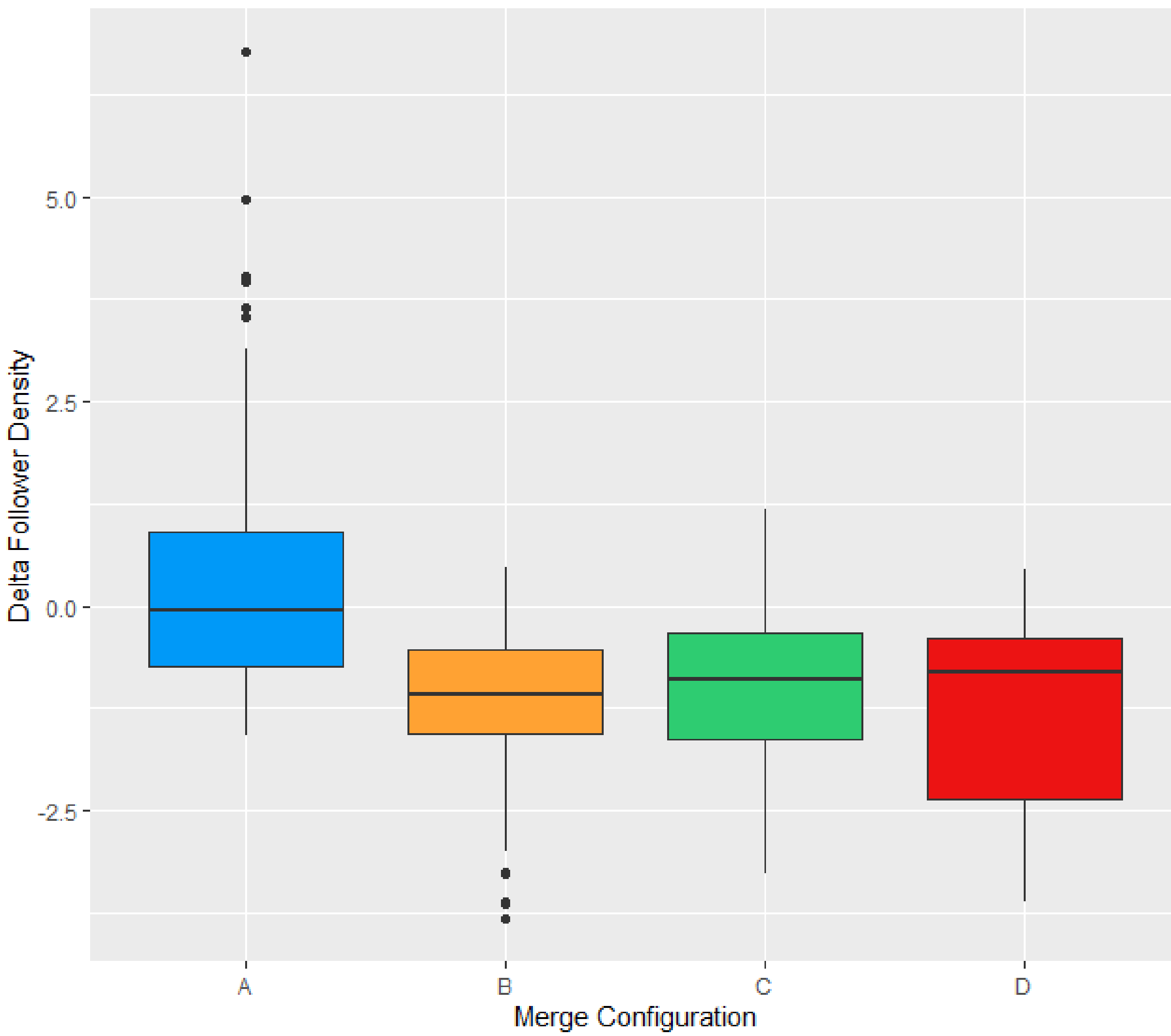


Figure 5-7. Difference in Upstream and Downstream Follower Density by Merge Configuration

Table 5-6. Descriptive Statistics for Difference in Upstream and Downstream Follower Density by Merge Configuration

| Statistic | Merge Configuration | | | |
|---|---|---|---|---|
| | **A** | **B** | **C** | **D** |
| Count | 54 | 54 | 54 | 54 |
| Mean | 0.553 | –1.23 | –0.996 | –1.26 |
| Median | –0.0635 | –1.08 | –0.903 | –0.804 |
| Std. Dev. | 5.13 | 7.31 | 5.88 | 6.24 |
| Minimum | –1.58 | –3.82 | –3.28 | –3.62 |
| Maximum | 6.78 | 0.472 | 1.18 | 0.442 |

*Analysis of Variance*

### Difference in Average Speed

ANOVA yielded F-values of 22.41, 11.93, and 23.73 for the differences in average speed, % followers, and follower density, respectively. These F-values indicate, for a 99+% confidence level, that the null hypothesis that the effect on the change in each respective performance measure between the start and end of the passing lane for all four merge configurations is equal. Thus, at least one of the four merge configurations has a statistically significant effect that is different from the other configurations. The Tukey-Kramer (T-K) Honest Significant Difference (HSD) test is used to identify which configurations have a different impact from the other configurations. The T-K HSD results are shown in Table 5-7. The highlighted (gray) rows indicate the merge configurations that are statistically significantly different from one another.

Table 5-7. T-K HSD Results

| | Δ Avg. Speed | | Δ % Followers | | Δ Follower Density | |
|---|---|---|---|---|---|---|
| | **Estimate** | **t-value** | **Estimate** | **t-value** | **Estimate** | **t-value** |
| B - A == 0 | 3.0109 | 5.717 | –7.007 | –5.881 | –1.78617 | –7.098 |
| C - A == 0 | 2.8591 | 5.429 | –4.636 | –3.891 | –1.54956 | –6.158 |
| D - A == 0 | 4.139 | 7.859 | –3.812 | –3.199 | –1.81574 | –7.216 |
| C - B == 0 | –0.1518 | –0.288 | 2.371 | 1.99 | 0.23661 | 0.94 |
| D - B == 0 | 1.128 | 2.142 | 3.195 | 2.682 | –0.02957 | –0.118 |
| D - C == 0 | 1.2799 | 2.43 | 0.824 | 0.692 | –0.26619 | –1.058 |

For the difference in average speed, all merge configurations have a statistically significantly different impact, except for configuration B versus configuration C. For the difference in % followers, all merge configurations have a statistically significantly different impact, except for configuration C versus configuration D. For the difference in follower density, only merge configuration A is statistically significantly different from the other merge configurations. The reference t-value is 1.645, corresponding to a 90% confidence level for a 2-tailed (i.e., $\alpha = 0.05$) test.

## 5.4. Conclusions and Recommendations

This task was focused on examining the impact of four different merge configurations on the difference in several performance measure values from immediately upstream to immediately downstream of the passing lane link. The ANOVA and T-K HSD results demonstrate that a specific merge configuration can have a smaller or larger impact on the change in value of key performance measures from upstream to downstream of the passing lane link. For example, referring to the values in Table 5-7, it is expected that merge configuration A would perform more poorly than merge configurations B, C, and D with respect to the change in average speed. Comparing merge configuration A to merge configuration D, for example, merge configuration A would result in an approximately 4.1 mi/h greater speed reduction, on average across the range of input conditions tested, from immediately upstream to immediately downstream of the passing lane relative to merge configuration D. Likewise, merge configuration D would perform slightly better than merge configurations B and C for the difference in average speed (1.1 and 1.3 mi/h, respectively).

Similarly, merge configuration A performs worse than merge configurations B, C, and D with respect to the change in % followers. That is, merge configurations B, C, and D all result in a larger reduction of % followers from upstream to downstream of the passing lane link than merge configuration A. Of the four configurations, merge configuration B provides the best performance for % followers, albeit the difference between merge configurations C and D is not statistically significant.

For follower density, the only statistically significant differences are between merge configuration A and merge configurations B, C, and D, where merge configuration A is again the worst performing among the four configurations.

Observations of the passing lane operations across numerous simulations confirmed that these results are logical. Merge configuration A is generally the worst performing configuration because slower vehicles, many of which are large trucks, have to change lanes twice. The differences between merge configurations B, C, and D are generally much smaller than when each of those configurations is compared to merge configuration A. As mentioned previously, there is not statistically significant difference between merge configurations B, C, and D for follower density.

When comparing merge configurations C and D, the two configurations for which slower and faster vehicles are each required to make one lane change, there is no statistically significant difference with respect to the % followers and follower density performance measures, but there is for average speed. Merge configuration D provides slightly better performance than merge configuration C, on average a 1.28 mi/h greater improvement in the change in average speed from immediately upstream to immediately downstream of the passing lane. In merge configuration D, large trucks are able to find gaps in the faster lane that are less likely to impact passenger cars. Generally, the trucks are looking to merge into the regular lane with plenty of margin before the end of the passing lane and they will defer to passenger cars and pull in behind them. With merge configuration C, passenger cars are generally trying to get past as many slower vehicles, especially trucks, in the adjacent lane as possible and will often move into to the continuous lane shortly before the passing lane ends. Such 'last-second' lane changes, especially in front of a truck can cause minor shockwave disturbances and lead to a more negative impact on average speed when compared to merge configuration D. See Appendix D (Modeling Speeds of Heavy Vehicles for Simulation), Section D.5.4, of the NCHRP Project 17-65 (Washburn, et al., 2018) report for more discussion about this issue.

Of course, with these results based on simulation, they are a function of the algorithms built into the simulation software (SwashSim). A description of the simulation logic used for passing lanes can be found in the SwashSim documentation (Washburn, 2023). Briefly, this logic was developed based on standard driving rules, empirical measurements as part of NCHRP Project 17-65, and other literature as documented in NCHRP Project 17-65. However, the field sites examined as part of NCHRP Project 17-65 were all of type configuration A. Thus, more research is needed, particularly empirical measurements from multiple merge configurations. Merge configuration is not currently considered in the HCM two-lane highway analysis methodology, but the results from this research suggest that it should be considered in further research.

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