APPENDIX 10A - AUXILIARY LANES

An auxiliary lane is an additional lane segment designed to effectively manage and restore existing capacity currently degraded by operational performance. An auxiliary lane is expected to restore (but not increase) effective existing system capacity caused by poor operations and address existing and future safety issues related to unique geometric and operational factors (e.g., intersections, grades, ramp spacing, and queuing build-up). These are locations where ODOT does not expect a statistically significant increase in vehicular capacity to the adjacent (i.e., upstream/downstream) roadway system.

The ODOT definition above is supplementary to the existing design (i.e., Highway Design Manual Section 601.2) and operational (i.e., Highway Capacity Manual Chapter 13 & 15) definitions, but is meant to be compatible with the current working environment and supports the overall objective stated below. All these definitions differ to some degree, and all should be taken in for full context and understanding.

The purpose of this document is to identify the auxiliary lane length that can be supported by data and where the excess length would operate more like a mainline or through lane (e.g., cruising) versus the auxiliary lane function (e.g., merging). This will help determine the point where a proposed improvement may act more like a system capacity increase than for addressing point operation and safety.

The application of these methodologies in this appendix are intended at the programming or scoping level where project descriptions and specific data is limited and where many projects need to be evaluated in a short timeframe. An example would be assessing for any new capacity-adding lane-mile additions to calculate greenhouse gas emissions for the STIP (Statewide Transportation Improvement Plan). Please note that these higher-level methodologies are not a substitute for full operational analysis.

In addition, Chapter 10 and Chapter 14 have additional considerations for auxiliary lanes such as multimodal impacts.

System capacity increases are long-distance improvements where the improvement is likely to make a difference by increasing demand and overall throughput into the project area. These are improvements that would be traditionally added to a travel demand model network. Point (or local) capacity is limited to the auxiliary lane location where the mainline lanes upstream and downstream of the auxiliary lane are the same, so the overall demand and throughput coming into the auxiliary lane section is unchanged. Throughput essentially is an "effective capacity" as it is based on the volumes that can travel through a particular section and it is limited by the number of upstream lanes. Auxiliary lanes, by their nature, will increase the total number of lanes in terms of point capacity as they restore the throughput (capacity) that was diminished because of trucks, slow vehicles, lane changes, etc. These allow the overall throughput of the mainline facility to be maintained throughout without creating a bottleneck section. Point capacity improvements would generally not be added to travel demand model networks as they would be too short to affect overall demand, or their purpose is safety-based rather than capacity-based (i.e., passing or climbing lanes).

Where possible, the data and analysis needs are generally at the scoping and programming levels. More detailed information can be used (e.g., project-level peak hour volumes) if available; however, the methodologies presented in this appendix are not a substitute for full operational analysis.

Freeway Weaving Lanes

These reduce turbulence caused by closely spaced diverging (i.e., to off-ramps) and merging (i.e., from on-ramps) traffic streams. Overall length of the weaving lane is controlled by the operational relationships between the mainline and the ramp volumes. Higher weaving flows will result in longer lengths required to safely satisfy the necessary weaving movements.

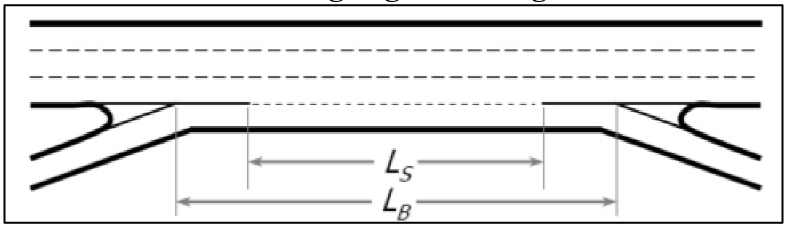
The analyst should be aware of any special considerations like grades which may affect acceleration/deceleration and the overall auxiliary lane length outside of the operational needs discussed here. Weaving lanes affected by these special considerations will require a more substantial analysis up to and beyond the typical capacity analysis shown in Highway Capacity Manual (HCM) Chapter 13. In addition, how weaving lanes are defined and accommodated in a travel demand model (i.e., shorter lengths may not be coded and lane assumptions for longer ones may differ anywhere from reduced to full capacity) may not be consistent across different models. These modeling differences need to be considered when developing volumes to use for full operational analysis or if daily volumes are taken directly from a travel demand model for application to this methodology.

It is possible to have a weaving lane that is longer than maximum operational distance needed to meet the weaving demand. In this case it functions like a basic freeway segment with separate merge and diverge sections with the auxiliary lane functioning like a regular through lane.

HCM Exhibit 13-11 and the related Equation 13-4 offer a comparison between the short weaving length (L_S; distance between the end/start of solid white striping where lane changing is discouraged) and the computed maximum length.

Exhibit 10A-1 below from the HCM shows the difference between the short length and the typical measured gore-to-gore length. If solid extension striping is not present, then the longer base length (L_B) is used.

Exhibit 10A-1: Weaving Segment Lengths



L_S = Weaving section short length between ends of solid lane stripes

L_B = Weaving section base length between the gore points

The volume ratio (VR) needs to be computed and compared with the maximum weaving lengths in Exhibit 10A-3 to determine if the given weaving segment length as defined in Exhibit 10A-1 is less than the indicated distances. The volume ratio is defined as the ratio between the weaving flows (i.e., sum of the freeway to off-ramp and freeway to on-ramp flows) and the total demand flow in the weaving section. Exhibit 10A-2 shows the individual flows that make up a weaving section. Exhibit 10A-3 has lengths for when the on and off ramps that form the weaving section are both a single lane and when one or the other is two lanes. Weaving sections with multilane ramps have more required lane changes needed to make the weaving movements.

If the weaving section has successive on or off-ramps (i.e., when the weaving lane is started with a loop on-ramp and then a diagonal on-ramp merges in downstream), then the ramp flows should be summed to a single value for application in this methodology. Both ramp volumes in this case would be subject to the same weaving or non-weaving flow patterns as for a single on or off-ramp. Weaving extents in this case would be from the ramp that starts the weaving lane to the one that ends it across the pair of interchanges.

Ramp-to-ramp demand is usually defaulted to zero in this screening process as its contribution will generally not make a substantial difference, especially if ramps are just single lanes. Ramp-to-ramp flows can be used if dual lane ramps and nested weaving lanes exist, but on or off-ramp flow will need to be sizable fractions of the total mainline volume to make much difference. However, it can be used if available by adding it into the VR equation denominator.

Note that the ramp-to-ramp demand cannot be easily computed from individual volumes and requires some sort of additional origin-destination (O-D) information from a special field study, a travel demand model, or "big-data/information" provider (i.e., "XXX"). The level-of-effort required to use O-D data for ramp-to-ramp flows is inconsistent with programming/scoping applications and should only be used at the refinement and project levels. It is recommended that if ramp-to-ramp volumes are desired to be included, then these need to be based on external O-D sources rather than the Oregon default ramp-to-ramp calculation shown in APM Appendix 11C.

Ramp-to-ramp flows outside of directly connecting highways together may be functioning as a local street replacement, which is discouraged in the Oregon Transportation Plan (see Strategy 1.3.2). Weaving auxiliary lanes can encourage inappropriate local use of the freeway system as an inadvertent effect when trying to solve a mainline operational and/or safety issue. The analyst needs to be aware of the surrounding local context to determine why these movements occur. For example, are drivers avoiding congestion at local intersections and streets and doing undesirable "short-hop" on/off local movements or is there a geographical obstruction such as a river with limited crossings that concentrates regional trips?

$$VR = \frac{(v_{FR}+v_{RF})}{(v_{FF}+v_{RF}+v_{FR}+v_{RR})}$$

Where:

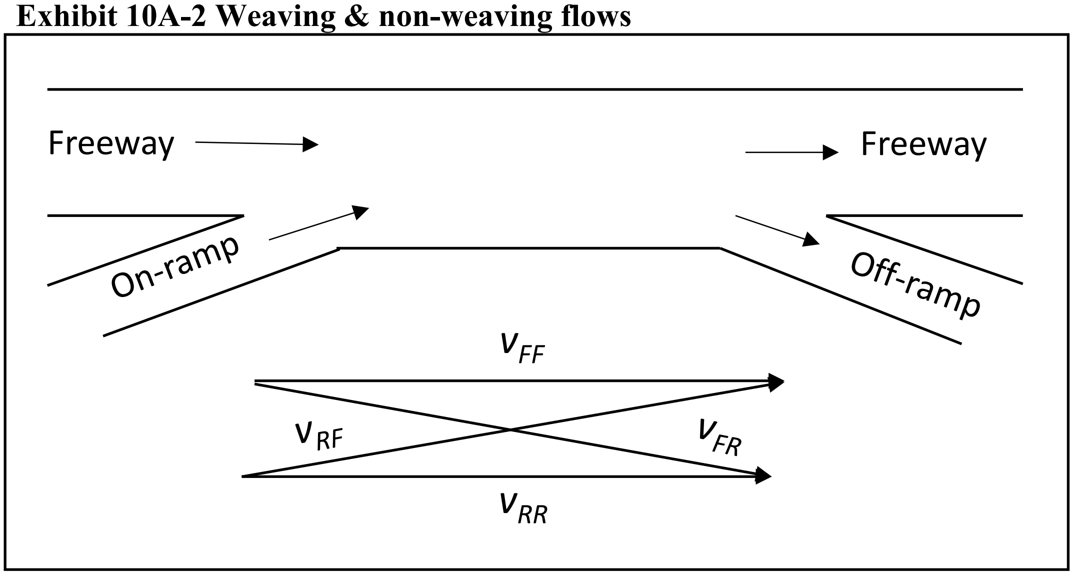
v_{FR} = Freeway to off-ramp flow (i.e., off-ramp volume)

v_{RF} = On-ramp to freeway flow (i.e., on-ramp volume)

v_{FF} = Freeway through flow

v_{RR}= on-ramp to off-ramp flow (optional, i.e., ramp-to-ramp flow)

Exhibit 10A-2 Weaving & non-weaving flows



The full HCM procedure uses passenger cars per hour (pc/h) in the computation of flow rates, but since VR is dimensionless any consistent units can be used such as peak hour volumes (PHV), average daily traffic (ADT) or average annual daily traffic (AADT). The quickest way for obtaining volumes for computing VR are from the daily volumes contained in the interchange ramp volume diagrams available at: https://www.oregon.gov/odot/Data/Pages/Traffic-Counting.aspx.

The examples show an application of the methodology using AADT ramp data for single and multiple interchange pairs. If peak hour volumes are available, then they should be used instead of AADT for a single interchange pair as volume patterns can vary significantly from average or annual average daily conditions. Since the multiple interchange procedure is an extension of the single pair process (and may not exactly match geometric or operational conditions because of the simplifications needed for nested and overlapping sections), AADT should be the only volumes used.

Where possible, predictions of which section should operate as an auxiliary lane or not, should be verified against field observations of existing conditions as a reality check.

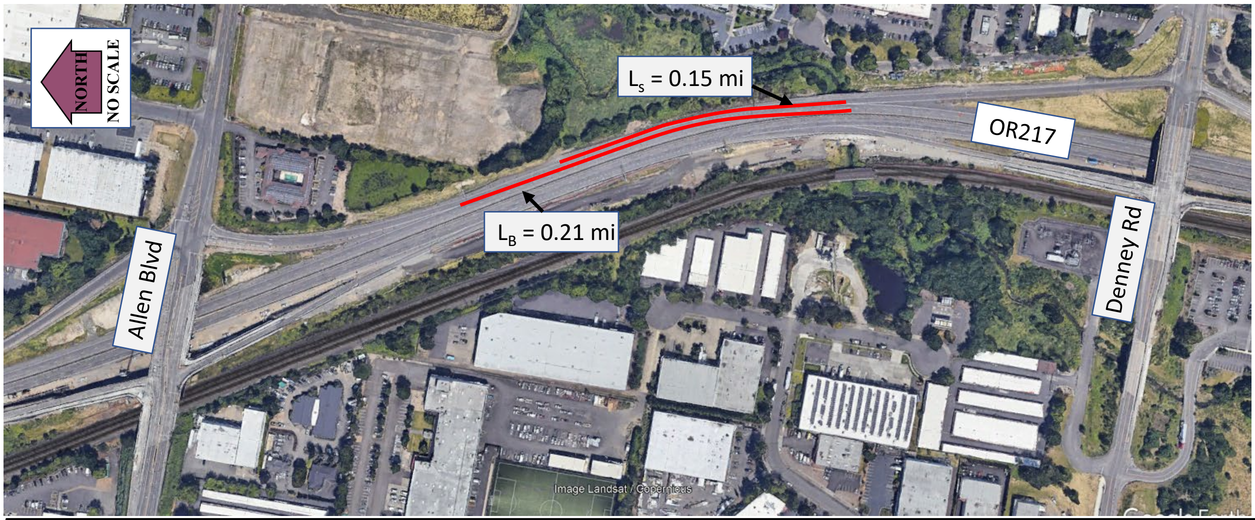
Caution should be taken when this procedure is applying existing available traffic volumes with future geometric improvements, as volume patterns may be different. More detailed analysis may be warranted by large projects with circumstances beyond the ability of this screening methodology, including auxiliary lanes on bridges, at the edge of an urban growth boundary, and in complex multi-lane or nested auxiliary lanes sections.

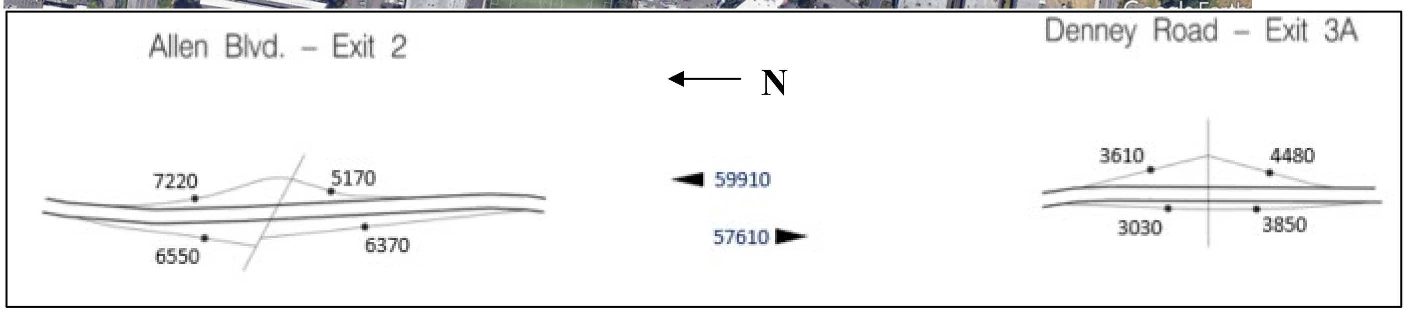
Exhibit 10A-3: Maximum (Operational) Weaving Length (mi)[1]

VR	Single lane on/off ramps	Dual lane on and/or off ramp[2]
0.1	0.7	0.4
0.2	0.9	0.6
0.3	1.1	0.8
0.4	1.3	0.9
0.5	1.5	1.2
0.6	n/a	1.4
0.7	n/a	1.6

[1]Maximum weaving length calculations based on HCM 7th Edition Equation 13-4.
[2]The dual on/off ramps are full lanes and not widened for ramp metering.

Paired Interchange Example: OR217 NB between Denney Road and Allen Boulevard



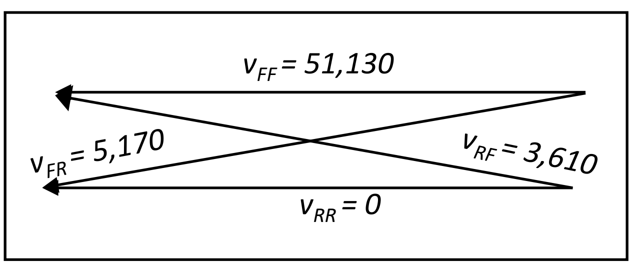


Volumes: 2019 AADT from ramp interchange diagrams (shown above in figure)

Ramp types: single lane on & off ramps

Weaving section base (L_B) and short length (L_S): L_B = 0.21 mi and L_S = 0.15 mi measured from an aerial image (i.e., Google Earth, Bing, etc.) with noted solid stripe at off-ramp so L_S<L_B.

OR217 NB daily volume between ramps from volume diagram = 59, 910 vpd



Denney Road on-ramp volume (v_{RF}) = 3,610 vpd

Allen Boulevard off-ramp volume (v_{FR}) = 5,170 vpd

v_{FF} = 59,910 – 3,610 – 5,170 = 51,130 vpd

[Note: The volume between the ramps, when available, is the denominator in the VR equation. Not all the applicable weaving sections have the daily volumes between the ramps shown in the diagrams, so additional adjustments may be necessary. This calculation is also consistent with the HCM in reminding the analyst to work in flow paths rather than discrete points.]

VR = (3,610 + 5,170) / (51,130 + 5,170 + 3,610) = 0.15

Interpolating VR in Exhibit 10A-3 results in a maximum operational weaving length of 0.8 mile.

The weaving lane short length is 0.15 mile, which is shorter than the maximum 0.8 mile, so all the weaving lane length will be used for weaving movements.

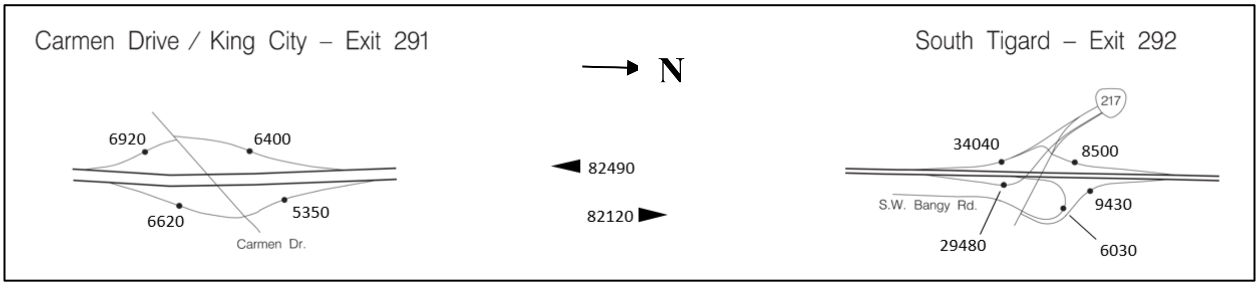
An extension of the methodology is used for weaving lanes that extend across multiple interchange pairs. By inspection, any weaving lane that is longer than 1.6 miles (longest section possible in Exhibit 10A-3), will not operate completely as an auxiliary lane but will have some aspect of a general through lane. Weaving lanes that cross multiple interchanges but are less than 1.6 miles may work completely as an auxiliary lane depending on the VR calculation. The VR calculation is worked from the beginning and the ending of the weaving lane section to determine how much of the full segment could operate as an auxiliary lane. Theoretically, this could allow up to 1.6 miles from each end (3.2 miles total), but the more realistic maximum range would be 2.0 to 2.5 miles given average volume ratios. This longer weaving lane length would still have a section in the middle that would operate more like a general through lane.

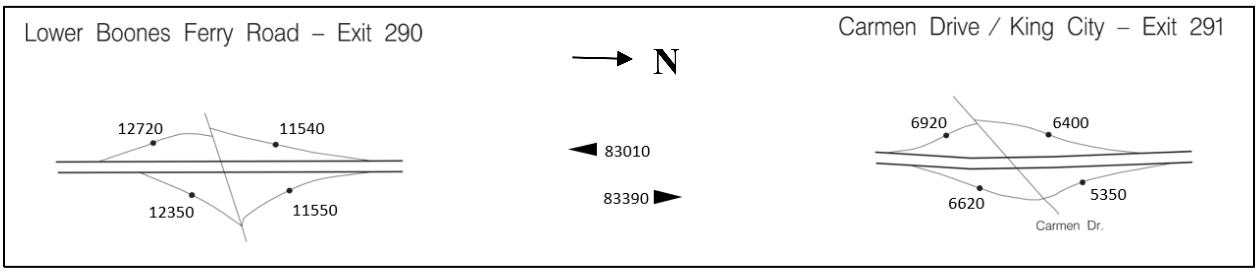
Nested weaving lanes would need to be analyzed on a lane-by-lane basis according to their beginning and end points. Typically, these occur with one weaving lane covering more than one interchange pair and a second weaving lane connecting two adjacent interchanges together. In this case, the longer lane would use the multiple interchange technique and the shorter one would use the standard paired interchange method.

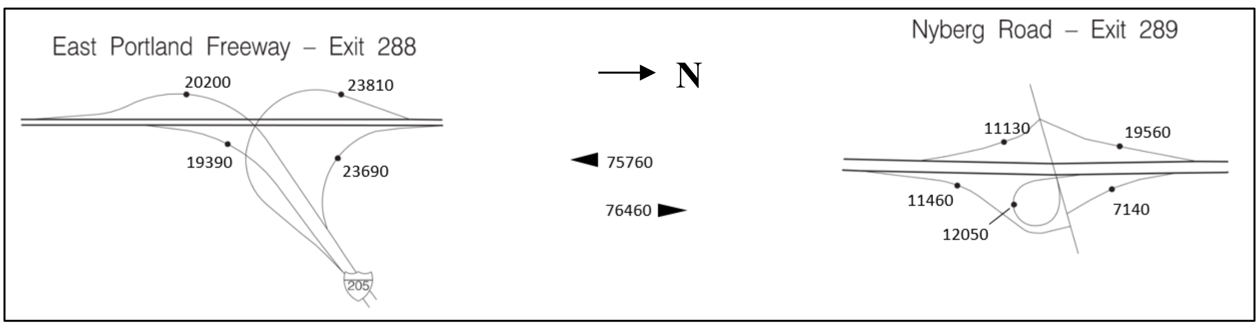
Like mentioned previously, sequential on or off-ramp volumes should be summed together as it is assumed that both would have similar weaving patterns. Note that these would create overlapping weaving areas but keeping with the higher-level theme of this screening methodology, this is simplified operationally and geometrically. These areas would eventually require breaking into separate merge/diverge/basic/weave segments in a full operational facility analysis to determine how much would operate as weaving versus a through lane.

Multiple Interchange Example: Southbound I5 from OR217 to I205

Volumes – 2019 AADT from ramp interchange volume diagrams as shown:







From the Road Assets and Mileage ODOT internet webpage (https://www.oregon.gov/odot/Data/pages/road-assets-mileage.aspx) obtain the milepoints of the necessary on & off-ramps that define the weaving lane segment(s). Volumes from the diagrams and number of lanes are noted from aerial imagery and added to the table below.

Ramp	Milepoint	Volume (vpd)	# Ramp Lanes
Exit 292 (OR217) On	291.86	34,040	2[1]
Exit 291 (Carmen Dr) Off	291.50	6,400	2[1]
Exit 291 On	291.02	6,920	1
Exit 290 (Lower Boones Ferry Rd) Off	290.70	11,540	1
Exit 290 On	290.25	12,720	1[1]
Exit 289 (Nyberg Rd) Off	289.75	19,560	2
Exit 289 On	289.27	11,130	1[1]
Exit 288 (I205) Off	288.97	23,810	2[1]

[1]Nested auxiliary lane on outside lane

The entire multi-interchange weaving lane goes from the Exit 292 on-ramp south to the Exit 288 off-ramp for a total gore-to-gore distance of 2.89 miles which is longer than the 1.6-mile maximum distance. This means that this section will not completely operate as an auxiliary lane but will also have some general through lane operational aspects. Portions of the overall section will operate as an auxiliary lane, however, so this screening analysis would determine what sections would operate as that. In addition, there are several nested weaving lanes between interchange pairs (e.g., Exits 292 & 291, 290 & 289, and 289 & 288) that would need evaluation using the previous paired interchange method.

For measuring purposes, the overall weaving distance does not include the lane drop section south of the Exit 288 off-ramp. This lane segment is an auxiliary lane put in for safety reasons to allow for extra distance for weaving traffic from the on-ramp to merge into the mainline without becoming "trapped" and either doing a forced movement to the off-ramp or doing unexpected lane changes/maneuvers or braking. This type of auxiliary lane falls into the "other" category described in the last section of this appendix.

Since the whole multi-interchange weaving section does not work as an auxiliary lane, a first trial is done to determine the furthest working extent southbound from Exit 292. This would be to the next most significant interchange (based on visual inspection of crossroad width and ramp terminal intersection size and on/off volumes) which would be Exit 290, Lower Boones Ferry Road.

Exit 292 on-ramp volume = 34,040 vpd

Exit 290 off-ramp volume = 11,540 vpd

Average I5 volume of the Exit 292-291 and Exit 291-290 mainline sections between the Exit 292 on-ramp and the Exit 290 off-ramp = (82,490 vpd+83,010 vpd)/2 = 82,750 vpd

Freeway through flow = volume between ramps – on-ramp volume – off-ramp volume = 82,750 – 34,040 – 11,540 = 37,170 vpd

VR = (34,040+11,540) / (37,170 + 34,040+ 11,540) = 0.55

Interpolating this with Exhibit 10A-3 comes up with a maximum length of 1.3 miles given that one of the ramps has dual lanes. The distance between the ramp points is MP 291.86 – 290.70 = 1.16 miles which is less than the maximum 1.3 miles. This means that the section from Exit 292 to 290 will operate as an auxiliary weaving lane, but since the remaining distance (0.14 miles) is less than the distance to the next interchange, the section beyond the Exit 290 off-ramp will have through lane operational characteristics.

In addition, the nested weaving lane between Exit 292 and 291 should be evaluated before moving onto the next section. The nested lane evaluation uses the paired interchange methodology.

Exit 292 on-ramp volume = 34,040 vpd

Exit 291 off-ramp volume = 6,400 vpd

I5 volume between the ramps = 82,490 vpd

Freeway through flow = volume between ramps – on-ramp volume – off-ramp volume = 82,490 – 34,040 – 6,400 = 42,050 vpd

VR = (34,040 + 6,400) / (42,450 + 34,040 + 6,400) = 0.49

Interpolating this with Exhibit 10A-3 comes up with a maximum length of 1.2 miles. This is longer than the 0.36 miles between the interchanges, so the nested lane will completely function as an auxiliary lane.

Second, the upstream portion from the endpoint also needs to be checked as that may also work as an auxiliary weaving lane section. This would be the section between Exit 289 (Nyberg Road) and Exit 288 (I205) to start.

Exit 289 on-ramp volume = 11,130 vpd

Exit 288 off-ramp volume = 23,810 vpd

I5 volume between the ramps = 75,760 vpd

Freeway through flow = volume between ramps – on-ramp volume – off-ramp volume = 75,760 – 11,130 – 23,810 = 40,820 vpd

VR = (11,130 + 23,810) / (40,820 + 11,130 + 23,810) = 0.46

Interpolating this with Exhibit 10A-3 comes up with a maximum length of 1.08 miles which is longer than the 0.36 miles between the ramps. This means the section between the Exit 289 on-ramp and the Exit 288 off-ramp would also function as a weaving lane for its entire length. There is also an additional nested weaving lane between the Exit 289 off-ramp and the Exit 288 on-ramp. The same volumes and VR apply, so that additional weaving lane would also function as an auxiliary lane for its entire length.

Third, the calculation should be run to see if the section upstream from the Exit 288 off-ramp can reach the Exit 290 on-ramp:

Exit 290 on-ramp volume = 12,720 vpd

Exit 288 off-ramp volume = 23,810 vpd

Average I5 volume between the ramps = 75,760 vpd [Exit 288-289] + 84,190 [Exit 289-290]/2 = 79,975 vpd

Freeway though volume = volume between ramps – on-ramp volume – off-ramp volume = 79,975 – 12,720 - 23,810 = 43,445 vpd

VR = (12,720 + 23,810) / (43,445 + 12,720 + 23,810) = 0.46

Interpolating this with Exhibit 10A-3 comes up with the same maximum length of 1.08 miles but is shorter than the 1.28 miles between the ramps, so this means that the section beyond Exit 289 will not function completely as an auxiliary lane. Lastly, the remaining nested weaving lane between Exit 290 and 289 should be evaluated:

Exit 290 on-ramp volume = 12,720 vpd

Exit 289 off-ramp volume = 19,560 vpd

I5 volume between the ramps = 84,190 vpd

Freeway through volume = volume between ramps – on-ramp volume – off-ramp volume = 84,190 – 12,720 – 19,560 = 51,910 vpd

VR = (12,720 + 19,560) / (71,470 + 12,720 + 19,560) = 0.38

Interpolating this with Exhibit 10A-3 comes up with a maximum length of 0.88 miles for two-lane on and off-ramps. This is longer than the 0.50 miles between the interchanges, so the nested lane will completely function as an auxiliary lane.

In summary, the 1.15-mile section from the Exit 292 off-ramp to the Exit 290 off-ramp, and the 0.30-mile section from the Exit 289 on-ramp to the Exit 288 off-ramp would function as a weaving freeway auxiliary lane. The remaining 1.45-mile section between the Exit 290 off-ramp to the Exit 289 on-ramp is predicted to operate more like a general-purpose lane. The nested weaving lanes between Exit 292 & 291, Exit 290 & 289, and Exit 289 & 288 also are predicted operate fully as auxiliary lanes. An operational analysis using peak hour volumes and full facility segmentation would still need to be performed to determine final values.

Passing Lanes

Passing lanes are used on rural two-lane highways to break up platoons of following vehicles which over time and distance are limited by the speed of the lead vehicle and end up reducing the effective capacity of the roadway. Passing lanes on significantly long or steep grades are essentially climbing lanes which are covered in the next section. Passing lane sections are normally located in sections with no intersections and ideally no driveways to prevent vehicles from slowing or stopping in the passing lane which would be outside typical driver expectation. Passing lanes improve the overall directional performance of the highway by increasing average speeds and decreasing travel time. Passing lanes also improve safety as they provide protected space to pass so less vehicles attempt passing in the opposing travel lane which has a much higher crash risk.

The capacity of a passing lane is less than a regular travel lane because of the tendency to bunch vehicles up at the end as they merge back together into a single lane. Within the passing lane section, more vehicles can be accommodated, but these do not add system capacity to the roadway as there is only a single lane entering and leaving. The lack of increasing system capacity and the limited operational effects of a single passing lane section are the primary reasons that these are needed to be spaced periodically along a roadway. Outside of the passing lane's influence area, vehicles will eventually drop into the following mode in a platoon and overall speeds will decrease with a corresponding increase in travel time.

Exhibit 15-A2 in the HCM gives a table of optimum passing lane lengths that maximizes the impact on reducing the number of percent following vehicles. Beyond the optimum point percent following vehicles still reduce but trends flatten out especially at lower flow rates (HCM Exhibit 15-A1). Also, lengths beyond this point would also tend to attract cruising vehicles in the passing lane.

Exhibit 10A-4 shows the optimum passing lane length. Lengths longer than the indicated value for the corresponding directional hourly or daily vehicles will likely operate more like a through lane. If directional volumes are higher than 800 vph in a single direction, then this is approaching or exceeding the maximum volume of a two-lane section. Either the directional hourly or daily (ADT or AADT) volume can be used depending on what is available to determine the optimum passing lane length.

Exhibit 10A-4: Optimum Passing Lane Lengths

Directional hourly volume (vph)	Directional daily volume (vpd)[1,2]	Optimum passing lane length (mi)
200	1,350	0.9
300	2,000	1.0
400	2,650	1.2
500	3,350	1.2
600	4,000	1.6
700	4,650	1.9
800	5,350	2.0

[1]Daily volumes are based on the typical 15% K-factor (daily volume factor that shows the percentage of the peak hour in the whole day) used for rural conditions. Alternately, K30 factors from specific automatic traffic recorders (ATR) can be used to estimate the daily volume from the hourly volume.

[2]Use a directional factor default of 50% for converting daily volumes from the Transportation Volume Tables. Specific directional factors can be obtained from ATRs or from available traffic counts if a closer representation of the subject section is desired.

Climbing Lanes

Climbing lanes are like passing lanes as they improve the overall operation of the roadway and do not add to the system capacity as the same number of lanes enter and leave the section. However, the presence of and overall distance of a climbing lane is determined by the resulting truck speeds. Steep and/or long grades affect truck speeds significantly to the point that eventually speeds can be reduced to a basic crawl speed which affects the effective carrying capacity of the roadway. In most cases, these are in rural areas of rolling or mountainous terrain, but they can exist in urban areas (e.g., northbound I5 at Barber Boulevard or US26 west of the Vista Ridge tunnel).

Like passing lanes, climbing lanes break up platoons of following vehicles and improve the overall speed and travel times along a roadway. These do not add to the system capacity as vehicles need to merge back together just past the top of the grade where truck speeds increase to within 10 mph of the posted speed to minimize speed differentials and related safety impacts. Unlike passing lanes, the length of a climbing lane is defined by the overall length of the grade or grades where the truck speed drops below 10 mph from the posted speed (or 85th percentile operating speed) to a point just past the top of a grade (or series of grades) to where truck speed increases to within 10 mph of other traffic.

Given an approximate 60 mph truck speed for an interstate-type semi-trailer, this means that a grade of at least 5% for about 5,000 feet is needed to drop the speed 10 mph which would be the likely trigger for a climbing lane. Exhibit 10A-5 shows the minimum length of a climbing lane for the given percent grades. The climbing lane will generally be longer than this as trucks would eventually slow to their crawl speed until the grade lessened, the summit reached, and there was enough length to accelerate trucks to within 10 mph of the mainline speed. <u>While less than a mile is needed to trigger a need for a climbing lane, the actual lane could be miles long</u>. If a climbing lane is proposed for a roadway section that does not meet the minimum distances in Exhibit 10A-5, then this would be considered a passing lane instead and Exhibit 10A-4 should be used to determine whether the section will wholly operate as a passing lane or be mixed with through lane characteristics. Climbing lanes will require full HCM Chapter 15 analysis to determine resulting truck speeds, average grades, full length requirements, and overall operations as grades are generally made up of many different slopes.

Exhibit 10A-5: Minimum Climbing Lane Length[1]

Grade (%)	Minimum Length of Grade (ft)
5	5000
6	2000
7	1500

[1]Distance necessary to drop the typical truck (e.g., WB-67) speed 10 mph below the posted speed for the given grade percentage.

Turn Lanes

Turn lane length is determined by the combination of the 95th percentile storage queue, the deceleration distance, and the taper. The deceleration and taper distances are fixed based on design speed as they are designed to allow a vehicle to slow at a reasonable braking rate and be able to stop at the end of the stopped queue. Therefore, this distance would not be considered a capacity-adding length. The largest effort will be to estimate the 95th percentile queue for the design year which is the design queue. Queue estimation ranges from a simple rule of thumb and equation application to full deterministic and micro-simulation (for locations that approach or are over capacity) and are included in the following sections as applicable.

Rural turn lanes

Unsignalized rural turn lanes are mainly built for increasing safety to limit the potential for high speed rear-end collisions in the mainline lanes with waiting left-turning vehicles or slowing right-turning vehicles. While capacity could be improved at a spot location, safety is the primary concern to address in high-speed rural environments when turn lanes are constructed where turning, slowing, or stopped vehicles are generally not expected. Rural turn lanes should not be considered system or local capacity improvements. Storage length can be estimated by using the "major left turn" equation in APM Section 12.5. If turning volumes exceed 300 vph or the conflicting volume exceeds 2,000 vph then micro-simulation is required. In addition, mainline turn lanes should meet the operational turn lane criteria in APM Section 12.2.

Urban unsignalized turn lanes

Urban unsignalized turn lanes are generally constructed to meet vehicular safety and capacity needs. High-speed locations (45 mph +) such as urban expressways where unsignalized turn lanes are constructed are there for safety reasons like the rural conditions noted above. The placement of these also should not be considered a capacity improvement.

Turn lanes on lower speed roadways (<45 mph) should be checked. Storage lengths in general are based on 95th percentile queues which should cover the need at each location. Designing to the maximum queue is unnecessary. Turn lane length is a function of the storage length plus a deceleration/taper length so these will be location specific. Highway Design Manual Section 506 has basic lengths for these depending on speed of the mainline. Right turn lanes will typically have a length of 250 – 450' assuming the minimum 50' storage and design speeds from 25 to 60 mph. Right turn lanes should only be placed (and generally need to be only considered) when right turn volumes are high at significant streets and driveways. This is because they have safety considerations for pedestrians and bicyclists (See HDM Section 506.11 & APM Chapter 10 for more information) and may also impact mode choice. In addition, mainline turn lanes should meet the operational turn lane criteria in APM Section 12.2.

It is also possible to have a continuous right turn lane that serves multiple closely spaced driveways or streets, but these should terminate at or before an intersection. If a continuous right turn lane is longer than a block it will likely not serve only turning uses. It could potentially carry through vehicles that would turn at the next intersection but would have normally been in a full through lane initially.

Left turn lanes will typically have a length of 225 – 500’ assuming a minimum 100’ storage and design speeds from 25 to 60 mph. The left turn storage length needs to be estimated to verify if the minimum design length is adequate using the procedure in APM Section 12.5. This procedure uses the turning volume and conflicting flow volume. If turning volumes exceed 300 vph or the conflicting volume exceeds 2,000 vph then micro-simulation is required.

Stopped minor street turn lanes

These are mainly for reducing delay and related queues/storage distance needs from waiting left-turning vehicles. While a separate turn lane will also increase the capacity of the minor approach (assuming that there is a significant right turn volume demand), it is likely that the operations of the left-turn lane still will control the actual reported v/c ratio. Stopped minor street left turn movements have the least amount of capacity available to them as they need to yield to all other conflicting movements, so any capacity increase is likely to be small. Typically turn lanes (outside of taper lengths) could be as short as 50’ for a right turn lane to around 300’ for a higher volume left turn lane.

Storage length can be estimated for minor street left and right turn lanes for several common configurations using the procedure in APM Section 12.5 which is a function of minor approach volume and conflicting major approach volume. Deterministic analysis (Highway Capacity Manual methods found in Sidra, Vistro, Synchro, etc.) will be required if the turn lane configurations are not covered by the APM Section 12.5 equations. If left-turning volume exceeds 300 vph or related conflicting flow exceeds 2,000 vph or if right-turning volume exceeds 250 vph and related conflicting flows are 1,500 vph then micro-simulation will be required. Additionally, if local conditions are overcapacity (e.g., stopped mainline traffic frequently blocks the subject approach) then micro-simulation will be required to determine the required length.

Two-way left-turn lanes (TWLTL)

These are a type of center median which are installed where there is a high number of turns to/from streets or private driveways. These provide a safe waiting space to turn and allow a space to turn into for a two-stage left turn from a driveway when mainline volumes are high. These are more common in urban and suburban areas where access density is higher, but speeds are lower. These are common in projects that convert former county roadways to an urban cross-section. These are primarily installed for access and vehicular safety reasons and should not be considered an auxiliary lane (as vehicles need to physically cross over a double yellow median stripe) or a feature that adds capacity to a roadway. While there are analysis techniques to analyze the difference between single and two-stage left turns from driveways in available software tools (e.g., Synchro), there are no methods available to analyze the actual operations of a TWLTL.

Signalized turn lanes

These are mainly built to maximize the capacity of the street approaches and improve overall intersection efficiency. Individual lane volumes and storage needs/network limitations (i.e., spacing back to next intersection) will control the actual signal phasing requirements and overall lane configurations.

APM Section 10.12.9 has some additional guidance about adding turn lanes at intersections. Turn lanes at signalized intersections with over 300’ of storage should be used with caution to avoid adverse impacts to bicyclists (APM Section 14.4.5), especially when left and right turn lanes are present. Signalized turn lanes are typically in the 100- 250’ storage range and instances over 300’ are rare but may exist in special cases where high turning volumes exist but available right-of-way prevents adding dual lanes.

Dual turn lanes will add capacity over a single lane and usually have full receiving lanes, however, in some cases the crossroad is not widened to seamlessly accommodate the turn lanes. In these cases, a receiving/drop lane is installed to allow traffic to merge back efficiently into the mainline through lane which should not be considered a capacity expansion as it would be too short to have any space that would be used as a through lane. Receiving lanes that continue for a considerable distance (i.e., over ¼ mile in a rural area, or over a city block) and eventually drop would allow some through traffic operation and would still be counted as through lanes from the originating intersection to the first lane ends sign. The overall area context must be a consideration as high volumes and/or speeds may require longer receiving lane distances than this general guidance.

APM Sections 13.5.2 & 13.5.3 have signalized queuing rule of thumb methods for left and right turn lanes. New or additional turn lanes need to be evaluated against the impact to pedestrian exposure by increasing crossing distance (see Chapter 14). For more specific lengths and configurations, this will require intersection analysis, potentially with micro-simulation if area conditions are close to or exceeding capacity.

Auxiliary through lanes

These are limited length through lanes added upstream and dropped downstream of a signalized intersection. Typically, these have been used as a way for an intersection to meet an operational standard under existing conditions with street widening deferred to a later date. This type will add to the capacity with about 15% of the directional traffic generally predicted to be in the auxiliary through lane. Microsimulation is required to determine adequacy and overall length. Overall, this lane type is heavily discouraged and is generally not allowed on state highways. This is because they have numerous concerns with vehicular and pedestrian safety. These concerns include vehicles driving significantly faster using it as a passing lane along with the increased pedestrian exposure. More information is included in APM Section 13.3.

Other Types

Certain speed change lanes such as for on-ramp merge and off-ramp diverge operations, right-turn acceleration lanes and left-turn median acceleration lanes do not fit well into the "auxiliary lane" mold. These require fixed lengths to get traffic from the lower entering speed such as from the end of a loop on-ramp roadway or weigh station or after making a right or left turn maneuver to typically within 10 mph of the mainline posted speed. Speed differentials need to be less than 10 mph to avoid inadvertently increasing the potential crash risk of high-speed rear end collisions. Likewise, deceleration lanes allow room for vehicles to slow down before entering a sharper curve such as on an off-ramp at an interchange or rest area and are also dependent on the desired departure and mainline design speeds. In general, design speed is used for most types; however, median acceleration lanes use posted speeds to define lengths. Because these are placed for safety and operational reasons to minimize speed differentials between the starting and ending points and are controlled by actual physical acceleration needs these are not considered to add capacity. Refer to the Highway Design Manual Sections 506.12 to 506.15 and Section 600 for more information.