Appendix D

Potential Changes Proposed for Consideration in the Next Edition of the *Green Book*

This section provides potential changes proposed for consideration to the next edition of the *Green Book*, based upon the findings and conclusions of this research. The proposals are based upon a review of the 2004 *Green Book*. Proposed modifications are specified for selected sections in Chapters 3 and 10 as follows, using track changes.

Chapter 3 (Beginning on Page 231 of 2004 *Green Book*)

VERTICAL ALIGNMENT

Terrain

The topography of the land traversed has an influence on the alignment of roads and streets. Topography affects horizontal alignment, but has an even more pronounced effect on vertical alignment. To characterize variations in topography, engineers generally separate it into three classifications according to terrain.

In level terrain, highway sight distances, as governed by both horizontal and vertical restrictions, are generally long or can be made to be so without construction difficulty or major expense.

In rolling terrain, natural slopes consistently rise above and fall below the road or street grade, and occasional steep slopes offer some restriction to normal horizontal and vertical roadway alignment.

In mountainous terrain, longitudinal and transverse changes in the elevation of the ground with respect to the road or street are abrupt, and benching and side hill excavation are frequently needed to obtain acceptable horizontal and vertical alignment.

Terrain classifications pertain to the general character of a specific route corridor. Routes in valleys, passes, or mountainous areas that have all the characteristics of roads or streets traversing level or rolling terrain should be classified as level or rolling. In general, rolling terrain generates steeper grades than level terrain, causing trucks to reduce speeds below those of passenger cars; mountainous terrain has even greater effects, causing some trucks to operate at crawl speeds.

Grades

Roads and streets should be designed to encourage uniform operation throughout. As discussed earlier in this chapter, design speeds are used as a means toward this end by correlation of various geometric features of the road or street. Design criteria have been determined for many highway features, but few conclusions have been reached on the appropriate relationship of roadway grades to design speed. Vehicle operating characteristics on grades are discussed and established relationships of grades and their lengths to design speed are developed below.

Vehicle Operating Characteristics on Grades

Passenger cars. The practices of passenger car drivers on grades vary greatly, but it is generally accepted that nearly all passenger cars can readily negotiate grades as steep as 4 ~~percent~~ to 5 percent without an appreciable loss in speed below that normally maintained on

level roadways, except for cars with high weight/power ratios, including some compact and subcompact cars.

Studies show that, under uncongested conditions, operation on a 3 percent upgrade, has only a slight effect on passenger car speeds compared to operations on level terrain. On steeper upgrades, speeds decrease progressively with increases in the grade. On downgrades, passenger car speeds generally are slightly higher than on level sections, but local conditions govern.

Trucks. The effect of grades on truck speeds is much more pronounced than on speeds of passenger cars. The average speed of trucks on level sections of highway approximates the average speed of passenger cars. Trucks generally increase speed by up to about 5 percent on downgrades and decrease speed by 7 percent or more on upgrades as compared to their operation on the level. On upgrades, the maximum speed that can be maintained by a truck is dependent primarily on the length and steepness of the grade and the truck's weight/power ratio, which is the gross vehicle weight divided by the net engine power. Other factors that affect the average truck speed on a grade are the entering speed, the aerodynamic resistance, and skill of the driver. The last two factors cause only minor variations in the average speed.

Extensive studies of truck performance have been conducted to determine the separate and combined effects of roadway grade, tractive effort, and gross vehicle weight (**33, 34, 35, 36, 37, 38, 39)**.

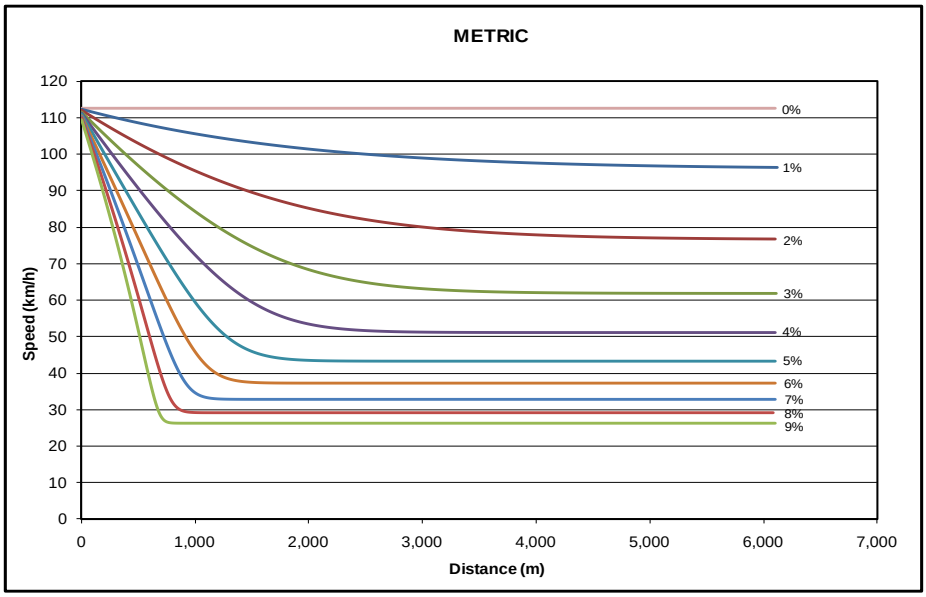
The effect of rate and length of grade on the speed of a typical heavy truck is shown in Exhibits 3-55 and 3-56. From Exhibit 3-55 it can be determined how far a truck, starting its climb from any speed up to approximately 120 km/h [70 mph], travels up various grades or combinations of grades before a certain or uniform speed is reached. For instance, with an entering speed of approximately 110 km/h [70 mph], the truck travels about 950 m [2,700 ft] up a 6 percent grade before its speed is reduced to 60 km/h [35 mph]. If the entering speed is 60 km/h [35 mph], the speed at the end of a 300-m [1,000-ft] climb is about 43 km/h [26 mph]. This is determined by starting on the curve for a 6 percent grade corresponding to 60 km/h [35 mph] for which the distance is 750 m [2,500 ft], and proceeding along it to the point where the distance is 300 m [1,000 ft] more, or 1 050 m [3,500 ft], for which the speed is about 43 km/h [26 mph]. Exhibit 3-56 shows the performance on grade for a truck that approaches the grade at or below crawl speed. The truck is able to accelerate to a speed of 40 km/h [25 mph] or more only on grades of less than 3.5 percent. These data serve as a valuable guide for design in appraising the effect of trucks on traffic operation for a given set of profile conditions.

Travel time (and, therefore, speed) of trucks on grades is directly related to the weight/power ratio. Trucks of the same weight/power ratio typically have similar operating characteristics. Hence, this ratio is of considerable assistance in anticipating the performance of trucks. Normally, the weight/power ratio is expressed in terms of gross weight and net power, in units of kg/kW [wt/hp]; while the metric unit kg is a unit of mass, rather than weight, it is commonly used to represent the weight of object. It has been found that trucks with weight/power ratios of about 120 kg/kW [200 lb/hp] have acceptable operating characteristics from the standpoint of the highway user. Such a weight/power ratio assures a minimum speed of about 60 km/h [35 mph] on a 3 percent upgrade. There is evidence that the automotive industry

would find a weight/power ratio of this magnitude acceptable as a minimum goal in the design of commercial vehicles. There is also evidence that carrier operators are voluntarily recognizing this ratio as the minimum performance control in the loads placed on trucks of different power, the overall result being that weight/power ratio of trucks on highways has improved in recent years. Ratios developed from information obtained in conjunction with the nationwide brake performance studies conducted between 1949 and 1985 show, for example, that for a gross vehicle weight of 18 000 kg [40,000 lb], the average weight/power ratio decreased from about 220 kg/kW [360 lb/hp] in 1949, to about 130 kg/kW [210 lb/hp] in 1975; the weight/power ratio continued to fall to about 80 kg/kW [130 lb/hp] in 1985. This decreased weight/power ratio means greater power and better climbing ability for trucks on upgrades.

There is a trend toward larger and heavier trucks with as many as three trailer units allowed on certain highways in some states. Studies indicate that as the number of axles increases, the weight/power ratio increases. Taking all factors into account, it appears conservative to use a weight/power ratio of 120 kg/kW [200 lb/hp] in determining critical length of grade. However, there are locations where a weight/power ratio as high as 120 kg/kW [200 lb/hp] is not appropriate. Where this occurs, designers are encouraged to utilize either a more representative weight/power ratio or an alternate method that more closely fits the conditions. Exhibits 3-57 through 3-62 provide speed-distance curves for trucks with weight/power ratios in the range from 85 to 110 kg/kW [140 to 180 lb/hp].

Recreational vehicles. Consideration of recreational vehicles on grades is not as critical as consideration of trucks. However, on certain routes such as designated recreational routes, where a low percentage of trucks may not warrant a truck climbing lane, sufficient recreational vehicle traffic may indicate a need for an additional lane. This can be evaluated by using the design charts in Exhibit 3-63~~57~~ in the same manner as for trucks described in the preceding section of this chapter. Recreational vehicles include self-contained motor homes, pickup campers, and towed trailers of numerous sizes. Because the characteristics of recreational vehicles vary so much, it is difficult to establish a single design vehicle. However, one study on the speed of vehicles on grades included recreational vehicles (**40**). The critical vehicle was considered to be a vehicle pulling a travel trailer, and the charts in Exhibit 3-63~~57~~ for a typical recreational vehicle is based on that assumption.



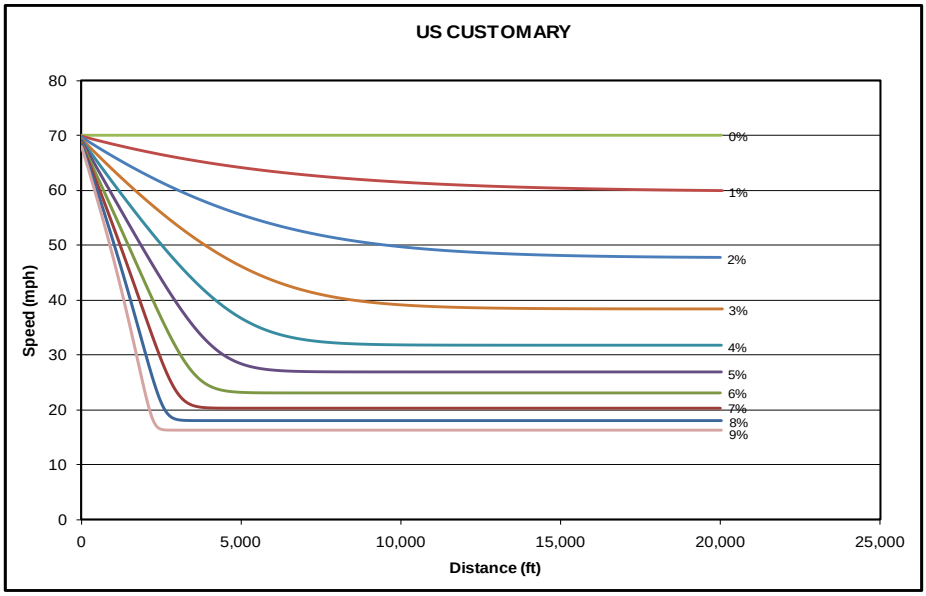
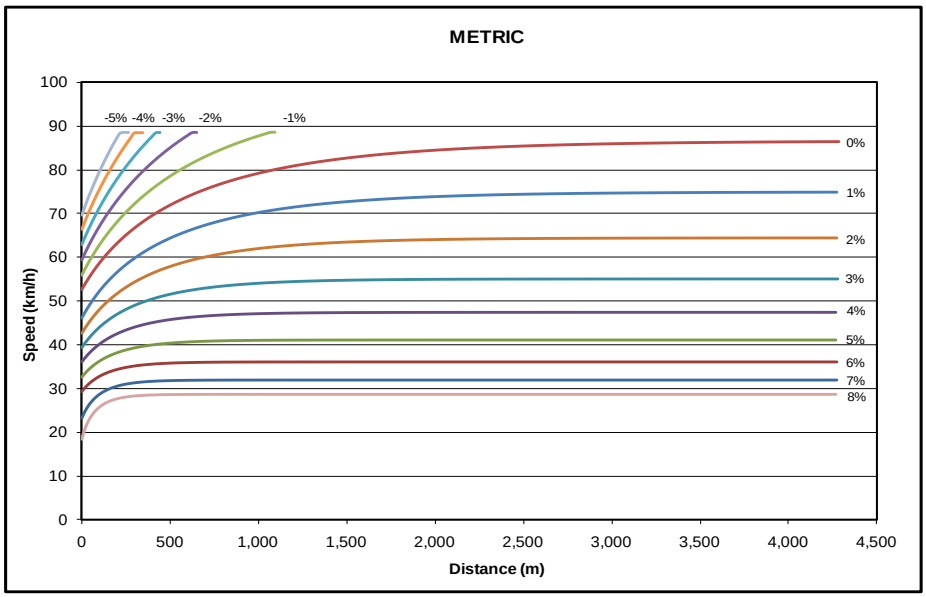


Exhibit 3-55. Speed-Distance Curves for a Typical Heavy Truck of 120 kg/kW [200 lb/hp] for Deceleration on Upgrades

Comment [djt1]: The speed-distance curves in this exhibit are modified making use of the Truck Speed Profile Model (TSPM).



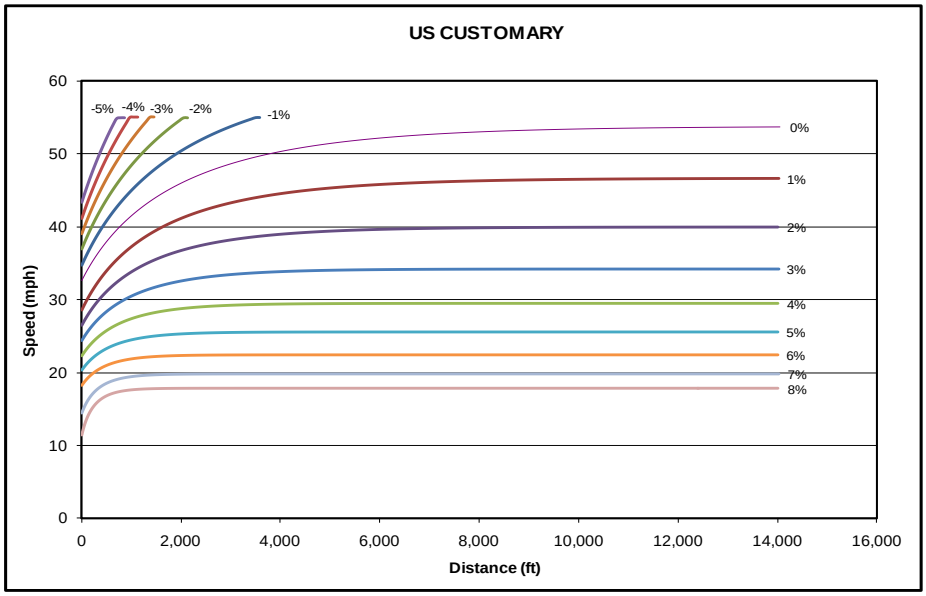
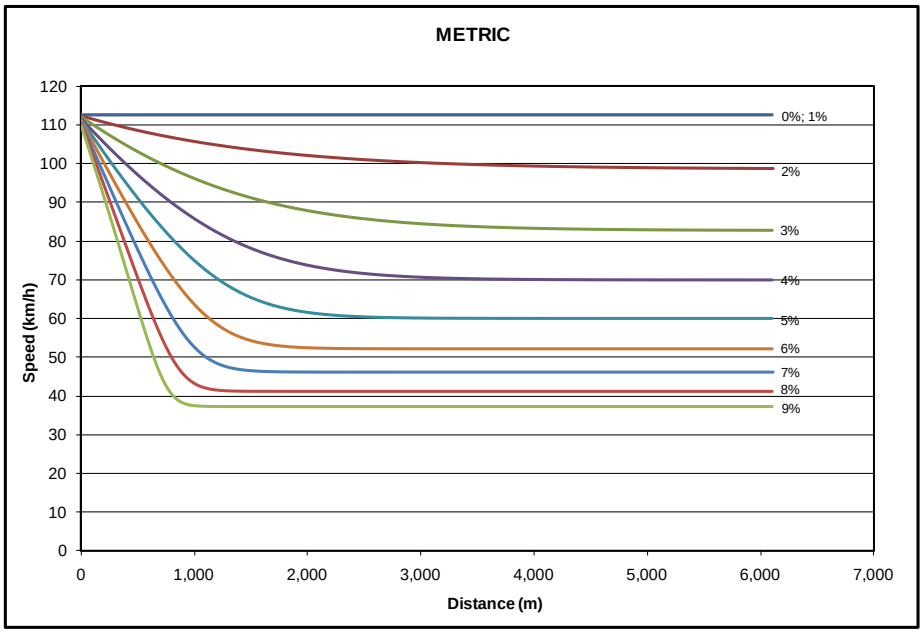


Exhibit 3-56. Speed-Distance Curves for Acceleration of a Typical Heavy Truck of 120 kg/kW [200 lb/hp] on Upgrades and Downgrades

Comment [djt2]: The speed-distance curves in this exhibit are modified making use of the Truck Speed Profile Model (TSPM).



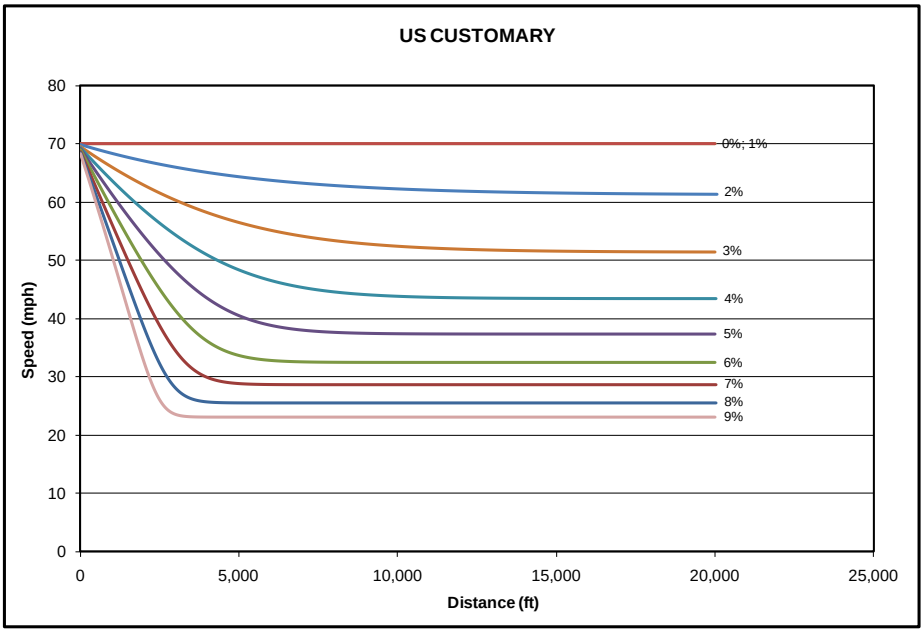
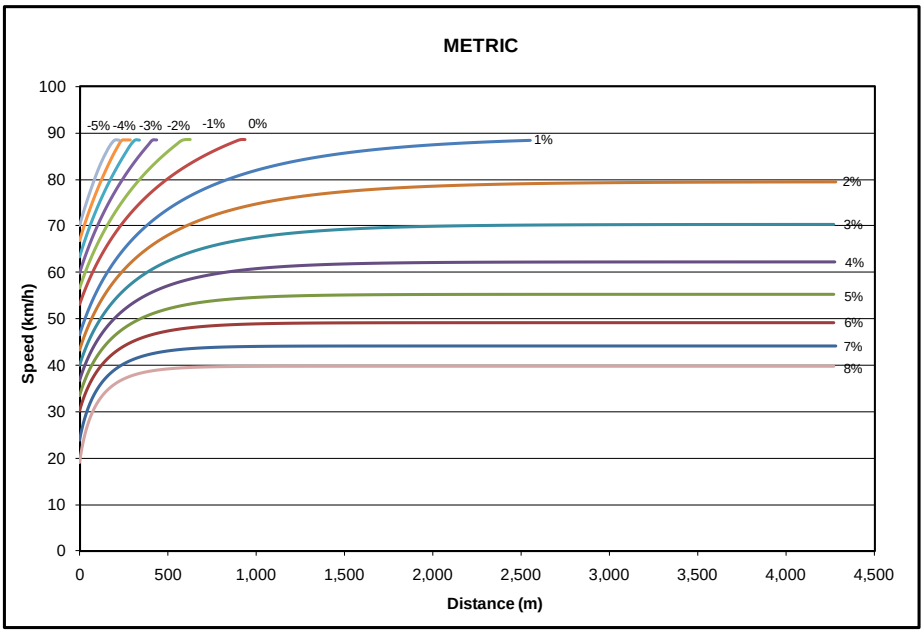


Exhibit 3-57. Speed-Distance Curves for a Typical Heavy Truck of 85 kg/kW [140 lb/hp] for Deceleration on Upgrades

Comment [djt3]: This is a new exhibit.



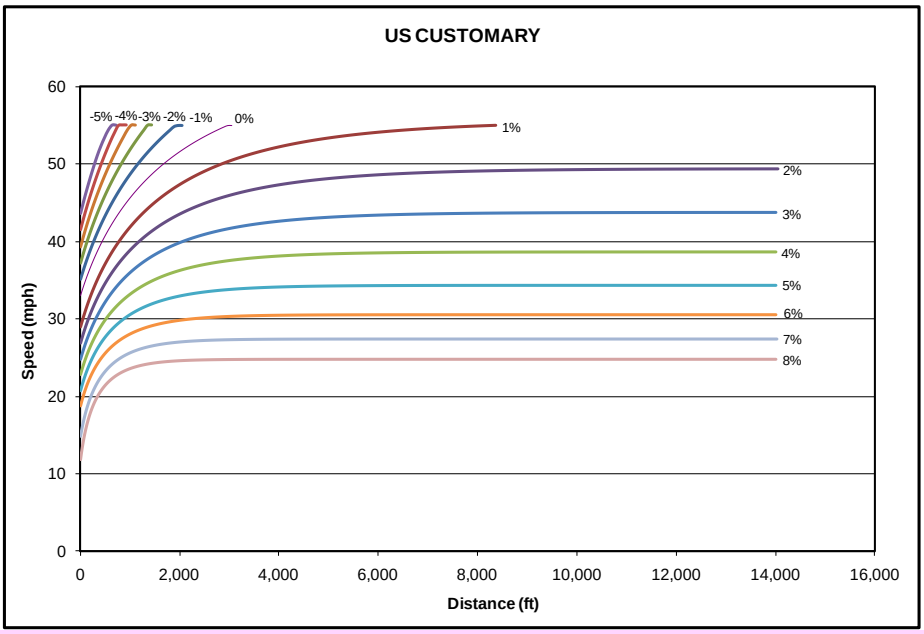
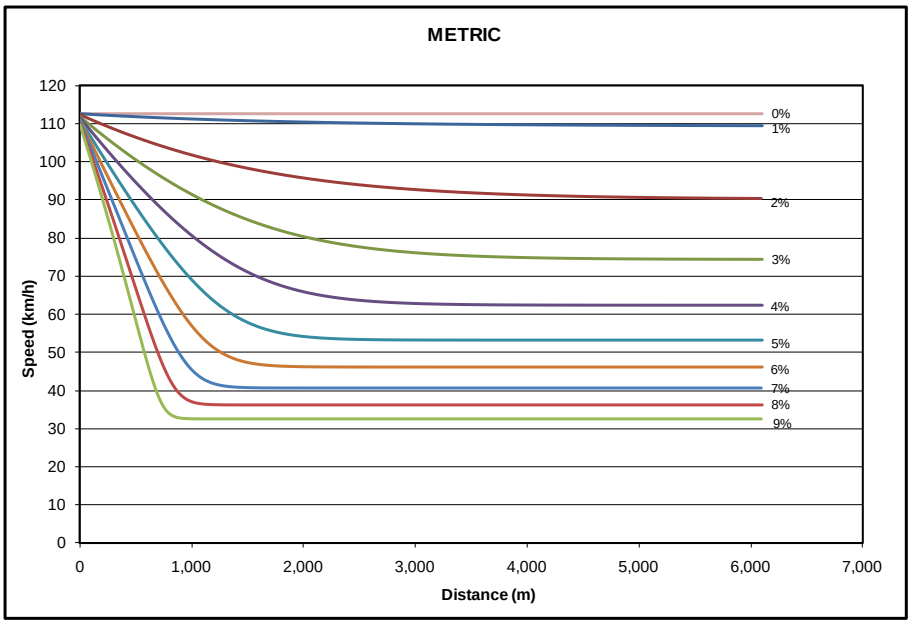


Exhibit 3-58. Speed-Distance Curves for Acceleration of a Heavy Truck of 85 kg/kW [140 lb/hp] on Upgrades and Downgrades

Comment [djt4]: This is a new exhibit.



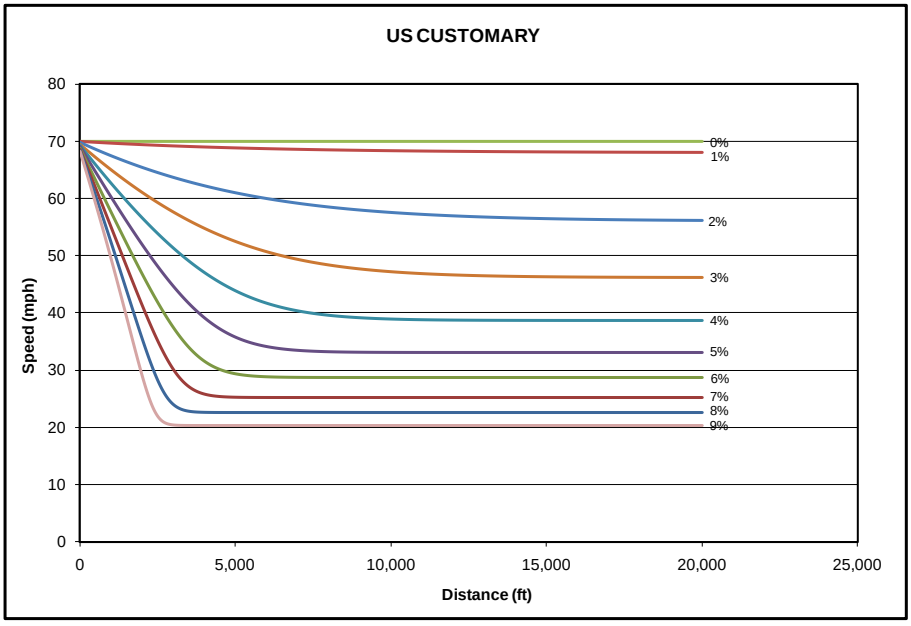
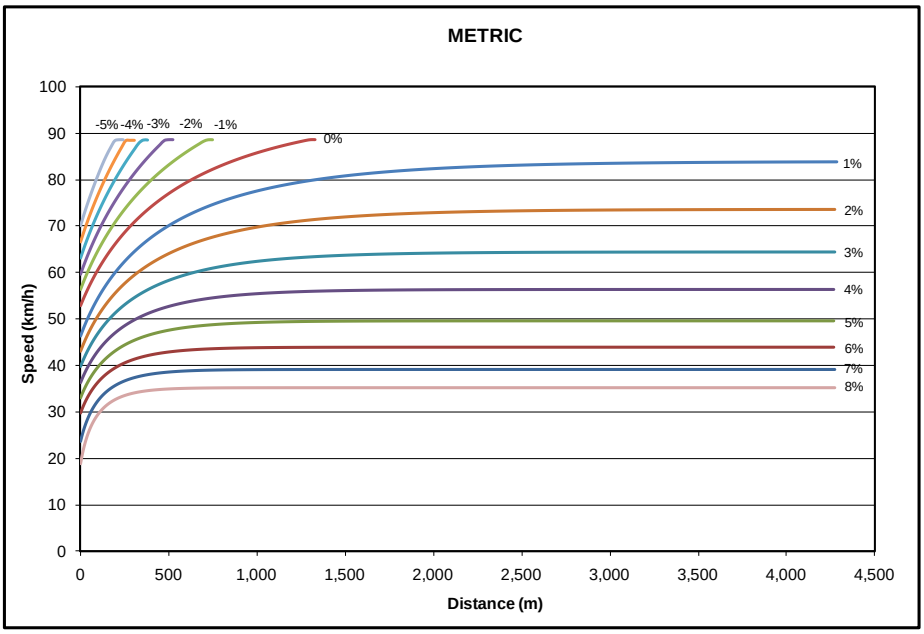


Exhibit 3-59. Speed-Distance Curves for a Typical Heavy Truck of 97 kg/kW [160 lb/hp] for Deceleration on Upgrades

Comment [djt5]: This is a new exhibit.



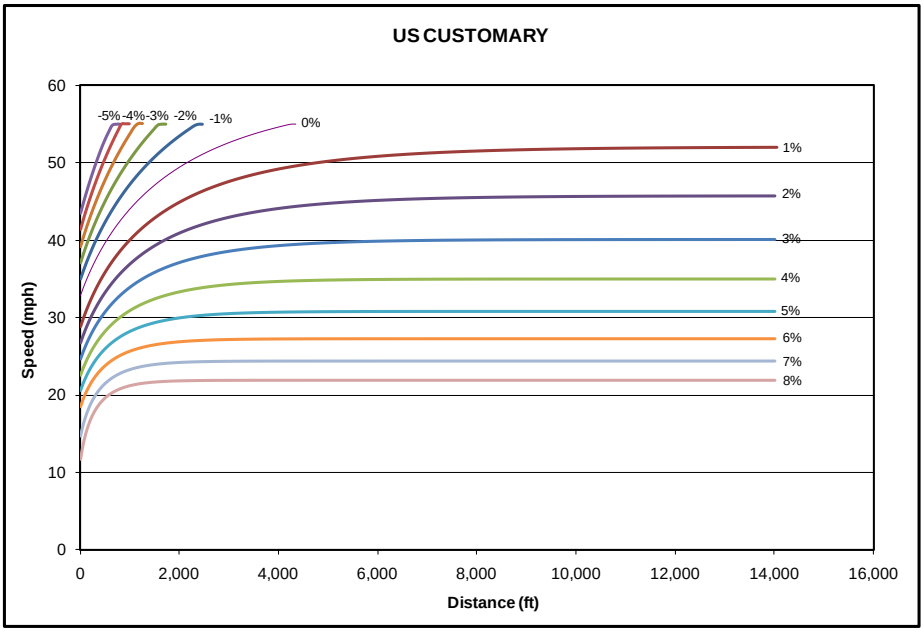
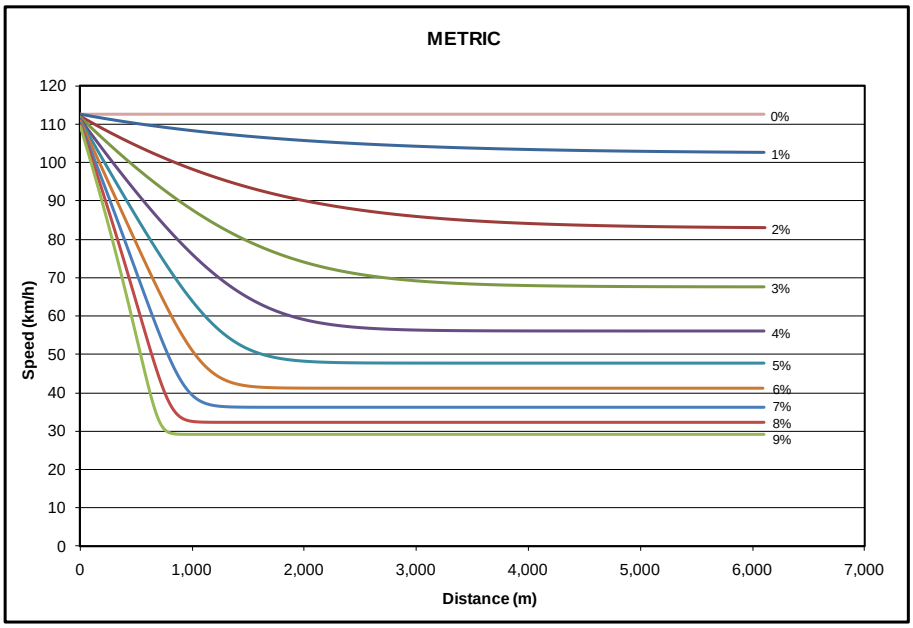


Exhibit 3-60. Speed-Distance Curves for Acceleration of a Heavy Truck of 97 kg/kW [160 lb/hp] on Upgrades and Downgrades

Comment [djt6]: This is a new exhibit.



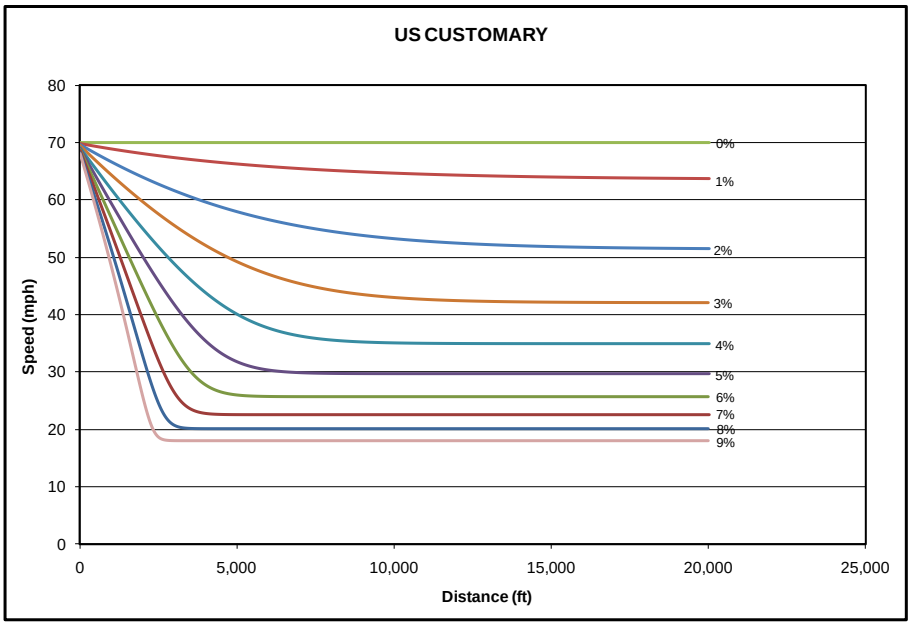
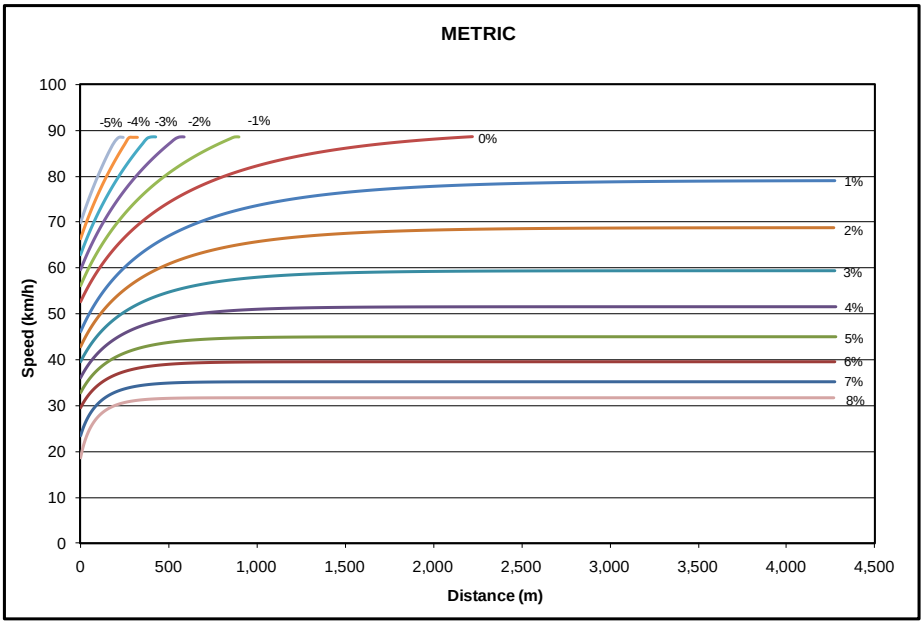


Exhibit 3-61. Speed-Distance Curves for a Typical Heavy Truck of 110 kg/kW [180 lb/hp] for Deceleration on Upgrades

Comment [djt7]: This is a new exhibit.



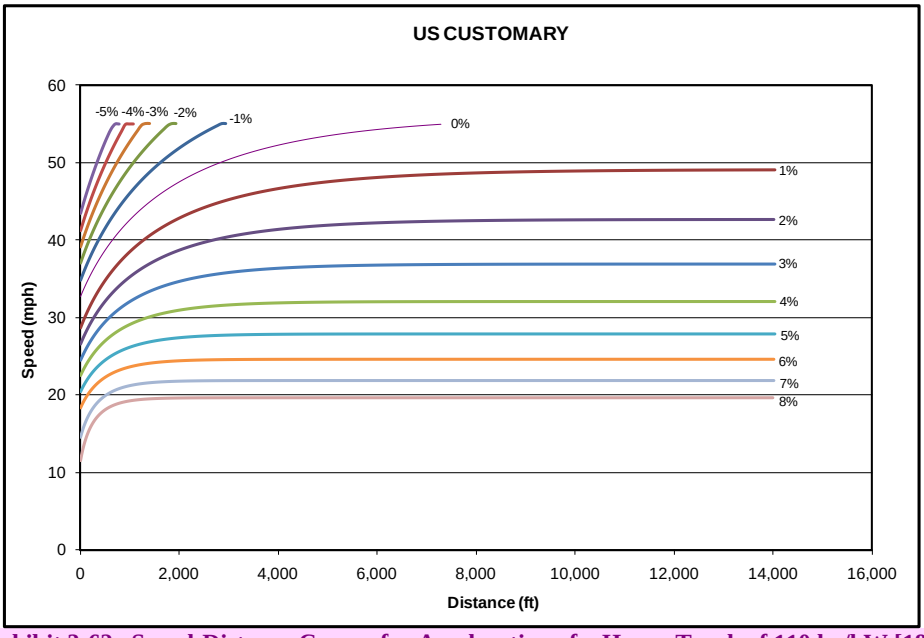


Exhibit 3-62. Speed-Distance Curves for Acceleration of a Heavy Truck of 110 kg/kW [180 lb/hp] on Upgrades and Downgrades

Comment [djt8]: This is a new exhibit.

Control Grades for Design

Maximum grades. On the basis of the data in Exhibits 3-55 through 3-64~~58~~, and according to the grade controls now in use in a large number of States, reasonable guidelines for maximum grades for use in design can be established. Maximum grades of about 5 percent are considered appropriate for a design speed of 110 km/h [70 mph]. For a design speed of 50 km/h [30 mph], maximum grades generally are in the range of 7 to 12 percent, depending on terrain. If only the more important highways are considered, it appears that maximum grades of 7 or 8 percent are representative of current design practice for a 50-km/h [30-mph] design speed. Control grades for design speeds from 60 to 100 km/h [40 to 60 mph] fall between the above extremes. Maximum grade controls for each functional class of highway and street are presented in Chapters 5 through 8.

The maximum design grade should be used only infrequently; in most cases, grades should be less than the maximum design grade. At the other extreme, for short grades less than 150 m [500 ft] in length and for one-way downgrades, the maximum grade may be about 1 percent steeper than other locations; for low-volume rural highways, the maximum grade may be 2 percent steeper.

Minimum grades. Flat grades can typically be used without problem on uncurbed highways where the cross slope is adequate to drain the pavement surface laterally. With curbed highways or streets, longitudinal grades should be provided to facilitate surface drainage. An appropriate minimum grade is typically 0.5 percent, but grades of 0.30 percent may be used where there is a high-type pavement accurately sloped and supported on firm subgrade. Use of even flatter grades may be justified in special cases as discussed in subsequent chapters. Particular attention should be given to the design of storm water inlets and their spacing to keep the spread of water on the traveled way within tolerable limits. Roadside channels and median swales frequently need grades steeper than the roadway profile for adequate drainage. Drainage channels are discussed in Chapter 4.

Chapter 10 (Beginning on Page 844 of 2004 *Green Book*)

Speed-change lanes. Drivers leaving a highway at an interchange are required to reduce speed as they exit on a ramp. Drivers entering a highway from a turning roadway accelerate until the desired highway speed is reached. Because the change in speed is usually substantial, provision should be made for acceleration and deceleration to be accomplished on auxiliary lanes to minimize interference with through traffic and to reduce crash potential. Such an auxiliary lane, including tapered areas, may be referred to as a speed-change lane. The terms “speed-change lane,” “deceleration lane,” or “acceleration lane” as used herein apply broadly to the added lane joining the traveled way of the highway with that of the turning roadway and do not necessarily imply a definite lane of uniform width. This additional lane is a part of the elongated ramp terminal area.

A speed-change lane should have sufficient length to enable a driver to make the appropriate change in speed between the highway and the turning roadway in a safe and comfortable manner.

Moreover, in the case of an acceleration lane, there should be additional length to permit adjustments in speeds of both through and entering vehicles so that the driver of the entering vehicle can position himself opposite a gap in the through-traffic stream and maneuver into it before reaching the end of the acceleration lane. This latter consideration also influences both the configuration and length of an acceleration lane.

Two general forms of speed-change lanes are: (1) the taper and (2) the parallel type. The taper type provides a direct entry or exit at a flat angle, whereas the parallel type has an added lane for changing speed. Either type, when properly designed, will operate satisfactorily. However, the parallel type is still favored in certain areas. Furthermore, some agencies use the taper type for exits and the parallel type for entrances.

See Chapter 9 for discussion of speed-change lanes as applicable to at-grade intersections.

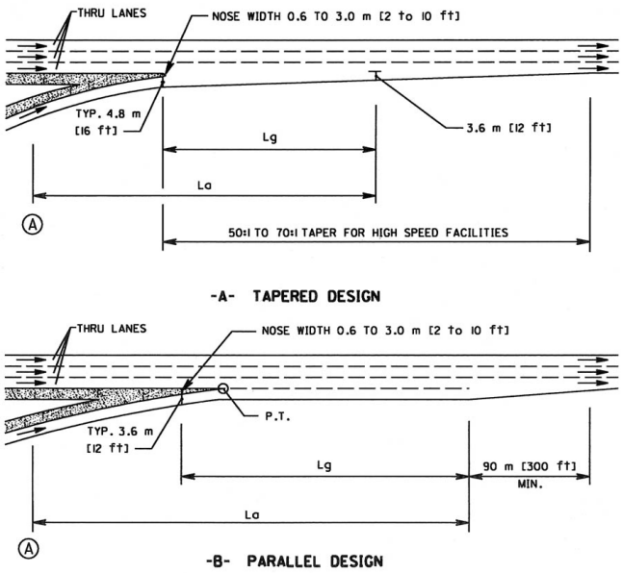
Single-Lane Free-Flow Terminals, Entrances

Taper-type entrance. When properly designed, the taper-type entrance usually operates smoothly at all volumes up to and including the design capacity of merging areas. By relatively minor speed adjustment, the entering driver can see and use an available gap in the through-traffic stream. A typical single-lane, taper-type entrance terminal is shown in Exhibit 10-69A.

The entrance is merged into the freeway with a long, uniform taper. Operational studies show a desirable rate of taper of about 50:1 to 70:1 (longitudinal to lateral) between the outer edge of the acceleration lane and the edge of the through-traffic lane. The gap acceptance length, L_g is also a consideration in the design of taper-type entrances, as illustrated in Exhibit 10-69A.

The geometrics of the ramp proper should be such that motorists may attain a merge speed that is within 10 km/h [5 mph] of the operating speed of the freeway by the time they reach the point where the left edge of the ramp joins the traveled way of the freeway. For consistency of application, this point of convergence of the left edge of the ramp and the right edge of the through lane may be assumed to occur where the right edge of the ramp traveled way is 3.6 m [12 ft] from the right edge of the through lane of the freeway. While it is desirable for motorists to merge onto the freeway at speeds near the operating speed of the freeway, some motorists may choose to enter the freeway at speeds below the operating speed of the freeway without using the full length of the speed-change lane. Taper-type entrances are preferred over parallel-type entrances because motorists tend to merge closer to freeway speeds at taper-type entrances.

The distance needed for acceleration in advance of this point of convergence is governed by the speed differential between the operating speed o~~n~~f the ~~entrance curve~~controlling feature of the ramp and the operating speed of the highway. In the case of a straight ramp, the controlling feature is the crossroad terminal, and in the case of a loop ramp, the controlling feature is the entrance curve to the acceleration lane. At crossroad terminals where many vehicles do not begin accelerating from a stopped position, it is reasonable to assume higher initial speeds than zero when determining minimum acceleration lengths. Exhibit 10-70 shows minimum lengths of acceleration distances for entrance terminals. Exhibit 10-69 shows the minimum lengths for gap acceptance. Referring to Exhibit 10-69, the larger value of the acceleration length (L_a) or the gap acceptance (L_g) length is suggested for use in the design of the ramp entrance.



NOTES:

1. La IS THE REQUIRED ACCELERATION LENGTH AS SHOWN IN EXHIBIT 10-70 OR AS ADJUSTED BY EXHIBIT 10-71.
2. POINT (A) CONTROLS SPEED ON THE RAMP. La SHOULD NOT START BACK ON THE CURVATURE OF THE RAMP UNLESS THE RADIUS EQUALS 300 m [1000 ft] OR MORE.
3. Lg IS REQUIRED GAP ACCEPTANCE LENGTH. Lg SHOULD BE A MINIMUM OF 90 TO 150 m [300 to 500 ft] DEPENDING ON THE NOSE WIDTH.
4. THE VALUE OF La OR Lg, WHICHEVER PRODUCES THE GREATER DISTANCE DOWNSTREAM FROM WHERE THE NOSE EQUAL 0.6 m [2 ft], IS SUGGESTED FOR USE IN THE DESIGN OF THE RAMP ENTRANCE.

Exhibit 10-69. Typical Single-Lane Entrance Ramps

Where the minimum values for nose width (0.6 m [2 ft]), lane width 4.8 m [16 ft]), and taper rate (50:1) are used with high traffic volumes, taper lengths longer than the larger of L_a or L_g may be needed to avoid inferior operation and to reduce fairly sharp moves into the mainline traffic stream. Where grades are present on ramps, speed-change lengths should be adjusted in accordance with Exhibit 10-71.

The design values in Exhibit 10-70 are conservative estimates based upon free-merge conditions (i.e., free-flow conditions) for passenger cars. In situations where free-merge conditions are expected in the foreseeable future and constraints make it difficult to provide the recommended minimum acceleration lengths, the minimum acceleration lane lengths can be reduced by 15 percent without causing expected operational problems. Additionally, if trucks are considered a significant percentage of traffic that they should be the design vehicle, acceleration

lane lengths designed to better accommodate heavier design vehicles can be derived using Exhibits 3-55 through 3-62 in Chapter 3.

Parallel-type entrances. The parallel-type entrance provides an added lane of sufficient length to enable a vehicle to accelerate to near-freeway speed prior to merging. A taper is provided at the end of the added lane. The process of entering the freeway is similar to a lane change to the left. The driver is able to use the side-view and rear-view mirrors to monitor surrounding traffic.

A typical design of a parallel-type entrance is shown in Exhibit 10-69B. Desirably, a curve with a radius of 300 m [1,000 ft] or more and a length of at least 60 m [200 ft] should be provided in advance of the added lane. If this curve has a short radius, motorists tend to drive directly onto the freeway without using the acceleration lane. This behavior results in undesirable merging operations.

The taper at the downstream end of a parallel-type acceleration lane should be a suitable length to guide the vehicle gradually onto the through lane of the freeway. A taper length of approximately 90 m [300 ft] is suitable for design speeds up to 110 km/h [70 mph].

The length of a parallel-type acceleration lane is generally measured from the point where the left edge of the traveled way of the ramp joins the traveled way of the freeway to the beginning of the downstream taper. Whereas, in the case of the taper type entrance, acceleration is accomplished on the ramp upstream of the point of convergence of the two roadways, acceleration usually takes place downstream from this point in the case of the parallel type. However, a part of the ramp proper may also be considered in the acceleration length, provided the curve approaching the acceleration lane has a long radius of approximately 300 m [1,000 ft] or more, and the motorist on the ramp has an unobstructed view of traffic on the freeway to his or her left. The minimum acceleration lengths for entrance terminals are given in Exhibit 10-70, and the adjustments for grades are given in Exhibit 10-71.

The operational and safety benefits of long acceleration lanes provided by parallel-type entrances are well recognized. If parallel-type entrances are used, they are most appropriate at ramps expected to experience congested conditions because ~~A~~a long acceleration lane provides more time for the merging vehicles to find an opening in the through-traffic stream. An acceleration lane length of at least 360 m [1,200 ft], plus the taper, is desirable wherever it is anticipated that the ramp and freeway will frequently carry traffic volumes approximately equal to the design capacity of the merging area.

Metric

Acceleration length, L (m) for ~~entrance curve~~ design speed of controlling feature, V_N (km/h)

Highway		Stop condition	20	30	40	50	60	70	80
Design speed, V (km/h)	Merge ~~S~~speed ~~reached~~, V_a (km/h)	and average running speed (i..e., initial speed) at controlling feature, V_a' (km/h)							
		0	20	28	35	42	51	63	70
50	37	60	50	30	—	—	—	—	—
60	45	95	80	65	45	—	—	—	—
70	53	150	130	110	90	65	—	—	—
80	60	200	180	165	145	115	65	—	—
90	67	260	245	225	205	175	125	35	—
100	74	345	325	305	285	255	205	110	40
110	81	430	410	390	370	340	290	200	125
120	88	545	530	515	490	460	410	325	245

Note: Uniform 50:1 to 70:1 tapers are recommended where lengths of acceleration lanes exceed 400 m.

~~Note: Uniform 50:1 to 70:1 tapers are recommended where lengths of acceleration lanes exceed 400 m.~~ V = design speed of highway (km/h)

V_a = 10 km/h below the average running speed on the highway (km/h)

V_N = design speed of controlling feature (km/h)

V_a' = average running speed at controlling feature (km/h)

US Customary

Acceleration length, L (ft) for ~~entrance curve~~ design speed of controlling feature (mph)

Highway		Stop condition	15	20	25	30	35	40	45	50
Design speed, V (mph)	Merge ~~S~~speed ~~reached~~, V_a (mph)	and average running speed (i.e., initial speed) at controlling feature, V_a' (mph)								
		0	14	18	22	26	30	36	40	44
30	23	180	140	—	—	—	—	—	—	—
35	27	280	220	160	—	—	—	—	—	—
40	31	360	300	270	210	120	—	—	—	—
45	35	560	490	440	380	280	160	—	—	—
50	39	720	660	610	550	450	350	130	—	—
55	43	960	900	810	780	670	550	320	150	—
60	47	1200	1140	1100	1020	910	800	550	420	180
65	50	1410	1350	1310	1220	1120	1000	770	600	370
70	53	1620	1560	1520	1420	1350	1230	1000	820	580
75	55	1790	1730	1630	1580	1510	1420	1160	1040	780

Note: -Uniform 50:1 to 70:1 tapers are recommended where lengths of acceleration lanes exceed 1,300 ft.

V = design speed of highway (mph)

V_a = 5 mph below the average running speed on the highway (mph)

V_N = design speed of controlling feature (mph)

V_a' = average running speed at controlling feature (mph)

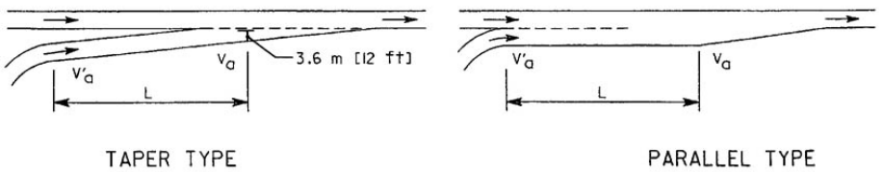


Exhibit 10-70. Minimum Acceleration Lengths for Entrance Terminals with Flat Grades of Two Percent or Less

Metric							US Customary					
Design speed of highway (km/h)	Deceleration lanes						Design speed of highway (mph)	Deceleration lanes				
	Ratio of length on grade to length on level for design speed of turning curve (km/h)[a]							Ratio of length on grade to length on level for design speed of turning curve (mph)[a]				
All speeds	3 to 4% upgrade				3 to 4% downgrade		All speeds	3 to 4% upgrade			3 to 4% downgrade	
	0.9				1.2			0.9			1.2	
All speeds	5 to 6% upgrade				5 to 6% downgrade		All speeds	5 to 6% upgrade			5 to 6% downgrade	
	0.8				1.35			0.8			1.35	
Design speed of highway (km/h)	Acceleration lanes						Design speed of highway (mph)	Acceleration lanes				
	Ratio of length on grade to length of level for design speed of turning curve (km/h)[a]							Ratio of length on grade to length of level for design speed of turning curve (mph)[a]				
	40	50	60	70	80	All speeds		20	30	40	50	All speeds
	3 to 4% upgrade					3 to 4% downgrade		3 to 4% upgrade				3 to 4% downgrade
60	1.3	1.4	1.4	—	—	0.7	40	1.3	1.3	—	—	0.7
70	1.3	1.4	1.4	1.5	—	0.65	45	1.3	1.35	—	—	0.675
80	1.4	1.5	1.5	1.5	1.6	0.65	50	1.3	1.4	1.4	—	0.65
90	1.4	1.5	1.5	1.5	1.6	0.6	55	1.35	1.45	1.45	—	0.625
100	1.5	1.6	1.7	1.7	1.8	0.6	60	1.4	1.5	1.5	1.6	0.6
110	1.5	1.6	1.7	1.7	1.8	0.6	65	1.45	1.55	1.6	1.7	0.6
120	1.5	1.6	1.7	1.7	1.8	0.6	70	1.5	1.6	1.7	1.8	0.6
	5 to 6% upgrade					5 to 6% downgrade		5 to 6% upgrade				5 to 6% downgrade
60	1.5	1.5	—	—	—	0.6	40	1.5	1.5	—	—	0.6
70	1.5	1.6	1.7	—	—	0.6	45	1.5	1.6	—	—	0.575
80	1.5	1.7	1.9	1.8	—	0.55	50	1.5	1.7	1.9	—	0.55
90	1.6	1.8	2.0	2.1	2.2	0.55	55	1.6	1.8	2.05	—	0.525
100	1.7	1.9	2.2	2.4	2.5	0.5	60	1.7	1.9	2.2	2.5	0.5
110	2.0	2.2	2.6	2.8	3.0	0.5	65	1.85	2.05	2.4	2.75	0.5
120	2.3	2.5	3.0	3.2	3.5	0.5	70	2.0	2.2	2.6	3.0	0.5

[a] Ratio from this table multiplied by the length in Exhibit 10-70 or Exhibit 10-73 gives length of speed change lane on grade.

Exhibit 10-71. Speed Change Lane Adjustment Factors as a Function of Grade

Single-Lane Free-Flow Terminals, Exits

Taper-type exits. The taper-type exit fits the direct path preferred by most drivers, permitting them to follow an easy path within the diverging area. The taper-type exit terminal beginning with an outer edge alignment break usually provides a clear indication of the point of departure from the through lane and has generally been found to operate smoothly on high-volume freeways. The divergence angle is usually between 2 and 5 degrees.

Studies of this type of terminal show that most vehicles leave the through lane at relatively high speeds, thereby reducing the potential for rear-end collisions as a result of deceleration on the through lane. The speed change can be achieved off the traveled way as the exiting vehicle moves along the taper onto the ramp proper. Exhibit 10-72A shows a typical design for a taper-type exit.

Vehicles should decelerate after clearing the through-traffic lane and before reaching the point limiting design speed for the ramp proper. The length available for deceleration may be assumed to extend from a point where the right edge of the tapered wedge is about 3.6 m [12 ft] from the right edge of the right through lane, to the location of the controlling feature of the ramp. This feature may be the point of initial curvature ~~of the exit ramp~~ (i.e., the first horizontal curve on the ramp), or it may be the crossroad terminal for a straight ramp. The length provided between these points should be at least as great as the distance needed to accomplish the appropriate deceleration, which is governed by the speed of traffic on the through lane and the speed to be attained on the ramp. Deceleration may end in a complete stop, as at a crossroad terminal for a diamond interchange, or the critical speed may be governed by the curvature of the ramp roadway. Minimum deceleration lengths for various combinations of design speeds for the highway and for the ramp roadway are given in Exhibit 10-73. Grade adjustments are given in Exhibit 10-71.

Although it is desirable that vehicles do not decelerate on the freeway mainline prior to diverging on the speed-change lane, a recent study (Torbic et al.) found that this does occur. Because the values listed in Exhibit 10-73 for minimum deceleration length on exit ramps do not account for any deceleration in the through lane, these design values provide a conservative estimate for design. It is still prudent for the designer to assume that all deceleration takes place in the speed-change lane when determining the minimum deceleration lane length.

The taper-type exit terminal design can be used advantageously in developing the desired long, narrow, triangular emergency maneuver area just upstream from the exit nose located at a proper offset from both the through lane and separate ramp lane. The taper configuration also works well in the length-width superelevation adjustments to obtain a ramp cross slope different from that of the through lane.

The width of the recovery area or the distance between the inner edges of the diverging lanes at the ramp nose is usually 6.0 to 9.0 m [20 to 30 ft]. This entire area should be paved to provide a maneuver and recovery area, but the desired travel path for the ramp roadway should be clearly delineated by pavement markings.

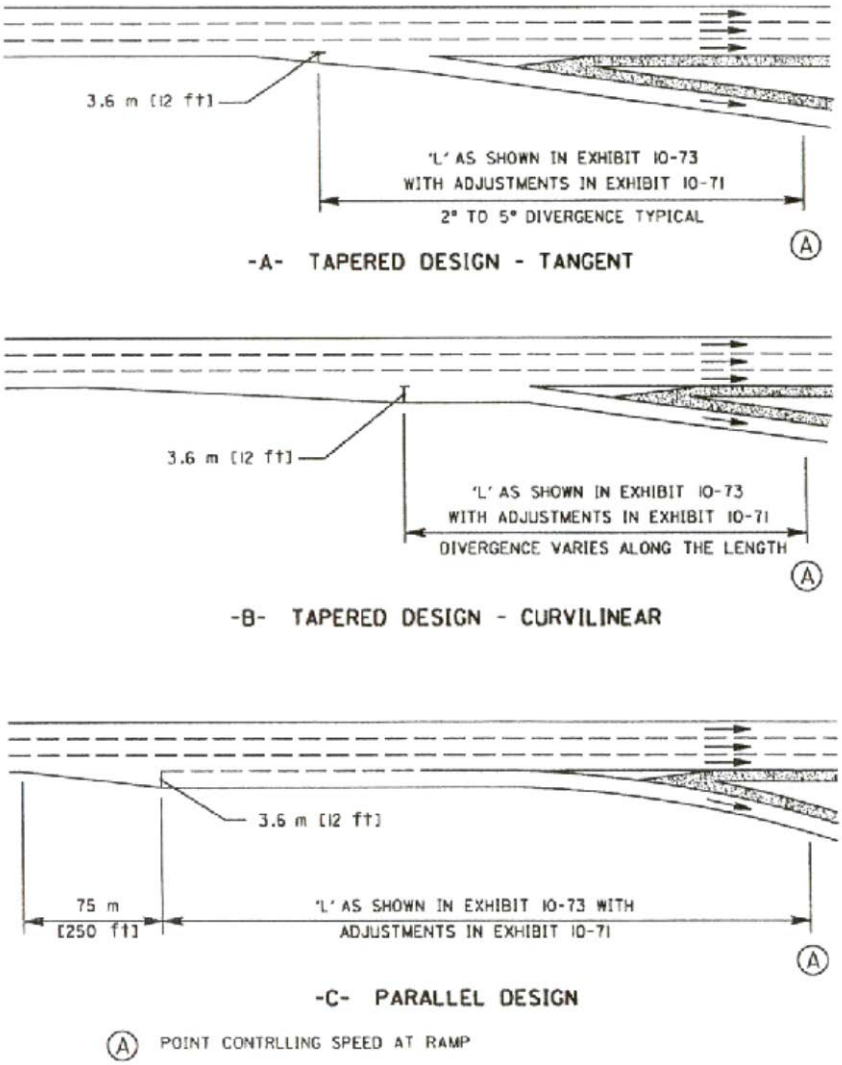


Exhibit 10-72. Exit Ramps—Single Lane

Metric

Deceleration length, L (m) for design speed of controlling feature ~~exit curve~~ V_N (km/h)

Highway design speed, V (km/h)	Diverge~~Speed reached~~speed, V_a (km/h)	Stop condition	20	30	40	50	60	70	80
		For average running speed at controlling feature ~~on exit curve~~ V_a' (km/h)							
		0	20	28	35	42	51	63	70
50	47	75	70	60	45	—	—	—	—
60	55	95	90	80	65	55	—	—	—
70	63	110	105	95	85	70	55	—	—
80	70	130	125	115	100	90	80	55	—
90	77	145	140	135	120	110	100	75	60
100	85	170	165	155	145	135	120	100	85
110	91	180	180	170	160	150	140	120	105
120	98	200	195	185	175	170	155	140	120

Note: Uniform 15:1 to 25:1 tapers are recommended at deceleration lanes.

V = design speed of highway (km/h)
V_a = average running speed on highway (km/h)
V_N = design speed of controlling feature ~~exit curve~~ (km/h)
V_a' = average running speed at controlling feature ~~on exit curve~~ (km/h)

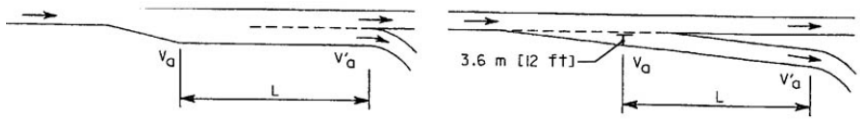
US Customary

Deceleration length, L (ft) for design speed of controlling feature ~~exit curve~~, V_N (mph)

Highway design speed, V (mph)	~~Speed reached~~ Diverge speed, V_a (mph)	Stop condition	15	20	25	30	35	40	45	50
		For average running speed at controlling feature ~~on exit curve~~, V_a' (mph)								
		0	14	18	22	26	30	36	40	44
30	28	235	200	170	140	—	—	—	—	—
35	32	280	250	210	185	150	—	—	—	—
40	36	320	295	265	235	185	155	—	—	—
45	40	385	350	325	295	250	220	—	—	—
50	44	435	405	385	355	315	285	225	175	—
55	48	480	455	440	410	380	350	285	235	—
60	52	530	500	480	460	430	405	350	300	240
65	55	570	540	520	500	470	440	390	340	280
70	58	615	590	570	550	520	490	440	390	340
75	61	660	635	620	600	575	535	490	440	390

Note: Uniform 15:1 to 25:1 tapers are recommended at deceleration lanes.

V =design speed of highway (mph)
V_a =average running speed on highway (mph)
V_N=design speed of controlling feature ~~exit curve~~ (mph)
V_a' = average running speed at controlling feature ~~on exit curve~~ (mph)



PARALLEL TYPE

TAPER TYPE

Exhibit 10-73. Minimum Deceleration Lengths for Exit Terminals with Flat Grades of Two Percent or Less

Parallel-type exits. A parallel-type exit terminal usually begins with a taper, followed by an added lane that is parallel to the traveled way. A typical parallel-type exit terminal is shown in Exhibit 10-72C. This type of terminal provides an inviting exit area, because the foreshortened view of the taper and the added width are very apparent. Parallel-type exits operate best when drivers choose to exit the through lane sufficiently in advance of the exit nose to permit deceleration to occur on the added lane (deceleration lane) and allows them to follow a path similar to that encouraged by a taper design. Drivers who do not exit the through lane sufficiently in advance of the exit nose will likely utilize a more abrupt reverse-curve maneuver, which is somewhat unnatural and can sometimes result in the driver slowing in the through lane. In locations where both the mainline and ramp carry high volumes of traffic, the deceleration lane provided by the parallel-type exit provides storage for vehicles that would otherwise undesirably queue up on the through lane or on a shoulder, if available.

The length of a parallel-type deceleration lane is usually measured from the point where the added lane attains a 3.6 m [12 ft] width to the point where the alignment of the ramp roadway departs from the alignment of the freeway. Where the ramp proper is curved, it is desirable to provide a transition at the end of the deceleration lane. A compound curve may be used with the initial curve desirably and should have a long radius of about 300 m [1,000 ft] or more. A transition or a long radius curve is also desirable if the deceleration lane connects with a relatively straight ramp. In such cases, a portion of the ramp may be considered as a part of the deceleration length, thus shortening to some extent the appropriate length of contiguous parallel lane. Minimum lengths are given in Exhibit 10-73, and adjustments for grades are given in Exhibit 10-71. Longer parallel-type deceleration lanes are more likely than shorter lanes to be used properly. Lengths of at least 240 m [800 ft] are desirable.

The designer should consider that providing deceleration lanes longer than the minimum values listed in *Green Book* Exhibit 10-73 may promote more casual deceleration by exiting drivers, particularly under uncongested or lightly congested conditions. This is not necessarily a negative result, but it does change the operational characteristics of the ramp, as those drivers may maintain higher speeds further into the speed-change lane and possibly into the ramp proper.

The taper portion of a parallel-type deceleration lane should have a taper of approximately 15:1 to 25:1 [longitudinal:transverse]. A long taper indicates the general path to be followed and reduces the unused portion of the deceleration lane. However, a long taper tends to entice the through driver into the deceleration lane. A short taper produces a better “target” to the approaching driver, giving a positive indication of the added lane ahead.

References

American Association of State Highway and Transportation Officials (AASHTO), *A Policy on Geometric Design of Highways and Streets*. Washington, D.C., 2004.

Torbic, D.J, J.M. Hutton, C.D. Bokenkroger, D.W. Harwood, D.K. Gilmore, M.M. Knoshaug, J.J. Ronchetto, M.A. Brewer, K. Fitzpatrick, S.T. Chrysler, and J. Stanley. *NCHRP Report 730: Design Guidance for Freeway Mainline Ramp Terminals*, Transportation Research Board of the National Academies, Washington, D.C., 2012.