Design of Interchange At-Grade Ramp Terminals

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Ramps consist of three components: two terminal ends and a connecting roadway. The terminals can be designed as high-speed transitions between higher-function roadways or as at-grade intersections. At-grade terminal designs may be applicable at both ends of a ramp. Decisions regarding the appropriate type of terminal design are affected by the functional classification of the intersecting roadways. When an at-grade intersection is chosen as the ramp terminal design, all related controls and criteria should be based on the information presented in Chapter IX of the 1990 AASHTO geometric design policy. Operationally, when an at-grade intersection is used as the terminal, the traffic capacity analyses should be based on the intersection capacity procedures outlined in the 1985 *Highway Capacity Manual*. It is suggested that AASHTO geometric design policy emphasize the appropriateness of at-grade intersections as ramp terminals. The use of functional classification and the high-speed and low-speed design criteria for at-grade ramp terminal designs is illustrated and clarified.

t is recognized that interchange ramps are made up of three istinct components, two terminal ends and a connecting oadway (*1*, p. 958). The connecting roadway is often referred o as the ramp proper, or simply the ramp. The terminal ends nay be either free-flow ramp/freeway junctions, which typially accommodate speed-change lanes, or connecting ramp/ rterial or ramp collector junctions, which may be designed s at-grade intersections (*1*, p. 978). Whereas these concepts re generally acknowledged, design criteria and controls for ne at-grade terminal ends of ramps are difficult to assimilate om the information in the AASHTO *Policy on Geometric esign of Highways and Streets* (referred to herein as the olicy). Furthermore, although issues such as functional hiarchy and design speed are implied, specific guidance reirding the application of relevant criteria to at-grade ramp rminals is not explicitly provided in the current Policy.

The objective of this paper is to identify the key design conderations of the at-grade intersection portion of an interiange ramp terminal. The paper also stresses the importance design consistency and hierarchy of movement in selecting e appropriate design criteria for the connecting ramp and rning roadway. Specific design considerations of the ramp oper are provided by Harwood and Mason in this Record.

ACKGROUND

ASHTO Policy is very clear that there are three types of tersections: at-grade intersections, highway grade separations without ramps, and interchanges. It also states that "each has a field of usage in which it is practical, but the limits are not sharply defined" (*1*, p. 857). Interchanges are considered to be practical for all types of intersecting roads for any range of design speeds (*1*, p. 859).

At-grade intersections are common along arterial, collector, and local roadways as an acceptable means for moving traffic between the intersecting roadways or providing ingress and egress to adjacent land uses. The site-specific design of an at-grade intersection is an element of the overall design of a grade-separated interchange. The terminal end of an interchange ramp that forms the at-grade intersection is logically designed according to intersection criteria. Consider the at-grade ramp terminal portions of a diamond interchange. The diamond interchange type of ramp design provides the needed transition between roadways of a higher type design (e.g., freeways) and roadways of a lower type design (e.g., collector streets).

The use of an at-grade terminal design is not restricted to a diamond-type interchange. At-grade terminal design is also applicable at both ends of a ramp for a one-quadrant design along with other types of ramps that interchange traffic between an arterial and a collector, or between a collector and the on-site circulation system of an adjacent property. The current Policy certainly provides the latitude to consider these types of designs.

The intent of the AASHTO Policy is to "provide guidance to the designer by referencing a recommended range of values for critical dimensions" (*1*, p. xliii). The Policy also indicates that "sufficient flexibility is permitted to encourage independent designs tailored to particular situations" (*1*, p. xliii). Previous geometric design criteria (pre-1984 AASHTO design policies) and operational quality assessments (pre-1985 editions of the *Highway Capacity Manual*) were primarily based on traffic volume ranges. Designs performed using the earlier criteria were produced under the assumption that highways with comparable design speeds and traffic volumes would be constructed to the same standards and assumed to provide similar levels of service. It has been demonstrated, however, that there can be considerable difference in their actual operation.

Functional classification–based design criteria reflect level-of-service calculations that vary according to the function of the highway facility (*1*, p. 16). The underlying design philosophy is based on the principle of "hierarchy of movement" (*1*, pp. 1–2). Current geometric design principles begin with the establishment of the functional classification of a road segment, and then the design criteria for a road segment are

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selected to be commensurate with its function (*1*, pp. 16–17). Function is reflected by use of high- and low-design speed ranges and various measures of level of service (e.g., density, delay, reserve capacity, etc.).

Design speed is selected on the basis of factors such as topography, adjacent land use, and functional classification of the highway. The designer selects as high a design speed as is reasonable to achieve the desired degree of safety, mobility, and efficiency while considering environmental quality, economics, aesthetics, and social or political constraints (*1*, p. 63). Given these design considerations, the Policy states that "once selected, all pertinent features of the highway, including the ramps, should be related to design speed to obtain a balanced design" (*1*, p. 63).

This statement implies that if a driver has been traveling on a high-speed facility and leaves via a ramp to a lower-speed facility, the terminal end design features (i.e., curvature, superelevation, etc.) near the high-speed roadway should be based on "high-speed" design criteria. The high-speed design continues through the ramp proper until it approaches the other terminal. On the approach to the downstream end of ramps, the high-speed design would continue if the end terminal is a ramp-freeway junction. If the end terminal is a ramp/arterial or ramp/collector junction, however, a low-speed design (at-grade intersection) could be used. Likewise, once drivers enter or have been traveling on a lower functional classification roadway or street system, they have already come to accept and expect the driving environment of a low-speed road/street system. Ramps from these types of facilities could be designed using "low-speed" design criteria.

Ramp terminal criteria should be dependent on a "functional classification" for the connecting roadway. In other words, the design of the terminal ends need not be higher than the types of facilities that are being connected. The Policy supports this opinion and states that "generally, the horizontal and vertical alinement standard of ramps is below that of the intersecting highway, but in some cases it may be equal" (*1*, p. 958). This statement is the basis for the various considerations that need to be examined when selecting ramp design speed.

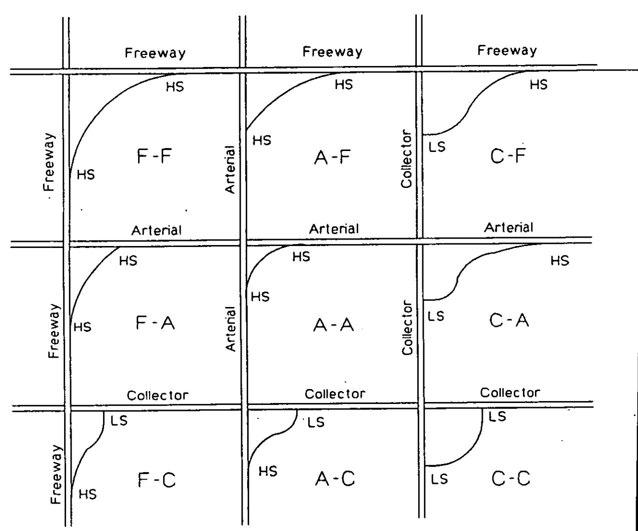
There are significant differences between design criteria applicable to low- and high-design speeds. The differences between high- and low-speed design criteria are clearly cited in the Policy. The maximum limit for low-speed design is 40 mph, and the minimum limit for high-speed design is 50 mph (*1*, p. 68). A geometric feature directly controlled by design speed selection is horizontal curvature. The parameter specifically affected by the selection of high-speed versus low-speed design criteria is f, the side friction factor, more correctly referred to as net lateral acceleration. When selecting maximum allowable side friction factors for design, the point is chosen at which the lateral force would permit a driver to experience discomfort that will, in turn, cause the driver to react instinctively to avoid higher speed (*1*, p. 143). This lateral acceleration is well below the value at which the driver would be forced "up against the door." A *g* force of that magnitude would be at the limit of available friction as when the driver is in an emergency locked wheel skid or "yawing" maneuver around a horizontal curve. In such situations, the *g* forces are, at a minimum, twice the lateral acceleration rates used in low-speed urban street design. At lower running speeds, drivers tolerate more discomfort by accepting increased lateral *g* forces. This recognition is the Policy's justification that permits a designer to use increased side friction levels for the design of low-speed horizontal curves (*1*, p. 143).

DESIGN OF THE GEOMETRIC ELEMENTS

Functional Classification

The first consideration in the design of a ramp and its at-grade terminals is the functional classification of the roadways that are being served by the ramp connector. When conditions permit, roadways are designed such that the geometric characteristics of the ramp and the at-grade terminals satisfy the "desirable" criteria recommended by AASHTO Policy. Unfortunately, constraints such as existing topography, other existing features, and cost-effectiveness often require that the design elements of an interchange be set at or near the minimum values.

To meet driver expectancy and provide a smooth transitio from one roadway to another, the design characteristics o the connecting ramp and its terminal ends should be base on the functional classification of the two intersecting road ways. For example, consider the difference in the at-grad terminal and the ramp proper geometries required by Con figuration F-A (freeway to arterial) as compared with Con figuration F-C (freeway to connector) in Figure 1. In Config uration F-A, the ramp proper serves traffic leaving a high speed freeway facility and entering a moderate- to high-spee arterial roadway. The ramp terminal's geometrics correspon to the applicable design criteria for the respective roadways The at-grade ramp terminus exiting the Interstate accom modates high travel speeds. Whereas the terminus tying th



(1) HS = high-speed; LS = low-speed

(2) The use of higher design speed criteria (i.e., lower 'f' values) would be considered where topography, environment and economics permit.

FIGURE 1 Minimum design speed criteria for ramp terminals.

ramp proper into the arterial would not be designed to as great a design speed as necessary for a freeway terminus, it would still have a greater design speed than that of a terminus on a collector. Likewise, a ramp connecting an arterial roadway to a collector roadway is not designed to the same standard as a ramp connecting a freeway to a major arterial roadway.

According to AASHTO policy, the freeway is not a functional class in itself and instead is normally classified as a principal arterial (*1*, p. 15). Freeways, however, have unique geometric criteria that demand a separate design designation apart from other arterials. A roadway connecting two freeways, F-F in Figure 1, requires the use of high-speed design criteria commensurate with the geometrics of the freeway proper. The connecting roadway provides design consistency and meets the driver's expectations. It follows that driver expectancy could be violated if a "short" roadway connecting a low-speed urban street to a collector, C-C in Figure 1, roadway was designed according to high-speed design criteria. It is likely that a driver would increase operating speed through the short roadway section and subsequently be confronted with low-speed operating conditions immediately downstream.

Ramp Design Speed

Ramp design speed selection depends primarily on the function (and respective design speed) of the adjacent/connecting roadways. If the terminus end is the junction between a low-speed, low functional class roadway, the geometric elements are designed to accommodate the driver's expectancy and, as such, provide a design consistent with a low-speed design roadway. When the terminus connects with a high-speed, high functional class roadway, the design controls are selected for high-speed operations. As site conditions permit, the ramp proper provides the location to transition between geometric design requirements of the different functional roadway classes. When the functional classes of the intersecting roadways are similar, the geometry of the ramp proper is typically based on an equal or lower design speed value.

AASHTO Policy provides guide values for ramp design speeds as related to highway design speed (Table X-1, 1990). Table X-1, in the 1990 Policy, also contains the note "see Table III-6" for finding the corresponding minimum radius for each design speed. Table III-6, in the 1990 Policy, contains the minimum radii for rural highways and high-speed urban streets. These radii are based on the friction values that are considered appropriate for high-speed designs. The Policy explicitly states that these values "do not pertain to the ramp terminals, which should be properly transitioned and provided with speed-change facilities adequate for the highway speed involved" (*1*, p. 960). The Policy specifically recommends that at-grade terminal designs are "predicated on near-minimum turning conditions such as those given in Chapter IX, At-Grade Intersections" (*1*, p. 960).

Sight Distance

Providing adequate sight distance is an important element in the design of interchange at-grade ramp terminals. The Policy states that "although ramp terminals may be considered part of the interchange design . . . the terminals should be planned in accordance with design principles for at-grade intersections and with particular attention to sight distance characteristics." Therefore, intersection sight distance for at-grade ramp terminals on through roadways that are considered "low speed" in Figure 1 may be calculated using Case III—Stop Control on the Minor Roads in the Sight Distance section of the At-Grade Intersections, Chapter IX of the 1990 Policy.

The only difference concerning sight distance between at-grade intersection and at-grade ramp terminals is the added concern of the grade separation structure that supports the intersecting roadway. Where a through roadway is an underpass, the major highway abutment or columns of the grade separation structure can limit the sight distance available to the driver at the ramp terminal. In the same manner, where the through roadway is an overpass, the minor roadway parapet or bridge railing can limit the available sight distance. Figure IX-42-A, in the 1990 Policy, graphically depicts the limiting of the sight distance by the grade separation structure.

Special consideration must be taken in the design of vertical curves in the area of ramp terminals. The necessary sight distance lengths for the left-turning vehicle from the ramp terminal may make it necessary to design longer vertical curves than would be otherwise necessary to provide adequate stopping sight distance for drivers on the through roadway. According to Fitzpatrick and Mason, "K values needed to produce vertical curves that will provide the required sight distances . . . are generally greater than the Green Book K values used for vertical curve design" (*2*, p. 2). Figure IX-42-B, in the 1990 Policy, graphically depicts the possibility of the vertical curvature limiting the at-grade ramp terminal sight distance.

Ramp terminals on high-speed roadways, as shown in Figure 1, are considered free-flow facilities and have different concerns than do at-grade ramp terminals on low-speed roadways. For exiting vehicles on a high-speed roadway, sight distance must be provided to allow the vehicle to leave the through lanes of traffic without hindering the through traffic (*1*, p. 980). The sight distance considerations permit a driver to see the roadway exit (ramp terminal) and make the necessary lane changes in a safe and timely manner causing no hindrance to the through traffic. The important feature in providing this sight distance is the location of the grade separation structure for the intersecting road with respect to the free-flow ramp terminal. According to AASHTO, sight distance provided to exiting vehicles "comparable to guidelines set out under the decision sight distance discussion is recommended where practical" (*1*, p. 1033). The same sight distance consideration is necessary for the drivers of vehicles entering the roadway from a ramp terminal. Sight distance should be provided so that the entering driver is able to see the through roadway and make a well-informed decision on timing the entry into the through traffic lanes.

Certain inconsistencies are found in the sight distance recommended by the 1990 AASHTO Green Book for at-grade ramp terminals as compared with the at-grade intersection recommended values. Fitzpatrick and Mason found that the required sight distance values ("looking left"), for ramp terminal designs cited in Table IX-9 of the 1984 Policy, are 21 percent lower than the distances recommended for the same design speeds in the at-grade intersection procedure (Figure

IX-27 of the 1984 Policy) (2). The reason for this discrepancy is the use of different travel distance assumptions in each of the procedures.

The changes that occurred from the 1984 Policy to the 1990 Policy relative to recommended sight distance did not alleviate this inconsistency. Table 1 demonstrates this inconsistency. A specific explanation has not been found by the authors as to why the at-grade ramp sight distance values are different from those offered for at-grade intersections.

Horizontal Curvature

On the basis of the previous discussion, turning roadways (ramps) can fall into two distinct groups: high speed, for design speeds greater than 50 mph, and low speed, for design speeds less than 40 mph. For design speeds in the range of 40 to 50 mph, it is desirable to choose a design speed based on the higher functional classification of the connected roads for the choice of radius criteria. Low-speed design minimum radii and corresponding friction factors are found in Table III-17, in the 1990 Policy "Minimum Radii for Intersection Curves"; high-speed design radii and corresponding friction factors are found in Table III-6 in the 1990 Policy. These two tables have been consolidated to form Table 2, which can be used in conjunction with Figure 1 to select the maximum lateral acceleration values (side-friction factors) that are

1. Commensurate with the functional classification of the connecting roadways,

TABLE 1 Comparison of Recommended Sight Distance Values for P Vehicle

Assumed Design Speed on the Crossroad Through the Interchange	1984 Green Book Table IX-9[1]	1984 Green Book Figure IX-27[2] (B-1 Curve)	1990 Green Book Table IX-9[3]	1990 Green Book Figure IX-40[4] (B-2b Curve)
70	740	950	710	1550
60	630	825	610	1150
50	530	675	510	850
40	420	550	410	575
30	320	425	310	375

1 Values from column in 1984 AASHTO Green Book, Table IX-9 labeled "Sight Distance Required to Permit Design Vehicle to Turn Left from Ramp to Crossroad (ft)."

2 Values from B-1 Curve in 1984 AASHTO Policy Figure IX-27. B-1 curve is labeled in figure as follows: "Safe sight distance for P vehicle turning left into two-lane highway across P vehicle approaching from left."

3 Values from column in 1990 AASHTO Green Book, Table IX-9 labeled "Sight Distance Required to Permit Design Vehicle to Turn Left from Ramp to Crossroad (ft) and not Interfere with oncoming traffic from left."

4 Values from B-2b Curve in 1990 AASHTO Green Book Figure IX-40. B-1 curve is labeled in figure as follows: "Sight distance for P vehicle to turn left into two-lane highway and attain 85% of design speed without being overtaken by a vehicle approaching from the right reducing speed from design speed to 85% of design speed."

TABLE 2 Maximum Lateral Acceleration Values (Side Friction Factors, f) for Horizontal Curve Design at Ramp Terminals [1]

Functional Classification[2]		Minimum turning roadways			High speed merge/diverge		
	[3]	20	30	40	50	60	70
Freeway	HS	N/A	.16	0.15	0.14	0.12	0.10
	LS	N/A	N/A	N/A	N/A	N/A	N/A
Arterial	HS	0.17	.16	0.15	0.12	0.12	0.10
	LS	0.27	0.23	0.16	N/A	N/A	N/A
Collector	HS	0.17	0.16	0.15	0.14	N/A	N/A
	LS	0.27	0.23	0.16	N/A	N/A	N/A
Local	HS	0.17	0.16	0.15	N/A	N/A	N/A
	LS	0.27	0.23	0.16	N/A	N/A	N/A

[1] For horizontal curve departing or entering the adjacent roadway.
[2] From and/or to which the ramp proper is connecting.
[3] HS = high-speed criteria (Ref. AASHTO 1990, p. 154, Table III-6).
LS = low-speed criteria (Ref. AASHTO 1990, p. 197. Table III-17).

2. Consistent with the respective design speed criteria, and
3. Appropriate for the site-specific ramp terminal configuration.

The horizontal curve design controls for ramps that are recommended in the 1990 Policy are based on the criteria first presented in the 1954 *Policy on Geometric Design of Rural Highways* (*3*). Later geometric design policies, the 1957 and 1973 Urban Policy and 1965 Rural Policy, and, most recently, the 1984 Policy, serve as the foundation for the 1990 Policy (*4–7*). The following illustrates the continued consistency in horizontal curve design policy for ramps from 1954 to the present.

Table III-17 in the 1990 Policy recommends horizontal curvature for design speeds of less than 40 mph. This table is similar to Table VII-3 in the 1954 Policy and Table VII-3 in the 1965 Policy. Inspection of the three tables results in the same minimum radii for the corresponding design speed. For example, in the 1954 Policy for a "turning" design speed of 20 mph, a minimum radius of 90 ft is suggested. The recommended value for 20 mph in the 1990 Policy is also 90 ft.

The minimum radii recommended by Table III-17 in the 1990 Policy for low-speed turning roadways (less than 40 mph) are based on side friction factors that are significantly higher than the side friction factors recommended in Table III-6 of the current Policy for curves on open highways (high-speed facilities). This difference is explained by AASHTO as due to "drivers' acceptance and use of higher side friction in operating around curves at intersections as opposed to side friction accepted and used on the through highway" (*1*, p. 195).

Table X-1 in the 1990 Policy offers recommended values for ramp design speed that relate to the connecting highway design speed. The values recommended in 1990 are similar to the values originally recommended in Table IX-2 of the 1954 Policy. The similarity to Table X-1 in the 1990 Policy values is also evident in Table IX-2 of the 1965 Policy and Table J-1 of the 1973 Policy. The recommended radii (of the 1965 and 1973 Policies) corresponding to the desirable and minimum ramp design speed are based on the higher side friction factors associated with low-speed turning roadways. This method for calculating the minimum radius differs from Table X-1 in the 1990 Policy, which refers to Table III-6 in the 1990 Policy for the corresponding minimum radius. Again, Table III-6 in the 1990 Policy uses the lower side friction factors associated with high-speed facility design. This change took place when the 1965 Rural Policy and 1973 Urban Policy were combined into the 1984 version of the Policy.

Minimum-turning roadway curves for various speeds, such as those for intersection design, are presented in Chapter III of the 1990 Policy. The criteria in Chapter III apply directly to design of the curves for at-grade ramp terminals and, in some cases, the ramp proper (*1*, p. 961). The recommended radius values for minimum-turning roadways are calculated on the basis of the side friction factors cited as low-speed urban street design criteria.

The current Policy states that "three segments of a ramp should be analyzed to determine superelevation rates that would be compatible with the design speed and the configuration of the ramp. The exit terminal, the ramp proper, and the entrance terminal should be studied in combination to ascertain the design speed and superelevation rates" (*1*, p. 966). Calculation of a radius of horizontal curvature requires selection of both a "limiting" side friction factor and a "maximum" rate of superelevation as governed by local practice. The selection of the limiting side friction, however, is fundamentally determined on the basis of the functional class of the roadway and its commensurate "design speed" criteria.

The 1990 Policy states further, "On ramps designed for speeds of 40 mph or less, superelevation ranges given in Table IX-12 in the 1990 Policy are appropriate for design of the ramp proper" (*1*, p. 966). The specific radius values listed in Table IX-12 in the 1990 Policy are the results of using Table IX-12 in the 1990 Policy superelevation ranges and the respective f values of the low-speed urban street criteria.

OPERATIONAL CONSIDERATIONS

Operationally, a ramp that is designed with at-grade intersection terminals is neither a fully developed "classic" interchange nor simply an at-grade intersection. As with the geometric design aspects, discretion must be used in determining what procedures yield reasonable estimates of actual operating conditions.

Two sections of the *Highway Capacity Manual* address this type of design: the ramp capacity procedures and the intersection capacity procedures. The ramp capacity procedures are based on a set of regression equations that are used to predict Lane 1 (shoulder lane) volumes. These regression equations were developed as part of a U.S. Bureau of Public Roads (BPR) study that was done in the early 1960s. Observations were made and data recorded as part of "219 studies conducted at 195 ramp-freeway connections" (*8*). The regression equations and the techniques for using them to asses ramp terminal capacity have remained essentially unchanged through the development of the 1965 *Highway Capacity Manual* (*9*), and they are still used currently in practice from the 1985 *Highway Capacity Manual* (*10*). Whereas many of the ramp terminals that were observed had geometric features that could be considered inadequate by today's standards (i.e., no acceleration lanes or tight, intersectionlike geometry), use of these ramp capacity procedures to analyze at-grade terminals is inappropriate in most cases. The BPR procedures state that traffic coming onto the ramp "supplied via a traffic signal or an ordinary street network . . . is outside the scope of this study" and "if ramp traffic is supplied by another freeway or expressway, the 'diverging' [movement] from that facility is within the scope of this study." This attitude is echoed by the current manual, which states:

> The ramp-street junction can be of a type permitting uncontrolled merging of diverging movements to take place, or it can take the form of an at-grade intersection.
>
> This chapter [Chapter 5, Ramps and Ramp Junctions] provides procedures for the capacity analysis of ramp-freeway junctions and ramp roadways. At-grade intersections may be analyzed using the procedures of Chapter 9, Signalized Intersections, or Chapter 10, Unsignalized Intersections. (*10*)

All of these sources acknowledge that the operations of low-speed design, at-grade ramp terminal ends are comparable with at-grade intersection operations. In other words, if the terminal treatment is geometrically designed as an at-grade intersection, it follows that it should be analyzed as an at-grade intersection. Using intersection capacity procedures for low-speed at-grade ramp terminals will generally yield lower overall capacity values than the ramp capacity procedures as well.

The ramp roadway discussion in the HCM provides limited guidance concerning the operating characteristics of ramp roadways or what is termed the ramp proper. The section on ramp capacity procedures offers some guidance regarding the ramp proper in Table 5-5 of the 1985 HCM (*10*). The various conditions regarding the use of this table, however, are vague, and Table 5-5 refers only to the ramp roadway itself. Even though up to 1,700 pcph may be accommodated in a single-lane ramp, this does not guarantee that such a volume can be accommodated through a single-lane ramp terminal or at the ramp-street junction.

The HCM states that as a general rule of thumb, where volumes exceed 1,500 pcph, a two-lane ramp-freeway terminal will be needed, and a two-lane ramp should be provided. Furthermore, even where a one-lane ramp and ramp terminal are sufficient from the capacity point of view, a two-lane ramp is generally provided if the ramp is located on a steep grade or has minimal geometrics. This rule of thumb is used because with corrections for trucks and mixing traffic in the right lane, the capacity of the ramp terminal is frequently below the 1,700 value for the high-end capacity of ramp roadways. For safety and operational reasons, it is highly unlikely that there would be a two-lane exit terminal with a one-lane ramp roadway. Therefore, the heuristic is put forth as a rule that will cover a more frequently occurring situation. The HCM recognizes that this does not cover all situations and gives the designer the option of using a two-lane ramp roadway without a two-lane terminal for three reasons. They are as follows:

1. The ramp is longer than 1,000 ft, to provide opportunities to pass stalled or slow-moving vehicles.
2. Queues are expected to form on the ramp from a controlled ramp-street junction, to provide additional storage.
3. The ramp is located on a steep grade or has minimal geometric (*9*).

A brief overview of the historical development of Table 5-5 illustrates the need to carefully apply the cited guidelines. The HCM states that "there is very little information concerning the operational characteristics of ramp roadways" (*11*). Table 5-5 exists to provide guidance on "approximate service flow rates for ramp roadways" (*10*).

Table 5-5 is based on information found in a report on capacity analysis techniques for freeway facilities reported by Leisch in 1974 (*11*, p. 26). Leisch's values are slightly different from the HCM table, but it is easily recognized as the predecessor of Table 5-5. After 30 years, the values for ramp roadway capacity are still approximations without empirical basis.

SUMMARY AND CONCLUSIONS

A critical review of the historical development of interchange ramp terminal geometric design has demonstrated the following:

- AASHTO Policy clearly recognizes geometric design based on "low-speed" and "high-speed" criteria.
- The selected design speed criteria should be commensurate with the functional classification of the roadway.
- The geometric design philosophy of horizontal curve design has not changed since its inception.
- The geometric design of at-grade ramp terminals is established on the basis of the adjacent and intersecting roadway design speed.
- At-grade terminal designs forming at-grade intersections are predicated on near-minimum turning conditions of Chapter IX of the AASHTO Policy.

The review has demonstrated the following concerning operations:

- The capacity analysis techniques for ramp terminals in the current *Highway Capacity Manual* are based on observations made at ramps that were part of ramp-freeway junctions.
- The intersection capacity analysis techniques (signalized and stop control) in the current *Highway Capacity Manual* are appropriate for the analysis of at-grade ramp terminals.
- Current values for estimating the capacities of ramp roadways (the ramp proper) are based on 30-year-old approximations.

RECOMMENDATIONS

The following statements should be considered in future geometric design policies and research:

- Minimum design speed criteria should be cited for at-grade ramp terminal design on the basis of the functional classification of terminal roadway.
- The information prepared for Figure 1 could be used to clarify the use of high and low design speeds in relation to functional classification of the terminal roadway.
- A presentation similar to Table 2 could be used to combine the maximum lateral acceleration values relating to the functional classification of the terminal roadway and respective high- or low-speed criteria.
- Future research should investigate whether the allowable maximum lateral acceleration values have changed since the selection of the original values first published in the 1954 rural Policy.
- The inconsistencies between at-grade intersection sight distance values and the ramp terminal sight distance values should be examined.

In addition, the following statements are recommended for consideration regarding operational analysis of ramp terminals:

- Ramp terminals designed according to high-speed design criteria should be analyzed according to ramp junction capacity analysis procedures.
- At-grade ramp terminals designed according to low-speed design criteria should be analyzed according to at-grade intersection capacity analysis procedures.
- The capacity estimation procedures in the current *Highway Capacity Manual* should be modified to reflect the higher volumes that are frequently observed at these types of facilities

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