Interchange Ramp Characteristics
(Selection and Design)

by

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INTRODUCTION

The purpose of this paper is to provide an interchange designer with information to aid in the selection of the type of ramp to use, that may best fit a specific project. It is suggested that the designer acquire the references listed, for a through treatment of interchange ramps. These references are available on the internet, at no cost, except for the AASHTO Greenbook.

Also included is a section on the role the ramp geometry plays on vehicle performance, that is the, acceleration, deceleration and rollover, of tractor-trailer trucks.

Two types of ramp terminals used on freeways for both the exiting and entering process, are the parallel type and the taper type. State Departments of Transportation have their preferred types. In a survey conducted by the Transportation Research Board, they reported, that forty-one of forty-five states responding to the survey, use a taper type for exits and thirty-four of forty-five use the parallel type for entrances.

Three parts of an interchange ramp are:

1. The terminal on the high speed facility (mainline), includes the deceleration or acceleration portion, the speed change lane, (SCL) and the tapered ends. (Figure 1)
2. The body of the ramp, composed of tangents and curves.
3. The terminal at the crossing highway. This terminal could be an intersection at an arterial highway or it could merge into a second high speed facility.

The most important requirement of a ramp terminal is that diverging and merging maneuvers be accomplished safely, orderly and comfortably. Exiting drivers should not be forced to begin breaking until they are clear of the mainline. Entering drivers should be able to merge into gaps on the mainline, at speeds that are close to the through traffic speeds.

Primary guidance for all ramp terminal selection and design is from the current edition of “A Policy on Geometric Design of Highways and Streets”, and the State’s Highway Design Manual, where the project is located.
MAINLINE RAMP TERMINAL TYPES

TAPERED ENTRANCE TERMINAL

A few State Departments of Transportation use the tapered type entrance ramp terminal, this type is merged with the mainline using a 50:1 or 70:1 taper rate. AASHTO and FHWA recommend that the entering vehicle be at a speed that is at least within 5 mph of the operating speed of the mainline, at a point where the left edge of the ramp travel way has joined the right edge of the mainline travelway.

Tapered acceleration ramps operate best under free-flow or lightly congested traffic conditions.

RESEARCH FINDINGS ON TAPERED ENTRANCE RAMPS (Ref. 2)

- Tapered entrance ramps function best under free-flow or lightly congested traffic conditions.
- A 50:1 taper design led to drivers using a greater portion of the ramp than a parallel design of the same length.
- A tapered ramp led to a more effective process allowing an increase in the ability to determine an acceptable gap.
- Behavior studies in Nebraska indicated that drivers took a tapered merging path on parallel ramps when available gaps in right-lane through traffic were available.
- Tapered ramps are preferred on roadways designed for a design speed of 65 mph or greater.

The above indicates using tapered entrance ramps in rural locations where traffic is light and speeds are high.
PARALLEL ENTRANCE TERMINAL

The FHWA, in Ref. (1), states that there are safety and operational benefits of long acceleration lengths provided by the parallel type entrance ramp and recommends their use in new interchanges and for reconstruction or reconfiguring existing interchanges. A length of 1200-feet is desirable for parallel entrance ramps, (longer if there is a plus-grade, greater than 2-percent). The acceleration lane should end with a taper 300-feet long.

FHWA reports that studies have shown that parallel entrance ramps are safer than the tapered type.

The majority of State D.O.T.’s use the parallel design for entrance terminals. The preferred approach curve to the parallel acceleration lane should have a curve radius of 1000-feet and a length of at least 200-feet. When a smaller radius curve is used, drivers tend to go directly onto the mainline and not use the acceleration lane. This could create an undesirable merging situation.

A parallel entrance terminal should be considered when: (Ref. 2)

- Under moderate to heavy traffic conditions, either due to peak-hour traffic or the potential for frequent traffic delays. In this condition merging into a gap is the primary purpose, rather than acceleration, since through speeds would be lower due to congestion.
- Where the entrance carries a large volume of trucks. (New York D.O.T. considers 2-percent or less truck traffic to be a very low percentage. A high percentage of trucks is 10-percent or more. For Interstates and freeways a DDHV of 250 trucks also indicates a high percentage.)
- Where there is a large volume of trucks and the entrance is located on a steep up-grade.
- The mainline roadway has a design speed of 60 mph or less.
- The Highway Design Handbook for Older Drivers recommends the use parallel entrance ramps (Ref. 3).
- Parallel entrance terminals provide a longer, full lane width, in which drivers can accelerate or find an acceptable gap in the mainline traffic. During congested traffic conditions, this can be an advantage.

TAPERED EXIT TERMINAL
The majority of State DOT’s use the taper design for exit terminals. Research performed on existing exit ramps has determined that:

- The majority of drivers take a tapered path when exiting the mainline, regardless if the ramp is the tapered or parallel type.

- The tapered design tends to cause driver to use the speed change portion of the ramp properly.

- An advantage of the tapered exit, is that it encourages drivers to maintain their speed until they clear the mainline.

A tapered exit should **not** be used when:

- A portion, or all, of the taper would be on a bridge.

- The exit must be placed on an existing mainline curve to the left.

- The available deceleration length is too short.

- Where an exit must be located on an existing mainline, beyond the crest of a vertical curve.
PARALLEL EXIT TERMINAL

Parallel exit terminals provide a better delineation of an exit location. This can prevent the accidental use of the ramp by drivers who want to remain on the mainline.

Parallel exit terminals should be used when:

- The exit ramp departs from a curve on the mainline.
- When a portion, or all, of the taper would be on a bridge.
- When the available deceleration length is insufficient.
- Where the exit must be located beyond the crest of a vertical curve.
- If the ramp is subject to heavy traffic conditions, such as occurs during peak hours.
- Parallel ramps should be considered when the mainline speed is 60 mph or less.
- If the capacity of the ramp intersection is not sufficient, and the ramp traffic backs up on the mainline.
TWO LANE RAMPS

Two Lane Entrance Ramp

Figure A-1, on page 816, of the 2004 Greenbook depicts a tapered two lane entrance. The left lane of the ramp is merged into the mainline, before the right lane is merged. In reference (16) this type of merging situation is discouraged, this inside merge does not meet driver exceptions, because lanes are merged or dropped on the outside or right and not on the left. For a two lane entrance the parallel type would operate better.

Two Lane Exit Ramp

An auxiliary lane should be added to the mainline, in advance of a two lane exit. Tapered exits with adequate deceleration lengths are preferred. If required, an auxiliary lane of the appropriate length and with the required advance signing should be used.
FIGURE 1 – TYPICAL GORE

1. SEE 2004 AASHTO GREENBOOK, P.614, OR STATE STANDARDS FOR DIMENSION. (ONLY USED AT EXIT TERMINALS)

2. SEE STATE STANDARDS FOR DIMENSION.
DIRECTIONAL RAMPS

Two types of directional ramps (Figures 2 and 3) are used to connect the crossing of two high speed highways, Direct Connection and Semi-Direct connection. Connection ramps are designed for higher speeds, usually with a 50 mph design speed.

Direct connection ramps exit on the left side of the high speed highway and enter the crossing high speed highway on either the left side or right side. These ramps are normally two lanes and therefore the left side exit should be designed as a major fork and the entrance as a branch connection.

Semi-direct connection ramps exit on the right side and enter the crossing mainline on the right side or left side. They are also normally two lanes wide. The two lane exit is usually developed using an auxiliary lane. The entrance, depending on volume, can be a typical two lane entrance. Texas Transportation Research Institute, Evaluation of Vehicle Speeds on Freeway to Freeway Connector Ramps, 2002. (Ref. 6), states that:

- Provide adequate deceleration and acceleration distances for tractor-trailers and other heavy vehicles. Designers should consider increasing the AASHTO lengths by 30 to 50 percent. This lengthening would assist truck drivers in exiting and entering the mainline traffic.

Since all directional ramps require at least a one level grade separation, the distance required to effect the separation can be found using Exhibit 10-8, on page 768, of the 2004 edition of the Geometric Design of Highways and Streets, (The AASHTO Greenbook).
The following can be used to determine a rough estimate of the bridge structure depth, which would be added to the required vertical clearance which yields the “Profile Rise” to be used in Exhibit 10-8.

### CONCRETE GIRDERS

<table>
<thead>
<tr>
<th>MAXIMUM SPAN LENGTH</th>
<th>GIRDER DEPTH</th>
</tr>
</thead>
<tbody>
<tr>
<td>40’ – 60’</td>
<td>3’ 0”</td>
</tr>
<tr>
<td>55’ – 80’</td>
<td>3’ 9”</td>
</tr>
<tr>
<td>70’ – 100’</td>
<td>4’ 6”</td>
</tr>
<tr>
<td>90’ – 120’</td>
<td>5’ 3”</td>
</tr>
<tr>
<td>110’ – 140’</td>
<td>6’ 0”</td>
</tr>
</tbody>
</table>

For Bridge Spans greater than 140’ and depths greater than 6’ 0” use:

\[
\text{Span} = \frac{\text{Girder Depth}}{25}
\]

Girder Depth plus 2” for Haunches and an 8” Slab Thickness would be the total Structure Depth
FIGURE 2 — DIRECT CONNECTION RAMP
FIGURE 3 – SEMI–DIRECT CONNECTION RAMP
LOOP RAMPS

There are two types of loop ramps.

A. The normal loop ramp used to exit a high speed facility and intersects at a crossroad is shown on Figure A. This type of loop is sometimes referred to as a controlled loop. Traffic control devices will be used at the intersection with the crossroad.

B. A free flow loop can be used to exit a high speed highway and join a high speed highway or an arterial highway.

Loop ramps can be designed with one circular arc (Figure C) or composed of two or three compounded arcs (Figure B). The three arc loop is sometimes referred to as a “tear drop”. The sequence of curves in a tear drop should be a flat curve followed by a sharper curve and the third curve should also be flat (flat/sharp/flat). The first and third curves could be spiral curves. **A sharp/flat/sharp, curve sequence should not be used.** This type of arc arrangement will mislead the driver to thinking the low advisory speed was only warranted by the first curve (sharp), the driver will therefore begin accelerating in order to merge at the end of the ramp. When the vehicle arrives at the third curve (sharp), there could be loss of control, skidding, and in the case of a truck, a rollover could occur.
**FIGURE A — CONTROLLED LOOP**

**FIGURE B — FREE FLOW LOOP**
*(THREE COMPOUNDED CURVES)*

**FIGURE C — FREE FLOW LOOP**
TWO-LANE LOOP RAMPS

The Georgia Department of Transportation performed the initial research on two-lane loop ramps in 1988 (Ref 7). Currently the Transportation Research Board is soliciting proposal for research for two-lane loops. In metro Atlanta two-lane loop ramps have proven to be safe and efficient. Georgia DOT constructed two-lane loops at two locations in 1988.

1. Georgia 400 exit to Holcomb Bridge Road. This is a suburban location, has 2,000 vehicles in the P.M. peak. The ADT at this location is over 14,000. The total paved width including shoulders is 38’-6”, the outside shoulder is 10’-8” wide and the inside shoulder is 4’-4” wide and the travel way width is 23’-6”.

2. The I-75 Northbound exit to I-285 Westbound, is an interstate to interstate connection. The volume on this ramp is over 1,000 vehicles during the P.M. peak with 18-percent trucks. The ADT at this location is 15,500 with 19-percent being trucks. The total paved width is 47’-10”, with an inside paved shoulder of 7’-0”, an outside shoulder of 4’-0” and a travelway width of 36’-10”. This extra wide travelway width was used because of the high truck volumes.

For a two-lane loop ramp that would service passenger cars and an insignificant amount of trucks the travelway width could be as shown in Exhibit 3-51, Page 220, of the 2004 AASHTO Greenbook.

If there is a high truck ADT or if this two-lane loop was connecting two freeways, the extra wide lanes used in No. 2 above, may be the best solution.

The two-lane loop connecting I-75 to I-285, is an economical substitute for a direct or semi-direct ramp.

Ample distance must be provided to correctly design the exit. An additional lane of from 1,300 feet to 1,500 feet should be used. Consult the State’s Highway Design Manuel for specific requirements for a two-lane exit. The Indiana DOT uses a total length from the beginning of the first taper to the gore nose of 2500 feet for volumes of 1500 vph and 3500-feet for turning volumes of 3000 vph.

The I-75 to I-285 ramp has been used as an economical replacement for Direct or Semi-Direct Connection ramp.
LEFT HAND RAMPS

Left hand exit and entrance ramps are considered less desirable, because they do not meet driver expectancy. With the preponderance of right-hand exits and entrances, left side “moves” tend to confuse and surprise drivers, even with the proper signing.

Some reasons for avoiding the use of left hand moves are:

- Decisions and maneuvering take place in the high-speed lanes.
- Trucks, which may be restricted to the right side mainline lanes, are forced to cross several traffic lanes to reach the left hand exit, or to return to the right side lane from a left hand entrance.
- There is a reduced visibility for drivers when they are forced to merge to their right, from a left hand entrance. This problem is magnified when the entering vehicle is a truck.

Since left hand ramps are less desirable, published design parameters are not available.

If an existing left hand ramp cannot be removed, the following can be used to help mitigate this condition.

- Provide decision sight distance at left hand exits
- Provide additional advance signing.
- Provide an auxiliary lane in advance of left side exits and beyond left side entrances.
- Provide at least 75-percent of the mainline design speed for the ramp design speed.
HEAVY TRUCK CONSIDERATIONS

TRUCK ROLLOVER

It is possible for some types of heavy trucks, with certain loading conditions to rollover while traveling at speeds below the design speed of the curve.

When a vehicle travels along a curved roadway or makes a turn, centrifugal and centripetal forces act on it. The amount of the centrifugal force, or lateral acceleration is generated by the vehicle’s speed and degree of curvature (sharpness). The centripetal force, which is also a function of the vehicle speed and sharpness of the curve that must counteract the centrifugal force comes from the friction developed between the vehicle’s tires and the roadway pavement. The superelevation of the roadway provides an additional retarding effect on the lateral acceleration, and therefore lessens the side friction demand. Both forces act through the center of gravity of the vehicle.

The “Simplified Curve” equation from Ref. (8), which is used to determine the side friction factor is:

\[
f = \frac{V^2}{15R} - e
\]  

\( \text{f} = \) Side friction factor (g’s)  
\( V = \) Design speed (mph)  
\( R = \) Radius (ft.)  
\( e = \) Superelevation (ft./ft.)

In highway design the side friction factor is considered to be equivalent to the lateral acceleration. The angular difference, created by the superelevation, between the lines of action of the two forces acting on the vehicle is small, and therefore is not considered relevant in highway curve design.

The equation to determine the lateral acceleration which is equivalent to the “simplified curve” equation is:

\[
a = \frac{V^2}{15R} - e
\]  

\( a = \) Lateral acceleration (g’s)
a=Lateral acceleration (g’s)

To convert equations (1) and (2) to the metric system, replace the 15 with 127, meters for feet and km/h for mph.

Side friction factors values used in highway design are based on providing a feeling of comfort to the driver and occupants of passenger cars. These values may not be satisfactory for heavy trucks, especially at low design speeds.

The center of gravity of a loaded heavy truck is much higher than that of a passenger car. The centrifugal force acting on a truck produces an overturning moment before the truck would begin to skid. The opposite is true of passenger cars, they will skid long before they overturn.

SunCam Course No. 170, “Horizontal Curve Design to Prevent the Rollover of Heavy Trucks”, provides a comprehensive treatment of heavy truck rollover.
ROLOVER THRESHOLD

The University of Michigan Transportation Research Institute, (UMTRI) has been researching how highway geometry relates to the characteristics of heavy trucks, since 1970. Their research has yielded peak values of lateral acceleration a heavy truck can withstand. Values greater than these peak values will cause the truck to rollover.

The equation developed by UMTRI to determine this peak value is:

\[ a = \frac{RT - SM}{1.15} \]  

(3)

RT = The rollover threshold value for a specific truck and loading condition.
SM = UMTRI arbitrarily set the Safety Margin at 0.10 g’s, which would cover the contingency of a heavy truck going 40 mph on a curve with a design speed of 30 mph.
1.15 provides a tolerance corresponding to the level of steering fluctuations, which have been measured in tests of the normal driving of a tractor trailer through interchange ramp curves.

Since lateral acceleration is equivalent to side friction factor, the value of “a” determined in equation (3) should be substituted for “f” in equation (1).

SUPERELEVATION TRANSITION METHODS

For superelevation, most State D.O.T.s use the two-thirds/one-third rule, that is, two-thirds of the transition from the normal cross slope (after any crown has been removed) to full superelevation, is placed on the tangent prior to the beginning of the curve (PC) and the remaining one-third is accomplished on the curve. There are some D.O.T.s that use 80-percent of the transition on the tangent and 20-percent on the curve. If a spiral curve is used, the full superelevation will be at the beginning of the circular curve (S.C.). Placing all of the transition on the tangent, so there would be full superelevation at the P.C. is not recommended by AASHTO.

Studies of driver characteristics have shown that most drivers enter curves at speeds above the posted speed, and they begin accelerating as they approach the end of the curve. Therefore the rollover of heavy trucks should be checked at the beginning of the circular curve, the P.C. and at the end of the curve, the P.T., since these locations will have less than the full amount of the required superelevation.

ROLOVER THRESHOLD VALUES
Rollover Threshold Values for a tractor with a trailer 46-feet long (from Ref. 9), for double trailers, see page 22 of Ref. (9)

<table>
<thead>
<tr>
<th>Type of Load</th>
<th>Gross Vehicle Weight (lbs.)</th>
<th>Trailer Width (ft.)</th>
<th>Tractor Width (ft.)</th>
<th>Rollover Threshold (g’s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Medium Density</td>
<td>80,000</td>
<td>8.0</td>
<td>8.0</td>
<td>0.35</td>
</tr>
<tr>
<td>Freight (31 lbs./cu. ft.)</td>
<td>80,000</td>
<td>8.5</td>
<td>8.0</td>
<td>0.36</td>
</tr>
<tr>
<td>Low Density</td>
<td>77,000</td>
<td>8.0</td>
<td>8.0</td>
<td>0.24</td>
</tr>
<tr>
<td>Freight (16.5 lbs./cu. ft.)</td>
<td>80,000</td>
<td>8.5</td>
<td>8.0</td>
<td>0.24</td>
</tr>
</tbody>
</table>

Rollover Threshold Values from Ref. (5)

- Gasoline Tanker – 0.32 g’s
  Gross Vehicle Weight of 80,000 lbs.

- Cryogenic (circular) Tanker, - 0.26 g’s
  Gross Vehicle Weight of 80,000 lbs.

- Typical Freight Load - 0.28 g’s
  Less than Total Load
  Gross Vehicle Weight of 73,000 lbs.

- The New York D.O.T. uses a rollover threshold value of 0.36 g’s for tractor trailers with a gross vehicle weight of 80,000 lbs. or more. For high center of gravity tanker trucks they use 0.22 g’s. (Ref 10)

- The Institute of Transportation Engineers, in Ref. (11), uses the rollover threshold values from Ref. (9)

Equation (3) and the rollover threshold values from Ref. (9) were incorporated into an automatic warning system to prevent truck rollover on curved ramps, installed on the Capital Beltway in Maryland and Virginia Ref. (13).
EXAMPLE

A ramp is to be designed for a design speed of 45 mph, a superelevation rate of 0.08 ft./ft. The minimum allowable radius is 587 feet, and the selected rollover threshold is 0.28.

\[ a = \frac{RT - SM}{1.15} \]

\[ a = \frac{0.28 - 0.10}{1.15} \]

\[ a = 0.16 \]

Using the two thirds – one third method of superelevation transition, the superelevation at the P.C. would be 0.06, check the rollover speed at the P.C., and using “a” for the required side friction factor.

\[ V = \sqrt{15R(e + f)} \]

\[ V = \sqrt{15 \times 587(0.06 + 0.16)} \]

\[ V = 44 \text{ mph} \]

If 80-percent of the superelevation is placed on the tangent and 20-percent in the curve, the superelevation rate at the P.C. would be 0.07 and the rollover speed at the P.C. would be 45 mph.

Another solution would be to determine the radius required to comply with the design speed of 45 mph, with the rollover threshold of 0.28

\[ R = \frac{V^2}{15(e + f)} \]

\[ R = \frac{45^2}{15(0.06 + 0.16)} \]

\[ R = 614' \]

Using the 614 feet radius combined with the 80-percent of the superelevation transition on the tangent would yield a rollover speed of 46 mph. So, if there is available right-of-way for a radius
of 614 feet and sufficient tangent length available to use, the 80/20 transition method, that would be a better solution.

SELECTING A ROLLOVER THRESHOLD VALUE

Selecting a rollover threshold value of 0.28 would provide for a tractor trailer that is 8.5 feet wide with low density freight. It would also cover a “Typical Freight Load” and gasoline tanker trucks. This rollover threshold value would also be the most conservative. From Table 2 it can be seen that the 614 foot radius at speeds of 45 mph or less would have to be increased; only 59-feet at 30 mph and 27-feet for a design speed of 45 mph. If increasing the radius did not increase right-of-way requirements then the 0.28 value would be a good choice.

As in all highway design the driver should not be surprised, this is especially true for truck drivers, since their vehicle is more difficult to control then a passenger car.

The following highway locations where the designer should be aware of the needs of heavy trucks are:

- Exit and Entrance Ramps
- Loop Ramps
- Compound Curves
- Sharp curves at the end of steep down grades
- Reverse Curves
- Broken Back Curves
- All locations where minimum curve criteria is used, along with low design speeds
TABLE 1
ROLLOVER SPEEDS FOR SIX-PERCENT SUPERELEVATION AT THE P.C.

<table>
<thead>
<tr>
<th>Design Speed (mph)</th>
<th>Minimum Radius (ft.)</th>
<th>Super-elevation at the PC</th>
<th>Rollover Speed (mph)</th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>RT = 0.28</td>
<td>RT = 0.31</td>
<td>RT = 0.34</td>
<td>RT = 0.36</td>
</tr>
<tr>
<td>30</td>
<td>214</td>
<td>0.06</td>
<td>26</td>
<td>28</td>
<td>29</td>
<td>30</td>
</tr>
<tr>
<td>35</td>
<td>314</td>
<td>0.06</td>
<td>32</td>
<td>34</td>
<td>36</td>
<td>37</td>
</tr>
<tr>
<td>40</td>
<td>444</td>
<td>0.06</td>
<td>38</td>
<td>40</td>
<td>42</td>
<td>44</td>
</tr>
<tr>
<td>45</td>
<td>587</td>
<td>0.06</td>
<td>44</td>
<td>46</td>
<td>49</td>
<td>50</td>
</tr>
<tr>
<td>50</td>
<td>758</td>
<td>0.06</td>
<td>50</td>
<td>53</td>
<td>55</td>
<td>57</td>
</tr>
</tbody>
</table>

TABLE 2
MINIMUM RADII FOR SELECTED ROLLOVER THRESHOLD VALUES

<table>
<thead>
<tr>
<th>Design Speed (mph)</th>
<th>Super-elevation (ft./ft.)</th>
<th>Super-elevation at the PC</th>
<th>AASHTO Min. Radius (ft.)</th>
<th>Radius Required for Specific Rollover Thresholds</th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>RT = 0.28 (g’s)</td>
<td>RT = 0.31</td>
<td>RT = 0.34</td>
<td>RT = 0.36</td>
</tr>
<tr>
<td>30</td>
<td>0.08</td>
<td>0.06</td>
<td>214</td>
<td>277</td>
<td>247</td>
<td>223</td>
<td>210</td>
</tr>
<tr>
<td>35</td>
<td>0.08</td>
<td>0.06</td>
<td>314</td>
<td>376</td>
<td>336</td>
<td>304</td>
<td>286</td>
</tr>
<tr>
<td>40</td>
<td>0.08</td>
<td>0.06</td>
<td>444</td>
<td>492</td>
<td>439</td>
<td>397</td>
<td>373</td>
</tr>
<tr>
<td>45</td>
<td>0.08</td>
<td>0.06</td>
<td>587</td>
<td>622</td>
<td>556</td>
<td>502</td>
<td>472</td>
</tr>
<tr>
<td>50</td>
<td>0.08</td>
<td>0.06</td>
<td>758</td>
<td>768</td>
<td>686</td>
<td>620</td>
<td>583</td>
</tr>
</tbody>
</table>
TRUCK ACCELERATION AND DECELERATION

The more the difference in speed between the tractor-trailer merging onto a freeway and the speed of the vehicles on the freeway, the greater the chances for a collision (Ref. 17). This research shows that those large speed differentials do exist and therefore longer acceleration lane lengths are needed, so that the majority of tractor-trailer trucks can accelerate and merge into the traffic flow at speeds closer to the operating speed of the mainline.

When there is a high volume of trucks entering the mainline, where the speed limit is 65-mph, an acceleration lane length of 2,700 feet is required, just to allow an average vehicle, traveling on a level grade, to get within 10-mph of the mainline posted speed.

Designer should consider using a 30 to 50-percent increase in the acceleration and deceleration lengths as shown in the “Greenbook” to accommodate heavy trucks (Refs. 2, 5 and 6). This would better enable the truck driver when in exiting and entering mainline traffic.

The graphs on the following two pages were generated from an Excel spreadsheet, which is provided by the FHWA.
Interchange Ramp Characteristics - Selection and Design
A SunCam online continuing education course

30 to 50 mph: 180 lbs/HP @ 1% Up-Grade

30 to 50 mph: 200 lbs/HP @ 1% Up-Grade

30 to 50 mph: 220 lbs/HP @ 1% Up-Grade
Interchange Ramp Characteristics - Selection and Design
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40 to 65 mph: 180 lbs/HP @ 1% Up-Grade

40 to 65 mph: 200 lbs/HP @ 1% Up-Grade

40 to 65 mph: 220 lbs/HP @ 1% Up-Grade
DESIGN REVIEW

The Federal Highway Administration has adopted the AASHTO publication, “A Policy on Design Standards Interstate Standards”. These standards are applicable to the National Highway System, including the Interstate System. These standards also reference “A Policy on Geometric Design of Highways and Streets” (Greenbook).

The Greenbook (Chapter 10, Grade Separations and Interchanges) provides the standards that should be met for interchange ramps. Most State Departments of Transportation have developed their own standards that meet or exceed the AASHTO standards.

This Design Review will be based on the “13 Controlling Criteria” that have been established by the FHWA, which are listed below:

1. Design Speed
2. Lane Width
3. Shoulder Width
4. Bridge Width
5. Horizontal Alignment
6. Superelevation
7. Vertical Alignment
8. Grade
9. Stopping Sight Distance
10. Cross Slope
11. Vertical Clearance
12. Lateral Clearance
13. Structural Capacity
1. Design Speed

Design speeds should not be less than those shown in Exhibit 10-56, Page 826, of the AASHTO Greenbook.

The desirable design speed for ramps should be 70 to 85-percent of the mainline design speed. The minimum design speed should be at least 50-percent of the mainline.

For Directional ramps the design speed should be at least 85-percent of the mainline.

For Loop ramps the design speed can be in the low range, that is within 50-percent of the mainline design speed.

Not meeting the low range (50-percent) speeds in Exhibit 10-56 may require a design exception.

The correct deceleration length is dependent on the design speed of the mainline and the first controlling factor on the exit ramp, which could be a horizontal curve, stopping sight distance of a vertical curve or the end of a traffic queue.

2. Lane Width

All State DOT’s have their preferred ramp travelway width. The width may have to be increased along a curve to provide for passing a stalled vehicle, or to provide horizontal stopping sight distance.

The range in ramp widths for a one lane ramp, per the AASHTO Greenbook, for both rural and urban locations, is 12 to 30-feet. A formal design exception is required if this width is not provided.
3. **Shoulder Width**

States have their specific shoulder widths for all ramp types. The minimum shoulder width per the AASHTO Greenbook is from 1 to 10-feet.

On ramps, the widest widths, per State standards should always be considered first. Shoulders provide important functions, a place for disabled vehicles, a place for police enforcement activities, maintenance activities, where drivers can move around crash sites, a clear recovery area, and shoulders can be widened beyond minimums to improve the stopping sight distances along horizontal curves.

The State in which the project is located, has their required shoulder widths. The width of the inside shoulder may have to be increased to provide horizontal stopping sight distance. Check the State’s requirements for any increase in shoulder width when concrete barriers or guardrail are used, generally the shoulder width is increased 2-feet, when this occurs.

4. **Bridge Width**

The width of the ramp travelway plus the width of the two paved shoulders should be carried across the bridge. State D.O.T.’s have their specific requirements. A design exception is required when a ramp bridge is proposed with narrower travel lanes and shoulders.

5. **Horizontal Alignment**

Horizontal alignment refers only to the horizontal curvature of the ramp. The Design Speed determines the minimum radius, the maximum superelevation and the stopping sight distance.

The ramp alignment before a curve influences the approach speed. The crash frequency increases as the speed differential from the approach on a tangent section to the curve. This may happen if the approach is on a downgrade or if the curve is not apparent to the driver on the approach. At ramps, especially loop ramps, if there is insufficient deceleration length can cause vehicles to run off the road and heavy trucks to overturn. Research has confirmed that some heavy trucks can overturn at speeds slightly below the design speed.

When compound curves are used the ratio of the larger radius to the smaller radius should not exceed 2:1.
The inside shoulder width or lane width may have to be increased to provide the required stopping sight distance along a curve. This should be checked where guardrail or concrete barrier is located on the inside of a ramp curve. The tangent distance between reverse curves must be long enough to provide for the correct superelevation transition.

6. Superelevation

State D.O.T. criteria controls the maximum rate of superelevation to be used. Superelevation is used to counteract the lateral acceleration, caused by the vehicle traveling along the curve. A design exception is required if the State’s superelevation standards are not met. If the designer selects a maximum rate that is higher than the State’s maximum rate, a design exception is still required.

Prior to the publication of the 2001 Greenbook, rates of superelevation used for ramps could be based on the “rates for turning roadways”. This provided the designer with a range in superelevation rates that were useable for ramp design. This no longer applies; ramp superelevation rates are now the same as those for all highways, that are “open road conditions”.

In the 1990 Greenbook, rates for ramp superelevation are taken from Table 1X-12. If the radius of the ramp curve is 600-feet and the design speed is 40-mph, the superelevation rate for the ramp could range from 7-percent to 9-percent, regardless of the State’s maximum superelevation rate. The designer may find when reconstructing an existing interchange, some substandard superelevation will exist.
7. Vertical Alignment

Vertical Alignment relates only to the crest and sag vertical curves. The “K” values used to determine the lengths of the vertical curves are based on grade and design speed, which are covered separately.

The vertical alignment should provide Decision Sight Distance to exit ramps. Also the “breakover” between the mainline and the ramp travelway should comply with the project’s criteria. This criteria must comply with the maximums and minimums stated in the Greenbook.

8. Grade

The State D.O.T.’s criteria for maximum and minimum grades must be adhered to. These grades comply with the AASHTO criteria, and therefore if they are not complied with, a Design Exception will be required.

A safety concern is that drivers of tractor-trailer trucks can lose control on steep down-grades. This problem is compounded when there is a horizontal curve at the end of the steep down-grade. On steep up-grades, slow moving trucks create a safety and operations problem.

9. Stopping Sight Distance

Stopping Sight Distance must be provided at all horizontal and vertical curves. If the ramp is lighted, the length of sag vertical curve can be based on the “comfort” equation (see page 274 of the 2004 AASHTO Greenbook). The curve lengths using the “comfort” equation result in sag vertical curves being about one-half of the minimum curve length of those based on headlight criteria.

On horizontal curves, where guardrail and concrete barriers are required on the inside of the curve, the lane width and/or shoulder width may have to be increased to provide the required stopping sight distance. Also stopping sight distance should be checked where retaining walls, bridge piers, cut-slopes, etc. may obstruct sight distance.
Decision Sight Distance (page 116, 2004 AASTO Greenbook) should be used at exit ramps and lane drops. When stopping sight distance is not provided at all locations along the ramp, a Design Exception is required. If lighting is provided vertical curves meeting the “comfort” criteria do not require a Design Exception. A Design Exception is not required if Decision Sight Distance is not provided.

10. Cross Slope

Ramp travelway cross slopes should be a minimum of 1.5-percent and desirably 2-percent. In States with intense rainfall the cross slope could be increased to 2.5-percent. The cross slopes on paved shoulders should be from 2 to 6-percent, but should never be less than the cross slope of the travelway.

The cross slope break-over between the ramp travelway and the shoulder, on the high side of superelevation, should be less than 8-percent. If this break over exceeds 8-percent a Design Exception is required. If the design vehicle is a WB-60 or if there is a substantial amount of heavy truck traffic, consideration should be given to crowning the shoulder or sloping the shoulder in the same direction and rate as the superelevated travelway.

11. Vertical Clearance

Vertical Clearance applies to the ramp travelway and shoulders.

The minimum vertical clearance from the highest point on the ramp are: to the bottom of a bridge 16-feet, to the bottom of a sign truss and pedestrian bridge, 17-feet. Additionally, an allowance of at least 0.5-feet should be added for future resurfacing. The minimum vertical clearance to falsework is 14.4-feet (Ref. 17)

A Design Exception is required if these clearance dimensions cannot be met.

12. Horizontal Clearance

Horizontal clearance is defined as the lateral offset to an obstruction. Lateral offset is not the Clear Zone, which is the clear recovery area, free of obstacles and steep slopes, that allows vehicles to safely recover after they have run off the roadway. Lateral offset to obstructions is one of the 13 controlling criteria, Clear Zone is not.
The lateral offset to obstructions is generally not applicable to interchange ramps, since there will be an established Clear Zone for the ramp, and any obstructions within the Clear Zone will be shielded. There may be above ground utilities at the intersection of the ramp and the cross road, and the lateral offset distances would apply.

13. Structural Capacity

Structural Capacity is not part of this Design Review.

Reference (1) and Reference 14 provide a very comprehensive review of design requirements.
### Glossary

<table>
<thead>
<tr>
<th>Abbreviation</th>
<th>Description</th>
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<tbody>
<tr>
<td>DDHV</td>
<td>Directional Design Hourly Volume. The traffic volume in the peak direction of flow, during the design hour.</td>
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<tr>
<td>DHV</td>
<td>Design Hourly Volume. The one hour volume in the design year selected for determining dimensions and configurations of a highway’s design elements.</td>
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<tr>
<td>AADT</td>
<td>Annual Average Daily Traffic. The total yearly volume of traffic divided by the number of days in a year.</td>
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<tr>
<td>ADT</td>
<td>Average Daily Traffic. The average traffic volume in a time period greater than one day and less than one year, divided by the number days in that time period.</td>
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<tr>
<td>85th Percentile Speed</td>
<td>A speed value that is exceeded by 15-percent of the vehicles in a traffic stream.</td>
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<tr>
<td>Free Flow</td>
<td>A flow of traffic unaffected by upstream or downstream conditions.</td>
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<tr>
<td>Falsework</td>
<td>Temporary work that may include forms, shoring, form travelers, temporary bridges, etc.</td>
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REFERENCES

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