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A METHOD TO EVALUATE AND IMPROVE THE OPERATIONAL SAFETY OF
TRUCKS ON HIGHWAY RAMPS

by

BEATRICE ISAACS

A dissertation submitted to the Graduate Faculty in
Engineering in partial fulfillment of the requirements for
the degree of Doctor of Philosophy, The City University of
New York

1995

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This manuscript has been read and accepted for the Graduate Faculty in Engineering in satisfaction of the dissertation requirement for the degree of Doctor of Philosophy.

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Abstract

A METHOD TO EVALUATE AND IMPROVE THE OPERATIONAL SAFETY OF
TRUCKS ON HIGHWAY RAMPS

by

Beatrice Isaacs

Adviser: Professor Claire E. McKnight

The motor carrier industry is vital to the United States economy. At the same time however, this country pays a high price in deaths, injuries, property damage, congestion, and pollution that result from truck accidents. While vehicle and driver deficiencies are major factors in truck accidents, a basic cause is that trucks and the highways on which they operate are not compatible.

The sharp curvature and downgrades on many highway ramps make them likely sites for truck rollovers. The hazard is exacerbated by the AASHTO guidelines to which the ramps are built, in that they are deficient for safe operation by trucks.

Every highway operating agency in the United States has safety enhancement programs in place. Typically, however, the effectiveness of these programs is diminished by

reliance on incomplete and misleading accident data to identify sites and priorities for improvement.

A proactive approach to identification and prioritization of sites needing safety improvement is developed in this dissertation. It is based on a survey in which experts rate the relative extent to which deficiencies in each of the characteristics of a ramp might contribute to a truck rollover. The experts' ratings are derived from their knowledge of physics, highway design, and truck operations. There is no reliance on accident data, making this approach truly proactive.

Fuzzy mathematics is used in the synthesis of the survey results, in recognition of the fact that an opinion, however expert, is not precise. The final product of the survey is a set of ratings which describe the relative hazard of specific ramp and deceleration lane characteristics. These ratings can be used to rank all ramps in a jurisdiction in order of relative hazard. An example of the use of the ratings beyond a simple ranking is demonstrated in a cost-effectiveness analysis, where the ratings are used as a measure of effectiveness.

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A sincere thank you is extended to Dick Dalphin, Don Leone, Saleh Keshawarz and Michael Nowak because they are

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This work is dedicated to Prof. G. Donald Brandt, for opening the door, and to Ken Bobetsky, for pushing me off the fence.

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Chapter 1

INTRODUCTION

A truck rollover is a costly event. A rollover can result in personal injuries or fatalities; property damage to vehicles, cargo, the roadway and roadside; traffic congestion with resulting air pollution; and secondary effects such as economic disruption. The curvature of highway ramps and the large differential in posted speeds between ramps and highways make ramps prime candidates for rollover sites.

Every state in the United States and most urban jurisdictions have highway safety programs in place. In most of these programs, however, the focus for future safety initiatives is based on flawed data from past accidents. This diminishes the effectiveness of highway safety efforts.

This dissertation presents the development of a method to identify and evaluate ramp characteristics that present a rollover hazard for trucks. It is based on expert opinion, evolved from knowledge of physics, truck operations and highway geometry, and avoids reliance on accident data. Agencies responsible for highway safety enhancement can use the hazard evaluations in planning and prioritizing future projects, preferably in cost-effectiveness analysis. A cost-effectiveness analysis "...systematically examines the cost

and effectiveness (by using some dimensional measures) of alternative methods for accomplishing an objective." (Glennon & Harwood 1978). It allows consideration of costs and benefits that cannot be measured in dollars, and does not force the analyst to make arbitrary dollar assignments to factors. This approach ensures that "...in the selection of a preferred system, the greatest return is obtained for the capital invested." (Wattleworth 1972).

1.1 Trucks on U.S. Highways

The motor carrier industry is a vital part of the American economy. In 1990, trucks carried 735 billion ton-miles of freight. This was 25.6 percent of the freight shipped in the United States, up from 21.7 percent in 1981 (Association of American Railroads 1991).

The motor carrier share of the freight industry is likely to continue to increase. One reason for this is that shippers are increasingly attracted by the cost-savings and convenience of intermodal shipping. Tractor-trailers, which can provide "door-to-door" service, are ideal collectors and distributors of containers and trailers for which the line-haul portion of the shipment has been done by rail or water.

An even more important factor has to do with the changing configuration, and resultant enhanced productivity, of tractor-trailers since the enactment of the Surface Transportation Assistance Act (STAA) of 1982. STAA is a

federal law which preempted the varying restrictive limits that many states had set on truck sizes. It forced states to allow longer, wider and heavier trucks on a designated National Network which consists of almost all of the 44,000 mile Interstate system, 139,000 miles of other Primary system roads, and access roads for passage between highways, terminals and service facilities (TRB 1989). States were also required to allow operation of twin trailer trucks (one tractor pulling two 28-foot trailers) and were prohibited from setting maximum overall limits on tractor-trailer length (TRB 1989). Many states have gone beyond STAA requirements in liberalizing tractor-trailer size limits. Doubles, which are single tractors pulling two trailers of any length, have become commonplace. Triples, single tractors pulling three trailers, are allowed in some states, and 53-foot long trailers, five feet longer than the minimum limit set by STAA, can be seen with increasing frequency.

Although safety was specified in the STAA as the primary criterion in designating the National Network, the presence of these STAA-allowed vehicles on roads that were designed for smaller vehicles has raised serious safety concerns.

The 1980 Motor Carrier Act, which deregulated the trucking industry, also had the unintended effect of exacerbating the truck-related safety deficiencies of our highways. Since deregulation, the economic pressures created by increased competition have led to many instances of trucks

being operated at excessive rates of speed, truck drivers violating hours of service regulations, and poor truck maintenance. As a result, a truck driver's ability to compensate for a deficient roadway is diminished.

All these factors contribute to making the highway system less safe than the American public would like.

1.2 AASHTO Inadequacies

Guidelines for the design of the basic geometric characteristics of a highway are developed and published by the American Association of State Highway and Transportation Officials (AASHTO). The guidelines are based on the requirements of "design vehicles", vehicles designated by AASHTO as being representative of the various sizes and classes of vehicles using the highways. Much of the incompatibility between trucks and roads today comes from the fact that the AASHTO guidelines that were in effect when most of the Interstate system was built, from the 1950s through the 1970s, were based on design vehicles that were considerably smaller than STAA vehicles and had less stringent design requirements.

However, the disparity between STAA vehicles and AASHTO design vehicles is rooted in an even more basic problem. Results of a growing body of research conclude that AASHTO guidelines are deficient even for the design vehicles on which they are supposed to be based. AASHTO design criteria

that have been noted as being particularly deficient regarding truck operations are passing sight distance guidelines (Waller 1982), at-grade intersections with minor roads (Fambro et al. 1988), and highway exit ramps (Ervin, MacAdam and Barnes 1986).

Ervin's 1986 study of exit ramps investigated the influence of ramp design on the stability and control of heavy trucks, and found that the ramp design guidelines published by AASHTO in 1984 and earlier make little or no allowance for the special requirements of trucks. The study noted that AASHTO policy on side friction factor is determined by the effects on automobiles of such factors as driver discomfort, smooth tires, poor pavement texture and wet weather. While these factors are also relevant to trucks, the primary truck factor--low rollover threshold, is ignored. AASHTO design speed recommendations for highway exit ramps range from 10 to 35 miles per hour less than the design speed of the highway itself (AASHTO 1990). This requires a significant speed reduction if exiting vehicles are to negotiate the ramp safely. Trucks need longer deceleration distances than passenger cars do, but recommended deceleration lane lengths (AASHTO 1990) are based on the operating characteristics of passenger cars. The truck-related deficiencies in the 1984 and earlier guidelines were not corrected in the most recent AASHTO Policy published in 1990.

Ramps are a critical part of the highway system. As a ramp is typically no more than two lanes wide, a truck rollover is likely to block it completely, and a blocked ramp can cause massive congestion and disruption of the highway system for miles around.

1.3 Approaches to Correction

Many methods have been developed for the purpose of identifying and prioritizing highway locations in need of safety improvement. Some have been put into practice; others exist only in theory. The methods vary widely. Some use mathematical and statistical techniques to develop predictive models. Examples are an encroachment probability model for use in roadside safety enhancement (Sicking and Ross 1986) and a Bayesian model which uses accident history and traffic volume to identify hazardous location (Higle and Witkowski 1988). Other methods use accident history and observation to identify hazardous sites, and then base safety programs on the expected economic benefits if the hazard were removed (Hunter et al. 1978). A newer approach is to compare the geometric and operational characteristics of highway segments with the characteristics of segments that are known to be hazardous. This is the basis for a Road Safety Audit, an accident prevention strategy that was developed about five years ago and is being adopted in a growing number of countries (ITE 1995).

Many highway safety methodologies take a proactive approach, while others are reactive, but almost all rely on accident data. Some safety programs, typically in non-freeway urban systems, rely entirely on past accident frequency to identify sites needing correction. An example is the method developed for the New York City Department of Transportation (NYCDOT) to prioritize problem intersections in Brooklyn (Falcocchio et al. 1994). Methods that focus on safety improvement of freeway segments generally go beyond simple consideration of numbers of accidents and incorporate operational and geometric characteristics into the accident history database.

1.4 Deficiencies in Data

Reliance on accident data in developing remediation programs and methodologies is troublesome. One issue is that of striking a reasonable balance between a reactive and proactive approach. Many localities take an extreme reactive approach, in which nothing is done until a sufficient number of accidents have occurred. This does not seem to be in the best public interest. Neither, however, is an approach at the opposite extreme which ignores accident history. At present, there is no generally accepted philosophy regarding the appropriate level of reliance on accident data, and the issue is resolved case by case depending on the inclinations of those developing the particular safety enhancement

program.

An equally significant issue is that of data reliability. Reliance on accident data can skew the focus and evaluation of a remediation program if the data itself is skewed, and there is considerable evidence that most highway accident databases are neither complete nor accurate. A study prepared for the Office of Technology Assessment (OTA) of the United States Congress found that "...with few exceptions, existing information systems have deficiencies that limit their value in supporting safety policies and programs." (United States Congress 1988,173). The existing databases that cover truck accidents are built from two sources: accident information reported to the government by the carriers themselves and police accident reports. Stein's 1993 study evaluated the sufficiency of truck accident data reported by motor carriers to the federal Office of Motor Carriers (OMC) by comparing it with data from an independent truck safety study conducted by the Insurance Institute for Highway Safety. Results showed that only 39 percent of truck crashes which met OMC reportability criteria had actually been reported. These findings corroborated those of earlier studies (Stein 1993). When the OMC data is used for in-depth studies of accident characteristics and causative factors, as has been the case, the deficiencies in the data constitute significant biases that may cause these studies to be invalid. Past studies that used OMC data to estimate the

frequency of crash characteristics, such as truck overturn or jackknife, were found to have significantly underestimated the occurrence of these events (Stein 1993).

Stein's study focused on underreporting of truck accidents, but the problem is broader than that. Even when accidents are reported, the reports are made on forms which are not adequate for safety analysis, with detail being sacrificed to avoid excessive length. Reports made by police officers at crash sites are no less deficient than those made by carriers. Accidents are usually investigated by police officers who have little training in accident causation. The reported cause of an accident, therefore, will usually be the last, most obvious, event in a chain of interactions between the driver, vehicle and roadway, but not necessarily the actual cause. More often than not, the completed forms will be illegible, incomplete and inaccurate.

While the problem of inadequate data applies to all aspects of the highway system, the OTA study found that "...truck safety information systems lag considerably behind their modal counterparts in coverage and accuracy." (United States Congress 1988).

There are many federal and state initiatives aimed at improving data quality. However, even if data were more accurate, a basic change in the manner in which it is used would be required to produce optimal results, particularly regarding trucks. When accident data forms the basis for a

highway safety program, the typical approach is for remedial action to be triggered by numbers of accidents, rather than severity. This ignores the fact that a single severe accident, e.g. a truck rollover with injuries and cargo spill, may be far more costly to society than a hundred fender-benders. Given that truck accidents tend to be infrequent but severe, current practice makes it unlikely that highway locations at which trucks are especially at risk will be flagged for corrective action.

For all the reasons mentioned above, reliance on accident data is not the best approach to highway safety improvement. A more effective use of data would be to corroborate safety evaluations which are based on theoretical principles and engineering judgment. In-depth study of locations where actual numbers of accidents conflict with predictions based on theory or expert opinion could provide new insight into accident causation.

1.5 Highway Safety Programs

It is universally agreed that highway safety is desirable. The topic becomes controversial when the question of how much to spend for what level of safety arises. Putting aside the moral and philosophical aspects of the issue, and recognizing that agencies responsible for highway safety operate within fiscal constraints imposed from outside, a reasonable goal is to make highways as safe as

possible within existing constraints. This requires a systematic, orderly approach to identifying and prioritizing safety initiatives. Such an approach is dictated by the Intermodal Surface Transportation Efficiency Act (ISTEA) of 1991. ISTEA requires states to take a "systems" approach to all work done in safety, congestion, and four other areas. Systems must be in place by the fiscal year following September 30, 1995 or States risk having 10 percent of their federal-aid funds withheld. System components should include an inventory, monitoring and data collection, impacts modeling, strategy selection with a strong emphasis on cost-effectiveness, and plans for implementing selected strategies.

Even if it were not required, an orderly approach is beneficial to the State in that it helps protect against tort liability by providing evidence that the State is making every reasonable effort to ensure highway safety. Further, improved safety increases highway capacity without incurring costs of new construction.

1.6 Contents of Dissertation

This dissertation presents the development of sufficiency ratings for highway ramps based on expert opinion of the relative level of hazard presented by specific characteristics. Use of these ratings is then demonstrated in cost-effectiveness analysis, where effectiveness is

measured by the reduction of the level of hazard on the ramp.

The effects of truck and roadway characteristics in truck rollovers, and the costs of rollovers, are discussed in Chapter 2. Chapter 3 gives an overview of the proposed methodology. Chapter 4 describes the survey of experts and development of hazard ratings, numerical values developed from expert consensus on the level of hazard presented by specific characteristics of a ramp. To acknowledge the fact that an opinion on a level of hazard is subjective and cannot be described with numerical precision, fuzzy mathematics techniques are used in the development of the hazard ratings. Chapter 5 describes corrective measures that can reduce the likelihood of a truck rollover. Cost-effectiveness is discussed in Chapter 6, and use of the hazard ratings in cost-effectiveness analysis is demonstrated in Chapter 7. Topics for further study are discussed in Chapter 8.

Chapter 2

TRUCK ROLLOVERS

2.1 Mechanics of Rollover

A vehicle traveling on a horizontal curve is subjected to a centrifugal force which pushes it outward away from the center of the curve. The magnitude of this force is mV^2/R , where m is the vehicle mass, V is the velocity and R is the radius of the curve. In order for the vehicle to stay on the road, an opposing centripetal force must be developed at the pavement-tire interface. The combination of superelevation (e) and side friction (f) allow this force to be developed.

On any given curve, the amount of superelevation to be provided is a design decision, and superelevation remains constant once the road is built. Friction, however, is not constant. It depends on the amount of friction demanded, which is a function of the speed of the vehicle, and the friction available, which varies with the surface condition of the pavement and the vehicle's tires.

The relationship between these two forces can be seen in the basic equation used for designing highway curves

$$(2.1) \quad e + f = \frac{V^2}{15 R}$$

in which both sides represent lateral acceleration. The right side represents centrifugal acceleration, the tendency of the vehicle to move to the outside of the curve, while the left side represents the opposing centripetal acceleration.

To clarify the relationship between this equation and the rollover threshold of a truck, the equation can be rewritten to show superelevation as a reduction in centrifugal acceleration

$$(2.2) \quad f = \frac{V^2}{15R} - e$$

where both sides of the equation are equal to net lateral acceleration.

The maximum lateral acceleration that any vehicle can reach without rolling over, is known as the rollover threshold. Vehicle configuration, including loading in the case of trucks, is the primary determinant of rollover threshold. Relevant variables are center-of-gravity height above the ground, track width (center-to-center lateral dimension between tires), and, to a lesser extent, suspension and tire stiffness.

Although rollover thresholds of trucks vary, the high center-of-gravity of trucks make them far more likely to overturn than passenger cars. Truck configuration is the primary determinant of center-of-gravity location, but the type of freight being transported is also a major factor. A tractor-semitrailer carrying low-density freight, where the

load is constrained by bulking out rather than by weight limits, will have a much higher center-of-gravity than a similar vehicle where the trailer can be only partially filled with high-density freight. The greater track width of the wider trucks allowed by the 1982 Surface Transportation Assistance Act gives them a higher rollover threshold than older trucks, thereby decreasing their propensity to roll over. Rollover thresholds of some typical truck configurations and loadings are shown in Fig. 1.

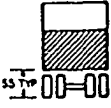
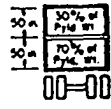
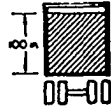


CASE	CONFIGURATION	WEIGHT	PAYLOAD	ROLLOVER
		(lbs)	CG	
		GVW	HEIGHT	(G's)
A.	 Full Gross, Medium-Density Freight (3.4 lb/ft ³) 53 in	80,000	83.5	.34
B.	 "Typical" LTL Freight Load 50% of Payload 70% of Payload 50 in	75,000	95.0	.28
C.	 Full Gross, Full Cube, Homogeneous Freight (10.7 lb/ft ³) 100 in	80,000	105.0	.24
D.	 Full Gross Gasoline Tanker 88.6 in	80,000	88.6	.32
E.	 Cryogenic Tanker (He ₂ and H ₂)	80,000	100.	.26

Fig.1 Typical Rollover Thresholds

Because of their different rollover thresholds, passenger cars and trucks are subject to different failure modes when the friction demands are excessive as the vehicle

rounds a curve. Typically, the level of friction demanded by a passenger car will be below the vehicle's rollover threshold even when it exceeds the friction available at the pavement-tire interface. Therefore, barring other factors that would induce a rollover, a passenger car will skid off the road before it turns over. A truck, however, will exceed its rollover threshold before the maximum available friction is demanded. Therefore, a truck is most likely to roll over, rather than skid off the road.

Fig. 2, adapted from a paper on the roll stability of heavy trucks (Ervin 1983), illustrates the mechanics of rollover.

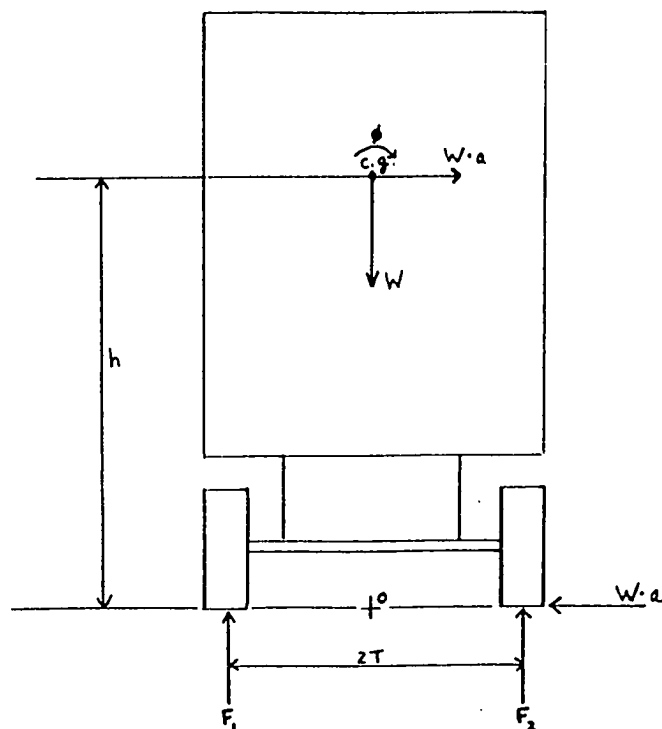


Fig. 2 Rigid Vehicle Roll Model

In this figure: W = vehicle weight
 a = steady lateral acceleration
 T = 1/2 vehicle track width
 h = height of center-of-gravity above ground
 ϕ = vehicle roll angle

As Ervin notes, three moments act on the vehicle:

$-W \cdot a \cdot h$ the primary overturning moment
 $(F_2 - F_1) \cdot T$ the "restoring" moment
 $-W \cdot h \cdot \phi$ the moment caused by the roll motion which shifts the center-of-gravity toward the outside wheels. (Assume a small roll angle, where $\tan \phi \approx \phi$).

Equilibrium is described by

$$(2.3) \quad W \cdot a \cdot h = (F_2 - F_1) \cdot T - (W \cdot h \cdot \phi)$$

If the vehicle is not in equilibrium, it is rolling over. Sensitivity of equilibrium to the center-of-gravity height and track width can be noted here, and the ratio T/h is thus another way to define a vehicle's rollover threshold.

Rollover is a special problem for multi-trailer vehicles (twins, doubles, triples), which are three to four times more likely to overturn than conventional tractor-trailers. Close to seventy percent of twin-trailer accidents involve rollover, usually of the rear trailer (United States Congress

1988). Beyond the factors which cause rollovers in conventional trucks, these vehicles experience rearward amplification, the phenomenon in which motions of the tractor are amplified in each successive trailer. At the same time, their coupling mechanisms are ineffective in transmitting roll movements from one unit to the next, preventing the driver from sensing instability of the rear trailer (TRB 1989). This combination of factors make multi-trailer trucks especially prone to rollovers.

2.2 Geometric Characteristics Relevant to Rollovers

Roadway characteristics that are factors in whether or not a truck will overturn include:

- Deceleration lane length

- Type of Transition Curve

- Ramp Characteristics, including:

- curve radius and superelevation

- type of compound curve

- presence of outside curb

- drop at pavement edge

- cross-slope difference at pavement edge

- lane width

- Pavement surface condition

- Downgrade

Location of speed advisory signs for the ramp is also a relevant factor.

2.2.1 Length of Deceleration Lane

Velocity is the most significant factor in producing the centrifugal force that acts on a vehicle as it goes around a curve. It is therefore also the primary determinant in whether or not the vehicle will encounter difficulties in safely negotiating the curve.

Because of space constraints, highway ramps usually incorporate tight (short-radius) curves which can be safely negotiated only at a speed much lower than the safe operating speed on the highway itself. Therefore, a deceleration lane must be provided on which a vehicle leaving the highway can decelerate to a speed appropriate for the curved portion of the ramp. The AASHTO design policy (AASHTO 1990) states that a "speed-change lane should, as a minimum requirement, have sufficient length to enable a driver to make the necessary change between the speed of operation on the highway and the speed on the turning roadway in a safe and comfortable manner."

The minimum length requirement for a deceleration lane is a function of speed reduction required, pavement surface condition (see 2.2.6), downgrade (see 2.2.7), braking capabilities of the vehicles using the lane, and the level of friction that can be developed at the pavement-tire interface. Recommended deceleration lengths are tabulated in the AASHTO design policy, but underlying assumptions made in calculating these lengths make them inadequate for most

trucks. The tabulated lengths are developed from passenger car braking characteristics and an assumption of available friction developed by a passenger car making a locked-wheel (skidding) stop on a wet roadway. The assumption of locked wheel deceleration leads to the assumption that all the friction that can be developed at the pavement-tire interface will be available. However, locked wheel operation is exceedingly hazardous for tractor-trailers, in that there is a high risk of loss of control of the vehicle (Harwood Vbl.II 1990, Fancher 1986, Mason and Briggs 1985, Cleveland et al. 1985). Therefore, deceleration lanes intended for use by trucks should be designed assuming a lower level of available friction. This would lead to greater length requirements.

The inadequate policy for deceleration lane design makes it likely that a truck will enter the curved portion of a ramp at an excessive speed. An example of the ensuing difficulties can be seen in one of the cases examined in a study of the impact of geometric characteristics on truck operations and safety (Ervin et al. 1986). The site, which had a short deceleration lane leading to a very short-radius curve, was the location of "frequent incidents of both jackknife (due to overbraking) and rollover (due to excessive speed in the curve)."

2.2.2 Type of Transition Curve

Superelevation, the upward sloping of the road surface toward the outside of a curve, is needed in order to keep the

developing lateral friction force from exceeding acceptable limits. A deceleration lane-to-ramp segment of a highway must include a transition between the superelevated cross-section of a curved ramp and the preceding normal crowned cross-section. The possible ways in which the transition can be achieved are through use of a spiral curve, compound curves, developing part of the superelevation on the tangent section and the remainder on the curve, and developing all the superelevation on the tangent so that the entire curve is fully superelevated.

A spiral curve, in which the degree of curvature and level of superelevation change gradually until both reach the desired maximum values, is the ideal way in which to effect the transition. The smooth transition allowed by a spiral enhances safety by reducing the likelihood that a driver will need to make sudden steering maneuvers, a particularly hazardous prospect for a tractor-trailer. Many agencies do not use spirals, justifying their policy by noting that drivers tend to provide their own spirals by steering into circular curves gradually. This, however, requires drivers to shift the placement of their vehicles within their own lanes, and sometimes into other lanes. This requirement is not safe, especially for tractor-trailers, where offtracking, the tendency of the rear wheels of the turning vehicle to follow a path inward of the front wheels at low speeds and outward of the front wheels at high speeds, may have already

caused the vehicle to encroach on adjacent lanes.

A compound curve, in which each successive segment has sharper curvature, does not provide as smooth a transition as a spiral.

In practice, the most common transition is one where some portion of the superelevation is developed on the tangent preceding the curve. AASHTO recommends that one-half to two-thirds of the maximum superelevation be developed prior to the point at which curvature begins (AASHTO 1990).

2.2.3 Curve Radius and Superelevation on Ramp

The radius of a curve and its degree of curvature (sharpness) are inversely proportional to each other. A long radius is necessary in order to have a shallow curve which can be easily traversed by a vehicle. Space constraints, however, often lead to the construction of sharply curved, short-radius ramps.

The basic equation for curve design (AASHTO 1990)

$$(2.1) \quad e + f = \frac{V^2}{15R}$$

shows the relationship between the variables that define the curve. In this equation, V is the design speed, slightly higher than operating speed in theory but frequently exceeded by operating speeds in fact. The values of radius (R) and superelevation (e) are selected by the designer based on design speed requirements. Once the ramp is built, R and e remain constant, and the level of friction (f) that develops

is thus dependent on the geometry of the ramp and the operating speed selected by a driver.

The curve equation demonstrates that even the independent variables, V , R , and e , are not truly independent of each other, and also depend on the magnitude of the level of friction that may safely be allowed to develop. This interrelationship can be seen in the AASHTO design charts (AASHTO 1990), which present these variables as functions of each other.

The variables in curve design are constrained by practical limits. R_{\max} is limited by space available. Superelevation must be limited by local climatic conditions and expected vehicle speeds. Superelevation on a ramp that is likely to become icy must not be so great as to cause a standing or slow-moving vehicle to slide across the road into the center of the curve. On dry pavements, superelevation that is excessive for slow-moving vehicles forces the driver to steer opposite to the direction of the turn in order to avoid sliding into the center. This is unnatural and confusing to a driver, and therefore diminishes safety. As described in Section 2.1, demand for friction must be limited by safety concerns and must be particularly sensitive to the requirements of trucks.

Just as AASHTO policy on friction factor is deficient for trucks on tangent sections (see 2.2.1), it is similarly deficient on curves. Recommended levels of friction are

based on the performance characteristics of passenger cars, considering the maximum side friction capacity of car tires which can be sustained without skidding on wet pavements with tread-worn tires (Ervin Vol.I 1986). The stated intention in AASHTO is to use a conservative value of friction, one which would cause a passenger car driver to feel discomfort and instinctively avoid higher speed. Ervin points out that AASHTO policy intends a substantially larger margin of safety than is, in fact, achieved with heavy trucks. He also notes that the risk of loss of control on AASHTO-recommended ramps is continual for certain trucks, rather than being temporally dependent upon vehicle and pavement maintenance and weather as it is for passenger cars. Therefore, design values of friction given in AASHTO are not appropriate for ramps used by trucks, and given the interaction between R , e , and f , curves designed according to AASHTO guidelines are not safe for trucks.

2.2.4 Type of Compound Curve on Ramp

Use of compound curves, consisting of several adjacent curved segments with different radii, is sometimes necessary in order to produce a ramp configuration that fits into the available space. With the differing radii, vehicles must operate at varying speeds in order to keep friction demands within safe levels, but only one speed advisory is usually posted for a ramp.

Considering a short radius curve as sharp and a long

radius curve as flat, possible compound curve configurations are

sharp → flat

flat → sharp

sharp → flat → sharp

flat → sharp → flat

Assuming the ramp is properly built, and the posted speed is appropriate for trucks, the sharp to flat configuration is safe. However, any configuration in which a flat curve precedes a sharper one can be exceedingly hazardous. Given the single speed advisory, a driver is likely to assume that the entire ramp has only the initial, flat, curvature, and that this is the curvature that warranted the posted speed. The probability of a vehicle entering the sharper curve at an speed is thus very high.

2.2.5 Presence of Outside Curb, Drop at Pavement Edge, Cross-Slope Difference at Pavement Edge, Lane Width

Any discontinuity in the pavement surface at the outer edge of a curved ramp can become a tripping mechanism which can cause a truck to roll over. Insufficient lane width intensifies this hazard.

A curb along the outside of a ramp curve is one form in which this type of hazard exists. Ervin's investigation of geometric features of truck accident locations (Ervin Vol.I 1986) notes four accidents where outside curbs triggered a rollover. Use of curbs to delineate the high side of the

pavement was a recommended AASHTO policy in the past (AASHTO 1965). The recommendation was dropped in 1984, but curbed ramps built to previous guidelines still exist on many roadways.

A similar risk is present with a sharp drop in elevation between the traveled lane and the shoulder, and with an excessive cross-slope difference at the pavement edge. A hazardous drop at the pavement edge can develop when vehicles repeatedly leave the traveled lane and encroach on the shoulder. Such encroachments can break up the shoulder surface, given the lesser pavement strength on the shoulder, and cause erosion and rutting (TRB 1989). A hazardous cross-slope difference exists at the high edge of a steeply superelevated lane which is adjacent to a sharply downward-sloping embankment.

All three of these hazards are exacerbated by inadequate lane width, especially when tractor-trailers are experiencing high-speed off-tracking, or the driver is steering toward the outside of the curve to compensate for an inadequate radius. The wider trucks allowed by the 1982 Surface Transportation Assistance Act, 102 inches wide versus the previous maximum of 96 inches, are especially at risk. AASHTO has long recommended that lanes be widened on horizontal curves (AASHTO 1965, 1984, 1990), but this is often precluded by physical constraints on ramps.

2.2.6 Pavement Surface Condition

Consideration must be given in highway design to the probability of a pavement surface being wet, snow-covered or icy, as these conditions prevent vehicles from developing the level of traction that would be available on a dry surface. On a deceleration lane, pavement surface condition is a major factor in determining whether a vehicle will be able to decelerate to a speed appropriate for the ramp. AASHTO design guidelines are based mainly on passenger car operating capability on wet pavements, and frequently result in highway segments that are barely adequate for the safe operation of a truck. Empty or lightly-loaded trucks are at greatest risk, as they are unable to adequately expel water at the tire-pavement footprint or develop sufficient traction for controlled operation (Ervin Vol.I 1986). For all vehicles, wet and snowy conditions are more severe than dry, and icy is extremely hazardous.

2.2.7 Downgrade

A downgrade on a deceleration lane and/or ramp can be hazardous if the increased braking demands exceed a vehicle's braking capability. An unintentional increase in velocity if a vehicle coasts downhill is another possible danger, particularly if the downgrade is followed by a sharp curve.

Trucks are at special risk on downgrades, regardless of whether the drivers are braking or allowing the vehicles to coast. A truck's ability to brake is less than that of a

passenger car, and trucks coasting downhill exhibit only about half the level of drag forces, per unit weight, produced by cars (Ervin Vol.I 1986). The smaller drag forces mean that trucks experience a much lower level of natural deceleration while coasting than do passenger cars. The likelihood of a truck traveling at an excessive speed at the end of a downgrade is therefore high.

The most recent AASHTO policy (AASHTO 1990) states that "...gradients up to 8 percent do not cause a hazard due to excessive acceleration." This contradicts an older policy (AASHTO 1965) which recommended grade limitations of 3 to 4 percent for sites having a high proportion of truck or bus traffic.

2.3 AASHTO Inadequacy

Highway design in the United States is based on guidelines published by AASHTO. The first AASHTO policy was published in 1940, and periodic updates have followed. The most recent policy was published in 1990. The AASHTO policies cover the design of the geometric characteristics of a highway, while specifications for sign placement, striping and other traffic control devices are given in the Manual on Uniform Traffic Control Devices (MUTCD).

AASHTO makes use of "design vehicles" in order to develop charts and tables of recommended numerical values for the geometric features of a highway. The stated intention is

for these design vehicles to be representative of the most demanding vehicles in the basic classifications, passenger cars, trucks, and buses/RVs, thus producing a highway which can safely accommodate all vehicles. However, the changing nature of the vehicle population, as opposed to the unchanging nature of a roadway, leads to a dilemma. At any given time, a highway will be designed using guidelines that were developed for vehicles of an earlier time. These guidelines may or may not still be appropriate. The same highway will then be used for years for a traffic mix in which the vehicle composition will change in size, configuration and operating characteristics over time. The connection between the requirements of the vehicles for safe operation and the capacity of the road to accommodate these requirements may thus be exceedingly tenuous. The hazards encountered, and created, by STAA vehicles operating on an Interstate system built to 1954 and 1965 guidelines are evidence of the difficulty in setting policies that are compatible with a broad range of vehicles, some of which do not even yet exist.

In addition to this basic dilemma, researchers have been reporting for about twenty years that AASHTO policies are inadequate. While some of the criticism is simply that AASHTO is not updated in a timely fashion (Glennon 1989), most of it demonstrates that the guidelines are insufficient even for the design vehicles on which they are based.

Glennon and Harwood (1978) have also noted that while AASHTO policies "...name the goals of safety, efficiency, economy, and comfort, they do not operationally define these goals." This leads to a piecemeal approach to highway design in which the stated goals are frequently not attained.

AASHTO policies on stopping distances have been examined in depth by researchers. Findings note that braking distances used, which are based on locked-wheel skidding on poor-condition wet pavement surfaces are not appropriate for speeds over 30 mph (Cleveland et al. 1985); that braking distances do not consider the substantially greater distances required by a truck (Harwood et al. 1989); that the assumption that a truck driver's greater eye height compensates for the greater stopping distance required is not valid on a horizontal curve with obstructions (Mason and Briggs 1985); that the basic functional AASHTO stopping sight distance model has not changed since 1940 (Neuman et al. 1983) and that its parameters do not correspond to the ability to decelerate a vehicle or to human visual limitations (Glennon 1989).

A major deficiency in AASHTO that significantly reduces the safety of highway ramps, as well as other curved segments, is that it does not recognize the reduction in a vehicle's stopping ability caused by horizontal curvature. The amount of friction available for decelerating or stopping is reduced by the friction used in cornering. When cornering

friction is considered, stopping sight distance requirements can increase by as much as 300 feet (Neuman et al. 1983). This deficiency combined with the inappropriate levels of friction recommended for use in design calculations, discussed in Sections 2.2.1 and 2.2.3, make trucks especially vulnerable.

Guidelines for highway ramp design are particularly deficient, making little or no allowance for the special requirements of trucks. The most important truck characteristic pertinent to safe operation on a curved ramp, the low rollover threshold of a truck, is ignored. The AASHTO policy refers the designer of a curved ramp to the guidelines for horizontal curves. Exactly the same criteria are used to design a highway segment with horizontal curvature and a ramp, with the only difference being a much lower design speed for the ramp. AASHTO design speed recommendations for highway ramps range from 10 to 35 mph less than the design speed for the preceding highway. Because guidelines for deceleration lane lengths are based on auto capabilities, trucks are frequently unable to decelerate sufficiently before entering the curved portion of the ramp.

Compounding the inadequacy of the AASHTO policy is the lack of coordination with MUTCD. Appropriate traffic control measures can be helpful in alleviating the hazards caused by deficiencies in geometric design. For example, timely placement of special speed reduction signs for trucks

approaching sharply curved ramps could be useful in avoiding rollovers or jackknife accidents. Some states, e.g. California, Maryland, Pennsylvania, have adopted this measure, but it is entirely a state initiative. The safety-enhancing opportunities that could result from better coordination of the AASHTO Policy and MUTCD are not addressed in these documents.

Most of the highway system in the United States is in place, and emphasis is now on resurfacing, restoration, and rehabilitation (3R) projects. These 3R projects present many opportunities for safety enhancement if designers avail themselves of current knowledge rather than blindly follow past practices. The highway design community is aware of these opportunities; they are discussed in depth in a recent Transportation Research Board publication (TRB 1987).

2.4 Costs of Truck Rollovers

Most agencies which have jurisdiction over highway operations and safety, and other, less specialized agencies, such as the National Safety Council, collect and publish data on the costs of various types of highway accidents. Most of these published costs, however, at best are not very informative, and at worst are misleading, and neglect that portion of highway accident costs that cannot be measured in dollars.

Every accident is unique. Therefore, precise costs for

categories of accidents cannot be formulated and average costs are not very enlightening. When actual costs arising from a specific accident are entered into a database, they become blurred. Therefore, cost estimates developed from data in a database are likely to be misleading. This is also true when accident costs are simulated. The variability of accidents makes theoretical simulation difficult. A 1988 study (Teal 1988) which simulated traffic delays caused by truck incidents on urban freeways noted substantial variations in the simulation output, often with the standard deviation exceeding the mean.

The difficulty in pinpointing actual numbers of truck accidents also leads to imprecision in evaluating costs of truck accidents. This issue is discussed in Section 2.4.1.

Despite the difficulty in assessing costs, it is clear that truck accidents are costly. This section will discuss the likely consequences of a truck rollover on a highway ramp. Types of costs will be described and dollar values will be given wherever relevant and available.

2.4.1 Truck Accident Rates

Trucks are vital to the U.S. economy. In 1989, they carried 42 percent of all domestic tonnage and accounted for 78 percent of domestic freight revenues, approximately \$257 billion or 4.9 percent of the Gross National Product (Grenzeback & Woodle 1992).

Data on numbers of truck accidents and truck accident

rates are not so clear, and must be interpreted with caution to avoid distortion. Published numbers of truck accidents vary widely. For example, in 1984 the National Highway Traffic Safety Administration reported 364,000 accidents involving trucks, while the Federal Highway Administration reported 37,000 (National Transportation Safety Board 1988). Clearly, the two agencies use different definitions of "truck accident", as do other agencies that monitor highway safety.

There is further conflict in interpretation of truck accident data, with disagreement over the most valid basis for comparison of accident rates between trucks and other types of vehicles, and between different types of trucks.

Highway accident rates are usually reported in terms of vehicle-miles of travel in order to consider exposure. However, since the enactment of the 1982 Surface Transportation Assistance Act (STAA), a growing number of larger tractor-trailers and multi-unit (twins, doubles, triples) tractor-trailers have appeared on U.S. highways, each of which would have previously been illegal. All of these STAA vehicles carry more cargo per vehicle-mile than the older, smaller tractor-trailers, thereby reducing overall exposure and raising the question of whether cargo-miles is a more appropriate measure of comparison than vehicle-miles.

Many studies have been undertaken in attempts to evaluate the comparative operational safety of STAA and older

vehicles. Results have been unclear at best, and sometimes conflicting, largely because of uncertainty in how to incorporate variations in travel on different types of roads, in cargo capacity, and in exposure. The operating characteristics of STAA vehicles appear to result in a somewhat higher accident involvement than the earlier, smaller vehicles, but the net effect vis-a-vis safety of combining the operating characteristics of STAA vehicles with their increased cargo capacity is not known (TRB 1989).

Many studies report that while the severity of truck accidents is greater than that of accidents involving other types of vehicles, trucks have a lower accident rate. However, the lower overall involvement of trucks appears to be largely due to the fact that they travel mostly on Interstates and toll roads, where the accident rate for all vehicles is lower than on other roads. When trucks are compared to other vehicles on the same roads, trucks turn out to have a higher accident rate (Jones 1984).

Almost all published studies agree that accidents involving large trucks are more severe than those involving only other types of vehicles (Bowman & Lum 1990, Polus & Mahalel 1985, Michie 1986). Heavy trucks are overrepresented in fatal accidents, with the risk of a fatality proportional to the weight of the truck. Also, the average number of injuries in accidents involving trucks is greater than in accidents involving only passenger cars and buses, despite

the fact that the number of passengers in a truck is generally lower than in any other vehicle (Polus & Mahalel 1985).

Reports of the percentage of tractor-trailer accidents that occur on ramps are not consistent, varying from 5.7 percent (Bowman & Lum 1990) to 16 percent (Polus & Mahalel 1985). Approximately 17 percent of tractor-trailer rollovers occur on ramps (Freedman et al. 1992). It is estimated that two thirds of "major incidents" on any highway, incidents which have an average duration of more than three and a half hours, are the results of overturns, spills and shifted loads. These incidents tend to occur on ramps (Grenzeback et al. 1990). While the primary cause of these rollovers is usually attributed to excess speed, many older highways in the United States have ramps with inadequate deceleration lanes and improper superelevation for today's larger trucks, deficiencies which are significant factors in rollovers.

2.4.2 Types of Costs Resulting From Truck Rollovers

The consequences of any highway accident can include personal injuries, fatalities, property damage to vehicles, the roadway and roadside, congestion, and a need for police and emergency medical and clean-up services. With truck involvement, there is the additional possibility of fuel spills and cargo spills, and cargo damage is likely. Accident costs for trucks vary widely, depending not only on vehicle damage, but also on freeway facility damage and the

type and degree of cargo loss.

The following example, described in the December 31, 1988 issue of the Status Report published by the Insurance Institute for Highway Safety, is given to illustrate the scale of costs of a truck rollover on a highway ramp. Neither the accident nor the costs should be considered representative, as there is no such thing as a "typical" accident. The accident, in which a 45-foot tractor-trailer loaded with 8,000 gallons of gasoline rolled over on a ramp in the Virginia portion of the Washington, D.C. beltway, occurred in 1988. The driver escaped without injury, but the tractor and trailer were totally destroyed by fire. Emergency response consisted of 46 vehicles, of which 30 were fire trucks. There was extensive damage to the roadway and several traffic lanes were closed for three months. The costs that were measurable in dollars were:

tractor & 43-foot gasoline tanker	\$107,000
8,000 gallons of gasoline	8,000
fire and rescue service	186,000
police	9,019
clean-up - contractor	10,500
" " - Virginia Highway Department	47,070
towing	4,100
State police	undetermined
bridge and road repairs	<u>200,000</u>
Total	\$571,789

The aftermath of the accident included several months of significant congestion, the attendant costs of which cannot be easily measured in dollars.

2.4.3 Costs of Personal Injuries and Fatalities

Costs arising from personal injuries include medical expenses, time missed from work resulting in lost wages for an employee and lost productivity for an employer, and pain and suffering. Injuries that are incapacitating are clearly more costly than those that are not incapacitating, but specific dollar costs vary widely from one accident to another.

Agencies concerned with highway safety customarily assign dollar costs to the personal injuries that can be incurred in an accident. However, these values are not consistent from one agency to another. In a 1991 study performed for the Federal Highway Administration, the Urban Institute recommended an increase in the assigned accident costs used in evaluations of highway safety improvements. The recommendations were that the cost of a single incapacitating personal injury be raised from \$47,000 to \$169,500, while the cost of a non-incapacitating injury go from \$18,900 to \$33,000. (Urban Transportation Monitor 1991). While these recommended values were much higher than those that had been used by most highway transportation professionals up to that time, with increases of 260 percent

and 75 percent respectively, they are lower than those currently used by the Federal Aviation Administration. Further discrepancies between agencies can be seen by comparing the \$47,000 and \$18,900 values used by the Federal Highway Administration prior to 1991, with the Caltrans practice during the same time period, of assigning a dollar cost ranging from \$10,300 to \$11,150 for personal injuries resulting from truck accidents (Teal 1988).

Appropriate assignment of costs in fatal accidents is even less clear. While actuarial tables list value of human life based on age at time of death and potential earnings, designation of the appropriate value of life to use in accident studies is always controversial. Regardless of the manner in which the cost of fatal accidents is assessed, and bypassing the philosophical view that human life is priceless, the cost of fatalities is clearly much higher than the cost of personal injury or property-damage-only (PDO) accidents. In 1985, Caltrans reported 3,171 accidents on the Los Angeles freeway system, of which 41 were fatal. At an estimated cost of \$556,500 per accident, the total cost for fatal accidents was more than \$22.8 million. This was more than three times the total \$6.7 million cost of PDO accidents in the same year, despite the fact that there were more than fifty times as many PDO's as fatalities. It was also more than double the \$10.8 million cost of almost twenty-five times as many personal injury accidents. (Teal 1988).

2.4.4. Costs of Property Damage

According to a 1986 Texas Transportation Institute study, urban property-damage-only accidents involving trucks have a 20 percent higher cost than those involving only automobiles (Bowman & Lum 1990).

Of all costs resulting from a truck rollover, property damage is the easiest to quantify. The cost of repairing a vehicle, and the value of damaged or destroyed cargo, can be determined fairly precisely, as can the cost of repairing damage to the roadway and roadside. However, there are further costs of damage to public property that are more difficult to measure. Beyond the cost of materials and labor, secondary issues arise. How promptly should repairs be done? Should crews be pulled from regular maintenance jobs or paid overtime to do immediate repairs? If repairs are not made immediately, to what level of risk or inconvenience is the public being exposed? If a work crew is pulled from another job, what is the cost to the public of the delay in completing the originally scheduled job? The final measure of costs of roadway and roadside damage therefore cannot be stated solely in dollars, but must acknowledge the fact that the public is being subjected to risk and/or inconvenience.

2.4.5 Emergency Services

The severity of truck accidents usually causes a greater need for emergency services than accidents in which trucks

are not involved. Police and medical personnel are required. Fire department equipment may be needed to extinguish fire, but more often to clean the roadway after a fuel spill. Wreckers and tow trucks are needed to right an overturned truck and to clear spilled cargo from the roadway.

Spilled loads are not common events, but they result in major disruptions when they occur. In 1985, Caltrans reported 64 major spilled load incidents on California freeways, where two or more lanes were closed for two or more hours. More than half of these incidents were associated with an overturned truck (Teal 1988).

Containing and clearing spilled fuel or cargo can be difficult and must be done with caution. The range of equipment and skills that should be available for such tasks can be estimated from the following list of cargo reported spilled in Chicago in 1986: steel coils, concrete block, turkey parts, live cattle, junk cars, gravel, metal shavings, cabin cruisers, drums of unknown chemicals, tubs of cheese, plastic pellets, sunflower seeds, water pipes, live chickens, and garbage (Federal Highway Administration 1991). When diesel fuel or other combustible liquids are spilled, special care must be taken to keep these substances from entering highway drains and sewers so that they do not contaminate and the surrounding area.

The argument can be made that emergency services should not be considered as one of the costs of truck rollovers.

Emergency personnel are employed and on the payroll, regardless of whether they have to respond to an emergency or not. Emergency equipment is owned by the locality regardless of whether it is in a garage or at the scene of a rollover. However, the presence of the emergency personnel and equipment at the scene of an incident, i.e a truck rollover, precludes their ability to respond to any other incident requiring emergency services that may occur during that time. The most appropriate measure of the cost of emergency services is therefore in terms of increased risk to the public. If a locality finds the level of risk unacceptable, it would need to expand availability of emergency services, a cost that can then be measured in dollars.

2.4.6 Congestion

Congestion is the most pervasive outcome of a truck rollover, but its costs are the most diffuse and difficult to measure. The costs of congestion include air pollution, wasted time, wasted fuel, increased driving time, wear and tear on vehicles, impeded flow of freight, and degradation of quality of life. Secondary accidents are likely to occur. A driver coming upon an unexpected queue may be unable to avoid colliding with a vehicle at the end of the queue, and building frustration when traffic is at a standstill or moving at crawl speed may cause drivers to take unreasonable risks which result in accidents.

While recurring congestion is a result of inadequate

highway capacity, accidents and vehicle breakdowns are the major cause of non-recurrent congestion. The Federal Highway Administration estimates that 60 percent of vehicle-hours lost to congestion are due to "incidents". While accidents account for only 10 percent of reported incidents, they cause disproportionately greater congestion than vehicle disablements, such as flat tires. This is partly because of the nature of an accident, where damaged vehicles or an overturned truck and spilled cargo and fuel may block the roadway, and also because of the presence of emergency vehicles which result in rubbernecking. The "hit-and-miss" nature of incidents masks the true impact they have on our highway systems, and makes it extremely difficult to quantify those impacts with any precision. This is especially true of congestion. It is known, however, that truck rollovers are the accident type where the resultant congestion has the longest duration (Recker et al.1990). In his study of congestion on the Los Angeles freeway system, Teal (1988) noted that truck incidents represent 15 to 22 percent of total delay costs, even though truck vehicle-miles represent only about 8 percent of the total vehicle-miles of travel on the system. He also noted that truck incidents were disproportionately concentrated in peak periods, times at which they would have the greatest negative impact.

Most agencies estimate the level of congestion caused by an accident from the number of lanes closed and the length of

time they are closed. Major accidents, a category into which truck rollovers fit, cause an estimated 2,500-5,000 vehicle-hours of delay per incident. Very severe accidents, typically those involving hazardous materials, last ten to twelve hours and cause 30,000-40,000 vehicle-hours of delay. While these are rare, their impacts can be catastrophic. It would appear that keeping records of actual hours of lane closures and delay would be feasible, but most metropolitan areas lack reliable counts of incidents, adequate measures of traffic impacts, and consistent estimates of overall highway congestion (Grenzeback & Woodle 1992).

While some effects of congestion, such as air pollution, can be measured, others are more difficult to track. The connection between certain effects and congestion as a causative factor may even be overlooked. An example of this is the case of an overturned truck blocking an exit ramp, forcing drivers to exit the highway at an unintended, unfamiliar location. These drivers are likely to be lost, confused, upset or angry, mental states which increase the probability of their becoming involved in an accident. The record of such an accident would note the location at which it occurred, but not the reason for the vehicle being at that location. The initial police report, if there is one, might include this information, but any connection between a blocked exit ramp and an accident on secondary roads or streets would be lost by the time the accident record became

part of a database.

Various schemes and methods have been devised to put a dollar value on vehicle delay costs resulting from congestion. Possibly the most widely known is the approach developed by AASHTO (AASHTO 1977) which assigns different dollar values to Low, Medium and High time savings and losses. This approach is based on the premise that small changes in travel time have little utility and therefore little economic value, but as time saved or lost increases, the economic value becomes significant. A 1988 study which simulated the costs of truck incidents on freeways was based on this approach. Annual delay costs of truck incidents on the Los Angeles County freeway system were estimated at almost \$300 million (Teal 1988).

Although this approach, and others, produce results in dollars based on reasonable theoretical suppositions, the fact remains that these dollar values are arbitrary and cannot be judged to be either correct or incorrect. This is not intended to suggest that cost estimates are without any value. Rather, they should be viewed as estimates of ranges of costs, which can be useful in making comparisons as long as all pertinent factors are taken into consideration.

The impact of an accident as regards congestion is closely correlated with the speed and adequacy of the response. Congestion can be minimized when incidents are cleared as quickly as possible and traffic is diverted before

vehicles are caught up in an incident queue. Because of this, more than thirty cities have instituted Incident Management Programs by which congestion is minimized through implementation of a systematic plan for quick and appropriate action. The speed and adequacy of the response are a special problem when a large truck is involved in an accident. Police officers, who are expected to coordinate response efforts, generally have little experience with major truck accidents because of the infrequency with which they occur. This is compounded by the lack of experience of tow-truck operators in dealing with large trucks. In most areas that do not have Incident Management programs, tow truck operators are called by rotation, and are likely to appear at the scene of a truck accident with inadequate or improper equipment (Grenzeback & Woodle 1992). These problems can double or triple the time required to clear an accident.

A 1984 staff study by the Federal Highway Administration found that freeway congestion in the thirty-seven largest metropolitan areas in the United States was responsible for two billion vehicle-hours of delay at a cost of \$16 billion. The study anticipated that those figures could rise to eight billion vehicle-hours and \$88 billion annually by the year 2005 (Grenzeback & Woodle 1992). The need to respond to this and similar predictions led to one of the provisions of the Intermodal Surface Transportation Efficiency Act (ISTEA) of 1991 which requires local governments to implement Congestion

Management Systems which will alleviate congestion in the most efficient, cost-effective manner possible. Management systems in other areas, such as pavements, bridges, and public transit, are also required by ISTEA. However, congestion is singled out for special attention in that the requirement is not only to alleviate existing congestion, but also to ensure that congestion does not develop in areas where it does not currently exist. Incident Management Programs respond directly to the systematic alleviation of non-recurring, incident-caused congestion.

Although trucks are frequently blamed as a major cause of highway incidents, congestion has a significant negative effect on the motor carrier industry. Highway congestion impedes the flow of freight, resulting in lost revenues and dissatisfied shippers. The delays caused by congestion are particularly significant in light of recent changes in American business practices forced by global competition. An increasing number of American businesses are engaging in "just-in-time" manufacturing and distribution, are using overseas suppliers, and are placing greater emphasis on quality and customer service. Motor carriers are being asked to provide faster, more reliable and more cost-effective service, demands which cannot be accommodated when highway congestion is encountered. The concern of the motor carrier industry is that shippers might elect to move their goods by rail if trucking does not allow the shipper to be

competitive. An additional concern of the motor carrier industry is that incidents involving trucks exacerbate the public's negative image of trucking and truck safety, leading to public demands for restrictions on trucking.

Chapter 3

OVERVIEW OF DEVELOPMENT AND USE OF METHOD

3.1 General Approach

While the method developed in this work is focused on reducing the incidence of truck rollovers on highway ramps, it has the broader purpose of demonstrating the feasibility and value of a proactive approach to highway safety. Work done by agencies in developing safety initiatives based on this method will also be useful in subsequent safety enhancement programs.

No ramp or highway segment exists independently. The adequacy of a ramp cannot be evaluated without also considering the preceding deceleration lane and highway. The geometric characteristics of any part of a highway are obviously the major factor in defining the level of safety at that location, but those characteristics also have a significant effect on what happens downstream. Therefore, a safety initiative at one site will have spillover benefits; safety at adjacent sites will be improved, and the agency will acquire information on which further programs can be based.

3.2 Development and Purpose of Hazard Ratings

In view of the large body of literature in which the deficiencies of AASHTO policies are described, the work done herein bypasses AASHTO and makes use of expert knowledge and opinion to produce guidelines for a proactive method to improve truck safety on ramps.

Use of expert opinion is an accepted approach in decision-making involving the expenditure of public funds. The process usually involves the application of standard techniques, such as the Delphi method, to the varied opinions of a group of experts. Ultimately, a single decision, or envelope of feasible decisions, is identified.

The method described herein uses expert opinion somewhat differently, in that the results of the survey of experts provide a set of ratings which non-experts can use in conjunction with other guidelines to make a decision. Similar use of an expert survey is made in a current Pennsylvania Department of Transportation (PennDOT) project to promote safe driving, in which experts were asked to rate the likelihood and consequences of inappropriate driving behaviors (Mason et al. 1992). The survey results are guiding PennDOT employees in directing educational efforts toward driver populations and behaviors where change is most desired.

In this dissertation, the results of a survey of experts are synthesized into numerical Hazard Ratings which describe

the relative level of hazard of specific characteristics of highway ramps. Agencies responsible for highway safety can use the Hazard Ratings in two ways. First, the ratings can be used simply to evaluate the relative level of hazard that exists on specific ramps. This allows the agency to develop a priority listing of ramps that need safety improvement. Second, by using the Hazard Ratings to measure the reduction in the existing level of hazard on a ramp that would result from specific improvements, the Hazard Ratings can be used in a cost-effectiveness analysis.

While there are many accepted approaches to public policy decision making, cost-effectiveness is believed to allow public officials, working within budget constraints, to achieve the greatest public benefit. It is, therefore, the generally preferred approach.

3.3 Use of Hazard Ratings

Use of the Hazard Ratings is demonstrated in a method following a cost-effectiveness approach. The method is intended to guide highway agencies in identifying the most cost-effective measures to alleviate rollover hazards to trucks on ramps. An agency using the method will take the following steps.

- a. Inventory the characteristics of all ramps in the jurisdiction.
- b. Determine the relative level of hazard present

at each ramp by applying a rating model. The model is based on a survey of expert opinion on the magnitude of hazards that could cause trucks to roll over. Development of the model is fully discussed in Chapter 4.

c. Apply the method described in Chapter 7, in which possible corrective measures at each ramp are rated according to cost-effectiveness.

The final result is a priority listing, based on cost-effectiveness, for possible safety programs to enhance the compatibility of trucks and ramps.

Chapter 4

Development of Hazard Ratings

4.1 Preparation of Survey Instrument

A questionnaire asking for opinions on the relative truck rollover hazard presented by specific geometric deficiencies and other conditions was developed and sent to a panel of experts.

Development of the questionnaire was complicated by two factors. One was the fact that the ultimate goal of the survey was not intended to be a simple tabulation of responses. Rather, the survey results were intended to provide the core for a model that will be part of a methodology. The responses, therefore, had to be elicited in a form in which they could be used later.

The second factor was the interactive nature of most of the relevant geometric characteristics, and the fact that the adequacy of each is closely related to the speed at which a truck is travelling. This precluded a simple questionnaire in which each characteristic could be evaluated independently.

The first version of the questionnaire asked for ratings of all combinations of velocities and interacting geometric characteristics. This format was much too long, and called for expert opinion to be calibrated to an unreasonably high level. The format was therefore changed, with the revised

version asking experts for their opinions of the relative hazard presented by deficiencies in each characteristic, with the others held constant. For some characteristics, separate evaluations were asked for different speed ranges. While this approach neglects the interaction of characteristics, it is as much as could reasonably be asked, and would produce a relative ranking. A range of 0-10 was selected for the ratings.

The overall question of the adequacy of a ramp was broken down into two parts:

1. the adequacy of the deceleration lane, which reflects the likelihood, and hazard, of a truck entering the ramp at too high a speed;
2. the adequacy of the ramp itself.

Consideration of the adequacy of the ramp geometry for the posted speed was selected as the clearest, most useful approach. This measures the existing hazard on a ramp and would later lead logically to the first, lowest cost, possible corrective measure: reduction of speed posted for the ramp. It also leads to the most logical approach to the evaluation of the deceleration lane, i.e. rating the adequacy of the distance available in which the truck must decelerate from the highway operating speed to the posted speed on the ramp.

The manner of evaluating the hazard associated with ramp curvature was especially sensitive because the interaction of

the radius, superelevation, friction factor, and velocity makes independent evaluations of these variables meaningless. The approach selected as most direct and potentially useful was to drop friction factor from the evaluation, because it is a totally dependent variable, and to rate the adequacy of the radius given the existing superelevation and posted speed.

Two assumptions about truck velocity were made in preparing the questionnaire. The first was that trucks operate on the highway at the same or lower speeds than other traffic. The hazard is greater if this assumption is incorrect, but higher operating speeds for trucks could be noted during inventory-taking, and taken into consideration. The other assumption was that trucks are attempting to comply with the speed posted for the ramp. While the speed of any one vehicle in a traffic stream is generally considered a random variable, this is not the case when a truck driver repeatedly operates on the same route. In such a case, driver familiarity with the ramp would increase the probability of the truck negotiating it safely regardless of the posted speed. In this case, an erroneous assumption of compliance with the posted speed would either err on the side of safety or be irrelevant.

Several different physical formats for the questionnaire were tested. Formats involving use of graphs, on which experts would sketch curves, and boxes in which numbers would

be filled in, were rejected as being too cumbersome and not sufficiently clear. The final format, using preprinted numbers to be circled, and the cover letter that was sent with the questionnaire, can be seen in Appendix A.

4.2 Responses

Thirty-three questionnaires were mailed out to a panel of experts culled from recent literature on truck operations, truck safety, and highway design relative to trucks. The experts included researchers from academia and from public or private agencies such as Insurance Institute of Highway Safety, as well as highway design practitioners from state DOT's. Of the thirty-three, twenty responded. Names and affiliations of the respondents are listed in Appendix B.

Raw tallies of the responses for each characteristic are shown in Appendix C.

4.3 Analysis of Responses

Twenty responses were considered sufficient to provide a basis for the model, as they came from a carefully selected panel of experts and were not a random sampling. However, because of the inordinate effect that any one or two responses out of twenty could have on the final results, the raw data was reviewed to determine if any responses were likely to skew the results and to see if there were any gross

misunderstandings of what the question was intended to ask. Dropping such responses would be justified, provided there were only a few. Many such responses would have pointed to a deficiency in the questionnaire, which would then have required further review.

Each expert's ratings were reviewed to see if any experts were consistently out of the range of the other responses. This was done by identifying experts whose ratings were consistently higher or lower than the others, or consistently beyond the range of $\mu \pm 2\sigma$. The effect that these responses had on means and other statistics were then evaluated. If any one or two experts was skewing results, consideration would be given to use of a factor to bring these into line with the others.

The responses were also reviewed to see if there was a clear consensus on a single numerical hazard rating for each characteristic, or whether there was a range of responses leading to a less clearly defined rating.

4.3.1 Statistics

A statistical analysis of the experts' responses was done using SAS software. Results are shown in Appendix D.

4.3.2 Major Deviations from the Mean

A Fortran program, LFTFLD, was written to determine whether the ratings of any of the experts had consistently large deviations above or below the mean. The program

compares each expert's rating for each characteristic to the mean as determined by SAS. The algebraic sum of the deviations is then computed for each expert. As plus and minus deviations cancel each other, a large sum of deviations identifies an expert who may tend to consistently make evaluations in a more extreme manner.

LFTFLD is written in Watfiv, which does not allow a data entry to be skipped if the expert omitted a particular rating. Use of zero to fill in for a non-response was precluded because zero was within the range of actual possible responses. Therefore, LFTFLD was run in two different ways:

- a. replacing a non-response with the mean
- b. replacing a non-response with 9.9

After running LFTFLD and reviewing the sums of the deviations, the cut-off point to define a large sum of deviations was arbitrarily taken as ± 60 . Results of the LFTFLD analysis show the following experts to rate in a consistently high or low manner.

<u>Non-response replaced by mean</u>	<u>Non-response replaced by 9.9</u>
Expert #9 (-143.65)	Expert #9 (-143.65)
#15 (-63.37)	#14 (+60.55)
#19 (-97.01)	#18 (+69.65)
#23 (+76.45)	#19 (-74.15)
#29 (-111.95)	#20 (+98.25)
#34 (+113.43)	#23 (+76.45)
	#24 (+179.85)
	#29 (-111.95)
	#31 (+61.75)
	#34 (+117.95)

The actual responses of these experts were then reviewed to see if the large deviations were caused by only a few ratings or by substitutions for non-answers made while running LFTFLD, or if there was a consistent pattern of either high or low ratings. The review showed that the sum of deviations from the mean for experts #14, 18, 20, 24, and 31 appeared excessive only because their non-responses had been replaced by 9.9. They were not earmarked when non-responses were replaced by the mean. This was checked by comparing only their actual responses with the means. The largest total deviation was 45.4; the range was 28→45.4.

Experts #9, 15, 19, 23, 29, and 34 appeared to be out of line with the others. Their responses were set aside for further analysis later to see the effect they had on the final ratings.

Ratings from all the experts for all characteristics were reviewed to see the extent of dispersion beyond the range of $\mu \pm 2\sigma$. A significant number of out-of-range ratings for a characteristic could indicate a complete lack of consensus, rendering the final rating suspect. Out-of-range ratings were also reviewed to see if many came from experts earmarked in the LFTFLD analysis, as this might indicate that only a few ratings were putting them in LFTFLD, and also to see if the ratings from any other experts consistently exceeded $\mu \pm 2\sigma$.

Results of this review showed that each characteristic

had, at most, two responses that were outside the range of $\mu \pm 2\sigma$. Most had one or none. Other than the experts earmarked in the LFTFLD analysis, most experts with out-of-range ratings had only one or two, or were just minimally out-of-range. Expert #1, however, had seven out-of-range ratings and #6 had five.

4.3.3 Effect of Major Deviations from the Mean

Based on the LFTFLD and $\mu \pm 2\sigma$ reviews, Experts #1, 6, 9, 15, 19, 23, 29, and 34 appear to be somewhat out of step with the others. The extent of their effect on the mean was evaluated by first recalculating the mean for each characteristic, omitting the responses of each of these experts, one at a time. Then the percent difference in the mean, with and without the response of these experts, was calculated to see how much effect these responses actually had on the mean.

Results showed no cases of a big difference in the mean caused by a single expert. There were several cases where the percent difference is large, but there would be minimal or no difference in the relative hazard rating; e.g. mean (all experts) = 0.25; mean (without #6) = 0.16. The difference is 36%, but the hazard rating, scaled from 0-10, would be zero either way.

No expert deviated consistently from the mean. In each case, the amount of deviation varied from characteristic to characteristic. Therefore, a factor to raise or lower all

responses from an expert is inappropriate.

4.3.4 Data Review Conclusions

Based on the evaluation of expert ratings described above, it was decided to include all responses from all the experts in constructing the functions for the Hazard Ratings.

The only exceptions are the Pavement Surface ratings from experts #27 and 31. Both noted in their questionnaires that they were rating an Icy surface as least hazardous, because a truck would slide off the icy pavement before it rolled over. A Dry surface was rated as most hazardous because the rollover would occur on the roadway itself. This interpretation misses the point, which is whether conditions on the road are likely to lead to a rollover, and these responses will be disregarded.

Comments from many of the experts also led to the decision to drop Curve Length as one of the determinants of the degree of hazard on a ramp. There appears to be no consensus whatsoever on the effect of this characteristic on the possibility of a rollover. Five of the experts did not respond at all in this category, and several who did respond stated bluntly that they were just guessing.

4.4 Application of Fuzzy Mathematics

Although the experts who participated in the survey used specific numbers to express their opinions, these numbers

must be taken as approximate rather than precise values. An opinion, by definition, cannot be precise. For this reason, fuzzy mathematics, which acknowledges imprecision, was used in the synthesis of the experts' ratings into a single number to be used for evaluating the hazard presented by existing ramp characteristics in the proposed method.

4.4.1 Definition of Fuzzy Sets

Fuzzy sets are useful in clarifying the expression of certain quantitative statements that cannot be made with precision. They do not compensate for random error. Rather, an acknowledgement of fuzziness is appropriate when the imprecision is based on human uncertainty, or on vagueness in the problem description, or in cases where there is no sharp transition in level of correctness between various members of the set.

The use of fuzzy sets allows the degree of membership of all elements in a set to be noted. For example, if the fuzzy set of "red" is considered, crimson has a very high degree of membership, pink and magenta have a somewhat lesser degree of membership, orange has less, and green does not belong to the fuzzy set of red at all. This is a clearer description of what "red" is than a statement that pink, magenta and orange are "reddish", which implies that each is equally red or not red.

If the safety of an intersection is described by a fuzzy set ranging from 0 to 10, where 0 is not safe and 10 is

perfectly safe, an uncontrolled intersection might have a degree of membership of 0 or 1, an intersection with two-way stop signs might have a degree of membership of around 3 or 4, an intersection with four-way stop signs about 5 or 6, and an intersection controlled by a traffic signal would have the highest degree of membership, about 9 or 10. The fact that the signalized intersection, which is the safest, is not represented by precisely 10 is a reflection of the fact that perfect safety is not attainable.

Most of the early work in developing fuzzy logic and mathematical techniques was done by L.A. Zadeh in the 1960's. The work has undergone extensive development since then, with widespread applications in science, engineering and other fields. It can be applied to many mathematical problems, and is especially useful in converting verbal to mathematical statements.

4.4.2 Mathematical Expression of Fuzzy Sets

$A = \mu_1|x_1 + \mu_2|x_2 + \dots + \mu_n|x_n$ describes a fuzzy subset A of a universe of discourse U, where μ ranges from 0 to 1 and represents the level of membership of each term $x_1, x_2 \dots x_n$ in A. For example, in the fuzzy subset red of the universe of discourse of all colors, scarlet has a grade of membership of approximately 1, magenta approximately .7, pink approximately .4, orange approximately .2, and green approximately 0.

Fuzzy sets do not have standardized functions (Murthy

and Sinha 1990), nor is there a standard approach to estimating the membership function which describes a fuzzy set. Zadeh has described " π " and "S" functions which conveniently describe many fuzzy sets (Zadeh 1976). The π -function is a bell-shaped curve, similar to a normal distribution in statistics, but with finite values of zero at each end. The S-function is half the π -function. The description of a fuzzy set however is not limited to such previously defined functions, and any function that describes the set in question may be used. Usually, the membership function is developed from a statistical analysis or a group decision approach.

4.4.3 Mathematical Operations on Fuzzy Sets

The extension principle, developed by Dubois and Prade (1978), allows standard arithmetic operations, such as addition, subtraction, multiplication and division, to be applied to fuzzy sets. The extension principle operates as follows. Consider a fuzzy number A which is a function of the fuzzy numbers M and N, i.e. $A = F(M,N)$, where $M = f(x)$ and $N = g(y)$. The membership function of A is then defined as $h(z) = \max[\min(f(x),g(y))]$, where $z = f(x,y)$. In applying the principle, all combinations of x and y that yield the same value of z are determined, and the minimum of each is selected. The maximum of these minima is the membership value of z.

For example, consider the following membership

functions:

$$M = f(x) = 1|3 + .7|4 + .3|5$$

$$N = g(y) = .2|1 + .5|2 + .7|3 + 1|4$$

$A = M + N$ is computed as shown in Table 1.

Table 1. Addition of Two Fuzzy Sets

Possible Sums of $x + y$	Combinations That Produce These Sums	Membership Values of Those Combinations
4	(3+1)	(1, .2) [3+1]
5	(3+2), (4+1)	(1, .5) [3+2]; (.7, .2) [4+1]
6	(3+3), (4+2), (5+1)	(1, .7) [3+3]; (.7, .5) [4+2]; (.3, .2) [5+1]
7	(3+4), (4+3), (5+2)	(1, 1) [3+4]; (.7, .7) [4+3]; (.3, .5) [5+2]
8	(4+4), (5+3)	(.7, 1) [4+4]; (.3, .7) [5+3]
9	(5+4)	(.3, 1) [5+4]

Minimum Membership Value of Each Combination

$$(M+N) = \max \{ (.2)|4; (.5)|5, (.2)|5; (.7)|6, (.5)|6, (.2)|6; (1)|7, (.7)|7, (.3)|7; (.7)|8, (.3)|8; (.3)|9 \}$$

Fuzzy Set Representing $A = M + N$

$$A = (M+N) = .2|4 + .5|5 + .7|6 + 1|7 + .7|8 + .3|9$$

Similarly, $B = M*N$ is calculated as shown in Table 2.

Table 2. Multiplication of Two Fuzzy Sets

Possible Products of $x*y$	Combinations That Produce These Products	Membership Values of Those Combinations
3	(3,1)	(1, .2) [3*1]
4	(4,1)	(.7, .2) [4*1]
5	(5,1)	(.3, .2) [5*1]
6	(3,2)	(1, .5) [3*2]
8	(4,2)	(.7, .5) [4*2]
9	(3,3)	(1, .8) [3*3]
10	(5,2)	(.3, .5) [5*2]
12	(4,3), (3,4)	(.7, .8) [4*3]; (1, 1) [3*4]
15	(5,3)	(.3, .8) [5*3]
16	(4,4)	(.7, 1) [4*4]
20	(5,4)	(.3, 1) [5*4]

Minimum Membership Value of Each Combination

$$(M*N) = \max \{ (.2) | 3; (.2) | 4; (.2) | 5; (.5) | 6; (.5) | 8; (.8) | 9; (.3) | 10; (.7) | 12, (1) | 12; (.3) | 15; (.7) | 16; (.3) | 20 \}$$

Fuzzy Set Representing $B = M * N$

$$B = (M*N) = .2 | 3 + .2 | 4 + .2 | 5 + .5 | 6 + .5 | 8 + .8 | 9 + .3 | 10 + 1 | 12 + .3 | 15 + .7 | 16 + .3 | 20$$

Similar procedures are followed for subtraction, division, exponentiation, or any other arithmetic operation.

In addition to the standard arithmetic operations, other fuzzy operations may be performed to enhance the expression of fuzzy sets. These include fuzzy concentration, which decreases the membership of all elements of the set except those with a degree of membership of 0 or 1; fuzzy dilation, which does the opposite; and fuzzy intensification, which is a combination of the two.

4.4.4 Algebraic Operations on Fuzzy Sets

Algebraic operations can be performed on fuzzy sets using either the Direct Method or the Indirect Method. Both methods produce the same final result; therefore, the choice of method is based solely on convenience. Generally, the Indirect Method requires fewer calculations and is less cumbersome to use.

4.4.4.1 Direct Method

Difficulties that arise when the Direct Method is used can be seen in the calculation of the weighted average of fuzzy sets. The procedure is the same as calculating the weighted average of ordinary numbers, but fuzzy rather than ordinary arithmetic operations are used.

For example, consider a fuzzy weighted average

$$(4.1) \quad R = \frac{(X*A) + (Y*B) + (Z*C)}{(A+B+C)}$$

where X , Y and Z are fuzzy sets which rate the hazards of certain components of a structure, and A , B and C are weighting factors which reflect the importance of those components. The fuzzy weighted average, which represents a single evaluation of the level of hazard that exists in that structure, is computed by applying fuzzy addition, multiplication and division. However, application of the extension principle to perform the fuzzy mathematics will require lengthy calculations, even with the use of a computer. If the values in sets X , Y and Z each range from 0 to 10, and the values in sets A , B and C range from 0 to 5, the values in the fuzzy set representing the denominator will range from 0 to 15, and the numerator will be represented by a fuzzy set ranging from 0 to 150, with a level of membership specified for each value in both sets. For this reason, unless the sets involved are very small, it is usually more convenient to use the Indirect Method for algebraic operations on fuzzy sets.

4.4.4.2 Indirect Method

The Indirect Method of computing a fuzzy weighted average involves the use of α -cuts. An α -cut of a fuzzy number defines the range of membership values in which every value is greater than some value of α . Possible values of α range from 0 to 1. An α -cut may be taken at any level of α selected by the analyst. Use of α -cuts allow a function to be described to any desired level of precision, with

increased precision obtained by increasing the number of α -cuts. For example, α -cuts of 0.0, 0.2, 0.4, 0.6, 0.8 and 1.0 can be used to describe a membership function. A more precise description of the same function would be obtained by using α -cuts of 0.00, 0.05, 0.10, 0.15, 0.20, 0.25, etc.

Representation of α -cuts of the fuzzy number "approximately 5" are shown in Figure 3 and Table 3, taken from Murthy and Sinha (1990). In this example, "approximately 5" is described by a π -function, but α -cuts can be used with functions of any shape.

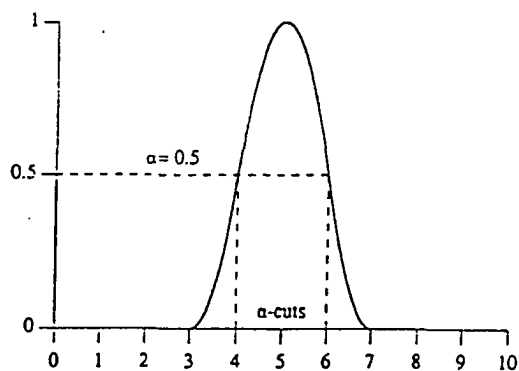


Fig.3 Fuzzy Function
"Approximately 5"

Table 3. α -Cuts of
"Approximately 5"

α	Range
0.0	(3.00, 7.00)
0.1	(3.45, 6.55)
0.2	(3.63, 6.37)
0.3	(3.77, 6.23)
0.4	(3.89, 6.11)
0.5	(4.00, 6.00)
0.6	(4.11, 5.89)
0.7	(4.23, 5.77)
0.8	(4.37, 5.63)
0.9	(4.55, 5.45)
1.0	(5.00, 5.00)

Using the Indirect Method, a fuzzy weighted average is calculated by applying arithmetic operations to the α -cuts rather than to the fuzzy function itself, thus reducing the

complexity of the calculations. The fuzzy weighted average of the α -cuts can then be reduced to a single ordinary number that can be used in practical applications.

In Section 4.5, the Indirect Method is applied to fuzzy functions that represent expert opinion on the relative hazard of specific ramp characteristics.

4.5 Application of Fuzzy Mathematics to Results of Expert Opinion Survey

The intended use of the results of the survey of experts, described in Sections 4.2 and 4.3, was to produce a single number called a Hazard Rating which reflects the relative likelihood that a specific ramp characteristic would lead to a truck rollover. The Hazard Ratings would later be applied to the results of a ramp inspection to produce a "Notice Rating" describing the level of hazard that exists on that ramp. While the Notice Rating for a specific ramp would not have absolute meaning, it would be an accurate guide in determining the relative level of hazard of each of the ramps in a jurisdiction.

The synthesis of the survey responses on the relative hazard and importance of each characteristic into fuzzy sets, and then into a single ordinary number, for each characteristic, required the following steps:

- a. development of a membership function for each characteristic, based on the responses of the experts

- b. determination of an appropriate equation to calculate the Hazard Ratings
- c. taking α -cuts at selected levels of the membership functions of each characteristic
- d. calculation of the α -cuts for the terms in the Hazard Rating equation, using the α -cuts of each membership function
- e. synthesis of the calculated α -cuts into a single ordinary number

4.5.1 Development of Membership Functions for Each Characteristic

Questionnaire responses reflecting expert opinion on the level of hazard of specific ramp characteristics were plotted on bar graphs (Appendix E). The graphs were then studied to determine the mathematical functions that most nearly described the responses. While responses for some characteristics fit the description of π or S functions, most did not fit into any standard category. As the work to be done does not require that the membership function be described in algebraic terms or be a continuous function, the membership functions were left in the form described by the bar graphs. The only changes made were to normalize the functions and adjust them for convexity.

For example, the bar graph of survey responses describing the relative hazard of a drop greater than 4 inches at the pavement edge is shown in Figure 4.

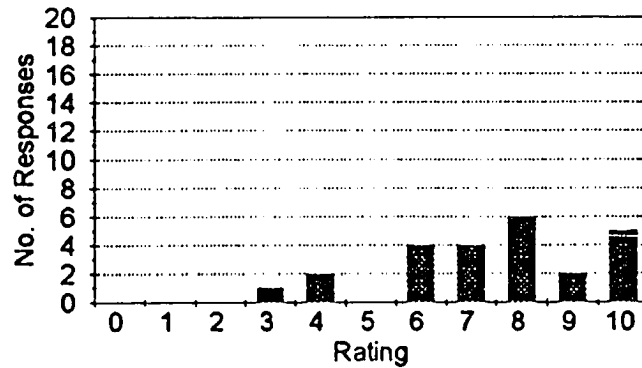


Fig. 4 Drop > 4 Inches

The normalized fuzzy membership function described by this graph is:

$$f(x) = .17|3 + .33|4 + 0|5 + .67|6 + .67|7 + 1|8 + .33|9 + .83|10$$

Adjustment for convexity produces:

$$f(x) = .17|3 + .33|4 + .50|5 + .67|6 + .67|7 + 1|8 + .91|9 + .83|10$$

4.5.2 Development of Equation to Calculate Hazard Ratings

The characteristics of a ramp that might contribute to the likelihood of a truck rollover interact in a complex manner. Despite considerable work by many researchers, definitive mathematical expressions to describe these complexities have not yet been developed. For this work it was determined that the sum of the hazard ratings for all the

characteristics of a ramp would reflect the relative hazard with sufficient accuracy.

The following equation was used to compute the Hazard Rating of each characteristic that might exist on a ramp

$$(4.2) \quad HR = r_1 * w_1 * w_2$$

In this equation, r_1 , w_1 , and w_2 represent the α -cuts of the membership functions of the ramp or deceleration lane characteristic, the relative importance of the deceleration lane or ramp, and the relative importance of the ramp characteristic, respectively, and HR represents the α -cuts of the Hazard Rating. Note that for deceleration lane characteristics, this equation reduces to

$$(4.3) \quad HR = r_1 * w_1$$

There are 2^N possible combinations of α -cuts of the fuzzy variables that are used in calculations with fuzzy sets, where N is the number of fuzzy variables. In this case, $N = 2$ for the deceleration lane and $N = 3$ for the ramp itself, resulting in four combinations for each deceleration lane characteristic and eight for each ramp characteristic. Each calculation is performed with the end points of the range of fuzzy sets at that level of α , or the α -cuts of the Hazard Ratings.

The procedure used to calculate the α -cuts of the fuzzy

sets representing the Hazard Ratings is demonstrated in the next section.

4.5.3 Calculation of the Hazard Rating for a Ramp Characteristic Using the Indirect Method

The relative hazard and relative importance of each characteristic of a ramp is described by the membership functions shown in Appendix F. These functions were developed from the bar graphs in Appendix E, and have been normalized and adjusted for convexity where necessary.

To demonstrate the procedure for calculating the Hazard Rating for a ramp characteristic, the rating for a drop greater than four inches at the pavement edge will be calculated.

The membership functions that are pertinent are:

r_1 , drop greater than four inches:

.17 | 3 + .33 | 4 + .50 | 5 + .67 | 7 + 1 | 8 + .91 | 9 + .83 | 10

w_1 , relative importance of ramp (compared to deceleration Lane):

.14 | 2 + .14 | 3 + .14 | 4 + .43 | 5 + .43 | 6 + 1 | 7 +
.57 | 8 + .28 | 9 + .28 | 10

w_2 , relative importance of drop at pavement edge (compared to other characteristics of the ramp):

.2 | 2 + .2 | 3 + .6 | 4 + .6 | 5 + 1 | 6 + .6 | 7 + .6 | 8 +
.6 | 9 + .6 | 10

The α -cuts for these variables, and the bar graphs from which they have been taken, are shown in Figures 5 through

7. The graphs show the membership functions after they have been normalized and adjusted for convexity.

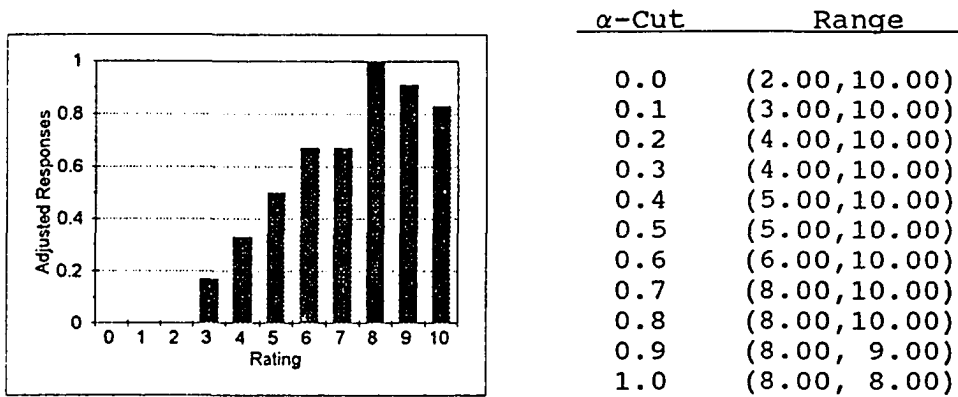


Fig. 5 r_1 , Drop > 4 Inches

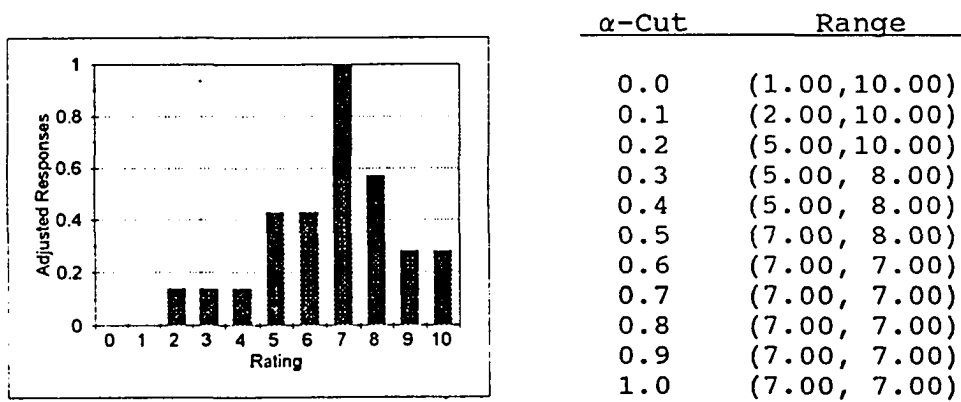
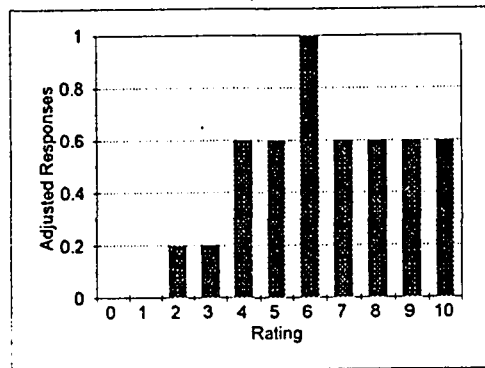


Fig. 6 w_1 , Relative Importance of Ramp



α -Cut	Range
0.0	(1.00, 10.00)
0.1	(2.00, 10.00)
0.2	(2.00, 10.00)
0.3	(4.00, 10.00)
0.4	(4.00, 10.00)
0.5	(4.00, 10.00)
0.6	(4.00, 10.00)
0.7	(6.00, 6.00)
0.8	(6.00, 6.00)
0.9	(6.00, 6.00)
1.0	(6.00, 6.00)

Fig. 7 w_2 , Relative Importance of Drop At Pavement Edge

The calculation is done in two parts. In the first part, the eight possible combinations of the α -cuts of the variables are used to find eight values of HR by solving Equation 4.2. This procedure is performed independently for each level of α . The eight combinations used for $\alpha = 0.0$ and $\alpha = 0.1$, and the results of the calculation are shown in Table 4. The same procedure is followed for $\alpha = 0.2$ through $\alpha = 1.0$.

In the second part of the calculation, the α -cuts of the fuzzy Hazard Ratings are used to compute a single ordinary number which represents the Hazard Rating for a pavement edge drop greater than four inches. First, the maximum and minimum results, HR, of the eight calculations for each level of α are noted. Results for all values of α are shown in columns 1 and 2 of Table 5. These results represent the

Table 4. Example of Calculations Using α -Cuts

$\alpha = 0.0$				$\alpha = 0.1$			
r_1	w_1	w_2	HR	r_1	w_1	w_2	HR
2	1	1	2	3	2	2	12
2	1	10	20	3	2	10	60
2	10	1	20	3	10	2	60
2	10	10	200	3	10	10	300
10	1	1	10	10	2	2	40
10	1	10	100	10	2	10	200
10	10	1	100	10	10	2	200
10	10	10	1000	10	10	10	1000

range of values of the α -cuts of the Hazard Ratings. The midpoint of the range is then determined (column 3) and multiplied by the value of α . The results are shown in column 4.

The sum of all values in column 4 will then be divided by the sum of all values of α , producing a single ordinary number which is the relevant Hazard Rating. In this example 2186.3 is divided by 5.5, producing 397.51. This final result is rounded off to 398, which is the Hazard Rating that represents the relative level of hazard presented by a drop

greater than four inches at the pavement edge.

Table 5. Calculation of Ordinary Number From α -Cuts

Col. 1	Col. 2	Col. 3	Col. 4
α	Range	Midpoint	Cols. 1 x 3
0.0	2, 1000	501	0.0
0.1	12, 1000	506	50.6
0.2	40, 1000	520	104.0
0.3	80, 800	440	132.0
0.4	100, 800	450	180.0
0.5	140, 800	470	235.0
0.6	168, 700	434	260.0
0.7	336, 420	378	264.6
0.8	336, 420	378	302.4
0.9	336, 378	357	321.3
<u>1.0</u>	336, 336	336	<u>336.0</u>
5.5			2186.3

The procedure described herein was applied to all the ramp characteristics rated by the experts. The results are shown in Table 6. Table 6 also includes an ordinary weighted average of the expert opinion on a factor to use when a ramp

is part of an interchange.

During the survey of experts, the relative importance of deceleration lane characteristics was not included in the questionnaire. As a result, there is no w_2 term in the equation used to calculate the HR values, and the Hazard Ratings for deceleration lane deficiencies are considerably lower than those for the ramp itself. If any agency using the Hazard Ratings believes that this undermines their usefulness in the decision-making process, they can develop w_2 terms by surveying their own engineers. The need for weighting factors for the deceleration lane characteristics became apparent during the development of the Hazard Ratings, and will be included in future iterations of the survey.

A method in which the Hazard Ratings are used to alleviate the truck rollover hazard on ramps in a jurisdiction is described in Chapter 7. In this method, the relative hazard that exists on a ramp is expressed by a Notice Rating, which is the sum of all Hazard Ratings that are relevant for that ramp. Cost-effectiveness analysis is then used to determine priorities for corrective action. The Hazard Ratings are used again in the cost-effectiveness analysis, by using the reduction in the Notice Rating to measure the effectiveness of a proposed corrective action.

Table 6. Hazard Ratings

		<u>Calculated Values</u>	<u>Rounded Values</u>
Deceleration Lane Length			
$V \leq 40$ mph	100% adequate	0.15	0
	80% "	4.65	5
	60% "	10.85	11
	40% "	14.90	15
	20% "	31.17	31
Deceleration Lane Length			
$40 \text{ mph} < V \leq 60$ mph	100% adequate	0	0
	80% "	5.44	5
	60% "	17.22	17
	40% "	22.68	23
	20% "	30.54	31
Deceleration Lane Length			
$V > 60$ mph	100% adequate	0	0
	80% "	8.41	8
	60% "	20.47	20
	40% "	27.96	28
	20% "	32.27	32
Deceleration Lane Downgrade			
	0%	0.29	0
	1-2%	7.38	7
	3-4%	16.94	17
	5-6%	30.97	31
Deceleration Lane Pavement			
	Dry	0.36	0
	Wet	18.15	18
	Snow	22.73	23
	Ice	32.31	32
Type of Transition Curve			
	Spiral	17.10	17
	Compound	172.67	173
	One on Tangent and Curve	147.96	148
	All on Tangent	183.24	184

(continued on next page)

Table 6. (continued)

		<u>Calculated Values</u>	<u>Rounded Values</u>
Radius	V ≤ 20 mph		
	100% adequate	6.36	6
	80% "	198.67	199
	60% "	289.45	290
	40% "	453.43	453
	20% "	609.35	609
Radius	20 mph < V ≤ 40 mph		
	100% adequate	8.18	8
	80% "	164.64	165
	60% "	263.47	263
	40% "	448.09	448
	20% "	610.22	610
Radius	V > 40 mph		
	100% adequate	1.82	2
	80% "	182.05	182
	60% "	356.44	356
	40% "	514.15	514
	20% "	619.84	620
Cross-Slope Difference	6%	115.81	116
	8%	217.98	218
	10%	293.49	293
	12%	396.15	396
Lane Width	≥ 13 feet	18.44	18
	12 "	108.65	109
	11 "	214.09	214
	10 "	327.14	327
	9 "	430.72	431
	≤ 8 "	435.00	435
Ramp Downgrade	0%	21.82	22
	1-2%	95.33	95
	3-4%	216.68	217
	5-6%	362.55	363
	> 6%	495.31	495

(continued on next page)

Table 6. (continued)

	<u>Calculated Values</u>	<u>Rounded Values</u>
Drop > 4" at Pavement Edge	397.51	398
Presence of Outside Curb	495.80	496
Type of Compound Curve		
sharp → flat	235.91	236
flat → sharp	260.54	261
sharp → flat → sharp	402.82	403
flat → sharp → flat	321.86	322

If ramps are components of interchanges, use a factor of 1.4.

Chapter 5

CORRECTIVE MEASURES

5.1 General Approach to Corrective Measures

The goal of achieving safe ramps must be considered from two viewpoints, geometric and operational. A ramp that has all the geometric characteristics necessary for safe travel by a truck may become the site of a rollover if the truck is traveling too fast. The opposite, a ramp with geometric deficiencies that can safely accommodate a truck that is moving sufficiently slowly, is also conceivable but is not a realistic expectation. Therefore, the corrections that should be considered at a ramp in need of safety enhancement should include measures to induce truck drivers to operate their vehicles at appropriate speeds, as well as measures that correct geometric deficiencies.

Appropriate operating speed on a ramp is particularly important for tractor-trailers. Drivers who are familiar with a ramp are generally aware of the speeds at which they can travel safely, even if these are higher than the posted speeds. A tractor-trailer, however, may be loaded differently each time it travels on a particular highway, and driver familiarity with the ramp may not be sufficient to

compensate for varying rollover thresholds of the tractor-trailer.

Measures that can be taken to reduce the risk of a truck rolling over on a ramp are presented in this section, starting with the simplest and increasing in scope to the most complex, i.e. total reconstruction of the ramp. They are not presented in order of cost, nor are costs given, because of extreme variations in cost in different locations. However, when the method described in Chapter 7 is applied to a specific ramp, corrective measures should be considered in order of increasing cost.

Correcting ramp deficiencies that might lead to a rollover requires a qualitative, as well as quantitative approach. As noted earlier, a truck may roll over on a ramp that is geometrically "correct" in theory if the driver's actions are inappropriate. Therefore, selection of possible corrective measures for a specific ramp, and decisions on effectiveness of these measures, must rely heavily on the knowledge and experience of local traffic engineers who can provide unique insight into traffic operations and driver behavior on that particular ramp.

5.2 Speed Reduction Measures

5.2.1 Calculation of Appropriate Speed for Ramp

Before any effort is made to reduce truck speeds on a ramp, the appropriate truck speed for the existing geometry

of the ramp must be determined. This can be done by using a slightly modified form of the basic equation for highway curve design (AASHTO 1990)

$$(2.2) \quad f = \frac{V^2}{15R} - e$$

where f = side friction factor (dimensionless)

V = velocity (miles/hour)

R = curve radius (feet)

e = superelevation (dimensionless)

As noted in Section 2.1, the side friction factor is mathematically equivalent to the net lateral acceleration, and the maximum lateral acceleration a truck can withstand without rolling over is called the rollover threshold. Therefore, a safe speed at which a tractor-trailer can operate on a specific ramp can be calculated by estimating the maximum acceptable lateral acceleration, replacing the friction term in the basic curve equation with this quantity, and solving the equation for velocity.

A method of calculating maximum acceptable lateral acceleration is given in a study sponsored by the Insurance Institute for Highway Safety (IIHS) (Freedman, Olson and Zador 1992). This method, which is based on work done for the Federal Highway Administration (Ervin 1986), uses the equation

$$(5.1) \quad a_{y_{max}} = \frac{(R-M)}{S}$$

where $a_{y_{max}}$ = maximum acceptable lateral acceleration
(multiples of gravitational acceleration
constant, g)

R = rollover threshold (multiples of g)

M = margin of safety factor (multiples of g)

S = steering factor (dimensionless)

Selection of appropriate values of R and M requires the judgment of local traffic engineers, based on their knowledge of types of trucks using the ramp and the frequency with which they do so. Rollover thresholds for typical truck configurations are given in Figure 1, but local engineers must decide on a reasonable balance between the use of an average value for rollover threshold, R, along with a high margin of safety factor for M, as opposed to the use of the worst case rollover threshold with a lower margin of safety factor. In the work done for IIHS, 0.27g was chosen as the generalized fleet rollover threshold and 0.11g was chosen as the factor representing a margin of safety. The steering factor, S, compensates for the difficulties encountered in steering a truck at lateral accelerations above 0.14g; the IIHS study used 1.15 as the value of the steering factor.

These calculations may reveal that the speed posted for a ramp exceeds that at which a tractor-trailer can operate safely. If the calculated speed is one at which trucks can

reasonably be expected to operate, the posted speed should be reduced. Depending on conditions at a particular ramp, the speed reduction could apply to all vehicles or might be posted for trucks only. However, if the geometry of the ramp is so extreme that the calculated "safe" speed is unreasonably low, reduction of the posted speed is not a sensible option and attention should be shifted to improving the physical characteristics of the ramp.

5.2.2 Compliance with Posted Speed

After verification that the speed advisory on a ramp is appropriate, at least in theory, the level of compliance by truck drivers must be determined. Two questions must be answered: are trucks entering the ramp at excessive speed, or are they entering the ramp at appropriate speeds which then increase to an excessive rate after they are on the ramp? The answers to these questions will lead to different corrective measures. If trucks are not being operated at excessive speeds, speed reduction measures are irrelevant.

Any standard method of measuring a vehicle's speed can be used in this application, but care should be taken to measure speed both on the ramp and at the entrance. Simple observation should indicate likely spots for speed checks on the ramp itself.

If a considerable number of trucks are being operated at excessive speeds, interviews with truck drivers may provide insight into the causes of this behavior and guidance in

selection of appropriate corrective measures.

5.2.3 Excessive Speed at Ramp Entrance

A tractor-trailer will enter a ramp at an excessive speed if the deceleration lane is too short or if the driver deliberately chooses to ignore the speed advisory.

A 1989 study done for the Federal Highway Administration notes that current guidelines for deceleration lane length are deficient for tractor-trailers (Firestine, McGee and Toeg 1989). The guidelines underestimate actual truck speeds and overestimate truck braking capabilities, ignoring the fact that average truck speeds today are at least equal to automobile speeds. At equal speeds, trucks require longer deceleration distances than passenger cars, a need which is not accommodated by design guidelines. The standard method of measuring the length of the deceleration lane adds to the problem. Current guidelines (AASHTO 1990) call for deceleration lane length measurement to start at the beginning of the taper, thus assuming that drivers are decelerating while still traveling in the through lane. This is not a realistic assumption, nor is it safe.

5.2.4 Inadequate Deceleration Lane Length

Correction of a deficiency in deceleration lane length starts with a determination of the actual distance required for a tractor-trailer to decelerate to the posted speed by the time it enters the ramp. This can be done using lengths

reported in results of recent research, where appropriate, or by calculating the values directly.

Table 7, adapted from the FHWA work (Firestine, McGee and Toeg 1989) gives minimum deceleration lengths for exit ramps that would be appropriate for heavy trucks. Assumptions made when the lengths in the table were calculated were that trucks coast in gear for three seconds and then brake at a rate of 0.15g's. The lengths given in the table are fifteen to fifty-five percent longer (depending on highway design speed) than those required for passenger cars, and are recommended for deceleration lanes with high truck volume and where the ramp has an early tight curve.

Table 7

**Deceleration Length (ft.)
For Design Speed of Exit Curve (mph.)**

HIGHWAY Design Speed (mph)	Avg. Running Speed (mph)	Stop	EXIT Design Speed (mph)									
			15	20	25	30	35	40	45	50		
30	28	271	227	198	162							
40	36	413	370	341	305	262	212					
50	44	585	541	512	477	434	384	295	227			
60	52	785	741	712	677	634	584	495	427	352		
65	55	867	823	795	759	716	666	577	509	434		
70	58	954	910	881	845	802	752	664	596	521		

Note, however, that the values of highway running speed listed in the table are averages. They may be considerably lower than the actual speeds of many vehicles, particularly on Interstates and major freeways which were built to specifications for 70 mph design speeds and on which actual speeds approach or exceed the design speed. Therefore, before the deceleration lane lengths given in the table are used, the actual highway speeds of trucks in the vicinity must be determined. Then, a decision must be made, based on the frequency with which the highest truck speeds occur, on whether an average of all truck speeds should be used in deceleration lane length calculations or if an average of some percentile of the highest truck speeds is more suitable. If the "typical" tractor-trailer speed is found to be higher than that given in the table, required deceleration lane lengths must be increased beyond those shown.

When the deceleration lane length is increased to meet the requirements of a tractor-trailer, care must be taken in measuring the length, starting the measurement from the point where the taper has developed a full lane width of twelve feet.

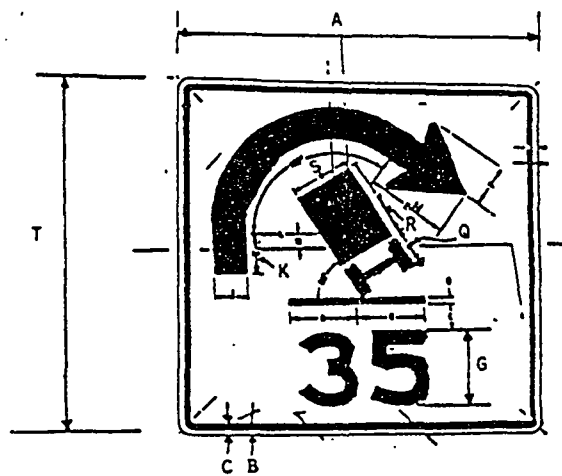
If it seems likely that truck drivers will comply with the posted speed advisory and enter the ramp at a proper rate of speed if they have sufficient distance in which to decelerate, the deceleration lane should be signed as specified in the Manual on Uniform Traffic Control Devices

for Streets and Highways (MUTCD) (Federal Highway Administration 1988). Standards for Exit Speed signs are given in Par.2C-36, and call for 48 inch by 60 inch rectangular signs with black lettering on a yellow background. The signs are specified as advisory, not warning, and together with a deceleration lane of sufficient length, should provide safe entrance to the ramp for truck drivers who comply with the posted speed.

5.2.5 Noncompliance with Posted Speed

Stronger measures must be taken if drivers are deliberately ignoring the speed advisory. A variety of schemes have been tried in different locations with varying degrees of success. Local traffic engineers should decide which measures are most likely to work at a specific location, either using measures which have been tried elsewhere or possibly experimenting with their own ideas. Implementation of any measure must always be followed by an observation period to determine whether, in fact, the intended speed reduction has been achieved.

Some states have adapted standard speed advisory or warning signs to emphasize the potential rollover hazard for trucks. These rectangular or diamond-shaped black on yellow signs show a tipping truck and an arrow indicating a curve. The speed may be shown on the sign itself or on a supplemental panel. Signs of this type that are in use in Maryland and Pennsylvania are shown in Figure 8.



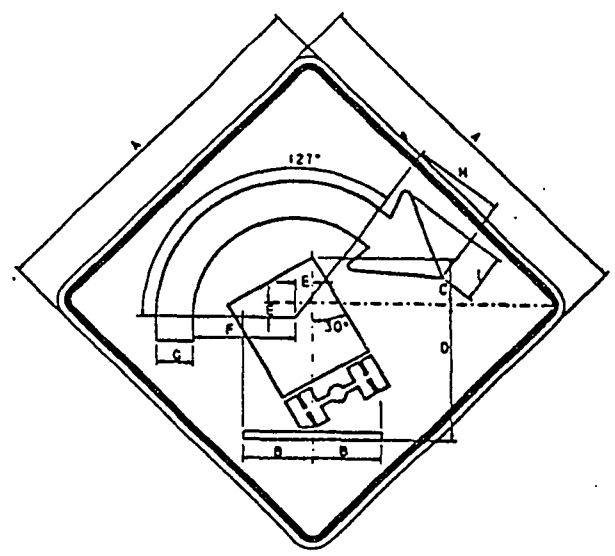
TIPPING TRUCK

		DIMENSIONS (INCHES)																	
NO.	1/4" Champ	A	B	C	D	E	F	G	H	J	K	L	M	N	P	Q	R	S	T
35		96	1 1/4	2 1/4	-	7	15 1/4	2 1/2	10	9	6	25	4	22	12	10	21	17	100

COLORS
 LEGEND - BLACK
 BACKGROUND - YELLOW (REFL)

MAY BE ORDERED FOR ANY SPEED.
 MAY BE ORDERED WITH LEFT TURNING ARROW AND RIGHT TIPPING TRUCK.
 MAY BE ORDERED IN EITHER DIAMOND OR RECTANGULAR SHAPE.
 SEE 7A FOR ARROWHEAD DETAIL.

Maryland's truck ramp signing.



COLORS: LEGEND AND BORDER BLACK
 BACKGROUND YELLOW (REFLECTORIZED)

SIGN SIZE	DIMENSIONS										MAR. CIN.	BOR. DER.	BLANK STD.	
	A	B	C	D	E	F	G	H	I	T				
48X48	48	9	5 1/2	18	2	13 1/2	5	11 1/2	6 1/2	1 1/2	1 1/2	1 1/2	1 1/2	83-48

Pennsylvania's truck ramp signing.

Fig. 8 Rollover Warning Signs

If signing alone is ineffective, compliance may be improved by flashing yellow warning lights that are mounted on the signs. Use of these lights, which are activated by excess speed, has been tried in Atlanta, Georgia. However, their usefulness in reducing tractor-trailer speeds is diminished by the fact that the sensors react only to speed and do not differentiate between cars and trucks, thus causing the lights to flash almost continuously (Fitzpatrick, Middleton, and Jasek 1992). In the project done for IIHS (Freedman, Olson and Zador 1992), truck drivers were asked for their opinions of a group of truck rollover warning signs that were being considered for use. The one they preferred had a yellow background and showed a silhouette of a tipping truck in black. The sign was a diamond-shaped warning sign. The speed was posted on a separate advisory speed panel, and a flashing yellow light was mounted to the right, above the warning sign. In this study, the light was activated manually, simply to determine whether or not it was effective in reducing truck speeds. Results showed that a flashing sign is more effective than one that does not flash, and suggest that the flashing sign is especially effective in reducing the speed of the most egregiously speeding trucks. While sensors are currently in use that will activate a flashing light on a basis of speed, the report on the IIHS work notes that future improved automated techniques may be able to activate warning signs on a basis of speed and

additional factors related to a specific truck's rollover threshold, such as axle loads, center of gravity height, and width.

At some locations, the use of rumble strips, along with signing, may promote compliance with speed advisories. Rumble strips alone are generally ineffective as speed control devices. Their primary function is to draw a driver's attention to traffic control devices or potential hazards, alerting the driver to a need to take action. Care must therefore be taken in placement of the rumble strips, so that they lead the driver's attention to the warning or traffic control device with enough time to take appropriate action.

Rumble strips are patterns, either raised or grooved, placed on the pavement surface of a roadway or shoulder. They provide motorists with an audible and tactile warning through the noise generated by tires passing over the strip and through vibration induced in the vehicle, which the driver senses through contact with the steering wheel and vehicle seat. They have been used since at least the 1950's by almost all state and toll road authorities, but despite their wide use, knowledge of their effectiveness is limited. They appear to be effective in accident reduction on intersection approaches; estimates of effectiveness in other applications are not available (TRB 1993). The only study of the effectiveness of rumble strips on approaches to

horizontal curves was done by the British Transport and Road Research Laboratory. This single study found that rumble strips were effective in reducing accidents related to high speed or lack of driver awareness (TRB 1993).

Rumble strips have some adverse aspects which should be considered before a decision is made to install them. Noise is a serious negative environmental impact, which has led most highway agencies to adopt policies against the use of rumble strips in residential areas. Motorist reaction to the noise and vibration is also a concern. Some motorists mistake these effects for a problem with their vehicles, while others are concerned about potential damage to the vehicle from the vibration. Maintenance of rumble strips can be difficult. Raised strips, particularly, may need frequent replacement because of damage, may exhibit poor adhesion with the asphalt surface, and may cause problems when snowplows are used.

Rumble strips appear to be less effective in slowing trucks than in slowing passenger cars. This is not well documented, but appears reasonable in that a truck driver may not notice the effects of the rumble strips because of the higher noise and vibration levels involved in normal truck operation (TRB 1993).

While rumble strips may be of limited value for the specific goal of reducing truck speed to prevent truck rollovers, their use may be appropriate in locations where

the intent is the reduction of speed of all vehicles.

Other, more unusual, measures to promote speed reduction have met with some success at the sites where they have been tested. Most are intended for all vehicles, not specifically trucks. Santa Barbara, California, has experimented with mobile roadside speedometers which allow approaching drivers to see the posted speed limit and their actual speed at the same time. The device consists of a speed limit sign, a doppler radar emitter and receiver to measure speeds, and a large display that shows the speed of an approaching vehicle. Significant speed reductions were achieved at sites where the speedometers were located, especially at those sites where excessive speeds were frequent, but long run usefulness of these devices has not been determined (Casey and Lund 1991).

The city of Calgary in Canada has had good results in tests of a different type of sign. A green metal sign, 1.3 meters by 2.3 meters (approximately 51 inches by 91 inches), with white lettering was mounted two meters (approximately 79 inches) above the ground. The messages on the sign read: DRIVERS NOT SPEEDING YESTERDAY and BEST RECORD. The relevant percentage was inserted each day, after each statement. The sign, which was used in conjunction with speed measurement devices for the tests, was placed after a sign indicating an upcoming speed limit reduction but before the reduced speed advisory itself. The sign was placed in a location with a high incidence of speeders, and appears to have had a

significant impact on speed reduction, with fairly long-term effect (Maroney and Dewar 1987).

Different speed limits for trucks and cars may be effective in some locations. In 1987 and 1988, forty states took advantage of a 1987 federal law allowing them to raise the speed limit on their rural Interstates from 55 to 65 mph. Ten of these states set lower speed limits for trucks, with seven states keeping the truck limit at 55 mph and three allowing 60 mph. Results of a study by Insurance Institute for Highway Safety (Baum et al. 1991) to determine whether the differential speed limits led to a significant difference in speeds were inconclusive. Truck speeds appeared to be only slightly lower in states with differential speed limits. However, a comparison of average truck speeds within an entire state showed that the percentage of trucks going faster than 70 mph was twice as large in uniform limit states as in differential limit states (Baum et al. 1991). This seems to suggest that the differential limit does have some salutary effect, despite the fact that most drivers may be exceeding the speed limit.

5.2.6 Excessive Speed on the Ramp

A truck driver who has entered a ramp at the posted speed may then accelerate to an excessive speed while on the ramp itself. The acceleration may be deliberate, in preparation for merging or if the driver assumes that the posted speed pertains to a single curve on which he is

already traveling. The latter is a serious problem on ramps with compound curves where a relatively flat curve is followed by a sharper one. The acceleration may also occur unintentionally, as a result of driver inattention on a steep downgrade.

Any of the measures intended to reduce speeds as the vehicle is entering the ramp may also be useful on the ramp itself. Others are intended to be implemented only on the ramp itself. One such measure that relies on optical illusion was tested in the city of Calgary, where transverse lines were painted on an exit ramp at progressively diminishing distances apart. The distance between the fluorescent white lines which began 100 meters (approximately 328 feet) after the ramp began, gradually decreased from 7.7 meters (approximately 25 feet) at the start to 2.75 meters (approximately 9 feet) nearing the end. The diminishing distances created an illusion of vehicle acceleration, increasing driver alertness. A significant reduction in speeds was achieved immediately, but the effects decayed over time. Speed monitoring was subsequently discontinued, so it is not known if speeds eventually returned to their original rates or settled at a plateau somewhere between the original and the lowest speeds achieved (Maroney and Dewar 1987).

Additional signing on the ramp, beyond that recommended in MUTCD, to warn drivers of compound curves and/or steep downgrades may be useful. The hazard presented by a compound

curve in which the sharper portion of the curve follows the flatter portion can be alleviated if drivers are aware of the need to maintain reduced speed, and are not misled into thinking that the entire curve is relatively flat. A Federal Highway Administration study (Firestine, McGee and Toeg 1989) recommends placement of a turn sign with an appropriate speed advisory plate along the flat section to provide drivers with an updated warning that a sharp curve still remains ahead. The final curve can be highlighted with a large arrow or with chevron alignment signs. Properly placed chevron signs can be particularly effective in making the driver aware of what is ahead. The MUTCD (Federal Highway Administration 1988) notes that the signs should be spaced so that drivers always have two in view along the curve, and that the signs should be visible for at least 500 feet.

Signs showing a silhouette of a tipping truck, reminding truck drivers to maintain reduced speed on curves and downgrades may be useful.

An increase in the length of the acceleration lane at the end of the ramp will provide an extra measure of safety by allowing sufficient distance in which a driver can accelerate to merging speed. However, this measure will be effective only with drivers who are familiar with the ramp. Others may still assume a need to accelerate too early.

5.2.7 Speed Control Measures - Summary

Signs of various sorts are generally the most effective

and least costly means of speed control. However, local engineers must use their judgment and knowledge of a specific location to determine the most appropriate measures for that particular site. They should not be limited to measures that are customary, published, or used in other locations. Innovation should be encouraged; whatever achieves the goal of speed reduction is correct.

Since there is no way to determine the effectiveness of any measure before it is implemented, the results of any action must be investigated to see if the desired results are being achieved. This applies to both traditional and innovative measures, and is recommended by the Federal Highway Administration (Freedman, Olson and Zador 1992). Tests of effectiveness should be part of the implementation process, and should be repeated periodically to assure long-term effectiveness. •

Maintenance of any device that is used in safety enhancement is vital. Signs, which are likely to be the primary speed-control measure, must be kept clean and reflective. Neglect of maintenance will not only eliminate the safety improvement achieved by the device, but can present a new hazard by diverting a driver's attention from the roadway for an excessive length of time as he tries to decipher an illegible sign.

5.3 Correction of Ramp Deficiencies

As with speed reduction measures, both traditional and innovative actions can be taken to alleviate hazardous conditions on a ramp. Again, local engineers should incorporate their knowledge of the operating characteristics of specific ramps when they develop lists of measures that may correct deficiencies at those sites. Their choices should be based entirely on their opinions of what may work, and not on cost; the cost effectiveness of all corrective measures that are considered for a specific ramp will be determined and ranked using the method described in Chapter 7.

Some corrective measures that should be considered are as follows.

5.3.1 Deficiencies in Friction, Superelevation or Radius

Pavement friction, superelevation, and curve radius are the geometric factors that, along with velocity, determine the magnitude of rollover forces acting on a truck. Because of the interaction between these factors, improvement in any one will alleviate deficiencies in the others.

In order to provide an optimal level of friction, a pavement on a freeway ramp should have what AASHTO (1990) describes as a "high-type" surface. While AASHTO recommends high-type pavements for locations with high traffic volume, the critical nature of ramps would justify this type of

pavement even where volume is moderate or less.

The friction available on an asphalt pavement can be increased by providing an overlay of open-graded asphalt concrete. The open-graded mix, in which the aggregate consists almost entirely of coarse particles, provides better surface friction than a dense-graded mix, where the inclusion of fine particles produces a smoother surface. On a Portland-cement pavement, tining will increase the available friction. In addition, additives and admixtures which enhance various properties of both asphalt and Portland-cement concrete are constantly being developed. Use of these should be considered where appropriate.

Researchers have found that rollover accidents tend to occur at sites with high levels of side friction demand (Firestine, McGee and Toeg 1989). This demand is a result of some combination of excessive speed and insufficient superelevation for the existing degree of curvature; therefore, an increase in the superelevation rate will enhance the safety of the ramp.

The optimal superelevation rate for a ramp varies with vehicle speed and curve radius. On any ramp, the shorter the radius (i.e. the sharper the curve), the greater the need for a combination of superelevation and side friction to counteract rollover forces. This need also increases as the velocity of a vehicle on the ramp increases. The theoretically optimal superelevation rate can be determined

by considering the curve radius and the range of speeds at which trucks on the ramp are travelling. For a realistic assessment, local climatic conditions which limit the amount of side friction that can develop, must also be considered. The centrifugal forces that develop on a slow-moving vehicle are relatively small, and if a slow-moving vehicle is travelling on a curve that has a high rate of superelevation, the centripetal force may exceed the centrifugal force. On a dry pavement, sufficient side friction can develop in the centrifugal direction to balance the lateral forces acting on the vehicle. However, on an icy or wet pavement, the amount of available side friction might be inadequate to keep the vehicle from sliding into the center of the curve. While term "slow-moving" cannot be defined precisely, the comparatively low design and posted speeds on the ramp along with the possibility of traffic congestion should make slow-moving vehicles a serious concern.

AASHTO recommendations for maximum superelevation rates range from 0.04 to 0.10 (AASHTO 1990), but these are for highway curves and do not reflect the more severe conditions that may exist on ramps. Therefore, a decision on the rate of superelevation to provide on any specific ramp must be based largely on engineering judgment.

When it appears impossible to provide a combination of side friction and superelevation that will allow safe travel by a truck on a particular ramp, the corrective measure that

should be considered is reconstruction of the ramp with a longer radius curve.

5.3.2 Cross-section Transitions

As a roadway changes from a tangent section to a curve, the normal crown cross-section changes to one that is superelevated. If the manner in which this change is achieved is deficient, the demand for side friction can reach high levels that increase the threat of a rollover.

There are several ways in which the transition to a fully superelevated cross-section is customarily made: by developing part of the superelevation on the tangent and part on the curve, by developing all the superelevation on the tangent, or by use of a spiral curve. From the standpoint of safety and efficient vehicle operation, spiral curves are vastly preferred.

A spiral curve begins with zero degree of curvature and a normal crown cross-section at the tangent end. As the spiral progresses to the junction with the circular curve, the increase in the degree of curvature, the tangent runout, and superelevation runoff are achieved at a constant rate. At the junction of the spiral and circular curves, both have the same degree of curvature and the pavement cross-section is fully superelevated. The development of superelevation at the same rate as the development of the degree of curvature reduces the likelihood of excessive demands for friction. AASHTO recommends the use of spirals, noting that they

provide safe, easy-to-drive paths (AASHTO 1990).

The other types of transition from normal crown to superelevation, while widely used, are deficient in two ways. When any superelevation is developed on the tangent, this unnecessary superelevation may force a driver to steer in the direction opposite to the upcoming curve in order to keep his vehicle on the road. This is an awkward, uncomfortable maneuver for the driver, and is therefore unsafe. At locations where part of the superelevation is developed on the tangent and part on the curve itself, in addition to the hazard presented by the unnecessary superelevation on the tangent, vehicles on the beginning portion of the curve where the superelevation is not yet fully developed may require an excessively high level of pavement friction.

Therefore, spiral curves should be constructed wherever there is an existing deficiency in cross-section transition, and should be included in any reconstruction project, e.g. lane or shoulder widening, curve flattening, etc., that is undertaken on a ramp.

5.3.3 Removal of Outside Curbs

A curb along the outside of a curved ramp may act as a tripping mechanism that can cause a truck to roll over. Simple removal of the curb will correct this deficiency.

Another approach to eliminating this hazard was used by Michigan DOT, where the curb on a ramp was incorporated into the pavement. This was done by adding a wedge of pavement to

cover the outside curb (Fitzpatrick, Middleton and Jasek 1992), and resulted in greater safety on the ramp beyond that achieved by just eliminating the curb hazard, in that the lane and shoulder were widened and the superelevation across both was increased.

5.3.4 Drop-off at Pavement Edge

A drop-off at the edge of the pavement may also act as a tripping mechanism that can cause a truck to roll over or jackknife. Tolerance for drop-off height is lower on roadways with significant proportions of heavy trucks or motorcycles (Glennon 1987), as these types of vehicles would have greater difficulty in recovering from a wheel encroachment on a low shoulder than would passenger cars or vans. Tractor-trailers experiencing outward high-speed off-tracking are especially at risk.

An unstabilized shoulder is the most likely cause of a hazardous drop-off. Surface erosion from vehicle encroachment or weathering can cause the shoulder elevation to drop as much as several inches below the pavement edge. The hazard can be corrected by adding material, preferably stabilized, to raise the shoulder elevation.

Glennon (1987) describes another alternative, in which the shape of the pavement edge is changed to more closely approximate a forty to sixty degree continuous taper by either adding an asphalt wedge or grinding the existing edge. This enhances the likelihood of recovery of a vehicle whose

wheel has left the pavement, but is probably more beneficial for small vehicles than for tractor-trailers.

5.3.5 Other Corrective Measures

Excessive differences in cross-slopes or sideslopes also present rollover hazards for trucks. These can be corrected by flattening the shoulder or sideslope. This potential hazard must be considered when a safety enhancement project on a ramp includes an increase in superelevation; care must be taken to adjust the adverse slope of the shoulder and sideslope accordingly, so that the difference does not become excessive.

Another possible safety measure that should be considered is to widen the shoulders on a ramp. The wider shoulders give the driver of a tractor-trailer greater margin for error, allowing greater flexibility in steering and minimizing the effects of off-tracking.

Reduction in the downgrade of a steep ramp is another measure that will make the ramp safer.

5.3.6 Corrective Measures - Summary

The preceding sections of this chapter have described some corrective measures which can be used to alleviate safety deficiencies on a ramp. However, safety enhancement projects cannot be standardized and should not be limited to the measures discussed.

Because of the unique geometry, operating

characteristics, and constraints of every ramp, and because of the interaction of these factors, safety enhancement must be planned on a site-specific basis. A geometric and operational evaluation of each ramp, done by local engineers, will lead to the most appropriate measures for inclusion in the cost-effectiveness analysis that will indicate the action to be taken.

Plans for safety enhancement of a ramp must also include consideration of the expected physical and operating characteristics of future vehicles. For example, use of semitrailers longer than 48 feet, the most common length today, is increasing rapidly. Use of 53-foot trailers will probably increase (Glauz 1987), intensifying off-tracking and other problems.

Over the past fifteen years, walls and barriers to contain large tractor-trailers have been developed, tested, and installed in some locations. While these walls and barriers stop the truck from rolling over, thereby reducing the severity of the accident, they do not prevent an accident from happening and are therefore not considered in this work.

Chapter 6

Cost-Effectiveness

6.1 Why Cost-Effectiveness?

Highway users evaluate the adequacy of a highway in a very personal manner. A driver who can go from one point to another, travelling at a speed chosen with relative freedom, will probably consider the road "good". The driver will give no thought to what makes that highway "good" unless he or she encounters a situation that prevents free movement, e.g. congestion, potholes, or involvement in an accident.

This type of personal anecdotal approach cannot be used by a highway operating agency. The need to provide goods and services that affect the common interest of citizens is one of the justifications for the existence of governments (Stokey and Zeckhauser 1978). In order to effectively discharge their responsibilities to provide highways on which the Level of Service (LOS) would be considered acceptable by the public, highway agencies must use an orderly, methodical approach to address safety, capacity and other relevant factors. Such an approach will guide the agency in using available funds to achieve the greatest public good.

Ideally, a public agency would have unlimited funds with which it could execute all projects for which benefits are

greater than costs. In reality, this is never the case. Budget constraints always limit the number and size of projects which can proceed. The agency must therefore set priorities.

Cost-effectiveness and benefit-cost analysis are approaches to public expenditure decision-making which aim to ensure that resources are put to their most effective use (Stokey and Zeckhauser 1978). Both require the systematic enumeration of all benefits and costs that will accrue to all members of society if a particular project is adopted. The two approaches differ in the manner in which benefits are measured. In benefit-cost analysis, both benefits and costs are measured in dollars. This allows many different types of benefits to be considered directly, simply by including their dollar value in the analysis. In cost-effectiveness, costs are usually measured in dollars, but a single surrogate is used to represent all benefits that will result from the action being considered. In the analysis, benefits resulting from the proposed action are measured in terms of effect on the surrogate only, with no direct evaluation of the benefits inferred by the surrogate. For example, in this work, benefits are measured by reduction in the level of hazard on a ramp. While this reduced level of hazard suggests a reduction in injuries, property damage, congestion, and other adverse effects of a rollover, none of these benefits are measured directly. In both approaches, the ratio of benefits

to costs, or vice versa, guides the analyst in determining whether and when it is worthwhile to proceed with a project.

An example of benefit-cost analysis is seen in the work by Rymer and Urbanik (1989) in developing warrants, based on delay savings, for increasingly complex interchanges. In their analysis, the authors place a dollar value on delay time, thus allowing the cost of delays and the benefits accrued from the reduction of delay time to be considered in dollars. Had this been a cost-effectiveness analysis, delay time would have been measured in units of time.

The basis of cost-effectiveness and benefit-cost analysis is that alternative methods are available for reaching an objective, and each requires resources and produces benefits. A cost-effectiveness analysis systematically evaluates alternative methods for accomplishing the objective by examining the costs and effectiveness of each (Glennon 1974). The results of the analysis, which reflect the benefit per unit of input, can then be used by the agency to set priorities and develop a plan for action that will result in the greatest achievement of the specific objective in question. A systematic approach is also beneficial in that it enhances the agency's ability to limit the influence of special interests and politics in the decision-making process. An agency with a clearly defined highway safety or risk control plan in place is also in a good position to defend tort liability cases.

6.2 Cost-Effectiveness Analysis

Cost-effectiveness analysis has four basic steps. First, all possible alternative actions must be identified. Then, the costs and benefits resulting from these actions are determined. Next, the benefit/cost (B/C) ratio is computed. The last step is to use the B/C ratio to rank the alternatives in order of desirability.

The types of alternatives which must be identified in the first step will vary depending on the purpose of the specific analysis. The alternatives may be many different projects to be implemented at many different locations, or various types of repair or rehabilitation projects at one specific location, or a combination of the two. For example, the goal of the analysis may be to identify bus stops at which shelters should be built, or it may be to investigate various ways to improve runway capacity at a single airport, or to decide which of the airports in the jurisdiction of that agency should proceed with capacity-enhancement projects. In the case of the bus shelters, the list of alternatives must include locations of all bus stops at which a shelter may be desirable, as well as all bus shelter designs which may be considered. To improve runway capacity at a single airport, alternatives would include various physical improvements to existing runways, construction of additional runways, and improved traffic control technology which could increase capacity by allowing shorter headways.

If the purpose of the analysis is to determine at which airports the capacity should be increased, all possible ways to increase capacity at all airports in the jurisdiction would have to be identified.

In the second step, all impacts, both good and bad, of the alternatives being considered must be identified. The extent of each impact is measured or estimated, and it is classified as a benefit, if good, or a cost if bad. In the private sector, the viewpoint from which to identify benefits and costs is usually clear and the classification process is simple. However, it becomes complex in an analysis done for a government agency. The society represented by a government agency is made up of diverse segments, each having different ideas of what is good and what is bad. These varying viewpoints complicate the government's job of classifying good and bad impacts of any proposed project. A further complication is that the costs and benefits of any single action may impact different segments of society. The question of whether to consider negative effects as reductions in benefits or as increases in cost also arises.

For example, if a private taxi company is considering adding taxis to its fleet, the purchase price of the vehicles, maintenance and operating costs, and additional salaries for drivers would clearly be costs, while increased fares would be benefits. In this case, both costs and benefits impact the company directly. In the case of bus

stop shelters for a municipal bus system, money spent in constructing the shelters is a cost borne by all taxpayers. The increased comfort of waiting passengers would clearly be a benefit, but would not impact all those who bore the cost. Additionally, more ambiguous benefits and costs, such as the reduced pedestrian LOS resulting from the reduction of sidewalk space, must also be considered. Many effects of a larger, more far-reaching project such as an airport expansion are even more ambiguous, and therefore more difficult to classify. While some costs and benefits are clear, such as the dollar expenditure needed for the physical expansion of the airport, others are less clear. For example, the resulting increase in air traffic will be good for the economic health of the area, but bad for residents of areas surrounding the airport who will experience more noise and increased vehicular traffic in their neighborhoods.

The issue of whether to classify a negative effect as a reduction in benefit or an increase in cost always arises in a public project. It can be resolved in several different ways, as shown in the following example (Newnan 1989):

On a proposed government project, the following consequences have been identified.

1. Initial cost of the project to be paid by the government is 100.
2. Present worth of future maintenance to be paid by the government is 40.

3. Present worth of benefits to the public is 300.
4. Present worth of additional public user costs is 60.

The benefit-cost (B/C) ratio can be computed in the following ways:

- a) All benefits divided by all costs.

$$\frac{300}{100 + 40 + 60} = \frac{300}{200} = 1.5$$

- b) Consider user costs as a "disbenefit".

$$\frac{300 - 60}{100 + 40} = \frac{240}{140} = 1.7$$

- c) Consider maintenance costs as a "disbenefit".

$$\frac{300 - 60 - 40}{100} = \frac{200}{100} = 2.0$$

Each of these methods of computation is valid. In the basic determination of whether a project is desirable ($B/C > 1$) or undesirable ($B/C < 1$), each method will lead to the same result. However, the analyst must be consistent in applying the same computation method to each alternative. Failure to do so can lead to erroneous rankings of projects in order of desirability.

For highway projects, AASHTO recommends use of the second approach: $B/C \text{ ratio} = \text{net public benefits/government costs}$ (AASHTO 1978). Generally, benefits and costs, other than the initial capital cost, will be identified on an

annual basis. They may be left in that form or converted to their present worth for use in the B/C analysis.

The option of doing nothing must always be considered as an alternative. While this option may sound neutral, the analyst should be aware that doing nothing may have long-range negative effects which should be acknowledged in numerical calculations. Consider a jurisdiction in which all roads going through commercial districts have a low LOS. If one road is improved while the others are left as they are, commercial activity along the unimproved roads will not remain as is, but is likely to decline.

Calculation of the B/C ratio for each alternative allows the analyst to select the most desirable alternative. The analysis should begin with the "do nothing" alternative, where the B/C ratio is zero because no costs are incurred. The most desirable alternative to implement will be identified in the first iteration as the alternative having the highest B/C ratio. To continue ranking the alternatives in descending order of desirability, the analyst must now shift to incremental analysis. After the initial selection, the problem is not to determine which of the original alternatives is the best second choice. Rather, the question is to identify the best alternative for further investment, given that some investment has already been made.

The procedure for incremental analysis is the same as for the initial analysis, but compares the current measure

being considered with the next-lower-cost measure already analyzed, noting incremental increase in costs (Δ costs) and benefits (Δ benefits). The B/C ratio is thus reflective of the increase in benefits over the increase in costs. An alternative earmarked as the next best choice using incremental analysis is not necessarily the one that would have been identified as next best choice using ordinary B/C or cost-effectiveness procedures. Successive applications of incremental analysis until all alternatives have been ranked will provide an orderly basis for expenditure of whatever funds become available.

Cost-effectiveness analysis tends to give higher priority to low-cost improvements, even when benefits appear to be relatively low. While this may appear questionable on the surface, the following example demonstrates the logic of the procedure (Glennon 1974). Note that in this example, cost is the numerator and benefit (hazard reduction) is the denominator of the cost-effectiveness (C/E) ratio. While this is the inverse of the usual ratio, and identifies alternatives with the lowest ratios as optimal choices, the procedure and results are the same.

<u>Improvement</u>	<u>Hazard Reduction</u>	<u>Annualized Cost (\$)</u>	<u>C/E Ratio</u>
Remove sign	0.05	2	40
Install guardrail at bridge abutment	0.15	24	160

In this example, the C/E ratio shows that sign removal is a much higher priority improvement, even though the hazard reduction of the guardrail is considerably greater. The point is that twelve such signs could be removed, with a total hazard reduction of 0.60, for the cost of one guardrail installation.

The application of cost-effectiveness analysis in the proposed method is described in detail in Chapter 7.

6.3 Applications of Cost-Effectiveness in Highway Engineering

Many examples of cost-effectiveness can be found in the literature on highway design, operation and safety. Past work has ranged from very specific applications, such as reduction of utility pole accidents (Zegeer and Cynecki 1984) and investigation of capacity enhancement of various interchange design configurations (Wattleworth and Ingram 1972), to a much broader view of overall systematic design consistency related to highway safety (Glennon and Harwood 1978).

Following are brief summaries of reports of some past applications of cost-effectiveness.

6.3.1 Reduction of Utility Pole Accidents

The work done by Zegeer and Cynecki (1984) addressed the problem of the high rate of accidents and fatalities involving utility poles. Possible countermeasures that were

considered included placing utility lines underground, increasing the lateral offset of poles, reducing the number of poles, a combination of increased lateral offset and reduced pole density, and breakaway poles.

To determine the effectiveness, or benefit, of each possible countermeasure, a model to predict utility pole accidents for various combinations of utility poles and roadway features was developed from statistical analyses of utility pole accident data. The model was used to predict accident frequency and severity for various combinations of utility pole and roadway features, and the reduction in accident frequency and severity resulting from various countermeasures.

In their analysis, costs are broken into two categories, direct and indirect. Direct costs include capital investment, maintenance and overhead, and right-of-way acquisition, while traffic delay and detour costs, utility company liability costs, utility service interruption costs, and other societal costs are indirect. Note that the direct costs are easily measurable in dollars, while the indirect costs are not. The authors point out that indirect costs may change drastically from one site to another for the same type of countermeasure. Indirect costs were therefore not included directly as costs, but reduction of these costs were inferred as a benefit of reduction in frequency and severity of accidents.

Having identified costs and benefits, the authors used B/C ratios and incremental analysis to develop guidelines for selection of cost-effective countermeasures for various types of roadway sections.

6.3.2 Interchange Design Configurations

Wattleworth and Ingram (1972) developed a cost-effectiveness method for analysis and comparison of alternative interchange design configurations. The method was intended to be applied to redesign of existing interchanges, as well as design of new ones. Costs considered were the initial costs of construction and land acquisition, dollar values obtained from the Florida DOT. The measure of effectiveness, or benefit, was the capacity of the interchange.

The report noted that application of cost-effectiveness and incremental analysis is highly desirable in that it allows the designer to use the results of the analysis of one design to lead to the next design, thus developing a sequential design process. It also noted that with the application of cost-effectiveness, the greatest return is obtained for the capital invested.

6.3.3 Roadside Safety Improvement Programs

Glennon (1974) developed a cost-effectiveness approach to prioritizing roadside safety improvements on freeways.

Costs were the annualized first costs of doing the

improvements, measured in dollars. A six percent interest rate was used in converting initial to annual costs, but Glennon notes that comparative program analysis is fairly insensitive to the interest rate. He also points out that maintenance costs could significantly alter the relative priorities for the various improvements, because some improvements reduce maintenance requirements while others increase them. However, incremental maintenance costs are hard to identify and are therefore usually considered outside the initial cost-effectiveness analysis.

To measure the effectiveness of different approaches, a probabilistic hazard index was developed. The benefit was the hazard reduction achieved by each alternative.

6.3.4 Highway Design Consistency Related to Safety

This work done by Glennon and Harwood (1978) uses the same cost-effectiveness approach as the examples just described, but has a much broader scope. The basis for the work is the premise that AASHTO policies address highway design with a piecemeal, rather than systematic, approach. This leads to design inconsistencies which frequently violate driver expectations, and thus diminish highway safety.

The authors note the increasing use of a systems engineering approach to design, in which the design process is focused on the total end result rather than merely the efficiency of each component part. Their view that "the primary principle in applying the systems approach to design

is to maximize system performance for a given cost or to minimize cost for a given performance" leads logically to cost-effectiveness. To enhance the usefulness of any cost-effectiveness method, they recommend that other optimization techniques, such as linear programming, dynamic programming, game theory, etc., be incorporated into the cost-effectiveness analysis. They also recommend that the method be "compartmentalized" so that subelements can easily be adapted to specific used needs without having to alter the entire method.

Glennon and Harwood recommend two different, but complementary, applications of cost-effectiveness analysis for highway safety. The measure of effectiveness in both is reduction in hazard, but one uses a predictive model to identify potential high accident locations, while the other identifies hazardous sites using actual accident-frequency data.

Chapter 7

PROPOSED METHOD

The proposed method, which uses ratings developed from expert opinion to correct ramp deficiencies which may lead to trucks rolling over, consists of the following steps:

1. Inventory of characteristics of ramps in jurisdiction
2. Classification of characteristics
3. Assignment of a hazard rating to each characteristic
4. Calculation of a Notice Rating for each ramp
5. Listing of possible corrective measures
6. Application of cost-effectiveness procedures

Step 1 requires on-site inspections. Steps 2 through 6 can be done manually or by computer.

7.1 Inventory of Ramp Characteristics

The first step is to compile an inventory of the following characteristics of each ramp and deceleration lane in the jurisdiction: (1) speed posted for deceleration lane, (2) deceleration lane length, (3) downgrade on deceleration lane, (4) type of transition between tangent section and curve, (5) type of compound curve, if any, (6) speed posted

for ramp, (7) radius of ramp curve or curves, (8) superelevation, (9) presence of outside curb, (10) presence of drop greater than 4 inches at pavement edge, (11) cross-slope difference between roadway and shoulder, (12) lane width, and (13) downgrade on ramp. The average operating speed on the highway and on the ramp must also be noted, and the inventory taker must indicate whether the ramp leads to local streets or if it is part of an interchange leading to another highway. Curve lengths must also be measured for possible use later in the cost-effectiveness analysis.

Standard procedures for making field measurements to determine lengths, slopes, and other geometric characteristics can be followed. However, it is recommended that satellite Global Positioning System (GPS) technology be used whenever possible. GPS technology, which is being adopted by most highway operating agencies, allows measurements to be made quickly and easily. This results in cost savings for the agency and greater safety for the inventory taker in the field, while the measurements are more accurate than what can be obtained manually.

7.2 Classification of Characteristics

The characteristics of the ramp and deceleration lane must be classified into the same categories of characteristics as those evaluated in the survey of experts. This will allow assignment of the appropriate hazard rating

to each characteristic and compilation of a Notice Rating for the ramp in the two subsequent steps.

7.2.1 Road Surface Condition

Road surface condition on the deceleration lane must be classified as dry, wet, snowy or icy. This characteristic is not inventoried. Rather, the appropriate category to use at a specific location is a policy decision made by local engineers based on their knowledge of ambient weather conditions.

An assumption has been made that the pavement surface condition on the deceleration lane will be a major factor in whether a truck can decelerate to an appropriate speed for the ramp, and that the pavement surface on the deceleration lane is therefore more significant than on the ramp itself.

7.2.2 Adequacy of Deceleration Lane Length

The inventoried characteristics relevant in determining the adequacy of a deceleration lane are highway operating speed, speed posted for ramp, downgrade on deceleration lane, and length of the deceleration lane.

7.2.2.1 Calculation of Required Length

The required length of the deceleration lane is computed as

$$(7.1) \quad d = 1.47 V_1^2 t + \frac{(V_1^2 - V_2^2)}{30 (f \pm G)}$$

where d = required length of deceleration lane (feet)
 V_1 = highway operating speed (miles/hour)
 V_2 = posted speed for ramp (miles/hour)
 t = perception-reaction time (seconds)
 G = gradient (%/100; + for upgrade,
- for downgrade)
 f = friction factor (dimensionless)

AASHTO recommends use of a perception-reaction time, t , of 2.5 seconds, which reflects the behavior of the general population. While experienced truck drivers are likely to react more quickly than most other drivers, this is counteracted by the inherent delay of approximately 0.5 seconds in brake application in the air-brake systems used in most tractor-trailers. Therefore, $t = 2.5$ seconds is appropriate in this application.

Every agency makes its own policy decision on the appropriate value of friction factor, f , to use in highway design. AASHTO publishes recommended values of f , but they are based on deceleration rates for a locked-wheel stop by a passenger car on a wet roadway, and should not be used where truck safety is a concern. Harwood (1990) recommended the use of $f = 0.16$, based on test results reported by Fancher (1986) which reflect the worst braking performance of an empty tractor-trailer with good tires on a road that is in poor condition and wet. A friction factor value of 0.16 is

considerably below the AASHTO-recommended range of 0.26 to 0.40, and is very conservative. However, AASHTO (1984) states that the f values used for design should be nearly all-inclusive, rather than average. Therefore, use of f based on worst performance actually conforms with AASHTO policy as well as with recommendations based on research in truck operations, and $f = 0.16$ is recommended.

Research has shown that trucks equipped with anti-lock brakes achieve deceleration rates very close to those of passenger cars (Fancher 1986). Therefore, if anti-lock brakes are mandated and installed in all trucks at some time in the future, the AASHTO-recommended values of f would be appropriate in the calculation of deceleration lane requirements for trucks. The National Highway Traffic Safety Administration (NHTSA) has issued a rule requiring antilock brakes on all new truck tractors after March 1, 1997 and all new air-braked trailers and single-unit trucks after March 1, 1998, but implementation may be delayed by Congress.

7.2.2.2 Determination of Adequacy

The actual length of the deceleration lane, L_{act} , is compared to the calculated required length, L_{req} . If L_{act} is less than L_{req} , the percent deficiency is calculated as

$$(7.2) \quad \% \text{ Deficiency} = \frac{L_{req} - L_{act}}{L_{req}} \times 100$$

This value is then subtracted from 100 percent to get percent adequacy. If L_{act} is greater than or equal to L_{req} , the

deceleration lane is 100 percent adequate.

7.2.2.3 Classification of Deceleration Lane

In addition to adequacy of length, deceleration lanes are classified by highway operating speed. The classifications for highway speed are: (1) less than 40 miles/hour, (2) 40 to 60 miles/hour, and (3) greater than 60 miles/hour.

Within these speed categories, the deceleration lanes are classified into subgroups based on adequacy of length. The subgroups are 100 percent adequate, 80 percent adequate, 60 percent adequate, 40 percent adequate, and 20 percent adequate. To account for rounding off in calculations, it is recommended that a tolerance of -1 percent be used in the classifications. For example, if a deceleration lane is found to be 39 percent adequate, it should be classified as 40 percent adequate, while a deceleration lane that is 38 percent adequate should be classified as 20 percent adequate.

7.2.3 Downgrade on Deceleration Lane

The deceleration lane downgrades are classified as 0 percent, 1-2 percent, 3-4 percent, and 5-6 percent.

This characteristic has already been used in the calculation of required deceleration lane length, but it is also considered separately to reflect the difficulty in decelerating on a downgrade and the possibility of unintentional acceleration.

7.2.4 Type of Transition from Tangent to Curve

The transition from a normal crown cross-section on the deceleration lane to a superelevated cross-section on a curved ramp is classified according to the following categories: (1) spiral, (2) compound curve, (3) simple curve; superelevation developed partly on tangent and partly on curve, (4) simple curve; all superelevation developed on tangent; i.e. entire curve has e_{max} .

7.2.5 Curve Radius

Determination of the adequacy of the radius of the curve on the ramp is based on the speed posted for the ramp and the existing superelevation.

The equation that describes the relationship between the radius, superelevation, speed and friction factor is

$$(2.1) \quad e + f = \frac{V^2}{15 R}$$

where e = superelevation (dimensionless)

f = friction factor

V = speed (miles/hour)

R = radius (feet)

This equation is used by AASHTO to calculate required superelevation for a specified radius and design speed. In this application, it is rewritten to solve for the minimum required radius:

$$(7.3) \quad R_{min} = \frac{V^2}{15 (e + f)}$$

In using this equation, V is the speed posted for the ramp. and f is equal to 0.16, as discussed in Section 7.2.2. If most of the superelevation has been developed prior to the curve, e_{max} should be used in the calculation. On ramps where a significant portion of the superelevation is developed on the ramp itself, e at the beginning of the curve should be used.

Once the required radius has been calculated, the following steps are taken to classify the adequacy of the existing radius.

First, the percent deficiency is calculated as

$$(7.4) \quad \% \text{ Deficiency} = \frac{R_{req} - R_{act}}{R_{req}} \times 100$$

This value is then subtracted from 100% to get percent adequacy. If R_{act} is greater than R_{req} , the radius is 100% adequate. Classification of radius adequacy is very similar to classification of deceleration lane adequacy described in Section 7.2.2, with classification both by adequacy and by speed. The classifications for adequacy of curve radius are 100% adequate, 80%, 60%, 40% and 20% adequate. A tolerance of -1 percent should be used. The speed classifications reflect the speed posted on the ramp. The categories are V less than 20 miles/hour, $20 < V < 40$ miles/hour, and V greater than 40 miles/hour.

7.2.6 Ramp Downgrade

The ramp downgrade is classified as 0 percent, 1 to 2 percent, 3 to 4 percent, 5 to 6 percent, and greater than 6 percent.

7.2.7 Lane Width

Categories for classification of lane width are greater than or equal to 13 feet, 12 feet, 11 feet, 10 feet, 9 feet and less than or equal to 8 feet.

7.2.8 Type of Compound Curve

The terms used in classifying a compound curve are sharp (S) and flat (F), where sharp indicates a curve with a relatively short radius and high degree of curvature, and flat indicates a curve with a relatively long radius and lesser degree of curvature.

Ramps on which a compound curve exists are classified by the relative location of the long and short radius curves. Classifications are S → F, F → S, S → F → S and F → S → F.

7.2.9 Presence of an Outside Curb

Whether or not a curb is present along the outside of the curve must be noted. The classifications in this category are simply yes or no.

7.2.10 Drop at Pavement Edge

Presence of a drop greater than 4 inches at the pavement edge must be noted. Classifications are yes or no.

7.2.11 Cross-slope Difference

The inventoried difference in cross-slope between the shoulder and the superelevated roadway is classified as <6 percent, 6 percent, 8 percent, 10 percent or 12 percent.

7.2.12 Interchange

The ramp must be classified as to whether or not it is part of an interchange.

7.3 Assignment of Hazard Ratings

The values in Table 6 are the synthesis of the survey of experts on the relative hazard of ramp and deceleration lane characteristics. A Hazard Rating taken from this table shall be assigned to every pertinent characteristic of every ramp being evaluated.

7.4 Calculation of Notice Rating and Additional Factors

The Notice Rating for a ramp is the sum of the Hazard Ratings assigned to the characteristics of the ramp. Additional multiplicative factors may be applied later to ramps that are part of the National Network or are on hazardous materials routes or are components of interchanges, to increase the likelihood of attention being focused on these ramps.

7.4.1 Calculation of Notice Rating

An example of the calculation of the Notice Rating is as follows.

Consider a ramp posted for 30 mph, leading to a local street from a highway on which the average operating speed is 65 mph. The deceleration lane, which has a 2 percent downgrade, has been determined to be 80 percent adequate in length. The curve radius has also been found to be 80 percent adequate. The ramp has 12-foot lanes, a 1 percent downgrade, and a cross-slope difference of 6 percent. The transition from a normal crown cross-section to full superelevation is achieved partly on the tangent and partly on the curve. The policy of the agency which is responsible for this ramp is to design for roads under wet conditions. Using Table 6, the Notice Rating is calculated as follows:

<u>Characteristic</u>	<u>Hazard Rating (HR)</u>
Decel. lane length 80%; V > 60mph	8
Decel. lane downgrade 2%	7
Decel. lane pavement	18
Transition curve	148
Curve radius 80%; V = 30 mph	165
12 ft. lanes	109
Ramp downgrade 1%	95
Cross-slope diff. 6%	<u>116</u>
Notice Rating =	662

7.4.2 Additional Factors

These factors, which range in value from 1.1 to 2.0, reflect the increased likelihood of a rollover and/or the greater severity of the consequences of a rollover when:

- (a) the ramp is part of an interchange
- (b) the ramp is part of the National Network designated for STAA trucks
- (c) the ramp is on a route on which hazardous materials are transported

The factors are applied to the benefit-cost ratio in the cost-effectiveness analysis, rather than directly to the Notice Rating of the ramp. Direct application to the Notice Rating would distort the benefit-cost ratio by making the effectiveness of corrective measures, or benefit, disproportionately small. This is avoided by using the factors to enhance the benefit-cost ratio itself.

A factor of 1.4 is used when the ramp is part of an interchange. This represents expert consensus on the increased level of hazard on a interchange, where a driver may deliberately maintain as much speed as possible in preparation for merging. The factors that are appropriate for ramps that are part of the National Network or are on hazardous materials routes vary from location to location, and must, therefore, be determined by local engineers. While an argument could be made for use of varying factors for ramps on the National Network or on hazardous materials

routes, depending on specific conditions at each ramp and the relative amount of harm a truck rollover might do, the process of assigning varying factors would be susceptible to political pressures and personal preferences. Such influences would destroy the integrity of the cost-effectiveness analysis. Therefore, these factors should remain constant within a jurisdiction.

Use of the factors is demonstrated in the cost-effectiveness application described in Section 7.6.

7.5 Listing of Possible Corrective Measures

All corrective measures which an agency might consider using must be listed in order of increasing cost. Cost of installation or construction only should be considered here, under the assumption that ordinary maintenance will prevent deterioration of the ramp after improvements have been made. The assumption that the ramp will be properly maintained may not be realistic, but whether a sufficient level of maintenance is provided for highways in a jurisdiction is the not the issue here. The cost of any corrective measures that require maintenance beyond the ordinary, e.g. a sign with flashing lights where bulbs might burn out, should include maintenance in the cost.

Possible corrective measures can range from low-cost to total reconstruction of the ramp. Where appropriate, costs should be noted in terms of unit length, area or volume, to

allow calculation of total cost for each measure at each ramp.

The corrective measures considered in the example in Section 7.6 are cumulative, rather than corrections of one deficiency at a time. This is a more realistic approach rather investigating the cost-effectiveness of individual corrections, one at a time. Any highway agency will try to minimize the cost of getting a crew and equipment out to a job site, and the cost of closing a ramp, by trying to do as much as possible at one time.

7.6 Application of Cost-Effectiveness Procedures

The cost-effectiveness procedure is demonstrated in the following example. Ramps used in this demonstration are described in Section 7.6.1. The Hazard Rating (HR) for each characteristic listed is given in parentheses. Possible corrective measures, and their effect on each ramp, are discussed in Section 7.6.2. Cost-effectiveness calculations and results are given in Section 7.6.3. To simplify the demonstration, it is assumed that all ramps are approximately the same length.

7.6.1 Description of Ramps

Consider four ramps in a jurisdiction where the policy is to design for wet roadway surfaces, and where factors of 1.3 and 1.8 have been adopted for ramps on the National

Network and on hazardous materials routes respectively.

Ramp 1:

Average highway speed = 55 mph; leads to local street. Deceleration lane has a 4% downgrade (HR = 17) and is 60% adequate in length (HR = 17). Superelevation is developed partly on the tangent and partly on the curve (HR = 148). Ramp is posted for 25 mph and the radius is 40% adequate (HR = 448). Lanes are 11 feet wide (HR = 214). Ramp downgrade is 1% (HR = 95) and cross-slope difference at pavement edge is 8% (HR = 218). Wet Pavement (HR = 18).

The Notice Rating is 1175.

Ramp 2:

Average highway speed = 65 mph; interchange; on National Network. Deceleration lane has no downgrade (HR = 0) and is 80% adequate in length (HR = 8). A compound curve is used for transition from deceleration lane to superelevated curve (HR = 173). Ramp is posted for 35 mph and the radius is 80% adequate (HR = 165). Lanes are 13 feet wide (HR = 18). Ramp has 3% upgrade (HR = 22; consider as 0% downgrade). Cross-slope difference is 6% (HR = 116) and the ramp has a sharp-to-flat compound curve (HR = 236). Wet pavement (HR = 18).

The Notice Rating is 756.

Ramp 3:

Average highway speed = 65 mph; leads to local street; on National Network and Hazmat route. Deceleration lane has a 1% downgrade (HR = 7) and is 80% adequate in length (HR = 8). A compound curve is used for the transition from deceleration lane to superelevated curve (HR = 173). The ramp is posted for 25 mph and the radius is 60% adequate (HR = 263). Lanes are 13 feet wide (HR = 18). The ramp has a 4% downgrade (HR = 217) and a flat-to-sharp compound curve (HR = 261). Wet pavement (HR = 18).

The Notice Rating is 965.

Ramp 4:

Average highway speed = 60 mph; leads to local street; on National Network. Deceleration lane has a 1% downgrade (HR = 7) and is 60% adequate (HR = 17). Superelevation is developed partly on the tangent and partly on the curve (HR = 148). Ramp is posted for 25 mph and the radius is 60% adequate (HR = 263). Lanes are 12 feet wide (HR = 109). The ramp has a 6-inch drop at the pavement edge (HR = 398), a cross-slope difference of 8% (HR = 218), and no downgrade (HR = 22). Wet pavement (HR = 18).

The Notice Rating is 1200.

7.6.2 Corrective Measures

Following are the corrective measures that the local operating agency would consider implementing. The costs given are rough estimates based on cost figures currently used by the Connecticut Department of Transportation (Means 1992) and include materials, labor and the contractor's overhead costs. The measures are listed in order of increasing cost.

Ordinary highway signs are by far the lowest cost corrective measure. However, their effectiveness depends entirely on the inclination of drivers to take messages seriously and heed warnings. Therefore, it is recommended that signs not be considered in the cost-effectiveness process, but that they be installed and their effectiveness evaluated before cost-effectiveness analysis is applied to more costly measures.

The following corrective measures are considered in this analysis.

A. Build-up and stabilize shoulder along curve

This measure will correct the 6-inch drop at pavement edge on Ramp 4. The cost is approximately \$4000 for borrow and compacting plus \$2000 for soil stabilization measures, for a total cost of \$6000.

B. Flatten cross-slope between superelevated curve and shoulder, + Measure A

In addition to the deficiency corrected in measure A, and assuming that the cross-slope difference will be flattened to less than 6%, this measure will remove the hazard caused by excessive cross-slope difference found on Ramps 1, 2 and 4. Assuming all three ramps have the same superelevation, total approximate costs will be \$20,000 for ramp 1, \$18,000 for ramp 2, and \$28,000 for ramp 4.

C. Increase superelevation on ramp, + Measure B

To simplify this example, and recognizing that there is a limit to the amount of superelevation that can be incorporated into a curve and that superelevation cannot fully compensate for an inadequate radius, it is assumed that increasing the superelevation will improve any radius deficiency by 20%. In this example, increased superelevation would be beneficial on all four ramps. Total approximate costs would be \$55,000 on Ramp 1; \$65,000 on Ramp 2; \$60,000 on Ramp 3; and \$78,000 on Ramp 4.

D. Spiral transition and increased superelevation, + Measure C

This measure provides a smoother, and therefore

safer, transition from the deceleration lane into the curved ramp, and also corrects the deficiencies addressed in Measures A, B and C. Total approximate costs are \$95,000 on Ramp 1; \$140,000 on Ramp 2; \$115,000 on Ramp 3; and \$130,000 on Ramp 4.

E. Increase Deceleration Lane Length to Fully Adequate for Posted Speed, + Measure D

Safety on all the ramps would be enhanced by correction of deceleration lane deficiencies. Total approximate costs are \$125,000 on Ramp 1; \$155,000 on Ramp 2; \$140,000 on Ramp 3; and \$160,000 on Ramp 4.

F. Complete Reconstruction of Ramp and Deceleration Lane (Including Transition Curve). Improve Downgrade by 2% Where Relevant

Total approximate costs are \$200,000 for Ramp 1; \$235,000 for Ramp 2; \$230,000 for Ramp 3; and \$210,000 for Ramp 4.

7.6.3 Cost-Effectiveness Analysis

The matrix for the cost-effectiveness analysis is shown in Table 8. Improvements A through F, and costs, are those described in Section 7.6.2. Note that benefit/cost ratios for Ramps 2, 3 and 4 are multiplied by 1.3 to reflect that they are part of the National Network, and the ratios for

Table 8. Cost-Effectiveness Matrix

RAMP		Corrective Measures			
		DO NOTHING	A	B	C
1	NR	1175	1175	957 (4)	772 (7)
	Cost (\$1000)	0	0	20	55
	Δ B	0	0	218	403 185
	Δ C	0	0	20	55 35
	ΔB/ΔC	0	0	10.90	7.33 5.29
	Enhanced ΔB/ΔC	0	0	10.90	7.33 5.29
2	NR	756	756	640 (3)	483 (6)
	Cost (\$1000)	0	0	18	65
	Δ B	0	0	116	273 157
	Δ C	0	0	18	65 47
	ΔB/ΔC	0	0	6.44	4.20 3.34
	Enhanced ΔB/ΔC	0	0	11.73	7.64 6.08
3	NR	965	965	965	867
	Cost (\$1000)	0	0	0	60
	Δ B	0	0	0	98
	Δ C	0	0	0	60
	ΔB/ΔC	0	0	0	1.63
	Enhanced ΔB/ΔC	0	0	0	3.82
4	NR	1200	802 (1)	584 (2)	486
	Cost (\$1000)	0	6	28	78
	Δ B	0	398	616 218	714 316 98
	Δ C	0	6	28 22	78 78 50
	ΔB/ΔC	0	66.33	22.00 9.91	9.15 4.39 1.96
	Enhanced ΔB/ΔC	0	86.23	28.6 12.88	11.9 5.71 2.55

(continued on next page)

Table 8. Cost-Effectiveness Matrix (continued)

RAMP		Corrective Measures								
		D			E			F		
1	NR	641			624			216 (8)		
	Cost (\$1000)	95			125			200		
	Δ B	534	316	131	551	333	148	959	741	556
	Δ C	95	75	40	125	105	70	200	180	145
	ΔB/ΔC	5.62	4.21		4.41	3.17		4.80	4.12	
	Enhanced ΔB/ΔC	5.62	4.21	3.28	4.41	3.17	2.11	4.80	4.12	3.83
2	NR	327 (9)			319			199 (11)		
	Cost (\$1000)	140			155			235		
	Δ B	429	313	156	437	321	164	557	441	284
	Δ C	140	122	75	155	137	90	233	217	170
	ΔB/ΔC	3.06	2.56		2.82	2.34		2.37	2.03	
	Enhanced ΔB/ΔC	5.58	4.67	3.78	5.13	4.26	0.97	4.31	3.70	2.45
3	NR	711			703			90 (5)		
	Cost (\$1000)	115			140			230		
	Δ B	254			262			875		
	Δ C	115			140			230		
	ΔB/ΔC	2.21			1.87			3.80		
	Enhanced ΔB/ΔC	5.17			4.38			8.90		
4	NR	355 (10)			338			206 (12)		
	Cost (\$1000)	130			160			210		
	Δ B	845	447	229	862	464	246	994	596	378
	Δ C	130	124	102	160	154	132	210	204	182
	ΔB/ΔC	6.50	3.60		5.39	3.01		4.73	2.92	
	Enhanced ΔB/ΔC	8.45	4.69	2.92	7.00	3.92	0.74	6.13	3.80	2.42

Ramps 2 and 3 have additionally been multiplied by 1.4 and 1.8 to reflect an interchange and a hazardous materials route respectively.

In this cost-effectiveness analysis, benefit, or effectiveness, is measured by the reduction in the Notice Rating on a ramp if the corrective measure being considered is implemented. Cost is the actual dollar cost of implementing the measure.

In the first iteration of the analysis, cost-effectiveness is computed by comparing increments of benefit and cost resulting from each possible improvement to the "do-nothing" option. The highest "enhanced B/C ratio" is 86.23 when measure A is applied to Ramp 4. This indicates that the most cost-effective action to take is to build up and stabilize the shoulder along the curve on Ramp 4. This option is more cost-effective than doing nothing; therefore the option of doing nothing drops out of consideration for this ramp.

Correcting the shoulder will reduce the Notice Rating on Ramp 4 to 802. This new Notice Rating now becomes the basis for comparison on Ramp 4, while the analysis for the other ramps remains unchanged.

The second iteration produces a high "enhanced B/C ratio" of 12.88, also on Ramp 4. This indicates that if additional funds are available, it would be cost-effective to reduce the cross-slope difference, as well as correct the

shoulder on Ramp 4. The Notice Rating on Ramp 4 would now be reduced to 584, and this becomes the basis for comparison for the cost-effectiveness of any other action on that ramp. Notice Ratings on Ramps 1, 2 and 3 remain unchanged.

The third iteration produces a high "enhanced B/C ratio" of 11.73 when Measure B is applied to Ramp 2. This indicates that the next most cost-effective action the agency could take, if funds are available, would be to reduce the cross-slope difference and stabilize the shoulder on Ramp 2. This would bring the Notice Rating on Ramp 2 down to 640; this new Notice Rating now becomes the basis for further analysis on Ramp 2. The option to do nothing and option A are not cost-effective on Ramp 2, and they drop out of consideration.

The procedure is repeated until all alternative corrective measures on all ramps have either been ranked in order of their cost-effectiveness, or have been shown not to be cost-effective and eliminated from consideration.

Results of the cost-effectiveness analysis of the four ramps and seven possible corrective measures, including the option to do nothing, are shown in Table 9. Note that the only option that is cost-effective on Ramp 3 is total reconstruction of the ramp.

Note also that Measure E, which involves correction of deceleration lane deficiencies, is not cost-effective on any ramp. It is possible that the relatively low Hazard Ratings for deceleration lane characteristics influenced the cost-

effectiveness analysis as far as this measure is concerned, because the low Hazard Ratings would result in relatively little change in the Notice Rating if the measure is implemented.

Table 9. Results of Cost-Effectiveness Analysis

<u>Iteration</u>	<u>Ramp</u>	<u>Action to be Taken</u>
1	4	Measure A
2	4	" B
3	2	" B
4	1	" B
5	3	" F
6	2	" C
7	1	" C
8	1	" F
9	2	" D
10	4	" D
11	2	" F
12	4	" F

The results appear to confirm that cost-effectiveness analysis tends to favor lower cost measures. Note, however, that the amount of any corrective measure that must be taken at any single location has a bearing on both cost and

benefit, and therefore also on the cost-effectiveness. In this example, all ramps were assumed to be approximately the same length. This would probably not be the case in an actual analysis, and length differences would be a significant factor in the cost of implementing the corrective measures.

The final result of a cost-effectiveness analysis is a priority listing of projects and a curve showing level of benefit for any level of expenditure. Results for this example are shown in Table 10 and Figure 9 respectively.

Table 10. Results of Cost-Effectiveness Analysis

<u>PROJECT</u>	<u>(\$1000) COST</u>	<u>(\$1000) CUM COST</u>	<u>Δ BENEFIT</u>	<u>CUM Δ BENEFIT</u>
4A	6		398	
4B	28	34	218	616
2B	18	52	116	732
1B	20	72	218	950
3F	230	302	875	1825
2C	65	367	157	1982
1C	55	422	35	2017
1F	200	622	556	2573
2D	140	762	156	2729
4D	130	892	229	2958
2F	235	1127	128	3086
4F	210	1337	149	3235

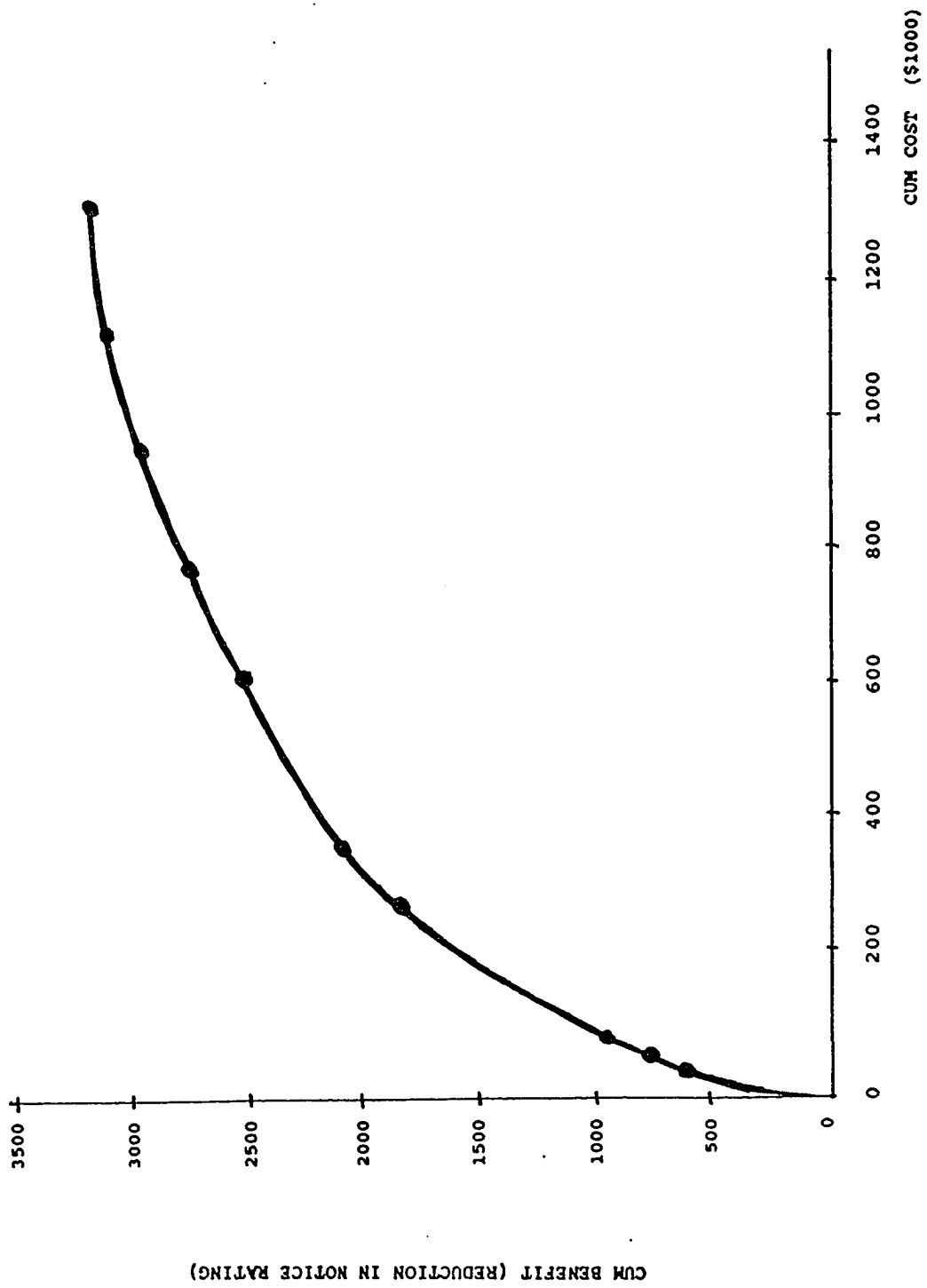


FIG. 9 CUMULATIVE COST-EFFECTIVENESS CURVE

Chapter 8

RECOMMENDATION FOR FUTURE RESEARCH

The work presented in this dissertation provides a basis for continued work in the future. There are two directions in which further analysis can be done.

First, the survey of experts should be expanded to provide more input for the development of the hazard ratings. This should result in greater confidence in the consensus, while analysis of variations in response may lead to a better understanding of what constitutes a hazard. Several experts who participated in this work suggested that experienced truck drivers be surveyed to elicit their opinions on the level of hazard presented by various ramp characteristics. Such a survey would be valuable in two ways. First, it would provide additional data for use in determining hazard ratings. Secondly, a comparison of opinions of those who use the ramps with opinions of those who design them may provide additional insight into the nature of hazards.

Another direction for continued work is to compare the actual accident histories of ramps with the hazard ratings based on expert opinion. Comparing what actually happens at a ramp with what an "expert" says should happen may provide valuable insight into what causes trucks to roll over.

Points of significant disagreement between accident history and predictions should be explored to uncover the reason for the disparity, while points of agreement will tend to confirm the expertise of the experts.

Deficiencies in the format of the questionnaire used in the survey of experts became apparent as the Hazard Ratings were being synthesized. The work done here would be improved by developing weighting factors for the importance of deceleration lane characteristics. It is also recommended that Hazard Ratings for ramp pavement surface be developed.

APPENDIX A
Questionnaire and Cover Letter

THE CITY COLLEGE
OF
THE CITY UNIVERSITY OF NEW YORK
NEW YORK, N.Y. 10031

SCHOOL OF ENGINEERING
DEPARTMENT OF CIVIL ENGINEERING

(212) 690-

May 9, 1991

Dear

I am a Ph.D. student in Transportation Engineering at the City College of New York, working on a dissertation entitled "Reducing Truck Rollovers on Highway Ramps: A Cost Effectiveness Approach."

I have come across your work, particularly on , while reading the literature on deficiencies in geometric design practices relative to trucks. For this reason I am requesting the benefit of your expertise.

My dissertation will have three basic parts:

1. Creation of a Hazard Index that quantifies the relative degree of rollover hazard to trucks, presented by the geometric characteristics of the ramp.

2. Development of a procedure that will guide any responsible agency in taking an inventory of characteristics of existing ramps in its jurisdiction. The agency can then use the Hazard Index to evaluate the relative rollover hazard that exists on each of these ramps. This will produce a listing of ramps, ranked in order of relative hazard, appropriate for cost-effectiveness analysis.

3. Development of a procedure to evaluate the cost-effectiveness of various measures which would alleviate the ramp deficiencies.

The Hazard Index will be used twice:

first, to determine the existing level of relative hazard;

second, in the cost-effectiveness procedure, where benefit will be measured by reduction in relative hazard.

The Hazard Index is the heart of this work, and it is the reason I am writing to you. In spite of the large body of

literature on truck rollovers, the variability of study results make it unreasonable for me to ask you to specify actual numerical values to describe the level of hazard at a ramp. Therefore, I intend to use your expert opinion on the relative hazard of various geometric characteristics.

I would appreciate your contribution to the evaluation of the Hazard Index by filling out the enclosed questionnaire. If you have any questions, please call me at (212) 650-8015.

Results of my survey of experts will be sent to you, and I will be happy to share any other parts of the work at your request.

Yours truly,

Beatrice Isaacs

Questionnaire on the Relative Influence of Geometric
Characteristics on Truck Rollovers

Introduction

Assumptions

1. Trucks using the ramp are typically 102"-wide tractor-trailers, with a single 48 to 53 foot semitrailer, and a rollover threshold ranging approximately from .24g to .35g, depending on cargo and loading.

2. The trucks are in acceptable operating condition and are being operated by competent, unimpaired drivers. Vehicle and driver deficiencies are not being considered here.

3. The roadway and signs are properly maintained. The pavement is free of potholes; signs are clean, reflective, meet MUTCD guidelines for size and lettering, and are not obscured.

4. The ramp being evaluated in the first section of the questionnaire is an exit ramp, at the end of which the driver is expected to bring the truck to a stop. The extent, if any, to which an interchange ramp presents a greater hazard is considered on Page 8.

5. "Ideal" conditions are those set by the most stringent requirements of physics, AASHTO and recent literature on AASHTO deficiencies relative to trucks.

Variables in Hazard Index

The Hazard Index considers the extent to which deficiencies in the following variables create or magnify hazardous conditions on a highway exit ramp.

Deceleration Lane Characteristics

- A - length of deceleration lane
- B - road surface condition
- C - downgrade on deceleration lane

Ramp Characteristics

- D - type of transition curve
- E - type of compound curve
- F - curve radius for existing superelevation
- G - presence of an outside curb
- H - drop at pavement edge
- I - cross-slope difference at lane edge
- J - lane width
- K - downgrade on ramp
- L - curve length

The hazard presented by a truck being forced to enter the ramp at an excessive rate of speed because of an inadequate deceleration lane is evaluated by the first three variables (A, B, C). The characteristics of the ramp itself (variables D through L) are evaluated relative to their adequacy for the posted speed on the ramp.

Side-friction factor (f) is not listed as a separate variable, but is considered implicitly through the evaluation of other variables. When rollovers are considered, the concern is not whether there is sufficient side-friction to prevent sliding, but rather whether the unbalanced lateral acceleration exceeds the truck's rollover threshold.

Please use the following tables to indicate your opinion of the relative degree to which deficiencies in each of the characteristics listed above contribute to the likelihood of a truck rollover, with all other characteristics being held constant. For each characteristic, please circle the number, or range of numbers, that best represents your opinion.

A scale of 0 - 10 is used for all ratings, where 0 represents a "safe" condition and 10 represents the greatest degree of hazard for the characteristic being rated.

RatingsA - Length of Deceleration Lane

The adequacy of the existing length of the deceleration lane is expressed as a percentage of length needed, as determined by physics, AASHTO and recent literature, to decelerate from the highway operating speed to the speed posted for the ramp.

Highway Operating Speed < 40 mph

RELATIVE HAZARD

Length Adequacy (%)	Least Hazardous										Most Hazardous											
	0	1	2	3	4	5	6	7	8	9	10	0	1	2	3	4	5	6	7	8	9	10
100 (Best)	0	1	2	3	4	5	6	7	8	9	10	0	1	2	3	4	5	6	7	8	9	10
80	0	1	2	3	4	5	6	7	8	9	10	0	1	2	3	4	5	6	7	8	9	10
60	0	1	2	3	4	5	6	7	8	9	10	0	1	2	3	4	5	6	7	8	9	10
40	0	1	2	3	4	5	6	7	8	9	10	0	1	2	3	4	5	6	7	8	9	10
20 (Worst)	0	1	2	3	4	5	6	7	8	9	10	0	1	2	3	4	5	6	7	8	9	10

Highway Operating Speed 40 to 60 mph

Length Adequacy (%)	Least Hazardous										Most Hazardous											
	0	1	2	3	4	5	6	7	8	9	10	0	1	2	3	4	5	6	7	8	9	10
100 (Best)	0	1	2	3	4	5	6	7	8	9	10	0	1	2	3	4	5	6	7	8	9	10
80	0	1	2	3	4	5	6	7	8	9	10	0	1	2	3	4	5	6	7	8	9	10
60	0	1	2	3	4	5	6	7	8	9	10	0	1	2	3	4	5	6	7	8	9	10
40	0	1	2	3	4	5	6	7	8	9	10	0	1	2	3	4	5	6	7	8	9	10
20 (Worst)	0	1	2	3	4	5	6	7	8	9	10	0	1	2	3	4	5	6	7	8	9	10

Highway Operating Speed > 60 mph

Length Adequacy (%)	Least Hazardous										Most Hazardous											
	0	1	2	3	4	5	6	7	8	9	10	0	1	2	3	4	5	6	7	8	9	10
100 (Best)	0	1	2	3	4	5	6	7	8	9	10	0	1	2	3	4	5	6	7	8	9	10
80	0	1	2	3	4	5	6	7	8	9	10	0	1	2	3	4	5	6	7	8	9	10
60	0	1	2	3	4	5	6	7	8	9	10	0	1	2	3	4	5	6	7	8	9	10
40	0	1	2	3	4	5	6	7	8	9	10	0	1	2	3	4	5	6	7	8	9	10
20 (Worst)	0	1	2	3	4	5	6	7	8	9	10	0	1	2	3	4	5	6	7	8	9	10

B - Road Surface Condition

Condition	RELATIVE HAZARD										
	Least Hazardous					Most Hazardous					
Dry	0	1	2	3	4	5	6	7	8	9	10
Wet	0	1	2	3	4	5	6	7	8	9	10
Snow	0	1	2	3	4	5	6	7	8	9	10
Ice	0	1	2	3	4	5	6	7	8	9	10

C - Downgrade on Deceleration Lane

Gradient (%)	RELATIVE HAZARD										
	Least Hazardous					Most Hazardous					
0	0	1	2	3	4	5	6	7	8	9	10
1-2	0	1	2	3	4	5	6	7	8	9	10
3-4	0	1	2	3	4	5	6	7	8	9	10
5-6	0	1	2	3	4	5	6	7	8	9	10

D - Type of Transition Curve

Type of Curve	RELATIVE HAZARD										
	Least Hazardous					Most Hazardous					
Spiral	0	1	2	3	4	5	6	7	8	9	10
Compound	0	1	2	3	4	5	6	7	8	9	10
Simple curve; part of 'e' developed on tangent, part on curve	0	1	2	3	4	5	6	7	8	9	10
Simple curve; all e developed on tan.; entire curve has e_{min}	0	1	2	3	4	5	6	7	8	9	10

E - Type of Compound Curve

Type of Curve	RELATIVE HAZARD										
	Least Hazardous					Most Hazardous					
sharp → flat	0	1	2	3	4	5	6	7	8	9	10
flat → sharp	0	1	2	3	4	5	6	7	8	9	10
sharp-flat-sharp	0	1	2	3	4	5	6	7	8	9	10
flat-sharp-flat	0	1	2	3	4	5	6	7	8	9	10

F - Curve Radius for Existing Superelevation

The adequacy of the existing radius is expressed as a percentage of the radius determined to be necessary by physics, AASHTO and recent literature.

Posted Speed on Ramp < 20 mph

Radius Adequacy (%)	RELATIVE HAZARD										
	Least Hazardous					Most Hazardous					
100 (Best)	0	1	2	3	4	5	6	7	8	9	10
80	0	1	2	3	4	5	6	7	8	9	10
60	0	1	2	3	4	5	6	7	8	9	10
40	0	1	2	3	4	5	6	7	8	9	10
20 (Worst)	0	1	2	3	4	5	6	7	8	9	10

Posted Speed on Ramp 20 to 40 mph

Radius Adequacy (%)	RELATIVE HAZARD										
	Least Hazardous					Most Hazardous					
100 (Best)	0	1	2	3	4	5	6	7	8	9	10
80	0	1	2	3	4	5	6	7	8	9	10
60	0	1	2	3	4	5	6	7	8	9	10
40	0	1	2	3	4	5	6	7	8	9	10
20 (Worst)	0	1	2	3	4	5	6	7	8	9	10

Posted Speed on Ramp > 40 mph

Radius Adequacy (%)	RELATIVE HAZARD										
	Least Hazardous					Most Hazardous					
100 (Best)	0	1	2	3	4	5	6	7	8	9	10
80	0	1	2	3	4	5	6	7	8	9	10
60	0	1	2	3	4	5	6	7	8	9	10
40	0	1	2	3	4	5	6	7	8	9	10
20 (Worst)	0	1	2	3	4	5	6	7	8	9	10

G - Presence of an Outside Curb

	RELATIVE HAZARD										
	Least Hazardous						Most Hazardous				
Curb present	0	1	2	3	4	5	6	7	8	9	10
(No curb = 0)											

H - Drop at Pavement Edge

	RELATIVE HAZARD										
	Least Hazardous						Most Hazardous				
Drop > 4"	0	1	2	3	4	5	6	7	8	9	10
(Drop < 4" = 0)											

I - Cross-Slope Difference at Pavement Edge

The change in cross-slope between roadway and shoulder can present a rollover hazard if it is excessive. The 1990 AASHTO Green Book recommends that this difference not exceed 8%.

Algebraic Diff. in Cross-Slope (%)	RELATIVE HAZARD										
	Least Hazardous						Most Hazardous				
6	0	1	2	3	4	5	6	7	8	9	10
8	0	1	2	3	4	5	6	7	8	9	10
10	0	1	2	3	4	5	6	7	8	9	10
12	0	1	2	3	4	5	6	7	8	9	10

J - Lane Width

Lane Width (ft)	RELATIVE HAZARD										
	Least Hazardous						Most Hazardous				
> 13	0	1	2	3	4	5	6	7	8	9	10
12	0	1	2	3	4	5	6	7	8	9	10
11	0	1	2	3	4	5	6	7	8	9	10
10	0	1	2	3	4	5	6	7	8	9	10
9	0	1	2	3	4	5	6	7	8	9	10
< 8	0	1	2	3	4	5	6	7	8	9	10

K - Downgrade on Ramp

Gradient (%)	RELATIVE HAZARD										
	Least Hazardous					Most Hazardous					
0	0	1	2	3	4	5	6	7	8	9	10
1-2	0	1	2	3	4	5	6	7	8	9	10
3-4	0	1	2	3	4	5	6	7	8	9	10
5-6	0	1	2	3	4	5	6	7	8	9	10
> 6	0	1	2	3	4	5	6	7	8	9	10

L - Curve Length

Given a hazardous ramp, a driver may be able to compensate if the ramp is short.

Use the table to indicate your opinion of how increasing length might affect the relative hazard.

Length (feet)	RELATIVE HAZARD										
	Least Hazardous					Most Hazardous					
100	0	1	2	3	4	5	6	7	8	9	10
200	0	1	2	3	4	5	6	7	8	9	10
300	0	1	2	3	4	5	6	7	8	9	10
400	0	1	2	3	4	5	6	7	8	9	10
500	0	1	2	3	4	5	6	7	8	9	10
600	0	1	2	3	4	5	6	7	8	9	10

Questions

1. This questionnaire considers two different groups of characteristics: those on the deceleration lane and those on the ramp. In your opinion, how great is the relative effect of each group on the likelihood of a truck rollover.

RELATIVE IMPORTANCE	
	Least Most
Deceleration Lane	0 1 2 3 4 5 6 7 8 9 10
Ramp	0 1 2 3 4 5 6 7 8 9 10
Sum of the relative importance of deceleration lane and ramp should equal 10.	

2. Deficiencies in some geometric characteristics of a ramp are possibly more likely than others to lead to a rollover. Using a scale of 0 ~ 10, indicate your opinion of the relative importance of each.

	RELATIVE IMPORTANCE
Ramp Characteristic	Least Most
Type of transition curve	0 1 2 3 4 5 6 7 8 9 10
Type of compound curve	0 1 2 3 4 5 6 7 8 9 10
Curve radius for existing 'e'	0 1 2 3 4 5 6 7 8 9 10
Presence of an outside curb	0 1 2 3 4 5 6 7 8 9 10
Drop at pavement edge	0 1 2 3 4 5 6 7 8 9 10
Cross-slope diff. at lane edge	0 1 2 3 4 5 6 7 8 9 10
Lane width	0 1 2 3 4 5 6 7 8 9 10
Downgrade on ramp	0 1 2 3 4 5 6 7 8 9 10
Curve length	0 1 2 3 4 5 6 7 8 9 10

3. The geometric hazards on an exit ramp may be magnified on an interchange ramp because of the truck driver's desire to maintain as much speed as possible in preparation for merging.

Circle the factor by which the rating for an exit ramp should be multiplied to reflect this additional hazard.

1.0 1.1 1.2 1.3 1.4 1.5 1.6 1.7 1.8 1.9 2.0

If you think the hazard on an interchange is more than twice that of an exit ramp, write the multiplier here.

4. Changing vehicle characteristics (e.g., more combination vehicles, heavier vehicles, etc.) may increase the relative hazard presented by geometric characteristics of the roadway.

Circle the factor by which a rating of hazards for vehicles on the road today should be multiplied to reflect this additional hazard.

1.0 1.1 1.2 1.3 1.4 1.5 1.6 1.7 1.8 1.9 2.0

If you think the hazard will be more than double, write the multiplier here.

If you have any comments, I would appreciate knowing them.

Please return to: Beatrice Isaacs
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138 St. & Convent Ave.
New York, N.Y. 10031
(phone: 212 - 650-8015)

Thank you for your expert assistance.

APPENDIX B
Participants in Survey of Experts

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Austin, Texas

APPENDIX C
Questionnaire Responses - Raw Data

Characteristic

	<u>0</u>	<u>1</u>	<u>2</u>	<u>3</u>	<u>4</u>	<u>5</u>	<u>6</u>	<u>7</u>	<u>8</u>	<u>9</u>	<u>10</u>
A-Decel. Lane											
Length 100%	18		2	1	1						
V<40 80%	4	7	5	3	2	1					
60%		3	5	4	4	4	3	1			
40%			1	5	3	3	3	2	4	3	
20%				2	4	2	2	1	3	2	6
40<V<60											
100%	18		1	1	1	1					
80%	1	7	3	4	3	2	1	1			
60%			4	2	5	3	4		4	1	
40%				1	4	2	5	1	4	4	2
20%					2	1	3		6		10
V>60											
100%	15		1		1	1	1				
80%		2	7	3	2	3	2	1	1		1
60%				2	4	3	5	2	3	1	2
40%						2	5	3	2	6	4
20%						1	1	1	3	1	15
B-Surface											
Dry	13	1	2	2	2		1				2
Wet		1	1	2	4	5	5	3			
Snow				1	4	1	2	8		1	1
Ice	1				1	1		1		3	15
C-Decel. Down.											
0%	15	1	1	1	1						
1-2%	1	6	7	6	3	1					
3-4%				3	5	8	5	2	2		
5-6%						1	4	3	4	4	7
D-Trans. Curve											
Spiral	10	1	3	2	2	1	1				
Compound		3	2	3	2	2	1	2	2		3
eTan&Curve	1		1	4		7	3	2	1	1	1
eOnCurve	2		2	1	1	5	3	2	1	2	2
E-Comp. Curve											
S-F	2	1	3		1	1	3	2	2	1	3
F-S		1	2	1	4	1	3	1	3	1	2
S-F-S						1	4	1	4	4	7
F-S-F	1	1	1	1		4	3		5	1	2

Characteristic

		<u>0</u>	<u>1</u>	<u>2</u>	<u>3</u>	<u>4</u>	<u>5</u>	<u>6</u>	<u>7</u>	<u>8</u>	<u>9</u>	<u>10</u>
F-Radius												
V<20	100%	14	2	3	2	1					1	
	80%	2	2	6	2	6		1		1		1
	60%		2	1	3	5	2	3	2	1	1	1
	40%			2	1	1	3	5		4	2	3
	20%				2			2	2	2	4	1
20<V<40	100%	14	3	2	3	1						
	80%	1	3	6	2	3	3	1	1	1		
	60%			3		7	2	2	3	1	3	
	40%				1	2	1	5	3	2	4	3
	20%					1			2	2	4	4
V>40	100%	16	2		1	3	1					
	80%		5	3	2	2	4	3	1	1		
	60%			1	3	1	4	3	3	3	2	1
	40%				1		1	4	3	4	4	4
	20%					1				2	3	2
G-Curb												
			1	1	1		3	2	3	4	2	6
H-Drop>4"												
					1	2		4	4	6	2	5
I-C-S Diff.												
6%	3	1	6	5	2	3	1	1			1	
8%	1		1	2	4	6	4	2	2			1
10%				1	1	2	1	9	3	3	3	3
12%								1	3	2	6	10
J-LaneWidth												
>13	12	7	4	2	1	2	1	1	1	1	1	
12	5	4	4	2	4	2					1	1
11	1	2	3	4	4	3	1	1	2	1	1	1
10		1		2	2	5	3	1	2	2	2	4
9			1			3	5	2	2	3	3	6
<8				1			2	2	4			12
K-RampDown.												
0%	16	3	1	1	1							
1-2%	1	8	8	3	2							
3-4%		1	1	3	9	7				1		
5-6%				1	1	1	5	7	4	2	1	
>6%							2	2	4	4	10	

Characteristic

	<u>0</u>	<u>1</u>	<u>2</u>	<u>3</u>	<u>4</u>	<u>5</u>	<u>6</u>	<u>7</u>	<u>8</u>	<u>9</u>	<u>10</u>
L-CurveLength											
100	2	2	2	1		1	2			1	4
200		1	1	3	2	2	1		3	1	1
300			1	1	4	2	4	1	1		1
400			1	2	3	2	4	1	1		1
500		1	4		2	2	1	1	2		2
600	5				2	2	1	1			4
Relative Importance											
Dec.Lane		1	3	8	3	4	1	1	1		
Ramp			1	1	1	3	2	7	4	1	2
Trans.Curve			4	2	3	3		3	3	1	2
Comp.Curve			3	1	1	5	2	2	2	1	4
Radius				1	1		2	2	4	5	4
Curb	1				1	4	4	1	5	3	1
Drop			1		3	1	5	1	3	3	3
C-S Diff.			1	1	4	3	3	4	2	1	
Lane Width	1	1	3	1	2	3	3	2	3		1
Downgrade			2	1	3	3	4		2	4	1
Length		1		5	2	3	2	2	1		
Interchange	2	1		4		7	2				1
Future Vehs.	4	1	3	4		3	1				2

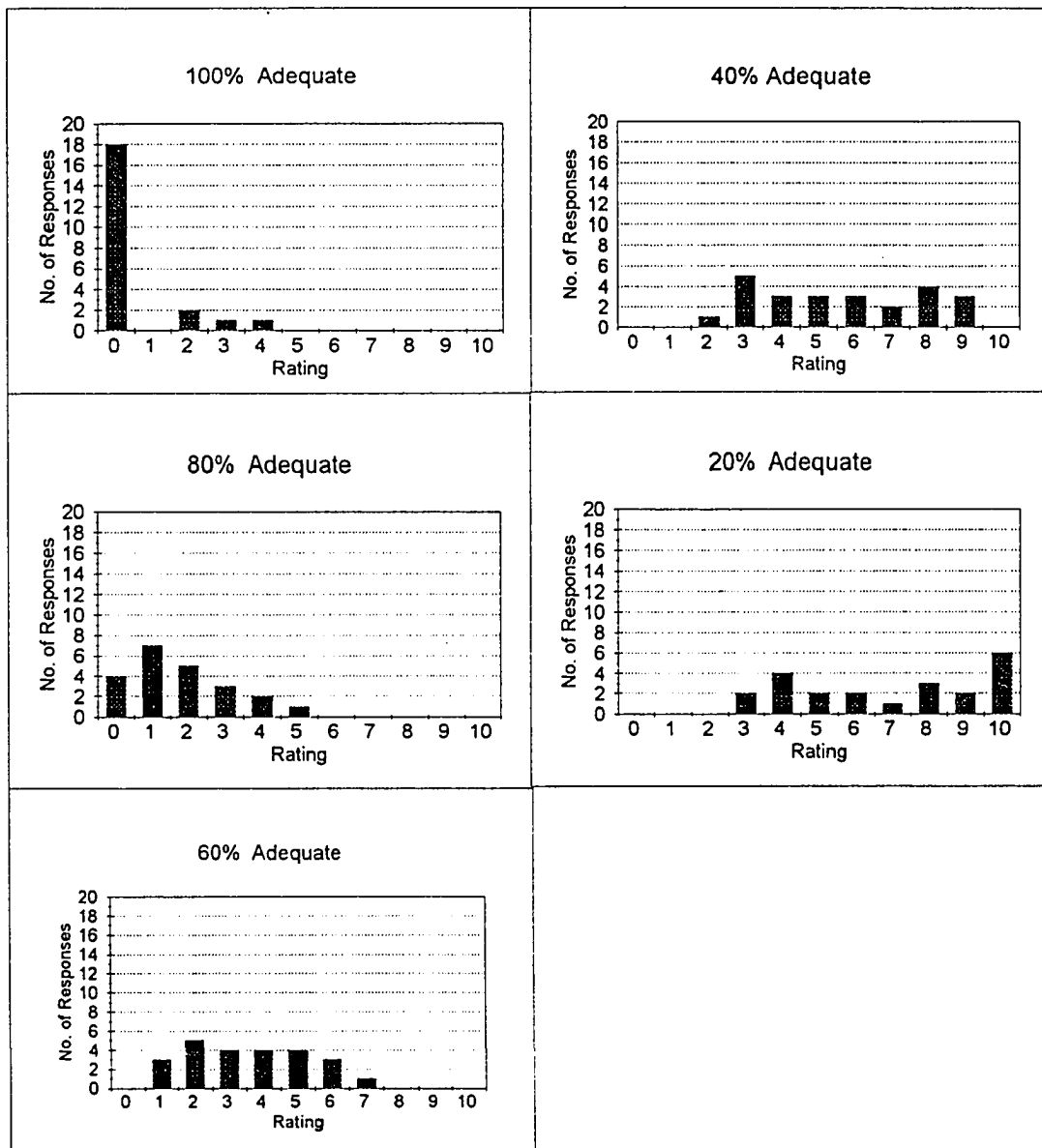
APPENDIX D
Statistical Analysis of Responses

<u>VARIABLE</u>	<u>N</u>	<u>MEAN</u>	<u>STANDARD DEVIATION</u>
DECL4100	20	0.250	0.786
DECL480	20	1.550	1.234
DECL460	20	3.500	1.878
DECL440	20	5.550	2.259
DECL420	20	7.250	2.531
DEC46100	20	0.300	0.979
DEC4680	20	2.500	1.792
DEC4660	20	4.775	2.167
DEC4640	20	6.725	2.111
DEC4620	20	8.450	1.905
DECG6100	20	0.350	1.182
DECG680	20	3.650	2.390
DECG660	20	6.100	2.174
DECG640	20	7.950	1.669
DECG620	20	9.450	1.099
DRYROAD	20	1.825	3.258
WETROAD	20	5.000	1.913
SNOWROAD	20	6.500	1.899
ICYROAD	20	8.750	2.668
DECFLAT	20	0.300	0.979
DECDWN12	20	2.150	1.148
DECDWN34	20	5.100	1.492
DECDWN56	20	8.000	1.828
SPIRAL	17	1.500	2.015
COMPOUND	17	4.853	3.287
SMPLHLFe	18	5.028	2.499
SMPLeMAX	18	5.361	3.105
SHARPFLT	16	5.375	3.258
FLATSHRP	17	5.882	2.684
SHPFLSHP	17	8.118	1.772
FLTSHFLT	17	6.176	2.926
RADL2100	19	1.184	2.256
RADL280	18	3.306	2.619
RADL260	18	5.028	2.569
RADL240	18	6.694	2.619
RADL220	18	8.028	2.404
RAD24100	19	0.711	1.146
RAD2480	18	3.306	2.217
RAD2460	18	5.417	2.418
RAD2440	18	7.194	2.204
RAD2420	18	8.583	1.699
RADG4100	19	0.711	1.484
RADG480	18	3.583	2.264
RADG460	18	5.917	2.365
RADG440	18	7.694	1.979
RADG420	18	9.139	1.607
CURBHAZ	20	7.475	2.593
DROPGT4	20	7.450	2.145

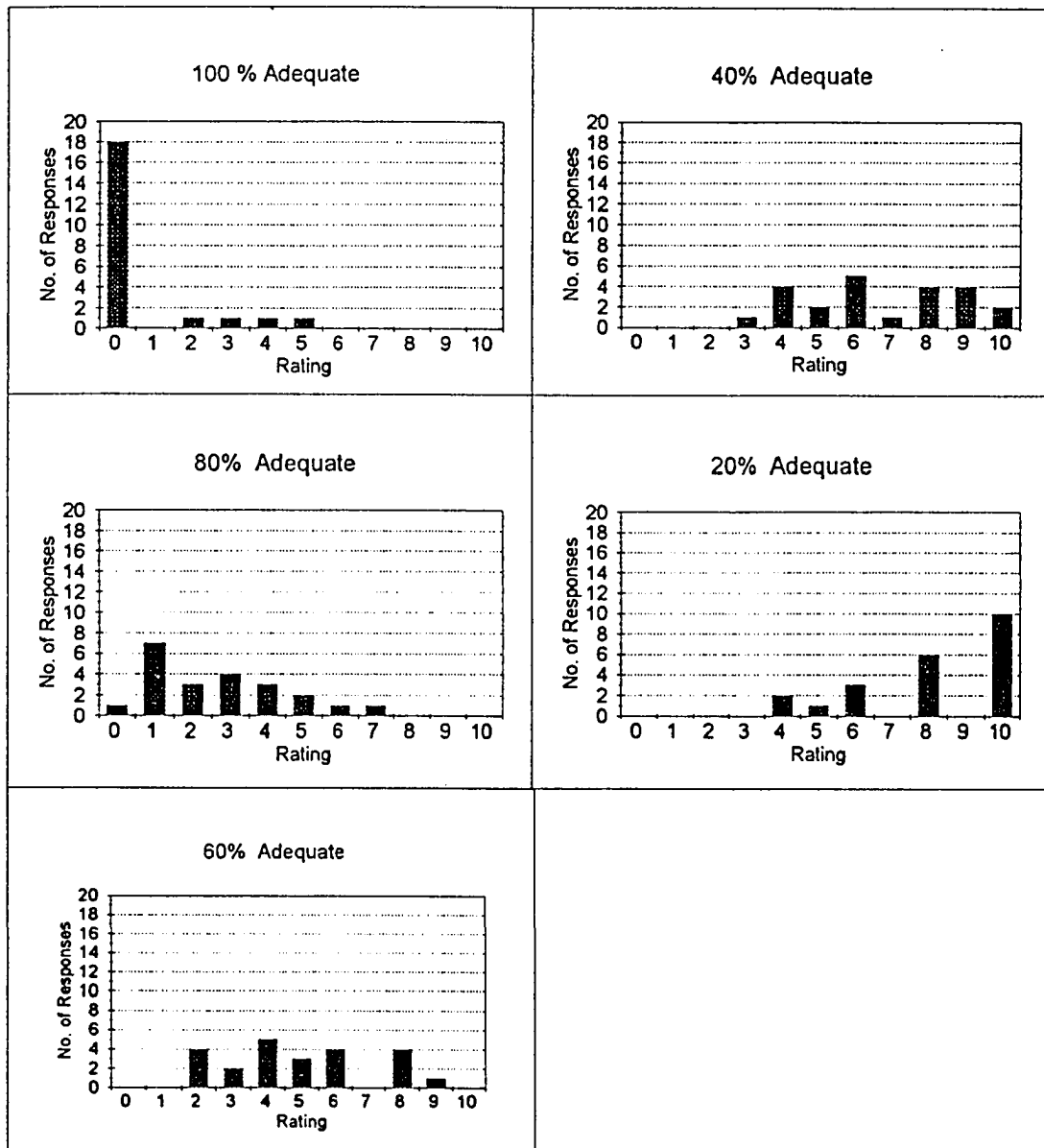
<u>VARIABLE</u>	<u>N</u>	<u>MEAN</u>	<u>STANDARD DEVIATION</u>
CSDIFF6	20	2.975	2.308
CSDIFF8	20	5.025	2.203
CSDIFF10	20	7.275	1.832
CSDIFF12	20	9.150	1.089
LANEGT13	20	0.725	1.229
LANE12	20	2.675	2.976
LANE11	20	4.225	2.803
LANE10	20	6.450	2.704
LANE9	20	7.600	2.280
LANELTS	19	8.789	1.932
RAMPFLAT	20	0.400	0.995
RAMPDN12	20	1.750	0.966
RAMPDN34	20	4.150	1.387
RAMPDN56	20	6.850	1.663
RAMPDNG6	20	8.900	1.373
CURVE100	13	5.462	4.075
CURVE200	13	5.462	2.904
CURVE300	13	5.385	2.142
CURVE400	13	5.154	2.193
CURVE500	13	4.923	3.200
CURVE600	13	4.538	4.332
DECLANE	20	3.950	1.701
RAMP	20	6.500	2.065
TRANSCRV	20	5.275	2.613
COMPSCRV	20	6.025	2.736
RADIUS	19	7.895	1.997
CURB	20	6.600	2.303
EDGEDROP	20	6.850	2.323
CSDIFF	19	5.632	1.862
WIDTH	20	4.950	2.685
DOWNRAMP	20	6.000	2.471
LENGTH	16	4.563	1.896
INTERCHG	17	1.412	0.245

APPENDIX E
Membership Functions - Graphs

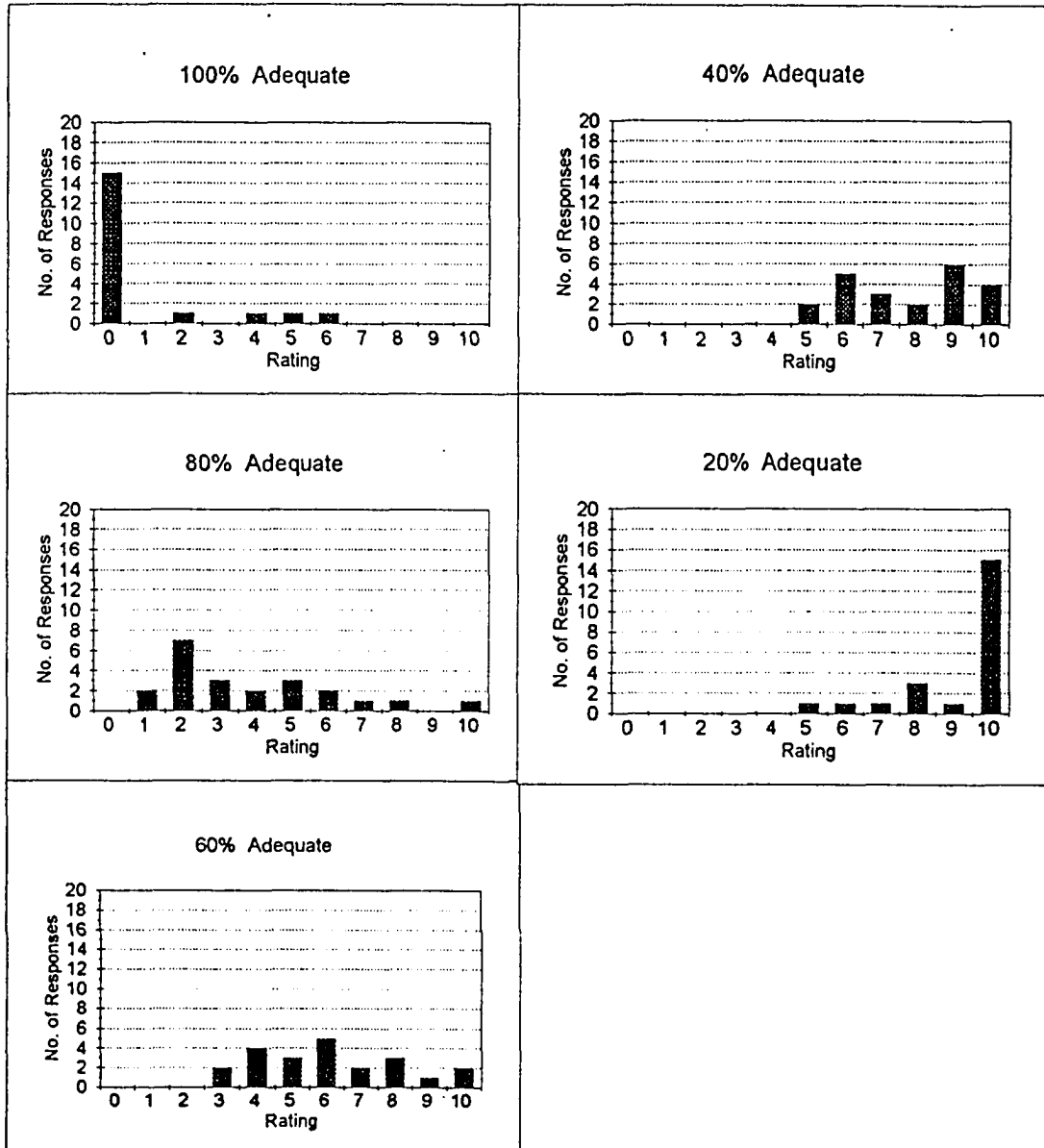
Deceleration Lane Length, V<40mph



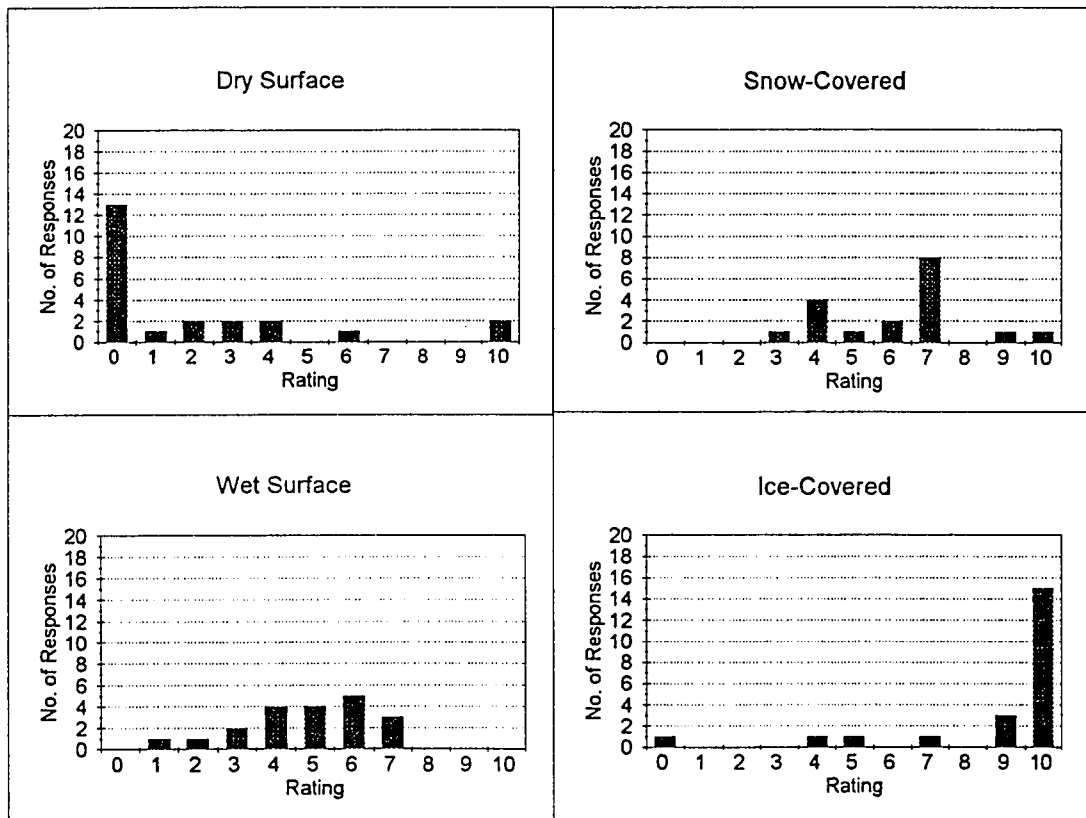
Deceleration Lane Length, 40<V<60 mph



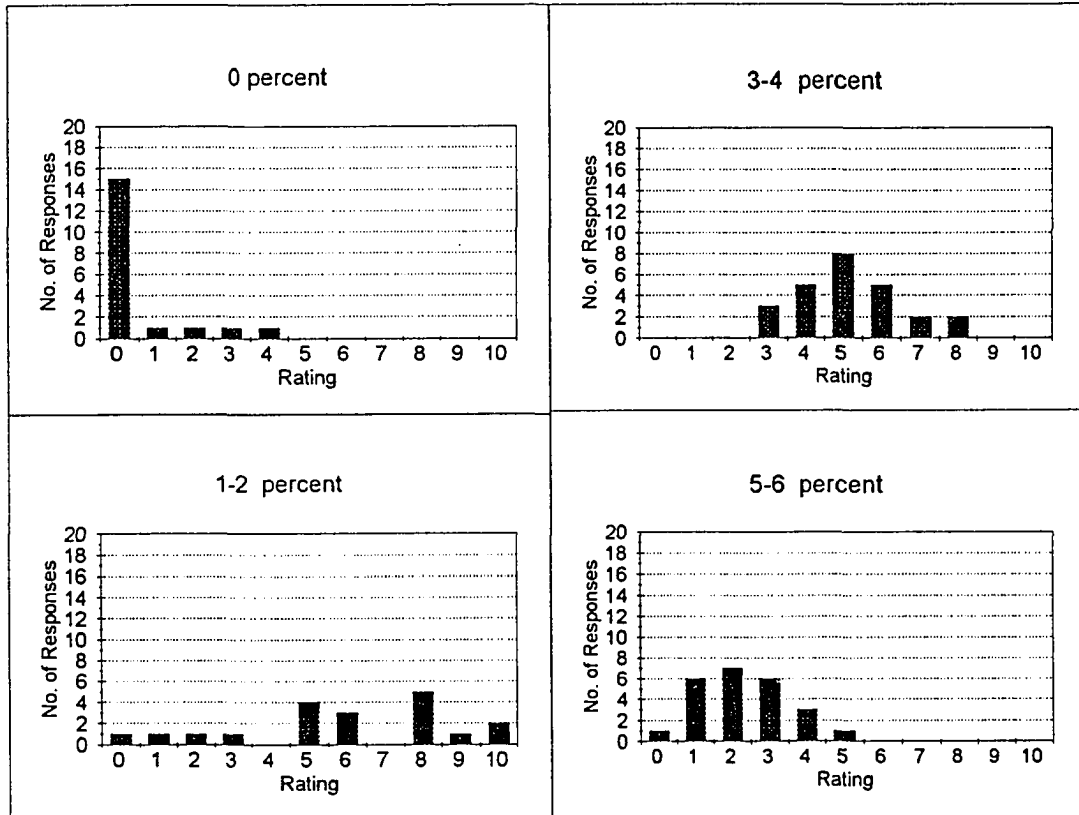
Deceleration Lane Length, V>60mph



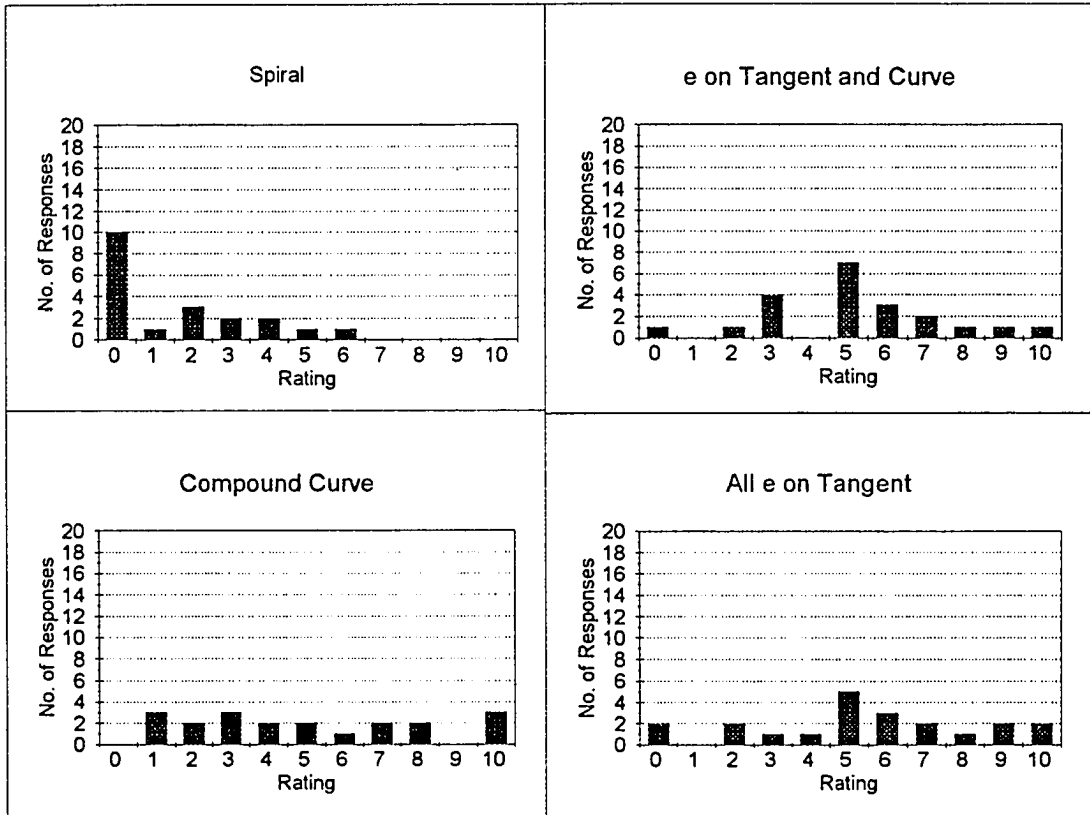
Pavement



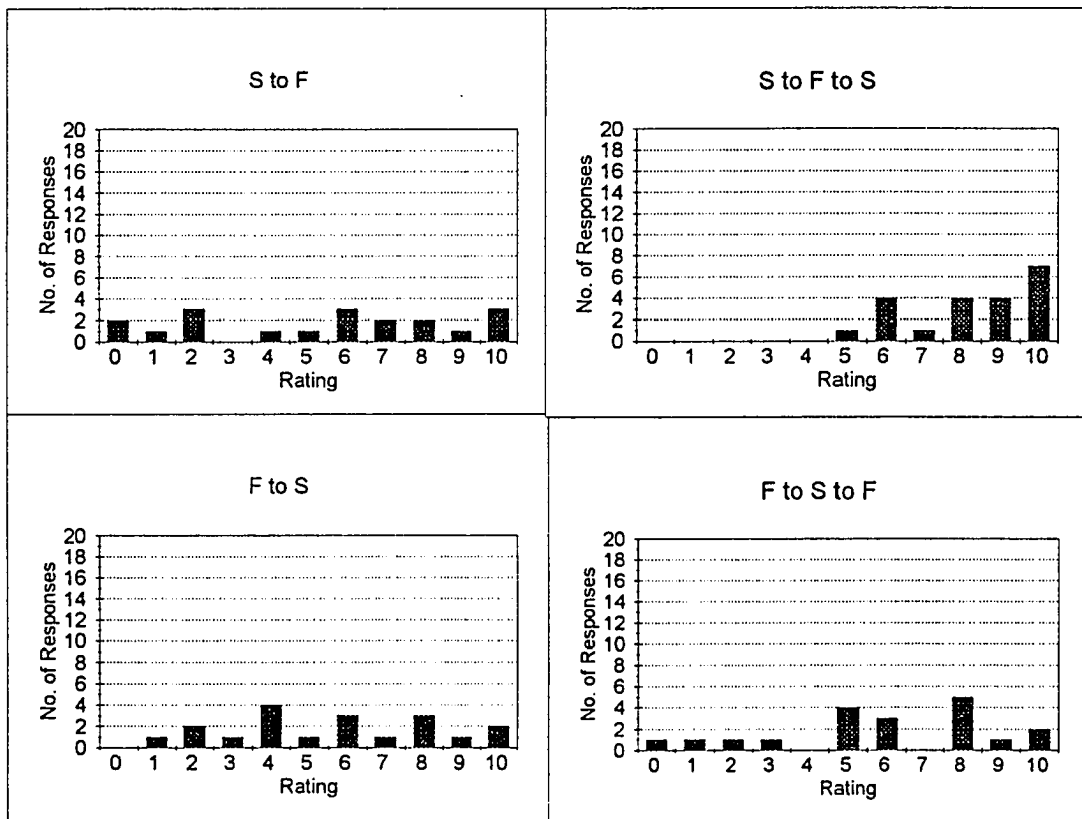
Downgrade on Deceleration Lane



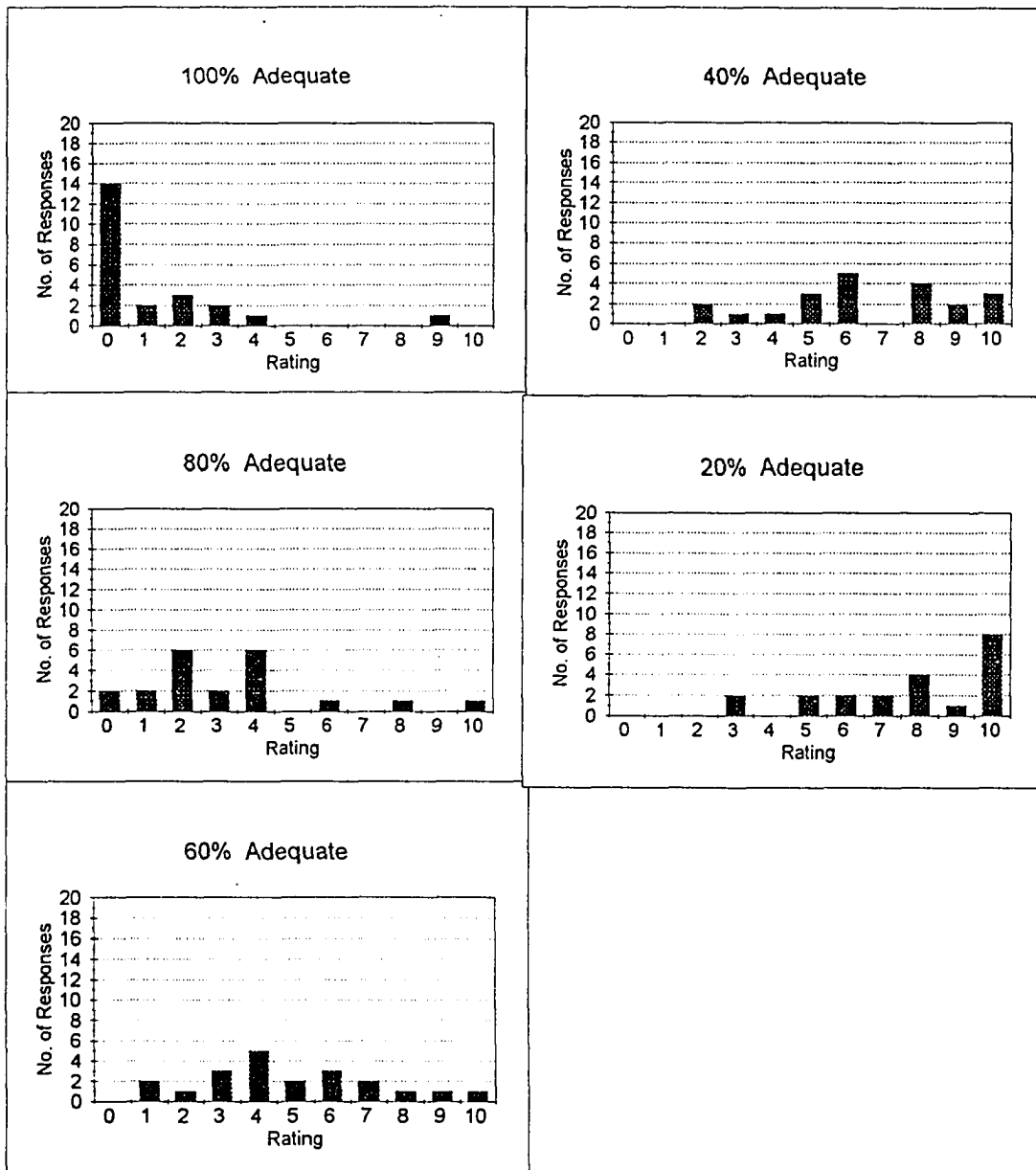
Transition Curve



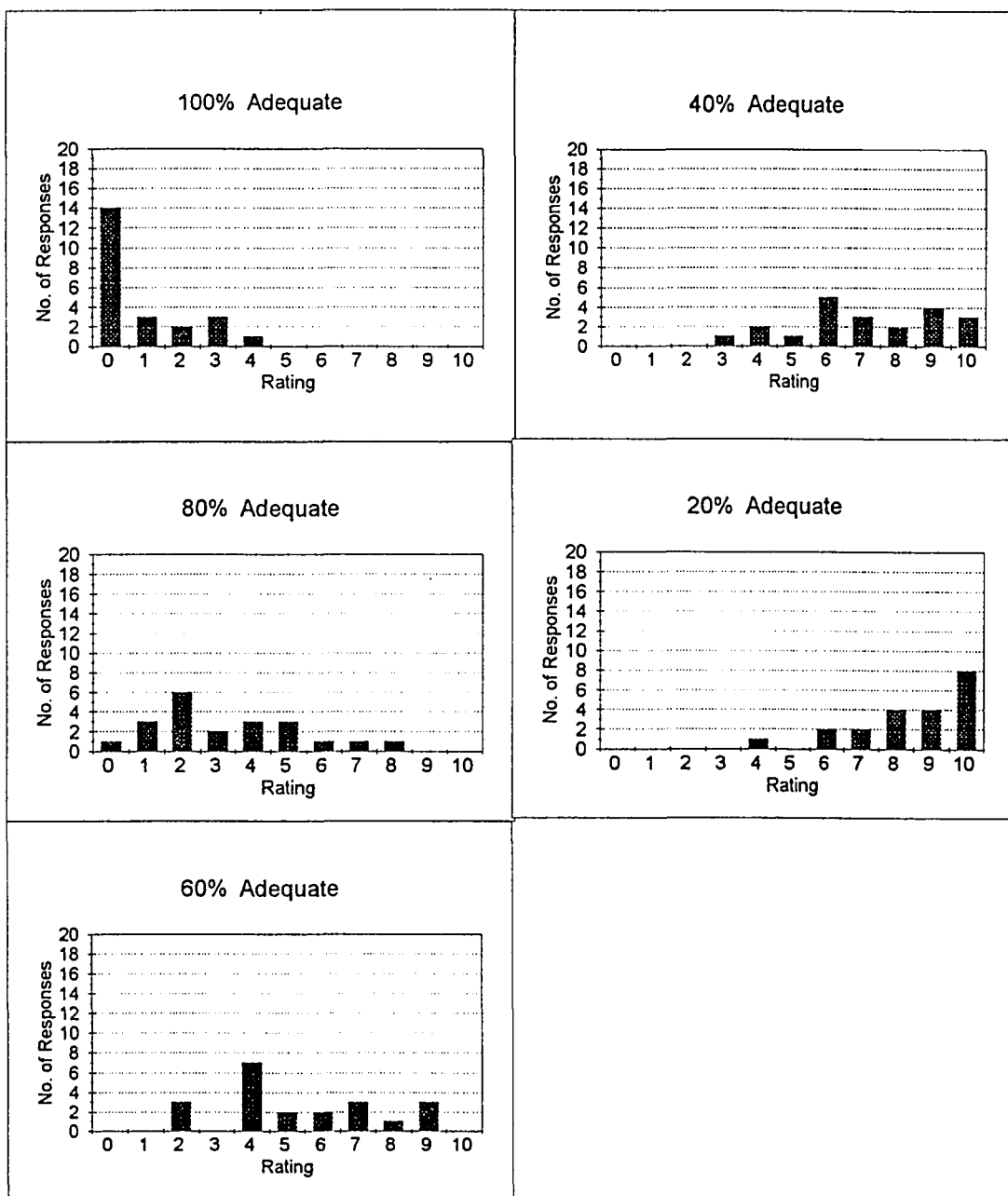
Type of Compound Curve



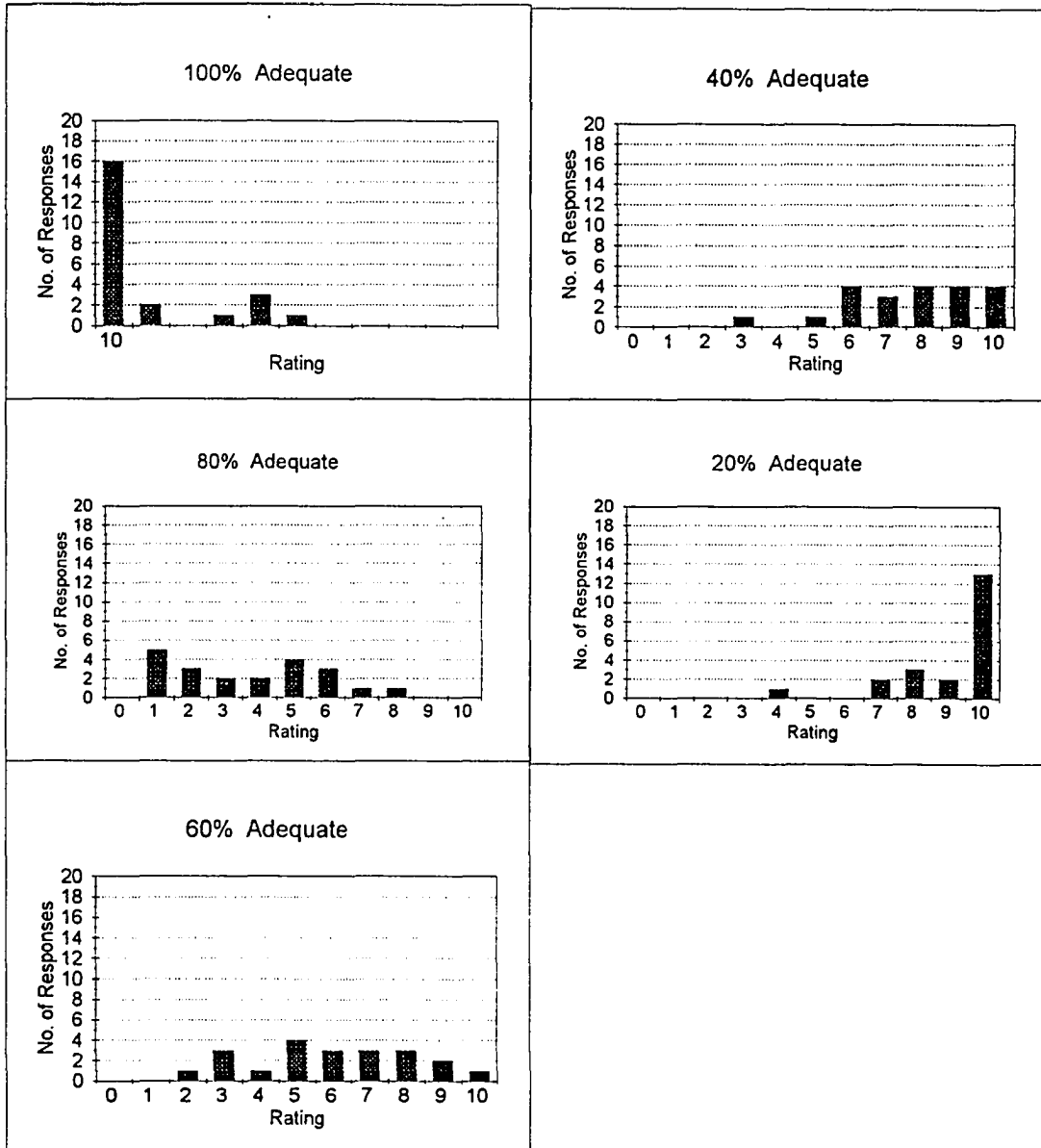
Adequacy of Radius, V < 20mph

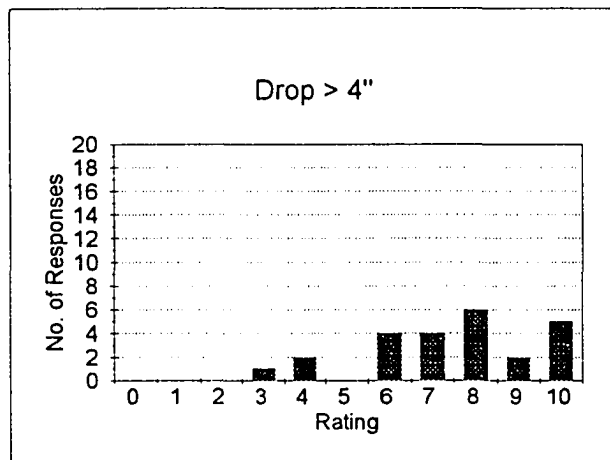
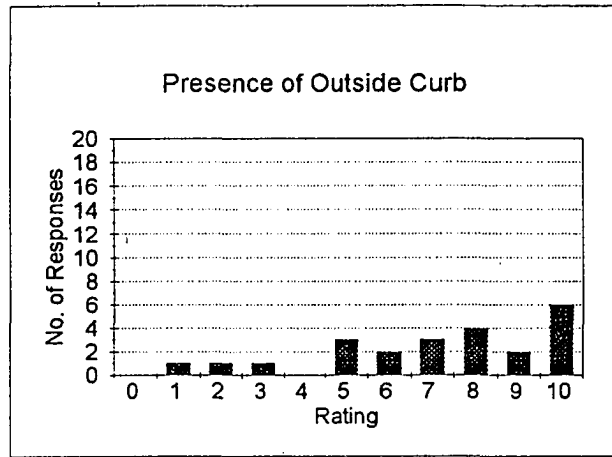


Adequacy of Radius, $20 < V < 40\text{mph}$

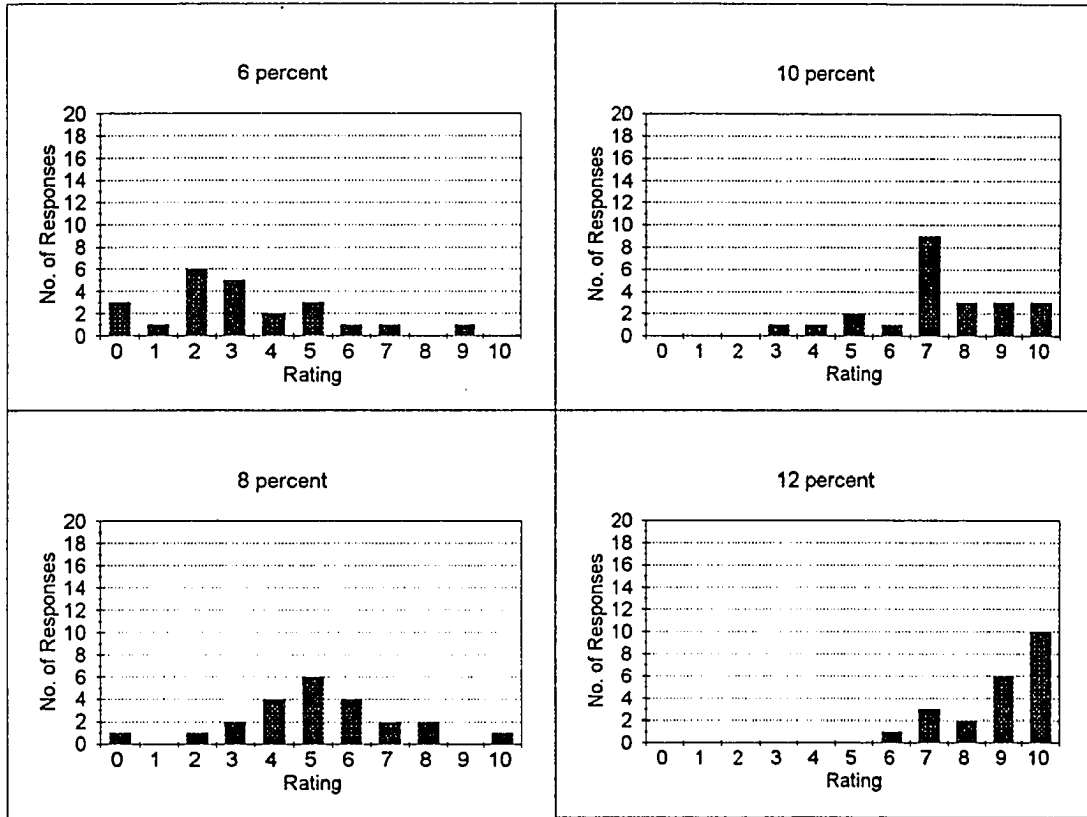


Adequacy of Radius, $V > 40\text{mph}$

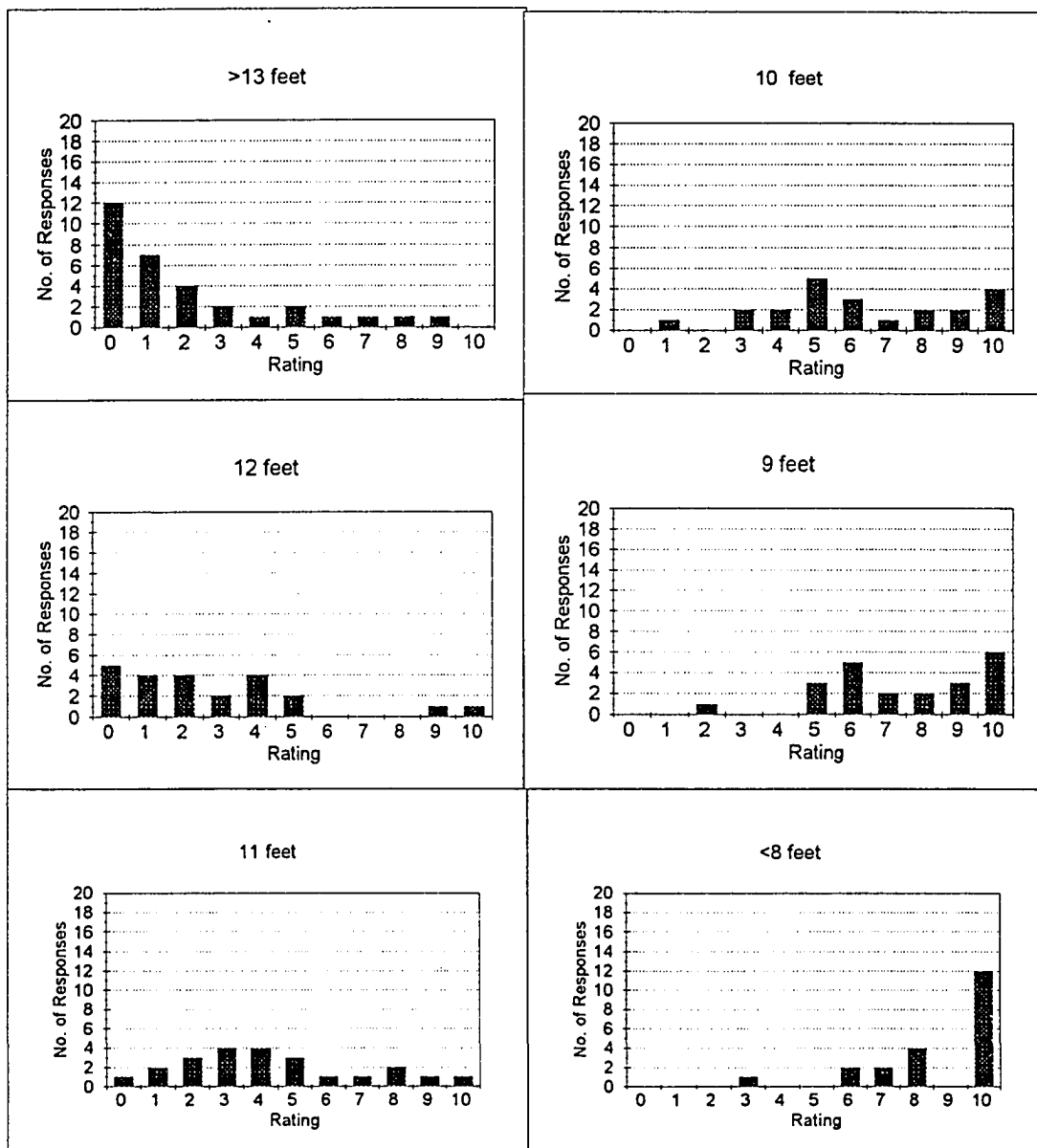




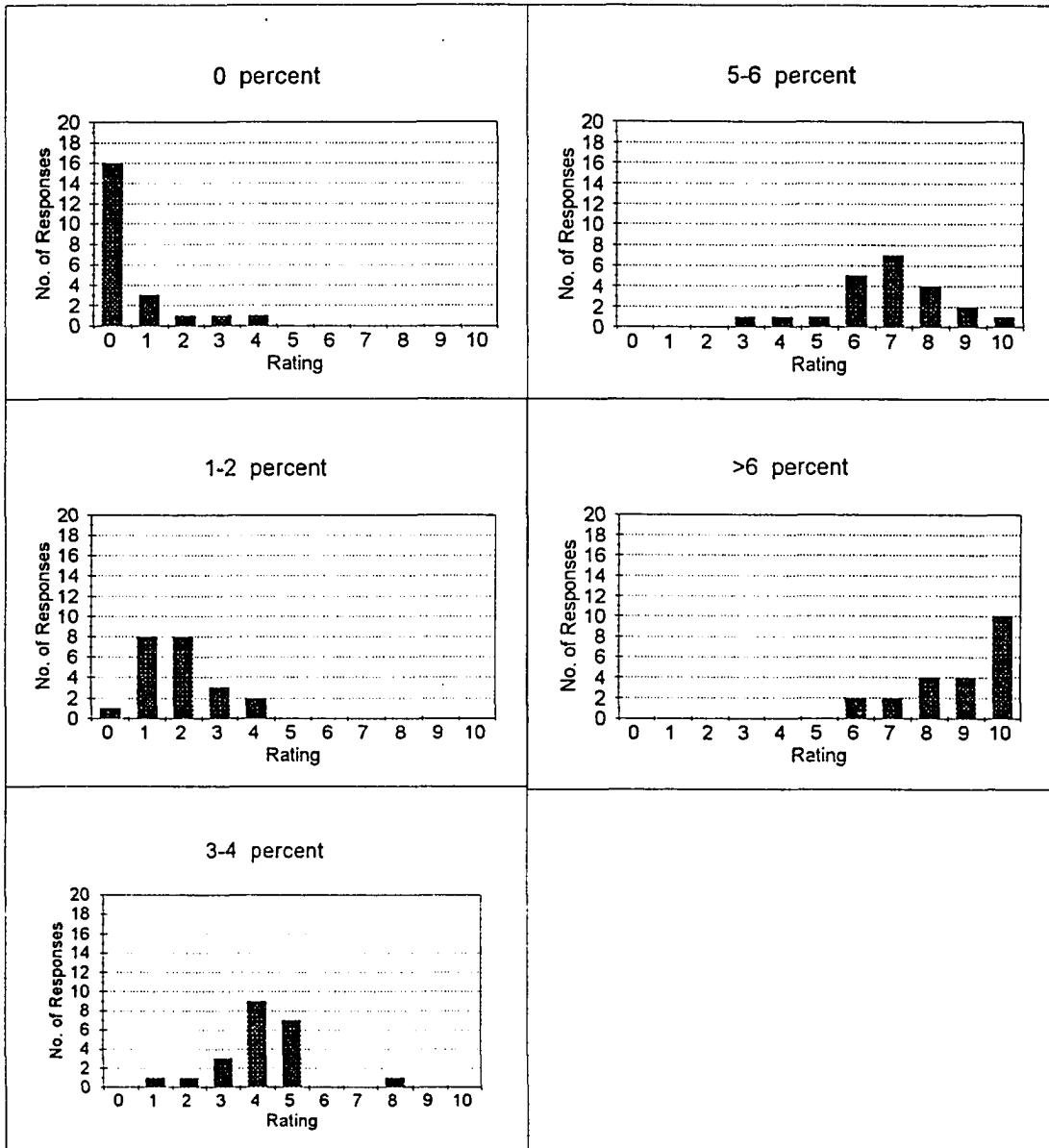
Cross-Slope Difference



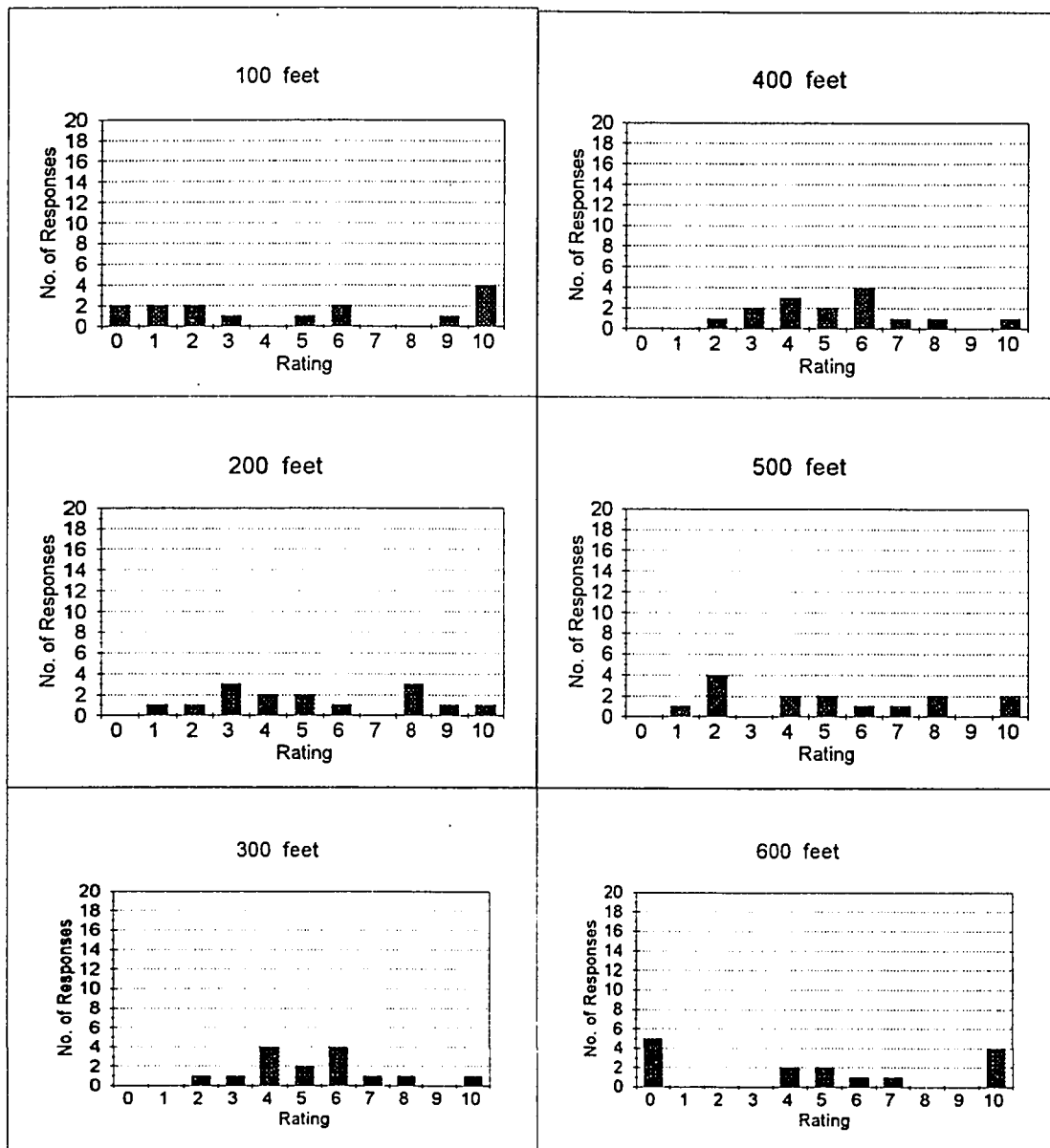
Lane Width



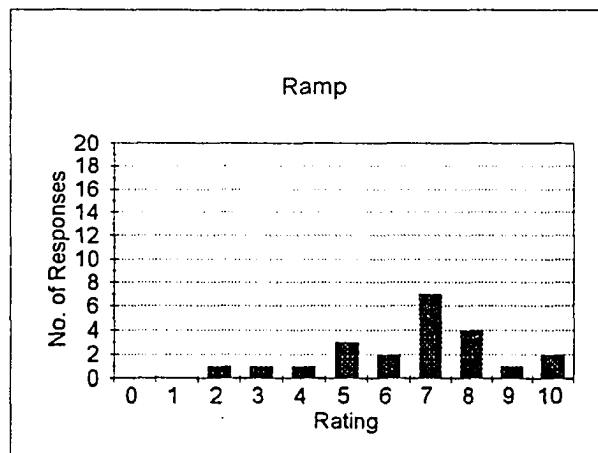
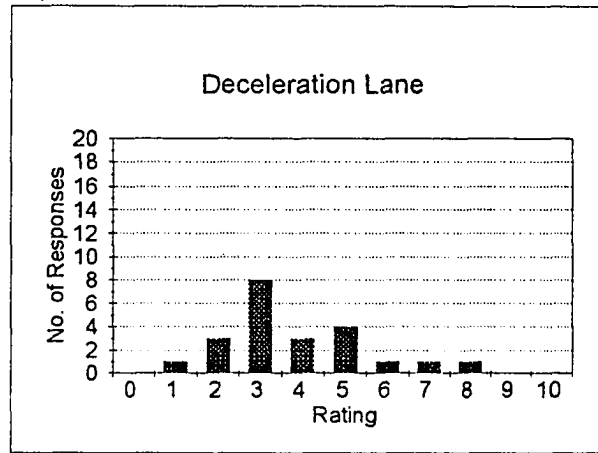
Ramp Downgrade



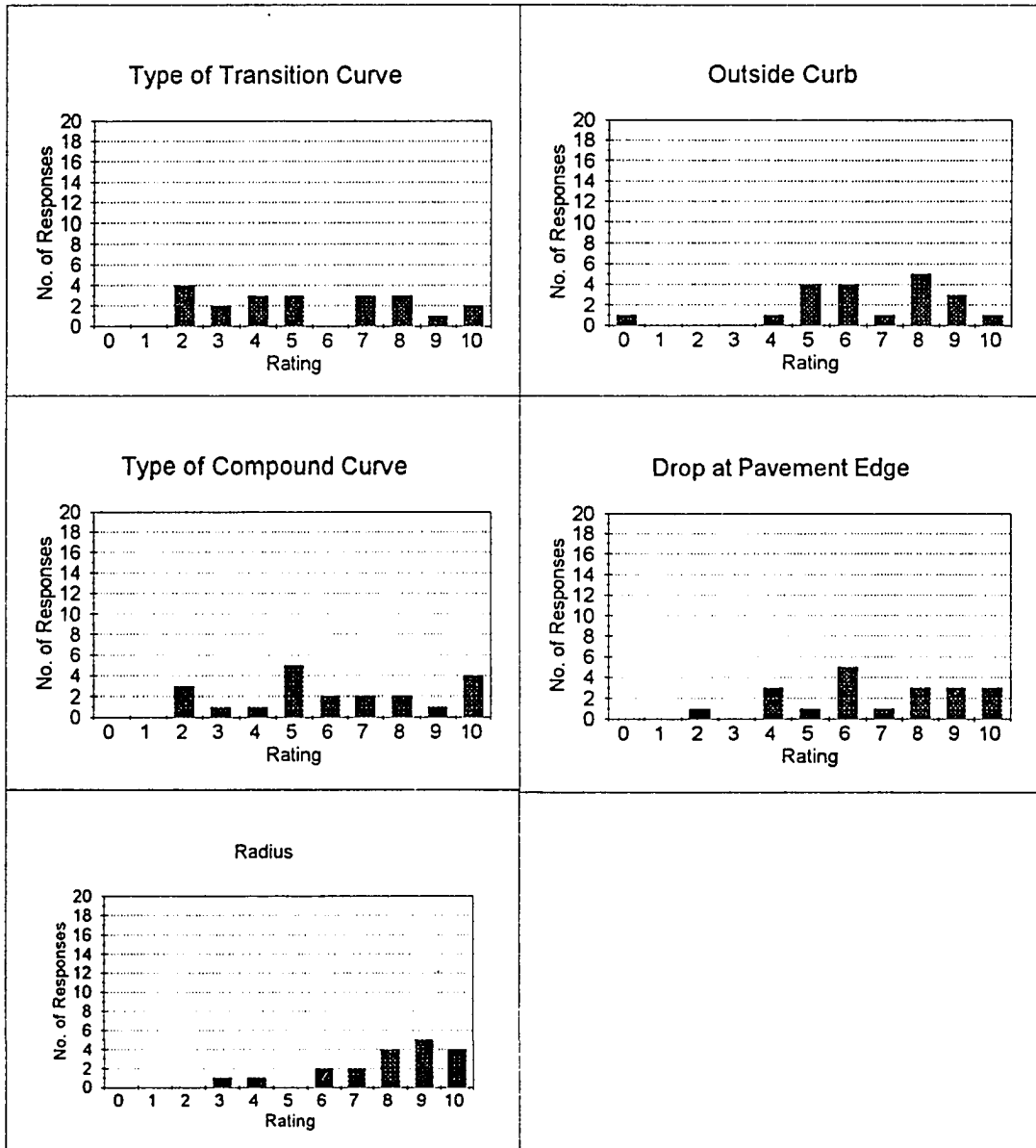
Curve Length



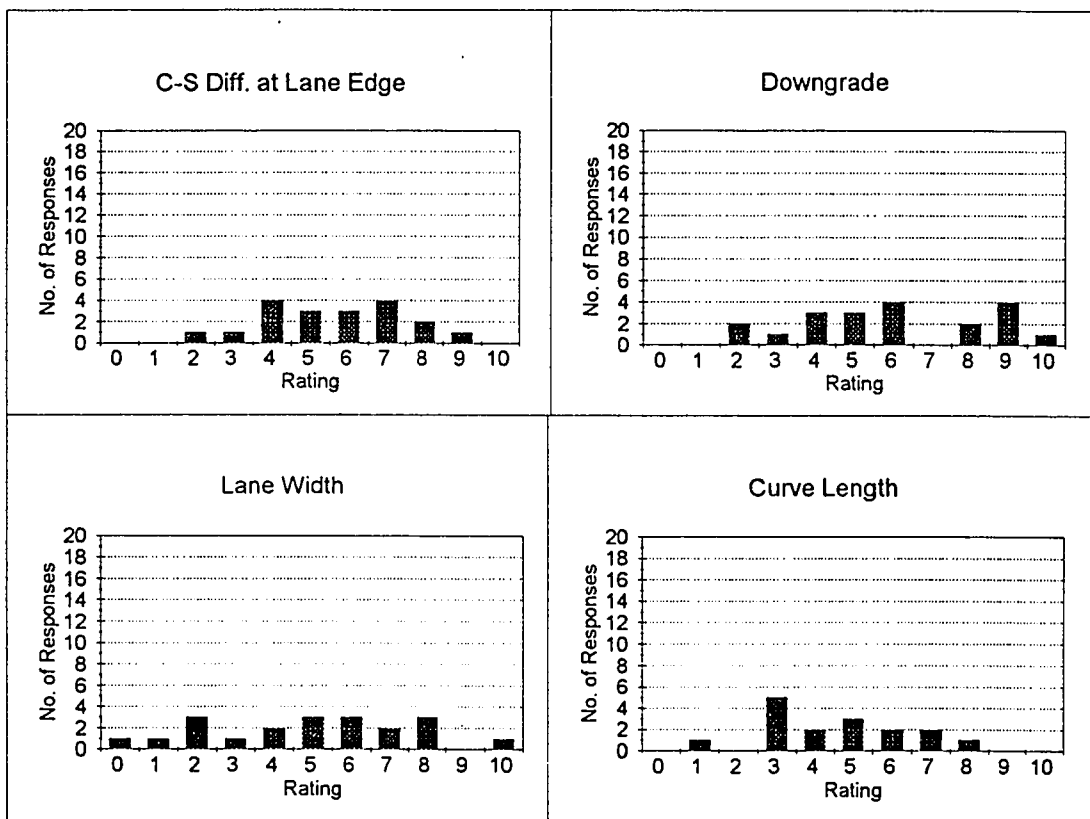
Relative Importance



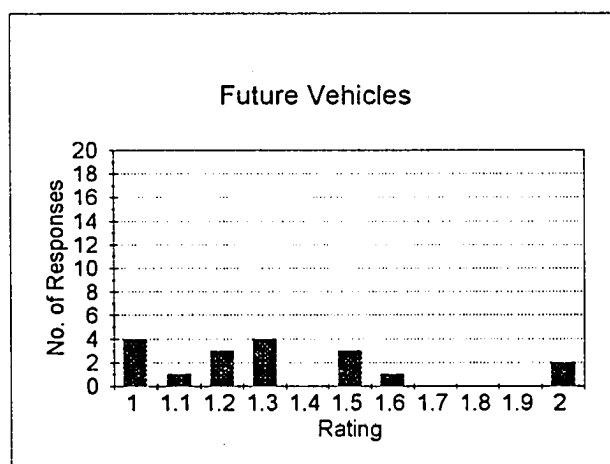
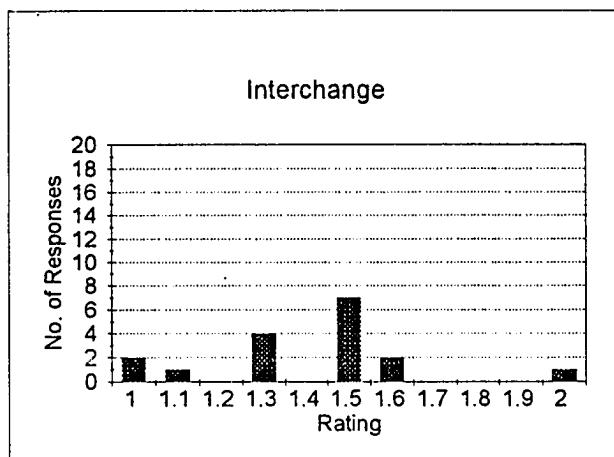
Relative Importance - Ramp Characteristics



Relative Importance - Ramp Characteristics



Factors



APPENDIX F
Membership Functions

MEMBERSHIP FUNCTIONS
DESCRIBED BY FUZZY SETS

Legend: Normalized Function
Adjusted for convexity where necessary

Deceleration Lane Length $V < 40\text{mph}$

100% Adequate	1 0, .1 2, .1 3, .1 4 1 0, .1 1, .1 2, .06 3, .06 4
80% "	.6 0, 1 1, .7 2, .4 3, .3 4, .1 5 .6 0, 1 1, .7 2, .4 3, .3 4, .1 5
60% "	.6 1, 1 2, .8 3, .8 4, .8 5, .6 6, .2 7
40% "	.2 2, 1 3, .6 4, .6 5, .6 6, .4 7, .8 8, .6 9 .2 2, 1 3, .6 4, .6 5, .6 6, .6 7, .6 8, .6 9
20% "	.3 3, .7 4, .3 5, .3 6, .2 7, .5 8, .3 9, 1 10 .3 3, .3 4, .3 5, .3 6, .3 7, .5 8, .5 9, 1 10

Deceleration Lane Length $40 < V < 60\text{mph}$

100% Adequate	1 0, .1 2, .1 3, .1 4, .1 5 1 0, .06 1, .06 2, .06 3, .06 4, .06 5
80% "	.1 0, 1 1, .4 2, .6 3, .4 4, .3 5, .1 6, .1 7 .14 0, 1 1, .57 2, .57 3, .43 4, .29 5, .14 6, .14 7
60% "	.8 2, .4 3, 1 4, .6 5, .8 6, .8 8, .2 9 .6 2, .6 3, 1 4, .8 5, .8 6, .8 7, .8 8, .2 9
40% "	.2 3, .8 4, .4 5, 1 6, .2 7, .8 8, .8 9, .4 10 .2 3, .6 4, .6 5, 1 6, .8 7, .8 8, .8 9, .4 10
20% "	.2 4, .1 5, .3 6, .6 8, 1 10 .2 4, .25 5, .3 6, .45 7, .6 8, .8 9, 1 10

Deceleration Lane Length V > 60mph

100% Adequate	1 0, .1 2, .1 4, .1 5, .1 6 1 0, .07 1, .07 2, .07 3, .07 4, .07 5, .07 6
80% "	.3 1, .1 2, .4 3, .3 4, .4 5, .3 6, .1 7, .1 8, .1 10 .29 1, .1 2, .43 3, .43 4, .43 5, .29 6, .14 7, .14 8, .14 9, .14 10
60% "	.4 3, .8 4, .6 5, 1 6, .4 7, .6 8, .2 9, .4 10 .4 3, .8 4, .9 5, 1 6, .8 7, .6 8, .5 9, .4 10
40% "	.3 5, .8 6, .5 7, .3 8, 1 9, .7 10 .33 5, .67 6, .67 7, .83 8, 1 9, .67 10
20% "	.1 5, .1 6, .1 7, .2 8, .1 9, 1 10 .07 5, .07 6, .07 7, .2 8, .2 9, 1 10

Deceleration Lane Pavement Surface

Dry	1 0, .1 1, .2 2, .2 3, .2 4, .1 6 1 0, .15 1, .15 2, .15 3, .15 4, .12 5, .08 6
Wet	.2 1, .2 2, .4 3, .8 4, .8 5, 1 6, .6 7
Snow	.1 3, .4 4, .3 6, 1 7, .1 9, 1 10 .13 3, .25 4, .25 5, .25 6, 1 7, .13 8, .13 9, .13 10
Ice	.1 4, .1 7, .2 9, 1 10 .07 4, .07 5, .07 6, .07 7, .14 8, .2 9, 1 10

Downgrade on Deceleration Lane

0%	1 0, .1 1, .1 2, .1 3, .1 4
1-2%	.1 0, .9 1, 1 2, .9 3, .4 4, .1 5
3-4%	.4 3, .6 4, 1 5, .6 6, .3 7, .3 8
5-6%	.1 5, .6 6, .4 7, .6 8, .6 9, 1 10 .14 5, .57 6, .57 7, .57 8, .57 9, 1 10

Transition Curve

Spiral	1 0, .1 1, .3 2, .2 3, .2 4, .1 5, .1 6 1 0, .3 1, .3 2, .2 3, .2 4, .1 5, .1 6
Compound	1 1, .7 2, 1 3, .7 4, .7 5, .3 6, .7 7, .7 8, 1 10 1 1, 1 2, 1 3, .67 4, .67 5, .67 6, .67 7, .67 8, .67 9, .67 10
e on Tangent and Curve	.1 0, .1 2, .6 3, 1 5, .4 6, .3 7, .1 8, .1 9, .1 10 .14 0, .14 1, .14 2, .57 3, .79 4, 1 5, .43 6, .29 7, .14 8, .14 9, .14 10
All e on Tangent	.4 0, .4 2, .2 3, .2 4, 1 5, .6 6, .4 7, .2 8, .4 9, .4 10 .2 0, .2 1, .4 2, .4 3, .4 4, 1 5, .6 6, .4 7, .4 8, .4 9, .4 10

Type of Compound Curve

S → F	.7 0, .3 1, 1 2, .3 4, .3 5, 1 6, .7 7, .7 8, .3 9, 1 10 .33 0, .33 1, .33 2, .33 3, .33 4, .33 5, 1 6, .67 7, .67 8, .33 9, .33 10
F → S	.3 1, .5 2, .3 3, 1 4, .3 5, .8 6, .3 7, .8 8, .3 9, .5 10 .25 1, .5 2, .75 3, 1 4, .88 5, .75 6, .75 7, .75 8, .63 9, .5 10
S → F → S	.1 5, .6 6, .1 7, .6 8, .6 9, 1 10 .14 5, .57 6, .57 7, .57 8, .57 9, 1 10
F → S → F	.2 0, .2 1, .2 2, .2 3, .8 5, .6 6, .1 8, .2 9, .4 10 .2 0, .2 1, .2 2, .2 3, .5 4, .8 5, .87 6, .93 7, 1 8, .7 9, .4 10

Adequacy of Radius V < 20mph

100% Adequate	1 0, .1 1, .2 2, .1 3, .1 4, .1 9 1 0, .2 1, .2 2, .14 3, .07 4
80% "	.3 0, .3 1, 1 2, .3 3, 1 4, .2 6, .2 8, .2 10 .33 0, .33 1, 1 2, 1 3, 1 4, .17 5, .17 6, .17 7, .17 8 .17 9, .17 10
60% "	.4 1, .2 2, .6 3, 1 4, .4 5, .6 6, .4 7, .2 8, .2 9, .2 10 .4 1, .5 2, .6 3, 1 4, .8 5, .6 6, .4 7, .2 8, .2 9, .2 10
40% "	.4 2, .2 3, .2 4, .6 5, 1 6, .8 8, .4 9, .6 10 .4 2, .4 3, .6 4, .6 5, 1 6, .9 7, .8 8, .7 9, .6 10
20% "	.3 3, .3 5, .3 6, .3 7, .5 8, .1 9, 1 10 .25 3, .25 4, .25 5, .25 6, .25 7, .5 8, .5 9, 1 10

Adequacy of Radius 20 < V < 40mph

100% Adequate	1 0, .2 1, .1 2, .2 3, .1 4 1 0, .2 1, .2 2, .2 3, .07 4
80% "	.2 0, .5 1, 1 2, .3 3, .5 4, .5 5, .2 6, .2 7, .2 8 .17 0, .5 1, 1 2, .75 3, .5 4, .5 5, .17 6, .17 7, .17 8
60% "	.4 2, 1 4, .3 5, .3 6, .4 7, .1 8, .4 9 .43 2, .72 3, 1 4, .43 5, .43 6, .43 7, .29 8, .29 9
40% "	.2 3, .4 4, .2 5, 1 6, .6 7, .4 8, .8 9, .6 10 .2 3, .4 4, .7 5, 1 6, .93 7, .86 8, .8 9, .6 10
20% "	.1 4, .3 6, .3 7, .5 8, .5 9, 1 10 .13 4, .19 5, .25 6, .25 7, .5 8, .5 9, 1 10

Adequacy of Radius V > 40mph

100% Adequate	1 0, .1 1, .1 3, .2 4, .1 5 1 0, .13 1, .13 2, .06 3, .06 4, .06 5
80% "	1 1, .6 2, .4 3, .4 4, .8 5, .6 6, .2 7, .2 8 1 1, .8 2, .8 3, .8 4, .8 5, .6 6, .2 7, .2 8
60% "	.3 2, .8 3, .3 4, 1 5, .8 6, .8 7, .8 8, .5 9, .3 10 .25 2, .75 3, .88 4, 1 5, .75 6, .75 7, .75 8, .5 9, .25 10
40% "	.3 3, .3 5, 1 6, .8 7, 1 8, 1 9, 1 10 .25 3, .25 4, .25 5, 1 6, 1 7, 1 8, 1 9, 1 10
20% "	.1 4, .2 7, .2 8, .2 9, 1 10 .15 7, .23 8, .23 9, 1 10

Presence of Outside Curb

.2| 1, .2| 2, .2| 3, .5| 5, .3| 6, .5| 7, .7| 8,
.3| 9, 1| 10
.17| 1, .17| 2, .17| 3, .17| 4, .33| 5, .33| 6,
.5| 7, .67| 8, .67| 9, 1| 10

Drop > 4 Inches

.2| 3, .3| 4, .7| 6, .7| 7, 1| 8, .3| 9, .8| 10
.17| 3, .33| 4, .5| 5, .67| 6, .67| 7, 1| 8,
.91| 9, .83| 10

Cross-Slope Difference

6%	.5 0, .2 1, 1 2, .8 3, .3 4, .5 5, .2 6, .2 7, .2 9 .5 0, .5 1, 1 2, .83 3, .5 4, .5 5, .17 6, .17 7, .17 8, .17 9
8%	.2 0, .2 2, .3 3, .7 4, 1 5, .7 6, .3 7, .3 8, .2 10 .17 0, .17 1, .17 2, .33 3, .67 4, 1 5, .67 6, .33 7, .33 8, .17 9, .17 10
10%	.1 3, .1 4, .2 5, .1 6, 1 7, .3 8, .3 9, .3 10 .11 3, .11 4, .22 5, .22 6, 1 7, .33 8, .33 9, .33 10
12%	.1 6, .3 7, .2 8, .6 9, 1 10 .1 6, .3 7, .3 8, .6 9, 1 10

Lane Width

≥ 13 ft.	1 0, .6 1, .3 2, .2 3, .1 4, .2 5, .1 6, .1 7, .1 8, .1 9 1 0, .58 1, .33 2, .17 3, .17 4, .17 5, .08 6, .08 7, .08 8, .08 9
12 ft.	1 0, .8 1, .8 2, .4 3, .8 4, .4 5, .2 9, .2 10 1 0, .8 1, .8 2, .8 3, .8 4, .4 5, .2 6, .2 7, .2 8, .2 9, .2 10
11 ft.	.3 0, .5 1, .8 2, 1 3, 1 4, .8 5, .3 6, .3 7, .5 8, .3 9, .3 10 .25 0, .5 1, .75 2, 1 3, 1 4, .75 5, .5 6, .5 7, .5 8, .25 9, .25 10
10 ft.	.2 1, .4 3, .4 4, 1 5, .6 6, .2 7, .4 8, .4 9, .8 10 .2 1, .3 2, .4 3, .4 4, 1 5, .8 6, .8 7, .8 8, .8 9, .8 9, .8 10
9 ft.	.2 2, .5 5, .8 6, .3 7, .3 8, .5 9, 1 10 .17 2, .17 3, .17 4, .5 5, .67 6, .67 7, .67 8, .67 9, 1 10
≤ 8 ft.	.1 3, .2 6, .2 7, .3 8, 1 10 .06 3, .06 4, .06 5, .11 6, .11 7, .22 8, .61 9, 1 10

Ramp Downgrade

0%	1 0, .2 1, .1 2, .1 3, .1 4
1-2%	.1 0, 1 1, 1 2, .4 3, .3 4
3-4%	.1 1, .1 2, .3 3, 1 4, .8 5, .1 8 .11 1, .11 2, .33 3, 1 4, .77 5
5-6%	.1 3, .1 4, .1 5, .7 6, 1 7, .6 8, .3 9, .1 10
> 6%	.2 6, .2 7, .4 8, .4 9, 1 10

Relative Importance

Deceleration Lane	.1 1, .4 2, 1 3, .4 4, .5 5, .1 6, .1 7, .1 8, .13 1, .38 2, 1 3, .5 4, .5 5, .13 6, .13 7, .13 8
Ramp	.1 2, .1 3, .1 4, .4 5, .3 6, 1 7, .6 8, .1 9, .3 10 .14 2, .14 3, .14 4, .43 5, .43 6, 1 7, .57 8, .28 9, .28 10

Relative Importance of Ramp Characteristics

Transition Curve	1 2, .5 3, .8 4, .8 5, .8 7, .8 8, .3 9, .5 10 1 2, .75 3, .75 4, .75 5, .75 6, .75 7, .75 8, .5 9, .5 10
Compound Curve	.6 2, .2 3, .2 4, 1 5, .4 6, .4 7, .4 8, .2 9, .8 10 .4 2, .4 3, .4 4, 1 5, .6 6, .6 7, .6 8, .6 9, .6 10
Radius Adequacy	.2 3, .2 4, .4 6, .4 7, .8 8, 1 9, .8 10 .2 3, .2 4, .2 5, .4 6, .4 7, .8 8, 1 9, .8 10
Outside Curb	.2 0, .2 4, .8 5, .8 6, .2 7, 1 8, .6 9, .2 10 .2 4, .8 5, .8 6, .8 7, 1 8, .6 9, .2 10

(continued)

Relative Importance of Ramp Characteristics (continued)

Drop > 4 Inches	.2 2, .6 4, .2 5, 1 6, .2 7, .6 8, .6 9, .6 10 .2 2, .2 3, .6 4, .6 5, 1 6, .6 7, .6 8, .6 9, .6 10
Cross-Slope Difference	.3 2, .3 3, 1 4, .8 5, .8 6, 1 7, .5 8, .3 9 .25 2, .25 3, 1 4, 1 5, 1 6, 1 7, .5 8, .25 9
Lane Width	.3 0, .3 1, 1 2, .3 3, .7 4, 1 5, 1 6, .7 7, 1 8, .3 1 .33 0, .5 1, .67 2, .67 3, .67 4, 1 5, 1 6, 1 7, 1 8, .66 9, .33 10
Downgrade	.5 2, .3 3, .8 4, .8 5, 1 6, .5 8, 1 9, .3 10 .25 2, .25 3, .75 4, .75 5, 1 6, 1 7, 1 8, 1 9, .25 10

Factor for Interchange

	.3 0, .1 1, .6 3, 1 5, .3 6, .1 10 .28 0, .37 1, .47 2, .57 3, .78 4, 1 5, .28 6
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