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Impact of Specific Geometric Features on Truck Operations and Safety at Interchanges

Final Technical Report
Volume I

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16. Abstract

The problem of truck loss-of-control accidents on interchange ramps is examined from the viewpoint of the suitability of highway geometric design, given the pecular stability and control limitations of heavy-duty trucks. Accident records were used to identify specific ramps which were overinvolved in jackknife, rollover, and run-off-road accidents. The ramp geometries were represented in a complex simulation of the dynamic behavior of representative tractor-semitrailer combinations. The calculated responses of heavy vehicles on each ramp were studied to illustrate how highway design features may have influenced the known accident experience at the site. Results show that various aspects of the AASHTO policy on geometric design result in a very slim margin of safety for the operation of heavy trucks on exit ramps. Problem features included side friction factors, superelevation transitions, compound curves, deceleration lanes, ramp downgrades, curbs on curved ramps, and wet surface friction on high-speed ramps. Potential countermeasures for the identified problems are suggested. Recommendations include a careful scoping of the prevalence of "problem ramps," nationally, initiation of efforts by State highway engineers to apply these findings to ramps having a known truck problem, and informing truck drivers of the situation involving slim safety margins.

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1.0 INTRODUCTION

This document constitutes the final report of a study undertaken to identify the nature of truck loss-of-control accidents occurring on freeway interchange ramps and to determine the role which the geometric design of ramps has played in causing such accidents. Since a principal focus of past research conducted by The University of Michigan Transportation Research Institute (UMTRI) has been in the area of truck dynamic response, the study dwells upon the mechanics of vehicle behavior and its sensitivity to details in ramp design. Recognizing that accidents generally occur only when driver, vehicle, and roadway factors combine disadvantageously, the study also considers the control actions of the truck driver and the likely influence of ramp design on driver strategy.

The project has investigated the problem of truck accidents on ramps by means of an approach which links actual ramp sites with a truck simulation methodology. The national accident record was first examined to identify States and, subsequently, individual ramp sites which suffered an abnormally high incidence of truck loss-of-control accidents. Individual sites, for which paper copies of accident reports were available, were selected for detailed study. Engineering drawings from these sites were converted into the geometric input format needed to calculate truck maneuvering response in a rather complete simulation model. Simulated vehicle responses are then interpreted in a fashion which relates geometric design of the ramp to the specific stability and control problems which showed up in the accident reports.

The study has found that certain of the peculiar types of accidents which involve heavy-duty trucks[1] on interchange ramps are clearly relatable to ramp design features. In fact, a principal finding of the work is that the geometric design policy of the American Association of State Highway and Transportation Officials (AASHTO), by which virtually all highway design is guided in the United States, provides for only a very slim margin of safety for the operation of certain trucks on interchange ramps. Additionally, it is

[1]Implies single or combination trucks having power units in class 7 or 8 (for which the gross vehicle weight rating of the truck or tractor is 26,000 lbs and up).

seen that specific ramps further exacerbate the safety margin situation through designs which do not meet even the minimum requirements indicated by AASHTO policy. Beyond questions of geometry, it is also apparent that traffic control devices are often selected and placed such that the warning which truck drivers may need to compensate for marginal geometric design is not provided.

The report is organized through text and appendixes to provide coverage of both roadway and vehicle-response aspects of the study. Beginning with presentations of the accident picture, in sections 1 and 2, the methodology for selecting specific ramps is outlined. Before presenting a summary of the geometric design analysis for the selected ramps, the principles of geometric design are reviewed, in section 4.1, so that the reader who is unfamiliar with ramp design practices can be introduced to the subject. In section 4.2, then, six primary features of ramp design which were found to be marginal relative to recommended practice are presented, with individual ramp sites used as examples. Appendix A provides a complete set of reviews of the geometric design for each of the selected ramps.

Section 5 presents the results of the simulation study, together with discussion of the apparent implications of the vehicle response problems for geometric design. The same six primary design features which were found to be marginal, above, are cited in terms of the mechanisms by which truck loss-of-control is threatened. Parameters used to represent truck combinations in the simulation are presented in appendix B, and the detailed plots describing vehicle response in each of the simulation runs are provided in appendix C.

Countermeasures for expanding the "margin of safety" at sites exhibiting any of these design features are presented in section 6. The overall conclusions and recommendations deriving from this study are presented in section 7.

2.0 EARLIER STUDIES REGARDING THE RELATIONSHIP OF HIGHWAY ACCIDENTS TO INTERCHANGE DESIGN VARIABLES

A list of investigators who have examined the influence of some of these geometric and design variables on accidents and/or operational aspects would include Cirillo, Browner, Fisher, Lundy, Pinnell and Buhr, Gray and Kauk, Langsner, Mullins and Keese, Hill, Wattleworth, Drew, Buhr, and Yates [1-13].

A major work concerning the relationship between accidents and the geometric and traffic characteristics at interchanges is the 1969 study published by the Bureau of Public Roads [1]. Although this is based on data obtained from 1959 to 1965, the bulk of the Interstate system had been constructed and a large number and variety of interchanges from 20 States were represented. In addition, this report provides a concise tabulation and description of the kinds of interchanges and their elements.

In this study, accident counts and accident rates were determined for interchanges of various types (full cloverleaf, diamond, etc.), and multiple linear regression models were developed to permit estimation of accident rates as a function of such variables as ADT, geometric features (e.g., curvature, presence and type of shoulders, etc.), and the proportion of commercial vehicles in the traffic stream. In the specific analyses of accidents at "extreme" sites (those with an unusually high number of reported accidents), commercial vehicles were subdivided into large trucks and buses. In general, they were found to be underrepresented in the accident population relative to their presence in the traffic stream. The congestive effect of commercial vehicles in traffic was judged to contribute to the accident probability, however. Relatively little detail was presented regarding accident type, although some was available in the data set.

In the regression models developed by the Bureau of Public Roads study, the dominant factor predicting accidents was traffic volume. Geometric variables contributed 0 to 9 percent of the explanation of variability in the overall accident and injury rates, and from 5 to 20 percent for the various subunits of interchanges (various types of ramps, etc.). The authors indicated that the geometrics on Interstate quality roads were generally improved to the point where the variation in geometrics had little influence on accidents. This conclusion was confirmed by Browner, who indicated that this finding did not imply that accident data were totally useless with respect to determining the effects of the geometric variables on overall safety [2]. Fisher, in reviewing four years of accident records on interchanges which included a number of older facilities, stated that higher (geometric) design standards simply reduced the total number of accidents but that the same types of accidents occurred at all interchanges [3]. He noted that older designs, with their attendent high accident rates, were actually helpful as subjects for study since mechanisms relating design to accident causation were more apparent. As in various similar studies, the bulk of Fisher's considerations of accident data simply relates the gross layout of interchanges to differing scenarios of car-to-car or car-to-object impact. Loss-of-control accidents involving single vehicles were often not dealt with as a specific class. (An exception to this is found, for example, in the work by Lundy which showed single-vehicle accidents to comprise a much larger percentage of all accidents at off-ramps than at on-ramps [4].)

In Browner's study, which correlated mass accident data with design variables at individual interchange sites, an "extreme analysis" was conducted by which isolated sites having an unusually large number of accidents per year relative to other sites of the same type were studied in closer detail [2]. The results of this analysis were reported in a manner which did distinguish "commercial vehicles" as a group from other motor vehicles. While it was seen that the commercial vehicles exhibited accident rates which averaged approximately half that seen with other vehicles (correcting for nominal exposure rates which are also reported), it is perhaps worthy to note that the commercial vehicle accident involvement rate was highest on loop-type ramp legs. This observation would appear to agree with the general conclusion, from the present study, that tight-radius curves and low advisory speeds such as are found on loop-type connections seem to be problematical for heavy trucks. Nevertheless, results from many investigators show that the whole range of motor vehicle types is more heavily involved in accidents at ramps having relatively tight curvature.

Among various studies reported in the literature, the interchange elements that have been examined for their effect on accidents have included acceleration and deceleration lanes, as well as the part of the mainline roadway immediately connected with the ramps. Generally,the elements of ramp geometrics that have been examined by regression techniques and other statistical treatments of large-volume data, include:

1) exit or entrance function

2) ramp curvature

3) length of ramp

4) length of the constant-radius section and length of transition curves

5) ramp width

6) gradient

7) superelevation and cross section

8) sight distance

9) length of weaving section

10) shoulder width

11) delineation

12) surface type

13) ramp sequence (i.e., exit ramp followed by entrance ramp, or vice versa

14) distance between successive ramps

15) left-hand or right-hand ramp

16) length of speed-change lane

17) shape of speed-change lane

18) taper rate

19) convergence or divergence angle

20) signing

21) lighting

22) smooth curve or elongated curve

23) rural or urban

Several past studies indicate that the effect of traffic volume overshadows that of ramp geometrics (e.g., Cirillo et al. and Buhr et al. [1,12]. It should be noted, however, that these studies tended to focus on the effect of a single geometric feature on overall accidents, one at a time.

No thorough investigations were made of the effect of the factors influencing truck accidents alone, nor the effect on truck accidents due to interactions among these factors. In addition, no studies were seen to have adopted the type of indepth methodology applied here, regardless of vehicle type. That is, it appears that no prior investigators have sought to examine the significance of ramp geometry by simulating the actual motion response of vehicles for the specific geometric site layouts of accident-involved road sections. Looking to the vehicle control issues involved, however, such an exercise would have trivial results for all but the heavy-duty commercial vehicle whose maneuvering limits are rather near the roadway design limits. Thus, the study documented in this report differs from prior work topically in that it considers the influences of ramp geometry through direct simulation of vehicle response on example geometries.

3.0 ACCIDENT DATA ANALYSIS

3.1 Objectives and Methodology

The accident data analysis task had two objectives. The first was to identify a number of individual ramps that were attractive for the simulation of truck dynamic response. These ramps were chosen on the basis of an overrepresentation of large-truck involvements or a history of a significant number of truck accidents. The second objective was to identify, through study of the accident data, the causes of truck accidents on ramps, both for the sake of guiding the simulation work and suggesting avenues for development of possible countermeasures.

Review of the literature and computerized accident files indicated that the automated files of accident data collected at the national level do not contain enough detail to identify the specific interchange, intersection, or ramp on which an accident occurred. Examples of files in this category are those of the Bureau of Motor Carrier Safety (BMCS), and the Fatal Accident Reporting System (FARS) and National Accident Sampling System (NASS) of the National Highway Traffic Safety Administration (NHTSA). While a number of State files provide geographic location of each accident and hence serve to identify specific interchanges, for example, many do not identify the specific ramp, and none give sufficient detail to describe the sequence of the crash. It is also typical that a hazardous driver action such as "speed too fast" is coded, which is of little utility in synthesizing countermeasures.

Accordingly, it was necessary to obtain hard copies of original accident reports which usually contain a schematic sketch or diagram prepared by an investigating police officer. Such diagrams, along with notes in accompanying narratives, provide a good deal of information on the specific ramp, the location on the ramp, and details describing the accident.

The procedure used to select ramps for individual study was as follows: Copies of example accident reporting forms from all States, together with instructions and coding protocols used to prepare these reports in some cases, were reviewed for content to determine which States might have the capability for identifying (a) accidents involving large trucks on ramps and (b) the ramp or interchange at which they occurred. A number of candidate States were selected based on the absolute and relative number of accidents in the 1980 BMCS accident file. Twelve States were selected for tentative inclusion based on both the BMCS data and the review of their respective accident reports. Michigan was included because of the convenience to both the accident and highway data, the close cooperation between UMTRI and MDOT, and the ready access to sites selected in the State.

Each selected State was then asked to identify a number of ramps or interchanges with a history of large-truck accident involvements. It would be desirable to select ramps with high accident rates based on the number or frequency of truck traversals as a measure of exposure. We learned early, however, that classification counts are generally not available on ramps. Counts are available on through lanes, but these do not provide reliable measures of exposure on ramps even when the counts are taken near or at the interchange. If the interchange is in a rural agricultural region or in a suburban bedroom community, very little truck traffic is likely to use the ramps. In contrast, an interchange serving an area of heavy industry or a large truck stop will experience very heavy truck traffic.

The responses of the States varied, of course, depending on their data processing capabilities and the details of their accident and highway files. The responses which were found to contain all of the elements needed to support the analysis of truck accidents at specific ramps were chosen for the detailed study.

A number of candidate ramps were selected in California, Illinois, Maryland, Michigan, Ohio, and Texas. Copies of the accident reports and copies of the police reports on the truck accidents at these sites were obtained, as well as construction plans giving their geometry. The accident reports were then examined to determine which ramps had a number of accidents that were related to geometrics and vehicle dynamics and therefore would be candidates for simulation.

The examination and review of accident reports consisted of evaluating each case and judging it either "relevant" or "irrelevant." Cases classed as irrelevant (to truck dynamics and ramp geometry) included those in which passenger cars or other vehicles were clearly culpable, such as a car colliding into the rear of a truck, or a car traveling the wrong way on a ramp. A substantial number coded as "ramp" accidents actually occurred on through lanes, e.g., a vehicle improperly merging from a ramp. Merging accidents were classed as irrelevant unless geometry might have been a contributing factor. Cases resulting from mechanical failure, such as tire blow-out, were also considered irrelevant. Relevant cases were taken to include all those in which, for example, the truck's speed was noted to be excessive or which resulted in run-off-road, overturn, jackknife, or collision subsequent to loss of control. The number of relevant cases was totaled for each candidate ramp, and those with the greater numbers were selected for study.

The procedures outlined above are described in detail in the sections that follow.

3.2 Available Accident Data Systems

The primary instrument which provides the basis of State accident data systems is the traffic-accident reporting form used by investigating police officers. The forms used by each of the States were examined to determine if the information necessary to the project could be available in automated files. For a number of States, the instruction manuals were available, and for a few States the data file coding manuals were also available.

Two information elements that are essential to the project are (a) the identification or coding of large trucks--excluding pickups and other units under 10,000 lbs (4.54 Mg) GVW--and (b) the identification of ramps as the accident site. In order to associate the accident with specific interchange design parameters, or to select sites for analysis of accident experience, it was also necessary that the data structure promote sufficient geographic location information such that either the specific ramps could be identified or a cross-index was provided for accessing the hard-copy accident reports. The latter were also important if hard-copy reports were to be used to obtain narrative or diagrammatic information not coded in the State data system.

The State report forms were reviewed for these four data items. The results are shown in table 1, with notes to explain specific entries. Three States (Montana, South Dakota, and Vermont) are not included because reports were not available; Alaska and Hawaii were omitted because of their remoteness.

Documentation in the form of coding manuals is available for the State data systems of a few States, and these are indicated by a separate line of data. Digital files of State data are available at UMTRI for Michigan, Pennsylvania, Texas, and Washington. Information on these files is also included.

A "Y" in the list indicates that the desired information is explicitly included on the report form.

The report number which indexes the hard copy to the digital record is not shown on a number of the blank reports on file at UMTRI. States having such apparent deficiency are indicated by a question mark. However, nearly all States assign and insert such a number at the State data-processing center.

Vehicle descriptions in the reports vary widely, as indicated in the list. A number of States provide vehicle identification numbers (VIN). In principle, the VIN provides a reliable identification of the type of vehicle. In practice, the automated interpretation of truck VINs is difficult and would become a very expensive method of separating large truck involvements from those of all other types of vehicles.

3.3 Selection of Candidate States Using the BMCS Accident Data File

The file of truck accident reports submitted to the BMCS provides a convenient data set for comparing States. While not all accidents involving large trucks are reported to the BMCS, primarily because of the reporting threshold, the file is, nevertheless, the largest national file of heavy-truck accidents. The accident report form and data file identify accidents which

Table 1. Content review of State accident reporting forms.

State[1]	Report Number	Large Truck Identification	Ramp Identification[2]	Location
Alabama	?	Y	Y, (diagram)	Y
Alaska	-	-	-	-
Arizona	Y	VIN[7]	Y, (diagram)	Y
Arkansas	?	Combinations only[5]	Diagram	Y
File	Y	Y	Y	Y
California	Y	Y	Diagram	Y
Colorado	Y	Y	Diagram	Y
Connecticut	Y	VIN[7]	Diagram	Y
Delaware	Y	7	Diagram	Y
Dist of Columbia	Y	No	Diagram	Y
Florida	Y	"Truck"	Diagram	Y
Georgia	Y	Y	Y, (diagram)	Y
Hawaii	-	-	-	-
Idaho	?	Y	Y, (diagram)	Y
Illinois	Y	Y[7]	Diagram	Y
Indiana	Y	Trac-trail[5]	Y, (diagram)	Y
Iowa[3]	Y	Y	Y, (diagram)	Y
Kansas	Y	VIN[7]	Diagram[8]	Y
Kentucky	Y	?	Diagram[8]	Y
Louisiana	Y	No. of Axles	Y, (diagram)	Y
Maine	Y	Y	Diagram	Y
Maryland	y	Y[5]	Y, (diagram)	Y
Massachusetts	Y(?)	7	Y, (diagram)	Y
Michigan	Y	Y	Diagram	Y
File	Y	Y	Y	Y
Minnesota[3]	?	Trac-trail[5]	Y, (diagram)[8]	y
Mississippi	Y	Trac-trail[5]	Diagram	Y
Missouri	Y	7	Diagram	Y
Montana	-	-	-	-
Nebraska[3]	?	VIN[7]	Diagram	y
Nevada	Y	7	Y	Y
New Hampshire	Y	Y	Diagram	Y
File	Y	Y	Y	Y
New Jersey	Y	Y[6]	Diagram[8]	Y
New Mexico	Y	Y	Y, (diagram)	Y
New York	?	7	Diagram[8]	Y
North Carolina	Y	VIN[9]	Y, (diagram)	Y
North Dakota	Y	7	Diagram	Y
Ohio	Y	Y	Y, (diagram)	Y
File	Y	Y	Y, (rural only)	Y
Oklahoma	Y	?	Y, (diagram)	Y
Oregon	Y	Y	Y, (diagram)	Y
Pennsylvania	?	Y	Diagram	Y
File	Y	Y	Y	Y
Rhode Island	?	VIN	Diagram	Y

Table 1. (Continued)

State[1]	Report Number	Large Truck Identification	Ramp Identification[2]	Location
South Carolina	Y	Y	Y, (diagram)	Y
South Dakota	-	-	-	-
File	Y	Y	Y (prior to 1977)	Y
Tennessee[4]	?	Y	Diagram	Y
Texas	Y	Y	Diagram	Y
File	Y	Y	Y	Y
Utah	?	Y[10]	Diagram	Y
Vermont	-	-	-	-
Virginia	Y	?	Diagram	Y
File	?	?	Y	Y
Washington[11]	Y	No	Diagram	Y
File	Y	Y	No	No
West Virginia	?	Y	Y	Y
Wisconsin	Y	Y	Diagram	Y
Wyoming[4]	Y	Y[6]	Y, (diagram)	Y

[1] The information relates to the police officers report. Information on coding of computerized files is given on a second line labeled "File" for each state for which we now have data.

[2] A "Y" indicates the form explicitly provides for indicating a ramp accident. Most states provide space for an officers diagram of the accident. If such a space is provided, it is indicated parenthetically.

[3] A separate report form is used for driver-reported information ,providing similar information.

[4] A separate report form is used for driver-reported information, but including less detail.

[5] Pickups and other light trucks are,or may be, included with heavier straight trucks.

[6] Pickups or trucks under 10 kips are differentiated from heavier units, without further description.

[7] The vehicle description is given by the investigating officer's own choice of words, and may not differentiate the size of trucks.

[8] A complete drawing may be provided, or a checking of cursory impact configurations may be used without indicating a ramp involvement.

[9] The form also specifies 2-axle, 3-axle, and tractor-trailer trucks.

[10] Vehicle weight and Vin's are also provided for by the form.

[11] Driver report.

occurred on expressway entrance and exit ramps, but these accidents probably do not include those occurring on speed-change lanes.

The number of ramp accidents by State is shown in table 2, along with the total number of truck accidents in the file. It should be recognized that the numbers shown in table 2 do not represent the total large-truck accident experience either in individual States or in the entire country. The BMCS regulations require that the carrier report the accident when it is a fatal or injury-producing accident, or if property damage is greater than $2,000. Many of the property damage cases reported by police never appear in BMCS files. Comparison of the numbers of large-truck accidents on ramps in the BMCS file for 1980 and several State files indicates that the latter include many more than the BMCS file. The ratios are about four for Texas and Pennsylvania (1979) and over ten for Michigan.

Ideally, we would like to have compared the accident experience among the States on the basis of exposure, mileage on ramps, ramp traversals, or even total truck vehicle miles in the State. Unfortunately, since no such exposure measures were readily available, it was necessary to seek suitable surrogate measures. A simple measure of ramp overrepresentation is the proportion of accidents which occur on ramps (given in the fourth column). However, this does not account for the proportion of travel which is on ramps or the relative number of interchanges per mile of highway.

In the absence of reliable exposure measures, two alternatives have been used. One is the miles of fully controlled access highways. This was further divided into urban and rural mileage on the hypothesis that the relative urban mileage might correlate with the number of interchanges per State. The source of highway mileage data is "Highway Statistics: 1979" published by the Federal Highway Administration (FHWA).[1] The mileage figures for four States (Connecticut, Mississippi, Rhode Island, and Washington) appeared to be unreasonably low. For these States, the total interstate mileage as given in the "Interstate System Route Log and Finder List" published by FHWA was used,

1The data in the 1980 statistics are the first to be based on the merged Highway Performance Monitoring System (HPMS). Twenty-six States were not yet able to report data in the HPMS format; thus, data are missing for these States. The 1979 data were used because of this high missing data rate in the 1980 publications.

Table 2. List of 1980 BMCS truck accidents by State, miles of full-access-control highways, and population in millions.

State	Accidents Ramp N	Accidents Total N	Accidents percent On Ramp	Miles of Full Access-Control Hwy. Rural	Miles of Full Access-Control Hwy. Urban	Miles of Full Access-Control Hwy. Total	Population
Alabama	21	732	2.9	440	135	575	3.894
Alaska	6	137	4.4	-	-	-	.402
Arizona	16	448	3.6	1019	85	1104	2.718
Arkansas	16	415	3.9	511	110	621	2.286
California	115	1468	7.8	2390	1318	3708	23.668
Colorado	31	529	5.9	774	117	891	2.890
Connecticut	24	319	7.5	-	-	346	3.108
Delaware	2	88	2.3	21	4	25	.594
D. C.	1	53	1.9	-	-	-	.638
Florida	41	1087	3.8	1186	230	1416	9.746
Georgia	58	1003	5.8	1037	184	1221	5.463
Hawaii	0	12	0.0	12	28	40	.965
Idaho	8	242	3.3	393	32	425	.944
Illinois	126	1950	6.5	1550	354	1904	11.427
Indiana	74	1297	5.7	792	174	966	5.490
Iowa	36	589	6.1	683	110	793	2.914
Kansas	24	591	4.1	619	105	724	2.364
Kentucky	42	542	7.7	1221	47	1268	3.661
Louisiana	35	724	4.8	569	98	667	4.206
Maine	2	68	2.9	52	11	63	1.125
Maryland	42	568	7.4	361	30	391	4.217
Massachusetts	23	307	7.5	56	566	622	5.737
Michigan	56	803	7.0	1379	329	1708	9.262
Minnesota	13	354	3.7	651	267	918	4.076
Mississippi	14	369	3.8	-	-	698	2.521
Missouri	51	912	5.6	906	368	1274	4.917
Montana	8	241	3.3	856	22	878	.787
Nebraska	7	336	2.1	450	46	496	1.570
Nevada	11	142	7.7	488	25	513	.800
New Hampshire	1	54	1.9	129	119	248	.921
New Jersey	87	924	9.4	126	281	407	7.365
New Mexico	13	357	3.6	868	92	960	1.303
New York	68	1300	5.2	573	461	1034	17.558
North Carolina	34	799	4.3	932	145	1077	5.882
North Dakota	5	106	4.7	501	24	525	.653
Ohio	159	1952	8.1	996	403	1399	10.798
Oklahoma	25	616	4.1	516	235	751	3.025
Oregon	38	613	6.2	558	166	724	2.633
Pennsylvania	123	1944	6.3	1213	497	1710	11.864
Rhode Island	7	51	13.7	-	-	148	.947
South Carolina	21	460	4.6	737	25	762	3.122
South Dakota	2	131	1.5	621	18	639	.691
Tennessee	63	863	7.3	834	189	1023	4.591
Texas	132	2519	5.2	2145	1272	3417	14.229
Utah	18	254	7.1	602	110	712	1.461
Vermont	0	37	0.0	283	45	328	.511
Virginia	43	751	5.7	931	169	1100	5.347
Washington	37	608	6.1	-	-	717	4.132
West Virginia	15	414	3.6	375	22	397	1.950
Wisconsin	34	607	5.6	670	189	859	4.706
Wyoming	15	435	3.4	840	30	870	.470
Missing Data	9	136	6.6				
Total	1852	32257	5.7	32879	9287	45368	-

but without the rural/urban split. State population, in millions, from the 1980 census is also included as an alternate exposure normalizer for truck travel.

A number of regressions were used to examine measures of overrepresentation of ramp accidents among the States using the highway mileage and population. Specifically, both forward and backward stepwise regressions of both the number of accidents on ramps (column 2) and the percentage of accidents which were on ramps (column 4) were run using the following independent variables:

1) total mileage of fully controlled access highway

2) proportion of the highway mileage which is urban

3) population

4) product of mileage and population (as an interaction term)

The models which resulted from the stepwise regressions were the following: for the number of ramp accidents--mileage, population, and the product of mileage and population; and for percent of accidents on ramps--population alone. Regressions using only these variables were then run so that cases (States) with missing data on the nonselected variables (namely, proportion of urban mileage) would be included. The residuals (excess of the actual number over the number predicted by the regression) may be interpreted as measures of overrepresentation.

Four variables have thus been developed which relate to total number and overrepresentation of ramp accidents in possible candidate States. These are (1) the total number of ramp accidents, (2) the percentage of accidents which occurred on ramps, (3) the residual of the number of ramp accidents in a regression model, and (4) the residual of the percentage on ramps in a regression model. Tables 3 through 6 give the States in decreasing order of each of these four variables.

It should be emphasized that the regressions are not intended to represent reliable predictive models, nor was an exhaustive search for reliable predictors undertaken. The objective was to develop a list of States

Table 3. List of 1980 BMCS truck accidents which are on ramps by State, in decreasing order of number on ramps.

	State	Accidents Ramp N	Accidents Total N	percent On Ramp
1	Ohio	159	1952	8.1
2	Texas	132	2519	5.2
3	Illinois	126	1950	6.5
4	Pennsylvania	123	1944	6.3
5	California	115	1468	7.8
6	New Jersey	87	924	9.4
7	Indiana	74	1297	5.7
8	New York	68	1300	5.2
9	Tennessee	63	863	7.3
10	Georgia	58	1003	5.8
11	Michigan	56	803	7.0
12	Missouri	51	912	5.6
13	Virginia	43	751	5.7
14	Kentucky	42	542	7.7
15	Maryland	42	568	7.4
16	Florida	41	1087	3.8
17	Oregon	38	613	6.2
18	Washington	37	608	6.1
19	Iowa	36	589	6.1
20	Louisiana	35	724	4.8
21	North Carolina	34	799	4.3
22	Wisconsin	34	607	5.6
23	Colorado	31	529	5.9
24	Oklahoma	25	616	4.1
25	Connecticut	24	319	7.5
26	Kansas	24	591	4.1
27	Massachusetts	23	307	7.5
28	Alabama	21	732	2.9
29	South Carolina	21	460	4.6
30	Utah	18	254	7.1
31	Arkansas	16	415	3.9
32	Arizona	16	448	3.6
33	West Virginia	15	414	3.6
34	Wyoming	15	435	3.4
35	Mississippi	14	369	3.8
36	Minnesota	13	354	3.7
37	New Mexico	13	357	3.6
38	Nevada	11	142	7.7
39	Idaho	8	242	3.3
40	Montana	8	241	3.3
41	Nebraska	7	336	2.1
42	Rhode Island	7	51	13.7
43	Alaska	6	137	4.4
44	North Dakota	5	106	4.7
45	Delaware	2	88	2.3
46	Maine	2	68	2.9
47	South Dakota	2	131	1.5
48	New Hampshire	1	54	1.9
49	D.C.	1	53	1.9
50	Hawaii	0	12	0.0
51	Vermont	0	37	0.0

Table 4. List of 1980 BMCS truck accidents which are on ramps by State, in decreasing order of percent on ramps.

	State	Ramp N	Accidents Total N	percent On Ramp
1	Rhode Island	7	51	13.7
2	New Jersey	87	924	9.4
3	Ohio	159	1952	8.1
4	California	115	1468	7.8
5	Nevada	11	142	7.7
6	Kentucky	42	542	7.7
7	Connecticut	24	319	7.5
8	Massachusetts	23	307	7.5
9	Maryland	42	568	7.4
10	Tennessee	63	863	7.3
11	Utah	18	254	7.1
12	Michigan	56	803	7.0
13	Illinois	126	1950	6.5
14	Pennsylvania	123	1944	6.3
15	Oregon	38	613	6.2
16	Iowa	36	589	6.1
17	Washington	37	608	6.1
18	Colorado	31	529	5.9
19	Georgia	58	1003	5.8
20	Indiana	74	1297	5.7
21	Virginia	43	751	5.7
22	Missouri	51	912	5.6
23	Wisconsin	34	607	5.6
24	New York	68	1300	5.2
25	Texas	132	2519	5.2
26	Louisiana	35	724	4.8
27	North Dakota	5	106	4.7
28	South Carolina	21	460	4.6
29	Alaska	6	137	4.4
30	North Carolina	34	799	4.3
31	Kansas	24	591	4.1
32	Oklahoma	25	616	4.1
33	Arkansas	16	415	3.9
34	Mississippi	14	369	3.8
35	Florida	41	1087	3.8
36	Minnesota	13	354	3.7
37	West Virginia	15	414	3.6
38	Arizona	16	448	3.6
39	New Mexico	13	357	3.6
40	Wyoming	15	435	3.4
41	Idaho	8	242	3.3
42	Montana	8	241	3.3
43	Alabama	21	732	2.9
44	Maine	2	68	2.9
45	Delaware	2	88	2.3
46	Nebraska	7	336	2.1
47	New Hampshire	1	54	1.9
48	Dist of Columbia	1	53	1.9
49	South Dakota	2	131	1.5
50	Hawaii	0	12	0.0
51	Vermont	0	37	0.0

Table 5. States ordered on residual of number of ramp accidents reported to the BMCS in 1980.

	State	Actual Number	Predicted Number	Residual
1	Ohio	159	84.522	74.478
2	New Jersey	87	48.270	38.730
3	Illinois	126	93.961	32.039
4	Pennsylvania	123	93.445	29.555
5	Indiana	74	48.545	25.455
6	Tennessee	63	44.832	18.168
7	Maryland	42	27.029	14.971
8	Texas	132	117.099	14.901
9	Oregon	38	25.682	12.318
10	Delaware	2	-8.428	10.428
11	Rhode Island	7	-1.940	8.940
12	Iowa	36	29.261	6.739
13	Connecticut	24	18.452	5.548
14	Maine	2	-3.419	5.419
15	Hawaii	0	-5.273	5.273
16	Georgia	58	54.192	3.808
17	Nevada	11	8.058	2.942
18	West Virginia	15	12.130	2.870
19	Washington	37	34.620	2.380
20	Idaho	8	6.305	1.695
21	Louisiana	35	33.827	1.173
22	New Hampshire	1	.754	.246
23	Kansas	24	24.047	-.047
24	Utah	18	18.202	-.202
25	Vermont	0	.424	-.424
26	Colorado	31	31.778	-.778
27	Missouri	51	52.591	-1.591
28	Wyoming	15	17.099	-2.099
29	North Dakota	5	7.483	-2.483
30	Oklahoma	25	28.792	-3.792
31	Kentucky	42	45.981	-3.981
32	Arkansas	16	20.671	-4.671
33	Nebraska	7	12.518	-5.518
34	Wisconsin	34	41.530	-7.530
35	Virginia	43	50.809	-7.809
36	Alabama	21	29.560	-8.560
37	South Carolina	21	29.672	-8.672
38	South Dakota	2	11.250	-9.250
39	Mississippi	14	24.280	-10.280
40	Montana	8	19.193	-11.193
41	New Mexico	13	24.642	-11.642
42	California	115	128.619	-13.619
43	North Carolina	34	53.211	-19.211
44	Massachusetts	23	42.281	-19.281
45	Arizona	16	36.625	-20.625
46	Michigan	56	81.978	-25.978
47	Minnesota	13	39.327	-26.327
48	Florida	41	79.602	-38.602
49	New York	68	117.063	-49.063
50	Alaska	6	-	-
51	Dist of Columbia	1	-	-

Table 6. States ordered on residual of percent of accidents reported to the BMCS in 1980 which were ramp accidents.

	State	Actual Percent	Predicted Percent	Residual
1	Rhode Island	13.700	4.390	9.310
2	New Jersey	9.400	5.567	3.833
3	Nevada	7.700	4.364	3.336
4	Kentucky	7.700	4.888	2.812
5	Connecticut	7.500	4.787	2.713
6	Utah	7.100	4.485	2.615
7	Maryland	7.400	4.990	2.410
8	Tennessee	7.300	5.059	2.241
9	Massachusetts	7.500	5.269	2.231
10	Ohio	8.100	6.197	1.903
11	Oregon	6.200	4.700	1.500
12	Iowa	6.100	4.751	1.349
13	Colorado	5.900	4.747	1.153
14	Washington	6.100	4.975	1.125
15	Michigan	7.000	5.915	1.085
16	Georgia	5.800	5.219	.581
17	Wisconsin	5.600	5.080	.520
18	Virginia	5.700	5.197	.503
19	Missouri	5.600	5.118	.482
20	Indiana	5.700	5.224	.476
21	North Dakota	4.700	4.337	.363
22	Illinois	6.500	6.312	.188
23	Alaska	4.400	4.291	.109
24	Pennsylvania	6.300	6.392	-.092
25	Louisiana	4.800	4.988	-.188
26	South Carolina	4.600	4.789	-.189
27	Kansas	4.100	4.650	-.550
28	Oklahoma	4.100	4.772	-.672
29	Arkansas	3.900	4.636	-.736
30	California	7.800	8.557	-.757
31	New Mexico	3.600	4.456	-.856
32	Mississippi	3.800	4.679	-.879
33	Wyoming	3.400	4.303	-.903
34	West Virginia	3.600	4.574	-.974
35	North Carolina	4.300	5.295	-.995
36	Montana	3.300	4.361	-1.061
37	Idaho	3.300	4.390	-1.090
38	Arizona	3.600	4.715	-1.115
39	Minnesota	3.700	4.964	-1.264
40	Maine	2.900	4.423	-1.523
41	Texas	5.200	6.826	-1.626
42	Delaware	2.300	4.326	-2.026
43	Alabama	2.900	4.931	-2.031
44	Florida	3.800	6.004	-2.204
45	New York	5.200	7.436	-2.236
46	Nebraska	2.100	4.505	-2.405
47	Dist of Columbia	1.900	4.334	-2.434
48	New Hampshire	1.900	4.386	-2.486
49	South Dakota	1.500	4.344	-2.844
50	Vermont	0.000	4.311	-4.311
51	Hawaii	0.000	4.394	-4.394

which should be included for consideration, and which should be further evaluated on the basis of availability of appropriate data and cooperation in providing accident and geometric data.

Those States which appear among the top ten on two or more of the lists were selected as a set appropriate for consideration as candidate States. Michigan was added because of the convenience of proximity, and because of past cooperation between Michigan agencies and UMTRI.

While Connecticut, Massachusetts, Nevada, and Rhode Island all appeared on the list, they were eliminated because of their small number of ramp accidents. The candidate States were:

1) Ohio
2) New Jersey
3) Tennessee
4) Maryland
5) Illinois
6) California
7) Pennsylvania
8) Texas
9) Indiana
10) Kentucky
11) Michigan

Table 2 indicates that the above States all collect appropriate and necessary data, although the vehicle identification in New Jersey, Maryland, and Illinois is marginal.

With the assistance of the sponsor, each of these States was contacted and asked for assistance. Each was asked to identify approximately six ramps (or interchanges) which have had a substantial involvement of large trucks in accidents. The selection was to be based on overinvolvement relative to average daily truck traffic on the ramp if this were available, or on large number of accidents if the truck ADT were not available. Geometric reports were also requested. Since Indiana, Kentucky, Pennsylvania, and Tennessee were unable to provide hard copies of police reports, they were not included in the consideration of individual ramp sites. The responses of the other

States were all positive but varied in details, which will be discussed separately.

Michigan -- The Michigan Department of Transportation (MDOT) provided information on the frequency of large-truck involvements on ramps using their accident files and the Michigan Accident Location Index (MALI). The MALI system provides automation of the geographic location of accidents with some information on location with respect to the elements of interchanges. The information provided was based on accident data on State trunklines from 1979 through 1981, and provides listings of individual interchanges which had eight or more large-truck involvements on ramps in the 3-year period.

Michigan has approximately 650 interchanges. Over the 3-year period, 1,543 large trucks were involved in accidents on ramps or connectors of 385 of these interchanges. Thus, 265 of the 650 interchanges had no large-truck accident experience. An additional 338 sites had fewer than 8 involvements in the 3 years. Nearly half of the truck involvements, 696, occurred at only 47 sites. The majority of these accidents (514) occurred in southeastern Michigan, with 305 in Wayne County--which includes Detroit. Those sites in southeastern Michigan which had over 10 involvements in 3 years number only 14, but accounted for 331 involvements or over 20 percent of those in the entire State. This nonuniform distribution is largely the result of the high traffic density in the southeastern corner of the State. To provide study sites from less urbanized areas, contacts were made with the district engineers of each of the southern highway districts which include interchanges on limited-access highways.

Accident reports were obtained for seven interchanges with the greatest truck involvement--five from the Detroit area and two in rural areas. A total of 238 accident reports were examined, and based on the review, 5 ramps were selected for continued study.

The predominant accidents at each of the sites were rollovers and jackknifes. As-constructed plans were obtained for the selected ramps, giving horizontal and vertical alignment, as well as the superelevation for each.

Maryland -- The Maryland Department of Transportation selected four sites for consideration. They provided 70 reports of large-truck involvements on ramps at the sites. Drawings giving the geometrics of the ramps were also provided. After review of the accident reports, one ramp was selected for simulation involving a large number of accidents which had occurred on wet pavement--both ran-off-road and jackknifes.

Ohio -- The Ohio Department of Transportation selected 6 interchanges and provided drawings and geometric data on the sites, along with copies of 80 accident reports. On the basis of the frequency and details of the 80 accidents, 5 ramps were selected for study.

California -- The California Department of Transportation suggested 9 possible ramps and provided geometric plans and 130 accident reports. Two ramps were selected for study, both of which had a history of truck rollovers but now incorporate advisory signs with a symbol depicting a truck which is rolling over, as emphasis for a speed advisory.

Illinois -- The Illinois Department of Transportation provided listings giving number of ramp and gore area truck accidents at all locations, with such involvements by county, route, intersecting route, and milepost. The locations of 2,747 accidents were listed for 1980 to 1982. The number of accidents by interchange were determined and the accident reports obtained for the five sites having the greatest number of accidents. The single ramp having the greatest incidence of truck involvements in which geometry was apparently a factor was selected for study.

Texas -- The Department of Highways and Public Transportation of Texas generated a tabulation of accident summaries (a number of key variables on each accident) for 476 large-truck accidents on ramps in 1981 to 1983. The accidents were tabulated by county and control section. Enough detail was tabulated to provide the milepoint of the crash, but failure to determine the involved ramp at each interchange led to the exclusion of Texas sites from the study.

New Jersey -- The New Jersey Department of Transportation provided 58 reports of truck accidents at 6 interchanges. Unfortunately, the report format did not provide sufficient graphic information to evaluate the role of either geometry or vehicle dynamics in the crashes. Because of this limitation, none of the New Jersey sites were selected for simulation and geometric specifications were not requested.

3.4 Summary of Site Selection

A total of 15 ramps at 11 interchanges in 5 States were selected for study by simulation. Each of the interchange ramps for which hard copies of accident reports were reviewed is described in appendix A, along with an analysis of the geometric design of the ramp. A total of over 800 accident reports were individually reviewed and evaluated. While scrutiny of these reports revealed that a number of the truck accidents were unrelated to the interests of this study, those seen as "relevant" were of great value in indicating the type of control loss problem which should be simulated.

3.5 Review of Indepth Accident Investigations

As a supplement to accident analyses described above, individual cases of large-truck accidents at interchanges which had been reported by indepth investigation teams were identified by computer, and the case material retrieved and reviewed. The purpose of this review was to provide some insight into the causes and circumstances of such involvements, using the results of investigations that provide much more detail than police reports. Insight gained from inspecting indepth reports assisted in formulating and interpreting computer simulations of generic vehicle behavior on the selected ramps.

Because of the coding conventions used, not all ramp accidents in the computerized file of indepth reported cases are related to truck performance characteristics or interchange design. Of the 52 ramp-related cases in the file, 29 were found to be irrelevant to the project. These included, for example, cases of accidents on through lanes that happened to occur near or beside entrance/exit ramps, accidents that resulted from traffic weaving as a consequence of a car improperly merging on entrance, and four cases of crashes induced by mechanical failure.

Three accidents involved the rear-end impact of other vehicles into trucks which had entered a freeway. One case involved the rear impact of a passenger car into a truck which was traveling much slower than the other traffic (for lack of an acceleration lane). Another case of rear impact resulted from evasive maneuvers of a truck attempting to avoid a braking car.

The remaining 23 cases were all single-vehicle accidents on ramps. Sixteen (70 percent) of these involved trucks which had lost control on a curved ramp and either ran off the road or overturned on the road. Ice or snow was undoubtedly a factor in three of these cases.

The file of indepth investigations is not a representative sample of any population of accidents. The truck-interchange accidents, for example, have a geographic basis. Over 80 percent of the available cases are from southern California (urban) or southeastern Michigan. This bias results both from the localized assignment of the teams collecting data and from the relative length of time that the various teams were in operation. Because of such biases and the small number of cases, meaningful statistics cannot be derived from these data. However, some insight into typical truck-interchange accidents is provided.

For example, curbs were involved as "tripping agents" in four cases--an observation which tends to confirm one of the principal findings of this study. In three cases, the drivers implicated load shifts. Other factors that appeared in the case reviews are limited sight distance and unanticipated reduction of radius of curvature either because of compound curves or because of lane changes being conducted while traveling on two-lane ramps. In a number of the cases, rollover occurred near the end of the ramp after leaving skid marks for a substantial distance.

Excessive speed (for the particular design) was the single most frequently implicated factor identified by the investigators.

4.0 CONSIDERATIONS OF GEOMETRIC DESIGN OF RAMPS

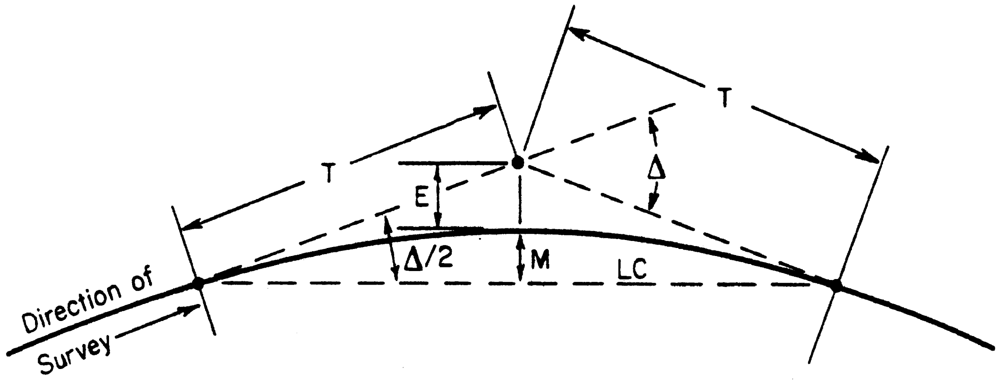
The geometric design of freeway ramps, as well as all other elements of the highway system, are guided by the AASHTO design policy. In order to provide an orderly basis for the discussion of selected ramp sites which have been represented in dynamic simulation of truck response, this section first presents a simplified overview of ramp design practice. This overview is basically a condensation of the AASHTO policy as it pertains to the primary geometric elements of ramp design, particularly treating those features which are highlighted later as having importance for the stability and control of heavy-duty vehicles.

Next, the geometric features of the 15 selected ramps are summarized. The reader is referred to appendix A for a complete review of each site, from the viewpoint of its nominal adherence to the recommended design policy.

4.1 Overview of Ramp Design Practice

The design practices pertaining to highway ramps will be reviewed with reference to both the prior design policy as presented in the "Blue Book" [14] and the current policy as presented in the "Green Book" [15]. Persons wishing to study the rationale behind AASHTO design policy, as well as the numerical and graphical form of the design guidelines, should consult the policies directly. The geometric design of ramps is discussed in separate subheadings below. The general elements of a horizontal circular highway curve are defined first, and then each of the individual design features are discussed in turn.

General Elements. Detailed in figure 1a is a simple horizontal circular highway curve without transitions. The figure illustrates the complete set of terms referring to geometry in the horizontal plane. Obviously, many of these terms are redundant in terms of actually defining the curve, but they have value to surveyors and those who lay out highways for construction. The four elements of chief importance in assessing horizontal highway curves are the point of curvature, PC, the point of tangency, PT, the radius, R (or degree of curvature, D), and the curve length, L. These terms will be used throughout



PC = Point of curvature
PI = Point of intersection
PT = Point of tangency
E = External distance
M = Middle ordinate distance
R = Length of radius of curve
T = Length of tangent (PC to PI and PI to PT)
D = Degree of curve (angle subtended at the center of curve by an arc of 100 feet)
L = Length of curve, ft.
LC = Long chord
Δ = External angle, deg.

$T = R \text{ tangent } \frac{\Delta}{2}$

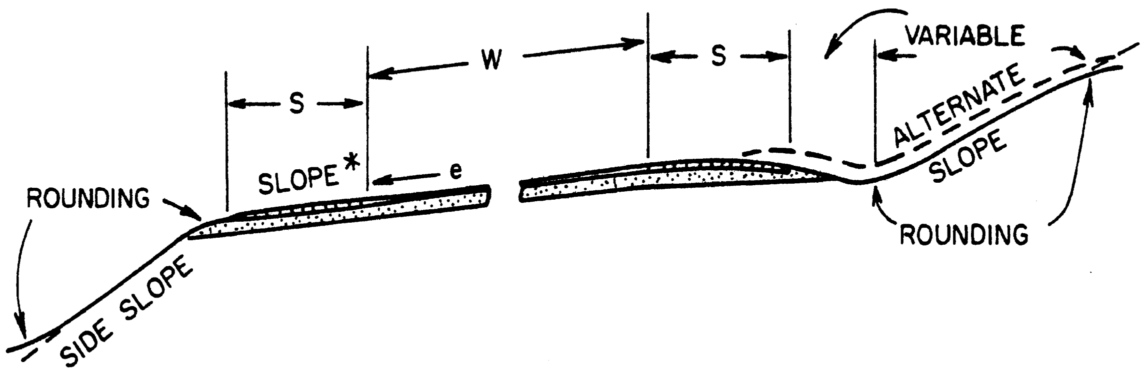
$E = R \text{ exsecant } \frac{\Delta}{2}$

$L = 100 \frac{\Delta}{D}$

$M = R \text{ versine } \frac{\Delta}{2}$

$LC = 2R \sin \frac{\Delta}{2}$

(a)



W = road width
S = shoulder
e = superelevation

(b)

Figure 1. Terminology employed in the design of horizontal curves.

this report to describe and analyze the curve geometry at individual sites. These elements describing the horizontal layout of curves are supplemented with the superelevation slope, e, resulting in a side friction factor, f, given the speed of vehicles traversing the curve.

Superelevation. Since curved sections of highway impose a centripetal acceleration upon vehicles, superelevation is employed in order to limit the lateral forces which the tires must generate and to assure comfort and ease-of-control for the driver. The term superelevation refers to the banking, or elevation of one edge of the pavement lane relative to the other, in curved highway sections. Superelevation is typically expressed in units of feet/foot describing the rate of superelevation developed across the road width. For example, a single-lane 18-foot-wide (5.5-m) roadway, whose difference in pavement edge elevations is 1.44 feet (.44 m), is superelevated at a rate of .08 feet/foot (1.44/18 = .08). Figure 1b details the cross section of a superelevated roadway. Maximum superelevation rates range from .07 or less in areas where ice and snow are common to .12 feet/foot in warm climates where very low pavement friction levels rarely, if ever, prevail. The basis for selecting the maximum value of superelevation to be achieved in a given curve will be presented later in conjunction with the determination of side friction factor.

Superelevation Development. While superelevated curves are beneficial, the manner of attaining the prescribed superelevation is important. The transition from a normal crowned cross section to a superelevated one must be gradual, and ideally not require the driver to reduce speed or make sudden steering maneuvers. For developing the full level of superelevation on curves, two types of transition sections are widely used, namely, a straight connecting transition length, or a connecting spiral curve.

A straight connecting transition length constitutes a portion of the tangent preceding the curve over which some fraction of the maximum superelevation value is attained. This type of transition design is the most commonly used. The highway lane is typically revolved about either the centerline, inside, or outside pavement edge over a substantial length until the full level of superelevation is achieved. The total transition length is composed of two length elements, the tangent runout, and the runoff length.

The tangent runout refers to the length necessary to remove the normal road crown. Some States, however, simply superelevate the roadway without removing the crown. In this instance, a tangent runout length is not employed. Tangent runouts range from 25 ft to 60 ft (7.6 m to 18.3 m), depending upon the road crown value being removed. The runoff length directly follows the tangent runout and is the length along which the maximum superelevation value is developed. Minimum runoff lengths range from 100 ft to 250 ft (30.5 m to 76.2 m). Lengths shorter than 100 ft (30.5 m) are to be avoided as they result in too abrupt a change in the highway profile.

Runoff lengths are determined recognizing that for appearance and comfort the runoff should not exceed a longitudinal slope of 1:200 with respect to the centerline. Noting that the desired length of the transition is speed dependent, maximum relative profile gradients for varying speeds have been outlined by AASHTO.

Placement of the runoff length with respect to the point of curvature (PC) and point of tangency (PT) varies from State to State. However, AASHTO recommends that at least 50 percent of the runoff length be located on the tangent portion preceding the PC and likewise beyond the PT of the curve when returning again to a normal road crown. AASHTO has concluded that the achievement of 60 percent of the superelevation on the tangent should be taken as the normal range for design control. When spiral transitions are employed, it is standard practice that the full level of superelevation be achieved on the spiral. Figure 2 is a plan view of a horizontal highway curve. Included in the figure are the transition elements discussed above, illustrating the typical sequence with which the maximum superelevation is developed in the horizontal curve.

Connecting a spiral curve to the curve PC and PT in order to attain and return from a maximum superelevated curve is also a widely accepted method. This method is encouraged when superelevating a sharp curve or connecting two curves with substantially varying radii. The length of the spiral can be determined by several means, namely, Euler's spiral equation or a clothoid, the Short equation originally developed for railroad track curves and included in the AASHTO policy, or Barnett's Transition Curve Tables. While using any of these methodds is acceptable practice, the selection of spiral length equal to

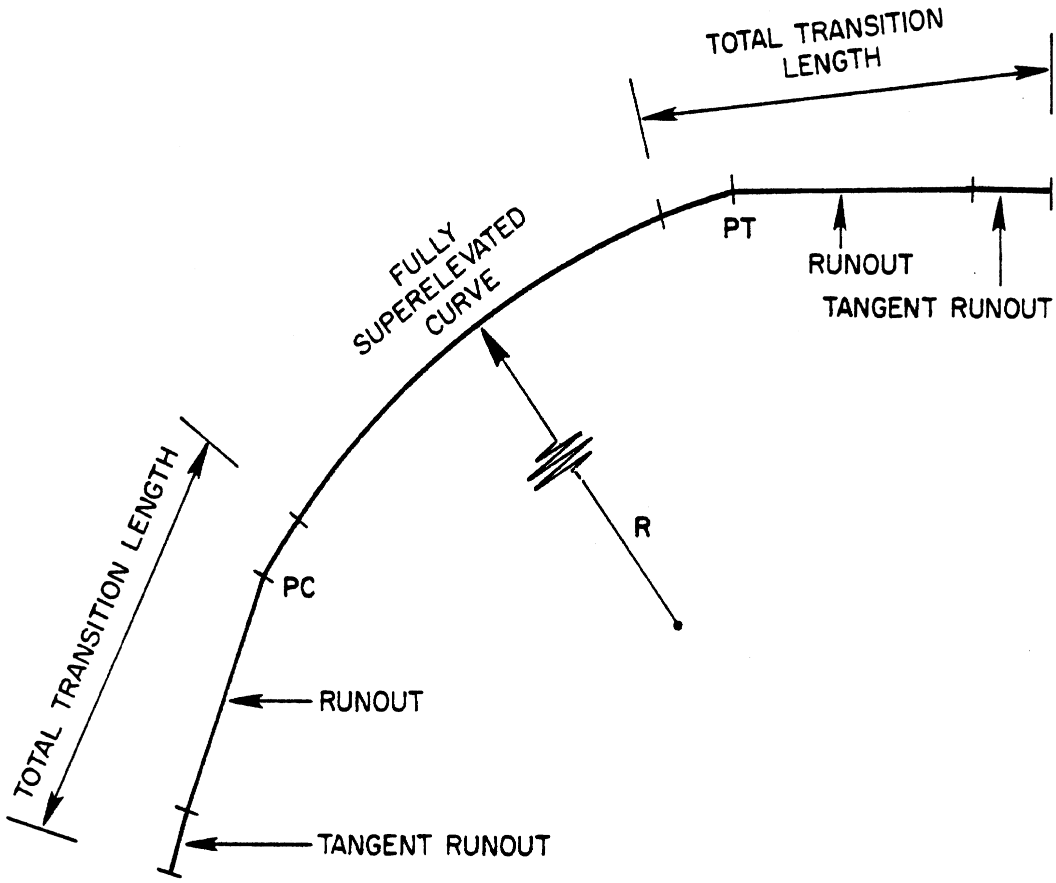


Figure 2. Terminology used in describing a straight section connecting transition.

the required transition runoff length discussed previously constitutes a practical and sufficiently accurate guideline for highway design.

Figure 3 illustrates the layout for connecting spirals as the means to develop superelevation. The use of this method assumes that the tangent runout has already been achieved such that the spiral takes the place of the runoff length discussed in the previous method. The TS notation refers to the point tangent to the spiral or the beginning of the spiral curve. Conversely, the SC refers to the point at which the spiral meets the constant radius curve.

Clearly, a spiral developing the maximum superelevation prior to the point of curvature provides a uniform means of applying superelevation to match the development of centripetal acceleration in vehicles. Such designs contrast with the situation of the "superelevation deficiency" prevailing in curves which have superelevation incompletely developed at the point of curvature. Nevertheless, spiral curves are not typically employed in U.S. highways.

Minimum Radius Equation. Superelevating curved highway sections counteracts the centripetal acceleration acting on the vehicle, and thus reduces the lateral force or "side friction" between the tires and pavement. The "minimum radius equation" simply expresses the steady-state relationship between curve radius, vehicle speed, superelevation, and side friction factor, as outlined in figure 4. Given constraints upon the tolerable levels of side friction factor and superelevation, a minimum value of radius is determined for a target design value of vehicle speed.

Side Friction Factor. The tolerable levels of side friction factor have been determined primarily through experiments addressing the threshold of discomfort of passenger-car drivers. These studies have illustrated that driver tolerance of the net acceleration level experienced along the lateral axis of the vehicle, equal in g's to the side friction factor, declines with vehicle speed. In order to assure comfortable driving through curves, AASHTO has defined maximum design values of side friction level as a function of speed. Based upon comfort considerations, these limits have been seen as affording ample levels of safety margin against the potential for "side

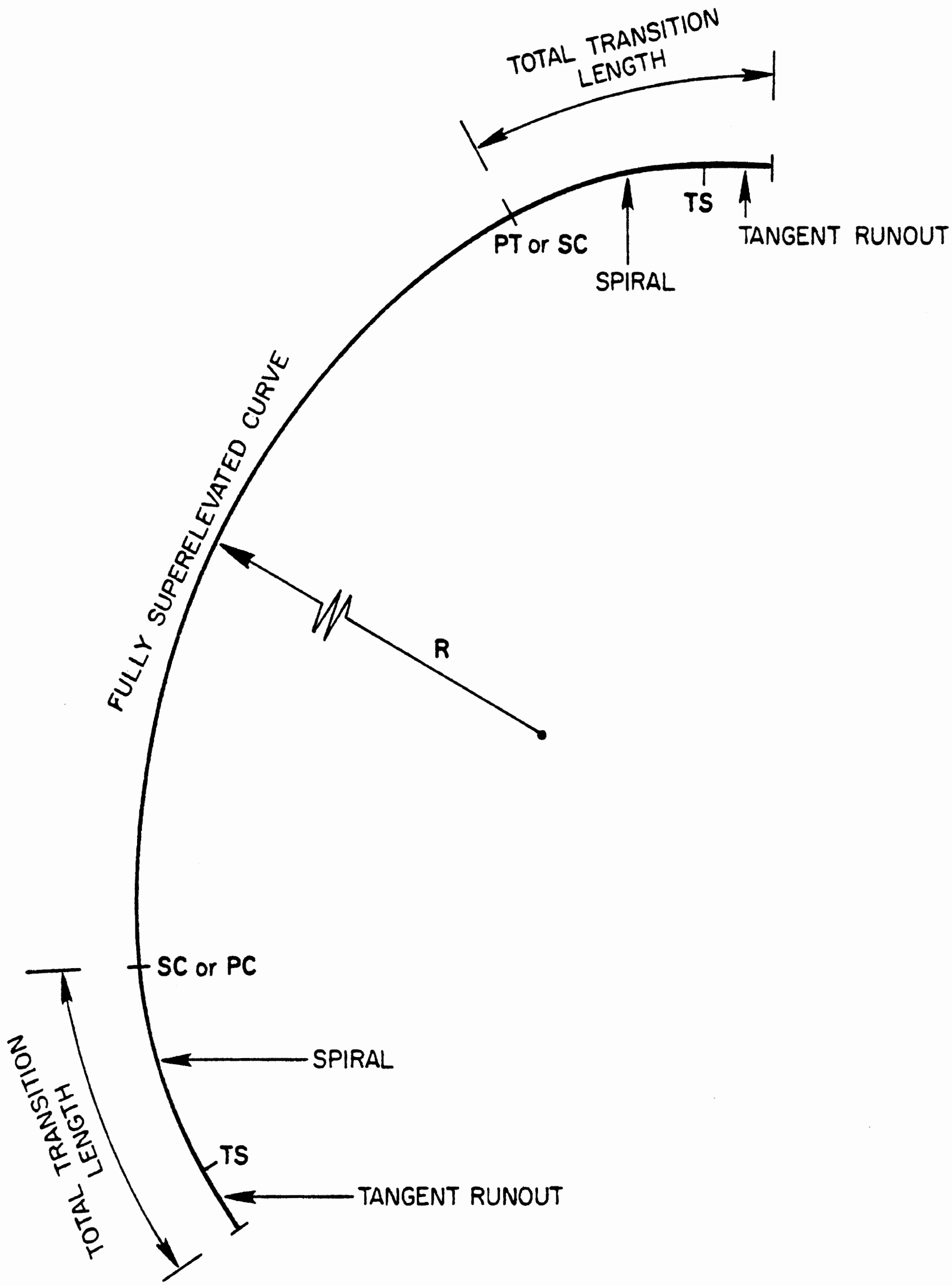
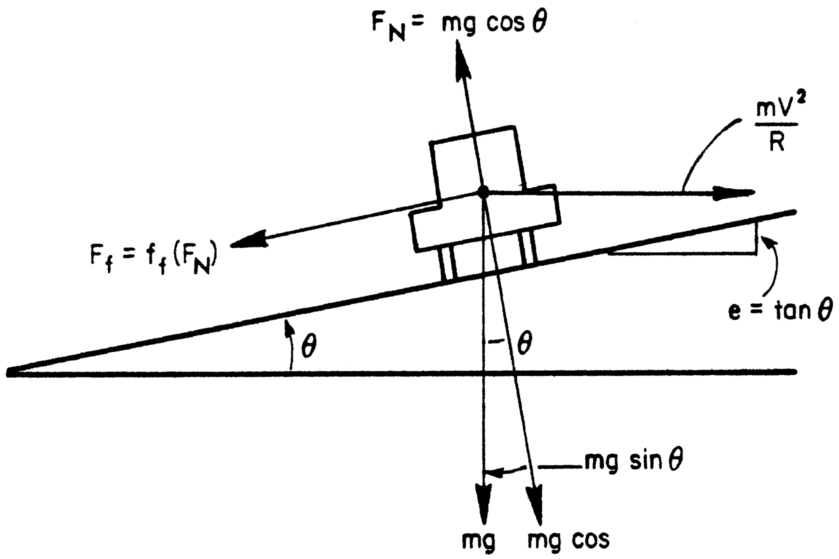


Figure 3. Terminology used in describing spiral curve transitions.



$$\frac{mV^2}{R} = mg\sin\theta + f_f(F_N)$$

$$\frac{mV^2}{R} = mg\sin\theta + f_f mg\cos\theta$$

dividing through by $g(\cos\theta)$ yields

$$\frac{V^2}{Rg(\cos\theta)} = \tan\theta + f_f : \theta\text{, small angle assumption}$$

$$\frac{V^2}{Rg} = e + f\text{: converting for mixed unit use yields}$$

$$\frac{V^2}{15R} = e + f\text{: where}$$

e, superelevation feet/foot
f, friction factor
V, velocity mi/hour
R, curve radius in feet

Figure 4. Derivation of the miniumum radius equation.

skidding" in which the lateral forces developed by car tires might reach a friction saturation condition. The contrast in the safety margin pertaining to passenger cars and heavy trucks, respectively, is a subject which is discussed in section 5.0 by way of commentary on the findings of this study.

As seen in table 7, from the current edition of the AASHTO Policy on Geometric Design, side friction factor is permitted to go as high as 0.17 at a speed of 20 mi/h (32 km/h). Since the sum of the superelevation value, e, plus the side friction factor, f, can be as high as 0.27 (maximum superelevation = 0.10), it is clear that the potential exists for instantaneous values of (f) to reach 0.20 and above when superelevation is being run off within the constant-radius curve.

Shoulder/Crown and Roadside Development. The shoulder/crown and roadside development is of particular importance on superelevated roadways. An excessive change in slope between the pavement edge and the shoulder can be hazardous if the driver takes the curve wide. Also, excessive drainage from the shoulder onto the roadway can create slippery driving conditions. Insufficient shoulder width can also be hazardous for disabled vehicles that cannot get clear of the through lane. Such elements were examined when reviewing the geometrics of each of the selected sites in this study. For ramps adjoining divided highways, the shoulder widths range from 4 ft to 10 ft (1.2 m to 3 m) for the left shoulder and 6 ft to 12 ft (1.8 m to 3.6 m) for the right shoulder. Indeed, full shoulders on both sides of one-way ramps are rarely provided; with one-way traffic, it is common to employ a wider shoulder on the right side as an adequate provision for disabled vehicles.

With paved shoulders on superelevated sections, it is recommended practice that the inner 2 ft (.6 m) of the shoulder width is to be maintained at the superelevation rate matching that of the travel lane. This provision avoids a gradebreak (or change in slope) of the pavement edge. For shoulder sections not maintaining the superelevation rate, a maximum recommended value of the gradebreak between the pavement and shoulder edges is .07 (.08 in the Green Book). For such paved shoulders, drainage considerations support the recommendation that they slope away from the roadway at a rate of 3/8 to 5/8 in/ft (.009 to .016 m/m) without curbs, and 1/4 in/ft (.006 m/m) with curbs.

Table 7. Maximum degree of curve and minimum radius determined for limiting values of e and f, rural highways and high-speed urban streets.

Design Speed (mph)	Maximum e	Maximum f	Total (e + f)	Maximum Degree of Curve	Rounded Maximum Degree of Curve	Minimum Radius (ft)
20	.04	.17	.21	44.97	45:0	127
30	.04	.16	.20	19.04	19.0	302
40	.04	.15	.19	10.17	10.0	573
50	.04	.14	.18	6.17	6.0	955
60	.04	.12	.16	3.81	3.75	1,528
20	.06	.17	.23	49.25	49.25	116
30	.06	.16	.22	20.94	21.0	273
40	.06	.15	.21	11.24	11.25	509
50	.06	.14	.20	6.85	6.75	849
60	.06	.12	.18	4.28	4.25	1,348
65	.06	.11	.17	3.45	3.5	1,637
70	.06	.10	.16	2.80	2.75	2,083
20	.08	.17	.25	53.54	53.5	107
30	.08	.16	.24	22.84	22.75	252
40	.08	.15	.23	12.31	12.25	468
50	.08	.14	.22	7.64	7.5	764
60	.08	.12	.20	4.76	4.75	1,206
65	.08	.11	.19	3.85	3.75	1,528
70	.08	.10	.18	3.15	3.0	1,910
20	.10	.17	.27	57.82	58.0	99
30	.10	.16	.26	24.75	24.75	231
40	.10	.15	.25	13.38	13.25	432
50	.10	.14	.24	8.22	8.25	694
60	.10	.12	.22	5.23	5.25	1,091
65	.10	.11	.21	4.26	4.25	1,348
70	.10	.10	.20	3.60	3.5	1,637

NOTE: In recognition of safety considerations, use of e_{max} = 0.04 should be limited to urban conditions.

The use of curbs on ramps was permitted in the prior AASHTO design policy (the Blue Book), but is no longer recommended as a current policy. In prior practice, curbs on the outer side of a curved ramp were seen as an effective delineator, but now are discouraged unless needed in particularly difficult drainage situations. Currently, when placing a curb on a high-speed ramp, it is recommended that only a mountable curb be used and that it be placed on the outside edge of the shoulder. The curb placement issue is given particular attention in the findings of this study because of a peculiar tendency of truck trailers to drift toward the outside during high-speed cornering.

Sideslopes on highways carrying high volumes of traffic and high speeds are desirably flat so that vehicles running off the roadway have an improved prospect for recovery. Rather substantial sideslope inclinations are designed into highways for which excessive cost burdens are posed by the cutting and filling of terrain in order to provide a more moderate sideslope. According to the height of the cut or fill, then, the AASHTO policy provides a design guide for sideslopes ranging from 6:1 to 1.75:1 slope as the cut depth goes from the range of 0 ft to 4 ft (0 m to 1.2 m) to greater than 20 ft (6.1 m). A sideslope of 6:1 is seen as offering a "good chance" for recovery by an automobile which runs off of the road. Findings of this study, however, will reflect upon the potential for loss of control of heavy trucks which travel on the sideslope area.

In addition to the inclination of the slope, itself, there is also design guidance on the matter of "rounding" the "hinge point" between the shoulder edge and the soil forming the sideslope. A continuous change in slope is desirable so that vehicles do not become airborne in crossing beyond the edge of the shoulder in an emergency.

Deceleration and Acceleration Lanes. Since vehicles entering and exiting the through lanes of high-speed expressways encounter a substantial change in velocity, provisions are made for additional travel lanes upon which the speed change is to occur. Specific design recommendations exist for the length of both deceleration and acceleration lanes, depending upon the combination of initial and final speeds which are expected. These lengths are based upon comfortable levels of braking and acceleration which are thought to apply to passenger-car operation. The recommendation for length of acceleration lanes reflects an assumed acceleration rate which varies nonlinearly from approximately 3 mi/h per second (.14 g's) at zero speed to 1 mi/h per second (.05 g's) at 50 mi/h (80 km/h). These length guidelines yield acceleration lanes which are intended to permit merging within 5 mi/h (8 km/h) of the average running speed on the through lanes. As will be seen, however, loaded truck combinations require considerably longer distances to accelerate such that truck drivers appear to adopt compensating practices which are less than desirable.

The recommended lengths of deceleration lanes are based upon an assumed deceleration rate of 6.2 mi/h per second (.28 g's) from an initial speed of 70 mi/ (113 km/h) to 4 mi/h (per second (.18 g's) from a speed of 30 mi/h (48 km/h). These values, found to be comfortable as braking rates in passenger cars, are assumed to apply following a 3-second interval during which the vehicle is coasting in gear. The coasting deceleration is on the order of 1.5 mi/h per second (0.08 g's). Deceleration lanes are to be constructed such that the target length computed per the AASHTO guideline begins at the point at which the right-hand edge of the tapered lane is 12 ft (3.7 m) from the right-hand edge of the through lane. The intention of this requirement is to assure that the decelerating vehicles have largely exited the through lanes at the time deceleration is initiated. As will be discussed later, the deceleration capability of heavy-duty trucks is sufficiently limited that the achievement of the assumed level of 0.28 g's is often impossible, such that the driver must begin braking before the beginning of the deceleration lane.

4.2 Summary of Ramp Geometric Features

The 15 selected ramps were each examined and analyzed relative to their conformance with the design policies of AASHTO. The detailed results of these analysis are presented in appendix A. The appended section considers each ramp, in turn, and makes observations regarding the shortcomings of each design relative to accepted design standards. The effort expended on geometric analyses does not, however, attempt to second-guess standard design practices. In section 5.3, however, results of dynamic simulations of trucks on each ramp site are discussed in terms of the apparent shortcomings of the AASHTO design policy, itself, as a means of providing suitable geometric designs for highways used heavily by trucks.

In this section, a summary of the observations arising from the geometric design analysis are presented. To the extent that the selected ramp sites are representative of ramp designs which result in an extraordinarily high incidence of truck accidents, the summary presented here serves to overview "problem designs," relative to truck safety hazards.

The complete set of sites can be grouped into six sets, each indicating a particular characteristic which appears to have been the primary feature leading to truck accidents. For each feature, the group of specific sites exhibiting the problem are identified. While, of course, each site is truly unique when all features of the design are considered, it does appear useful to propose such grouping insofar as it serves to highlight likely problem areas.

4.2.1 Poor Transition of Superelevation. Sites no. 1 and 10 were seen to be particularly poor in the transition of superelevation from either a tangent or a preceding curve section, into the critical curve on the ramp. The AASHTO Blue and Green Books provide a great deal of flexibility in the design of transitions. Nevertheless, it is recommended practice that 1/2 to 2/3 of the superelevation be developed prior to the point at which curvature begins. Further, if a spiral transition is to be provided, it is generally expected that the full superelevation level would be implemented over the length of the spiral--such that the fixed-radius curve begins with superelevation fully developed.

The site which conveniently illustrates the transition problem is site no. 1, as diagrammed in figure 5. The design incorporates a spiral transition to curve (3)--the curve in which truck rollover and jackknife accidents have been reported. The posted speed for this ramp is 35 mi/h (56 km/h), yielding a nominal side friction factor of 0.16. This nominal value is virtually in compliance with the AASHTO-recommended value of 0.155 for this speed level.

Figure 6 is a plot which shows that the "effective side friction factor," as it is developed continuously through curve (3) of ramp site no. 1, rises substantially above the 0.16 nominal value over most of the length of the ramp. That is, since the full superelevation level of 0.08 is developed over a distance

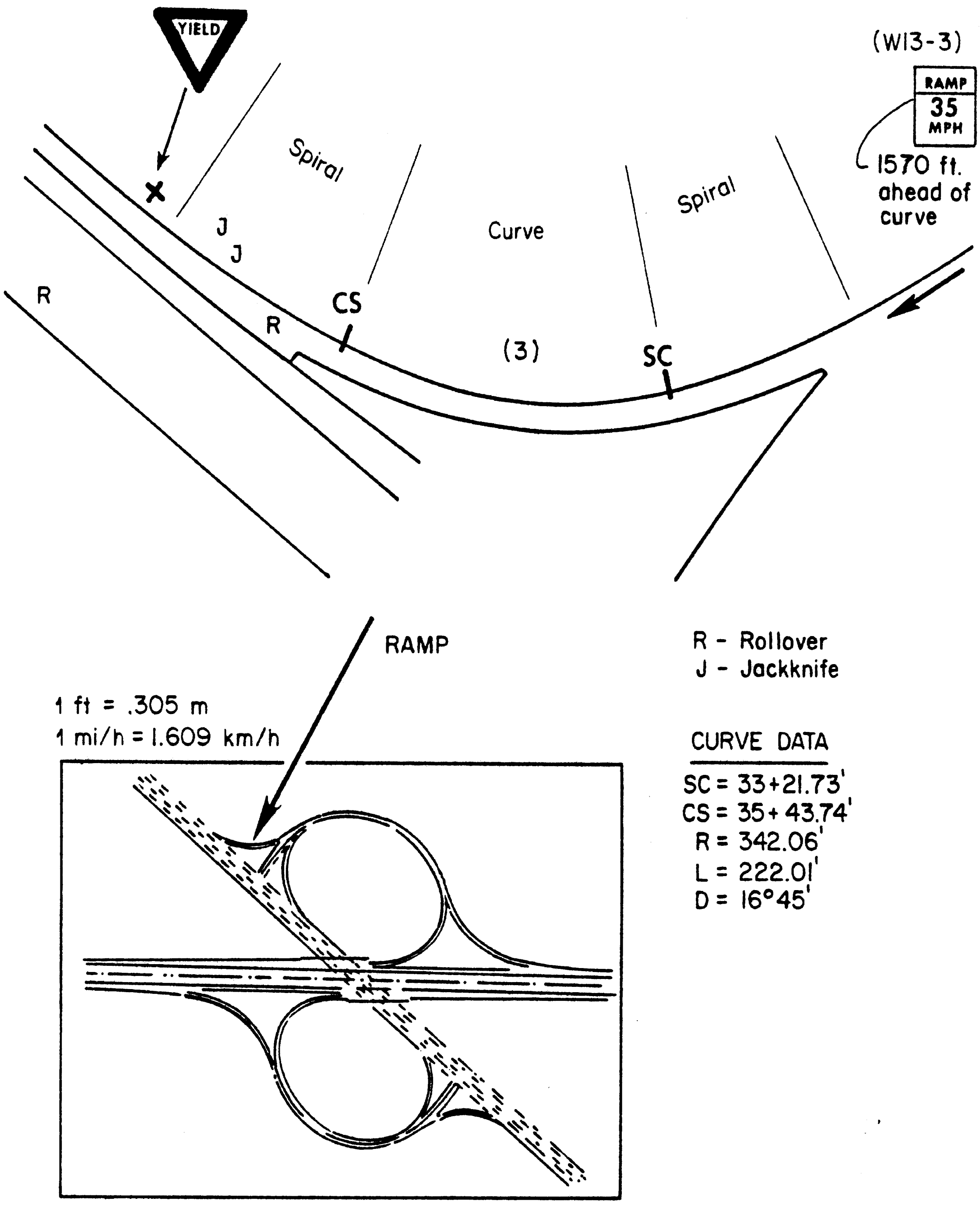


Figure 5. Layout of site no. 1.

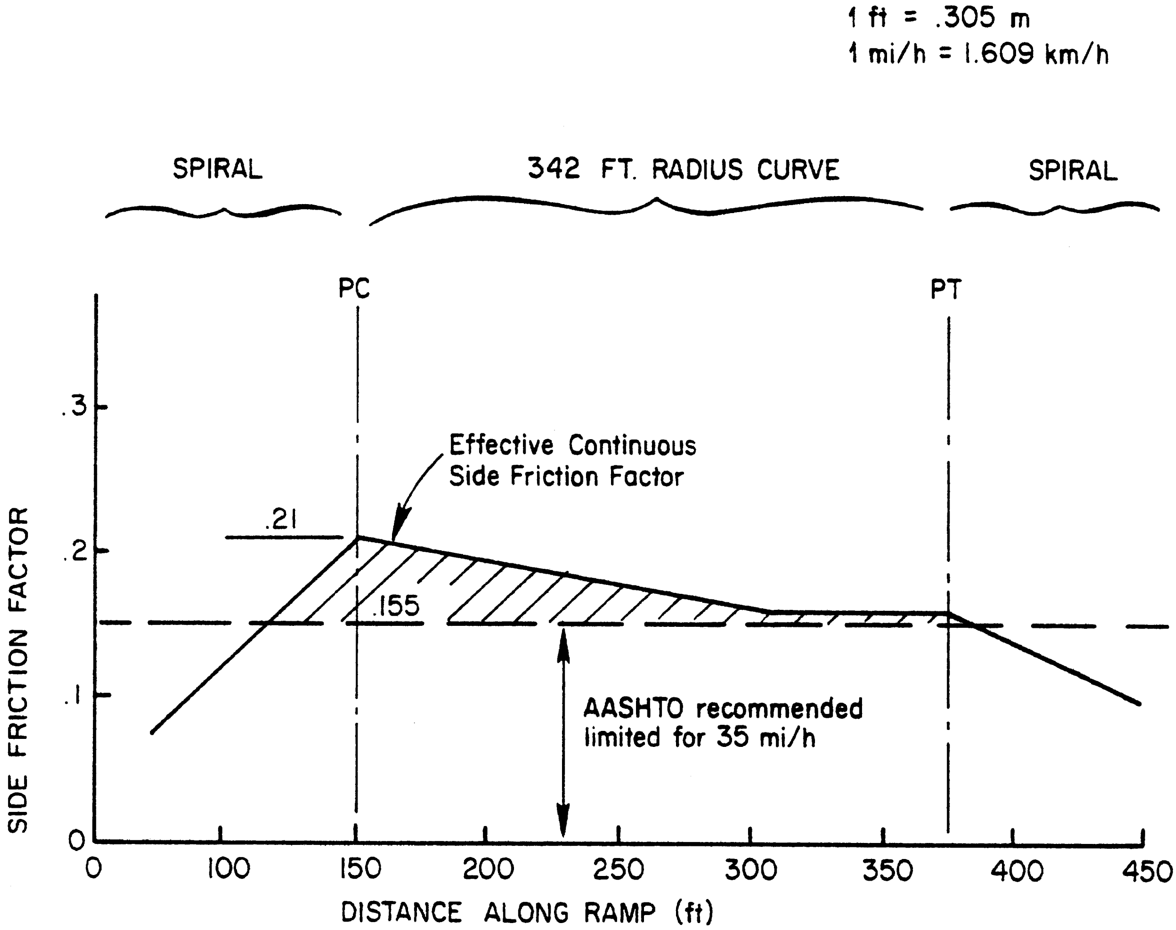


Figure 6. Continuous side friction factor developed at site no. 1.

which begins only 77 ft (23 m) ahead of the point of curvature and continues for another 179 ft (55 m) along the 222-ft (68-m) curve, the side friction factor peaks at a value of 0.21. Accordingly, vehicles traveling over this ramp experience centripetal acceleration levels as large as 0.21 g's as a consequence of the anamolous development of superelevation. Exposure of heavy-duty trucks to such conditions is shown in section 5.3 to pose an increased threat of vehicle rollover when a payload having a high center of gravity location is being hauled.

4.2.2 Abrupt Changes in Compound Curves. Three ramp sites (nos. 2, 11, and 15) appeared in the truck accident sample as having quite abrupt changes in the radii of successive portions of a compound curve. Two of these sites involved a classic arrangement in which a relatively flat-radius curve appears between two curves which are considerably tighter in radius. The AASHTO Green Book explicitly indicates that such a design is "not good practice."

An illustrative case of this problem appears in site no. 2 which is diagrammed in figure 7. This ramp, posted with an advisory speed of 25 mi/h (40 km/h), constitutes an inner loop of a two-quadrant, partial cloverleaf interchange. We see that curves (1) and (4) are configured with approximately 250 ft (76 m) radii while the centrally located curves, (2) and (3), both have 520 ft (158 m) radii. The ratio of the radius change from curves (3) to (4) is 2.08 which also exceeds the AASHTO guideline of a limiting ratio value of 2.0. Nevertheless, the more critical aspect of the design flaw appears to be the misleading nature of the overall ramp layout, in which the driver appears to have gained the impression that the low advisory speed was warranted simply by curve (1), such that the vehicle may be accelerated somewhat in preparation for merging at the end of the ramp. The increase in speed that has accumulated by the time of arrival in curve (4) seems to have been responsible for the numerous loss-of-control events in that portion of the ramp.

4.2.3 Short Deceleration Lane Preceding a Tight-Radius Exit. Four of the 15 sites (nos. 3, 4, 9, and 10) were noted to involve a tight-radius curve which was situated right at the end of the deceleration lane. Three of the sites were posted for a speed of 25 mi/h (40 km/h) and one was posted at 20 mi/h (32 km/h). In one of these cases, the length of the deceleration lane was decidedly shorter than AASHTO recommendations. As will be shown in section 5.3, however, even the recommended lengths of deceleration lanes tend to place relatively high burdens upon the braking

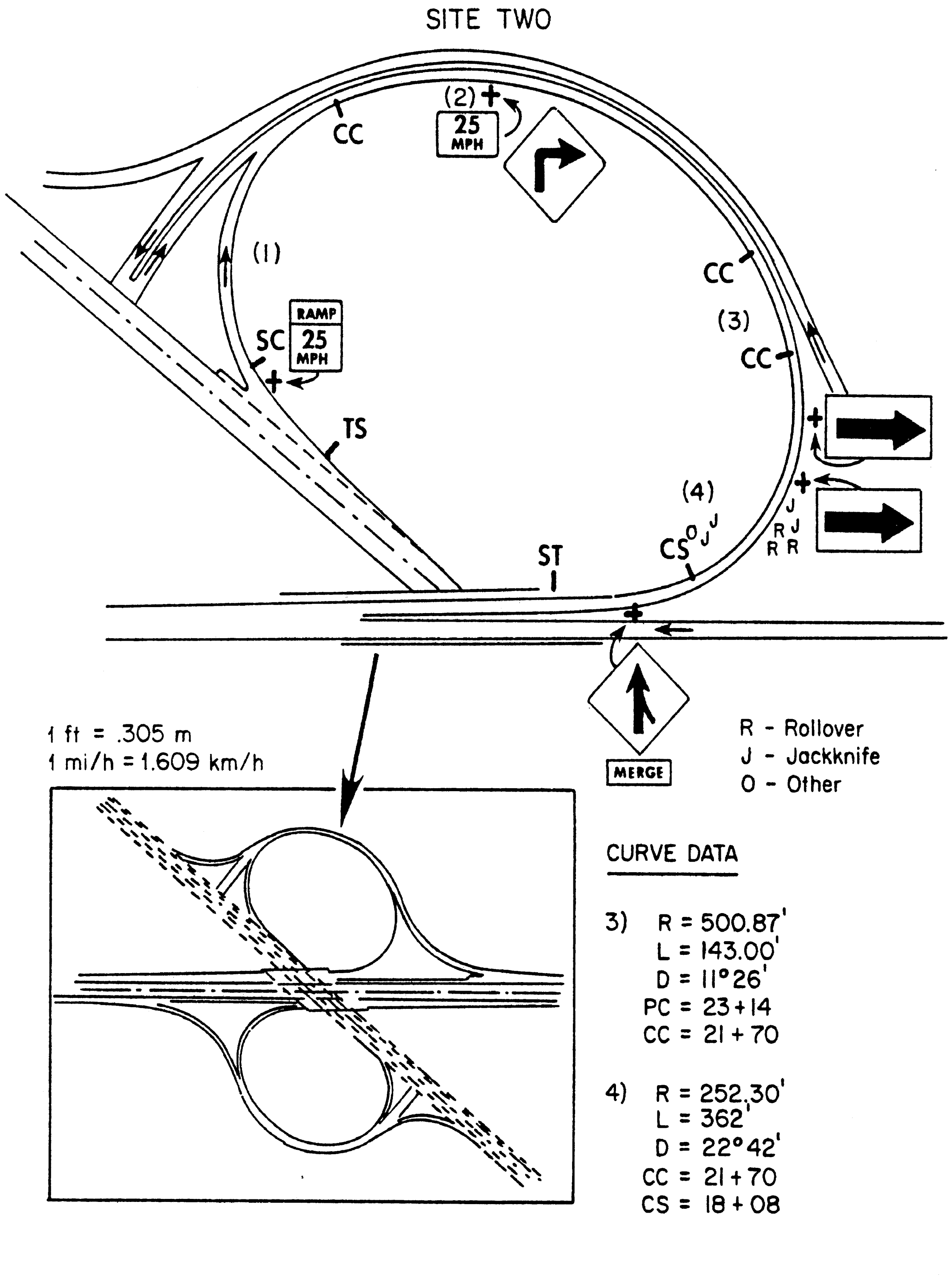


Figure 7. Layout of site no. 2.

capacity of heavy-duty truck combinations. The prospect of trucks facing a high deceleration challenge, together with a very short-radius curve condition, was seen to lead to frequent incidents of both jackknife (due to overbraking) and rollover (due to excessive speed in the curve).

Shown in figure 8 is a diagram of the layout of site 3, in which a tapered deceleration lane, 375 ft (114 m) in length, leads directly to a 250-ft (76-m) radius curve. The ramp is posted at an advisory speed of 25 mi/h (40 km/h). The AASHTO approach toward measurement of the length of deceleration lanes places the effective deceleration length of this lane at approximately 100 ft (30 m) (since the taper is not "counted" until it has progressed to a point 12 ft (4 m) to the right of the right edge of the through lane). Ramps of this type show truck accidents occurring under both wet and dry conditions as either excessive braking or excessive speed takes its toll.

4.2.4 Curbs Placed on the Outside of a Ramp Curve. The AASHTO Blue Book discusses the placement of a curb on the outer side of curvature on loops and direct connection segments as an effective way to delineate the high side of the pavement. In the Green Book, however, the use of curbs on intermediate- and high-speed ramp facilities is not recommended. Three sites (nos. 5, 13, and 14) appeared in the study with curbs placed along the outer side of curvature. In each case, truck rollover accidents were reported in which the curb could be implicated as a tripping mechanism contributing to the rollover outcome. While it may be that some of these incidents would have resulted in a rollover even without curbs present, it is clear that certain peculiar features of the trailer motions which occur in high-speed curves render an outer curb a special hazard. The mechanics of such motions are discussed in section 5.3.

A case in point is provided by site no. 13, shown in figure 9, in which the original design of this urban connecting ramp included a nominally "mountable" curb placed at 2 ft (.6 m) to the right of the right-hand lane edge. The low advisory speed of 30 mi/h (48 km/h), moderately tight radius (374 ft (114 m)), and poor placement of warning signs seemed to contribute to a frequent incidence of truck rollovers at this site. More recently, a wedged overlay of pavement was installed, eliminating the outer curb and providing one continuous surface, with 3 percent additional superelevation, from the left to right shoulder extremities. This modification was seen to strongly reduce the incidence of truck rollovers, although

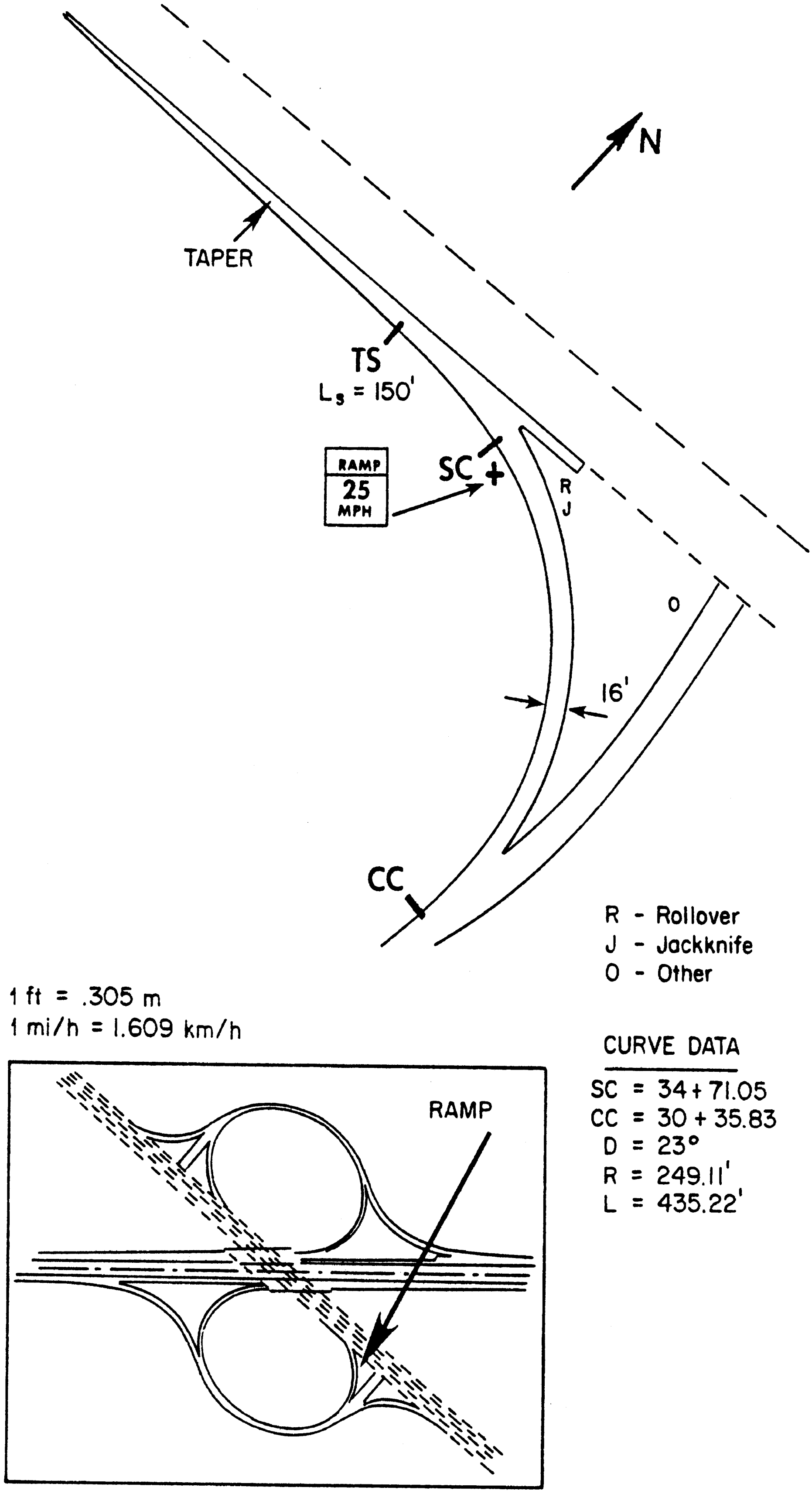
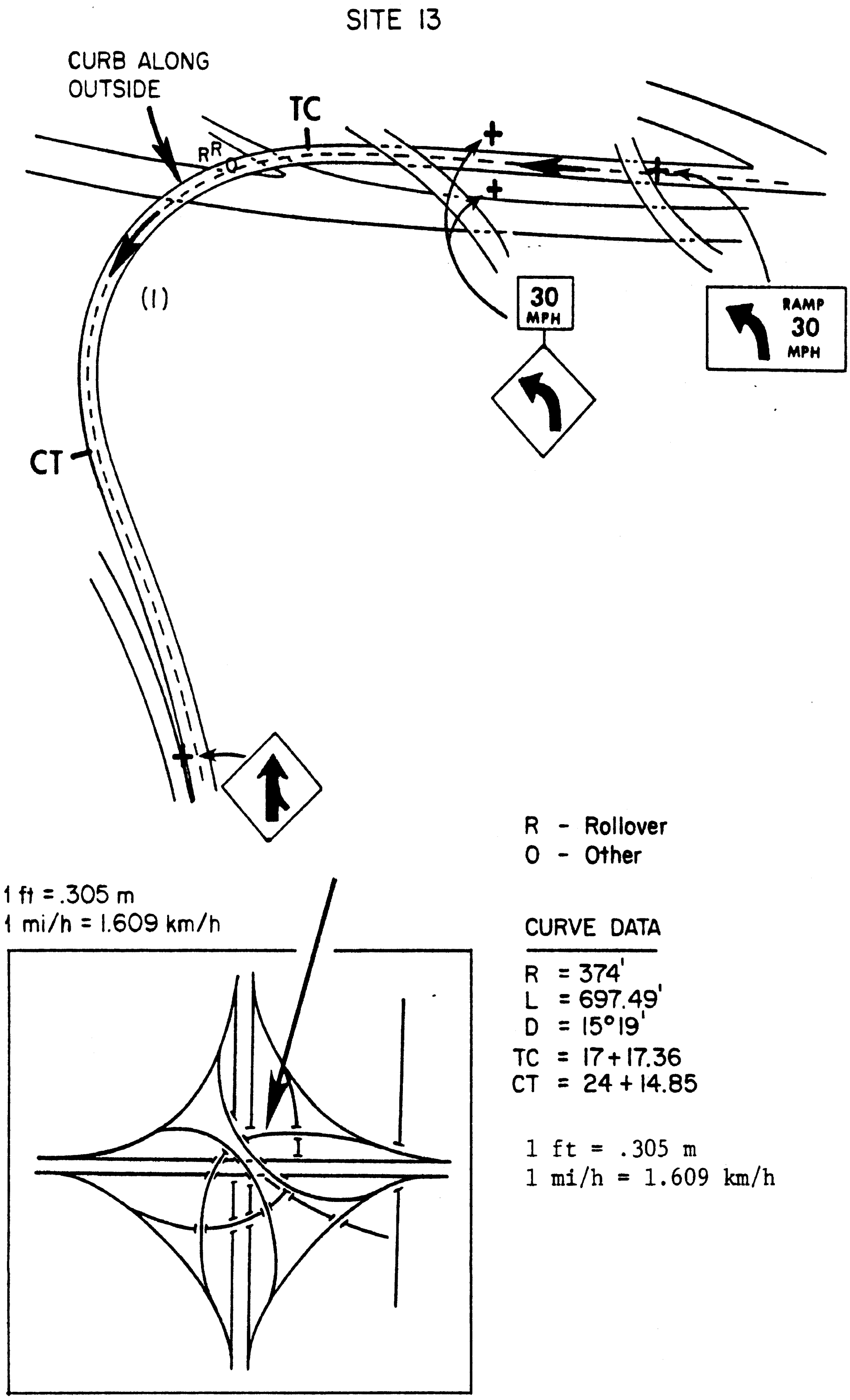


Figure 8. Layout of site no. 3.



1 ft = .305 m
1 mi/h = 1.609 km/h

Figure 9. Layout of site no. 13.

an occasional rollover accident was still being reported. The "wedging"-type countermeasure for elimination of outer curbs is discussed in more detail in section 6.1.

4.2.5 Substantial Downgrade Leading to a Tight Ramp Curve. Two ramp sites (nos. 7 and 8) were identified as suffering, at least in part, by the fact that a substantial downgrade preceded the point of curvature of a segment which placed the critical demand for the advisory speed. The accident problem in both of these cases involved excessive speed. While many other ramp sites showed problems that point at the speed condition, it was deemed pertinent that the downgrades involved here could well account for the speed differences needed for some trucks to approach the rollover condition. Grades on the order of 5 percent, with 500 ft (152 m) of grade length, can yield a speed increase of 10 mi/h (16 km/h) in trucks that are coasting.

Shown in figure 10 is the layout of site no. 8. This semidirect, urban ramp employs a 5.4 percent downgrade for a distance of 470 ft (143 m) leading up to the point of tangency of the 350-ft (107-m) curve. The rollover accidents are all clustered near the very end of this curve. Clearly, if the vehicles were simply traveling at excessive speed upon entering the curve, the rollovers would all have occurred much earlier in the ramp. The placement of all of the rollovers near the end of the curve suggests that a substantial speed increase was being achieved along the downgrade.

Both the AASHTO Blue and Green Books suggest that ramp downgrades as high as 8 percent do not cause hazard due to excessive acceleration. On the other hand, the Blue Book advises that sites having a higher proportion of truck and bus traffic should have grades limited to 3 or 4 percent. Although the culpability of the grade condition has only been inferred from the data describing the two sites studied here, it is clear that the acceleration potential which does exist for heavy trucks is sufficient to explain the rollover outcomes which were observed.

4.2.6 Reduced Friction Level on a High-Speed Ramp. Two sites (nos. 6 and 15) were identified as having experienced a frequent incidence of truck loss-of-control events in wet weather. At site no. 6, the incidence of jackknife with tractor-semitrailers was phenomenal for a 2-year period, following which the ramp was resurfaced such that the jackknife problem simply disappeared. The ramp geometric feature at issue seems to involve a sufficiently large radius that the vehicle can

SITE EIGHT

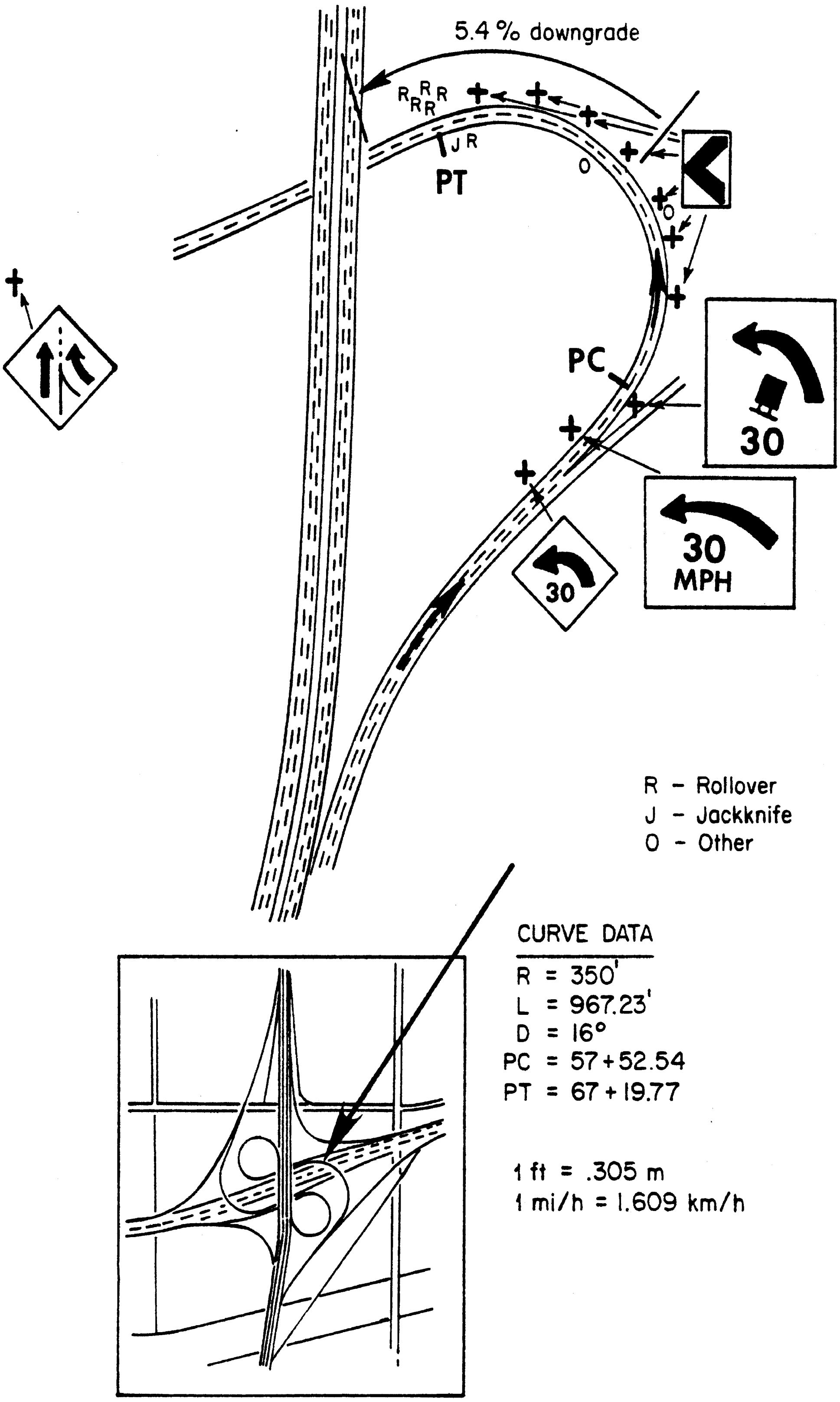
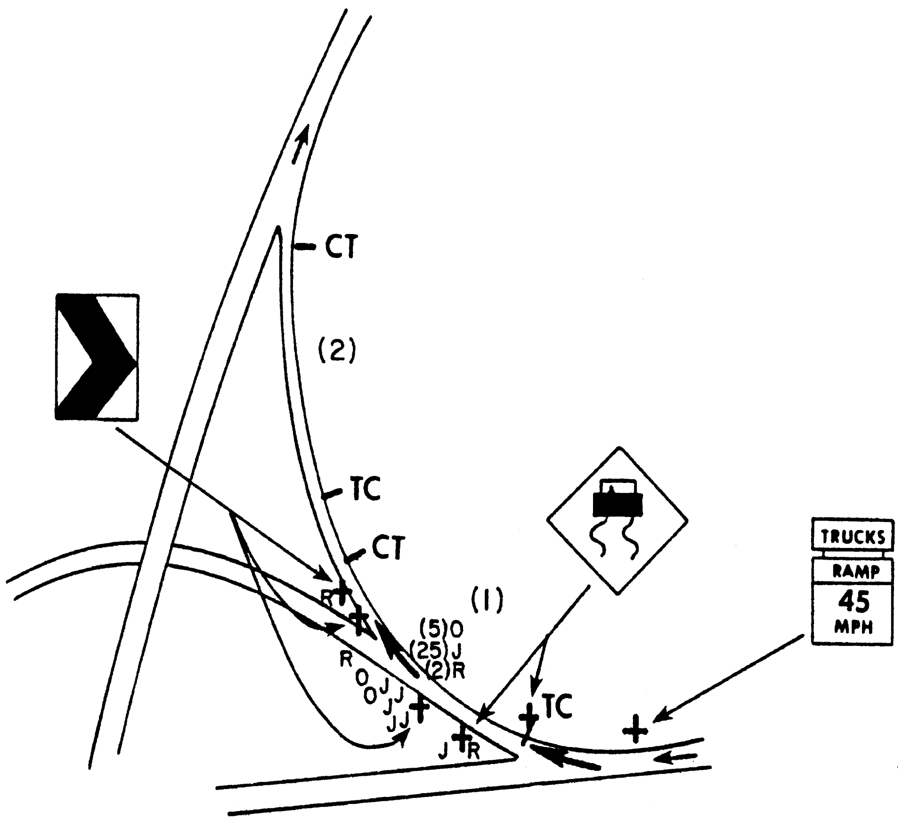


Figure 10. Layout of site no. 8.

travel at near the 55 mi/h (88 km/h) limit. With a poor pavement texture condition, then, the potential for a hydroplaning-like loss in tire/pavement friction results in loss of control.

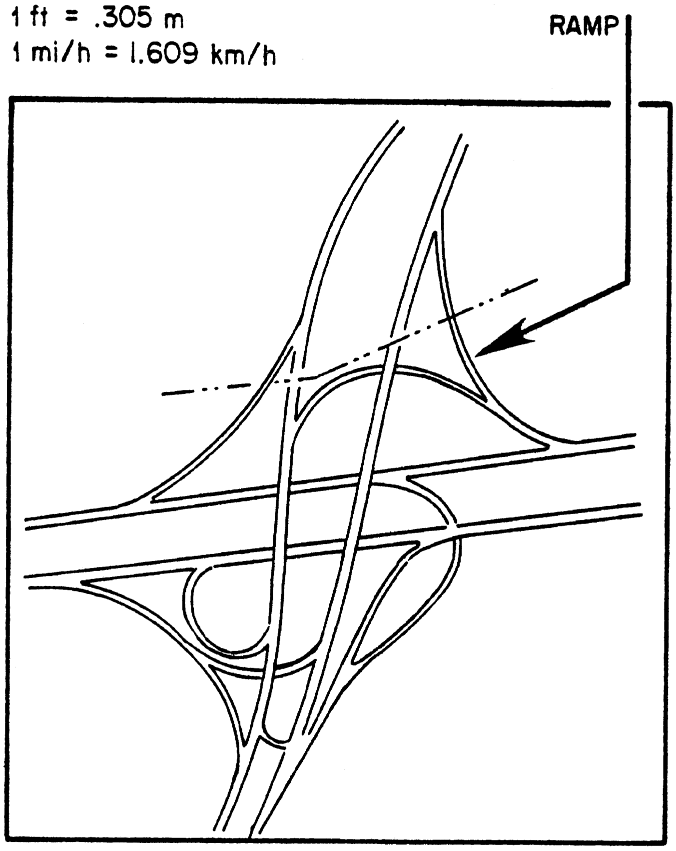
Shown in figure 11 is a diagram of the ramp at site no. 6. The curve radius of 1,400 ft (427 m) is such that the ramp was originally posted with an advisory speed of 55 mi/h (88 km/h) for all traffic. An inordinate number of truck accidents in wet weather led to a special posting of 45 mi/h (72 km/h) for truck traffic. The side friction factor for the 45 mi/h (72 km/h) speed is only 0.05, suggesting that travel through this ramp should be very straightforward. Nevertheless, until the surface was repaved, the rate of jackknife events in wet weather was so high that a number of accidents were actually observed by police officers who had come to the site to report a prior jackknife which had just occurred. At this site, as well as others in which both jackknife and rollover accidents were reported, it is interesting to note that the vehicles suffering jackknife most often come to rest on the inside of the curve while vehicles rolling over typically land on the outside of the curve. While the disposition of rollovers toward the outside of the curve involves straightforward mechanics, the tendency of jackknifed vehicles to land inside is less simply explained. In section 5.3, observations are made explaining this phenomenon and the basic tendency toward hydroplaning-type control problems is related to recent findings published in the literature.

SITE SIX



R - Rollover
J - Jackknife
O - Other

1 ft = .305 m
1 mi/h = 1.609 km/h



CURVE DATA

1) R = 1400.00'
L = 972.08'
D = 4° 5'
TC = 4 + 09.90'
CT = 13 + 81.98'

2) R = 1400.00'
L = 1645.63'
D = 4° 5'
TC = 16 + 73.09'
CT = 33 + 18.72'

1 ft = .305 m
1 mi/h = 1.609 km/h

Figure 11. Layout of site no. 6.

5.0 EXAMINATION OF VEHICLE DYNAMIC RESPONSE ON THE SELECTED RAMPS

In this section, the response properties of vehicle combinations on the selected ramps will be examined. While the principal effort in this regard has involved the simulation of specific truck configurations on each of the 15 selected ramps, the discussion below will also generalize upon the apparent impact of ramp design on the truck controllability problems which are observed. The representation of both the road sections and the vehicle in the simulation model will be described, with reference to appendixes for more detail. A general discussion of the basic loss-of-control modes with heavy-duty vehicles then precedes the presentation of results from the simulation activity.

5.1 Representation of Ramp Geometry in the Simulation Model

The UMTRI "Phase 4" simulation model was used to represent the dynamic response of the baseline tractor-semitrailer along each of the highway ramps [16]. The Phase 4 model is a nonlinear, time-domain simulation capable of representing commercial vehicles ranging from straight trucks to triples combinations. Each sprung mass (or body unit) has six degrees of freedom; each unsprung mass (or axle assembly) has a vertical bounce and roll degree of freedom. Spin degrees of freedom are also included at each wheel location for calculating tire traction forces during braking or combined braking/cornering maneuvers. For the parameter sets used in this study, the tire and suspension characteristics were described by realistic sets of nonlinear data derived from previous experimental measurements. Consequently, the basic computer representation of the baseline tractor-semitrailer used here can be considered to be fairly comprehensive and complete.

To simulate the negotiation of an actual highway ramp, the Phase 4 model was operated in a closed-loop or path-following mode through the use of a driver steering model. The driver model operates by "looking ahead" and steering the vehicle along the specified curve much like an actual driver does. By specifying the details of the highway geometry and path to be followed, the program user supplies the necessary information to the model to permit it to operate in this manner. The following paragraphs describe how the highway ramp geometry and path curvature information is processed by the vehicle and driver models.

In order to specify a road surface to the Phase 4 program, the elevation of the road surface as a function of the forward and lateral travel distance must be defined. For most highway ramp geometries, such as those examined within this study, such information would be obtained from highway drawings and then translated (or programmed) as a "road subroutine" which the vehicle model would access during a simulation run. Highway elevation, superelevation, and grade information is then obtained from the road subroutine for each wheel location on the vehicle as a function of time. Thus, the road surface and its respective surface gradients act as a set of time varying displacement inputs to each wheel assembly as the vehicle moves along the specified highway ramp geometry. Forces associated with wheel displacements are then transmitted to the vehicle masses through tire and suspension compliances. Longitudinal and lateral components of the gravitation force acting on the vehicle are also introduced by changes in road surface inclination.

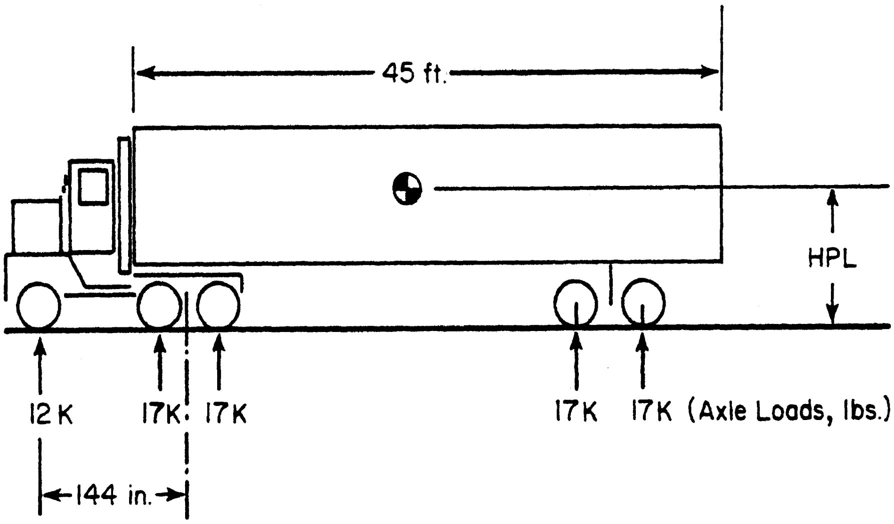
For actual highway ramps comprised of approach tangents, spiral transitions, and circular sections, the description of the highway road subroutine was coded as a sequence of corresponding mathematical formulae or tabular data. The required geometry consisted of a set of x-y path coordinates defining the road centerline, the corresponding elevation, superelevation, and grade at each coordinate pair. From this information and the known position of the vehicle relative to the road centerline, road surface elevation and inclination under each wheel location were calculated at each time step of the simulation.

The driver steering control model operates by selecting a steering-wheel angle to cause the vehicle to track a previewed section of the highway curve. At each instant of time, the driver model predicts future position of the vehicle based upon the current vehicle response and steering-wheel angle. This predicted position is then compared with the directly observed (or previewed) path position corresponding to the same future time. The error between the predicted vehicle position and previewed path is used to steer the vehicle. A time delay is also included to represent basic driver reaction time limitations normally present in the steering control process. The driver model parameters selected to represent driver steering characteristics within this study were similar to values used in previous studies for describing representative driver steering control behavior. The preview (or "look ahead") time used in the driver model was 2.0 seconds, while the time delay value was 0.25 seconds. Further details on the driver steering model and comparisons with experimental measurements for automobile and truck-driver systems can be found in references 17 to 20.

5.2 Characterization of Trucks Used in the Simulation

Tractor-semitrailers having differing design and loading parameters were represented in the simulation for examining the control problems posed by the various ramps. Since the Phase 4 simulation provides for a relatively complex treatment of the vehicle, the parameters describing each truck combination cover a large number of quantities. A complete listing of the quantities employed to describe the simulated vehicles is presented in appendix B. The appendix also lists an example output data set, showing the instantaneous values for a number of the key response variables as the vehicle proceeds along an example ramp.

The basic vehicle type which was described in each simulated case is illustrated in figure 12. The vehicle constitutes a typical cab-over-engine (COE) tractor, having a wheelbase of 144 inches (3.65 m), coupled to a 45-foot (13.7-m) van-type semitrailer. The tractor and semitrailer both incorporate tandem axle sets. The tires in every case represent a popular radial-ply, rib-tread selection, size 11 R 22.5, load range G. Other characteristics of the vehicle, encompassing steering system characteristics, inertial properties, frame stiffness, fifth wheel height and stiffness, and other geometric details were all



Vehicle Configuration	Payload Mass Center Height, HPL, (inches)	Nominal Vertical Spring rates, lb/in	
		Tractor tandem	Trailer tandem
Baseline	83.5	6000	9000
High - CG	105	5000	7300

1 ft = .305 m
1 in = .0254 m
1 lb/in = .179 kg/m
1 lb = .454 kg

Figure 12. Basic description of simulated vehicles.

selected to represent common hardware used in normal long-haul types of vehicles.

All of the ramp sites were examined initially with the vehicle combination set up in a "baseline" configuration. For these cases, the suspensions employed on both the tractor and semitrailer tandem axle locations were represented as the common "four-spring," multileaf type of design. The tractor steering axle was outfitted with a common multileaf suspension, rated at 12,000 lbs (5.5 Mg), for all configurations. The baseline vehicle was also distinguished by placement of the payload mass center at an elevation of 83.5 in (2.12 m). This figure is taken to represent a common loading which derives from a medium-density freight. The vehicle was loaded to the full gross weight allowance of the Federal Aid Highway System, 80,000 lbs (39.6 Mg), with the distribution of axle loads as shown in figure 12.

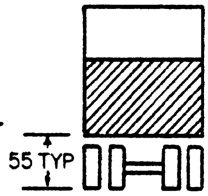
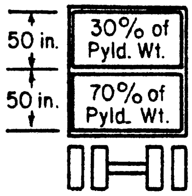
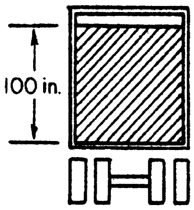
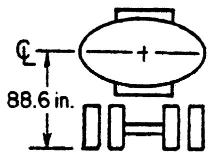
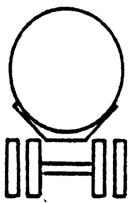
In order to also study the response of vehicles which lie at the extreme end of the spectrum of roll stability characteristics, another configuration called the "high-c.g." case was simulated. This vehicle was identical to the baseline configuration except that:

- the tandem suspensions at the tractor and semitrailer were made "softer" so as to represent actual suspensions which are known to exhibit lower levels of resistance to rolling motions. The tractor tandem suspension was configured to represent a popular torsion-bar suspension which is known to exhibit a low level of roll stiffness [21]. Likewise, the trailer tandem was represented as a low-stiffness four-spring variety such as had been measured previously [21].

- the payload mass center was placed at a height of 105 in (2.7 m) above the ground. This payload placement represents a case in commercial practice in which a homogenous freight is loaded to both fill the cubic capacity of the trailer while also reaching the full gross weight allowance. With such a loading, the payload mass center height is essentially placed midway between the floor and the roof of a typical 13.5-ft- (4.1-m) high trailer (allowing also for an approximate 7-inch (.18-m) clearance between the freight and the top of the trailer). This loading represents the highest mass center location which occurs in common trucking practice.

In addition to the "baseline" and "high-c.g." cases, the baseline vehicle was also represented in an empty condition for examining jackknife responses. One of the jackknife simulations, at site no. 13, was associated with studying a near-hydroplaning condition on wetted pavement. In this case, the tire/pavement friction limit was sequentially reduced in order to incite a jackknife response simply as a result of cornering in a slippery curve. Since the tire/road friction level deriving from hydrodynamic effects is known to be very sensitive to tire load, and since the "unloaded" vehicle condition dramatically unloads the tires on the tractor drive axle relative to those on the steering axle, differing frictional conditions were represented on the steering and drive axles of the tractor. Aside from the frictional limits, however, the other force and moment properties of the tire were represented in the same manner as was used on the dry surface.

The key response characteristic which is probed in the simulation of most of the ramps is the roll stability of the vehicle. As will be discussed, the results show that a small degree of speeding beyond the posted advisory value can yield rollover on some of the sites. To place the roll stability levels of the represented configuration in the context of wider truck practice, figure 13 shows five different tractor-semitrailer combinations, and the corresponding levels of "rollover threshold" which apply to each. The rollover threshold characterization describes the peak value of lateral acceleration, in units of g's, beyond which the vehicle would roll over in a steady turn. We see that cases A and C, which represent the respective "baseline" and "high-c.g." configurations mentioned above, yield rollover threshold values which are not dissimilar from those of other common vehicles. Of course, there are many tractor-trailer loading patterns which would yield higher values of rollover threshold than those shown in the figure. Nevertheless, the cases shown are by no means rare and they are introduced for consideration here in recognition of the rollover

CASE		CONFIGURATION	WEIGHT (lbs) GVW	PAYLOAD CG HEIGHT (in)	ROLLOVER THRESHOLD (g's)
A. "BASELINE"		Full Gross, Medium-Density Freight (34 lb/ft^3)	80,000	83.5	.34
B.		"Typical" LTL Freight Load	73,000	95.0	.28
C. "HIGH-CG"		Full Gross, Full Cube, Homogeneous Freight (18.7 lb/ft^3)	80,000	105.0	.24
D.		Full Gross Gasoline Tanker	80,000	88.6	.32
E.		Cryogenic Tanker (He_2 and H_2)	80,000	100.	.26

1 lb. = .454 kg
1 in. = .0254 m
1 lb/ft^3 = 16.01 kg/m^3

Figure 13. Rollover threshold values for various example vehicles.

challenges which the lower stability vehicle faces when operating on ramps such as many of those selected for this study.

5.3 Vehicle Response on Interchange Ramps

In this section, the results of the simulation exercise will be discussed. Simulations were performed covering the matrix of sites, speeds, and vehicle configurations listed in table 8. The basic scheme of selected conditions involved a run at the posted speed plus additional runs in the baseline condition at higher speeds, where the response was seen as instructive. For selected cases in which rollover of the vehicle seemed a probable outcome, runs were conducted with the "high-c.g." configuration in order to establish whether rollover would occur at or near the posted advisory speed.

Additional runs were made at site nos. 4 and 6 to illustrate the mechanics of jackknife responses under conditions of reduced tire/pavement friction. On site no. 4, involving a moderate initial curve at which a number of jackknife accidents were reported, the baseline vehicle in an empty configuration was braked to the point of jackknife, with the peak side friction coefficient placed at 0.5. On site no. 6, involving a large-radius ramp which apparently imposed very low friction conditions for trucks running at high speed, reduced friction levels were represented at the unloaded tractor drive wheels and trailer wheels while the vehicle traveled through the ramp at constant speed. Jackknife was induced in these cases by the saturation of lateral forces at the rear of the tractor, without braking. Since the study had no access to data describing tire/pavement friction levels, values for friction limits were simply hypothesized so that the characteristic responses could be demonstrated. (Regarding other cases in which rollover was simulated, of course, the friction level constituted a moot issue since the lateral acceleration levels needed to obtain rollover were well below the values needed to approach friction limits on any dry pavement.)

The simulation results are presented comprehensively in appendix C, including plots of the motion variables of the vehicle, with respect

Table 8. Simulation run conditions.

Site No.	Advisory Speed mi/h	Peculiar Site Characteristic	Simulation Speeds "Baseline"	"High C-G"
1	35	tight curve, end of ramp, poor transition	34,44,40 mi/h	35,40 mi/h
2	25	compound curves, tight-flat-tight	25,35,42	25
3	25	short decel lane, then tight curve	35,25	25,35
*4	20	low advisory spd, spiral to compound curve	55 (braking)	-----
5	20	tight curve, curb on outside	35,25,20	35,30,20
**6	45/55	large radius, highspeed, slippery wet	55,70	-----
7	35	downgrade leads to sharp curve	35,45	35,45
8	30	downgrade leads to sharp curve	30	30,35
9	25	sharp curve at end of moderate decel. lane	25	-----
10	25	poor transition, sharp curve right away	25	-----
11	25	compound curve, tight-flat-tight	25	-----
12	40	tight curve on central lanes of trumpet	40	45,50
13	30	tight curve, curb on outside	35	-----
14	25	tight curve, curb on outside	40	-----
15	30	compound curve, large ratio of radii	35	-----

* Site No. 4 - Braking from 55 mi/h towards 20 mi/h on μ = .50 surface

** Site No. 6 - Additional runs to simulate hydroplaning, Baseline Vehicle.

(Note: Posted advisories: 45 mi/h - trucks, 55 mi/h - others.)

(empty) 55 mi/h
μ tractor front = 0.50
μ tractor rear = 0.15
μ trailer = 0.15

(empty) 55 mi/h
μ tractor front = 0.50
μ tractor rear = 0.12
μ trailer = 0.12

to time. These results constitute a precise definition of the mechanics of rollover, jackknife, yaw response dynamics, and the like. In order to interpret the results in a manner which places a focus upon the interaction between the vehicle and the ramp geometrics, the discussion below will generalize upon the responses which were observed. Firstly, in section 5.3.1, the discussion presents certain general concepts concerning the types of loss-of-control problems which are manifest at interchange ramps by heavy-duty vehicles. In section 5.3.2, selected simulation runs are employed to illustrate the range of specific control problems which are seen to occur on the various types of ramps in the sample.

5.3.1 Mechanisms Leading to Loss of Control with Trucks. Heavy-duty vehicle combinations exhibit control limits in maneuvering conditions which can typically be handled with relative ease in passenger cars. Such limits are encountered under both braking and cornering conditions. The following discussion identifies the mechanisms which determine the types of truck control limits which appear to have been exceeded in the ramp accidents studied in this project. This generalized presentation will provide the basis for specific illustrations of loss-of-control in simulation results in the next section.

Rollover in a Steady Turn

The design of highway curves reflects an accounting of the lateral, or centripetal acceleration which is associated with travel around a fixed-radius arc. For example, the level of lateral acceleration experienced in a steady turn was expressed in section 4.1 by means of the AASHTO guidelines on the use of the "minimum radius equation." Since the net lateral acceleration, after accounting for superelevation as well as speed and path radius, acts on the vehicle at the height, H, of its center of gravity, c.g., it is pertinent to consider the implication of the steady-turn condition on vehicles having relatively high placement of the c.g..

Because the heavy vehicle carries freight in commercial quantities, its center of gravity is high, in absolute terms, and is certainly high relative to the width of the tire track. The height of the c.g. is directly involved as a determinant of roll stability since the size of the moment which tends to roll the vehicle over is proportional to the c.g. height. Correspondingly, the strength of the maximum moment which can be exerted in keeping the vehicle upright is proportional to the track width.

As illustrated in figure 14, the roll stability of heavy tractor-semitrailers is inherently distinguished from that of, say, passenger cars insofar as the ratios of the half-track widths, $T/2$, to the respective heights, H, of the center of gravity of cars and trucks are so different. Recognizing that the ratio, $T/2H$, describes a basic scaling of the roll stability level of any vehicle, it is instructive to note that loaded, heavy-duty, trucks exhibit a value of this ratio near 0.5 while cars yield a value near 1.3. The $T/2H$ term provides a crude first estimate of the level of lateral acceleration, in g's, which a vehicle will tolerate before rolling over in a steady turn.

When the stiffness of tires and suspension springs are taken into account, we find that the $T/2H$ estimate for common heavy trucks degrades to an actual rollover limit in the vicinity of 0.3 g's while cars will actually roll over near 1.2 g's. Further, it was shown in section 5.2 that a number of specific truck loadings cause such high c.g. placement that rollover will occur as low as 0.24 g's. The basic observation, then, is that loaded commercial vehicles exhibit very low "rollover threshold" limits relative to passenger cars and that the specific limit value for a given vehicle will depend very strongly upon the distribution of payload which is being carried. (More comprehensive treatments of the mechanics of rollover are presented in references 22 and 23.)

Since the rollover threshold of heavy trucks is so low, one can appreciate that rollover is a common outcome in response to severe steering maneuvers, or as a result of overspeeding on a highway curve. In particular, the limit condition which constrains truck movement on

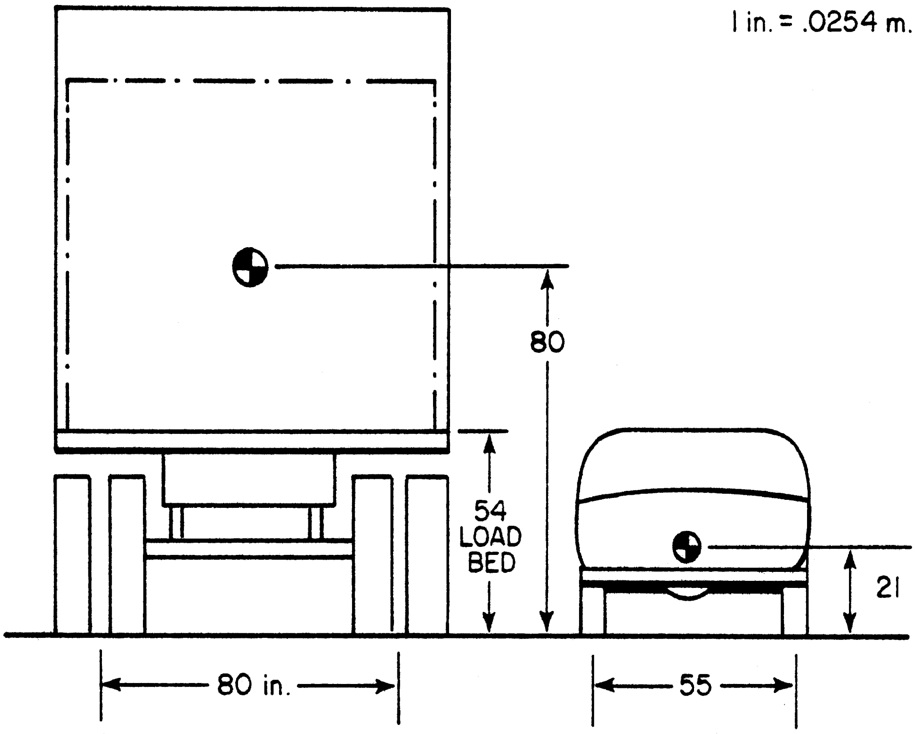


Figure 14. Center of gravity height and effective track width for loaded van-type semitrailer combination and passenger car.

highway curves is likely to be the rollover limit rather than the "skidding" condition which is the primary safety concern upon which AASHTO design guidelines are rationalized. If highways were to be designed with trucks specifically in mind, the constraints on curve geometrics would have to be expressed in terms which afforded a suitable margin of safety for the avoidance of truck rollover.

High-speed Offtracking

While the mechanics of rollover in steady turns is explainable through static phenomena, an additional mechanism involving a dynamic process can also come into play. This mechanism begins with a property which is exhibited to some extent by all vehicles, but especially by articulated vehicles having one or more trailers. The mechanism is termed "high-speed offtracking." With articulated truck combinations, the phenomenon involves the outboard drift of the rear of the trailer under the influence of lateral acceleration. That is, while trailing elements tend to track inboard of the tractor in tight turns at low speed, travel through curves at high speed tends to cause the trailer tires to track outboard of the tractor. The outboard orientation of the trailer is established by the need for the trailer tires to operate at a slip angle in order to develop the lateral forces needed for equilibrium in the turn. The extent of the outboard offset in wheel paths is a function of the trailer wheelbase(s), the tire cornering stiffness levels, and the lateral acceleration level which is achieved (see 20 and 24).

As shown in figure 15, one primary risk associated with the outboard tracking of trailer wheels arises when a curb is placed on the outside of the curve. Since the sideslipping tire is pointed inwards relative to the curve, (and, thus, is pointed away from an outside curb) the tire does not tend to mount even a "mountable" type curb such that large levels of side force are likely to accompany curb contact. The abrupt input of large side forces at the tire, in turn, produce large transient moments tending to overturn the vehicle. Thus, one can hypothesize that rollover deriving from high speed offtracking, followed

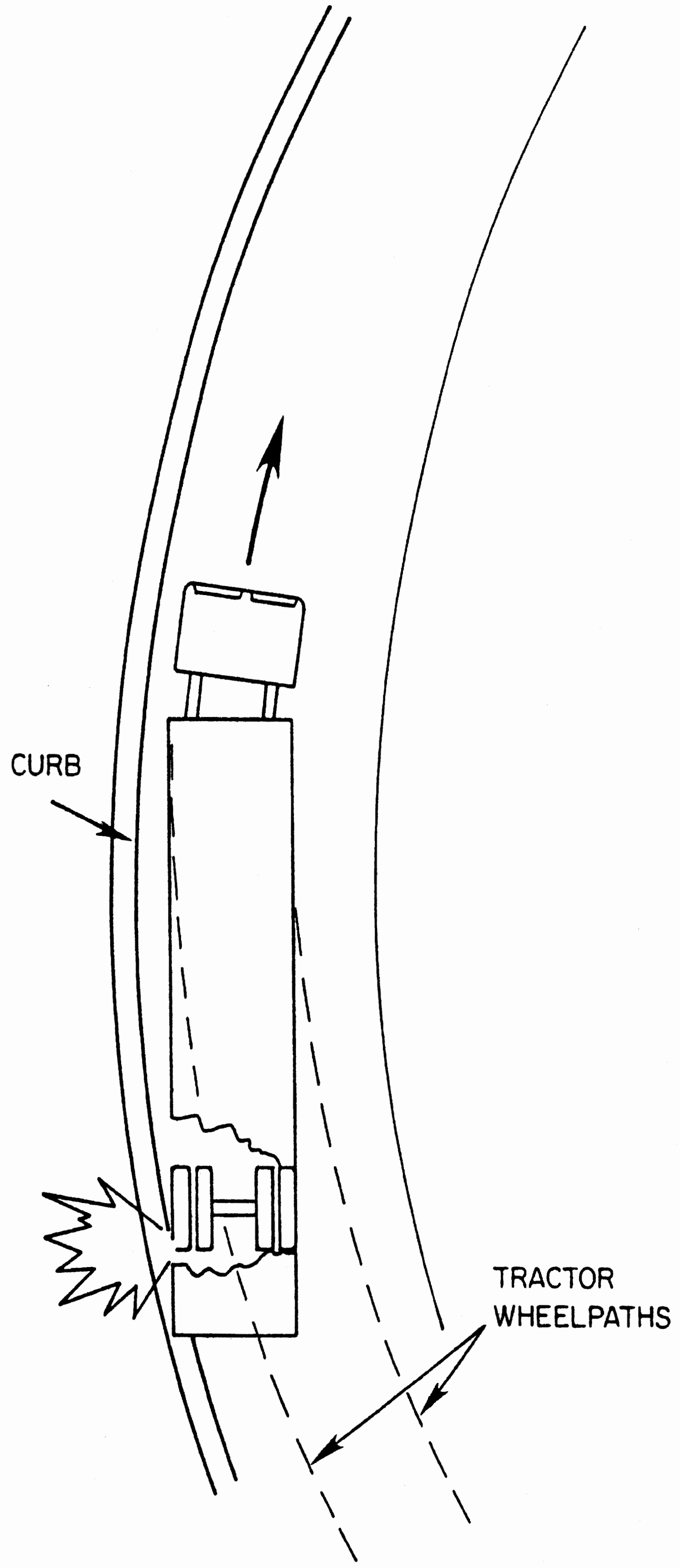


Figure 15. Trailer tires tracking outboard of the tractor risk striking an outside curb on a curve.

by curb contact, might occur at an operating speed which is well below the value needed to produce a simple static rollover in the same curve.

Limitations in Braking Control

Because trucks are designed to maximize cargo capacity, there is generally a large difference in the axle loading which is encountered in going from an empty to fully loaded state. Figure 16 illustrates the contrast between, for example, a typical midsized American car which experiences a 30 percent increase in total load, from empty to loaded, and a typical two-axle unit truck which experiences a 250 percent increase in load, from empty to loaded. Further, we note that the distribution of the relative increases in loading between the front and rear axles of the car is far more balanced than for the case of the truck. Because of the need to place the cab and engine of the truck in the vicinity of the front axle, the great bulk of the payload weight is borne on the truck's rear axle. Thus, we see the truck front axle experiencing only a 30 percent load change compared to a 500 percent change at the rear in covering the range of operating loads. The consequence of this situation for trucks and truck-tractors is that such vehicles are very overbraked in the rear when operating in the unloaded state.

Similarly, when a portion of the freight is removed from, say, the rear of a van trailer at an intermediate destination, the rear trailer axle becomes lightly loaded, and thus overbraked relative to the loaded axles, so that the trailer wheels tend to lock up prematurely. Because of these and other practical issues involving truck brake systems, themselves, the braking performance of trucks is categorically reduced below that of passenger cars.

Aside from the risk of collisions, limitations in stopping capability lead to one of two situations, namely, a) insufficient deceleration such that truck speed is kept up at a hazardous level or b) loss of control due to lockup of one or more axle sets. In the prior case, the risk due to elevated speed involves, say, the prospect of rollover on a curve. The occurrence of wheel lockup, on the other hand, comes about when an excessive level of braking is applied and may

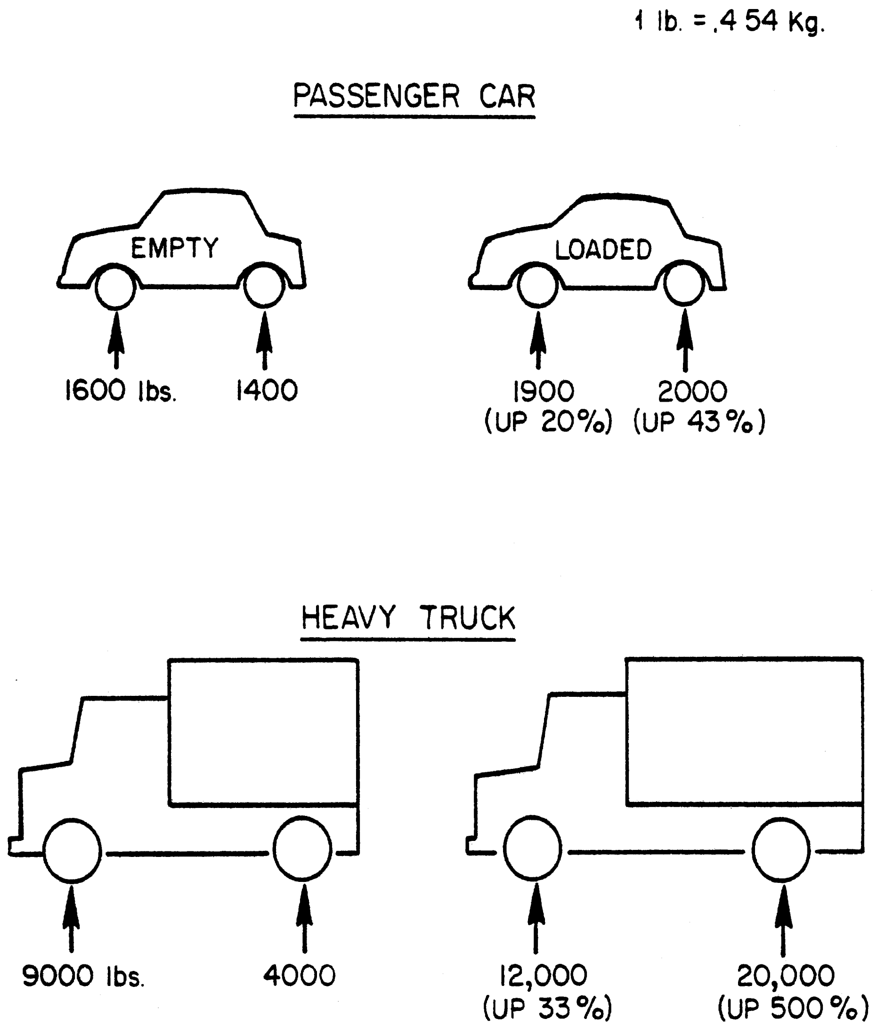


Figure 16. Difference in the empty vs. loaded states of car and heavy truck.

culminate in a rapid rotation of the tractor or trailer units of the vehicle combination relative to one another. The significance of the lockup event is that the pneumatic tire fails to sustain lateral forces when the wheel is locked. As shown in figure 17, lockup of tractor drive wheels, with the steering wheels still rolling, produces the powerful "jackknife" instability which rotates the tractor rapidly relative to the semitrailer. For a vehicle in a left-hand curve, the jackknife articulation motion shown in the figure would prevail upon lockup of tractor drive wheels. As suggested above, lockup of the drive wheels is a rather common occurrence when the vehicle is nearly empty since it is in that state that the drive wheels are very overbraked.

When trailer wheels are locked, a more slowly developing instability called "trailer swing" develops, as shown in figure 17. Although the increase in articulation angle due to the trailer swing motion develops moderately, the total lateral motion achieved at the rear of the trailer can become very large relative to, say, a lane width.

Although the lockup-induced instabilities are most commonly produced on slippery surfaces, they are certainly also possible on dry pavement. In fact, a stop, without wheel lockup, at a deceleration level of approximately 0.4 g's is considered a severe braking condition, even on dry pavement, with a heavy truck. As an historical note, the Federal Motor Vehicle Safety Standard, No. 121, requiring a deceleration capability of 0.41 g's for air-braked trucks stopping from 60 mi/h (96 km/h) was seen as imposing a serious challenge to the state of truck braking technology in the mid-1970's [25,26]. This standard, applying to stopping on dry pavement, implied a braking efficiency level of only 50 percent and was difficult to meet without wheel lockup, over the empty-to-full loading range, with many vehicles.

When the pavement is wet, it is not unreasonable to expect peak friction levels around 0.5 with car tires, and there is a substantial body of evidence that truck tires produce even lower traction levels under the same pavement conditions (see, e.g., [27,28]). Accordingly, an efficiency level of 50 percent suggests a maximum braking capability

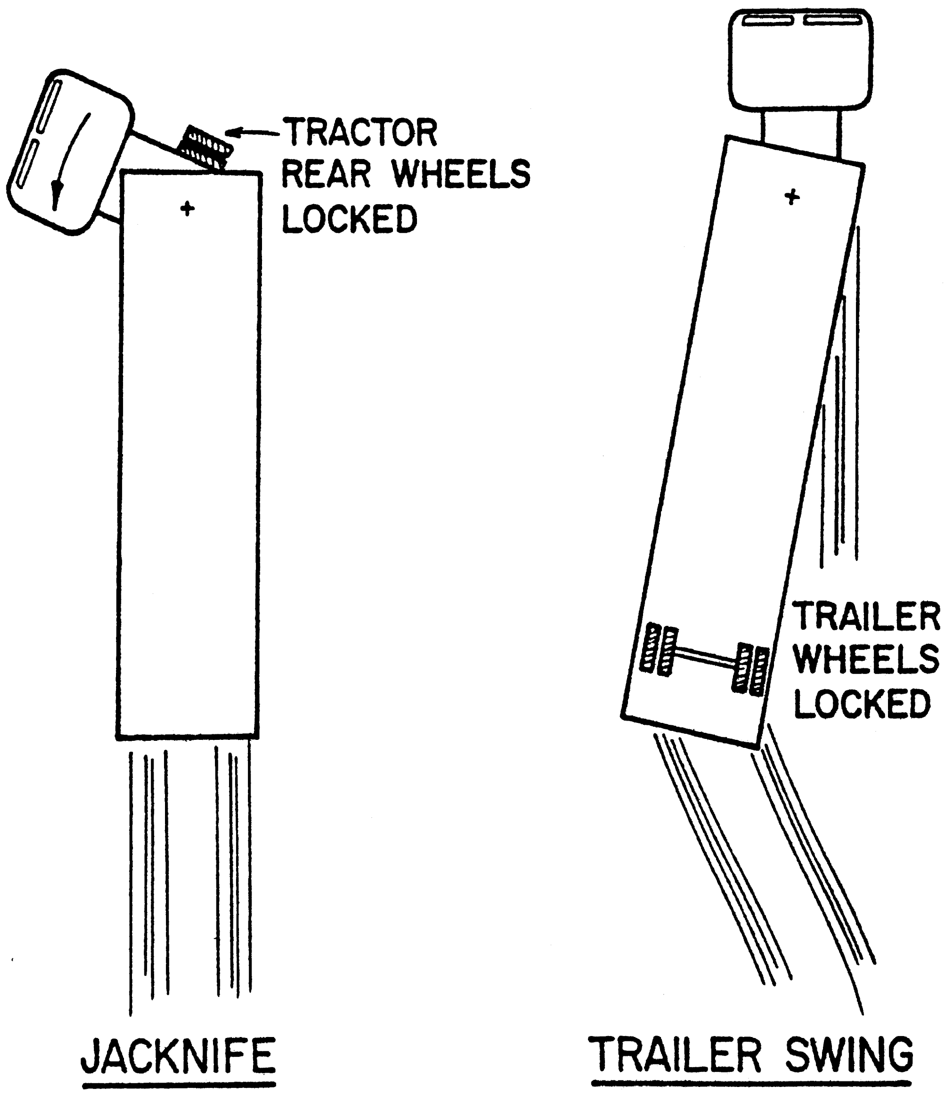


Figure 17. Tractor-semitrailer articulation instabilities during braking.

around 0.25 g's. Roadway layouts calling for braking levels approaching such values of deceleration tend to dramatically increase the probability that trucks will encounter the loss-of-control modes resulting from wheel lockup.

Speed Control on Short Downgrades

In order for loaded truck combinations to keep speed in check while operating on downgrades, the vehicle must develop retardation forces either through engine braking, the use of service brakes, or by means of a supplemental retarder device, if available. Further, each of these mechanisms is under the control of the driver, such that conscious decisions regarding downhill speed control are required. Even for short downgrades such as may be encountered on a ramp, the maintenance of speed requires a retardation strategy.

Although it can generally be said that trucks should have no difficulty in achieving the retardation levels needed to control speed on short grades, it is instructive to note that heavy-duty trucks exhibit considerably lower levels of "natural deceleration" response during coasting than do cars. The dominant "parasitic drag" mechanisms determining the coasting decelerations of cars and trucks are tire rolling resistance and aerodynamic drag. By both mechanisms, loaded trucks exhibit approximately half of the levels of drag forces, per unit vehicle weight, produced by cars such that the coasting decelerations are half, or less than those of cars (see, e.g. [29,30]). Accordingly, one could surmise that it is necessary that truck drivers be especially attentive to applying the needed supplementary retardation on ramp downgrades if excessive speed is to be avoided. As will be seen in the next section, such attention is particularly warranted on curved ramps because the implications of overspeeding on the risk of rollover are dramatic.

The Ability to Accelerate During Merging

Heavy-duty trucks clearly have a very reduced ability to accelerate up to highway speeds, relative to other vehicles. The acceleration capability is very simply limited by engine power level, given the large mass of a fully loaded unit. While no direct loss-of-control mode is seen to be associated with the task of accelerating a heavy vehicle, it is useful to note that the severe limitation in this performance area very likely influences the driving strategy of truck drivers in certain respects. For example, since some 5,000 feet (1,524 m) are required to accelerate a typical 80,000-lb (36.4-Mg) rig from 25 mi/h to 50 mi/h (40 to 80 km/h), it seems reasonable to expect that the driver will try to avoid slipping down to a reduced speed level whenever possible [31]. If "keeping speed up" implies a tendency to overdrive connecting ramps between freeways, for example, the poor acceleration capability of trucks becomes an indirect mechanism for promoting a risky driving behavior. Such behavior might lead to increasing throttle partway through a ramp, or minimizing retardation braking on a ramp downgrade in order to facilitate the subsequent merging process.

5.3.2 Summary of Simulation Results and Discussion of Design Implications. In this section samples of the simulation results will be presented as they illustrate the specific maneuvering problems which heavy trucks encounter on certain interchange ramps. The "problems" are organized in a manner which is similar to that used in section 4.2 to discuss the geometric designs of differing types of ramps selected for study. That is, the loss-of-control modes which have been linked to ramp designs are discussed in terms of the mode, itself, and the ramp features which precipitate it. Ramp design is examined in each case relative to the design policy of AASHTO, thus extending the considerations of section 4.2. In the following discussion, each of six problematic design features will be presented. The full presentation of simulation results covering the matrix of runs outlined earlier in table 8 appears in appendix C.

Case 1 Excessive Levels of Side Friction Demand

While a number of the other cases to be discussed will involve rollover under nominally static conditions, the first case is presented as evidence of the specific conclusion that AASHTO's allowance for side friction factor leaves a very slim margin of safety for many heavy truck combinations. This margin is further narrowed when superelevation is developed poorly. Nevertheless, regardless of the transition details and other features of ramp design, the mere presence of curves imposing side friction demands which approximate the value, in g's, of truck rollover thresholds, poses a special hazard to trucks.

The ramp case chosen for illustrating this problem is the exit ramp from site no. 1 which is shown again in figure 18. As discussed in section 4.2.1, this site incorporates a curve that is preceded and followed by spiral transitions in which the full superelevation is not achieved. Thus, the side friction level computed continuously along the curve peaks at 0.21 for the posted advisory speed of 35 mi/h (56 km/h).

Shown in figures 19 and 20 are simulation results illustrating the dynamic response at 35 mi/h and 40 mi/h (56 km/h and 64 km/h), respectively, of a tractor-semitrailer which is loaded with freight in the "high-c.g." configuration (payload mass center at 105 in (2.67 m)) and which is operated over the cited curve. The results show that the vehicle at 35 mi/h (56 km/h) experiences a near rollover, with a large amount of load being transferred from right- to left-side tires. The transient character of the maneuver is such, however, that the roll response has not fully developed at the occasion of the peak lateral acceleration level. Thus, the vehicle "just squeaks by" at the posted speed by virtue of the relatively short-lived peak demand condition.

In the 40 mi/h (64 km/h) case, figure 20, we see that the tire loads on the right side have reached zero at approximately 5.5 seconds into the run--at which time the vehicle is approximately 50 ft (15.2 m) beyond the point of curvature. Although the zero-load condition on the tractor's inside wheels signals an imminent rollover, the body of the vehicle is seen to be rolling over rather slowly such that it will not strike the ground for another few seconds. (The simulation model is not configured, however, to provide an accurate portrayal of roll motions well beyond the wheel-liftoff condition.)

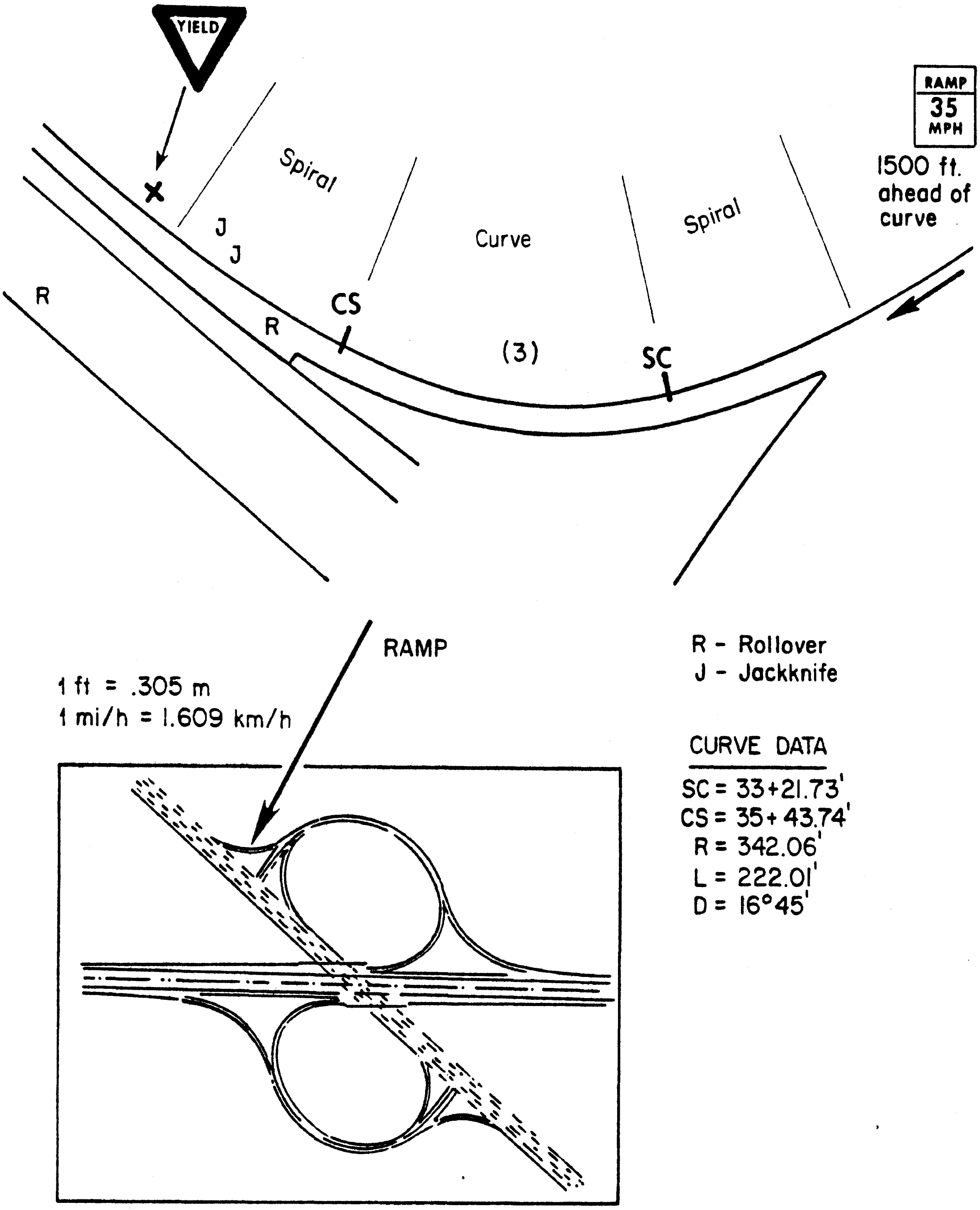


Figure 18. Layout of site no. 1.

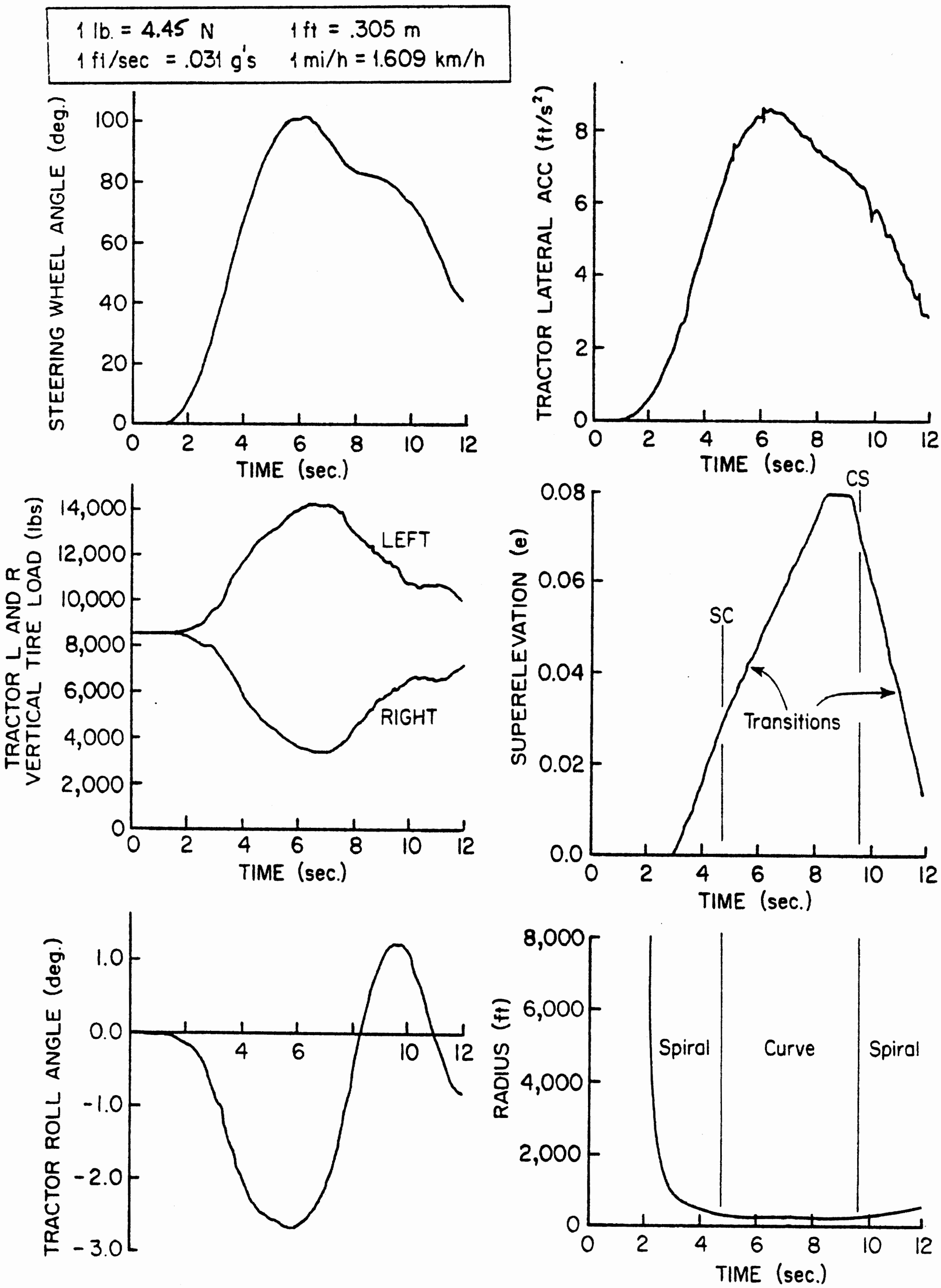


Figure 19. Vehicle response at site no. 1 - 35 mi/h.

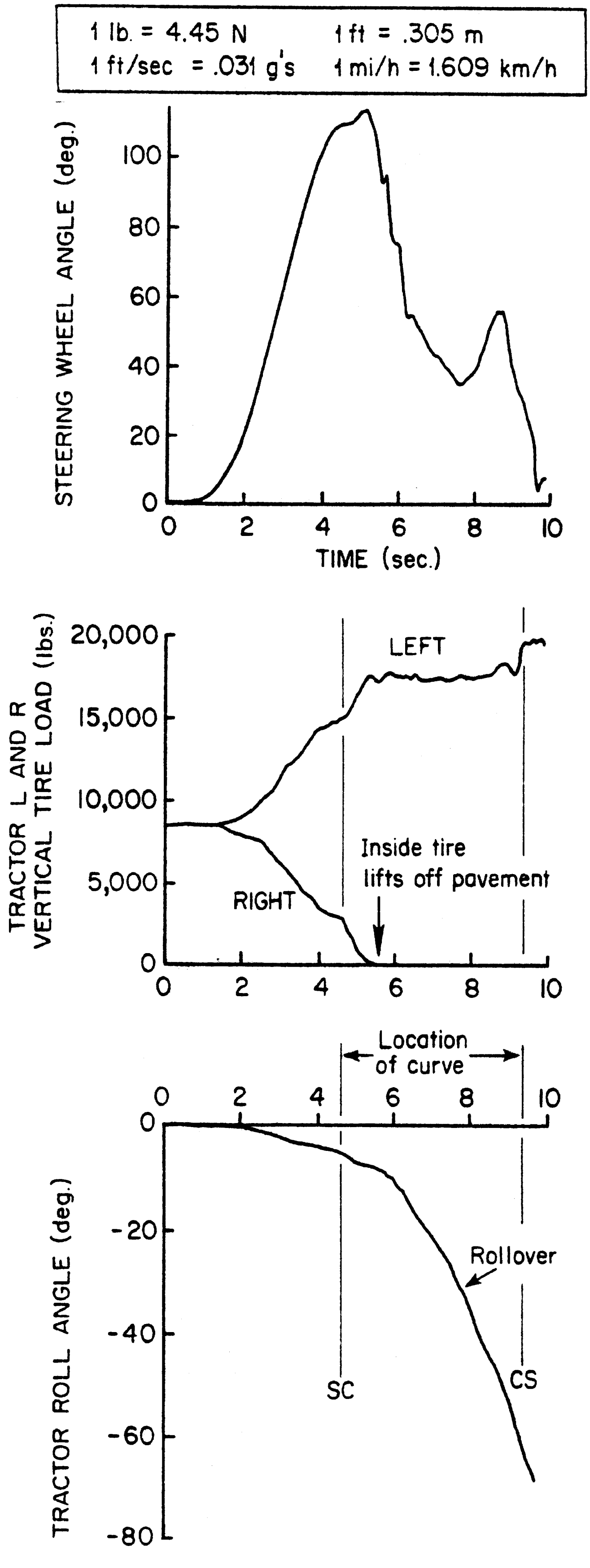


Figure 20. Vehicle response at site no. 1 - 40 mi/h.

Although, at first note, it seems very surprising that a common commercial vehicle will nearly rollover at the posted ramp speed on a primary U.S. highway, it is instructive to examine the margin of safety which is reflected in the side friction factor pertaining to the cited curve. Shown in figure 21 is a diagram providing further elaboration on the "continuous friction factor" condition at this site, plotted as a function of the longitudinal position along the ramp section. The figure presents the centripetal acceleration, (e+f), and the side friction factor, (f), pertaining to the 35 mi/h (56 km/h) advisory speed, plus a suggested "likely" side friction demand curve which is 15 percent above the (f) curve--providing a tolerance corresponding to the level of steering fluctuations which have been measured in tests of the normal driving of a tractor semitrailer through expressway ramps [32]. Because superelevation is not fully developed along the spiral transition, we see that the peak side friction factor reaches a value of 0.21 at the point of curvature, SC, and corresponds to a total peak demand level of 0.24, allowing for steering fluctuations. This demand level is essentially equal to the steady-state rollover threshold limit of fully loaded tractor semitrailers which lie at the low end of the stability range of vehicles in common service, as discussed earlier.

In order to reconcile the clear hazard that such a curve will pose for many heavy-duty vehicles, it is useful to note, first, that at the final superelevation value of 0.08 ft/ft, the curve would be characterized by a nominal friction factor of 0.16. This value is in virtual compliance with the AASHTO recommendation of a maximum of 0.155 for the side friction value in curves posted at 35 mi/h (56 km/h). The first "issue," then, concerns the very basic matter of the suitability of a design policy allowing friction factor levels of .155 (or .16), recognizing levels as low as 0.24. A full discussion of this matter would require review of a) the essential basis for the AASHTO policy on side friction factors and b) the mechanics and operational realities determining the roll stability levels of heavy commercial vehicles. While no comprehensive treatise can be attempted here, a minor elaboration on each point is warranted.

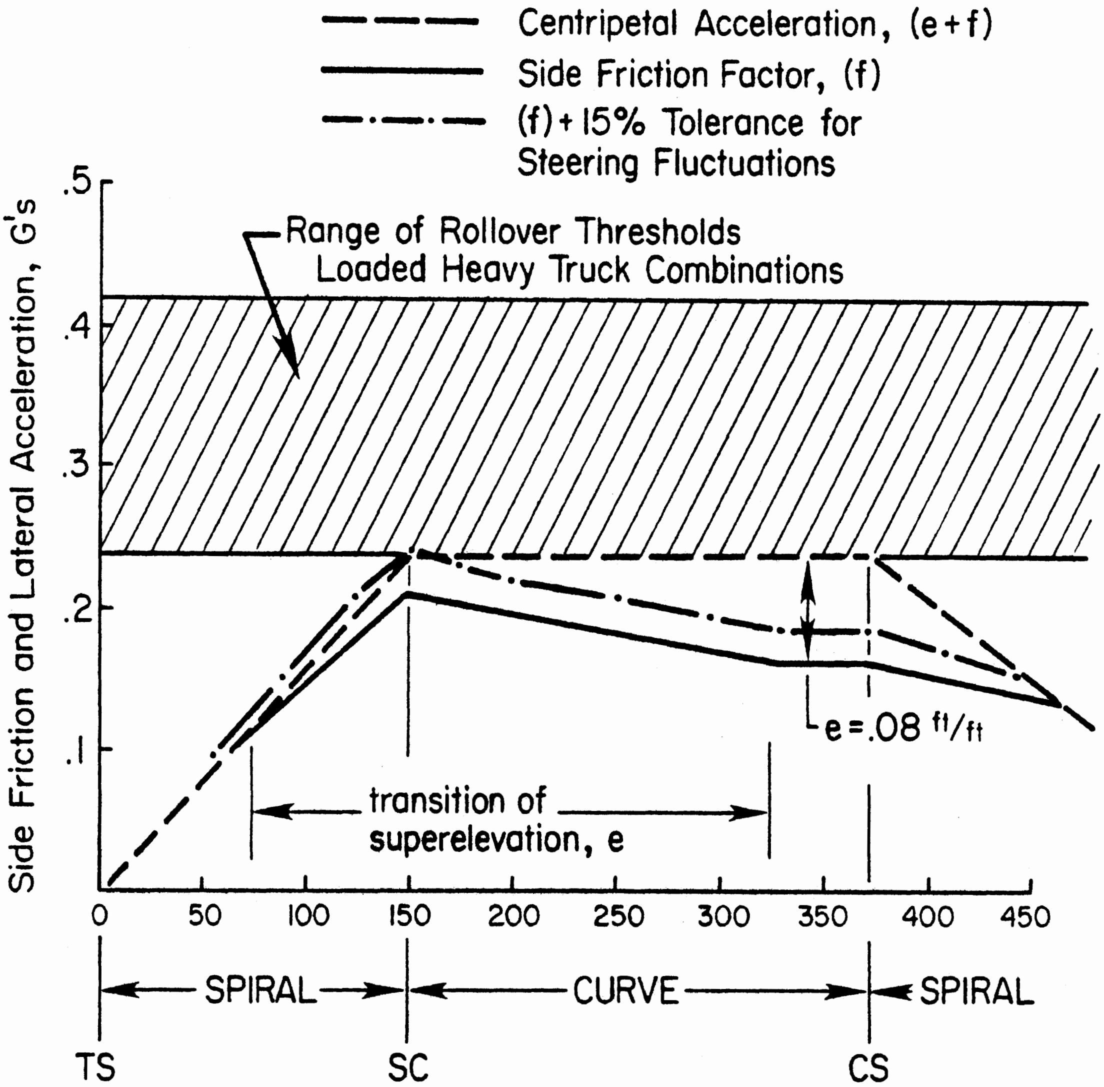


Figure 21. Elements of side friction demand, compared to range of truck rollover tolerance, for ramp curve at site no. 1.

The AASHTO policy [15] on side friction factor allowance is clearly based upon consideration of (1) the proximity of the friction demand level to the lateral traction limits of passenger cars, beyond which "side skidding" may occur and (2) the point of discomfort as noted by passenger car drivers. In this regard, it is clear that maximum recommended values for side friction factor have been set by AASHTO primarily in order to avoid driver discomfort. With regard to the margin of safety, it is apparent that this policy intends a substantially larger margin than is, in fact, achieved with heavy trucks which are at the lower (but by no means rare) end of the stability spectrum. For example, the discussion of the AASHTO policy in the Green Book identifies that the effective limit condition is established by the maximum side friction capacity of car tires (as low as 0.35 at 45 mi/h (72 km/h)) which can be sustained without skidding on wet pavements with tread-worn tires. Accordingly, the guidelines which limit the design value of side friction factor (to a maximum of 0.17 at 20 mi/h (32 km/h)) appear to reflect a substantial degree of conservatism in behalf of passenger cars. Indeed, the design policy for side friction factors has been derived to accommodate the limits of driver discomfort--at which levels the conservatism relative to side skidding is quite generous.

Considering the margin of safety for trucks, however, it is apparent that there also exists a fundamental difference between the respective probabilities that trucks and passenger cars will "bump against" their respective maneuvering limits at the time of traversing a demanding ramp. While, on the one hand, a car may be constrained by a 0.35 traction coefficient only when a) smooth tires and b) a poor pavement texture condition are combined with c) wet weather, an adversely loaded truck will be constrained by its low rollover threshold characteristic <u>continually</u> as it goes down the road. Accordingly, we see not only that the truck margin of safety on AASHTO-recommended ramps can be exceedingly narrow, in absolute terms, compared to the margins provided for cars but also that the risk of control loss for certain trucks is continual rather than temporally dependent upon vehicle and pavement maintenance factors and weather.

Although the transition of superelevation in this example is non-ideal, and certainly disapproved of by AASHTO as a design practice, the fact that a zero margin of safety exists with some trucks should not be dismissed as attributable simply to the transition anomaly. For the more common cases in which superelevation is transitioned without spirals, the AASHTO-preferred method would have two-thirds of the superelevation achieved at the point of curvature. Even this policy would still allow a side friction factor as high as 0.20 in the transition portion of the curve, thus yielding 0.23 as the effective side friction demand level, allowing for steering fluctuations. Thus, it appears that the problem which led to the identification of the ramp at site no. 1 as heavily involved in truck loss-of-control accidents is a) understandable in terms of ramp geometry and b) rather generally anticipated for ramp curves which are built to the limits of the recommended AASHTO practice. While, in practice, it may take a combination of poor signage, an awkward compound curve arrangement and other factors to render a particular site overinvolved in truck accidents such as site no. 1, side friction factors which peak around 0.20 clearly provide a very slim margin of safety against truck rollover.

It is also worthy to note that AASHTO design policy for low speed urban streets allows side friction factors up to 0.30! Such a level will surely yield rollover in a large fraction of the population of loaded commercial vehicles.

Case 2 Awkward Compound Curves

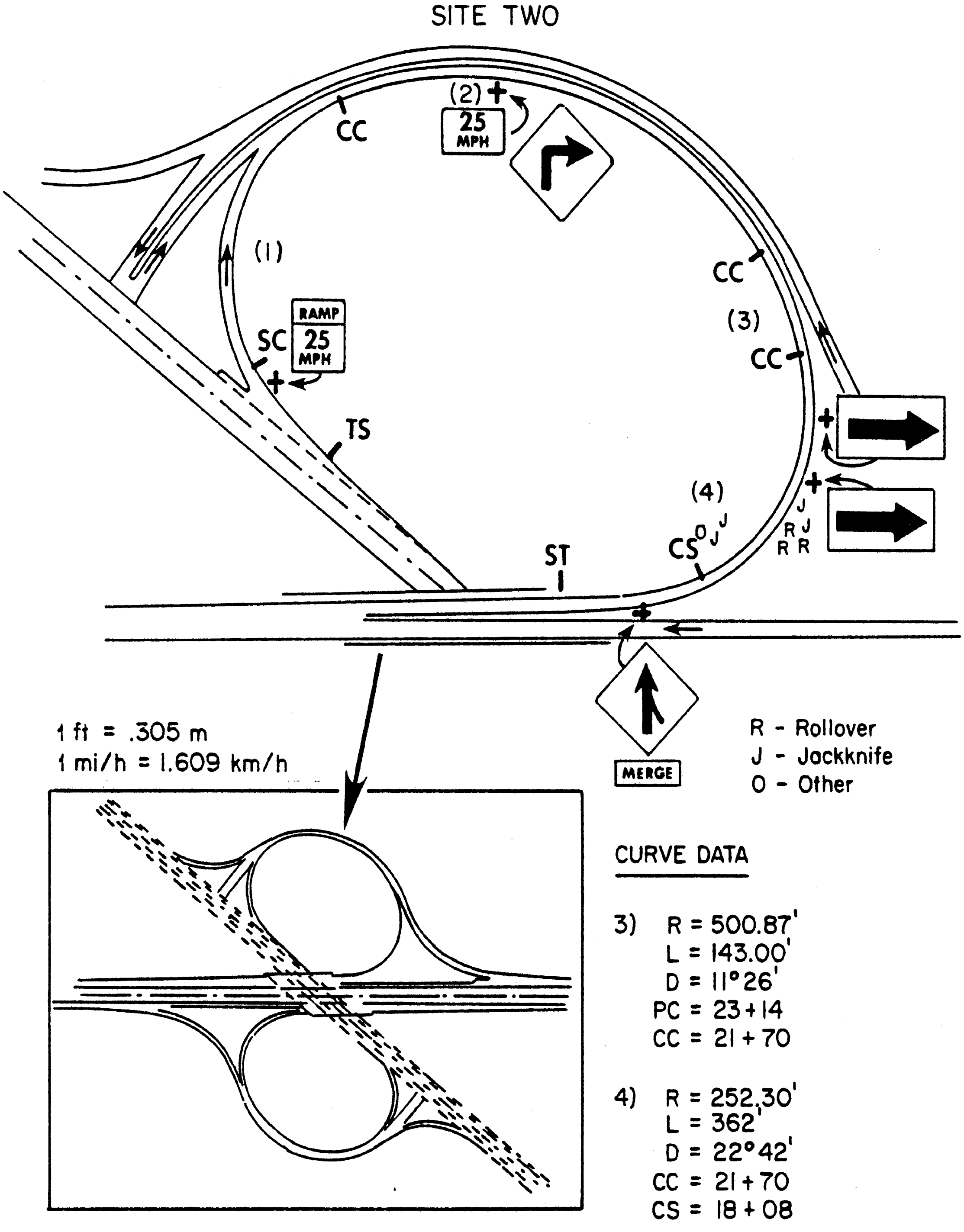
Three of the selected ramps incorporated multiple curved segments, each having a different side friction factor demand, although only one ramp speed was posted. At such sites, it appeared that truckers occasionally assumed, at some point along the ramp, that they had already passed the curve(s) which warranted the low value for the posted speed. Subsequently, they apparently sped up in preparation for the merging task, only to find that the remaining curve was at least as demanding of the low advisory speed condition as was the preceding portion of the ramp.

A case in point is ramp site no. 2, shown in figure 22--comprising a loop which has four curves within a partial cloverleaf, rural interchange. The ramp is posted at 25 mi/h (40 km/h), and involves two rather sharp curves at either end with two intermediate curves having more moderate radii. Listed below are the essential data for each of the four curves.

Curve No.	Radius (ft)	Length (ft)	Side Friction Factor
1	250	435	0.09
2	520	993	0.00
3	500	144	0.003
4	252	362	0.09

Spiral transitions to the tangent legs at both ends of this ramp provide that both curves (1) and (4) are superelevated at 0.08 ft/ft throughout their lengths. Thus, the nominal values listed above for side friction factor are also the maximum values. The AASHTO Green Book cautions against the use of compound curves of this type and particularly recommends against the design of a flatter curve section between two sharper curves.

Shown in figure 23 are simulation results for the baseline vehicle traveling through curves (3) and (4) at the posted advisory speed of 25 mi/h (40 km/h). Although the indicated responses do not represent any limit type of vehicle behavior, the abrupt jumps in lateral acceleration upon entry to curve (4) illustrate the nature of the apparent problem. If the driver increases his speed through curves (2) and (3), he may arrive in curve (4) at a speed which exceeds the vehicle's controllability limits. Indeed, as was illustrated in figure 22, the truck accidents occurring on this ramp were all clustered at the approximate midlength location of curve (4). Since curves (1) and (4) are both characterized by identical values of side friction factor, it can only be surmised that truck drivers a) reasonably satisfy the speed requirements of curve (1), but then b) misjudge the continuing need for retaining the low advisory speed while traveling the 1,100 ft (335 m) through the mild curves, (2) and (3). Results in appendix C show that



1 ft = .305 m
1 mi/h = 1.609 km/h

Figure 22. Layout of site no. 2.

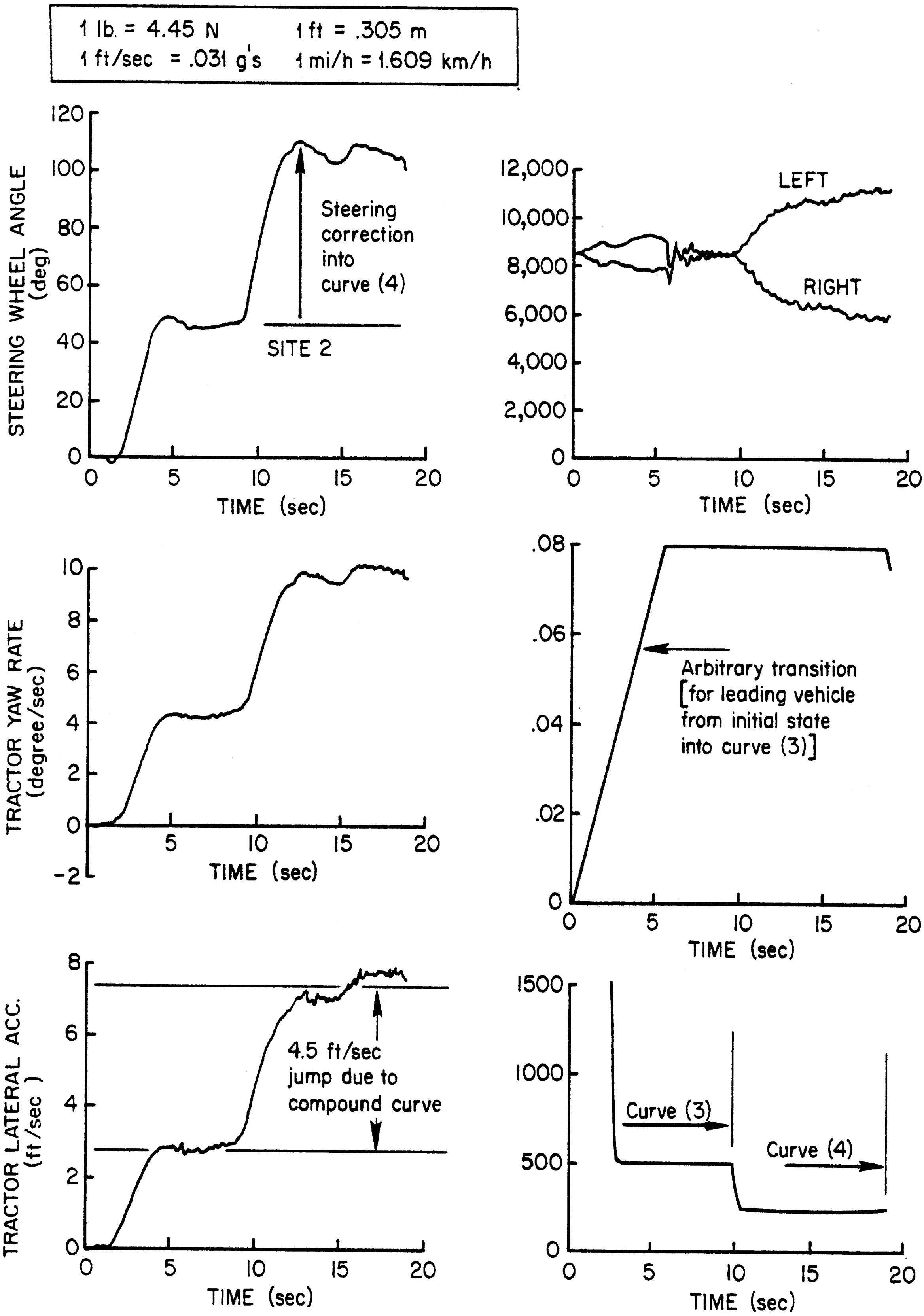


Figure 23. Vehicle response at site no. 2 - 25 mi/h baseline vehicle.

the baseline vehicle rolls over in curve (4) at a speed of 42 mi/h (68 km/h). By way of extrapolation, one can show that the "high-c.g." tractor-semitrailer would roll over in curve (4) if the driver permits his speed to exceed 34 mi/h (55 km/h).

In addition to a number of rollover incidents reported at this site, an equal occurrence of jackknife accidents is also reported, suggesting that heavy braking is probably being applied when the driver approaches curve (4) and perceives that general loss-of-control is being threatened. Jackknife due to overbraking will be illustrated in a subsequent case.

Curve (4) ends with a spiral transition and then a tangent acceleration lane which is 1,200 ft (367 m) long to the very end of the taper. The length of this lane, to the point at which the taper is within 12 ft (3.7 m) of the edge of the through lane, is approximately 1,000 ft (305 m). Although the acceleration lane is virtually equal in length to the minimum value recommended by AASHTO, the available distance for accelerating a loaded truck from the 25 mi/h (40 km/h) ramp speed up to, say, a 50 mi/h (80 km/h) free-merging speed is almost inconsequential, considering that such vehicles may require approximately 5,000 ft (1,524 m) as an acceleration distance. Thus, one could hypothesize that the very short lengths of acceleration lane available for bringing a fully loaded rig up to speed (both at this site and in the highway system, generally) serve to encourage the driver to achieve as much speed within the ramp as possible before merging. While we can readily criticize the truck driver who exceeds the posted ramp speed, it seems more realistic to observe that the sum of the highway geometric constraints imposed in this case have "boxed in" the driver and, perhaps, promoted the possibility of misjudgments.

Case 3 Short Deceleration Lane Leading to Tight-Radius Curve

The 1965 AASHTO Blue Book gives a definitive background rationale behind the recommended lengths of deceleration lanes. Notwithstanding the careful basis which is developed for designing such lanes to meet the needs and comfort threshold of car drivers, both the Blue and Green book specifications for deceleration lanes place a substantial burden upon the stopping capability of many heavy-duty truck combinations. The background figures in the Blue Book reveal that the "comfortable" level of deceleration for passenger car drivers slowing from 55 mi/h (88 km/h) is 0.24 g's. The recommended lengths for deceleration lanes are calculated to allow approximately 3 seconds of deceleration of the vehicle in gear, followed by braking at the "comfortable" passenger car rate. The Blue Book does note that trucks require longer stopping distances than cars to decelerate for the same difference in speed, but finds longer allowances for deceleration lanes unwarranted because "average speeds of trucks are generally lower than those of passenger cars." Although the Green Book does not restate the observation concerning truck speeds, the newer recommendations for length of deceleration lane are virtually identical to those in the 1965 policy. Further, it seems reasonable to observe that average truck speeds on U.S. highways today are at least equal to, and perhaps exceed, those of passenger cars.

The cases in which the length of deceleration lanes becomes a special problem for trucks are those in which the ramp incorporates a rather sharp curve right at the end of the deceleration lane such that the low value of advisory ramp speed must be achieved very quickly upon departure from the through roadway. Shown in figure 24 is an example of such an exit ramp, having a 249 ft (76 m) curve radius and a maximum superelevation value of 0.08 ft/ft. The side friction factor has a peak value of 0.13 at the advisory speed, given a transition which achieves approximately 50 percent of the full development of superelevation at the point of curvature.

The tapered exit begins 375 ft (114 m) ahead of the point of curvature, thus requiring a nominal deceleration of 0.21 g's even if braking begins immediately at the front edge of the taper, in order to achieve the 25 mi/h (40 km/h) ramp speed upon entry to the curve. The 0.21 g requirement allows no distance for delay in brake application beyond the leading edge of the taper and assumes that the vehicle will begin decelerating while still placed fully in the through lane. Even

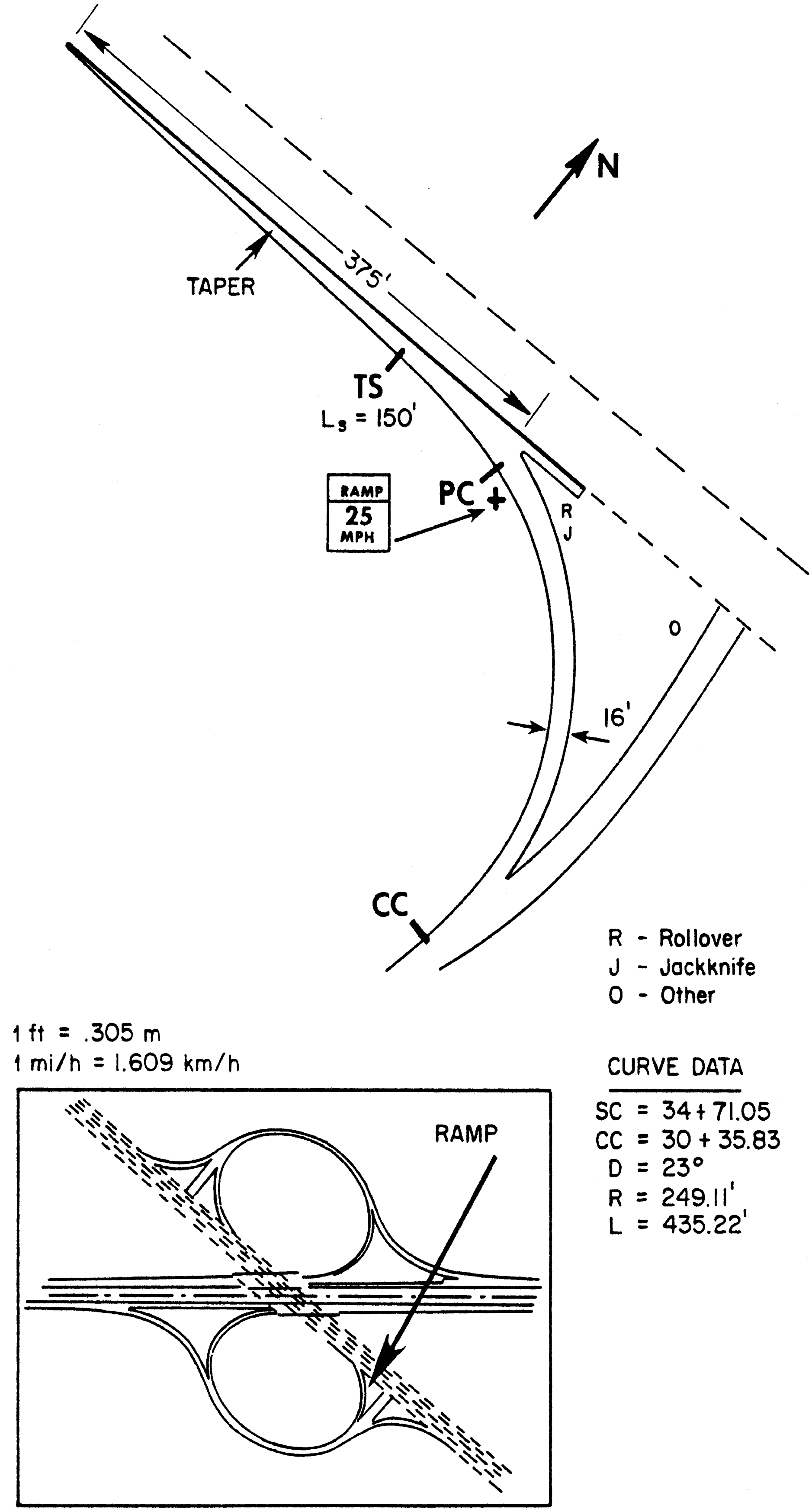


Figure 24. Layout of site no. 3.

per the AASHTO recommendations, this deceleration lane is very short, providing only approximately 100 ft (31 m) of roadway that should be "counted" for deceleration, recognizing that the acknowledged deceleration lane begins only at the point at which the taper has progressed 12 ft (3.7 m) from the right edge of the through lane.

The penalty paid by trucks which fail to achieve the required speed upon entry to this curve is, of course, most likely to be rollover. The accident data show both rollover and jackknife accidents occurring right at the beginning of the example curve. Of course, the jackknife accidents are seen as simply resulting from the overbraking behavior of truck drivers who are endeavoring to achieve a speed which is low enough to avoid rollover.

Simulation results shown in figure 25 illustrate the baseline tractor-semitrailer passing easily through the curve at site no. 3 at 25 mi/h (40 km/h), but barely escaping rollover at 35 mi/h (56 km/h). Other calculations for the "high-c.g." vehicle, presented in appendix C, show that the rig rolls over quickly upon entering the ramp at 35 mi/h (56 km/h). Thus, there is no question that the deceleration task must be accomplished by most loaded truck combinations if they are to safely negotiate curves having this degree of "demand."

Although the jackknife response was not simulated at site no. 3, an illustration of the basic jackknife phenomenon is shown in figure 26, in which overbraking during entry into a spiral transition (at site no. 4) causes the tractor drive axles to lock up such that tractor yaw rate diverges rapidly into the jackknife collision between tractor and semitrailer. The jackknife motion is very rapid, with the tractor reaching the point of collision with the trailer in approximately 2.5 seconds. (In many of the accident reports examined in this study, jackknifing on a curve resulted in the vehicle departing off of the inside of the curve. Apparently, this resting point is attained because the driver releases the brakes after the tractor has rotated through a modest articulation angle, whereupon all of the tractor tires recover their rolling condition such that large cornering forces are developed for propelling the rig off the road toward the inside. For vehicles

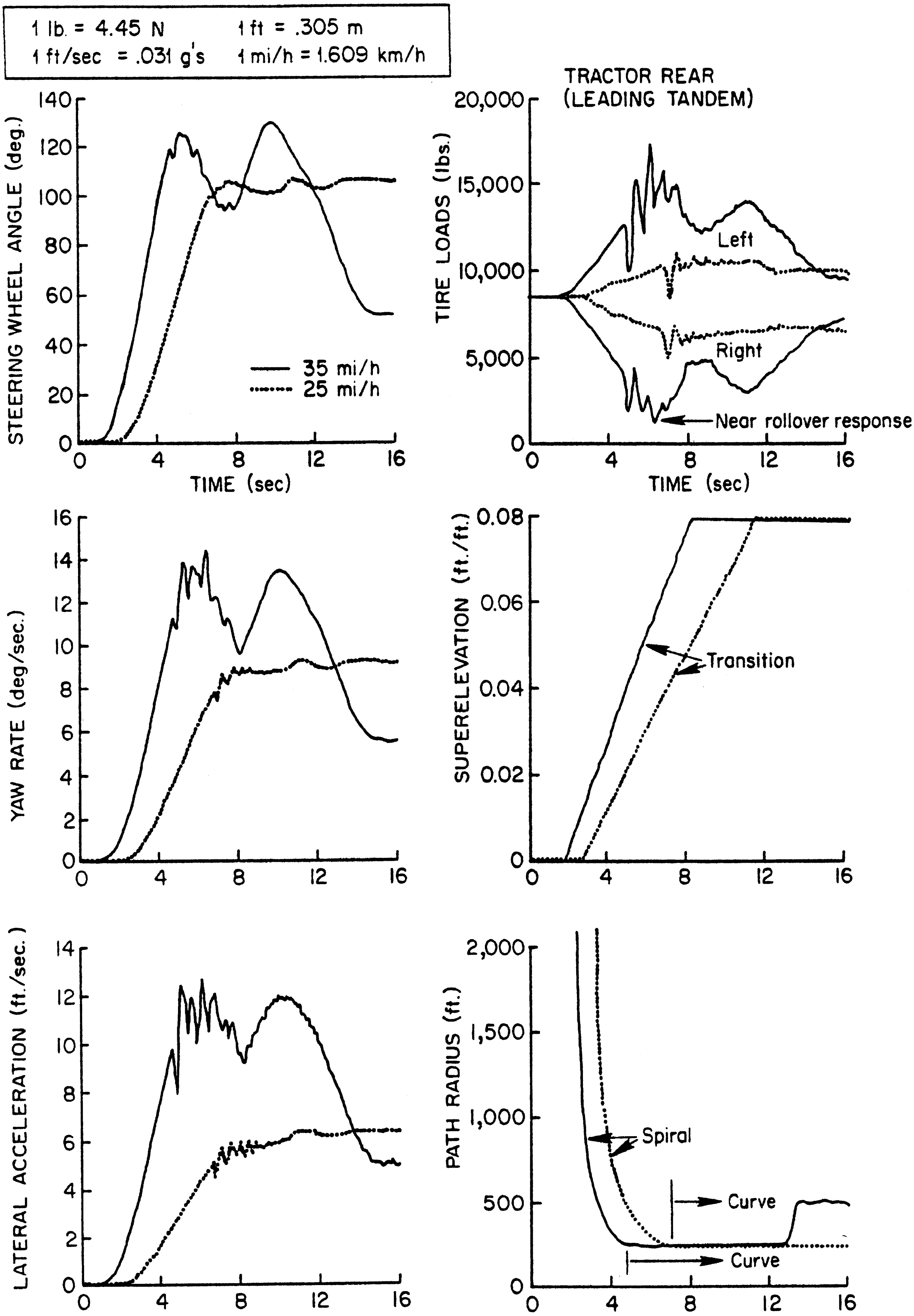


Figure 25. Vehicle response at site no. 3 - baseline vehicle, 25 and 35 mi/h.

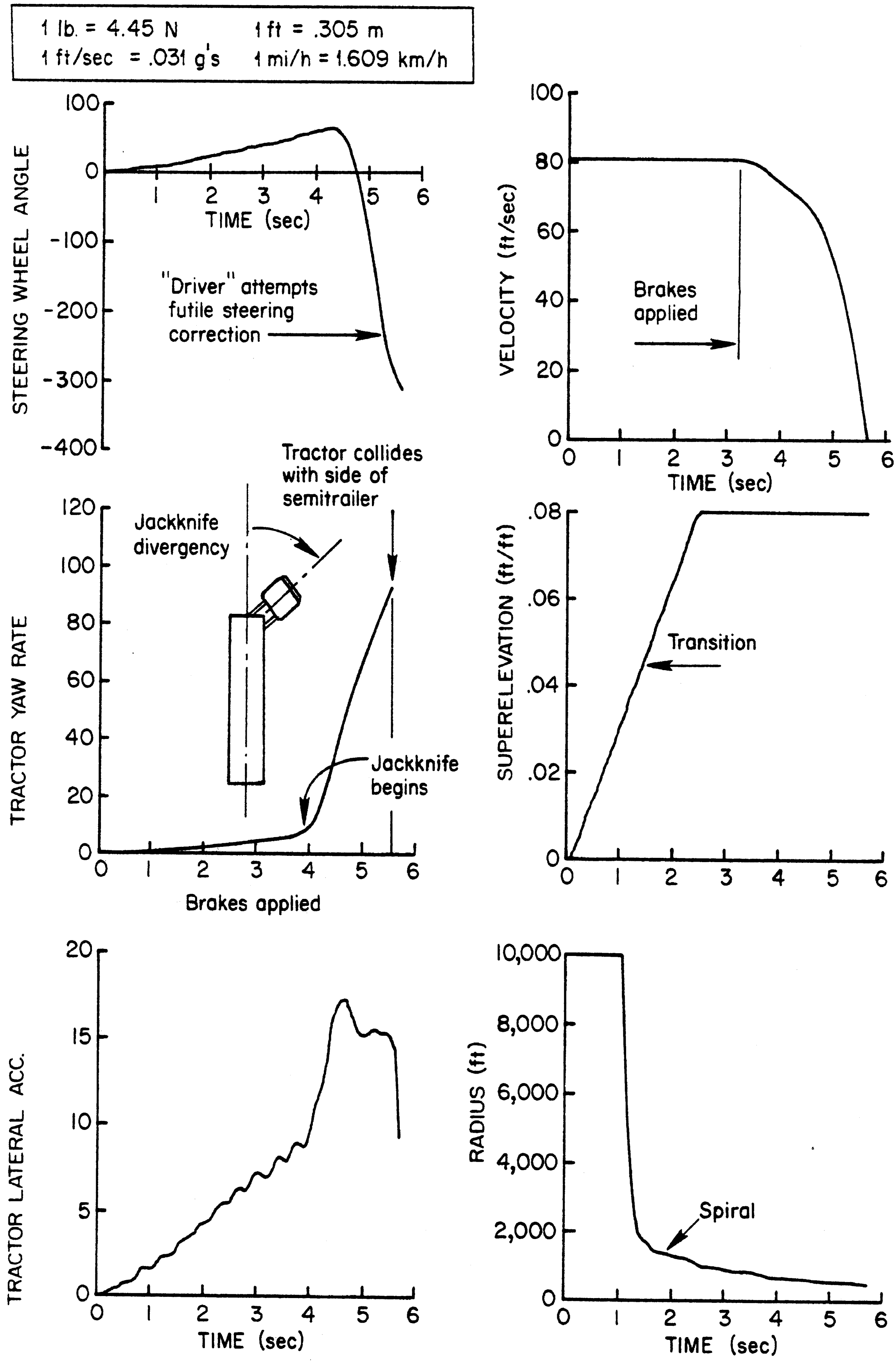


Figure 26. Vehicle response at site no. 4 - empty trailer, 50 mi/h brake pressure, 30 psi, pavement friction, 0.50.

which experience a complete jackknife rotation, with the tractor cab striking the semitrailer, the vehicle proceeds essentially along a tangent to its original path.)

The general issue posed by the case of site no. 3 is the extent to which deceleration requirements of the level represented in this case, and more generally of the level implicit in AASHTO policy, can be reasonably accomplished by heavy duty truck combinations. The discussion in section 5.3.1 suggested that the characteristic limits in deceleration capability for such vehicles are very low, with efficiencies falling at 50 percent, and below. In the particular case of short deceleration lanes leading to tight-radius curves, the vehicles which are especially vulnerable are those which tend to have both a poor stopping capability and a low rollover threshold, as well. (Note that an empty vehicle may have a quite low braking capability but is compensated, to a substantial degree, by a rather high rollover limit such that it can pass through the initial tight curve at well above the advisory speed without suffering rollover.) Under certain partial loading conditions, however, a vehicle can exhibit both a low level of roll stability and a very poor level of braking capability. In such cases, the unfavorable distribution of axle loads makes it very difficult for the truck to decelerate, even though the relatively high c.g. location demands that speed be reduced as required by the curve in order to avoid rollover.

The AASHTO policy for length of deceleration lanes clearly provides for more relaxed braking conditions than those needed on the example ramp, although trucks must "take liberties" with the design relative to the expected usage by passenger cars. In particular, the Green Book requires that deceleration length be measured on tapered exits beginning with the point at which 12 ft (3.65 m) of taper is achieved. By this standard, the example ramp would have been constructed with the taper beginning approximately 390 ft (119 m) sooner than it was. Trucks which begin braking right at the taper of such a deceleration lane, then, would experience only a moderate braking demand. Taking the recommended lengths of deceleration lanes, generally, trucks could make a compromise usage of the suggested design by simply applying brakes throughout the available length of the lane, thus forsaking the "luxury" of a 3-second period for coasting in gear. By this approach, for example, the 490 ft (149 m) value which the Green Book recommends for reducing speed from 55 mi/h to 25 mi/h (88 km/h to 40 km/h) would require a steady deceleration of 0.16 g's--a level which should be reasonably achievable by almost all trucks under most wet and dry conditions.

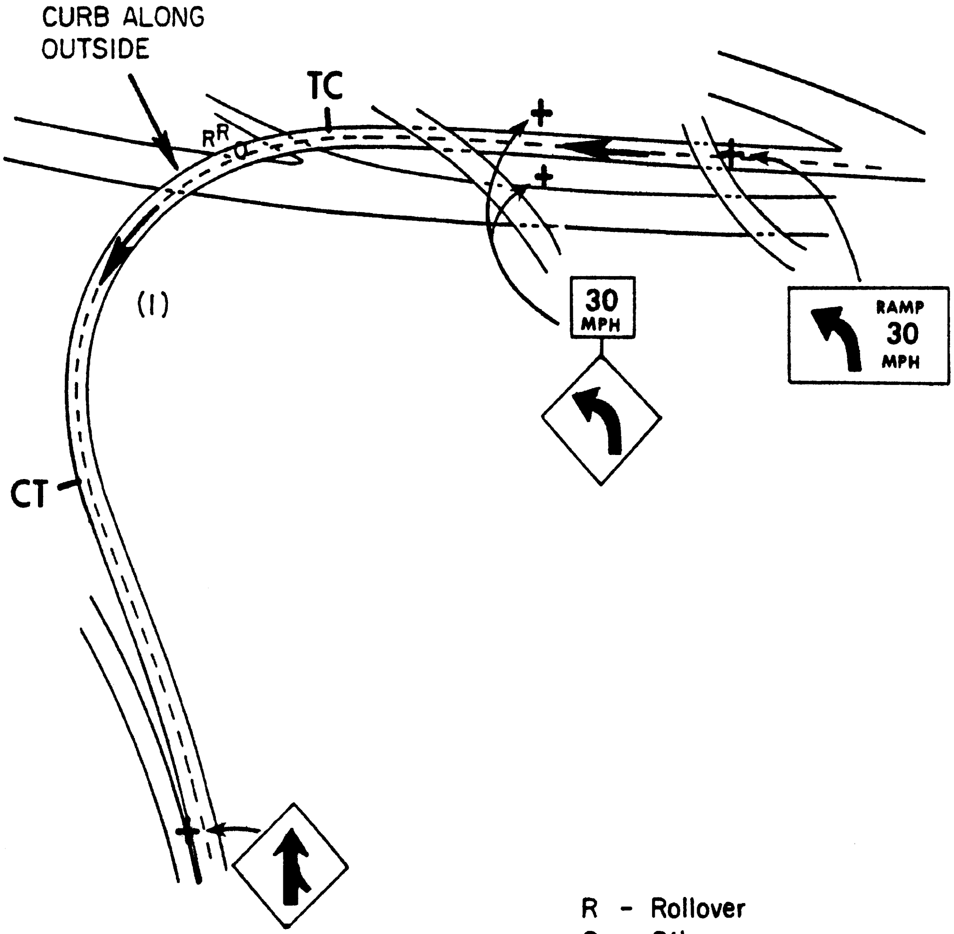
Moreover, the problem posed by short deceleration lanes is analagous to that encountered with allowances for side friction factor. Namely, design specifications which are selected to assure "comfortable" operation of passenger cars tend to pose demands which challenge the controllability limits of heavy-duty trucks.

Case 4 Curb Placed Along the Outside of Curve

As suggested earlier, every truck driver knows that the rear axles on the trailing elements of an articulated truck combination will track inboard of the path of the tractor during low-speed, tight-radius, turning maneuvers. This phenomenon has been called low-speed offtracking and has been recognized as a consideration in highway design for many years. It has been observed, however, that the trailers in tractor-semitrailer and doubles combinations tend to "fling out" in a turn as much as 2 ft to 3 ft (.6 m to .9 m) from the path of the tractor. The particular safety concern which arises from this behavioral characteristic is that the rearmost axles may strike a curb which is situated, on certain ramps, along the outer side of the curve. Since it is thought that truck drivers are generally unaware of this so-called "high-speed offtracking" phenomenon, the safety problem may be exacerbated by the harmful natural instinct of drivers who steer close to the outer curb, maximizing the turn radius while believing that the trailer axles always tend to go inboard.

Shown in figure 27 is the layout of an elevated ramp, site no. 13, at which truck rollover accidents appeared to have involved tripping at an outside curb. The ramp involves 2 12-ft (7 m) lanes constituting

SITE 13



R - Rollover
O - Other

1 ft = .305 m
1 mi/h = 1.609 km/h

CURVE DATA

R = 374'
L = 697.49'
D = 15°19'
TC = 17+17.36
CT = 24+14.85

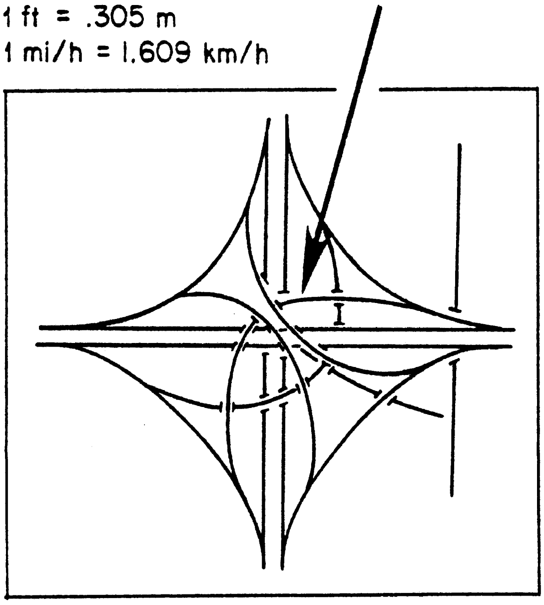


Figure 27. Layout of site no. 13.

an interchange leg between two urban expressways. The curve radius of 374 ft (114 m), together with a superelevation of 0.05 and an original ramp advisory speed of 35 mi/h (56 km/h) yielded a side friction factor of 0.17. The ramp incorporated a cross-section design, as shown in figure 28, with curbs provided to assist in channeling water drainage. The right-hand curb is a mountable type permitting access by disabled vehicles to a paved right shoulder.

This ramp provides, firstly, a relatively severe side friction demand, together with the curb which is within approximately 20 in (.51 m) of the lane edge along the outside of the curve. It would appear that truck combinations may have experienced sufficient outboard offtracking of the trailer axles, due to the substantial side friction factor, that the rearmost outer tire struck the "mountable" curb. Since the sideslipping tire, with its inward orientation, was unable to mount the curb, a lateral force response developed due to the curb contact, thus producing the additional roll moment needed to overturn the truck combination. More recently, the site has been modified with an overlay pavement which bridges the curb and provides a smooth, superelevated, surface from one shoulder edge to the other. The reduction in truck rollover accidents at the site have been attributed to the modification.

Although the response of baseline and "high-c.g." configuration vehicles were simulated at this site, with the results appearing in appendix C, the influence of curb contact was not addressed due to the absence of such a feature in the simulation model. Further, it is believed that no experimental data have ever been reported on the nature of the lateral forces which truck tires might develop as a result of such curb contact. Nevertheless, the prospect for curb contact at curbed ramps seems sufficiently likely that the matter deserves consideration as a design issue.

The practice of employing curbs on the outside of a curved ramp was among the approved design approaches cited in the 1965 AASHTO Blue Book. Even on loops or direct connection roadways having continuous-curve alignment in one direction, curbs along the outside edge were justified as providing "an effective delineator on the high side of the

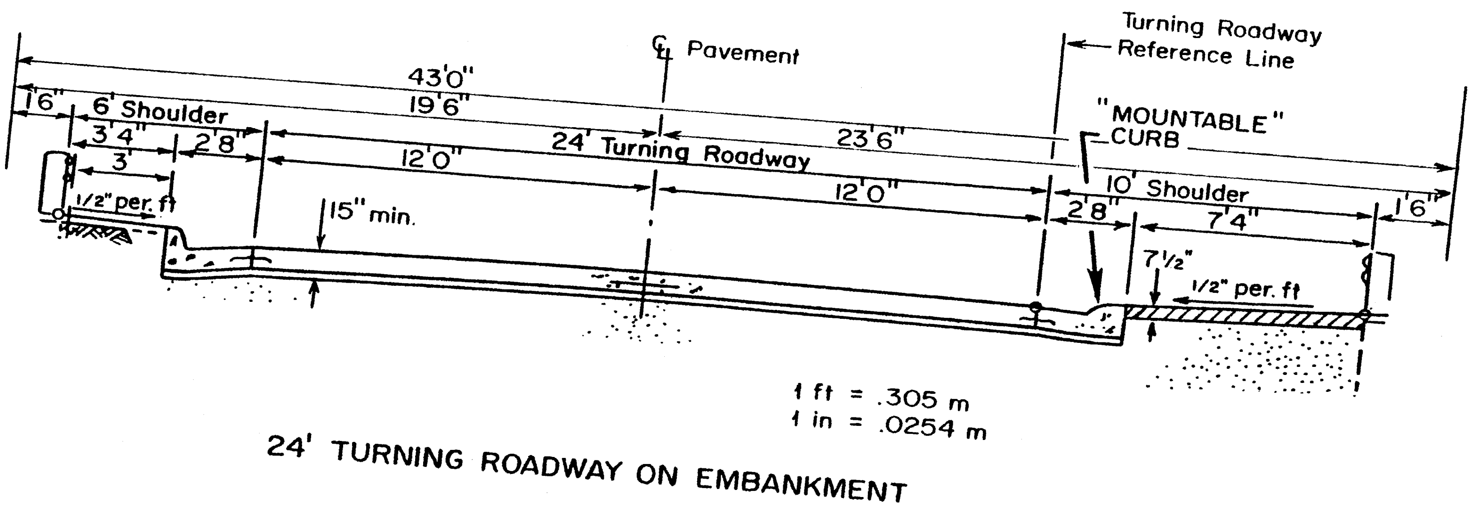


Figure 28. Curb placement at site no. 13.

pavement." In the more recent Green Book, AASHTO policy has apparently changed such that the use of curbs on intermediate and higher speed ramps is not recommended. In fact, the Green Book suggests that curbs be considered only to facilitate particularly difficult drainage situations. Indeed, it is clear that the use of a curb on the "high side" of a superelevated curve cannot be rationalized as an aid to drainage.

Case 5 Downgrade Leading to a Tight Curve

Because of the potential that truck drivers may neglect to provide the needed braking to restrain acceleration on ramp downgrades, excessive speed may develop, jeopardizing the ability of the truck to negotiate a tight curve later in the ramp. A case in point is represented by the ramp at site no. 8, shown in figure 29, at which a substantial number of rollovers were reported to have occurred near the end of the indicated 350-ft (107-m) radius curve. At the posted advisory speed of 30 mi/h (48 km/h) and superelevation level of 0.08, the side friction factor is 0.09. A downgrade begins ahead of the curve and reaches a peak of 5.4 percent approximately 470 ft (176 m) before the end of the curve. As shown in the figure, the state transportation agency has reacted to the rollover experience by placing a special warning sign showing a truck rolling over on the curve.

Shown in figure 30 are the simulation results for a "high-c.g." configuration tractor-semitrailer which coasts down the ramp after entering the curve at a speed of 35 mi/h (56 km/h). The grade causes the vehicle's speed to rise to a maximum of 42 mi/h (68 km/h), at which point the lateral acceleration level has climbed to a level producing rollover. The wheel liftoff condition is observed approximately 180 ft (55 m) ahead of the end of the curve. Given that complete rollover takes on the order of three seconds to develop, the simulated case would have deposited the truck at the same approximate location on the roadside at which the reported rollovers occurred.

SITE EIGHT

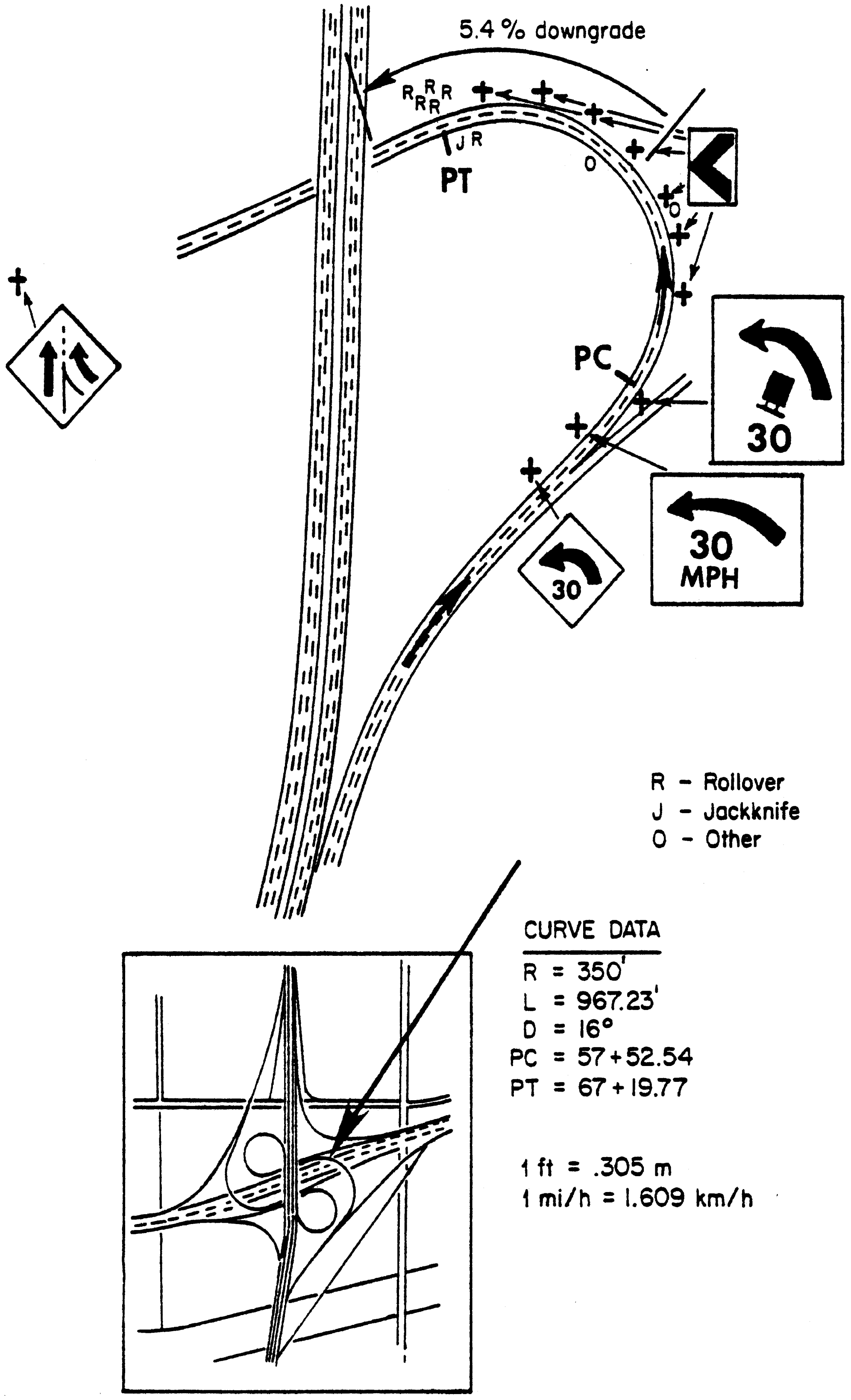


Figure 29. Layout of site no. 8.

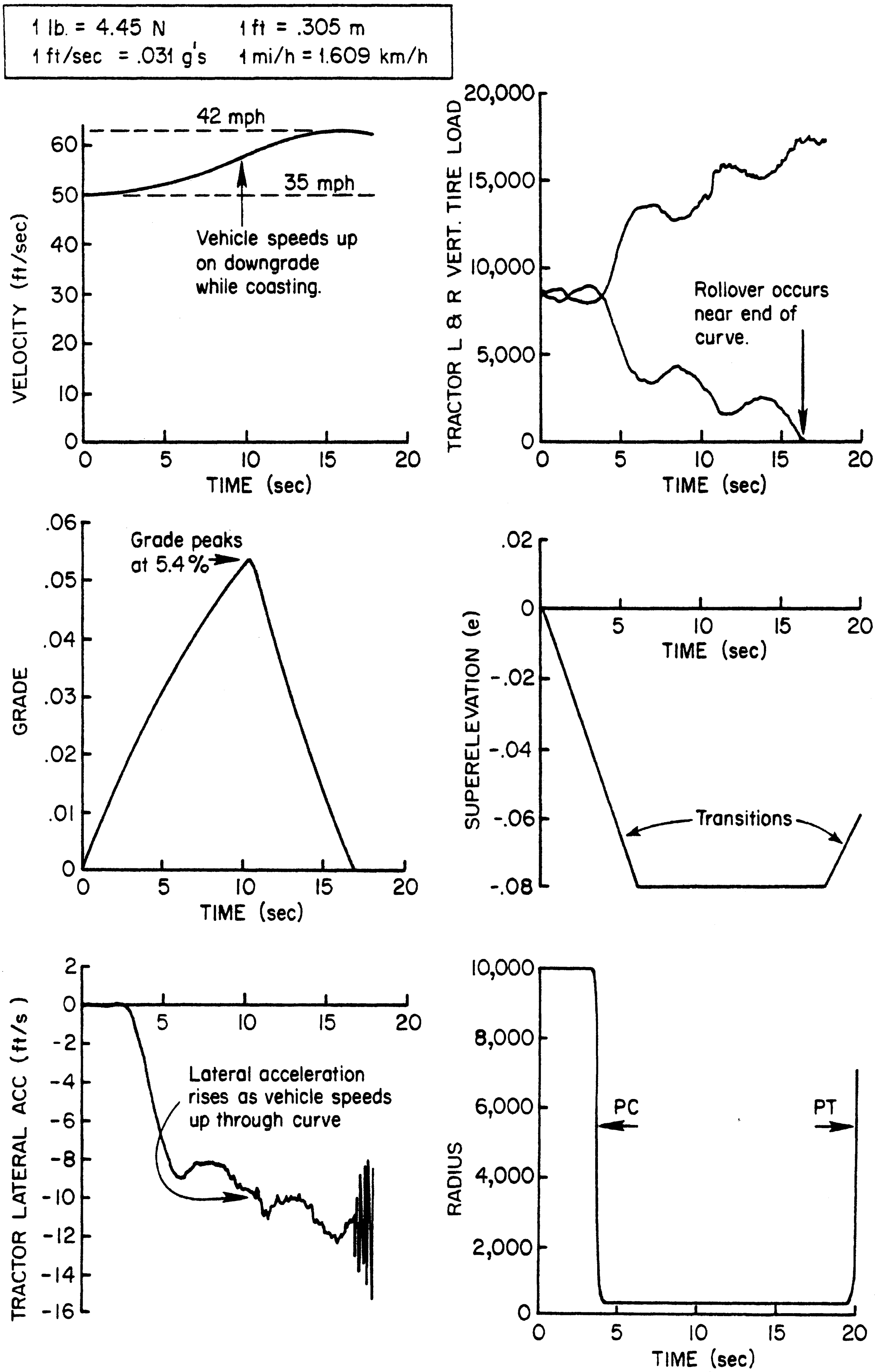


Figure 30. Vehicle response at site no. 8 - high c.g. vehicle, 42 mi/h.

The case illustrates that a relatively long downgrade ramp, coupled with a tight curve, can produce rollovers simply due to the coasting decelerations of inattentive or miscalculating drivers. Again, since this site (and the other downgrade site, no. 7,) provides connection from one freeway to another, the tendency of truckers to speed somewhat in anticipation of the merging task may act synergistically with the downgrade (which provides, after all, a quick means of accelerating the vehicle).

The implications of these observations for geometric design may be that there exists a critical sum of grade, grade length, side friction factor, and the differential between ramp speed and the speed of the through lanes with which merging is expected. That is, firstly, the integral of grade with respect to grade length will determine a potential increase in truck speed which could be attained through coasting. Secondly, the side friction factor of the curve remaining at the bottom of the grade will, together with the minimum truck rollover thresholds, determine the tolerance of the ramp for overspeeding. Thirdly, the differential in speeds may indicate the possible motivation level on the part of truck drivers to speed in anticipation of merging; the closer the ramp speed is to the speed of the through traffic, the less burdensome the merging process and the less inclined the trucker may be to speed.

Case 6 Poor Pavement Friction Level on High-speed Curve

Recent findings [33,34] indicating the potential for hydroplaning with lightly loaded truck tires offer a likely explanation for loss-of-control problems which are seen at certain ramp sites in wet weather. These findings are based upon the observation that at the very light tire loads associated with empty truck combinations the footprint with which a truck tire contacts the pavement is unusually incapable of expelling water. Accordingly, very lightly loaded truck tires are vulnerable to a pronounced traction deficiency on smooth, wet pavements. Since the loss of tire traction on wetted surfaces is clearly most pronounced when speed is high, potentially troublesome ramps are those having large radius curves such as at many "high-design" interchanges between two modern freeways. The applicable scenario leading to loss of control involves an unloaded truck combination, a high-speed turn which also poses a substantial side friction demand, and a poor pavement texture and/or water drainage characteristic.

An example ramp site which was found to provide a dramatic illustration of this phenomenon is illustrated in figure 31. The ramp constitutes a "broken-back" curve, 2,600 ft (793 m) in length, which is comprised of two curve segments of 1,400 ft (427 m) radius, with a 290-ft (88-m) tangent section connecting the two. The entire curved portion of the ramp plus the included tangent section was superelevated at 0.05 ft/ft (0.05 m/m), yielding a side friction factor of 0.05 at the special truck advisory speed of 45 mi/h (72 km/h). The accident evidence suggests, however, that many trucks simply sustain the 55 mi/h (88 km/h) speed which is posted for other vehicles, thus experiencing a side friction factor of 0.09.

Forty-four loss-of-control accidents occurred at this site with tractor-semitrailers over a 2-year period following opening of the new roadway. All 44 accidents occurred when the pavement was wet. Thirty-two of the accidents at this site involved tractor jackknife, five culminated in rollover, and seven involved other events such as simply running off the road or striking a guardrail. The ramp was resurfaced at the end of this 2-year period with a high-friction bituminous concrete overlay, after which the wet weather accident problem essentially disappeared. Although the police-reported accident forms provided no note of vehicle loading, the large number of loss-of-control incidents which involved running off of the road without rollover suggests that many of the semitrailers were lightly loaded or empty.

The fact that so many jackknifed rigs ran off the inside of the roadway suggests that overbraking, and then release, constituted a frequent mode of loss of control (as was described earlier). For such a control sequence, the jackknife divergency is basically uninfluenced by any details concerning ramp geometrics. Braking-induced control loss

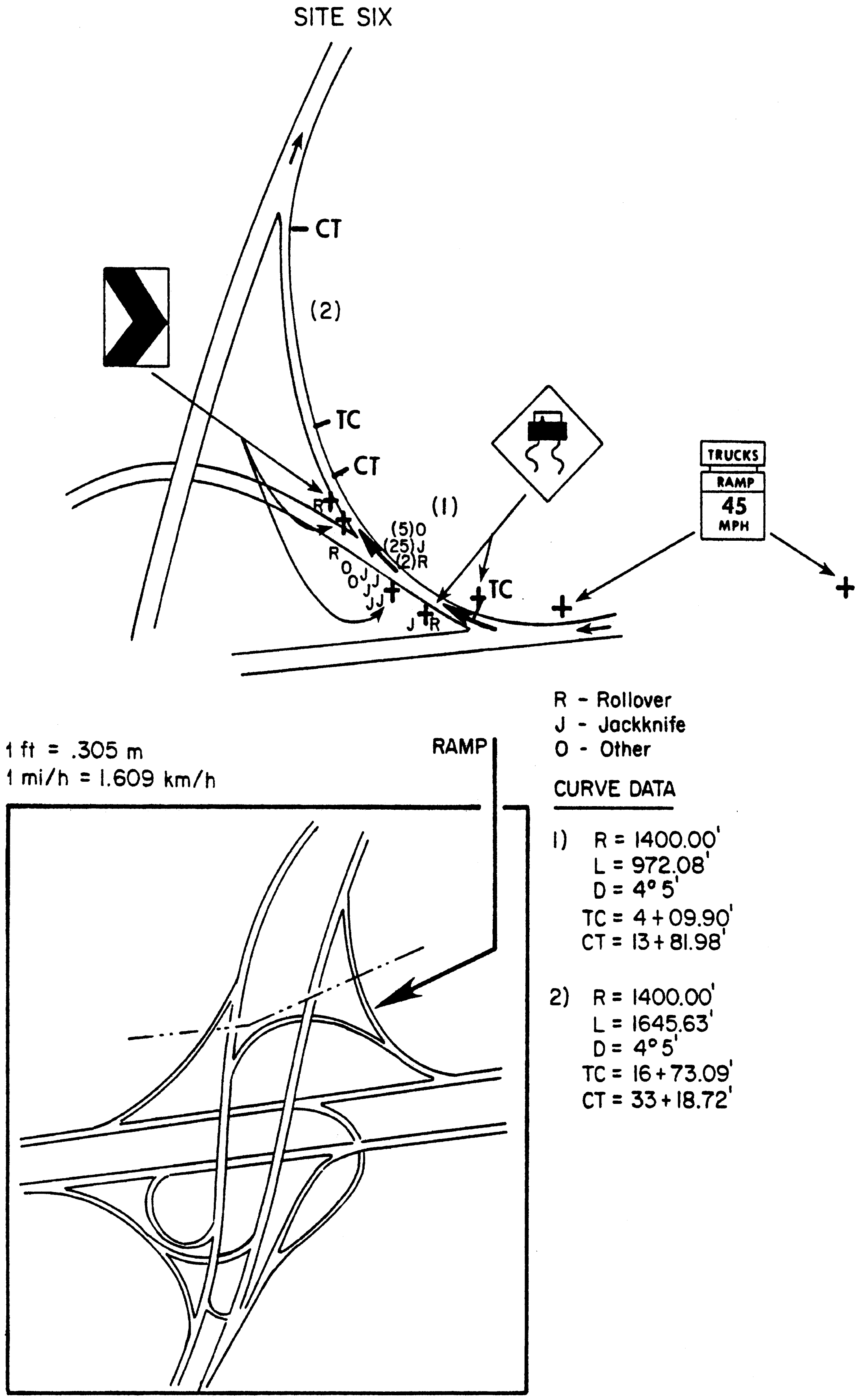


Figure 31. Layout of site no. 6.

simply requires a condition in which a) the driver is prompted to apply brakes more vigorously and b) a reduced tire/pavement friction level prevails at site no. 6. Because of the minimal instructive value in simulations of braking-induced jackknife, computations focused upon another form of jackknife which may also have caused a number of the incidents at this site. Recognizing the recent findings concerning the hydroplaning potential of lightly loaded truck tires, simulations were run looking for the conditions in which the light tractor rear tires would reach a side force saturation while an empty tractor semitrailer simply drove through the ramp at a steady speed of 55 mi/h (88 km/h).

Shown in figure 32 are simulation results representing such a case, with jackknife occurring right as the vehicle passes the short tangent section of the broken-back curve. The conditions producing loss of control in this example involve the assumption of a near-hydroplaning level (mu = 0.12) at the tractor rear and trailer tires, compared to a friction level at the front tires of 0.50. Again, this peculiar distribution of tire/pavement friction levels was rationalized on the basis of large differences in tire load among the respective axles and the corresponding implications for friction, considering the potential for strong hydrodynamic influences [33]. Static loads on front and rear tires were 4,700 lbs (2.1 mg) and 1,300 lbs (0.6 mg), respectively. The simulation results indicate that if the friction levels attain the identified values, the vehicle becomes sufficiently disturbed in traveling over the superelevated tangent portion of the curve that a rapid jackknife divergency is precipitated (upon saturating the lateral force output of the tractor rear tires).

The item of general importance illustrated in this case is that heavy-duty vehicles are now known to be unusual in their potential for loss of control on wetted pavements. It would appear that ramps which impose moderate to large demands for side friction factor while also permitting high-speed travel should be maintained with particular attention to pavement friction level and water drainage in order to safely accommodate lightly loaded truck combinations.

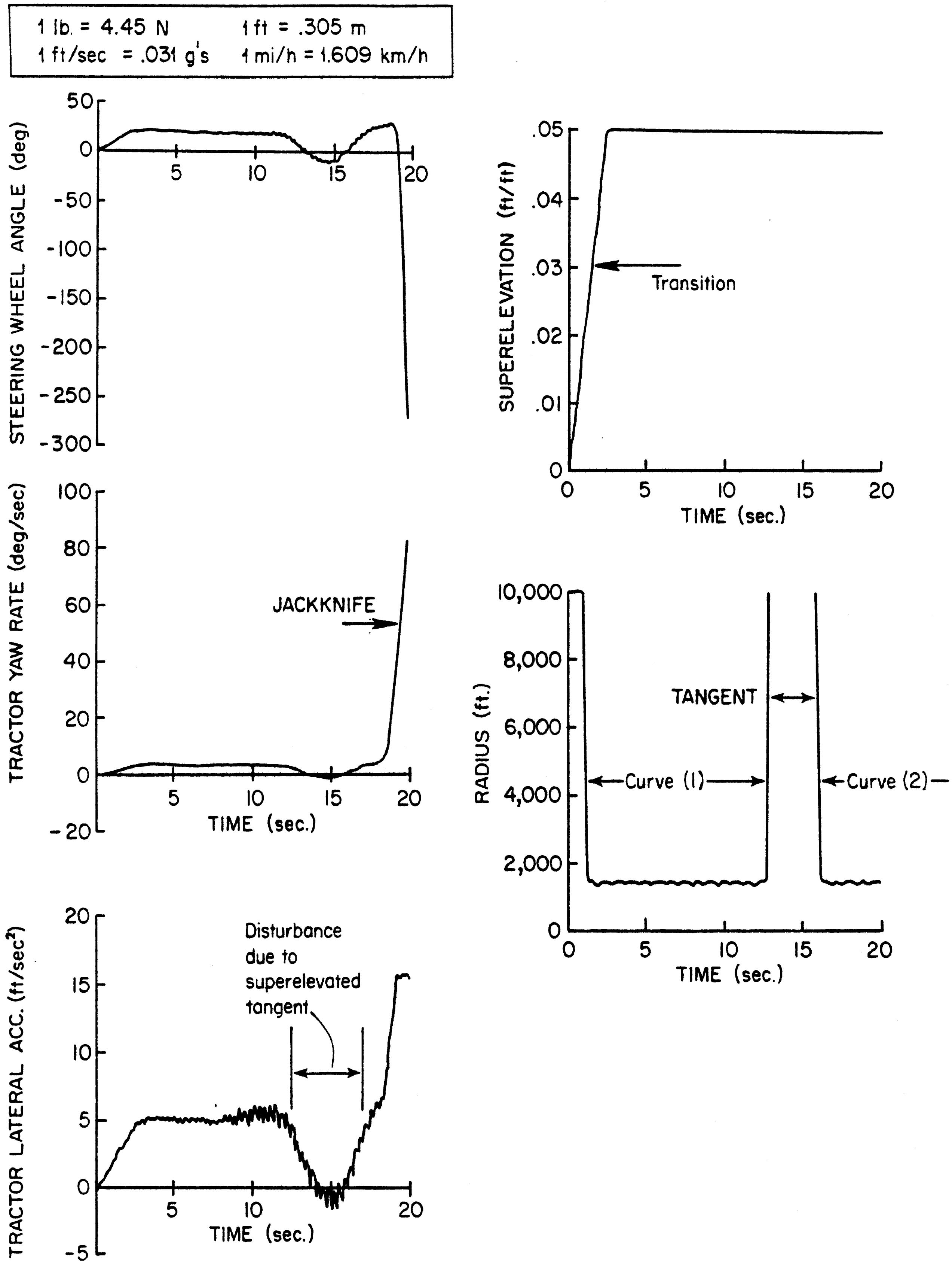


Figure 32. Vehicle response at site no. 6 - empty vehicle, 55 mi/h; tractor rear and trailer tires near hydroplaning (pavement friction, 0.12).

6.0 CONSIDERATIONS OF COUNTERMEASURES

The foregoing discussions and presentation of simulation results have served to suggest that there is a relationship between the geometric design of interchange ramps and the safety of truck operations. In some of the illustrated cases, it has been apparent that trucks have suffered frequent loss-of-control accidents even though the ramp site was nominally in compliance with AASHTO design guidelines. In other cases, the ramp geometric design was clearly in violation of good design practice. In this section of the report, the question of countermeasures will be addressed, considering the approach which might be taken to adjust the recommended practices of highway design to better reflect the needs of trucks and also the particular aspects of design which may be practicably modified at existing sites to solve current problems with truck safety. Again, the discussion will be organized around the six types of problems which were identified in the study.

Although this discussion addresses only those countermeasures pertaining to the highway, it is clear that improvements in truck stability and control properties constitute another realm of potential countermeasures. Indeed, most of the references on truck dynamics research cited in this report contain recommendations for the improvement of truck control qualities. Although enhancement of truck design and operating practices is expected to progress in the years ahead, it is likely that technical progress will proceed rather slowly and that the national trucking fleet will incorporate innovations very cautiously. Thus, it seems safe to suggest that highway-level countermeasures of the type discussed below are likely to be needed for many years to come.

6.1 The Maximum Value of Side Friction Factors

The discussion of the friction factor aspect of ramp design revealed that trucks having lower levels of rollover threshold could experience a virtually zero margin of safety on curves which were constructed according to AASHTO policy. The lack of safety margin was seen to be most pronounced on curves for which superelevation was not fully developed at the point of curvature--the predominant situation prevailing in most curve design in the

U.S. The countermeasure to this problem is, of course, simply to take steps to assure that friction factors are maintained at lower values, both through better transitioning and the reduction of maximum friction factor values sustained along steady curves. In particular, it seems reasonable to treat the friction factor as a continuous function along the roadway such that both transition and curve sections are directly controlled in the design policy. Using such an approach, one would specify the maximum value of peak friction factor which is allowed at any point on the road.

For example, if we consider that the low-stability tractor-semitrailer will roll over at a lateral acceleration level of 0.24 g's, we could formulate a constraint on peak side friction factor, f_{peak}, which also incorporates a safety margin value, SM, of lateral acceleration as follows:

$$(1.15) \times f_{peak} + SM = 0.24 \text{ g's}$$

This expression includes the factor, (1.15), which accounts for steering fluctuations, as discussed earlier. The safety margin, SM, is defined so that the tolerance remaining for truck operation within a 0.24 g rollover threshold is expressed directly. Arbitrarily setting the safety margin at SM = 0.10 g's would cover the contingency of a truck running at 40 mi/h (64 km/h), for example, on a ramp which is designed for 30 mi/h (48 km/h). Shown in table 9 are values for

- superelevation, (e)
- the maximum (f) values which AASHTO recommends for 30 mi/h (48 km/h)
- the (f_{peak}) value which derives if superelevation was developed to 50 percent of its full value at the point of curvature
- 1.15 x (f_{peak}) allowing for steering fluctuations
- the safety margin remaining between the (f_{peak}) value and a 0.24 g rollover threshold
- the maximum (f) value which might be adopted in order to achieve a safety margin of 0.10 g's

Table 9. Characteristics of side friction demand on curves having 50 percent superelevlation developed at the point of curvature

e	AASHTO Max f at 30 mi/h	Peak f with 50% (e) at PC	1.15xPeak (Allowing for Fluctuation)	Safety Margin for .24 g Truck (g's)	Max f Assuring SM = 0.1 g
.10	.16	.21	.24	0	.06
.08	.16	.20	.23	.01	.07
.06	.16	.19	.22	.02	.08
.04	.16	.18	.21	.03	.09

The table illustrates that safety margin values are zero or near-zero when AASHTO guidelines are followed and the peak values of side friction factor are considered. The "countermeasure," then, for the clear safety hazard associated with near-zero safety margins is to adopt maximum (f) values, as shown in the column at the far right. Although, in many cases, it is quite likely that the allowance of only a 0.07 friction factor with a superelevation level of 0.08, for example, would require prohibitively low ramp speeds and/or large curve radii, it would seem that roadways carrying very heavy volumes of truck traffic might warrant such treatment.

If roadways were to be constructed with spiral transitions, such that the 50 percent development assumption did not apply, the maximum (f) value could be boosted considerably, with full superelevation being achieved at the point of curvature. Again, providing for a safety margin of 0.10 g's, a maximum (f) value of 0.12 could be allowed on properly spiraled transitions. Further, since the limit condition of interest, namely, rollover, is not a function of speed, per se, the suggested limits on maximum (f) would apply for any design speed.

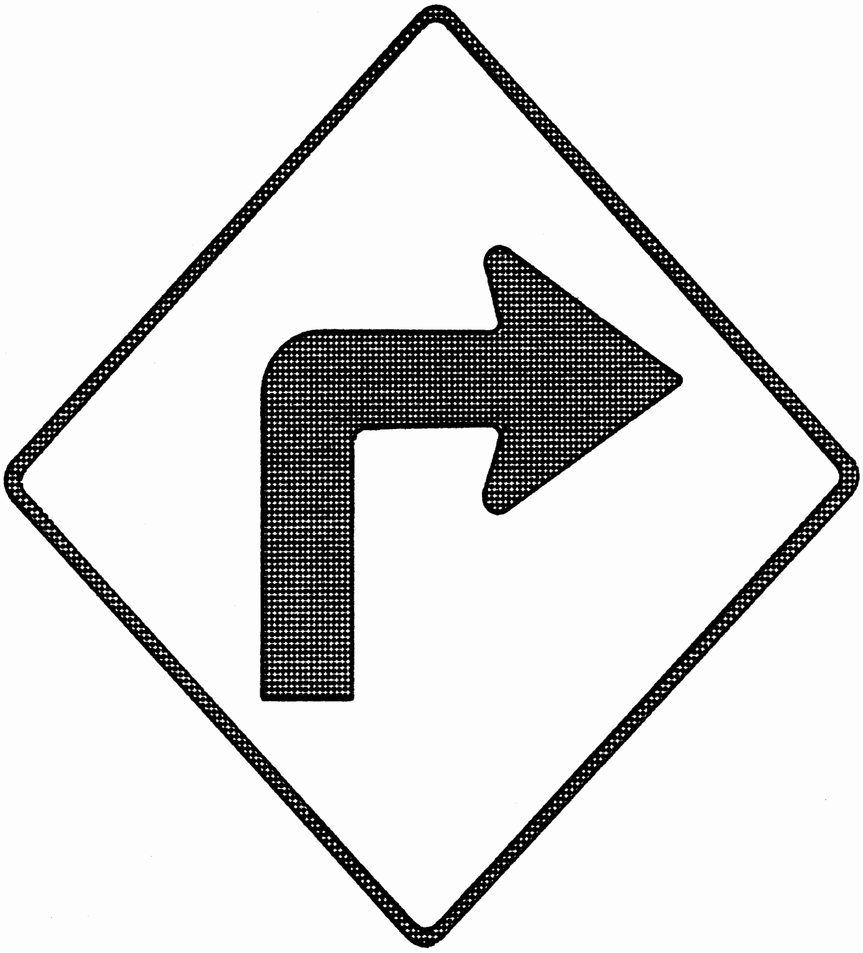
Beyond the prospect of designs which simply reduce the side friction demand in recognition of truck stability levels, the study also identified signing countermeasures which can be taken to reduce the accident probability at existing curves having frequent truck rollovers.

The first signing issue, of course, simply pertains to the advisory speed. At the most basic level, the question of the advisory value, per se, must be viewed as the first-line countermeasure, noting that the side friction factor is related to the square of the travel speed through the curve. Indeed, to say that side friction factors are too high is simply to say that the advisory speeds are too high. At the next level, one can consider special speed advisory signs for trucks, although we recognize that a greater potential exists for confusion when two values of speed advisory are present.

Together with the attention to speed advisory signs, there are associated conventions pertaining to the selection of curve warning signs. In particular, for ramps which are posted with advisories of 30 mi/h (56 km/h) and below, the proper warning sign identified in the Manual on Uniform Traffic Control Devices (MUTCD) is the "Turn" sign, no. (W1-1R or 1L), as shown in Figure 33. As pointed out in the detailed review of each site in appendix A, a number of problem sites did not incorporate curve warning signs of this type. Since the Turn sign tends to imply a sharper turn layout, this type of warning sign may well serve to emphasize the speed advisory signage.

The MUTCD also indicates that the Turn sign may be supplemented by the "Large Arrow" sign (W1-6), as well as an "Advisory Speed" plate (W13-1) affixed to the Turn sign. (Both of these latter signs were seen employed along the compound curve at site no. 2; see figure 22). At another level of treatment, the "Chevron Alignment" sign (W1-8) (such as seen outlining the curve sign at site no. 12, appendix A) may be used as a further supplement, or alternative, to the Large Arrow sign.

An additional aspect of the warning sign issue is the placement of the sign relative to the point of curvature. As pointed out in appendix A, a number of warning signs were seen to be placed closer to the point of curvature than is recommended in the MUTCD. Clearly, the placement of warning signs at a proper distance in advance of a challenging curve will provide more timely advice to the driver on the need to regulate speed.



W1 − 1R
30" x 30"

Figure 33. "Turn" sign recommended in the MUTCD for curves with advisory speeds of 30 mi/h (48 km/h) or less.

Beyond the realm of standard signing, the research team became aware that various States have designed special warning signs for use at ramp sites having a high incidence of truck accidents. While none of these signs are recognized in the MUTCD, some were claimed to be very effective as accident countermeasures. One sign which the State of California has used with success on problem ramps having sharp curves following a downgrade is shown in figure 34. The sign shows a truck which is in the process of rolling over, and posts an Advisory Speed plate. The sign is configured in a large square shape (in contrast to the diamond shape of all standard warning signs) and is posted partway along the downgrade section to warn against a hazardous speed increase. California's experience is that the sign has contributed to a wholesale reduction in the truck rollover rate at certain sites.

6.2 Compound Curves on Interchange Ramps

The problem which compound curves place upon the safety of truck operations is not unique to trucks. Further, since the highway design community has long recognized the problem posed by dramatically discontinuous curve designs, there is little that can be said peculiarly in behalf of trucks. The only additional insight which enters the picture with regard to trucks pertains to the strong motivation which truckers may have toward speeding on a central, flat portion of the ramp in order to facilitate merging with high-speed traffic. Existing curves having tight-flat-tight curve sequences seem to call for special signing treatment in order to alert trucks of the impending curve demands toward the end of the ramp. In this regard, the Turn sign with Advisory Speed plate can be installed in the flat section of the ramp to provide the driver with an "updated" warning that a sharp curve still remains ahead on the ramp. As mentioned above, the final sharp curve can also be highlighted with Large Arrow and Chevron Alignment signs as supplements.

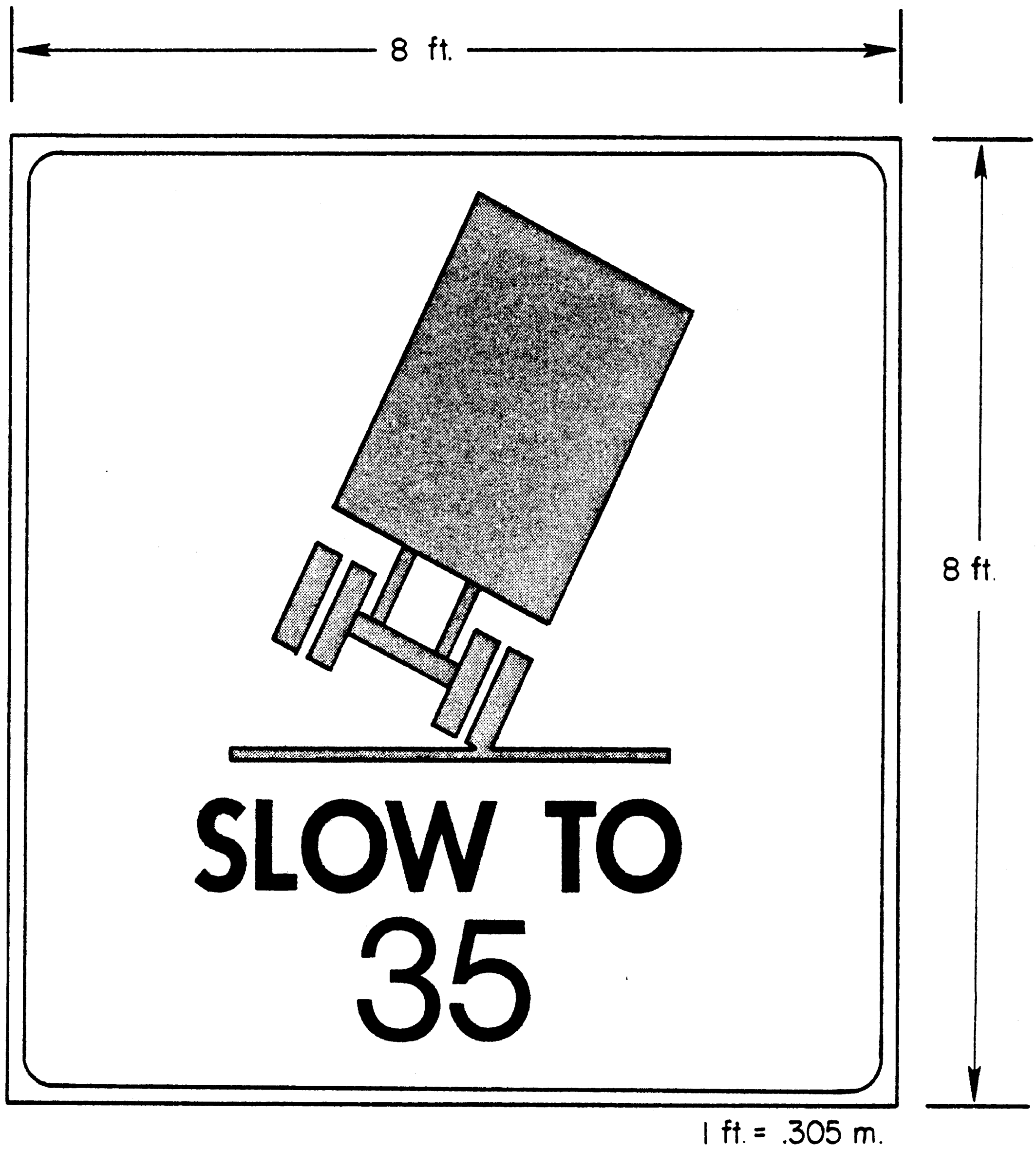


Figure 34. Warning sign used in California at ramps having history of truck rollovers.

6.3 Length of Deceleration Lanes

It was established that because of poor stopping capability, trucks are sorely pressed to attain the needed reductions in speed on the provided deceleration lanes which lead to tight-radius curves. At the root of the conflict is the AASHTO design assumption that a deceleration level of 0.28 g's can be accommodated comfortably at 70 mi/h (113 km/h), tapering down to 0.18 g's at 30 mi/h (48 km/h). Accordingly, the AASHTO guideline for length of deceleration lanes assumes that passenger cars are, first, coasting in gear for 3 seconds at a nominal deceleration rate of approximately 0.11 g's, following which the vehicle brakes at 0.28 g's to 0.18 g's as speed reduces. With trucks, however, we saw that braking efficiencies as low as 50 percent would render the maximum braking capability of only 0.25 g's for a vehicle decelerating on a surface having a peak friction potential of 0.5. Further, the deceleration level experienced by heavy trucks which are coasting in gear is considerably lower than the value used in the AASHTO calculation. Published data show that loaded truck combinations exhibit a coasting deceleration level (in gear) which is more on the order of 0.03 g's over the speed range from 30 mi/h to 60 mi/h (48 km/h to 97 km/h) [30].

Considering, again, that some safety margin should be provided to assure that trucks can, indeed, meet the deceleration requirements without wheel locking, it seems reasonable to assume a braking level of 0.15 g's (thus providing a 0.10 g safety margin below the nominal limit value of 0.25 g's). For a corresponding design formula, then, in which the truck coasts in gear for 3 seconds at 0.03 g's and then brakes at 0.15 g's, example results are shown in table 10 in terms of the length components of a deceleration lane.

Table 10. Elements determining length of deceleration lane for trucks.

1 mi/h = 1.6 km/h

1 ft = 0.3 m

Through Lane		Deceleration Lane				
Hwy. Design Speed, V (mi/h)	Ave. Running Speed, Va (mi/h)	3-Sec. Coasting Distance (ft)	Braking Dist. (ft) to Achieve Average Running Speed on Exit Curve		Total Decel. Length (ft) to Achieve Average Running Speed on Exit Curve	
			30 mi/h	40 mi/h	30 mi/h	40 mi/h
50	44	189	192	37	381	226
60	52	224	356	201	580	425
65	55	237	423	269	660	506
70	58	250	497	342	747	592

The table shows data using reference initial speed values as in the AASHTO Geen Book, but only for average running speeds on the exit curve of 30 mi/h and 40 mi/hh (48 km/h and 64 km/h). Column 3 lists the portion of the deceleration lane length which must be provided for coasting in gear. Columns 4 and 5 list the lengths needed for braking. The total needed length of deceleration lane, then, is the sum of the respective value in column 3 plus either column 4 or 5. For example, the total length of deceleration lane pertaining to a "Hwy. Design Speed" of 50 mi/h (80 km/h) and an "Ave. Running Speed" of 30 mi/h (48 km/h) on the exit curve is 381 ft (116 m). The 381 ft (116 m) value derives from the sum of 189 ft (58 m) as a coasting distance plus 192 ft (59 m) as a braking distance.

We see that the deceleration lane lengths required for trucks are 30 percent to 50 percent longer than those which appear in the AASHTO Green Book. Moreover, one can conclude that the realistic demands which trucks place upon length of deceleration lane are substantially longer than those imposed by passenger cars. It would seem rational that the sites which most warrant such lengths for deceleration lane are those in which (a) truck traffic volume is heavy, and (b) a tight curve is encountered early in the ramp, such that side friction factors are also demanding.

The problem of short deceleration lanes leading to tight-radius curves should also benefit from advance placement of warning signs. Both Advisory Speed and Curve Warning signs can be used at the beginning of the deceleration lane, or in advance of the lane when properly designated as exit advisories.

6.4 Curbs Placed Upon the Outside of Ramp Curves

The occurrence of truck rollover following the striking of a curb on the outside of a curve may not necessarily implicate the curb as a culpable element. Nevertheless, because curbs which are placed within a foot or two (.3 to .6 m) of the traveled way do present a tripping mechanism, they are seen as categorically undesirable at curved sites where the probability of truck rollover is relatively high simply because of the side friction value. Indeed, it would be straightforward to illustrate a range of vehicle paths which would be "survivable" without a curb at a given site, but which would induce rollover if a curb were placed close to the lane edge. Beyond this general rationale for eliminating curbs on curved ramps, however, the outboard offtracking of articulated vehicle combinations is seen as a special problem because it is apparent that truck drivers are broadly ignorant of this phenomenon and yet it has the potential for producing as much as 2.5 ft to 3 ft (.8 m to .9 m) of outboard offset in wheel paths. Accordingly, one approach toward a countermeasure strategy is to remove the curb from the outside of those lanes in which a substantial potential for outboard offtracking may occur.

Recognizing that the potential for outboard offtracking is determined peculiarly by the curve radius, it is straightforward to solve for the value of curve radius above which high-speed offtracking may pose a threat. The offtracking process is such that the trailers track inboard at low speeds, with greater values of inboard offset of wheel path for smaller radii. As lateral acceleration builds up, the tires then track increasingly more outboard until they "cross over" the zero offtracking condition and subtend paths which are net outboard.

The total offtracking, OT, exhibited for a given lateral acceleration value, A_y/g, is

$$OT = OT_{\text{zero speed}} - K\,(A_y/g)$$

where: the zero speed value, $OT_{\text{zero speed}}$, derives from the path radius, R, and various length parameters describing the vehicle (see, e.g., [35]).

K constitutes a high-speed offtracking gradient determined by the length parameters and the cornering compliance of the tires at each axle. (Cornering compliance expresses the rate of tire slip angle developed per unit of lateral acceleration, radians per g (see, e.g., reference 24).)

Since the extent of the zero speed, inboard, offtracking is inversely related to the radius of the turn, the ramp curves which are most prone to problems with curb contact through outboard offtracking are those which have curve radii above a certain value. Further, the magnitude of this "minimum radius" value is dependent upon the configuration of the vehicle which is involved. Shown in figure 35, for example, is an illustrative plot of the offtracking responses of a typical tractor-semitrailer and a doubles combination on a 500-ft (152-m) radius curve. We see that, at zero speed, both vehicles track inboard, but the double which tracked less inboard at zero speed is the first to cross over toward the outside. If one were to consider a constraint on the use of curbs on exit ramp curves, it is clear that the double shown here would constitute a more serious consideration than would the tractor-semitrailer.

The high-speed gradient shown in the figure can be determined from elemental wheelbase dimensions and tire stiffnesses in each vehicle configuration. For example, the conventional tractor with 48 ft (14.6 m) semitrailer equipped with bias-ply tires, will offtrack increasingly outboard at a rate of approximately 11.2 ft/g (3.4 m/g) while the corresponding double with 28 ft (8.5 m) trailers and bias-ply tires would exhibit a gradient of 12.5 ft/g (3.8 m/g). Knowing the vehicle properties determining the zero-speed offtracking as a function of curve radius (e.g., see reference 35) and

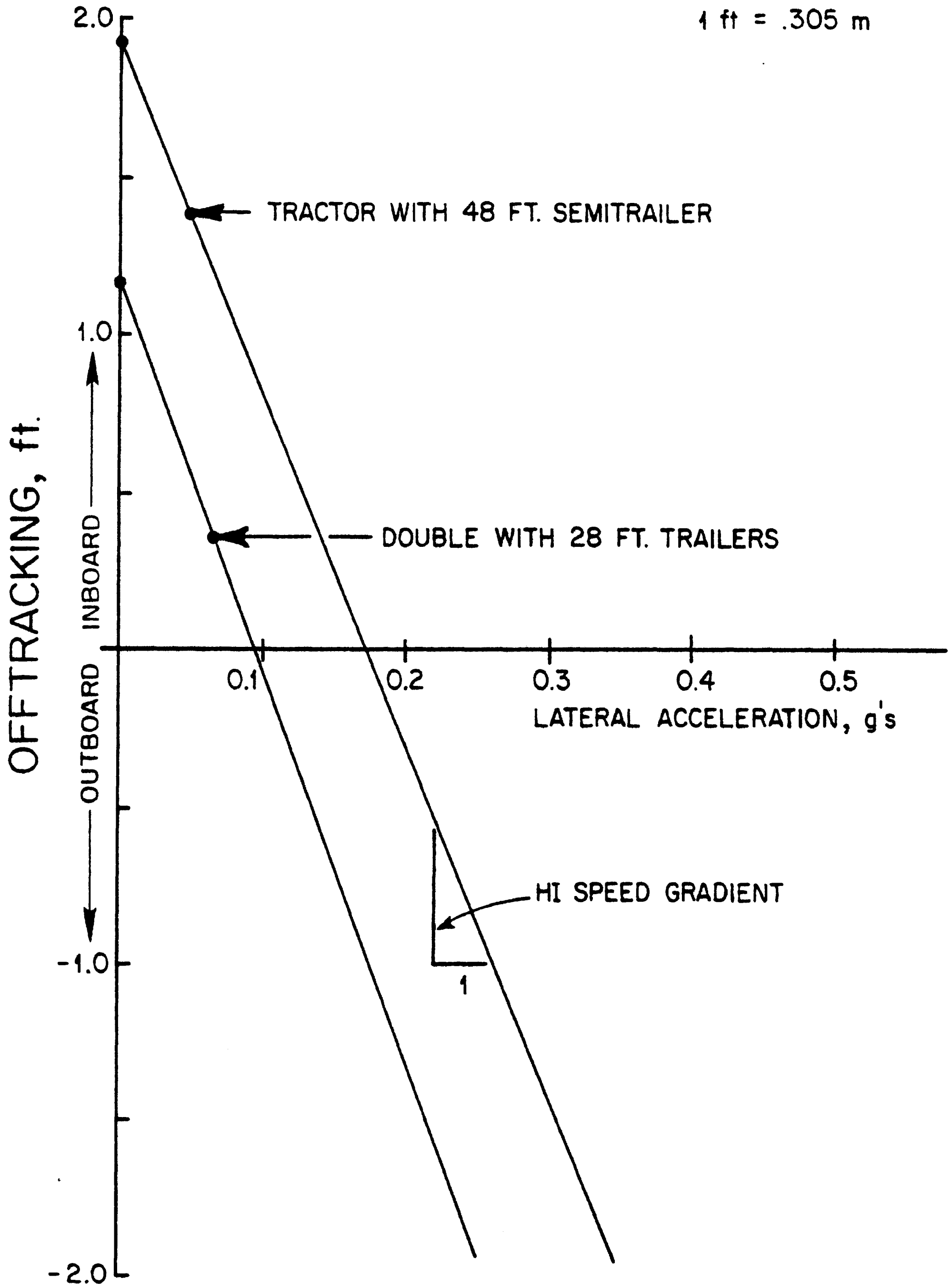


Figure 35. Example offtracking behavior of tractor-semitrailer and doubles combination on 500-ft (152-m) radius curve.

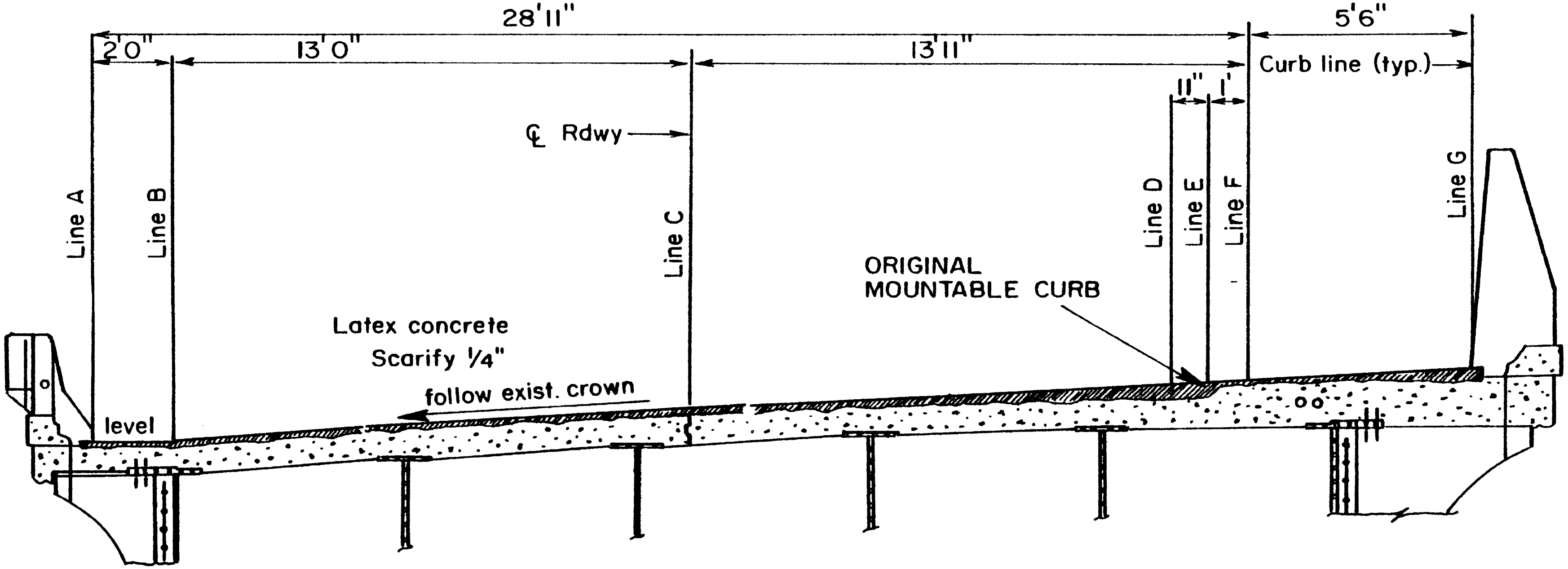
the values of high-speed gradient for each vehicle, it is possible to solve for the value of path radius above which outboard offtracking will exceed a selected value. If we stipulate, for example, that we wish to eliminate curbs from any curve on which a tractor-semitrailer or doubles combination might achieve a net outboard offtracking of 1 ft (0.3 m), while traveling at a limit lateral acceleration level of 0.3 g's, we find that

Maximum Radius Permitting Curbs = 370 ft (113 m) for tractor-semi

= 220 ft (67 m) for doubles combination

While the 1 ft (0.3 m) and 0.3 g stipulations are nominal selections which illustrate a countermeasure guideline, they also appear reasonable as values establishing some real potential for curb contact and trip-induced rollovers. Thus, it is suggested that for radii exceeding 370 ft (113 m) for tractor-semitrailers and 220 ft (67 m) for doubles, the extent of outboard offtracking arising at the 0.3 g condition will exceed 1 ft (0.3 m) such that a substantial threat from curb strike may prevail. For smaller radii, the inboard offtracking excursion is less than 1 ft (0.3 m) except at lateral acceleration levels exceeding 0.3 g. Since most loaded trucks exhibit rollover threshold values ranging from .24 g to, say, .42 g's, the example value of 0.3 g is rationalized as a guideline for the sake of protecting against the trip-induced rollover of the larger portion of the truck spectrum which would otherwise "survive" a smooth steady turn of that severity. For trucks having rollover thresholds below 0.3 g, of course, the example guideline is moot since those vehicles will roll over due to static phenomena before they exhibit the extent of outboard offtracking meeting this criterion.

Considering that the removal of certain existing curbs may be warranted, one such modification which was effectively implemented at certain ramps in the State of Michigan is sketched in figure 36. The existing ramp pavement was overlayed with a latex concrete such that the outer curb was eliminated and a smooth road plane provided, from shoulder edge to shoulder edge. The overlay also provided a means for wedging in additional superelevation, thereby reducing the side friction factor, as well. Such an overlay, without the need for compensating drainage treatments, emphasizes the likely general



OVERLAY THICKNESS (IN)

Line	A	B	C	D	E	F	G
Thickness	1.75	1.75	1.75	6.75	7.25	2.12	7.12

Figure 36. Overlay treatment employed by Michigan DOT spanning curb on outside of curve.

case--namely, that curbs existing on the "high side" of superelevated curves do not typically play a useful role with regard to drainage.

6.5 Downgrade Leading to Sharp Curve

Because the combination of parasitic drag and engine braking is quite low on loaded heavy trucks, a distinct potential for speed increase exists on even the relatively short lengths of downgrade which may exist on an interchange ramp. If the truck driver is inattentive, or otherwise inclined to accept a speed increase as an aid toward accomplishing the upcoming merging task, the grade can promote a hazardous condition in a subsequent curved portion of the ramp which may impose an excessive side friction demand at the elevated speed. The AASHTO Green Book suggests that downgrades having gradients as high as 8 percent do not cause hazard due to excessive acceleration. With trucks capable of only some 0.03 g's of coasting deceleration, however, an 8 percent grade which is, say, 500 ft (152 m) in length produces a speed increase from 30 mi/h to 40 mi/h (48 km/h to 64 km/h)--an increase which was seen in example cases to easily put many trucks in risk of rollover. Evaluation of the potential for speed increases on an individual site can be done using values for the average grade, G, and the length of grade, L. The acceleration from an initial speed, V_o, to a final speed, V_f, due to coasting down the grade with an inherent drag equal to 0.03 g's of deceleration is given by:

$$V_f = \sqrt{V_o^2 + L(G - .03)/.033}$$

where

L is in ft (m/3.28)

G is nondimensional, ft/ft

V_o and V_f are in mi/h ((km/h)(.62))

Although the driver can avoid this speed increase simply by applying sufficient braking to hold speed in check, the classification of sites using this formula can serve to distinguish potential problem ramps from those which have no such adverse potential. Relative to the "no adverse potential" case, the AASHTO Blue Book included a recommendation that gradients on ramp downgrades be limited to 3 percent or 4 percent where "an effective proportion of the ramp traffic consists of heavy trucks or buses." Clearly, the above formula indicates that a grade of 3 percent (.03 ft/ft) yields a zero speed increase for the nominal loaded truck. Thus, the Blue Book recommendation would appear to be much more suitable than the 8 percent allowance of the Green Book for controlling the kind of speed-increase problem which appeared to have caused numerous rollovers in the cases studied.

With regard to signing, MUTCD recommendations would nominally rule out usage of "Hill" (W7-1) or "Grade" (W7-3) warning signs on the short lengths of downgrade typically occurring on ramps. While curve warning and speed advisory signs certainly have a proper role, as discussed in the context of ramp curves, generally, the peculiar nature of the downgrade-and-curve ramp problem may well call for special signing directed at trucks such as noted earlier.

6.6 Reduced Pavement Friction on Curves

The selected case which was seen to involve numerous truck loss-of-control incidents in wet weather does not, by itself, implicate geometric design nor does it introduce a concern which is new to the highway engineering community. Clearly, the issue of pavement friction level has been a central concern in highway maintenance for many years. Perhaps the only countermeasure kind of rationale that can be applied to this subject is one of a more focused surveillance, recognizing that empty truck combinations experience a special hazard when running at high speed. The evidence suggests that the extent of this hazard exceeds that experienced with passenger cars. Nevertheless, the ability of conventional ASTM Skid Number measurements to adequately characterize the frictional condition which will be critical for trucks has not yet been demonstrated.

The current situation is that empty trucks can be expected to have control problems on high-speed facilities having deficient pavement friction quality, especially when substantial levels of side friction demand are imposed by the geometry--but, no clear guideline can be proposed for that critical value of ASTM Skid Number which reflects the truck's special sensitivities.

An additional complicating factor relating to concern over friction level on curved ramps, of course, relates to the fact that conventional ASTM E-274 Skid Number measurements are not made on the curved roadway, itself, due to the unstable articulation of the skid trailer during wheel lockup. To the extent that the pavement on a curved ramp may be of different materials or maintenance condition than the through lanes, skid number measurements made on the through lanes may not represent the friction conditions prevailing on the ramp. If it was suspected that a ramp having wet skidding accidents with trucks differed in friction quality from that of the through lanes, it might be advisable to employ an independent measurement of texture depth or some other surrogate means of estimating friction level on the ramp.

Alternatively, a recent FHWA research study has examined the feasibility of using a conventional ASTM Skid Trailer, with only one wheel braking, for conducting measurements on curved roadways [37]. Since the study finds that useful results can be obtained, given certain variations in technique, States might find it attractive to adopt this method for the characterization of ramp skid number where trucks have had frequent loss-of-control accidents on wet pavement.

7.0 CONCLUSIONS AND RECOMMENDATIONS

Most other studies which have examined the relationship between geometric design features and vehicle safety have proceeded from a statistical base of data. The safety significance of a given level or type of geometric feature was then determined through statistical inference, given the confidence limits of the analysis. Such approaches provide the basis for conclusions which are as general as the data base is representative. In the current study, a broad base of accident data was used only as a guide to the selection of a few individual sites. Thus, conclusions deriving from the anecdotal level of study of these sites cannot claim generality in any statistically supportable sense. Rather, the conclusions of this study have general significance only to the extent that (a) ramp design is related to the AASHTO design guidelines, which are intended to guide the limits of general design practice, and (b) safe truck operation was judged upon the basis of example vehicles whose relationship to the broad spectrum of commercial vehicles in the U.S. is (arguably) known.

In this sense, findings can be made which speak to the suitability of the AASHTO design policy given the physics of truck response and the authenticity of the example vehicles which were modeled in the simulation study. Such findings are "theoretical" except insofar as a limited number of accident cases have been employed in discovering and demonstrating the findings. Moreover, the conclusions of this study reflect the outcome of an analytical process in which computed vehicle behavior seems to confirm limited samples of accident experience. Given that rather little past research has addressed the question of truck safety versus highway design, the results may serve to alert the highway engineering community to the prospect that certain design practices may dramatically compromise the safety of truck operation on ramps.

The results of the study are expressed below, in the form of conclusions which are seen as having general importance and recommendations for future improvement of truck safety on interchange ramps.

7.1 Conclusions

Based upon the accident data study, evaluation of the geometric design of ramps which were found to be heavily involved in truck accidents, the simulation of truck performance on such ramps, and the consideration of accident countermeasures, the following can be concluded.

1) Truck loss-of-control accidents on interchange ramps are predominantly by rollover and jackknife events. The relatively "clean" character of the roadside on high-design highways results in rather few collisions with fixed objects off-road. On the other hand, because loaded trucks cannot generally penetrate far onto side slopes which are inclined at 6:1 or steeper without rolling over, many run-off-road incidents result in rollover.

2) Jackknife accidents predominate at sites where inadequate pavement friction levels prevail during wet weather. The loss-of-control mechanism can entail either a simple deficiency of lateral traction on curves due to near hydroplaning at the tractor drive wheels of empty vehicles or due to light braking which causes lockup of tractor drive wheels. Jackknife accidents are also found ahead of curves which appear to pose a threat of rollover to vehicles traveling near or above the advisory speed. Apparently, truck drivers apply excessive braking in an attempt to reduce speed before entering the curve, suffering the wheel lockup conditions causing jackknife before the curve is reached.

3) Rollover accidents are precipitated at sites having high levels of side friction demand, particularly if (a) superelevation is largely undeveloped at the point of curvature, (b) a curb is installed on the outside of the curve, close to the edge of the traveled way, (c) a relatively demanding curve is placed at the bottom of a substantial downgrade, (d) the curve appears early in a ramp which is preceded by a short deceleration lane, or (e) the curve is placed late in a compound curve which entails a sharp-flat-sharp sequence of curve radii.

4) The AASHTO policy for the geometric design of curves provides for virtually no margin of safety against rollover for certain trucks. This deficiency in the design policy is so startling that one can only assume that

the highway engineering community has simply not had data available showing the very low stability limits of commercial vehicles. The trucks of critical interest lie at the low end of the roll stability spectrum, primarily as a result of high payload centers of gravity, but exist in substantial numbers. Curves designed to suitably accommodate such trucks would have side friction factor values limited to approximately 50 percent of the current AASHTO-prescribed limits.

5) The AASHTO policy for the length of deceleration lanes does not provide for the deceleration of truck combinations in a manner analogous to the treatment for passenger cars. For trucks to decelerate safely within the AASHTO-prescribed lengths, the vehicle must apply service brakes over the full length of the deceleration lane--rather than being allowed an initial 3-second period for coasting in gear upon entering the lane, as is assumed in the AASHTO calculations. Deceleration lanes which would realistically reflect the braking constraints of trucks would be 30 percent to 50 percent longer than AASHTO guidelines suggest.

6) The tremendous mismatch between the provided lengths of acceleration lanes and the acceleration length demands of loaded trucks may be prompting the truck driver to speed in the later portions of many interchange ramps in order to mitigate the inevitable conflicts associated with merging. For ramps which entail a final sharp curve before the exit terminal, the increased-speed strategy threatens loss-of-control in this curve.

7) The AASHTO policy of accepting ramp downgrades as high as 8 percent may be ill-advised at sites on which a relatively sharp curve remains to be negotiated toward the bottom of the grade. The low values of parasitic and engine drag which exist relative to the total weight of the loaded truck combinations render a substantial acceleration capability while coasting in gear on downgrades. Thus, trucks are peculiarly capable of substantial speed increases, even on relatively short downgrade sections, unless the truck driver applies braking to keep speed in check.

8) Curve warning signs were observed to be improperly selected or, in certain cases, placed an insufficient distance ahead of the curve, considering the guidelines of the Manual on Uniform Traffic Control Devices [36]. Given

the limited capabilities of trucks to either withstand an excessive speed condition without rollover, or to safely achieve the braking levels needed to reduce speed in advance of a curve, the inadequate use of warning signs will constitute a more critical deficiency for truck operations than for cars.

7.2 Recommendations

Steps are recommended for implementation of the study findings in behalf of improved truck safety, in the short term, and more truck-cognizant highway design policies in the longer term.

1) Professional bodies concerned with geometric design should review the results of this study to determine the extent to which either direct adjustment of design limits, or at least discussion of the special concerns regarding trucks, are warranted in future revisions of AASHTO design policy. It seems reasonable to suggest that new highways intended to carry large volumes of truck traffic should be designed, if practicable, according to constraints such as outlined in Section 6, on countermeasures, rather than as permitted within current AASHTO policy. More generally, those involved in geometric design may also benefit simply from gaining a better view of the performance limits which can be expected of heavy-duty trucks.

2) A broad sampling of interchange ramps should be examined to determine the actual prevalence of design features such as those which were implicated here as being potentially troublesome for heavy trucks. The "examination" should include evaluation of:

- continuous side friction factors through all curves on each sampled ramp,
- the sequence and magnitude of radius changes along compound curves,
- length of deceleration lanes and relationship to sharp curves,
- downgrade slope and length prior to sharp curves,
- curbs placed close to the outside edge of curved lanes,

- information implicating low friction levels on high-speed ramp curves, and

- signing practices, given the recommendations of the MUTCD and the special considerations which trucks may need to achieve adequate warning.

This recommendation springs from a concern on the part of the research team that the designs found at the selected sites may be highly unusual. If such is the case, the technical community may still press to have the design policy revised in behalf of new highway construction, but no major countermeasure program may be needed for dealing with existing sites. On the other hand, if the design features implicated in this study are found to represent a substantial fraction of existing ramps, then a vigorous countermeasure activity may be warranted.

3) Should such a survey study determine that the occurrence of slim safety margins for trucks constitutes a broad national problem, efforts should be mounted at the national level to (a) evaluate and recommend countermeasures which may bbe implemented at existing sites and (b) develop a major program for guiding the application of such countermeasures and for encouraging their implementation around the country. The practical focus of the countermeasure evaluation program will likely be on signage. The effectiveness of both standard and special signing should be examined, particularly with the benefit of experience gained at the State level on the use of special warning signs for trucks.

4) To whatever extent the survey determines that ramps having "problem" design features exist, elements within the trucking industry, as well as Federal agencies concerned with trucking safety, should take steps to inform truck drivers on the safety precautions which are warranted. If, for example, many interchange ramps provide a near-zero margin of safety for trucks lying in the region of 0.24 g rollover threshold, a vigorous program of public information may be needed to advise truck drivers that the posted advisory

speeds can be patently misleading[1]. Since rollover is often fatal to the truck driver, the seriousness of the advisory speed matter reaches the level of a moral imperative.

In considering methods for communicating such safety messages to the truck driver, it is recommended that the current safety film initiative being mounted by a joint effort of the Motor Vehicle Manufacturers Association plus the trucking industry, drivers' union, and Federal safety agencies be monitored. If this distribution mechanism proves effective, additional safety films such as "What Truck Drivers Need to Know about Freeway Ramps" should be produced. It may also be useful to consider an accompanying film on "What Truck Drivers Need to Know about Highway Signs." Particularly if special truck warning signs are developed for broad national usage, it might be highly desirable to reinforce the signs' effectiveness with an explanatory message to drivers.

4) State departments of transportation would be well advised to initiate projects reviewing ramp sites having a history of accidents with heavy-duty trucks, in light of the findings of this study. Since many States contacted during the accident-data phase of this study showed the clear capability for identifying such individual ramp sites, there seems to be a ready mechanism for focusing the new knowledge directly on the biggest problem cases. While it may develop that geometric modifications to many of these sites is not currently practicable, the use of improved warning and advisory speed signing, or perhaps the removal of curbs, may still offer effective short-term countermeasures. Clearly, the magnitude of the safety problem at certain sites may warrant immediate corrective action, as best the highway engineer can formulate such action, without waiting for the formal evaluation of countermeasures as outlined above.

5) The assurance of adequate pavement friction level for safe operation of trucks calls for new research in the area of truck tire traction, with reference to the ASTM Skid Number characterization. The goal of this research

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[1] One large national fleet, operated by the Linde Division of the Union Carbide Corporation has already mandated that its drivers of cryogenic tankers travel at 10 mi/h (16 km/h) below the advisory speed on all freeway ramps.

would be a guideline by which States may modify their criteria for surface friction maintenance at certain sites having a large volume of high-speed truck traffic. The assumption, of course, is that the ASTM SN characterization is, indeed, sensitive to pavement texture and composition in a manner which is sufficiently similar to the traction sensitivities of truck tires that it will serve to usefully determine the pavement properties assuring minimum truck tire traction performance. To deal directly with the methodological problem of assessing skid numbers on ramp curves, it is recommended that FHWA's recent developments on this subject be presented to State highway departments in the context of measuring ramp skid numbers where trucks have experienced frequent wet-weather accidents.

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