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Contents

Aspects of Spiral Transition Curve Design

Joseph Craus and Abishai Polus, Department of Civil Engineering, Technion, Israel Institute of Technology

Some aspects of the design of spiral transition curves on highways are discussed, and a model for the relation between the design speed and the rate of change of centrifugal acceleration is presented. The model is based on the principle that higher speeds require higher comfort. An identity between the spiral length given by the model and the length of the superelevation runoff is assumed. A modified criterion for the maximum relative slope of the centerline and the edges of a two-lane pavement is proposed. The values for the rate of change of centrifugal acceleration and the maximum relative slope suggested are shown to be reasonable.

When the alignment of a highway changes directly from a tangent to a circular curve, the driver of an automobile on the highway is subjected to a sudden centrifugal force. The use of a spiral transition curve helps to avoid the sudden impact of this force as this curve follows the actual path of the vehicle more closely and improves the visual quality of the highway.

The mathematical expression for the minimum length of a spiral curve was developed by Shortt (6) and is given by

$$L_s = 3.15V^3/R_cC \tag{1}$$

where

L_c = minimum length of spiral curve (feet),
V = design speed (miles per hour),
R_c = radius of curve (feet), and
C = rate of change of centrifugal acceleration for a unit time interval (feet per second per second per second).

[This model was designed for comparison with the American Association of State Highway Officials (AASHO) standards, which are given in U.S. customary units; values in Tables 1 to 3, the text tables on page 3, and Figures 1 and 3 are not given in SI units.] The factor C is an empirical value that indicates the comfort and safety involved. For a given value of V and R_c, this factor determines the length of spiral needed.

The relation between the design speed and the rate of change of centrifugal acceleration and an evaluation of some practical aspects of a model of V versus C are presented in this paper. The model assumes an identity between the spiral length and the length of superrelevation runoff, and a modified criterion for the maximum relative slope of the centerline and the edge of a two-lane pavement is proposed. A criterion for the maximum radius that requires the use of a spiral transition curve in highway design is also proposed. Most of the analysis is based on the AASHO policy (1), but some aspects are derived from European practices, especially German standards (RAL) (5).

CURRENT PRACTICES FOR DETERMINING C-VALUES

There are a number of methods for determining the value of C at different speeds, but most of them give ranges rather than precise values.

The AASHO policy (1) suggests a range of C-values of 0.3 to 0.9 m/s^3 (1 to 3 ft/s^3), and tabulates values of C that vary linearly from 0.75 to 1.2 m/s^3 (2.5 to 4.0 ft/s^3) for speeds from 80 to 32 km/h (50 to 20 mph) respectively.

The values derived from the RAL Standards (5) are similar, i.e., C = 0.5 m/s^3 (1.6 ft/s^3). However, for speeds above 100 km/h (62 mph), the C-values are lower [e.g., 0.302 m/s^3 (0.99 ft/s^3) at 121 km/h (75 mph)] and decrease as the speed increases. Therefore, C depends on the speed.

The Northwestern University Traffic Institute (NUTI) Geometric Design Notes (4) describe C as the factor of comfort and safety in negotiating highway curves and recommend the use of C = 0.3 m/s^3 (1 ft/s^3) as desirable and C = 0.6 m/s^3 (2 ft/s^3) as a minimum. These notes also suggest that the maximum length of the spiral should also be considered and that the equivalent of an 8-s travel interval is appropriate.

SUGGESTED MODEL FOR C-VALUES

A few basic principles served as guides for the development of the model for the relation between C and V. These include the dynamic safety, simplicity, and practical validity. That C decreased with increasing values of V was established on two bases: The first of these is intuitive—C is often called the comfort coefficient, which suggests that, at high speeds, there should be a lower rate of change, i.e., a smaller amount of centrifugal acceleration acting on the driver in a unit time. The second is that calculations of the centrifugal acceleration (V^2/R) for different speeds and the appropriate minimum radii show that this decreases as the speed increases.

To obtain relatively high comfort at high speeds, increasing maneuver times are necessary. Thus, since C is the centrifugal acceleration for a unit time interval, it must decrease as the speed increases.

Many models for the determination of C have been investigated. The derivation of this model divided it into two parts—one for speeds above and one for speeds below 97 km/h (60 mph)—on the assumption that a more moderate change in C is required at higher speeds. Because a linear model does not differ appreciably from a parabolic one for the ranges of values that are considered appropriate, the linear model was chosen because of its simplicity. The model is given in Equations 2 and 3.

$$C = 2.5 - 0.033(V - 30) \quad (30 \leq V < 60) \tag{2}$$

$$C = 1.5 - 0.025(V - 60) \quad (60 \leq V \leq 80) \tag{3}$$

The cutoff points were established as follows: C = 0.3 m/s^3 1 ft/s^3) for a design speed of 129 km/h (80 mph), and C = 0.75 m/s^3 (2.5 ft/s^3) for a design speed of 48 km/h (30 mph). The latter value is somewhat lower than the currently accepted one but may give a more appropriate spiral length. The value of C = 0.45 m/s^3 (1.5 ft/s^3) was chosen for the design speed of 97 km/h (60 mph) since it agrees closely with the value of C = 0.5 m/s^3 (1.6 ft/s^3) for a design speed of 100 km/h (62 mph) that is commonly used in Europe. V versus C, based on Equations 2 and 3, is shown graphically in Figure 1.

Figure 1. Rate of change of centrifugal acceleration versus design speed.

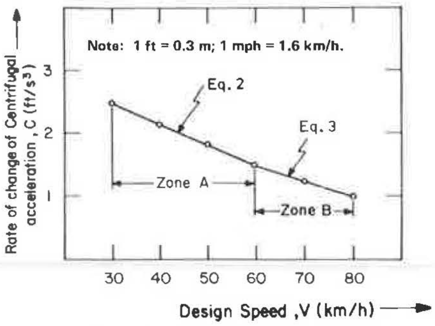


Figure 2. Elements of spiral for a given design speed.

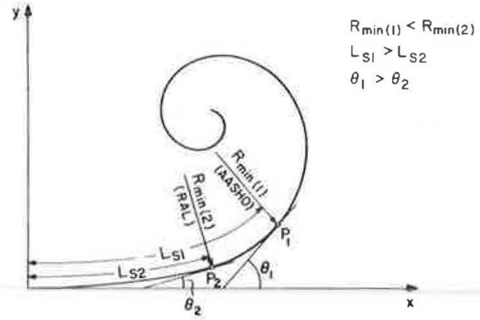


Table 1. L_S, t_M, p, and θ in relation to C at a superelevation of 6 percent.

V (mph)	C (ft/s³)	R[a] (ft)	L_s (ft)	t_M (s)	p (ft)	θ (°)
30	2.50	273	124.62	2.85	2.37	13.07
40	2.16	508	183.73	3.15	2.77	10.36
50	1.83	833	258.30	3.55	3.34	8.88
60	1.50	1263	359.14	4.11	4.26	8.15
70	1.25	1815	476.23	4.67	5.21	7.51
75	1.13	2206	535.50	4.81	5.42	6.95
80	1.00	2510	642.55	5.51	6.85	7.33

Note: 1 mph = 1.6 km/h; 1 ft = 0.305 m.
[a]Minimum radii suggested by AASHO (1) for a superelevation of 6 percent.

Table 2. L_S, t_M, p, and θ in relation to C at a superelevation of 8 percent.

V (mph)	C (ft/s³)	R[a] (ft)	L_s (ft)	t_M (s)	p (ft)	θ (°)
30	2.50	250	136.08	3.11	3.09	15.58
40	2.16	464	201.14	3.45	3.63	12.41
50	1.83	758	283.86	3.90	4.43	10.73
60	1.50	1143	396.85	4.50	5.74	9.95
70	1.25	1633	529.31	5.15	7.15	9.28
75	1.13	1974	598.70	5.48	7.56	8.68
80	1.00	2246	718.08	6.16	9.57	9.17

Note: 1 mph = 1.6 km/h; 1 ft = 0.305 m.
[a]Minimum radii suggested by AASHO (1) for a superelevation of 8 percent.

Table 3. Evaluation of spiral elements using different minimum radii.

Superelevation (%)	V (mph)	C (ft/s³)	R (ft)	L_s (ft)	t_M (s)	p (ft)	θ (°)
6	50	1.83	1147[a]	187.50	2.56	1.27	4.68
6	75	1.13	3278[a]	358.66	3.26	1.63	3.13
8	50	1.83	852[b]	247.20	3.37	2.98	8.30
8	75	1.13	2459[c]	476.10	4.33	3.84	5.55

Note: 1 mph = 1.6 km/h; 1 ft = 0.305 m.
[a]Minimum radii suggested by RAL (5). [b]Minimum radii suggested by Craus (3).

EVALUATION OF MODEL

To evaluate the suggested model, the values of the minimum length of spiral (L_s), the maneuver time (t_M), the offset from the initial tangent to the shifted circle (p), and the spiral angle (θ) (2) were calculated for the minimum values of radii suggested by AASHO (1) as appropriate for certain speeds at superelevations of 0.06 and 0.08. The results are summarized in Tables 1 and 2 respectively.

The following results are apparent.

1. The spiral length increases with increasing speed, but does not become unreasonably long.
2. The offset from the initial tangent to the shifted circle increases as the speed increases.
3. The spiral angle decreases as the speed increases [except between 121 and 129 km/h (75 and 80 mph)].
4. The spiral length and spiral angle, which are shown in Figure 2, depend only on the radius of the circular curve for a given design speed since the design speed determines C, which is actually the only factor defining the spiral; i.e., the spiral is defined by C (for a given speed), and its endpoint is determined by the radius.
5. The maneuver time increases continuously. (It can be proven that the maneuver time increases in a hyperbolic manner as V increases.) At speeds above 80 km/h (50 mph), this time is within the range set by the NUTI notes (4).

A further analysis of the spiral elements was carried out by using the suggested model for C and different minimum radii. (Since the same model for C is being used, the same spiral is being investigated, but its endpoints will be different.) Some results using the minimum radii recommended by RAL (5) for a superelevation of 0.06 and those recommended by Craus and others (3) for a superelevation of 0.08 are given in Table 3. The values of the spiral lengths and the maneuver times are reasonable and within appropriate boundaries. Since the radii recommended by AASHO (1) are smaller than those recommended by RAL (5) and by Craus (3), the values of L_s and θ given in Table 3 are smaller than the corresponding values given in Tables 1 and 2.

INTERCHANGEABILITY BETWEEN SUPERELEVATION RUNOFF AND SPIRAL LENGTH

The current policy of many design agencies is to use the whole length of the spiral curve to make the desired change in the cross slope. This common practice simplifies the construction and the calculations. The AASHO policy (1) assumes that, for the most part, the calculated values for the lengths of spiral and superelevation runoff do not differ very much. The consistency of this approach was verified by substituting various specific lengths that are suggested for the superelevation runoff of a two-lane pavement (as given in Table 3-2 of the AASHO policy. The values of L_s in Equation 1. The C-values derived in this way are given in the table below for three rates of superelevation and different speeds (1 mph = 1.6 km/h and 1 ft/s³ = 0.280 m/s³).

Design Speed (mph)	C (ft/s^3) e = 6%	e = 8%	e = 10%
30	2.79	2.30	2.00
40	3.11	2.52	2.30
50	3.21	2.69	2.33
60	3.31	2.72	2.36
65	3.41	2.79	2.43
70	3.34	2.79	2.50
75	3.11	2.59	2.30
80	3.25	2.75	2.50

At a constant superelevation rate the C-values increase with the speed, which contradicts the basic principle of the model. The use of higher C-values at high speeds in itself also leads to some discrepancies related to the safety and comfort concept of highway design.

Different maximum relative slopes (Δ) between the profiles of the edges and the centerline of a two-lane pavement should be used to make the lengths of the superelevation runoff satisfactory for use as lengths of spiral transitions. These values can be calculated by

$$\Delta = be/L_s = beR_cC/3.15V^3 \quad (4)$$

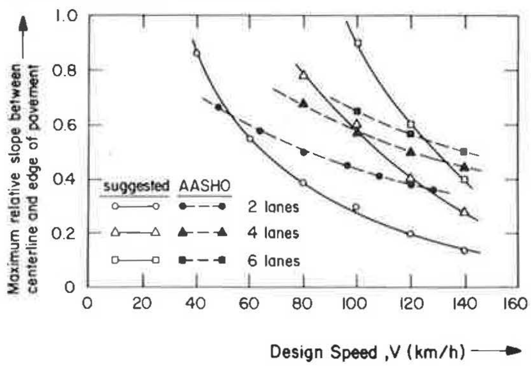
where

R_c = minimum radius suggested by AASHO for a given superelevation and design speed (feet),
C = value defined by Equations 2 and 3, and
b = lane width [12 ft (3.7 m)].

The results for three rates of superelevation are given in the following table (1 mph = 1.6 km).

V (mph)	Δ for e = 6% (%)	Δ for e = 8% (%)	Δ for e = 10% (%)
30	0.58	0.71	0.81
40	0.39	0.48	0.55
50	0.28	0.34	0.39
60	0.20	0.24	0.28
70	0.15	0.18	0.21
75	0.13	0.15	0.17
80	0.11	0.13	0.15

Figure 3. Suggested values of maximum relative slope between centerline and edge of pavement as a function of design speed (e = 8 percent).



Note: 1 km/h = 1.6 mph.

The Δ-values suggested by AASHO are a function of the design speed only, but those given in the above table are a function of both the design speed and the superelevation rate. These values are lower than the AASHO values, except at the low design speed of 30 mph (48 km/h) for the superelevations of 8 and 10 percent.

The maximum relative slope suggested by AASHO for a four-lane pavement is greater than the suggested value for a two-lane pavement by a factor of 1.33. However, since the length of the spiral should be determined mainly by dynamic and comfort considerations, rather than by the number of lanes, the Δ-value for four-lane highways should be doubled.

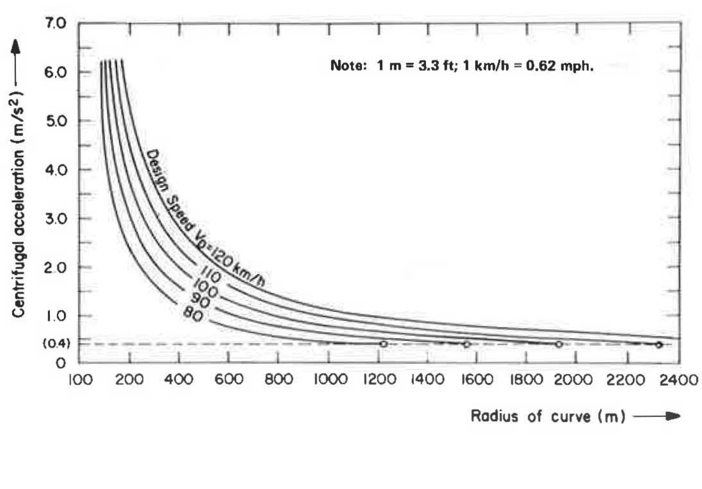
A further analysis that was based on the minimum radii suggested by Craus and others (3) for a superelevation of 8 percent and was carried out in SI units is given in Table 4. The maximum relative slope between the profiles of the centerline and the edges of four and six-lane pavements is doubled and tripled accordingly.

Table 4. Calculated values of Δ for minimum radii given by Craus for superelevation of 8 percent.

V (km/h)	C (ft/s^3)	C (m/s^3)	R (m)	Δ for Two-Lane Highways (%)	Δ for Four-Lane Highways (%)	Δ for Six-Lane Highways (%)
40	2.67	0.81	50	0.86	–	–
60	2.26	0.69	125	0.55	–	–
80	1.83	0.56	260	0.39	0.78	–
100	1.43	0.44	500	0.30	0.60	0.90
120	1.13	0.34	750	0.20	0.40	0.60
140	0.83	0.25	1080	0.14	0.28	0.40

Note: 1 km/h = 0.62 mph; 1 m = 3.28 ft.

Figure 4. Recommended criteria of maximum radii that require use of spiral transition curves.



Since it is logical to assume that a four-lane highway will have higher design speeds than will a two-lane highway, the Δ-values for a four-lane highway are given for speeds equal to or greater than 80 km/h (50 mph) and, similarly, the Δ-values for six-lane highways are given for design speeds equal to or greater than 100 km/h (62 mph). This analysis leads to the following conclusions.

1. The Δ-values between the profiles of the edges and the centerline of a two-lane pavement are lower than the values suggested by AASHO (1). The exact relation is shown in Figure 3. The use of lower Δ-values gives a more gradual superelevation runoff, and the identity in length with that of the spiral may result in a rather simplified design.

2. For multilane highways, the Δ-values suggested here are higher than the AASHO values at lower speeds but not at speeds above 100 km/h (62 mph) on four-lane highways and 120 km/h (75 mph) on six-lane highways.

MAXIMUM RADIUS FOR NECESSARY USE OF SPIRALS

The need for transition curves is most pronounced on sharper curves. On curves having larger radii there is less need for the use of spirals.

Several criteria have been suggested for the use of spirals. One method designates a single degree of curve that is applicable to all design speeds. Another method suggests the use of spiral curves when p, computed by Equation 1 with $C = 0.6\ m/s^3$ ($2\ ft/s^3$), is greater than 0.3 m (1 ft). The method given in the NUTI Geometric Design Notes (4) suggests that the spiral be used on curves that require a superelevation rate of 0.03 or more

The following assumptions suggest another criterion for the introduction of spiral curves. A gently curving alignment that requires little centrifugal-acceleration resistance should not require spirals.

The minimum amount of centrifugal acceleration for the introduction of spiral transition curves is $0.4\ m/s^2$ ($1.3\ ft/s^2$). The criterion for the maximum radii that will require use of a spiral is

$$V^2/R_c = 0.4 \qquad (5)$$

where R_c = maximum radius for necessary use of spiral transition (meters) and V = design speed (kilometers per hour). The values calculated by Equation 5 can be read directly from Figure 4, which shows that the centrifugal force varies hyperbolically with speed and radii.

The centrifigual-force criterion has two advantages. First, this criterion is based on the actual force that is imposed on the traveling vehicle. Second, this criterion agrees with the assumption, given by Spindler (7), that the safest and most comfortable situation is that in which the side-friction factor and the superelevation equally resist the centrifugal acceleration; i.e., the ratio of e to (e + f) should be 0.5.

CONCLUSIONS

A model for the relation between the rate of change of centrifugal acceleration on a spiral transition curve and the design speed is presented. The model has two regions and decreases linearly. The resulting spirals and their properties are discussed. The practical reasons of safety and uniformity were used as a guideline to the suggestion that the maximum relative slope between the edges of the pavement and the centerline should be greater than that recommended by AASHO. An identity between the superelevation runoff and the spiral lengths is assumed. The criterion of the maximum radius for the use of spiral transition curves is also discussed.

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New Concepts in Design-Speed Application

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The design-speed concept, as presently applied, does not preclude inconsistencies in highway alignment. The basic problem, particularly in the range of design speeds below 90 km/h (55 mph), is the tendency on the part of the driver to continually accelerate and decelerate. A secondary problem is the speed differential between automobiles and trucks. To overcome these weaknesses in current practice, a new concept in the definition and application of design speed is presented. The overall object is to meet driver expectations and to comply with his or her inherent characteristics to achieve operational consistency and improve driving comfort and safety. The principle used in the updated design-speed approach is the 15-km/h (10-mph) rule, which during periods of free-flow conditions, entails three considerations: (a) A reduction in design speed

should be avoided if possible, but if it is required, it should be no more than 15 km/h (10 mph); (b) within a given design speed, potential automobile speeds along the highway normally should vary no more than 15 km/h (10 mph); and (c) potential truck speeds generally should be no more than 15 km/h (10 mph) lower than automobile speeds on common lanes. The tool to accomplish these goals is a speed-profile technique. The potential automobile and truck speeds are plotted along the proposed highway improvement, taking into account the joint configuration of the horizontal and vertical alignments and the individual curvatures and gradients. The procedure is applicable to design of new facilities, but is even more useful for determining corrective measures to upgrade existing facilities.

The convenience and economy that a highway offers are related to the speed and safety of operation on it, and the speed that can be maintained is a direct function of the quality of the highway and the amount of traffic that it handles. Because of this, attempts to relate the potential speeds on the highway to its various geometric elements have been important for many decades, which has resulted in the introduction of the concept of highway design that is referred to as the design speed.

The application of this concept has served well, but in recent years a better understanding of the complexities associated with the driving task and the multitude of related human factors has shown the need to expand and improve on it.

The object of this paper is to present a new concept of the definition and application of design speed. The function of this concept in the design and redesign of highways will be to better meet driver expectations and to comply with his or her inherent characteristics. The results presented are intended to achieve operational consistency and improve driving comfort and safety through a more uniform and balanced design of highway alignment.

CONCEPT OF DESIGN SPEED

The concept of design speed was introduced during the 1930s, and its application was embraced in the 1940s. Two publications (1, 2) on highway geometrics had an important role in instituting this design guide, which with minor modification has been adhered to in the 1954, 1957, 1965, and 1973 American Association of Highway Officials (AASHO) Geometric Design Policies (3, 4, 5, 6). However, recent knowledge of traffic operations and driver characteristics has made it evident that the concept and its applications should be updated.

One problem in the use of design speed as it is applied today is that, primarily at lower speeds, the changing alignment causes variations in operating speeds; i.e., the horizontal curves that control the design speed along the highway cause the driver to increase speed on the flatter portions of the alignment and then require him or her to decrease speed on the sharper or controlling curves.

Another problem is that the design speed sometimes is lower than the driver's expectation and judgment of what the logical speed should be. The design speed must appear to be reasonable to the driver, whose sensitivity and judgment of a logical speed are very keen. He or she expects a high design speed in open country and flat terrain but recognizes, even if only inadvertently, the difficulty of the situation in mountainous terrain or in highly urbanized or built-up areas. Therefore, a speed that more nearly meets the driver's natural tendency must be used. On many highways, the elimination of some of the sharper curves to increase the design speed would make a more consistent and safer design. On others, the configuration of the alignment can be arranged to produce a more uniform speed if some curves of appropriate degree are introduced and others are nominally and compatibly sharpened.

The object is not only a logical and acceptable design speed but also one that produces a relatively uniform operating speed. This is fully met where high design speeds are used, but at low and intermediate design speeds, the portions of relatively flat alignments interspersed between the controlling portions tend to produce increases in operating speed that may exceed the design speed by substantial amounts.

This undesirable feature must be overcome. The communicative aspects of alignment design that interact with driver responses—the control and guidance of the vehicle and the pattern of operating speeds—are related to the configuration and composition of the longitudinal alignment. Thus, to compensate for the driver's physical and emotional states, to meet his or her expectations, and to comply with his or her inherent characteristics, it is necessary to achieve operational consistency and improve driving comfort and safety by the application of the appropriate design speed. Accordingly, this new concept of design speed recognizes the inadvertent increase in speed of drivers but limits it to a maximum of 15 km/h (10 mph). Design-speed designations remain the same [50, 65, 80, 95, 110, or 125 km/h (30, 40, 50, 60, 70, or 80 mph)], but, for speeds of less than 100 km/h, recognize a potential overdriving speed of 15 km/h; for example, a design speed of 50 km/h (30 mph) means a design-speed range of 50 to 65 km/h (30 to 40 mph), a design speed designated for 65 km/h (40 mph) means a design-speed range of 65 to 80 km/h (40 to 50 mph), and so on. This principle is summarized below.

1. Within a given design speed, the potential average automobile speeds should not vary more than 15 km/h (10 mph).
2. When a reduction in design speed is necessary, it should normally be no more than 15 km/h (10 mph).
3. On common lanes, potential average truck speeds should generally be no more than 15 km/h (10 mph) lower than average automobile speeds.

This device makes the composition of the highway a function of the design speed. What the driver sees and the message conveyed by the highway should induce a response on the part of the driver that encourages him or her to operate at a reasonably consistent speed. The physical makeup of the alignment should prevent or discourage increases in speed to more than 15 km/h (10 mph) beyond the designated design speed.

The basic concept of design speed, except for details of application, has the same general meaning and objectives today as it did nearly 40 years ago. The design speed primarily determines the quality of the highway and provides a consistent design. Once the design speed is selected in accordance with the driver's fundamental expectations and in conformity with the basic controls of the type of terrain, the extent of man-made features, and economic limitations, it is unhampered by other constraints, such as the constantly changing conditions on a highway caused by the effects of weather variations; differences in size, weight, and power of vehicles; the wide variation of behavior within the driving population; variations in traffic volumes during different periods of the day; and speed-limit restraints.

These constraints can have significant and sometimes dramatic effects on traffic operating speeds. However, they have no relation to the design of the highway after the design speed has been selected. Even the current imposition of the general 88-km/h (55-mph) speed limit on U.S. highways does not and should not have any effect on the selection of the design speed or the quality of the highway and the standards to which it should be designed. The highway and its basic configuration, which reflects its quality, will last for 30 years or more, but the speed limit, the form of energy used, and the type of vehicle employed can readily change in that period.

What actually happens is that the relative high quality of the highway provides a significant safety factor. A higher quality highway will have more leeway to handle various overloads and inconsistencies. The better design with a given number of lanes can carry the traffic at a higher level of service and with a larger margin of safety for minimizing hazardous maneuvers and accidents. Also, the larger the difference is between the design speed and the average speed of operation or the speed limit, the safer the operation of the highway will be; e.g., a highway that has a design speed of 110 km/h (70 mph) and a speed limit of 80 km/h (50 mph) is safer than a highway that has a design speed of 80 km/h (50 mph) and a speed limit of 80 km/h (50 mph).

This approach leads to the following reformulated definition: Design speed is a representative potential operating speed that is determined by the design and correlation of the physical (geometric) features of a highway. It is indicative of a nearly consistent maximum or near-maximum speed that a driver could safely maintain on the highway in ideal weather and with low traffic (free-flow) conditions and serves as an index or measure of the geometric quality of the highway.

The 15-km/h (10-mph) incremental speed value is based on several facts. Experience has shown that a driver can cope reasonably well with a 15-km/h (10-mph) speed adjustment in most circumstances. There is also evidence that variations from the average speed cause a definite increase in accident involvement (8). Also, there is a sharp rise in the ratio of the number of accidents occurring as the speed reduction of trucks (the difference between the average speed of trucks and the average speed of automobiles) increases relative to the number of accidents occurring when there is no speed reduction (i.e., trucks and automobiles can maintain the same speed). This is tabulated below (1 km/h = 0.6 mph).

Speed Reduction (km/h)	Ratio of Accidents	Speed Reduction (km/h)	Ratio of Accidents
8	2	24	8.9
16	3.7	32	15.9

This new definition requires a tool to regulate the design so as to maintain the necessary consistency and quality of the alignment. The speed profile (Figure 1), which charts potential automobile and truck speeds, can provide the necessary insight.

The application of the new design-speed concept also requires that, at the point at which potential speed increases occur, appropriately higher sight distances and increased superelevation on curves must be provided. Thus, the design will be fully sensitive to the way in which the driver operates. Moreover, should a change in design speed become necessary, the justification for the introduction of a reduced design speed should be obvious and psychologically acceptable to the driver.

DEVELOPMENT OF SPEED-PROFILE TECHNIQUE

The application of the design-speed concept requires that the speed profile be developed and analyzed for every project. On an existing facility, speed measurements (mean, average, or any percentile) at close intervals along the highway can be plotted against distance. Separate measurements and plots can be made for different classifications of vehicles; e.g., vehicles with passenger-automobile characteristics can represent one group and large trucks (dual-tired) can represent a second group and other subdivisions can be considered in special cases.

However, the use of the direct-measurement technique for every existing highway designated for improvement would be too time-consuming and expensive. Moreover, there are no completely appropriate or practical procedures available for the prediction of speed versus distance relations for determining or evaluating the operational characteristics on proposed improvements on old highways, or in the design of new ones. Some previous attempts to formulate such techniques have been reported by Leisch and others (10) and by Baluch (11).

The method for determining speed profiles given here has been devised from limited data, extensions of minimal research in vehicle operations, empirical speed relations, and experience judgments. However, despite the lack of direct research on some aspects of the problem, the results are believed to be sufficiently accurate for their purpose, can be effectively used immediately, and will serve as the essential tool for designing highway improvements directed toward optimizing operations and improving safety.

Speed-Profile Components

Speed profiles for a given type of highway may represent different conditions, such as speed measure (mean, average, or 85th percentile), traffic-volume condition (free-flow or specific traffic volumes), different types of vehicles, or intersection or interchange conditions along the route.

The type of speed profile developed here, particularly as it relates to the design speed, represents a basic set of conditions that allows the most direct way to evaluate and optimize the uninterrupted flow (geometric) characteristics of the highway. It applies to any type or classification of highway, but is normally used for rural or suburban highways on portions where there are no intersection or interchange problems.

The basic characteristics of the speed profile given here are predicated on the following assumptions:

1. Low volumes (free-flow conditions);
2. Average (running) traffic speeds;
3. Favorable roadway conditions such as daylight and good weather;
4. Top average speeds representative of freely moving vehicles (automobiles and trucks) on relatively straight open sections of roads, outside the influence of any other geometric constraints;
5. Average (running) speeds that agree with the low-volume relations of average running speed to design speed on horizontal curves;
6. Separate average (running) speeds for automobiles and for trucks plotted in juxtaposition [average truck speeds on or near level grades are assumed to be 8 km/h (5 mph) below average automobile speeds] (other values may be used as appropriate);
7. Truck selected to be representative for a particular highway (average weight to power ratio of 200 assumed) (other values may be used as appropriate);
8. Deceleration and acceleration for automobiles predicated primarily on and extrapolated from 1965 AASHO Geometric Design Policy (5); and
9. Deceleration and acceleration for trucks compiled from 1965 Highway Capacity Manual, 1965 AASHO Geometric Design Policy, and 1972 Federal Highway Administration Dynamic Design for Safety and (5, 10, 14).

Speed Values and Related Factors

The speed profile is represented by the average (running) speed of traffic during free-flow conditions. Its basic inputs evolve from a set of average speeds for free-flow, near-level highway conditions on tangent sections and horizontal curves.

Top Average Speed of Highway

Every highway or homogeneous section of one has what may be termed its top average speed that depends on the environment, the type of highway, the length of trips made on it, regional effects, and possibly speed limits and other restrictions. This relates to conditions of freely moving vehicles and indicates a driver's (average) desired speed under ideal traffic, roadway, and weather conditions on near-level and straight sections of road that are sufficiently removed from any constraining effects. Many such measurements have been recorded (12). The composite average speeds on main rural highways have increased from 80 km/h (50 mph) for automobiles and 72 km/h (45 mph) for trucks in 1951 to 100 km/h (62 mph) for automobiles and 92 km/h (57 mph) for trucks in 1972. [The recent constraint of the 88.5-km/h (55-mph) national speed limit has changed these relations somewhat.] The top average speeds for various types of vehicles and highways are summarized below (1 km/h = 0.62 mph).

	Avg Speed (km/h)		
Type of Highway	Automobiles	Trucks	All
Rural Interstate	107	96	104
Rural primary	94	87	93
Rural secondary	86	80	85
Urban Interstate	93	85	91
Urban primary	70	66	69
Urban secondary	64	62	64

Measurements of top average speeds of this kind should be a prerequisite to the reconstruction of old highways and an important input to the speed profile. In the design of new highways, measurements made on similar highways in the general corridor or region should be used. The values given below (or similarly derived values) are representative of low-volume, free-flowing conditions on open, near-level, and straight highways and may be used as a guide to top average speeds in the absence of specific data (1 km/h = 0.62 mph).

	Top Average Speed (km/h)	
Type of Facility	Favorable Highway Quality	Moderate Highway Quality
Rural highway		
Interstate	100	95
Primary—main	95	90
Primary—intermediate	90	80
Secondary	80	70
Urban highway		
Interstate	95	90
Arterial—main	80	70
Arterial—intermediate	70	65
Secondary—feeder	65	55

Speed on Curves

The curves that control the design speed (i.e., the maximum degree of curvature or the minimum radius for a given design speed) have been related, by actual measurements, to a set of corresponding average running speeds, as shown below (5) (1 km/h = 0.62 mph).

Design Speed (km/h)	Average Running Speed (km/h)	Design Speed (km/h)	Average Running Speed (km/h)
50	46	95	82
65	58	110	92
80	70	125	100

This basic relation between the design speed and average running speed is assumed to be applicable even where a flatter curvature is encountered so long as the design-speed designation is maintained. The problem arises when a driver is encouraged by a sufficiently long or inviting section of more gentle alignment to increase speed momentarily, even though he or she may reduce the speed to normal further along the road. In such a case, the operating speed (the average running speed) disrupts the above relation, which indicates that an equivalent higher design speed had been achieved temporarily. The proposed speed profile and the application of the 15-km/h (10-mph) rule would reveal this problem and provide the insight to adjust the design and possibly smooth out the differences.

For speed-profile purposes, what average speeds should be assigned to individual alignment curves? Figure 2 [adapted from the 1965 AASHO Geometric Design Policy (5)] shows the relations among the degree of curvature, the radius of the curve, and the average (maximum) speed that can be driven on it. The data given in this figure assume that the driver has no other constraints and that the approach speed to the curve is equal to or higher than the tabular value, although the actual average speed on the curve may be less because of the composition of the alignment and the type of highway. Lower speeds, and their numerical values in developing the speed profile, will automatically be determined by the driver's process of applying the required decelerations and accelerations.

Figure 2 can be used to establish the points on the initial plot of the speed profile.

Truck Speeds

Large trucks generally operate at lower speeds than do automobiles. On open highways and level grades the difference in average speeds is usually small, and it may be nearly the same when there are speed-limit controls. However, the overall average difference on main highways is approximately 8 km/h (5 mph) (12). The general averages under ideal conditions are that truck speeds are about 8 to 11 km/h (5 to 7 mph) lower than automobile speeds on high-quality facilities and 5 to 8 km/h (3 to 5 mph) lower on lower order facilities. The speed differences between the two types of vehicles are further reduced in less favorable traffic and highway conditions.

In developing the speed profile the 8-km/h (5-mph) speed difference should be applied uniformly along the speed profile under normal circumstances. This pertains to all near-level sections of roads, regardless of their horizontal alignment, although slightly different values could be used where justified by specific data.

On gradients, the speed differential between trucks and automobiles becomes significantly greater.

Speed-Change Characteristics

To maintain his or her desired speed a driver frequently accelerates, although at a moderate rate, after a deceleration that was caused by constraints or possible inconsistencies in the horizontal or vertical alignment of the highway. Because of the differences in the mechanical and operational characteristics of automobiles and trucks and in the drivers of each, the speed variations between the two types of vehicles differ drastically. Automobile speed changes are influenced largely by horizontal curvature, but truck speeds are affected by both horizontal and vertical alignment. Horizontal curvatures have the same basic constraints on both automobiles and trucks. Roadway gradients, on the other hand, have little effect on automobiles but cause trucks to have unusually high rates of deceleration and extremely low speeds on steep, sustained upgrades. Upward gradients of up to 4 and even 5 percent have little if any decelerating effect on the speed of automobiles; i.e., automobiles normally have no problem in maintaining constant-speed operation on such grades, and downward gradients of 3 or more percent have an accelerating effect on automobiles.

Because of these differences, the speeds of each type of vehicle must be analyzed separately. The driver characteristics of both the automobile driver and the truck operator also have a part in negotiating horizontal curves. In the case of the automobile, it is the driver characteristics, rather than the vehicle characteristics, that determine the speed, but in the case of truck operations, it is primarily the mechanical characteristics of the vehicle that determine the speed. This is an important fundamental difference that must be considered in determining decelerations and accelerations on a roadway occupied by both kinds of vehicles.

Sight distance seems to have little effect on the speed that drivers use on the highway. This is unfortunate since sight distance has a significant effect on safety. Drivers apparently are not able to properly judge the speeds that should be associated with sight restrictions and do not change speeds appropriately. Thus, the amount of sight distance or the lack of appropriate sight distance is not an element in the construction of the speed profile. However, extra sight distance should be provided on sections of highway that the speed profile shows to be high-speed operations.

Automobile Deceleration and Acceleration

The main limitations on the speed of automobiles are the horizontal curves. There is little information available on the deceleration and acceleration of vehicles as affected by changes in horizontal alignment of highways. Thus, in the absence of specific information that could be applied directly, a set of empirical rates based on the extension of values associated with intersection and interchange conditions in the AASHO Geometric Design Policies (5, 6) were developed.

Deceleration rates, which are equivalent to slowing

Figure 1. Example of speed profile.

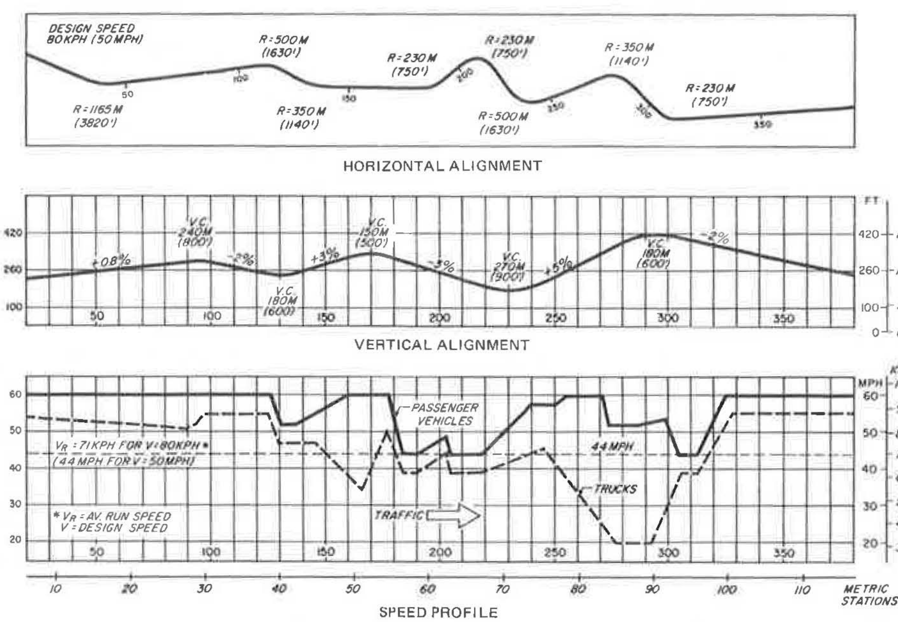


Figure 2. Speed versus curvature relations.

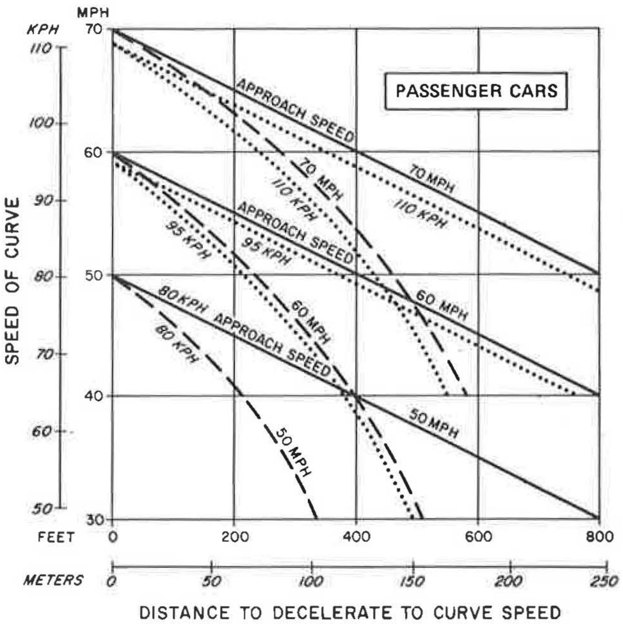
V- DESIGN SPEED, KPH	50			65			80			95			110		125
CORRESPONDING AVERAGE RUNNING SPEED, KPH													92	96	100
										82	86	90	92	96	100
							70	74	78	82	86	90	92 •	→	
				58	62	66	70	74	78	82	86 •	→			
	46	50	54	58	62	66	70	74	78 •	→					
D_M- MAX. DEGREE OF CURVE *	21.0	17.8	14.4	11.9	10.4	8.9	7.5	6.8	5.9	5.1	4.6	4.0	3.7	3.2	2.7
R_M- MIN. RADIUS, METERS	80	95	120	145	165	195	230	255	290	340	375	425	465	540	650

Note: 1 km/h = 0.62 mph; 1 m = 3.28 ft.

* $D_m = 1718.8734 \div R_m$ (BASED ON CENTRAL ANGLE SUBTENDING 30-METER ARC)

•→ FOR A DESIGNATED OR ESTIMATED DESIGN SPEED, ANY LARGER RADII BEYOND THE ARROW ARE ASSUMED TO HAVE THE SAME AVERAGE RUNNING SPEED AS AT THE ARROW.

Figure 3. Deceleration of automobiles approaching a curve that limits the speed.



Deceleration for required speed reduction of *25 KPH* (15 MPH) or less (based on deceleration in gear)

Deceleration for required speed reduction of *30 KPH* (20 MPH) or more (based on "light" braking)

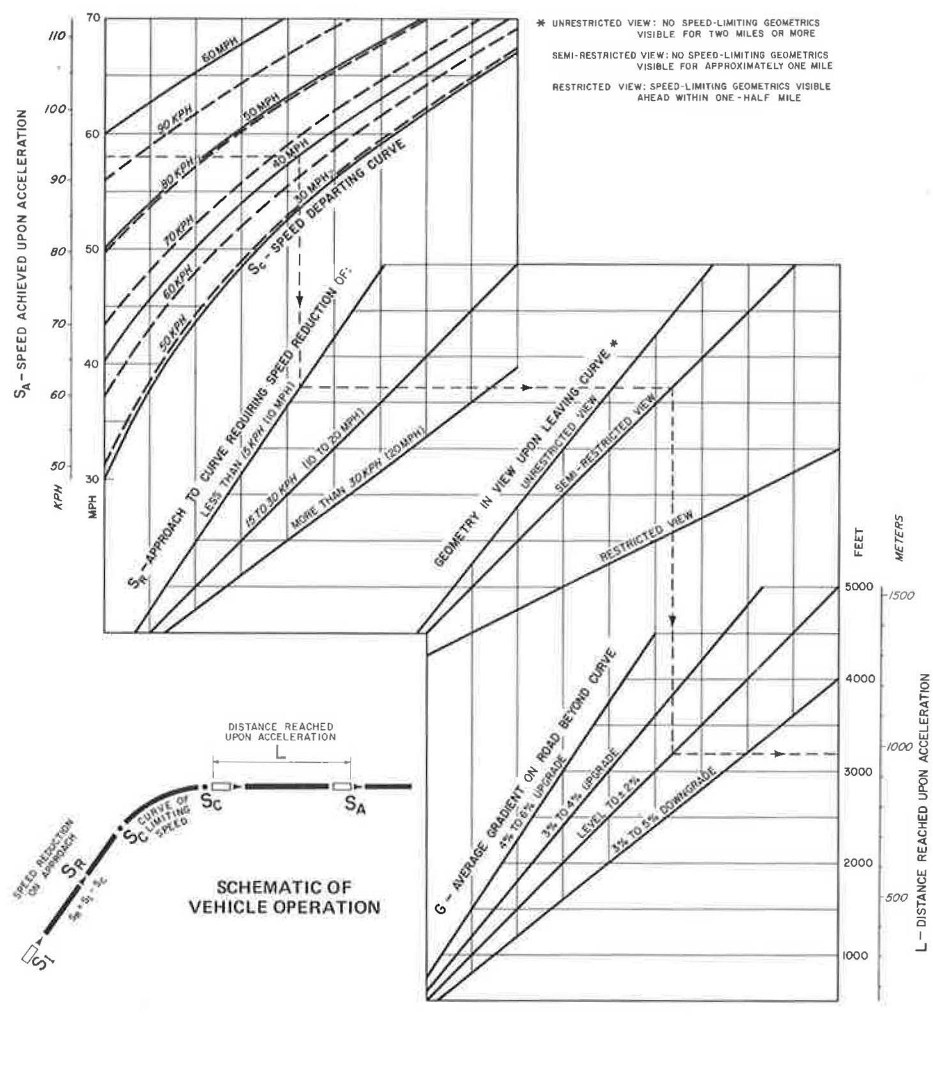
the vehicle in gear, and leisurely rates of deceleration for small and high reductions in speed respectively for approaching horizontal curves have been assumed as shown in Figure 3. The combination of Figures 2 and 3 (5) can be used directly in the development of speed profiles as the vehicle approaches and negotiates a horizontal curve.

The main factors affecting the acceleration of a passenger automobile after encountering a constraint in the alignment are (a) the distance and degree of restrictiveness of the geometry in sight on departing a limiting curve and (b) the difference between the limited speed on the curve being departed and the speed on the road preceding the curve. A driver will accelerate faster on departing a limiting curve if the road ahead has an unrestricted geometry for a considerable distance than if another limiting curve is visible ahead. Also, the more a driver has decelerated in advance of the curve, the faster he or she will accelerate from the curve to regain the previous speed or a higher speed. This rationale and the correspondingly modified normal acceleration rates from the 1965 AASHO Geometric Design Policy (5) were used to develop the speed-change behavioral model given in Figure 4.

Truck Deceleration and Acceleration

The operational characteristics of large trucks are reasonably well documented with respect to gradeability, or speed variations along roadway profiles, but not with respect to changes in horizontal alignment, except for

Figure 4. Acceleration of automobiles departing a curve that limits the speed.



speeds on curves and top average speeds on open highways. Nevertheless, there is sufficient information to assemble a reasonable procedure for the development of their speed profiles.

The weight of the vehicle relative to its power is the most important effect on its speed and on the speed variations as they are influenced by the highway profile (13). Also, vehicles of the same weight to power ratio have similar operating characteristics. Therefore, a representative truck (for purposes of speed characteristics) was established for a given highway or class of highways. An approximately 85th percentile weight to power ratio of the truck population on a given class of highways can be taken as the criterion for a representative truck for design purposes. According to the 1965 Highway Capacity Manual (14), trucks having weight to power ratios of 325 and 200 are typical on two-lane and multilane modern highways respectively, although improvements in truck design continue to lead to lower weight to power ratios.

For purposes of analysis and demonstration in this paper, a typical truck is taken to have a weight to power ratio of 200. The deceleration and acceleration characteristics used to predict speed profiles are predicated on this vehicle.

The speed profile of a truck has two parts. First, the profile with respect to the horizontal alignment, assuming a level or near-level gradeline, must be developed and second, the profile with respect to the vertical alignment only, assuming no restrictions in the horizontal alignment, must be developed. The actual speed profile is then determined as a composite of the two parts.

For the horizontal control portion of the speed profile for trucks, two charts are used—the first for deceleration and the second for acceleration on a level or near-level gradient. Figure 5 [derived from the AASHO 1965 Geometric Design Policy (5) and Leisch and others (10)] can be used to estimate the deceleration of trucks on approaching a horizontal curve that limits the speed. These values were derived on the basis of a 50 percent increase in the deceleration distances for automobiles. This relation is extrapolated from data that indicate that the stopping distances required for trucks are approximately 1.5 to 2 times those for automobiles. This figure is entered at the left with the average speed representative of the curve and reads to the right to the speed of the truck on the approach to the curve and then downward to the distance required to decelerate between the two speeds.

Figure 6 [compiled from Figure 7 and Baluch (11)] can be used to determine the distances required for trucks to accelerate, on level or near-level grades, on departing a horizontal curve that had had a constraining effect on their speed. These relations were compiled from several sources of gradeability characteristics. The figure approximates the distance required for the representative truck to accelerate from the limiting speed

Figure 5. Deceleration of trucks approaching a curve that limits the speed.

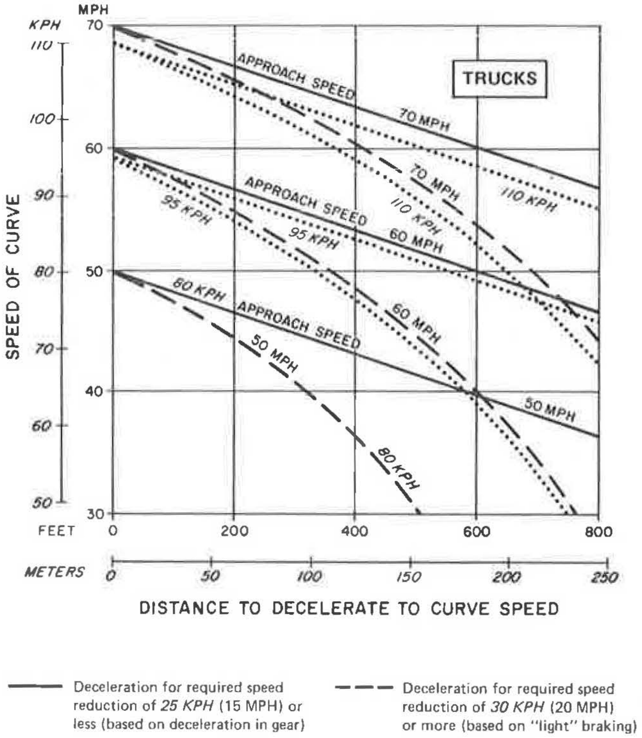
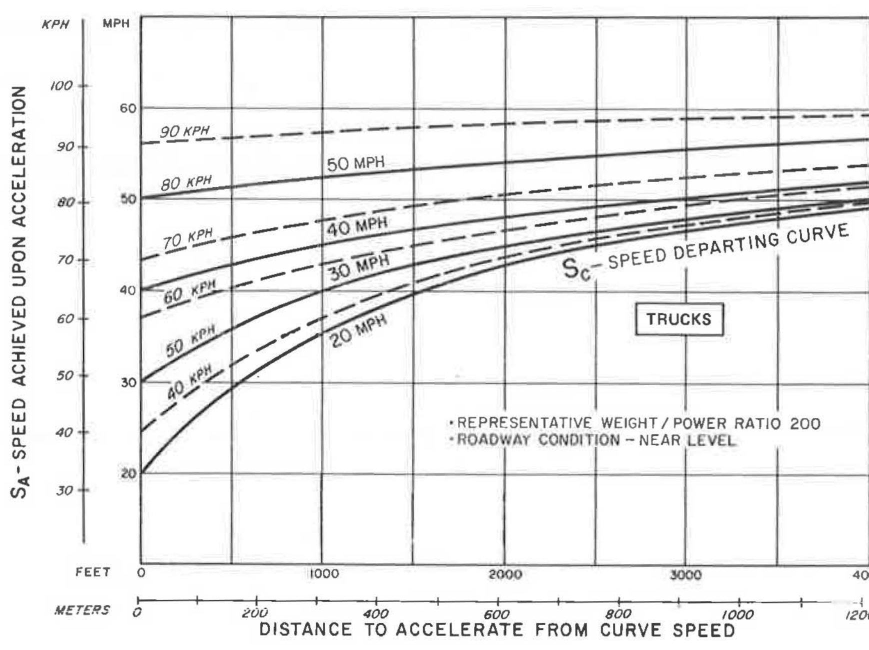


Figure 6. Acceleration of trucks departing a curve that limits the speed.



of the horizontal curve to a given speed beyond the curve.

For the vertical control portion of the speed profile for trucks, the speed versus distance relations give the representative truck. The solid curves show the deceleration distances based on the truck entering an upgrade at a speed of 80 km/h (50 mph). However, a lower entering speed can be used to determine the speed reduction for any distance upgrade by shifting the horizontal axis and, for a higher entering speed, the curves have been extrapolated for grades of 2 percent or more, up to 95 km/h (60 mph). The addition of travel distances to the left of zero will give the total distances along the grade for the various speed changes. After a certain distance of upgrade travel, a minimum constant (crawl) speed is reached.

APPLICATION OF SPEED-PROFILE ANALYSIS AS BASIS FOR DESIGN

The various graphs (Figures 3 through 7), the tabulation in Figure 2, Table 2, and the 8-km/h (5-mph) speed differential between automobiles and trucks can be used for the direct determination of speed profiles for various highways.

Figure 7. Speed versus distance relations for operation of trucks on grades.

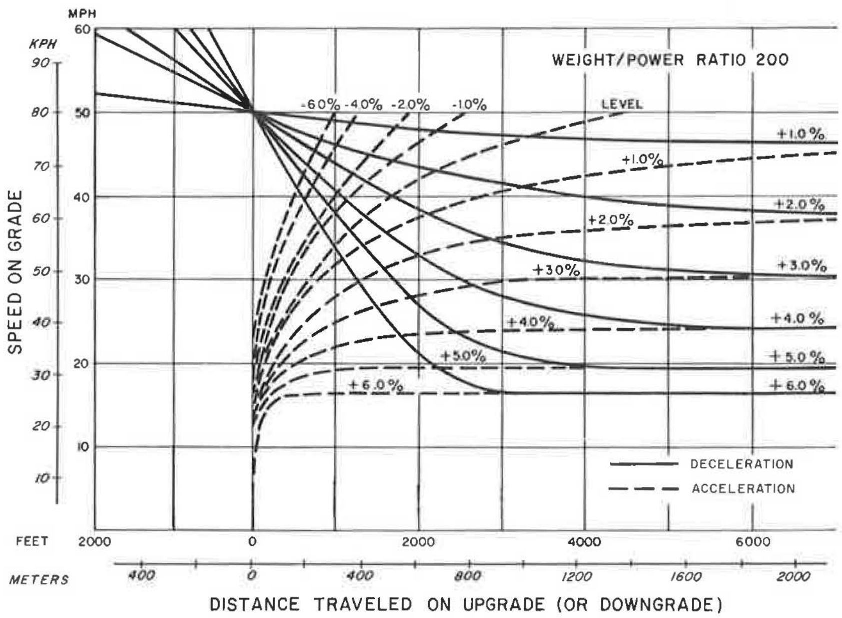
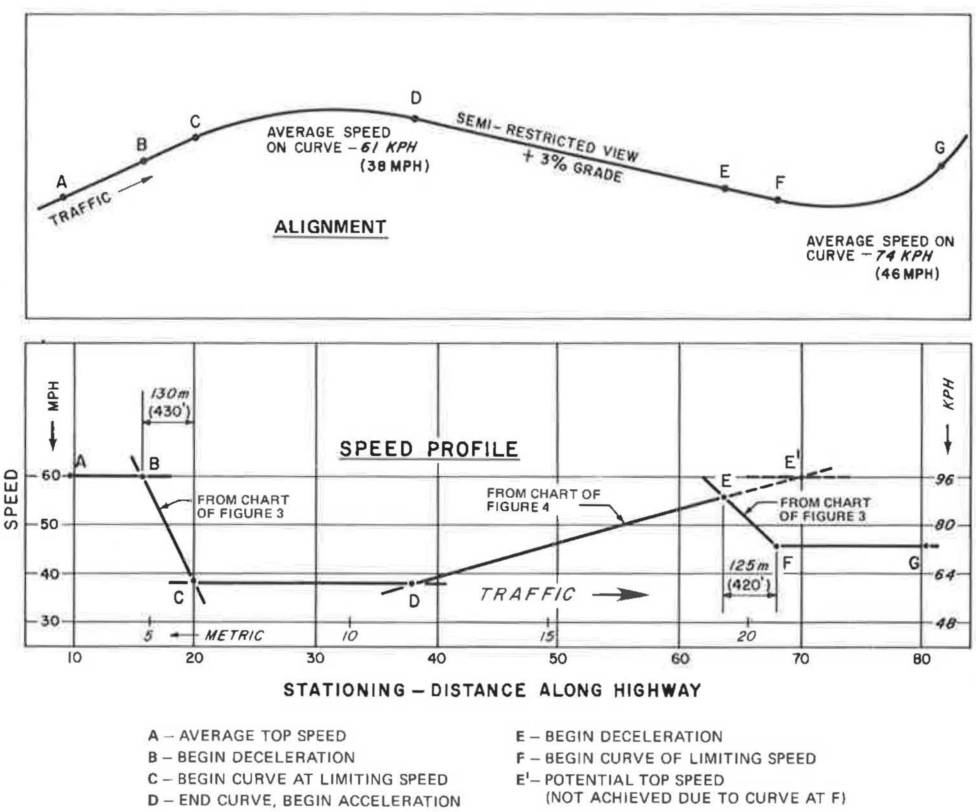


Figure 8. Development of speed profile for automobiles.



The speed profile provides a continuous plot of the average speeds of vehicles along the roadway in each direction of travel at times when the traffic is sufficiently light to represent the condition described as free flowing. By thus charting separate speed configurations, one for a representative automobile and the other for a representative truck, a complete (predicted) record of operating speeds along the course of a highway can be shown graphically. The form of and the associated variations within the speed profile, when plotted and displayed together with horizontal and vertical alignment of the highway, present a picture of the favorable and unfavorable operational characteristics that could not be seen by any other means.

The following illustrations are given to provide a better understanding of how the various charts and related material are used in constructing the speed profile. Figure 8 shows the construction of a speed profile for automobile operations on the indicated alignment. The points labeled A through G designate the principal controls that affect the profile. [The problem assumes that the top average speed is 95 to 96 km/h (60 mph) in agreement with Table 2, which applies to the conditions of the road at point A.] From Figure 2, the horizontal curve C to D (according to its radius) has an average speed of 61 km/h (38 mph). The deceleration approaching this curve at a speed reduction exceeding 30 km/h (20 mph) is 130 m (430 ft) (from Figure 3). The acceleration distance on leaving the curve at point D and driving toward the tentative point E′ is determined from

Figure 9. Use of speed versus distance relations for trucks (Figure 7).

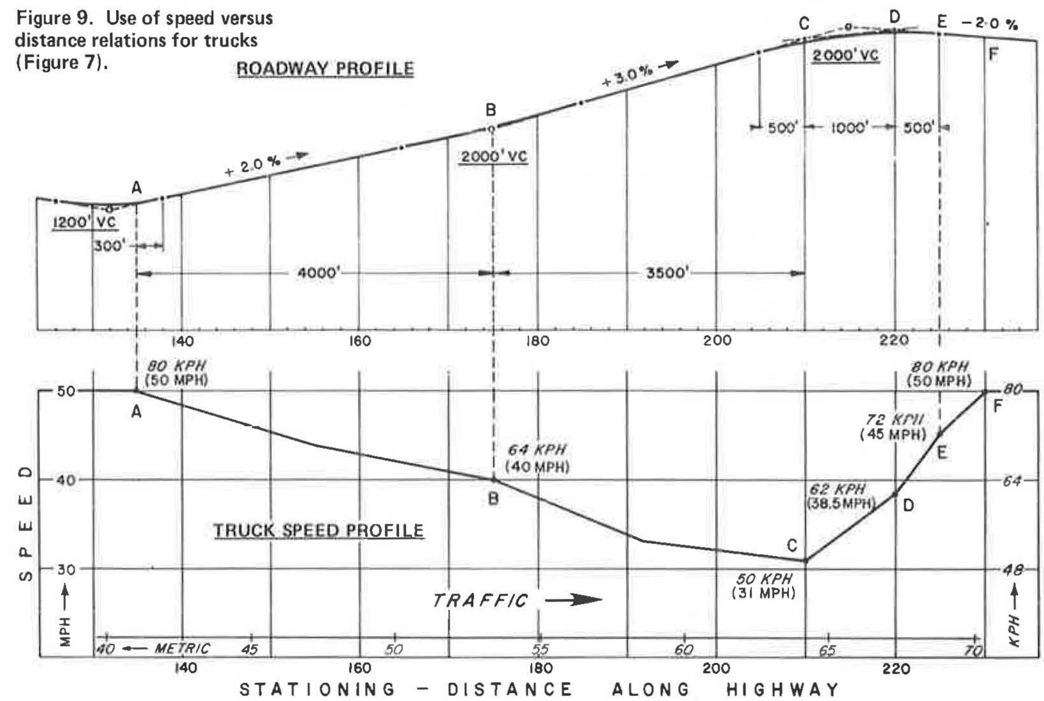


Figure 10. Development of speed profile for trucks.

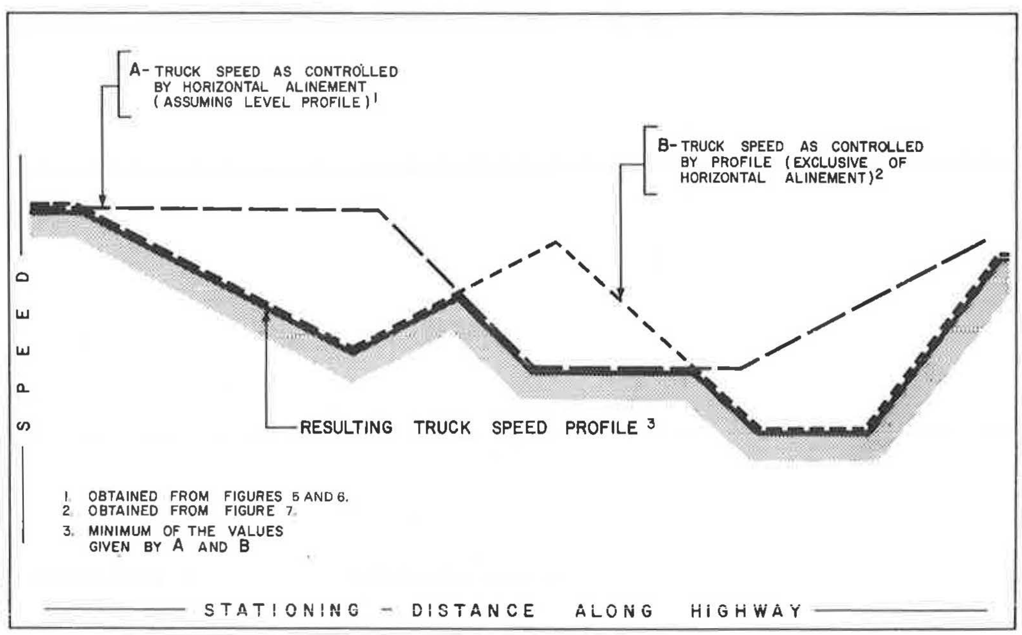


Figure 11. Application of speed profile.

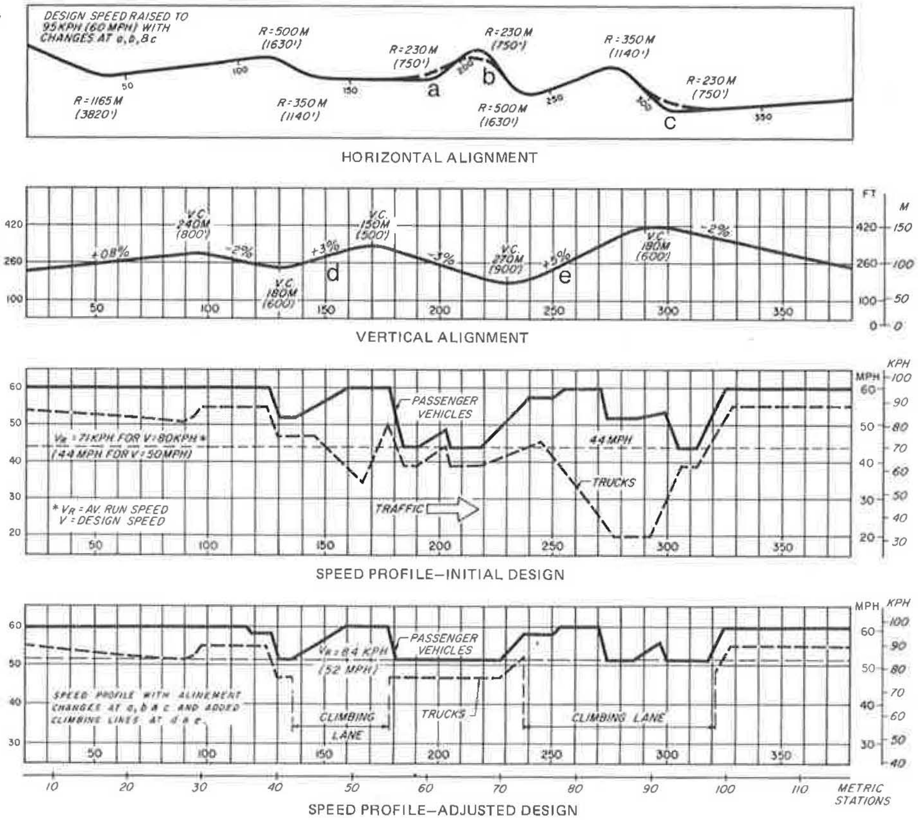


Figure 4. Point E, at which the vehicle begins to decelerate for the next horizontal curve, FG, is determined from Figure 3 by plotting in reverse the deceleration distance between the 74-km/h (48-mph) speed on the horizontal curve and a sufficiently high preselected hypothetical speed on the approach to form the intersection with the previous line. (Occasionally, a second attempt will be required to find the intersection point, if the first hypothetical speed assumed was incorrect.)

Another typical problem is the construction of the speed profile for trucks as controlled by the roadway profile by using Figure 7. An example of the development of a truck speed profile that is controlled by the vertical alignment of the roadway is shown in Figure 9. The technique for the use of Figure 7 by the progressive manipulation of horizontal and vertical projections has been discussed by Leisch (15). In applying the chart of Figure 7 to the roadway profile shown in the upper part of Figure 9, the accompanying truck speed profile was developed directly. It indicates a speed configuration that ranges from an 80-km/h (50-mph) approach to the minimum speed of 50 km/h (31 mph) on the roadway crest at point C and finally through an acceleration to a return to 80 km/h (50 mph) on the downgrade at point F.

The speed profile in Figure 9 is valid if there is no horizontal alignment constraint more severe than the vertical constraint. For this reason, truck speed profiles require the combination of the two effects. The most expedient way to do this is to plot two separate speed profiles for the truck: First, use only the horizontal controls (in effect assuming a near-level gradient) and, second, use only the vertical controls (in effect assuming a horizontally straight or nearly straight road).

The first (horizontally controlled) speed profile is developed in a manner similar to that demonstrated for the automobile (Figure 8). The primary differences are the use of average speeds 8 km/h (5 mph) lower than the corresponding automobile speeds for both the top average speed and the permissible average speed on horizontal curves and the deceleration and acceleration characteristics given in Figures 5 and 6.

The second (vertically controlled) speed profile is developed in the manner shown in Figure 9.

The final truck speed profile is then developed by the combination of the two as shown in Figure 10. The lower line of the combination, which is formed by superimposing the separate profiles, is the resultant truck speed profile and is highlighted by the shaded line.

An application of the speed profile to an actual problem involving an approximately 11-km (7-mile) section of highway is shown in Figure 11. The horizontal and vertical alignments of an existing highway (except for the dashed lines at points a, b, and c) are shown in the two upper blocks of the figure. The combination of the variable horizontal curvature based on the original design speed of 80 km/h (50 mph) and the undulating profile produces highly variable speeds as shown in the third block of the figure. This speed profile (shown for travel in one direction only) reveals the inconsistency of the design and its resulting operations. The speed profile for automobiles indicates variations that require a series of decelerations and accelerations, with changes in average speed of as much as 25 km/h (15 mph). The truck speed profile is even more erratic and shows critical variations not only in truck speeds, but also in the speed differential between automobiles and trucks.

The trouble spots can be easily found by examining the speed profile. The apparent problems are the inability of the alignment to achieve an appropriate design speed and consistent operation for automobiles and the effect of the combination of the horizontal and vertical alignments on truck operations. The lack of conformity to the 15-km/h (10-mph) rule is obvious, as are the locations of the relatively rapid changes in speed and those locations where the speed differential between automobiles and trucks exceeds 15 km/h (10 mph) [at several locations these differences are approximately 40 and 50 km/h (25 and 30 mph)].

Some situations are of particular concern. For example, between metric stations 43 and 50 (stations 140 and 165), trucks are rapidly decelerating, but automobiles are accelerating. Such a condition is very hazardous, particularly in a case such as this where the maximum speed differential between the two types of vehicles is nearly 45 km/h (28 mph). There is another undesirable situation between metric stations 91 and 93 (stations 300 and 305) where automobiles are decelerating to negotiate a horizontal curve, and trucks are continuing to accelerate on a downgrade.

These are some of the more obvious points of concern. A methodical study of the speed profile can provide a thorough insight into the operational characteristics of the highway and spot inconsistencies in the design and potentially hazardous locations. (Although it is ignored here, a plot of a sight-distance profile together with the speed profile can add to the perception of the problem and provide a more thorough diagnosis and evaluation.)

The use of the speed profile on an existing highway immediately shows the means for remedial measures. Any number of improvements can be suggested and depend largely on socioeconomic considerations. A rather minimal improvement that complies with the design principle of the 15-km/h (10-mph) rule is shown in the same figure. These improvements would be the realignment of the three horizontal curves (at a, b, and c) from a design speed of 80 km/h (50 mph) to a design speed of 95 km/h (60 mph) with a change in radius of from 230 to 350 m (750 to 1150 m) and the construction of truck-climbing lanes at locations d and e (approximately) metric stations 42 to 54 and 73 to 98 (stations 137 to 178 and 238 to 322).

The revised speed profile incorporating these improvements is shown in the bottom block of Figure 11. This profile shows compliance with the 15-km/h (10-mph) rule and that the design-speed designation has been changed from 80 to 95 km/h (50 to 60 mph), which provides a more consistent design and uniform operation. The speed variance of automobiles is within 15 km/h (10 mph), and truck speeds on common lanes are no more than 15 km/h (10 mph) lower than the speeds of automobiles. A similar speed profile was developed for travel in the opposite direction and showed the need for further profile adjustments.

SUMMARY

A key to consistent design and built-in operational uniformity for highways has been a goal of designers for the past half century. The principal guideline that has served as a common denominator in design has been expressed through the use of the design speed.

In this paper, the basic principle and intrinsic value of the design-speed concept as it has been used since the early 1940s is maintained as a general guideline, but the scope has been broadened to provide increased sensitivity toward design. The tool used for the updated design approach is the 15-km/h (10-mph) rule. During periods of free-flow conditions, when the design is most meaningful, this rule entails

1. Avoiding design-speed reductions along the highway except where the environment naturally and logically allows for them and maintaining these reductions at or below 15 km/h (10 mph),
2. Maintaining potential automobile speeds along the highway within a given design speed that should not vary more than 15 km/h (10 mph), and
3. Maintaining potential truck speeds at a value no more than 15 km/h (10 mph) lower than automobile speeds on common lanes.

The use of the new design-speed concept requires the application of a special tool—the plotting of a speed profile. The potential automobile and truck speeds are plotted along the proposed highway improvement, with separate plots for each direction, taking into account the joint configurations of the horizontal and vertical alignments and the individual curvatures and gradients.

A complete procedure for the development of speed profiles for free-flow conditions is described. The numerical values of speed and speed change for various conditions were derived from limited data, extensions of minimal research in vehicle operations, empirical speed relations, and experience judgments, but despite lack of direct research in some aspects of the problem, the results are believed to be reasonably accurate for their purposes and can be used effectively immediately.

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Communicative Aspects in Highway Design

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The operation of a vehicle is a complex process for the driver. To ease the task and to improve operations and safety, it is necessary to incorporate communicative aspects in highway design, i.e., to make clear to the driver the messages conveyed by the facility. Numerous geometric and control features that have been formulated in response to human-factors inputs are presented in this paper. Particular attention is directed to various features of design. Among these are alignment, sight distance and cross-sectional features, and operational uniformity, route continuity, and marking and signing. The suggested guidelines permit immediate application and are a starting point for improving design criteria on a larger scale. They could significantly improve the operational efficiency and safety on both existing and new facilities, but the designer's input—his or her philosophy and skill—must play an important role in meeting the objective of achieving optimum design.

Operating on today's highways is a complex process that involves the driver in many intricate and overlapping tasks of sensory detection, perception, analysis, and decision, which must be followed by responses of vehicle control, guidance, and navigation. The present highway and traffic environment is not fully adapted to the makeup of the driver. Yet, there has been some encouraging progress in highway design in considering the combined effects of driver, vehicle, and roadway.

Much attention has been directed to the study of human factors during the past several years, but extensive application of these factors has not yet taken place effectively. The highway and traffic engineer has not always taken advantage of this relatively new field, sometimes waiting for further research, but more often hesitating because of the natural tendency to maintain the status quo. However, while the field of human factors as related to the driver does need more research, there is already considerable knowledge that could be used in the design of highway geometrics and traffic-control devices.

Alexander and Lunenfeld (1) have developed a systematic approach and a procedure to identify a driver-performance level for what they term positive guidance—the selection of an appropriate and safe speed and path on the highway. (Their guidelines are primarily oriented toward immediate cost-effective improvements of existing trouble spots, but the principles apply equally to the rehabilitation of whole facilities and the design of new highways.) This paper presents specific design measures that respond to positive guidance and provide communicative features and is intended to complement their paper. Its broad objective is to make the highway, through design and control devices, responsive to the characteristics and needs of drivers so that they can comprehend and interpret the facility with comfort, efficiency, and safety.

Driver characteristics, such as height of eye, perception-reaction time, deceleration, lane-changing behavior, maintenance of headway between vehicles, and other aspects of operational behavior, have been used for many years as inputs to highway design (2). This has given guidance in the establishment of appropriate geometric standards, but further design improvements will require a better in-depth understanding of driver behavior and the reasons for it. Not only the physiological characteristics of the driver, but also his or her psychological and emotional makeup has a significant role in how to design a highway and control its operation.

The aspect of the human element in design requires more than the consideration of driver behavior and characteristics. It also involves the designer and his or her philosophy and skill. The designer applies standards, chooses the minimum or desirable values, considers the human factors that relate to the driver, and develops the composition of the facility in three dimensions. The designer's philosophy, attitude, awareness, and concern, as well as capability, are significant in determining the operational features of the highway. The concern for the well-being of the driver, the designer's (and the designing agency's) compassion for him or her, can dramatically affect the quality of the design. Thus, the effective use of the human element provides an output of proper, efficient, and safe design. This paper first explores the various inputs of driver behavior and characteristics and then presents a series of key features as design outputs that are either not covered by present standards or are used partially or in a limited way (3, 11).

HUMAN-FACTORS INPUTS

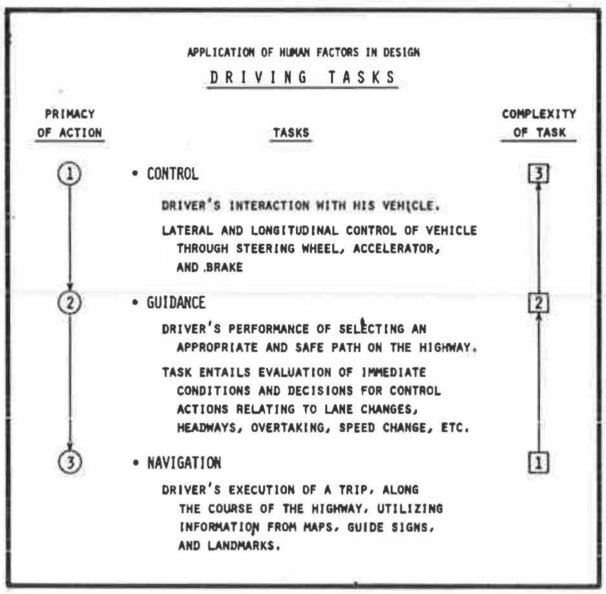
The reason for using human factors is to design facilities that operate more efficiently and safely. There is an immediate need to upgrade or supplement those elements of existing standards that do not respond to the more in-depth knowledge of driver behavior that is now available.

The design objectives, as outlined below, are threefold:

1. To compensate in the design for any momentary or temporary impairment of the driver due to his or her physiological and psychological state (anxiety, confusion, frustration, fatigue, monotony, or effects of alcohol, drugs, or illness);
2. To incorporate design features on the facility that meet driver expectations; and
3. To design the highway, geometrically and in coordination with control devices, so as to reduce and simplify the driving task.

In applying driver-behavior factors to performance, the total driving task should be considered as consisting of three subtasks—control, guidance, and navigation—as shown in Figure 1 (1, 5, 6). These subtasks frequently are not performed independently. At any given time, the driver is confronted with a multitude of information from a variety of sources. He or she must sift through this information, determine its relative importance, interpret it, decide on a course of action, and take that action within a limited time period. Under these circumstances the driver may experience anxiety, confusion, and harassment that can impair performance and affect safety. The driver will perform most efficiently and best if there is only one task at a time. Thus, the separation of the three types of tasks along

Figure 1. Driving tasks to be considered in design.



the course of the highway is a major objective in design. The designer requires knowledge of the performance levels of the driving tasks and the complexity and priority of action of each.

Another important design objective is that of meeting driver expectations. A driver develops a set of expectations of what the roadway will be like through experience (4). Thus, the highway conveys a certain message that he or she must interpret correctly. A list of some of the more common expectancies relating to through driving and turning maneuvers is outlined below.

1. The number of through lanes approaching and leaving a given area will be the same.
2. At a division point, the most important route will have the most lanes.
3. The most important route will be the most direct.
4. After the driver has entered a curve, there will be no speed reduction on it.
5. A speed reduction will be required to safely negotiate a connecting roadway between two major facilities.
6. All freeway exits will be on the right.
7. Left turns onto an intersecting roadway from an arterial street will be made from the left-hand lane.
8. To go to the right on an intersecting freeway, a driver will turn right in advance of the interchange structure.
9. Right turns are made from the right-hand lane: The driver will move to the right-hand lane and then continue to the desired intersection or exit point.
10. What appears to be a regular lane on a highway will be continuous and not be dropped.

Expectancies occur in all three parts of the driving task, and the configuration of the highway and its various elements should allow the driver to visually interpret the highway conditions and to accurately predict the maneuver, which should be reasonably consistent with what he or she normally would anticipate.

The third significant design objective is the recognition of and compensation for driver inherent characteristics (8). Drivers as a group have certain tendencies and desires, some of which are listed below.

1. Drivers desire and tend to travel at relatively high speeds, upward of 80 km/h (50 mph), where deterrents are few and free-flow characteristics are present.
2. Drivers entering and leaving curved roadways do so by negotiating a transitional path.
3. Drivers exiting and entering high-speed highways, via a turning roadway or ramp, do so by a direct and gradually diverging or merging maneuver.
4. Drivers traveling along a variable alignment tend to speed-up when the quality of the alignment improves.
5. Drivers tend to overdrive crest vertical curves on favorable horizontal alignments.
6. Drivers tend to overdrive turning roadways.
7. Drivers lose their sense of speed on long, sustained driving and tend to overdrive situations that require speed reductions.
8. Drivers orient themselves and choose their paths by following delineating features on or along the side of the highway.

Here, too, driver misjudgment, anxiety, frustration, fatigue, and monotony can be mitigated by appropriate design. Where the design fits the manner in which the driver operates—his or her inherent behavior and characteristics—there is a definite improvement in operational efficiency and safety.

By compensating for probable driver impairments, meeting driver expectations, and accounting for driver inherent characteristics in the design process, the driving task can be greatly reduced. However, there are a number of measures that are not necessarily covered by the above considerations that can also enhance driver-response capabilities and improve communicative operating conditions. The use of a three-dimensional approach considered dynamically from the driver's view can be the means for simplifying the operations. A large part of the answer to reducing the driver's task is simply to make it simple.

DESIGN OUTPUTS

When the human-factors inputs are coupled with a sensitivity on the part of the designer to achieve optimum operating conditions within the available resources and constraints, it is possible to upgrade the geometric features and the informational system of a highway much beyond what past practices have produced. The design features and guidelines given here are a series of measures and design elements that are derived largely on the basis of human-factors requirements. The list is not all-inclusive, but does cover many of the major design areas and is directed toward minimizing operational system failures. Most of the items are related to the visual or the communicative aspects of the highway, the messages conveyed by the facility, which allow the driver to relate to it with the minimum need to process information.

Design-Speed Application

A design speed is selected and used in an attempt to achieve a uniform design, primarily through correlation of the various features and elements of the highway that control or influence vehicle operation (2). The application of design speed should include those human-factors aspects that provide consistency in design.

The design-speed concept was well established during the 1940s and has enhanced design significantly, although its application under current conditions has produced some problems. The primary difficulty occurs on intermediate and low design-speed facilities on which variations in operating speed caused by changes in alignment cause inconsistent speeds. This inconsistency and its associated hazard are caused by the tendency of drivers to increase speed on flatter portions of the alignment, which then requires them to decrease speed on approaching sharper or controlling curves. Another problem is that the design speed is not always compatible with the driver's expectation and judgment.

Figure 2. Alignment coordination to more accurately convey highway form.

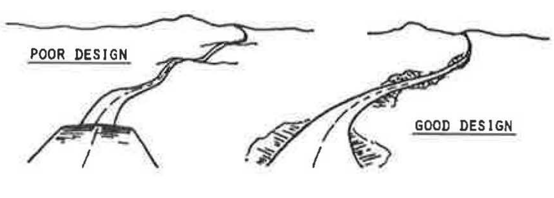


Figure 3. Balance and compatibility of horizontal alignment and vertical profile.

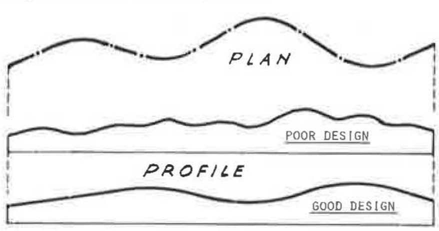


Figure 4. Preferred horizontal alignment.

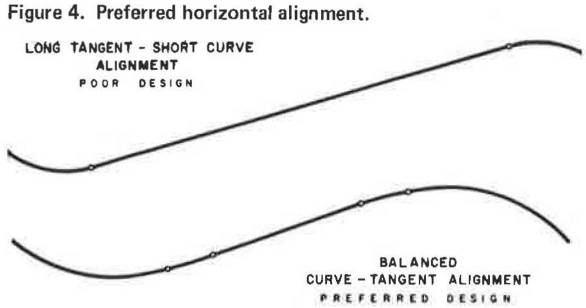


Figure 5. General guide for coordination of horizontal and vertical alignments.

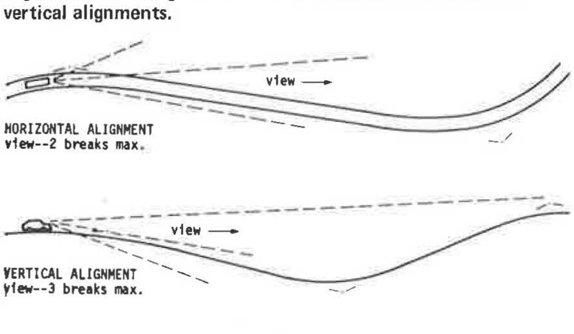
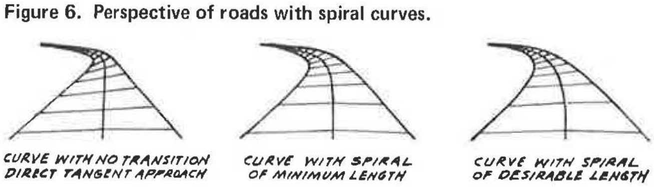


Figure 6. Perspective of roads with spiral curves.



Minimizing these two problems in the present use of the design-speed concept is an important objective. An updated concept and application of design speed that attempts to reach this goal by the use of the 15-km/h (10-mph) rule as a design principle is given by Leisch and Leisch in the preceding paper.

Alignment Design and Coordination

Coordinating the horizontal and the vertical alignments of a highway not only enhances its aesthetic quality, as shown in Figure 2, but also provides a positive means to more accurately present the shape and character of the highway to the driver proceeding along it. The smooth-flowing quality and avoidance of sight loss (the disappearing and reappearing of the road) are desirable goals.

A driver's foreshortened view of the roadway ahead gives him or her specific information on conditions that will soon be experienced. If a driver becomes apprehensive about an apparent condition seen some distance ahead, but then finds on arrival at the spot that the apprehensions were unnecessary, he or she must conclude that the visual information received previously was false. Such experience repeated many times at different places breaks down the visual communication from the highway to the driver, who then tends to lose his or her ability to judge, to orient, or to make valid choices. Thus, the messages conveyed by the facility must be consistent with its operational characteristics. This requires a design that provides a semblance of visual accuracy to the real character and condition of the roadway as it is viewed by the driver (8).

Although ideal coordination cannot always be achieved because of other controlling features, a degree of coordination usually can be achieved. The success in this depends largely on how this feature is considered, tested, and evaluated during the location stages of the design process, although once the location is selected, further adjustments and refinements to improve coordination can be made during the functional and preliminary design stages.

The proper combination of the horizontal alignment and the vertical profile, to present an accurate picture and improve driver confidence, can be made by following several general principles. The horizontal and vertical alignments should balance the horizontal and vertical portions of curves and tangents of somewhat similar length, as shown in Figure 3. Moreover, the vertices of the horizontal and vertical curve elements should be in general proximity.

Another feature to be considered is that of the relative proportions of tangent and curve elements on the horizontal alignment. A long tangent and short curve arrangement should be avoided: Figure 4 illustrates a treatment that produces more uniform operation and better aesthetics. To reduce the probability of misleading a driver who is negotiating a curved alignment, a sharp curve should not be preceded by a long flat curve unless there is an effective transition treatment. Individual elements of the vertical alignment also should maintain a balance; however, long tangent grades are often appropriate, if they do not reduce speed unduly and if the vertical curves do not obstruct the view of subsequent horizontal curves.

The use of these principles and special alignment models and computer graphics can produce reasonably good results. An additional general guide that can be used to coordinate the alignment and the profile, visualizing the whole in three dimensions, is given in the Federal Highway Administration seminar notes (8), where it is suggested that the likely (although not necessarily) maximum number of breaks or changes in the course of the longitudinal line should be no more than two horizontally or three vertically. This guideline is illustrated in Figure 5.

Accidents on curved portions of roads and skidding on curves have led to a further examination of curve-design practices. The loss of friction on roadway pavement surfaces with time is one problem, and the centripetal acceleration and balance of forces as the driver proceeds into and out of a circular curve are another. The manner in which superelevation is developed preceding a circular curve and the speed of approach by the driver, which is sometimes excessive, may combine with the substandard frictional quality of the pavement (particularly when it is wet) to produce an accident-prone situation. Steering adjustments on entering the curve, particularly when combined with braking, may cause an unbalance of forces that results in a horizonal skid.

Because a driver naturally steers a transitional path on entering or leaving a circular curve, a transition or spiral curve is desirable in combination with the circular curve, to fit the driver's inherent operational behavior. The spiral further acts as the element through which superelevation is developed. Thus, the combination of these two features allows the driver to operate gradually with minimum steering effort and maximum comfort, which reduces the probability of skidding. The transitional design also allows the driver to reduce speed smoothly over the length of the spiral should his or her approach be too fast for the circular curve.

The use of spirals improves the driver's operation and comfort, and makes steering easier and more accurate. Spirals also produce a smoother appearing transition that is more accurate to the character of the alignment and to the performance that the driver expects to experience. Perspective sketches of approach views to curves with and without spirals are shown in Figure 6.

Operational experience and engineering judgment clearly indicate the need for the use of spirals. However, the spiral lengths currently used are apparently too short. These lengths should be increased from the 60 to 90 m (200 to 300 ft) used at present to 120 to 180 m (400 to 600 ft) to take advantage of the transitional path provided for comfort, safety, and appearance. Recommended minimum lengths of spirals for sharp (minimum or near-minimum radii for the indicated design speed, i.e., the lowest 30 percent of radii) and flat (maximum radii on which spirals are first introduced at indicated design speed, i.e., highest 30 percent of radii) curves in relation to design speed are given below (1 m = 3.3 ft; 1 km/h = 0.62 mph).

Highway Design Speed (km/h)	Minimum Spiral Length (m)	
	Sharp Curve	Flat Curve
50	60	45
65	90	60
80	120	75
95	150	90
110	180	120

Sight Distance

Sight distance—the ability of the driver to see the road ahead—is probably the most important individual design feature from the standpoint of safety. Recently the American Association of State Highway and Transportation Officials (AASHTO) issued a new policy on stopping sight distances that provides larger values (as more desirable criteria) for design (2). These are defined as the desirable minimum required continuously along the highway and are shown below (1 m = 3.3 ft; 1 km/h = 0.62 mph).

Design Speed (km/h)	Sight Distance (m)		
	Stopping	Anticipatory	Passing
50	60	180	340
65	90	250	460
80	160	340	550
95	200	460	630
110	260	600	750

The complexity of operating on modern highways and the need to frequently process information on them require even longer sight distances at certain locations. This distance, the anticipatory sight distance, is particularly important in areas of potential hazard and at points requiring driver decisions (8). Such areas or points may involve intersections, interchange exits, lane drops, railroad grade crossings, drawbridges, toll-collection booths, or zones of design-speed reduction. This distance may also be referred to as the decision sight distance and is defined as the distance at which a driver can detect a hazard signal in an environment of visual clutter, recognize the threat of the potential hazard, select an appropriate speed and path, and perform the required maneuver safely and efficiently (1).

The anticipatory sight distance is not yet part of the AASHTO design criteria, although the concept of longer sight distances under such circumstances is certainly implied. Preliminary investigations have indicated that the distances are approximately 2.5 to 3 times the stopping sight distances, i.e., on highways having higher design speeds they are in the range of 460 to 750 m (1500 to 2500 ft). Their values, measured to the roadway surface, and their relations to other forms of sight distance requirements are shown above. A significant implication of the anticipatory sight distance is that these extra long distances would be provided only occasionally as needed for particular conditions. They would be provided during the location study stage and would be designed in a natural manner wherever feasible so as not to require undue cost; i.e., frequently the feature itself would be positioned where the extra sight distance was already available.

Longer stopping sight distances may be required in the operation of large trucks. There are circumstances where the increased height of eye within the truck has no advantage, i.e., on the inside of a horizontal curve with near-vertical obstruction, on steep downgrades, and particularly on combinations of the two. Stopping distances for large, loaded trucks may vary from 1.5 to 2 times those needed for passenger cars. The suggested stopping sight distances for trucks, measured from an eye height of 1.8 m (6 ft) to 15 cm (0.5 ft) above the roadway, are shown below (1 m = 3.3 ft; 1 km/h = 0.62 mph).

Design Speed (km/h)	Stopping Sight Distance (m)	Design Speed (km/h)	Stopping Sight Distance (m)
50	90	95	270
65	160	110	360
80	180		

Cross-Sectional Delineation

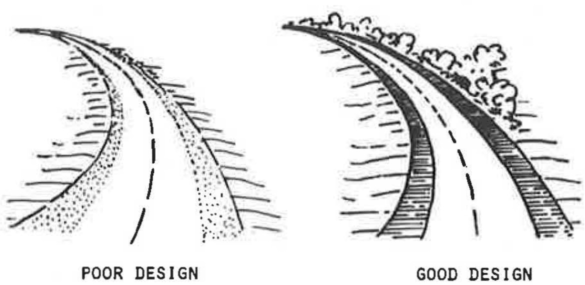
A distinctive cross section that communicates the character of the roadway clearly and accurately to the driver will give him or her valuable information about the facility ahead. This information can be transmitted to the driver most effectively by delineating the edges of the travelway and all of the significant elements in the cross section.

Delineation and contrast can be effected by the use of materials with different textures and colors on the various elements, by the application of pavement striping, and by variation in cross-slope surfaces, and is sometimes supplemented by outer curbing and delineators or other guide devices. Variations in roadside grading and planting can also assist in driver orientation and transmit the message conveyed by the facility. This is illustrated in Figure 7.

Delineation can clarify the three-dimensional form of the roadway as it unfolds before the driver and help him or her to gauge the rate of travel and the directional position. Variable landscape treatments and changing the form of the roadside grading to present a different appearance at each point along the roadway assist the driver in judging speed, position, and change of direction as he or she progresses along the facility. Such treatments provide the driver with a sense of the appropriate speed, introduce interest without distraction, and on long, sustained trips, tend to obviate drowsiness and monotony.

With respect to construction and maintenance practices, delineation may be a troublesome feature that adds to the cost of the facility. However, such cross-sectional refinement clarifies the three-dimensional form along the highway for the driver, orients him or her, and eases the driving task. Thus, the message conveyed by the facility can be expressed effectively by a properly executed cross-sectional delineation.

Figure 7. Delineation features for cross-section design.



Operational Uniformity and Interchange Design

Freeway and crossroad interactions depend on the configuration and arrangement of interchanges. In general, interchanges along most freeways have an inconsistent operational pattern—some have two exits from each freeway approach, some have one exit, some have left-hand ramps, and some have exits in advance of and some beyond the crossroad structure. This lack of uniformity creates an inconsistent pattern of signs and may cause undue lane changing and produce erratic maneuvers.

To increase efficiency and safety, interchanges should be patterned to provide an operational uniformity to meet driver expectancy (9). The arrangement of exits and entrances along a freeway or high-quality highway should be consistent, and the pattern of directional signing should be uniform.

Operational uniformity can be achieved by providing, at each interchange along the freeway route, a single

Table 1. Recommended values for spacing of entrance and exit ramps.

Configuration	L[a] (m) Desirable	Adequate	Absolute Minimum
Entrance-entrance or exit-exit			
Freeway	450	350	300
Collector-distributor road or freeway distributor	350	300	250
Exit-entrance			
Freeway	225	175	150
Collector-distributor road or freeway distributor	175	150	125
Turning roadways			
System interchange	350	300	250
Service interchange	300	250	200
Entrance-exit (weaving)			
System-to-service interchange			
Freeway	900	750	600
Collector-distributor road or freeway distributor	600	550	450
Service-to-service interchange			
Freeway	600	550	450
Collector-distributor road or freeway distributor	450	350	300

Note: 1 m = 3.3 ft.
[a] L = length defined in Figure 9.

Figure 8. Operational uniformity through consistent arrangement of successive exits.

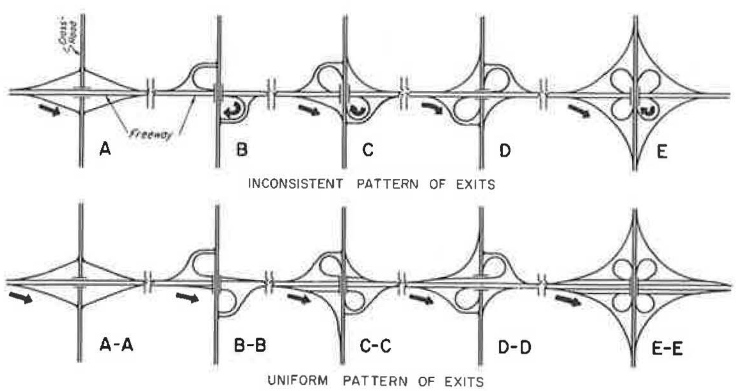
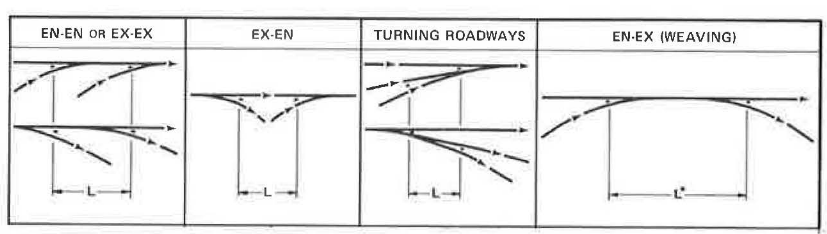


Figure 9. Configurations for entrances and exits.



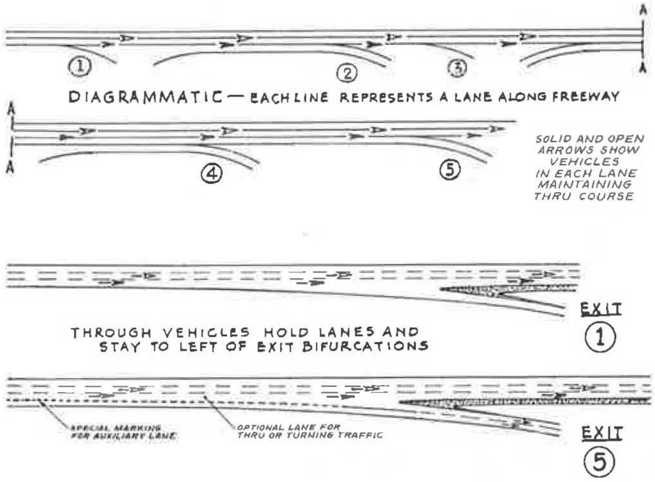
right-hand exit in advance of the crossroad, as illustrated in Figure 8. Such uniformity would require the construction of new freeways with this feature and the gradual replacement of outmoded interchanges by new ones that conform to the pattern. This would involve a long-range program, but its implementation would improve operations and safety. The anticipatory aspects provided by the visual and communicative features of a consistent exiting configuration would contribute significantly to the driver's confidence, assurance, and comfort.

Another feature of operational uniformity is that of the spacial relations, the sequencing and spacing of exits and entrances, along a route. The operation of the Interstate system throughout its development has demonstrated the need for greater spacing. Suggested values for new designs and for reconstruction of existing facilities that are built to the configurations illustrated in Figure 9 are given in Table 1.

Another communicative aspect in interchange design is the question of whether the crossroad passes over or under the major facility. The crossroad-over arrangement is much more favorable. The operational advantages of this configuration are more than simply the assistance of gravity for deceleration on up-ramps and acceleration on down-ramps. The advance view of the grade-separation structure and much of the exit ramp allows the driver exiting from the highway to fully appraise the situation and prepare for the necessary maneuver, and the driver on the entrance ramp has a good view of both the ramp and the freeway that enables him or her to pick a gap in traffic for merging, which provides visual assurance and the ability to perform more effectively.

For crossroads along highways in rural environments, particularly in sparsely settled areas where long trips at high, sustained speeds are common, the over arrangement also helps to lend interest and minimize driving monotony.

Figure 10. Operational characteristics with route continuity.



Lane Balance

The arrangement of lanes on facilities having diverging, merging, and weaving traffic, referred to as lane balance, is significant in aiding the driver in all levels of the driving task—vehicle control, guidance, and navigation. Lane balance is mandatory for achieving smooth operation, reducing lane changing to a minimum, and clarifying the paths to be followed. It also involves the addition of auxiliary lanes and the loss by the associated lane drops (10).

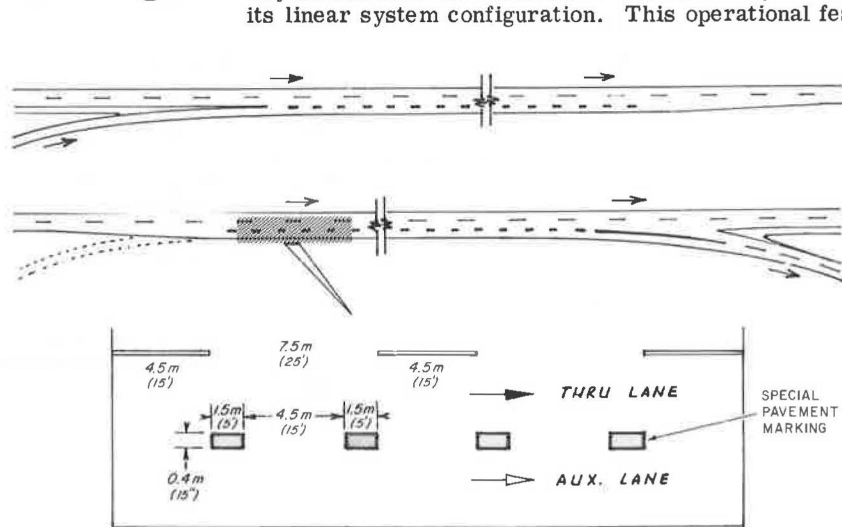
The lane relations at exits require one more lane going away than the combined number of lanes on the freeway preceding the exit. The arrangement of lanes at entrances should be such that the number of freeway lanes beyond the merge should be equal to the sum of the lanes preceding the merge or this sum minus one.

The use of auxiliary lanes to balance the traffic load and maintain a more uniform level of service on the highway is a relatively new technique. Such added lanes facilitate the positioning of drivers at exits and in bringing them onto the highway at entrances. Thus, the concept is very much related to route continuity and signing. An auxiliary lane has the potential for trapping a driver at its termination point or where it is continued onto a ramp or turning roadway. Consequently, the driver should be made aware when he is in or adjacent to an auxiliary lane, which should be marked in a special manner, as shown in Figures 10 and 11. The flexibility of operation with lane balance, especially as provided by the optional lane, is shown in the bottom illustration of Figure 10.

Route Continuity and Designation

To keep the driver, particularly one who is unfamiliar with the highway, on his or her desired route, the freeway should have the feature of route continuity built into its linear system configuration. This operational feature

Figure 11. Suggested lane markings for auxiliary lanes.



Note: 1 m = 3.3 ft.

is the provision of a direct path along and throughout the length of a designated route—the designation pertaining to the route number or to the freeway name (9).

Route continuity is achieved by observing operational uniformity (all exits follow the same pattern), by maintaining lane balance, and by favoring the through-route characteristic (irrespective of volume splits at bifurcations) for the designated route. Its application is shown by comparative examples in Figure 12. This arrangement automatically provides complete lane continuity and allows the through driver to maintain a lane position throughout the entire route, which provides constant

Figure 12. Route continuity.

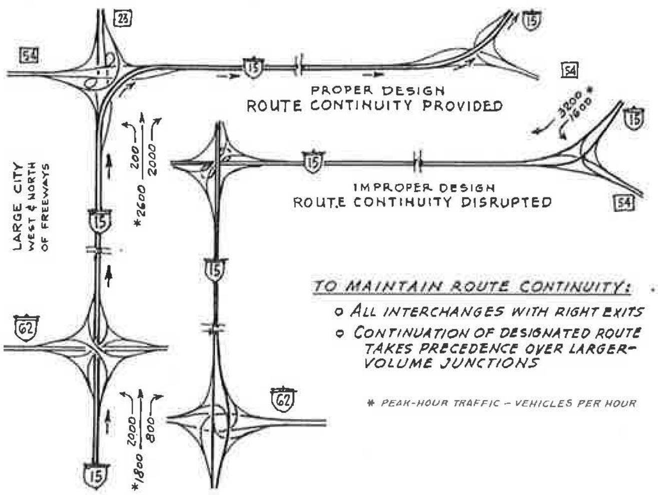


Figure 13. Route-designation configurations.

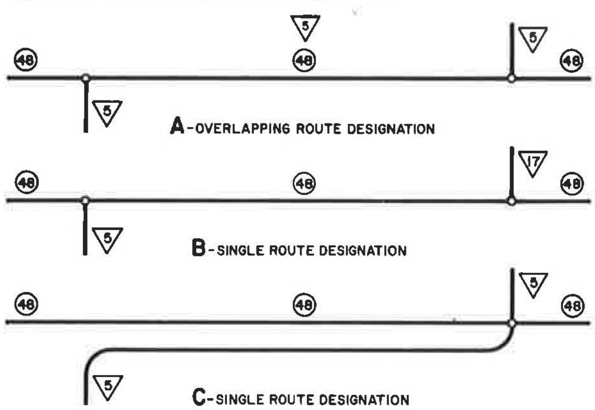
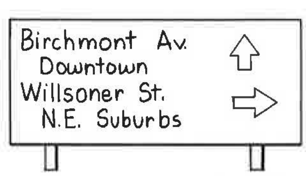


Figure 14. Signing for clarity and easy comprehension.



A. SIGN PROVIDING POOR READABILITY

B. SIGN PROVIDING GOOD READABILITY

Figure 15. Directional sign posting on rural freeways.

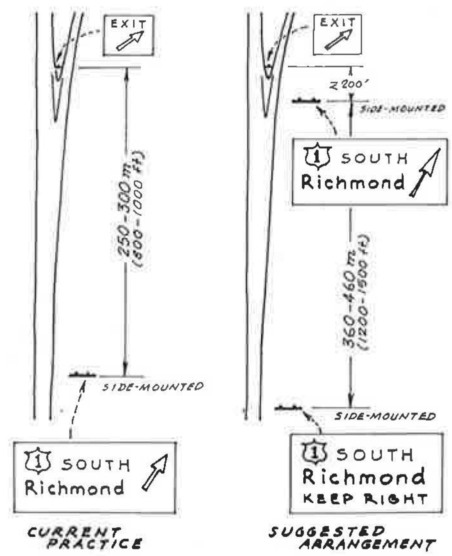


Figure 16. Directional signing for interchange with two-exit design (example).

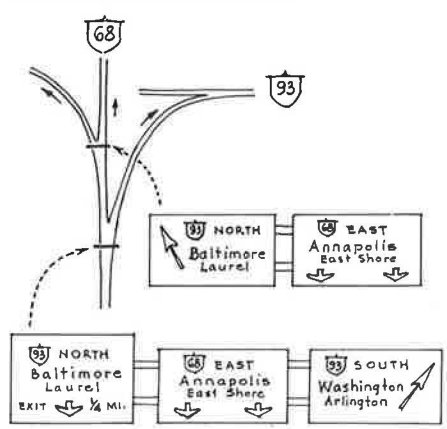


Figure 17. Directional signing for interchange with single-exit design (example).

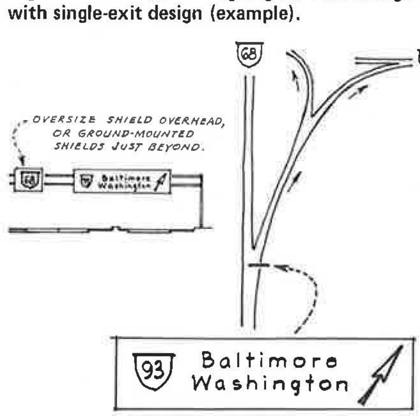
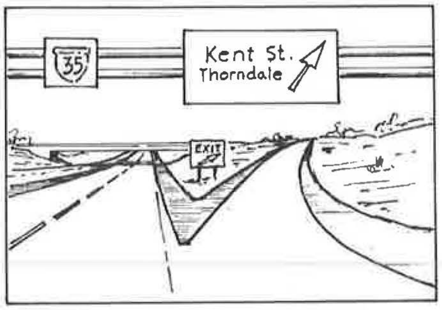


Figure 18. Coordination of geometry and signing at freeway exit.



visual assistance. As shown in Figure 10, the driver would need to diverge or change lanes only when choosing to exit from the designated route.

Route continuity significantly assists the driver in the driving task by eliminating lane changes for through drivers (except for passing) and by reducing lane changes and hazardous maneuvers for exiting traffic. The driver can operate with greater confidence, minimized anxiety, and the elimination of the elements of surprise and indecision.

A problem associated with route continuity is the need to redesignate and renumber various highway systems. Some numbers are now superfluous and should be dropped or reassigned. The overlapping of route numbers (roads that carry more than one number) on a given facility should be eliminated. This is illustrated in Figure 13. Although this will be a difficult task, the problem should eventually be resolved to clarify and simplify operations.

Marking and Signing

To perform guidance and navigational tasks, the driver receives important information from control devices, such as marking and signing. Although there are highly effective control devices with a large measure of uniformity in current use (12), there are some features that need review and updating in the light of human-factors applications.

The lane markings of auxiliary lanes and the associated lane drops should be in conspicuous contrast to normal lane lines. The special marking suggested for this purpose (Figure 11) consists of a highly conspicuous lane separation made up of 2.5-m (5-ft) long by 40-cm (15-in) wide painted elements. The message conveyed by such markings would soon become obvious to the driver. The basic lanes, those continuing through on the facility, would always advise the through driver to stay to the left of the marking; the exiting driver would be told to position him or herself to the right of the marking; and the entering driver would be alerted to the necessity of crossing this marking to continue on the highway. This marking provides the driver with prior information and couples it with a visual indicator for positive guidance.

The design, composition, and placement of signs, although well standardized, need some upgrading for clarity and comprehension. There should be more effective separation, grouping, and accenting with variable letter sizes and weights of the various message units on a sign. An example of what could be done is shown in Figure 14. Another area that needs improvement is that of the number of sign panels at a given location and the separation of panel positions.

One weakness of standard directional sign posting in rural and suburban areas is the separation of the destination sign from the physical point of exit, as shown on the left of Figure 15. Here, the sign may be obscured from the driver in heavy traffic by trucks or by momentary inattention, which will create indecision, anxiety, or confusion for him or her. Or, at night the driver may be misled by the directional arrow and turn off onto the roadside in advance of the exit. The recommended arrangement is to provide an advance-warning sign that does not have an arrow and a second sign with an arrow in the vicinity of the gore to give the driver, through the combination of signs, positive guidance for the exit. This is shown on the right of Figure 15.

Figures 16 and 17 show the advantages of providing operational uniformity by the use of single-exit designs on the right (rather than two-exit designs and possibly left-hand ramps). The reduction from five signs on the freeway to only one improves the informational system and simplifies and reduces the driving task.

The information on freeway exit signs should be placed so that it integrates with other information that the driver will use to make the decision. This frames the routing information and the geometric and marking information within his or her visual field. For example, as shown in Figure 18, the placement and design of the exit ramp, its nose, and the signing and striping should all direct the driver toward the desired destination in a positive, reassuring, and consistent manner.

The amount of navigational information supplied to the driver is frequently too much to cope with at one time. Thus, there is a need to reduce and break down the information. This can be done by minimizing the number of message units per sign and by reducing the number of panels at any one location, as suggested in the guideline given below.

Sign Panels at Location	Frequency of Use	Max Message Units per Sign		Max Message Units per Assembly	
		Desirable	Absolute	Desirable	Absolute
1	Frequently	5	6	5	6
2	Occasionally	4	5	8	10
3	Special case	3	4	9	11
4	Never	–	–	–	–

CONCLUSIONS

Although research in the field of human factors is continuing, there is already sufficient knowledge on the subject to permit its immediate application to the design and redesign of highway facilities and traffic-control devices.

Numerous geometric and control features and their response to human-factors inputs have been formulated and are discussed in this paper. Particular attention is directed to the features of design-speed application, alignment design and coordination, sight distance, cross-sectional delineation, operational uniformity and interchange design, lane balance, route continuity and designation, and marking and signing, and the way in which they are related to communicative aspects—the messages conveyed by the facility.

The guidelines suggested could be applied immediately and provide a starting point for improving design criteria on a larger scale and can significantly improve the operational efficiency and safety of both existing and new facilities.

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Characteristics of Trucks Operating on Grades

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The many changes in truck engine displacement and power have indicated a need to reassess current climbing-lane design practices. This study presents new data characterizing trucks (and combinations) on grades. Field data collected at several locations in central and east Texas were analyzed, and speed versus distance curves were developed for a range of grade profiles. From an evaluation of the speed versus distance curves for the designated critical-truck class, composite critical-length-of-grade charts were derived for an 88-km/h (55-mph) approach speed and a range of speed-reduction values.

The criteria currently used for the design of climbing lanes for trucks have been developed over the last four decades and are based primarily on theoretical formulations and limited field observations.

This paper presents the findings of a study that obtained new field data about the operating characteristics of trucks on selected grades and related these data to geometric design standards for highway grades, with particular emphasis on the capacity and safety aspects of vehicle climbing lanes. The result of the project was the development of revised design charts relating the length and percent of a grade to the performance of a vehicle on that grade.

Most of the previous research on truck hill-climbing ability has been directed toward measurement of the elements that affect the performance of the vehicle. The roadway conditions, including rolling resistance, have been studied by Taragin (1) in his theoretical equation, and traffic conditions have been studied by Schwender, Normann, and Granum (2). The current American Association of State Highway and Transportation Officials (AASHTO) policy for the design of truck climbing lanes is based principally on data collected in 1954, and most states currently use a modification that accounts for special state and regional characteristics (13).

The performance of a vehicle operating on a highway is a function of the numerous variables associated with the principal elements that govern vehicular motion. These elements are the vehicle itself, the roadway and the environment in which the vehicle operates, and the behavior of the vehicle operator. The identification and evaluation of these elements provided an additional framework for the field study. Each element was described and arrayed for analysis in the mathematical modeling phase of the study.

GENERAL EVALUATION OF VEHICLE GRADEABILITY

Because of the limited scope of the conventional force and energy equations, the mathematical models used in previous research have not been entirely successful in evaluating the effects of these variables in relation to actual vehicle performance. However, the models might be improved if new experimental data that represent the actual vehicle operating characteristics under a wide variety of roadway and environmental conditions were available (3). With this information, the performance characteristics of representative vehicles could be modeled, and the present design criteria for grades could be evaluated.

Collection of the field data necessary to adequately identify the operating characteristics of heavy vehicles on grades is a complex operation because of the numerous combinations of variables involved. However, a majority of the variables can be represented by field data from three major areas: (a) the pertinent physical characteristics of the vehicles under observation, (b) the speed versus distance profiles of the vehicles at selected field sites, and (c) the geometric and environmental external conditions under which the vehicles operate.

Table 1. Roadway parameters of test grades.

Roadway Parameters	Grades[a] A	B	C	D	E
Length of grade, m					
Approach	244	122	244	183	213
Grade section	747	762	1737	823	1006
Recovery area	152	107	244	213	183
Steepness of grade, %					
Approach	0	0	-0.5	-0.5	0
Grade section[b]	5	3.4	2.6	3.7	2.6
Recovery area	0	-0.5	-0.5	-0.5	-0.5
Horizontal alignment[c]					
Cross section					
Number of lanes	4	4	4	4	4
Lane width, m	3.35	3.35	3.35	3.01	3.35
Shoulder width, m	–	2.44	2.44	2.44	2.44
Median width, m	–	9.14	6.01	6.01	6.01

Note: 1 m = 3.28 ft.

[a] Grade A is on US-183, and grades B, C, D, and E are on US-59.
[b] Maximum average cumulative grade.
[c] All test sites have essentially straight horizontal alignments.

Vehicle Characteristics

There are many characteristics in the makeup of a vehicle that can directly influence its operating characteristics on a grade. The three principal ones are (a) the vehicle type or classification, (b) the gross weight of the vehicle, and (c) the power of the vehicle. It is not possible to evaluate every identifiable vehicle factor in an experimental program, but representative information about these three areas should be obtained.

Speed Versus Distance Profiles

The second major group of data needed in an experimental program should be the speed versus distance profiles of representative vehicles. From these profiles, relations between the vehicle weight, the percent grade, and the performance of the vehicle on the grade can be established. The speed characteristics that should be observed are

1. The speed of the vehicle on entering the grade,
2. The speed-reduction rate on successive sections of the grade (the deceleration rate),
3. The minimum steady-state speed reached during operation on the grade (the crawl speed), and
4. The acceleration characteristics of the vehicle on adjacent level or downgrade areas.

External Conditions

The third group of data needed should be records of the external factors that influence the operation of the heavier vehicles. The factors to be identified are the features of the roadway over which the test was run, the traffic and land use conditions at each test section, the climatic conditions at the time of each test, and the characteristics of the driver of each vehicle.

SITE SELECTION

The test sites were selected and characterized by the basic parameters associated with the roadway, the traffic, and the roadside. The search was concentrated in an area surrounding Austin, Texas, to minimize the cost. After an evaluation of several grades, several test sites were selected.

Roadway Parameters

Table 1 gives the roadway parameters for the grades selected. Each grade was divided into three distinct areas—approach, grade section, and recovery area—for the analysis of vehicle performance. (Ideally, the approach should be level so that the entry or approach speed for the vehicles will be relatively constant.)

The grade (test) section, over which vehicle performance was most closely monitored, begins where the grade actually begins, as determined from the profile plans, which were obtained from the Texas State Department of Highways and Public Transportation (SDHPT). The grade section is best described by the average cumulative grade (ACG) since this approximates the total effect of the grade on an ascending truck. The ACG is determined by computing the average grade for each 60-m (200-ft) section and then averaging this for the entire length.

Traffic Parameters

Speed estimates and average daily traffic compositions were obtained for each site. In 1972, 31 percent of the southbound vehicles on US-183 were combinations; the majority (80 percent) of them were five-axle, tractor semitrailer combinations. About 35 percent of the vehicles traveling on US-59 were trucks; of these, 42 percent (15 percent of the total vehicles) were combinations, and many carried logs or pulpwood.

Roadside Parameters

The conditions along the side of a roadway—the abutting land use, visual stimuli, and conflicting traffic flows—can indirectly or directly affect the operation of a vehicle by distracting or otherwise influencing its driver. At grade A, on US-183, the land on both sides of the roadway is agricultural, and there is a small gravel driveway at the top of the hill, past the study zone. There is a T-intersection with a seldom-traveled, farm-to-market roadway at the base of the grade, 90 m (300 ft) from the beginning, but this has little effect on the roadway since there is adequate stopping sight distance in both directions. There are no large signboards along the grade to distract the driver. The opposing traffic may influence vehicles in the inside lane because the four lanes are undivided, but all of the vehicles in this study were in the outside lane. Thus, it is assumed that the opposing traffic flow did not affect the vehicles tested on grade A.

On US-59, grades B and D have abutting land use that is agricultural. There is a light commercial area at the top of grade C that is visible from almost all points. Grade E has light commercial establishments along most of the grade, and there are crossovers in the median, and driveways and signs along all four grades. Thus, it is assumed that these interruptions and distractions will have some influence on driver behavior. Because of these differing effects on driver behavior, grade A was analyzed separately from grades B, C, D, and E.

DATA COLLECTION PROCEDURES

The techniques and instrumentation considered for this experimental program were evaluated by a number of criteria. In general, the equipment had to be relatively economical, practical for operation by project personnel,

Figure 1. Truck classification by frontal configuration.

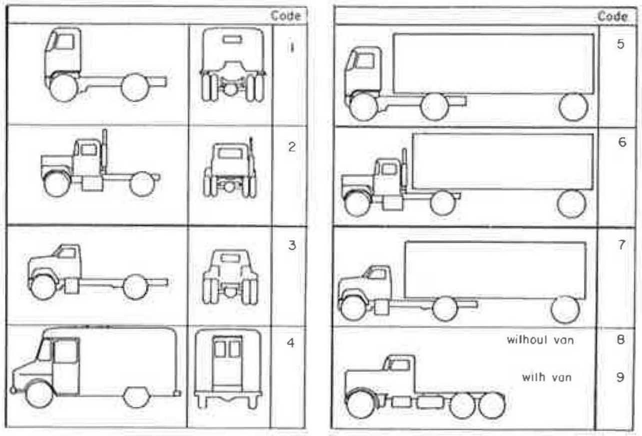
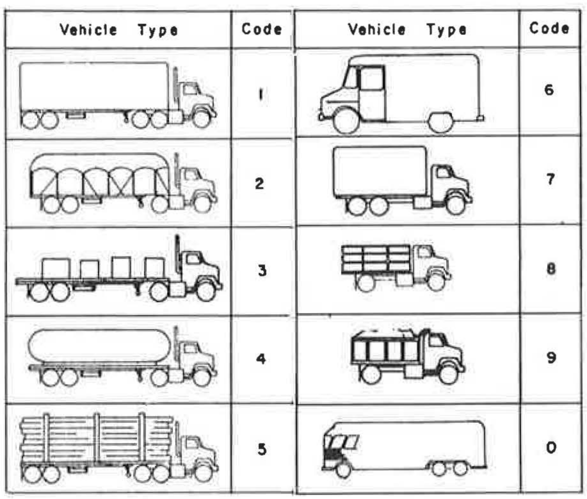


Figure 2. Truck classification by side configuration.



reliable, and accurate. It also had to be flexible and mobile enough to be used at different field sites and adaptable to the various conditions that might exist. As the system was going to be used for long periods of time, the availability of replacement equipment and the probable maintenance costs were also considered. Finally, the system had to be inconspicuous to the vehicle drivers so that the vehicles under test would be driven in a natural manner.

Speed-Reduction Profile

The three principal elements of data necessary for the construction of the speed versus distance profiles for each vehicle are (a) the speed reduction (deceleration) of the vehicle, (b) the entry speed for each grade, and (c) the minimum speed (crawl speed) reached on the grade.

The speed versus distance profile for each vehicle observed was determined by monitoring the time versus position relation for the vehicle as it progressed through the test section. Several systems were examined and evaluated (4) for possible use in obtaining this information. Two methods proved to be feasible: a photocell technique and an automobile-following method. A comparative study between the automobile-following technique and the photocell procedure showed that their accuracies were equal, but that the automobile-following procedure was much easier to adapt to local field conditions (12).

Physical Characteristics of Vehicles

The license numbers of the observed vehicles that were registered in Texas were submitted to the State Motor Vehicle Division (MVD) for a computer search of registration records. A typical registration printout gives the following useful information about a vehicle: (a) its empty weight, (b) its gross weight, (c) its age (model year), (d) its manufacturer, (e) its classification, (f) its identification number, (g) the name and address of its owner, (h) a number identifying its title on microfilm records, and (i) whether or not the engine is diesel (trucks). The microfilm records were then searched to determine the individual vehicle model numbers.

Vehicle Classification

From field observation and MVD records, each truck was assigned a particular classification according to its axle configuration. The following classes, which are based on SDHPT and AASHTO standards, were used:

Class	Description
Single-unit trucks	
SU-2A	2 axles, single wheels
SU-2D	2 axles, dual rear wheels
SU-3A	3 axles, all single wheels
SU-3D	3 axles, tandem dual rear wheels
Truck combinations	
2-S1	2-axle tractor, single-axle semitrailer
2-S2	2-axle tractor, 2-axle semitrailer
3-S1	3-axle tractor, single-axle semitrailer
3-S2	3-axle tractor, 2-axle semitrailer

However, because there are various models, types, loading configurations, and sizes within each axle class, each vehicle was also classified by its frontal and side configurations.

Figure 1 shows the classification by frontal configuration. The configuration is based on tractors pulling vans that are, on the average, 0.9 m (3 ft) higher than the cab and on tractors with trailers other than vans. Usually, other trailer combinations are lower than the tractor, which makes the frontal configuration of the tractor the governing factor.

Figure 2 illustrates the various side configurations. Since the vehicle lengths vary only slightly within a particular combination class, the side classifications are designed to include several axle groups each. The vehicle-type codes are described below.

Code	Description
1	Truck combinations with van trailers or cattle trailers
2	Truck combinations with trailers with stake sides
3	Truck combinations with flatbed trailers
4	Truck combinations with tank trailers
5	Truck combinations with logs
6	Single-unit truck vans
7	Single-unit, chassis-with-cab trucks with vans
8	Single-unit trucks with flatbeds or stake sides
9	Single-unit dump trucks (including concrete mixers)
0	Buses

Vehicle Gross Weight

A system that weighs vehicles in motion was used to determine the gross weight of each truck and truck combination in the field. This in-motion weighing system, which was developed by the Center for Highway

Research at the University of Texas at Austin and the SDHPT for use by its Planning and Research Division (5), uses special wheel-load transducers. Measurements of dynamic wheel forces are obtained and can be used to estimate the weight of a vehicle moving at speeds of up to 113 km/h (70 mph) with an accuracy within 10 percent. The use of this technique does not influence driver behavior, allows on-site weighing, and permits 100 percent sampling.

Basically, the system consists of two loop detectors and two loading pads placed in the outside lane of travel. The loops and scales are connected to a digital computer in a van beside the road. A display console with a special keyboard allows the van operator to classify each truck according to axle arrangement and type. The system information is stored on magnetic tape and later transferred to a computer listing.

The weighing-in-motion system produces the following information for each vehicle: (a) its speed, (b) its length, (c) the number of axles, (d) the distance between each two axles, (e) the gross weight of each axle, and (f) the gross weight of the vehicle.

Vehicle Power Rating

Gross power is the maximum power output of an engine without any encumbrances, and net power is the maximum power output of the engine as installed in the vehicle. Net power includes the effects of such encumbrances as the fan belt, alternator, water pump, and other standard accessories, and in recent years, any emission-control devices. If an engine is older or in a poor state of repair or both, the net power will be affected accordingly.

Because of all of the variations in standard equipment and the differences in emission controls and in maintenance procedures, it is difficult to establish a specific net power for any group of vehicles. Most manufacturers advertise and guarantee a specific gross power output for an engine, but a net power rating is available only theoretically. However, the gross-weight-to-power ratio on which AASHTO bases climbing-lane design theory uses net power.

For this reason, net power was used in the analysis in this study, and it had to be calculated for almost every vehicle. With a few exceptions, the annually published specifications for vehicle engines give gross power. The supplement to the Society of Automotive Engineers handbook (6) suggests reducing gross power by 10 percent to obtain net power. However, in view of the variations in standard accessories on different models of vehicles, the higher emission-control standards for new engines, and the decreasing state of repair on older vehicles, a standard 15 percent reduction in gross power was used to calculate the net power for the vehicles tested. This factor provides a margin of safety for design purposes. [A good discussion of the differing theories for using gross or net power has been given by the Western Highway Institute (7, 8).]

Driver Variables

The driver is probably the most difficult element to characterize in developing a complete speed profile for a particular class of vehicle, although truck drivers generally should not have large variations in experience and ability. In the attempt to identify more clearly the human aspects of truck operating characteristics, the field-data collection procedures were designed to reduce the influence on the driver of any awareness that he or she was being monitored and to provide a sample of driver age and experience. Additional information, such as the ages and years of experience of the drivers, was obtained from a questionnaire mailed to them.

DATA ANALYSIS

A stepwise multiple regression analysis was performed on the data. The procedures followed in preparing and performing the analysis included (a) a sample size calculation, (b) consideration of the variables to be analyzed, (c) determination of speed-history groups, (d) statistical multiple regression analysis to determine best-fit equations, (e) selection of the equations that best predict and describe the behavior of vehicles on grades, and (f) speed versus distance and critical-length-of-grade curve plotting.

Sample Size and Variables

A sample size of 22 vehicle speed histories was shown to give results that fell within a 95 percent confidence interval.

The factors used in characterizing the hill-climbing ability of vehicles were developed from past research and field observation. A total of 11 factors were analyzed:

1. Length of grade,
2. Percent of grade,
3. Approach speed,
4. Weight of the vehicle,
5. Power of the vehicle,
6. Frontal area of the vehicle,
7. Side area of the vehicle,
8. Length of the vehicle,
9. Experience of the driver,
10. Age of the driver, and
11. Age of the vehicle.

A stepwise regression analysis was performed on the data; in it the speed profile of each vehicle was entered as the dependent variable (Y), and the remaining factors or variables were entered as independent variables (X_1, X_2, X_3, . . ., X_n). The length of the grade was entered on a cumulative basis with respect to the starting point, which coincided with the vertical point of curvature. The percent of the grade was entered as an average percent weighted with respect to the point of curvature. The approach speed, the gross weight of the vehicle combination, and the net power of the vehicle were entered directly. The frontal area and the side area were each given as a relative number, from 0 to 9, that depends on the size of the vehicle and its type. The last three variables—driver experience, the age of the driver, and the age of the vehicle—were given in years. Since it was not possible to obtain sufficient information about the ages of the drivers and their experience for all of the vehicles and sites, these factors were included in a reduced sample.

Predictive Equations

The different vehicle categories were tabulated before the data analysis. Vehicle data from each site were first considered separately, and then data for similar vehicle categories from two or more sites were combined. Table 2 summarizes the predictive equations developed, their characteristics, the number of independent variables entering each equation, and the weighted percent grade for which each was developed. To combine data for similar vehicles on two or more grades into one equation, it was assumed that the slight horizontal curvature differences among the grades would not have differing

Table 2. Summary of predictive equations for trucks on grades.

Equation Number	Grade	Type of Vehicle	Sample Size	R^2	Coefficient of Variation (%)	Length of Section (m)	Weighted Percent Grade	No. of Variables in Equation
			Characteristics					
1	A	SU-2D	69	0.7721	9.9	1052	4.8	16
2[a]	A	2-S2	19	0.9818	5.4	1052	4.8	13
3	A	3-S2	37	0.8944	6.85	1052	4.8	29
4	A	3-S2	127	0.8309	10.84	1052	4.8	21
5	B	3-S2	37	0.7837	9.39	762	3.5	9
6	C	3-S2	37	0.8838	9.66	1737	2.6	15
7[b]	D	3-S2	34	0.7106	9.96	823	3.7	18
8[a]	E	3-S2	20	0.6463	8.56	1006	2.6	18
9[a]	A B, C, and D	2-S1	15	0.8653	9.6	927	4.9	10
10	B, C, and D	2-S2	43	0.8657	11.23	1094	3.1	6
11	B	Log	37	0.8275	9.2	762	3.5	14
12	C	Log	37	0.9244	8.5	1737	2.6	23
13[b]	D	Log	31	0.7824	10.7	823	3.7	12
14[c]	C	3-S2	26	0.9178	7.98	884	-2.3	7

Note: 1 m = 3.28 ft.
[a]Not considered, sample size too small. [b]Not usable, equation invalid. [c]Downgrade.

Table 3. Range of percent grade for test grades.

Grade	Average Percent Grade[a]	Distance (m)	Comments
A	6.00	457	Upgrade
	5.00[b]	914	Upgrade
B	5.00	457	Upgrade
	0.00	305	Level
C	2.50	1219	Upgrade
	3.00	975	Upgrade
	-2.87[c]	884	Downgrade
D[d]	4.00	518	Upgrade

Note: 1 m = 3.28 ft.
[a] Average percent grade of tangent section only.
[b]Grade varies from 4 to 6 percent; 5 is used as an average.
[c] Grade varies from -2.35 to -3.387 percent; -2.87 is used as an average.
[d]Not used after analysis of speed profiles (equation inadequate).

effects on the operating characteristics of the vehicles.

After the data were arrayed, 431 truck-speed histories were available for analysis. The 14 equations summarized in Table 2 were analyzed, and such factors as R^2, the sample size, the length of the grade, the vehicle type, and the coefficient of variation were evaluated before selecting equations to represent the operating characteristics of trucks on upgrades (11).

Equations 1, 3, 4, 5, 6, 10, 11, and 12 were selected. They represent the vehicle operating characteristics of single-unit trucks and semitrailers. Two of the equations, 11 and 12, represent log trucks. All of the equations have a correlation (R^2) above 83 percent. The coefficient of variation is within 10 for all but one equation.

These equations were developed by using a range of grade of 0 to 6 percent and a length of grade of 0 to 1980 m (0 to 6500 ft). A weighted-average percent grade of the tangent section only and the corresponding distance were used to represent the characteristics of each site. The resulting range of percentage and length for each site are given in Table 3.

Speed Profiles

From the information given in Table 3 and weighted-average grade profiles for the entire length of each grade, curves were plotted to apply the predictive equations. The average values for the variables in each equation are given in Table 4. The greatest total speed losses were those of 3-S2 semitrailer combinations and log trucks, which are characterized by equations 3, 5, 6, 11, and 12.

Figure 3 represents truck behavior over the selected upgrades given in Table 3 and provides direct comparisons between vehicles with different weight-to-power ratios [130, 228, 230, and 243 kg/kW (213, 375, 378, and 400 lb/hp)], varying grades (2.5 to 6 percent), and current design criteria versus those developed in this research. The family of curves compare speed versus distance and emphasize the differences between the current and the newly developed relations. The importance of approach speed can be seen by comparing the initial (or entering) speeds given.

The average gross-weight-to-net-power ratios of 3-S2 combinations and log trucks on US-59 are 218 and 324 kg/kW (359 and 385 lb/hp) respectively. The average ratio for all 3-S2 combinations on US-183 is 140 kg/kW. Present design procedures are based partly on the assumption that vehicle performance is a function of the weight-to-power ratio, but an analysis of Figure 3 shows that differences in the rates of deceleration on the upgrades depend mainly on the effect of the entering speed.

CRITICAL LENGTH OF GRADE

The term critical length of grade indicates the maximum length of a designated upgrade on which a vehicle can operate without an unreasonable reduction in speed. From the data given in Figure 3, a series of critical lengths of grade for 8, 16, 24, and 32-km/h (5, 10, 15, and 20-mph) reductions in speed, based on approach speed ranges rather than on specific speeds, are shown in Figures 4 and 5. The figures cover two speed ranges [79 to 87 and 84 to 100 km/h (49 to 54 and 52 to 62 mph)], are representative of trucks with weight-to-power ratios of approximately 225 and 234 kg/kW (370 and 385 lb/hp), and assume a reasonably level approach. Figure 4 was developed from log-truck speed profiles. These graphs are representative of all trucks, since they are based on the operating characteristics of the most critical trucks. The present critical-length-of-grade design curves are given in Figure 6 for comparison.

The procedure for the use of these graphs can be illustrated by reference to Figure 4. If an average percent grade of 4 and an approach speed between 79 and 87 km/h (49 and 54 mph) are assumed, the corresponding critical length of grade for a 24-km/h (15-mph) speed reduction is 378 m (1240 ft). If the approach speed is in a different range, the critical length of grade will also be different. The table below gives a comparison of the critical lengths of grade derived by the use of Figure 4 and an entering speed range of 79 to 87 km/h (49 to 54 mph), Figure 5 and an entering speed range of 84 to 100 km/h (52 to 62 mph), and Figure 6 and an entering speed of 76 km/h (47 mph) for a 24-km/h (15 mph) speed reduction (1 m = 3.28 ft).

Table 4. Characteristics of trucks by vehicle type and grade used in developing speed versus distance curves.

		Vehicle Characteristics							
Equation Number	Grade	Type of Vehicle	Approach Speed (km/h)	Net Power (kW)	Gross Weight (kg)	Weight/Power (kg/kW)	Age of Vehicle (years)	Frontal Area[a]	Side Area[a]
1	A	SU-2D	93	124	8 209	66	5.89	3.5	6.4
2	A	2-S2	90	139	15 057	107	3.76	5.3	2.1
3	A	3-S2	99	166	21 023	130	3.6	5.3	2.0
4	A	3-S2	97	158	21 578	140	4.15	4.6	2.3
Avg					21 301				
5	B	3-S2	97	151	34 345	223	2.46	2.9	3.9
6	C	3-S2	88	149	33 936	228	2.66	3.2	3.6
7	D	3-S2	100	152	33 781	223	2.6	3.2	3.6
8	E	3-S2	89	163	31 888	199	2.85	4.6	2.3
Avg					33 488	218			
9	B, C, and D	2-S2	80	139	30 887	223	4.02	3.0	5.0
10	B	Log	88	142	33 937	238	3.24	2.9	–
11	C	Log	77	145	34 138	230	3.14	2.8	–
12	D	Log	93	145	33 912	234	3.13	2.9	–
Avg					33 996	234			
13	A	2-S1	80	118	12 382	105	5.17	3.0	5.0
14	C	3-S2	53	151	34 602	230	2.61	3.4	3.4

Notes: 1 km/h = 0.62 mph; 1 kW = 1.34 hp; 1 kg = 2.204 lb.
Values represent an average (the mean) of those vehicles included in the study.
[a]Given on a scale from 0 through 9.

Figure 3. Speed versus distance curves for typical heavy trucks as compared with present design curves.

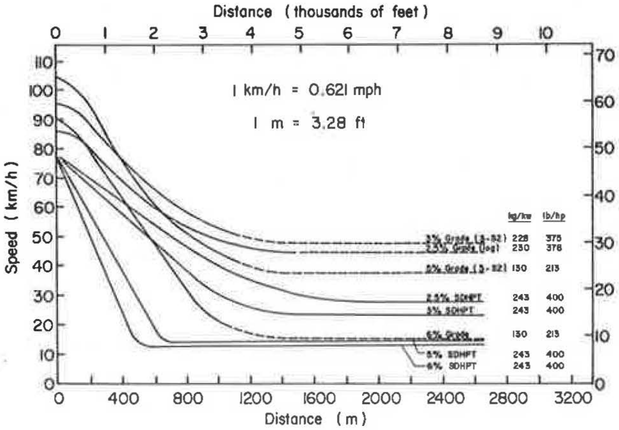


Figure 4. Critical lengths of grade for trucks with a weight-to-power ratio of 234 kg/kW (385 lb/hp) and an approach speed range of 79 to 87 km/h (49 to 54 mph).

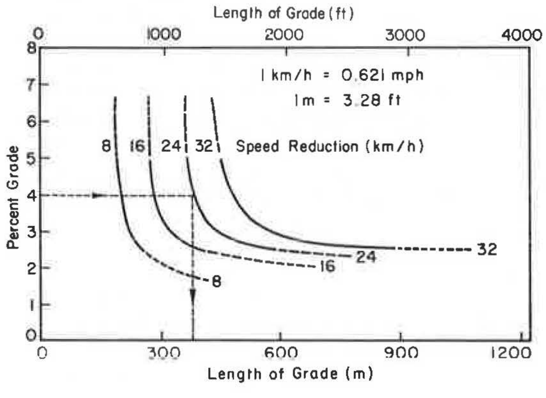


Figure 5. Critical lengths of grade for trucks with a weight-to-power ratio of 225 kg/kW (370 lb/hp) and an approach speed range of 84 to 100 km/h (52 to 62 mph).

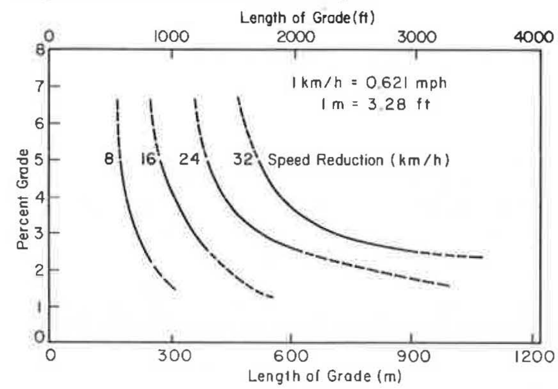
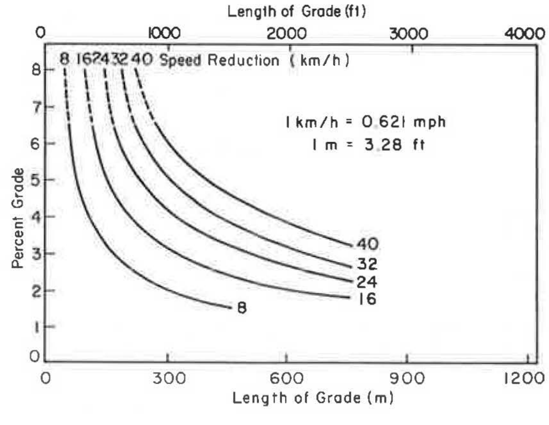


Figure 6. Current critical lengths of grade for trucks with a weight-to-power ratio of 243 kg/kW (400 lb/hp) and an approach speed of 76 km/h (47 mph).



	Critical Length of Grade (m)		
Grade (%)	Derived From Figure 4	Derived From Figure 5	Derived From Figure 6
3	1420	1700	1680
4	1240	1420	1100
5	1190	1280	750
6	1180	1200	600

The relations among approach speed, critical length of grade, and percent grade are not consistent between the two speed range categories, which confirms earlier findings that the distance and the percent grade are very important elements in any speed profile for trucks and truck combinations.

From these analyses, composite charts were developed for typical heavy trucks using an approach speed of 89 km/h (55 mph). Figure 7 shows the critical lengths of grade at different percent grades together with the associated speed reductions for a range of 8 to 32 km/h (5 to 20 mph) in 8-km/h (5-mph) increments.

Figure 8 shows the speed versus distance relations for a range of upgrades from 2 to 7 percent in 1 percent increments. The approach speed for all grades is 89 km/h (55 mph). The lengths of climbing lanes for the percent grade and speed-reduction criteria can be evaluated from these charts.

CONCLUSIONS AND RECOMMENDATIONS

The object of this study was to obtain new field data concerning motor-vehicle operating characteristics on selected grades and to relate these data to current and future geometric design standards for highway grades and the related capacity and safety aspects of vehicle climbing lanes.

The following recommendations, based on the findings of this research, are made.

1. The composite critical-length-of-grade speed versus distance curves (Figures 7 and 8) should be applied to the evaluation of the need for and the design of climbing lanes for trucks.
2. An approach speed of 89 km/h (55 mph) should be used for the evaluation and design of climbing lanes.

Further evaluation and study are recommended in the following areas:

1. The 16 versus the 24-km/h (10 versus 15-mph) speed-reduction criteria,
2. The current justifications for climbing lanes,
3. Vehicle equivalencies,
4. Roadway signing and marking of climbing lanes, and
5. The effect of driver behavior and experience on vehicle performance.

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The contents of this paper reflect the views of the authors, who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Texas State Department of Highways and Public Transportation or of the Federal Highway Administration, nor do they constitute a standard, specification, or regulation.

Figure 7. Critical lengths of grade for a composite truck weight-to-power ratio and an approach speed of 88 km/h (55 mph).

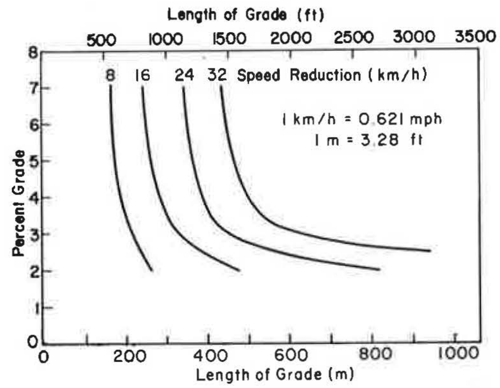
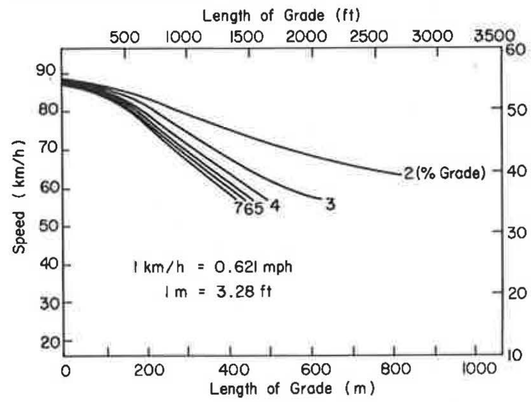


Figure 8. Composite speed versus distance curves for typical heavy trucks on selected upgrades.



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Rest-Area Wastewater Treatment

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To develop guidelines for rest-area wastewater-treatment systems that are capable of complying with the requirements of Public Law 92-500, the principal treatment situations encountered at rest areas have been identified. The quantity and quality of wastewater produced at rest areas were examined through a literature survey and through visits to various state highway departments. The common categories of wastewater-treatment systems in use at rest areas are discussed, and their capability for compliance with the requirements of Public Law 92-500 was investigated. An extended-aeration, activated-sludge plant was shown to be capable of meeting these requirements. Initial results of the study indicated that lack of adequate design criteria results in major problems in planning, designing, and operating rest-area wastewater-treatment facilities. To meet this deficiency, new design criteria and operation guidelines were developed to assist the state highway departments responsible for providing and maintaining wastewater-treatment facilities at rest areas.

The advent of the Interstate highway system has resulted in an increase in travel by a more mobile American public. To accommodate these travelers, the Federal Highway Administration (FHWA) and the state highway departments provide roadside rest areas on the highways.

One of the main problems in the construction of rest areas is that of providing adequate facilities for the treatment and disposal of wastewater. The wastewaters produced at rest areas are characterized by large variations in flow and composition, and the use of sophisticated treatment systems to accommodate these variations requires frequent attention from skilled operators, who are scarce.

The Federal Water-Pollution Control Act amendments of 1972 [Public Law 92-500 (Pub.L. 92-500)] and an increased public concern for environmental quality have prompted FHWA to initiate a program designed to minimize the environmental impact of the highway system. One of the purposes of this program is to develop a treatment technology for rest-area wastewater that will comply with the 1977 requirements of the 1972 amendments. This FHWA-funded research is designed to assist the state highway departments by providing information and guidelines for the design and upgrading of rest-area treatment and disposal facilities.

One segment of the FHWA research was a two-phase study by the Environmental Effects Laboratory of the U.S. Army Engineer Waterways Experiment Station (WES) at Vicksburg, Mississippi. The first phase emphasized the survey and assessment of the operating characteristics of existing rest-area treatment systems. The second phase emphasized the development of specific design and operating guidelines.

The phase 1 research collected information about the conditions of existing rest-area facilities. The types and sizes of existing wastewater-treatment systems and their operational characteristics and design parameters were inventoried. The applicable literature was reviewed, and field visits were made to 21 states.

The phase 2 effort consisted of the identification of wastewater-treatment systems that comply with the 1977 requirements of Pub.L. 92-500; the investigation of an extended-aeration, activated-sludge treatment plant with emphasis on its ability to meet the 1977 requirements of Pub.L. 92-500; and the preparation of design guidelines, criteria, and recommendations for selecting wastewater-treatment systems for rest areas.

LITERATURE REVIEW

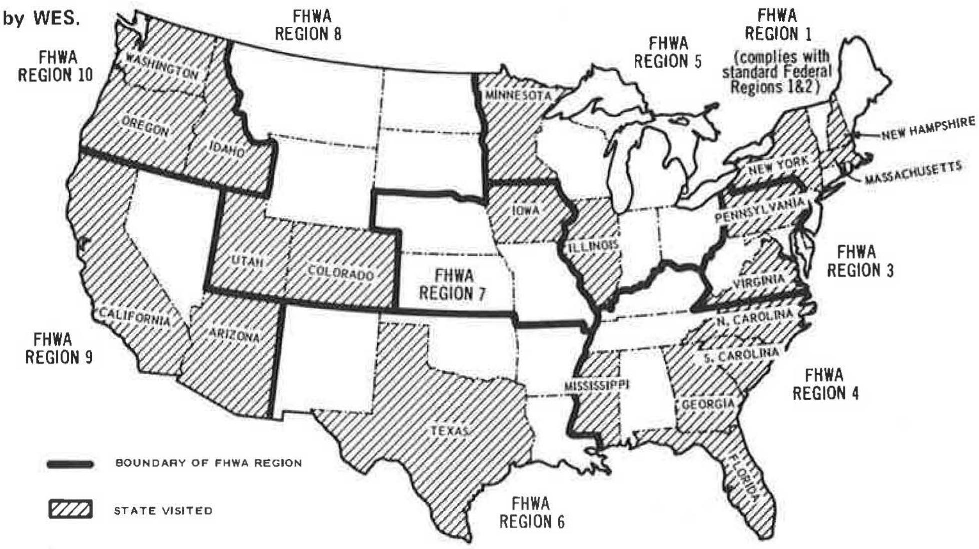
Before a wastewater treatment system is designed, the flow and concentrations of the various constituents of the wastewater must be determined or estimated. Because of the absence of more definitive information, most of the existing rest areas were designed by using the concentrations of an average domestic wastewater. However, since 1971, there have been four important studies of rest-area wastewater and the treatment facilities for it.

Washington Rest Areas

Sylvester and Seabloom (1) studied the wastewater characteristics at four rest areas in Washington. In comparing the characteristics of the rest-area wastewater with those of domestic wastes, they concluded that rest-area wastewater

1. Has essentially no grease or scum materials,
2. Is high in nitrogen, which indicates a preponderance of urine,
3. Has suspended solids (SS) and a 5-day biochemical oxygen demand (BOD_5) that are between those of weak and average domestic wastewaters,
4. Has a chemical oxygen demand (COD) equal to that of a strong wastewater (because of the paper content),

Figure 1. States visited by WES.



5. Has a phosphate content that corresponds to that of weak wastewater, and

6. Has a settleable solids content that is much greater than that of domestic wastewater (because of the high paper content).

Sylvester and Seabloom also found that the amount of wastewater produced per user varied from site to site and that there was a large fluctuation in the flow rate from hour to hour and from day to day. An assumed flow of 13.2 L (3.5 gal)/person/day and an associated BOD_5 of 165 mg/L were used as the basis for sizing different types of wastewater-treatment systems in Washington.

Indiana Rest Areas

In a similar study, Etzel and others (2) studied seven rest areas in Indiana and, from an analysis of their influents and effluents, concluded that "the plants are for the most part substantially underloaded (hydraulically) and accordingly BOD loadings are low." They propose a design flow of 18.9 L (5.0 gal)/person/day and a BOD_5 loading of 3.2 to 4.5 g (0.007 to 0.01 lb)/ person. They also recommend that comprehensive traffic data be collected before a rest-area facility is designed.

Illinois and Iowa Rest Areas

Pfeffer (3) studied the characteristics of rest-area wastewater in Illinois and Iowa and, in comparing the results of his study to those of the previous two, concluded that

> the range of average BOD_5 for the various rest areas is from 110 to 204 mg/L (average 150 mg/L). The suspended solids range from 56 to 230 mg/L with an average of 149 mg/L. These data suggested that rest-area wastewater is comparable with normal municipal waste.

Pfeffer also conducted a mail survey to identify design assumptions in various states. He recommends that 12 percent of the highway traffic (assuming an occupancy of 3.1 persons/vehicle) and a wastewater production of 18.9 L (5 gal)/person/day be used as design values for rest-area treatment facilities.

Other States

Zaltzman and others (4) have conducted an extensive study of rest areas. Various rest-area parameters were monitored in Florida, Tennessee, New Hampshire, Colorado, and Iowa. The most difficult parameters to predict, according to them, were accurate forecasts of the average daily traffic (ADT) and the percentage of the ADT that stops at the rest area. After the traffic approaching and that entering the rest areas were monitored, regression models were developed to predict the ADT entering the surveyed rest areas.

Zaltzman and others also sampled rest-area wastewater and concluded that it corresponded to a weak to medium-strength domestic wastewater with respect to BOD_5, COD, total oxygen concentration, SS, and pH. Nitrogen and phosphorus concentrations often exceeded those of strong domestic wastewater.

SURVEY OF REST AREAS

To obtain information about site-selection criteria, the facilities provided, and their flow characteristics and wastewater-treatment systems, WES visited state highway departments in the nine FHWA regions (Figure 1). The survey emphasized areas with well-developed comfort facilities that included flush toilets and, since 60 percent of the rest areas along Interstate highways provide these, it was concentrated there.

Each visit included an on-site survey of at least one rest area and a meeting with the FHWA and state-highway personnel responsible for rest-area construction and maintenance. In some cases, members of the state health agencies or pollution control agencies or both were also present.

Size of Rest Area

As the literature survey had shown, the size of the rest area is usually based on the ADT. Once the ADT at a location is determined, the number of vehicles that will stop at a rest area there is calculated as a percentage of this value. The number of occupants per vehicle is also calculated.

Most rest areas are sized to accommodate expected use for their design life, which is usually taken as 20 years. New York uses a design life of 15 years, and in California, some rest-area buildings are designed to reach capacity in 10 years, although the treatment facilities are designed for 20-year periods.

Rest-Area Components

In the past, rest areas consisted of parking spaces and restrooms. Today, however, they commonly have restrooms with flush toilets and sinks and various electrical equipment. Some rest areas have information centers, pet-walk areas, and scenic walks. In 11 of the 21 states surveyed, trailer dumping stations were provided. The waste from these dumping stations enters into the rest-area treatment facility or into a holding tank.

Most rest areas are landscaped with trees, grasses, and shrubbery. The incorporation of these can improve the functioning of evapotranspiration beds, screen treatment plants from the rest-area user, and shield the rest area from the highway. In Florida and in the midwestern and western states, these plants are sometimes spray irrigated with the waste effluent to supplement the rainfall.

Types of Treatment Systems

The WES field survey found that the criteria commonly used to select a treatment system are

1. Simplicity of operation and maintenance,
2. Freedom from odor, noise, and insects,
3. Capability to accommodate the widely fluctuating hydraulic and organic loadings that are caused by traffic and seasonal changes,
4. Treatment efficiency, and
5. Reasonable initial and operating costs.

One of the main objects of the field survey was to determine the types of treatment systems in use for rest-area wastewater and their distribution. Correlation of the type of wastewater-treatment system to various parameters, such as climate, soil type, or precipitation, was attempted, but the only relation found was the predominant use of evaporative lagoons in regions of low precipitation and high evaporation.

The principal types of rest-area treatment systems found by the survey were (a) septic tanks followed by leach fields, (b) facultative, aerobic, or totally evaporative lagoons, and (c) extended-aeration (EA), activated-sludge package plants.

Although discharge to a municipality is not a common disposal method, it is the most desirable. This method (a) relieves the state highway department of the responsibility of constructing on-site treatment and disposal systems, (b) reduces operation and maintenance (O&M) costs, and (c) fulfills the state's obligation to provide wastewater treatment and remove the wastewaters from the rest areas. However, since most rest areas are not near municipalities, on-site treatment facilities must usually be supplied.

Comparison of the design capacity and actual flows of wastewater-treatment systems at existing rest areas shows that the majority have been hydraulically overdesigned. In most cases, treatment capacities less than 57 000 L/d (15 000 gal/d) would have been adequate.

PROBLEMS ASSOCIATED WITH REST-AREA DESIGN

Inadequate Water Supply

One problem faced by the rest-area designer is that of an inadequate supply of water. At remotely located rest areas, water is often not available. At some facilities, water may be available but not potable, because of contaminants or high salt contents. Treatment to produce potable water is often not feasible, and the water supplies at these areas can be used only for flushing, lawn irrigation, or cleaning. If drinking water is to be available, it must be trucked in. At some locations where adequate water is not available, low-flush toilets have been used.

Lack of Adequate Design Criteria

Another problem in rest-area planning is the lack of adequate design criteria. The BOD loadings used in design calculations range from 2.7 to 9.1 g (0.006 to 0.02 lb)/person/d, and the hydraulic loadings range from 10.2 to 22.7 L (2.7 to 6.0 gal)/person/d. Because the characteristics of rest-area wastewaters are not always known, most states have based design criteria on the characteristics of domestic waste. There is also a lack of data about the variability of wastewater flows. Rest-area use depends on the number of vehicles that stop at the facility, and since traffic flows vary both hourly and seasonally, variations in wastewater-flow patterns may result in the failure of treatment facilities designed by inadequate criteria.

REQUIREMENTS OF PUBLIC LAW 92-500

Certain requirements of Public Law 92-500 are significant in the design and operation of wastewater treatment facilities for rest areas. Section 301(b) of that law requires compliance with effluent limitations that reflect the use of secondary treatment for wastewater discharges from publicly owned treatment facilities by July 1, 1977. Compliance with the limitations necessary to meet water-quality standards for the receiving water is also required by that date.

The Environmental Protection Agency (EPA) requirements for secondary treatment established a minimum national standard. The parameters BOD_5, SS, and pH of the wastewater discharge are specifically limited. Also, some states have promulgated state effluent limitations for BOD_5 and suspended solids that are more stringent than secondary treatment levels and have established limits for the nutrients, phosphorus and nitrogen.

Compliance with water-quality criteria (stream standards) may require achieving a higher degree of treatment than secondary. Water-quality criteria vary from state to state because of differences in climate, geography, and the major uses of the water. All state standards include an antidegradation statement and limits for dissolved oxygen (DO), pH, coliform bacteria, and nitrate nitrogen in streams used as public water supplies. The antidegradation statement may prohibit any additional discharge of pollutants to a water of high quality. Some states also limit ammonia nitrogen and phosphorus. The amount of pollutant permitted for discharge to a stream will depend on the flow and concentration of the effluent in relation to the flow and assimilative capacity of the stream. Discharge into a stream with an extremely low flow may require such a high degree of treatment that a no-discharge treatment facility becomes necessary. Therefore, the selection of a site for a rest area should include an evaluation of the location, classification, and assimilative capacity of the receiving stream.

The National Pollutant Discharge Elimination System (NPDES) permit program established by Public Law 92-500 requires that the EPA regional administrator issue a permit before the discharge of effluents containing pollutants. Operators of rest areas are required to obtain a permit for wastewater discharge.

The issuance of permits by states is not included as part of this program, but the procedures followed and the terms and conditions of the permits issued will be similar in most states. The state-program elements necessary for participation in the NPDES were published in the Federal Register on December 22, 1972.

One important factor in this system is the participation of the public by notices, hearings, and appeals. When an application for an NPDES permit is made, a draft permit based on the information contained in the application or obtained from a site inspection or both is developed. The public is notified of the intention to issue a permit, and interested persons can obtain copies of the application, draft permit, or other pertinent information. After a 30-d response period, the decision regarding the permit is made, and whether the permit is issued or not, both sides may request an additional hearing.

The section of the NPDES that specifies effluent limitations is the most pertinent to the permittee. For wastewater-treatment facilities, the 1977 limits for BOD, SS, fecal coliform bacteria, and pH are given below.

Parameter	Limit 30-Day Mean	7-Day Mean
BOD_5 (arithmetic mean)		
Influent > 200 mg/L, mg/L	30	45
Influent ≤ 200 mg/L, mg/L	15% of influent	45
SS (arithmetic mean)		
Influent > 200 mg/L, mg/L	30	45
Influent ≤ 200 mg/L, mg/L	15% of influent	45
Fecal coliform bacteria (geometric mean), count/100 mL	200	400
pH of effluent	≥6.0, ≤ 9.0	≥6.0, ≤ 9.0

Proposed rules in the Federal Register of August 15, 1975, eliminate the fecal coliform requirement for secondary treatment and exclude systems that treat only domestic waste and do not use chemical addition as a part of the treatment process from the pH requirement. In addition to the concentration limitations, the permits specify weekly and monthly quantitative limits by weight for BOD and SS. If it is necessary to maintain water-quality standards, other parameters such as ammonia nitrogen, phosphorus, and minimum DO may also be regulated by the permit.

If the final effluent limitations for an existing discharge cannot be achieved immediately, a schedule of compliance may be included to allow reasonable time for modification.

Figure 2. Methods for determining wastewater flows.

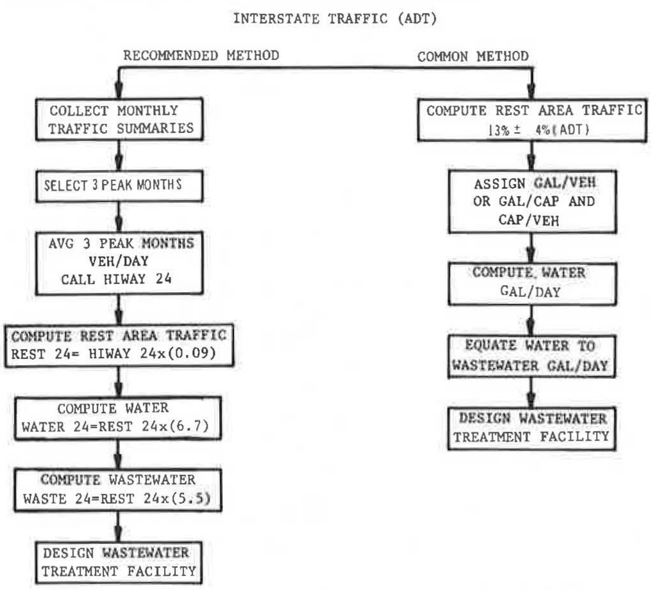


Table 1. Wastewater strengths at various rest areas.

State	Parameter BOD (mg/L)	COD (mg/L)	SS (mg/L)	pH
Colorado[a]				
High	156	507	504	8.3
Low	23	145	72	7.8
Mean	78	203	208	8.0
S.D.[b]	45	103	118	0.14
Florida[a]				
High	300	440	530	8.6
Low	140	216	28	6.8
Mean	181	342	186	7.4
S.D.[b]	43	60	111	0.55
Iowa[a]				
High	561	787	652	8.5
Low	59	140	38	7.1
Mean	210	383	224	7.9
S.D.[b]	137	209	153	0.35
New Hampshire[a]				
High	330	480	684	8.4
Low	90	197	1	6.4
Mean	203	330	208	7.2
S.D.[b]	62	82	165	0.65
Tennessee[a]				
High	223	883	310	8.7
Low	63	160	16	7.1
Mean	158	362	124	7.7
S.D.[b]	52	174	72	0.45
Mississippi[c]				
High	432	–	839	–
Low	12	–	4	–
Mean	124	–	140	–
S.D.[b]	86	–	145	–

[a]Data collected by Zaltzman and others (4).
[b]S.D. = standard deviation.
[c]Data collected by WES.

WATER USE, WASTEWATER PRODUCTION, AND WASTEWATER CHARACTERISTICS

Most of the rest areas designed to date have assumed a usage that is a percentage of the ADT. Water supply and wastewater-treatment facilities are then designed on the basis of an assigned occupancy per vehicle and a water use per occupant. These values are usually taken as 3.1 persons/vehicle and 18.9 L (5 gal)/person.

Zaltzman and others (4) have developed a method for predicting rest-area traffic, water use, and wastewater production. The contrast between this method and the method currently used by state highway departments for determining sizes of rest-area water supplies and wastewater-treatment facilities is shown in Figure 2. In the new method, monthly traffic summaries are collected for a 1-year period. The three peak months are selected from these summaries, and their average number of vehicles per day is calculated (HIWAY 24). HIWAY 24 is then used in the same way that the ADT is used at present. The number of vehicles per day stopping at a rest area (REST 24) is assumed to be 9 percent of HIWAY 24. Water use and wastewater production per vehicle vary from site to site; some typical values are given below (1 L = 0.26 gal).

State	Water Use (L/vehicle) Peak	Avg	Min	Wastewater Production (L/vehicle) Peak	Avg	Min
Florida	20.8	17.0	15.1	18.9	16.1	13.2
Tennessee	26.5	18.9	8.5	26.5	18.9	8.5
New Hampshire	26.5	24.6	20.8	22.7	21.8	16.1
Colorado	20.8	16.1	8.5	18.9	16.1	11.4
Iowa	20.8	16.1	8.5	20.8	16.1	8.5

Values given in these tables may be used as design guidelines. The concentrations of the constituents of the wastewater also affect the size of a wastewater-treatment system. Previously, rest-area wastewater was assumed to have characteristics similar to those of a medium-strength domestic wastewater (BOD_5 = 200 mg/L and SS = 200 mg/L). However, the data collected by Zaltzman and others and by WES (Table 1) indicate that BOD_5 and SS are in the range 125 to 200 mg/L, which corresponds to a weak domestic wastewater. Therefore, the values listed in Table 1 may be used as design guidelines.

COMMON REST-AREA TREATMENT METHODS

The following treatment methods are currently used at rest areas: septic tanks and leach fields or sand filters, oxidation ponds, and EA, activated-sludge package plants.

Septic Tanks

In a septic tank system, the wastewater enters directly into the tank, where it is retained (normally for 24 h). During this time, some of the solids settle to the tank bottom where anaerobic decomposition slightly reduces their volume. The remainder of the wastewater (including its SS, bacteria, soluble organics, and nutrients) becomes effluent and flows into the leach fields or sand filters.

Septic tanks are usually inexpensive, are easy to install and maintain, and can handle small variations in flow and periods of nonuse. However, high groundwater, high precipitation, and poor soil conditions may inhibit the effectiveness of the leach fields. Also, the soil in the leach fields may become clogged and the system may be hydraulically overtaxed, which will result in the surfacing of partially treated wastewater. In some areas, land requirements may prohibit the use of leach fields.

Oxidation Ponds (Lagoons)

In oxidation ponds, facultative bacteria metabolize the organic matter in the raw waste for energy and growth and produce carbon dioxide and water. Algae, on the other hand, use carbon dioxide, sunlight, and inorganic materials to produce algae protoplasm and oxygen, which are used by the bacteria to complete the cycle. The solids settle to the bottom for anaerobic decomposition.

Oxidation ponds are less susceptible to organic or hydraulic shock loading than are other treatment methods, their O&M costs are low, and they require little, if any, electrical or mechanical equipment. However, they usually achieve only a low degree of treatment and, in the summer, their effluents may be algae-laden. They may also present problems of odors and the pollution of groundwater. The use of oxidation ponds as the only means of treatment is limited because a high degree of treatment is generally required for rest-area wastewater effluents.

EA, Activated-Sludge Package Plants

Extended aeration is a modification of the conventional activated-sludge process and involves an aeration tank in which the incoming raw waste contacts a heterogeneous culture of microorganisms in the presence of oxygen. The bacteria use the organic matter as a source of food and energy, and the wastewater then flows into a clarifier where the biological culture is removed and returned to the aeration tank. These package plants can be installed on narrow rights-of-way or in areas of high precipitation. Additional units can be added if the volumes of wastewater increase. However, the use of EA systems is handicapped by high capital investment and O&M costs, as well as by the need for trained operators.

Land Treatment

Land treatment of liquid waste is divided into three variations: rapid infiltration, spray irrigation, and overland flow. Rapid infiltration is the application of a considerable depth of wastewater to a highly permeable soil for the generation or replenishment of the groundwater. Spray irrigation is the application of wastewater to vegetated land. Overland flow is the application of wastewater to an impermeable soil where the wastewater flows over a sloping terrain and is collected in a ditch or natural stream.

At rest areas, spray irrigation has been used as a tertiary treatment for effluents from EA, activated-sludge plants and oxidation ponds. This offers an advanced degree of treatment, low design and O&M costs, and removal of the discharge to surface areas. However, it may require land areas that can be isolated from the public.

Evaporative Lagoons (Ponds)

Evaporative lagoons are usually arranged in series. Raw wastewater flows into the first pond where much of the solid matter settles to the bottom and undergoes anaerobic decomposition; the remainder of the wastewater undergoes aerobic treatment. The partially treated wastewater then flows into a second pond where it evaporates or is used for spray irrigation.

These ponds eliminate discharge of the effluent, are inexpensive to operate and maintain, and can withstand large fluctuations in flow and shock loadings. The method is limited to areas where evaporation exceeds precipitation plus the volume of wastewater. Odor and nuisance (e.g., mosquito) problems must also be considered.

APPLICABILITY OF WASTEWATER-TREATMENT SYSTEMS TO USE AT REST AREAS

Because of the 1977 requirements of Public Law 92-500, rest-area wastewater-treatment systems must produce an effluent that meets the standards for secondary treatment. This applies to both existing and planned systems.

Evaporative lagoons and land-treatment systems (with the exception of overland flow) have no point-source discharge and therefore should meet the requirements of Public Law 92-500. However, in the design of these systems, the groundwater supplies must be protected against possible contamination by the wastewater.

Properly designed septic-tank-and-soil-absorption systems have also no discharge and should meet the requirements of Public Law 92-500. However, sound construction procedures must be followed to ensure correct operation of these systems and to protect groundwater supplies from possible contamination.

Facultative lagoons may not produce an effluent that meets the 1977 requirements of Public Law 92-500 because of the presence of algae. Algae not only contribute to a high suspended-solids concentration in the effluent, but also have an associated BOD_5 that often exceeds the effluent requirement. However, additional treatment of lagoon effluents to remove the algae may improve their quality enough to meet the requirements of Public Law 92-500. This additional treatment could be by slow sand filters, rock filters, and upflow sand filters. (However, recent EPA-proposed rule changes may alleviate the effluent suspended-solids requirements for lagoons.)

EA, activated-sludge package plants in use at schools, subdivisions, and apartment complexes have been shown to be capable of producing an effluent that meets the requirements of Public Law 92-500. However, EA plants in use at rest areas have not been shown to do so. Therefore, WES monitored an EA plant in use at a rest area to determine its compliance or noncompliance.

The EA plant monitored in this study was located at the Toomsuba Rest Area on the westbound lane of I-20, 19 km (12 miles) east of Meridian, Mississippi. This rest area is equipped with an information center, a parking area, picnic tables, drinking fountains, a trailer dumping station, and wastewater-treatment facilities.

The wastewater-treatment facilities consist of a comminutor; three aeration tanks, each having a capacity of 18 900 L (5000 gal), operated in series; a settling tank having a capacity of 9450 L (2500 gal); a sludge holding tank having a capacity of 4725 L (1125 gal) with recycle to the first aeration tank; a chlorine-contact chamber having a capacity of 1420 L (375 gal); and a postaeration chamber (Figure 3). The plant receives wastewater

Figure 3. Plant layout and sampling points.

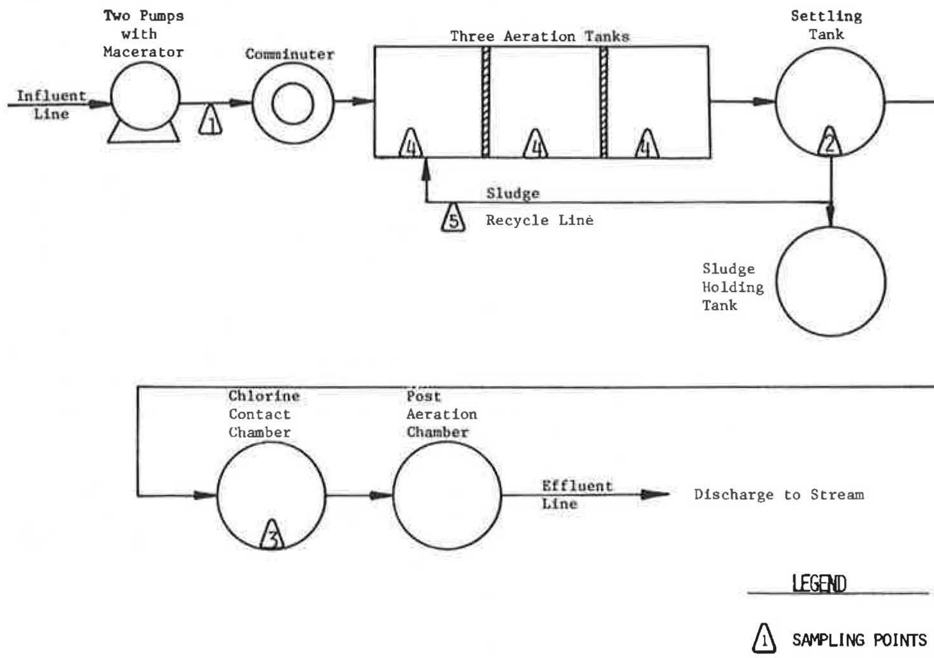


Figure 4. Influent and effluent 5-day biochemical oxygen demand.

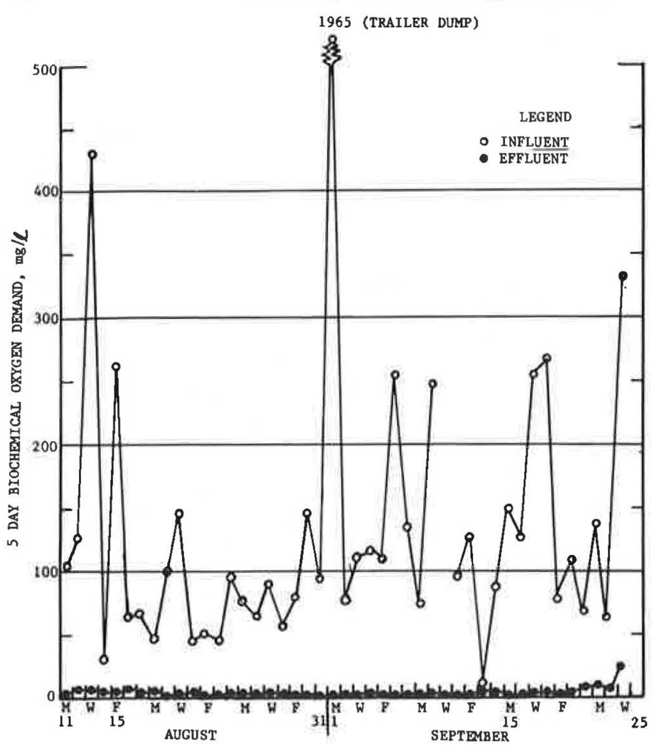
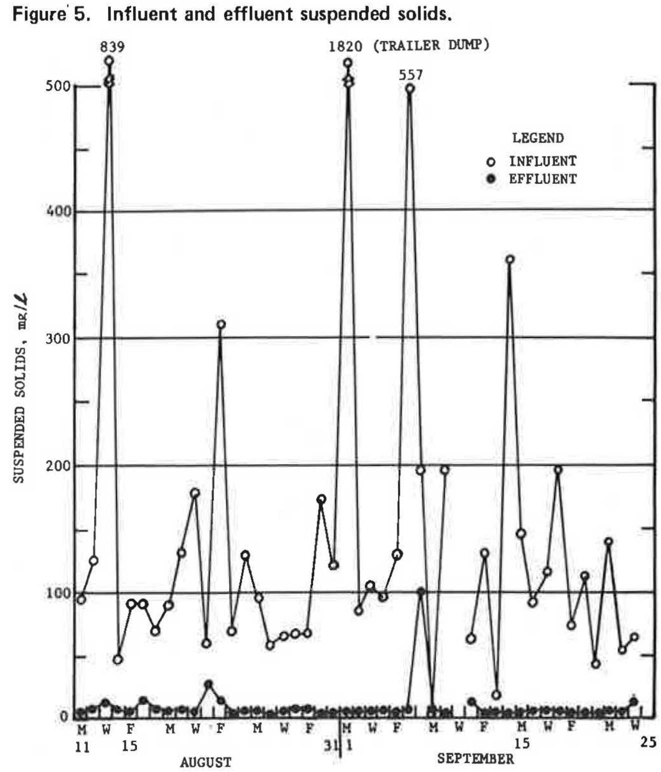


Figure 5. Influent and effluent suspended solids.



from a forced main serviced by two submersible, centrifugal pumps with macerators and discharges its effluent via a ditch to a nearby stream. It is currently operating under a permit from the Mississippi Air and Water Pollution Control Commission that was issued October 15, 1974.

Sampling points are located at the influent line just prior to the comminutor (1), the settling tank (2), the chlorine-contact tank (3), each aeration tank (4), and the sludge-return line (5) (Figure 3).

Random-grab samples of the influents and the final effluents were collected daily over a 45-d period between 8:00 a.m. and 6:30 p.m. Composite samples of the influent were also collected to establish the relation between the grab and composite samples.

The flow was measured with automatic flow recorders in the chlorine-contact chamber. Water use was measured with a 3.8-cm (1.5-in) water meter equipped with an impulse switch connected to an event recorder located on the water line to the hospitality house.

The analysis of the wastewater samples included ammonia; nitrate; phosphate; pH; total, settleable, suspended, dissolved, and volatile suspended solids; BOD_5; and fecal coliform bacteria. All analyses were carried out by procedures given in Standard Methods (5). Figures 4 and 5 illustrate influent and effluent data for BOD_5 and SS. Examination of the data shows that

1. All effluent BOD_5 grab and composite samples met the requirements of Public Law 92-500,
2. All 7-d mean BOD_5 met the requirements of Public Law 92-500,
3. All 30-d mean BOD_5 met the requirements of Public Law 92-500,
4. One SS grab was in excess of the Public Law 92-500 requirement,
5. All 7-d mean SS met the requirements of Public Law 92-500,
6. All 30-d mean SS met the requirements of Public Law 92-500,
7. All pH samples met the requirements of Public Law 92-500,
8. One 7-d mean fecal coliform bacteria failed to meet the requirements of Public Law 92-500 (this was attributed to malfunction of the chlorination unit), and
9. The overall plant efficiency (based on the average of the grab samples) was 97.5 percent removal of BOD_5 and 92.3 percent removal of SS.

Thus, the results of this study showed that a well-operated EA plant (when used at a rest area) can meet the 1977 requirements of Public Law 92-500.

CONCLUSIONS

1. Water use rates, wastewater-production rates, and wastewater characteristics, as related to the design of treatment facilities and to existing rest-area facilities, are extremely site specific. The design criteria currently in use by various states show a range of hydraulic-loading rates from 10.2 to 22.7 L (2.7 to 6.0 gal)/person/d and an organic (BOD_5) loading from 2.7 to 9.1 g (0.006 to 0.02 lb)/person/d.
2. Rest-area wastewater should not present any overall treatment problems from the standpoint of its composition, even in view of the following characteristics: The COD to BOD ratio is generally higher for wastewater from rest areas than for domestic wastewater, because of the presence of a larger fraction of nonbiodegradable organics. The ammonia nitrogen concentration of rest-area wastewater is higher than that of domestic wastewater because of its higher urine content. The variation in individual wastewater parameters is substantially greater for rest areas than for domestic sources, which creates greater difficulty in generating meaningful design criteria. Adequate nutrient (particularly nitrogen) and COD removals are sometimes difficult with secondary treatment systems, and tertiary treatment may be required at some rest areas.
3. The lack of adequate design criteria has created major problems in planning, designing, and operating rest-area wastewater-treatment facilities. In particular, the hydraulic and organic loading criteria assumed have led to the overdesign of most facilities.
4. The work of Zaltzman and others (4) and others has shown that a more accurate estimate of hydraulic loading rates is in the range of 16.1 to 21.8 L (4.25 to 5.75 gal)/vehicle/d, depending on geographical location and site-specific conditions, and assuming an average waste strength of 125 to 200 mg/L for BOD_5, and 125 to 200 mg/L for SS.
5. The commonest categories of wastewater-treatment systems in terms of use are biological and no-discharge systems. These systems include septic tanks followed by leach fields or sand filters; facultative, aerobic, or totally evaporative ponds; and EA, activated-sludge plants.
6. Wastewater-treatment systems that have been used at rest areas and that are capable of complying with the effluent limitations of Public Law 92-500 include septic tanks followed by leach fields or evapotranspiration beds, EA package plants, facultative lagoons followed by filtration or land treatment, and totally evaporative lagoons.
7. There is no single, best solution to the rest-area wastewater-treatment problem that can be applied on a national, regional, or even state-by-state basis. The design of each wastewater-treatment facility must be site specific.

ACKNOWLEDGMENT

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Abridgment

Evaluation of a Water-Reuse Concept for Highway Rest Areas

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Most of the water used at rest areas is used for flushing toilets. Therefore, if toilet-flushing needs can be met without using potable water, the potable-water supply needed for a rest area can be drastically reduced. In addition, if the water used for flushing the toilets can be returned to them, the waste-treatment objective becomes that of renovating the water for reuse rather than that of meeting water-quality standards.

The object of this work was (a) to develop a water-reuse concept for flushing toilets and (b) to evaluate this concept by using a bench-scale model.

WATER AND WASTEWATER AT REST AREAS

Studies in Virginia and other states have shown that the total water used at rest areas is 19 to 20 L (5 to 5.5 gal)/rest-room user (1). If this water is to be recycled and reused for flushing toilets, it is first necessary to identify the amount of potable water needed versus the amount of water that can be of a lower quality. A system that recycles and reuses water for flushing toilets must also take into account the amount of water wasted from lavatories and drinking fountains, which is another water input to the system. This input is one of the factors that control the amount of water that must be wasted from the system and was estimated from field observations to be between 5 and 10 percent of the total water use. This is within the range that can be calculated by using either (a) 11 to 19 L (3 to 5 gal)/flush/toilet plus 2.8 to 5.7 L (0.75 to 1.5 gal)/lavatory user or (b) 19 to 20 L (5 to 5.5 gal)/rest-room user with one-third to one-half of the rest-room users actually using the lavatory at a rate of 2.8 to 5.7 L (0.75 to 1.5 gal)/lavatory user.

Analyses of wastewater at rest areas have shown that both the 5-d biochemical oxygen demand (BOD_5) and the suspended solids are in the range of 150 to 180 mg/L (1). These results imply that the quality of rest-area wastewater is comparable to that of domestic wastewater; however, the total Kjeldahl nitrogen (TKN) of rest-area wastewater is between 75 and 100 mg/L, which is three to four times higher than that of domestic wastewater. Because the biological and chemical changes associated with nitrogen can have important effects on wastewater treatment processes as well as on the receiving streams, this high content of ammonia and organic nitrogen is very significant.

CONCEPT OF WATER RECYCLE AND REUSE

The basic requirements for a flushing fluid are the following: (a) no objectionable odor, (b) no objectionable color, (c) no foaming, (d) no suspended solids, (e) chemical and biological stability, and (f) low bacterial count. Fluids meeting these requirements can be produced by extended aeration or by aerated lagoons followed by filtration.

At present, there appear to be only two recycling systems for flushing fluids. These are the water-recycling system, in which the water from fountains and lavatories enters the system as a water input and becomes part of the recycled flushing water, and the mineral oil system, in which the fountain and lavatory wastewaters are either evaporated or separately treated. In either system, the water to the fountains and lavatories must be potable water and must be handled in the total treatment scheme. In a water-recycling system, this water from the fountains and lavatories can be used to maintain an equilibrium dissolved-solids concentration in the recycled water by wasting an equal volume of recycled water. Obviously, both water reuse and mineral oil recycling require that a volume of water equal to that produced from fountains and lavatories must be disposed of.

Originally designed for use on ships, the mineral oil-incineration system basically replaces flushing water with a colored mineral oil and either evaporates drinking-fountain and lavatory wastewaters or provides a separate treatment system for these wastes. A recycling system that is now in operation in Virginia can be added directly to rest areas that are already equipped with extended-aeration biological treatment. Approximately 90 to 95 percent of the water use at rest areas is for flushing toilets, and extended aeration followed by sand filtration produces a water of sufficient quality for this purpose. The advantages to this system are greatly reduced requirements for potable water and a reduced discharge of wastewater to streams. The cost-effectiveness of this type of system will be maximized when the system is added to rest areas that already have extended-aeration systems or aerated lagoons.

In a water-recycling system in which the mineral content is not reduced, there must be a waste of water from the system to limit the buildup of dissolved solids in the water. In a biological system, the effect of the dissolved solids on the microbial reactions must be taken into account. Solids inert to biological activity will build up to an equilibrium level that is controlled by the make-up water, i.e., the wastewater from lavatories and fountains. However, many organisms can adapt to a high salinity and efficiently degrade the organic constituents present in the wastewater.

The conceptual flow scheme shown in Figure 1 is based on the known performance of the extended-aeration system with high dissolved solids and the acceptability of applying settled extended-aeration effluent to a sand filter.

RESULTS OF BENCH-SCALE STUDIES

The bench-scale system used to study the recycle and reuse of wastewater included extended aeration followed by sedimentation and sand filtration. The sand filtration and recycling lines would be the only modifications necessary for the conversion of a conventional extended-aeration system to a water recycle-and-reuse system.

In the bench-scale study, recycled wastewater could not be used; therefore, a synthetic waste similar to rest-area wastes was formulated to duplicate certain specific parameters found in rest-area wastewater. The synthetic wastewater used in previous kinetic studies of bacterial growth was modified so that it would be similar to field wastewater in chemical oxygen demand (COD), pH, alkalinity, TKN, and chloride, calcium, and phosphorus concentrations (2). Because most of the compounds used to produce the COD were biodegradable organics, which is not the case in rest-area wastewater, the BOD_5 of the synthetic wastewater was greater than that of field wastewater. The total fixed-solids concentration in the synthetic waste was similar to that in the actual wastewater, but because the organics that determined the BOD_5 and COD also provided the volatile solids, the total solids were significantly higher in the synthetic wastewater.

Table 1. Quality of laboratory pilot-plant effluents at various recycle rates.

	Recycle Rate (%)			
Parameter	90[a]	0[a]	95[b]	95 With Color[b]
BOD_5, mg/L	14	9	15	12
COD, mg/L	155	109	875	846
Total dissolved solids, mg/L	5920	486	7644	7152
Total of fixed solids dissolved, mg/L	3860	365	5980	5572
Percent of dissolved solids fixed	65.2	75.1	78.2	77.9
TKN, mg/L	128	45.5	–	210
Ammonia nitrogen, mg/L	83	38.5	–	102
Mixed-liquor suspended solids, mg/L	7100	1726	7362	7846
Mixed-liquor, volatile suspended solids, mg/L	5900	1505	5936	6528
Percent volatile of mixed liquor	83.1	87.2	80.6	83.2
pH	5.4	5.6	6.2	6.2
Alkalinity, mg/L	10	26.3	346	314
Sludge-volume index	36.0	35.3	38.7	38.9
Chlorides, mg/L	597	–	1170	–
Efficiency of BOD removal[c]	99.5	96.9	99.7	99.8
Efficiency of COD removal[c]	96.1	72.8	89.1	89.4

[a] Synthetic-wastewater formula 1.
[b] Synthetic-wastewater formula 2.
[c] Based on weight of component in system effluent that was wasted.

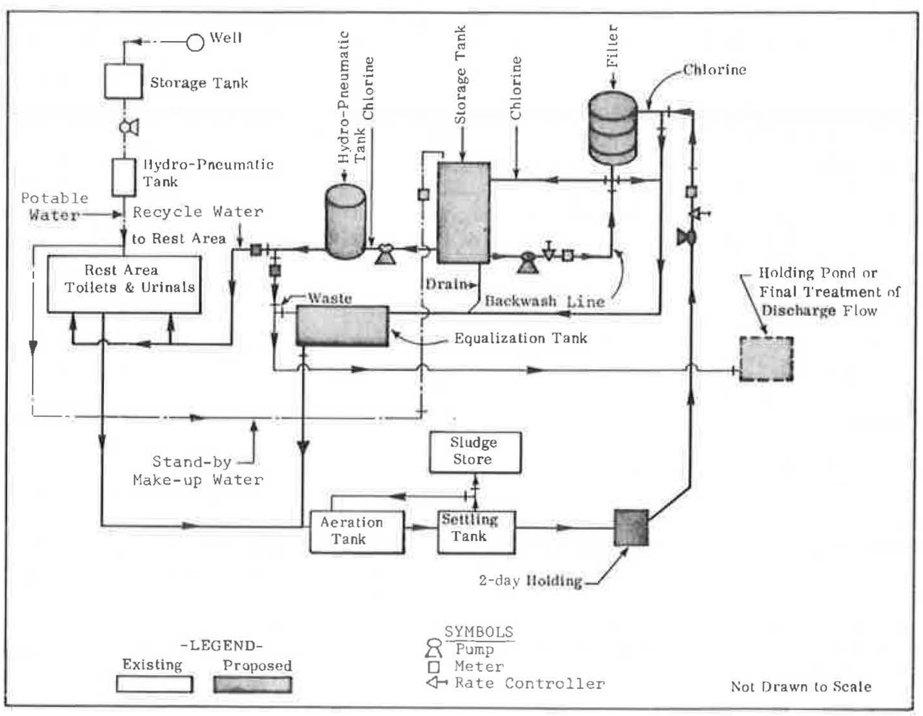
At the start-up, the biological system was seeded with a biomass obtained from a field extended-aeration unit. A recycle rate of zero percent was used to provide baseline information on the performance of the bench-scale system and to compare the system with a field-operated extended-aeration system.

At the zero recycle rate, the effluent from the bench-scale system contained 109 mg/L COD, 9 mg/L BOD_5, and 83 mg/L TKN. For comparison, an extended-aeration system at a rest area was found to be producing an effluent that contained 73 mg/L COD, 8 mg/L BOD_5, and 3 mg/L TKN from an influent raw wastewater containing 310 mg/L COD, 175 mg/L BOD_5, and 92 mg/L TKN. The effluents from the two systems compare favorably when it is considered that the BOD_5 of the synthetic wastewater had been made purposely higher to match the COD concentration so that the bench-scale system had a higher biodegradable organic load. Under these conditions, comparable nitrification was not expected.

Table 1 presents data obtained at different recycle rates. These data indicate that a buildup of nondegradable organics and inorganics will occur, but that there will not be a very large increase in biodegradable organics. Table 1 also includes the results from the use of sodium fluorescein to color the recycled water. The final steady-state water had a slight color, but appeared completely acceptable for flushing toilets. Chemically, biologically, and physically the system performed without difficulty and produced water that could be reused to transport human wastes. The biological system adjusted to the recycled water and did not show any signs of deterioration.

Synthetic wastewaters 1 and 2 in Table 1 differed only in the amount of alkalinity in the formulation. The increased alkalinity of formulation 2 was to enhance nitrification. Because pH, alkalinity, dissolved oxygen, and temperature greatly affect nitrification, a field recycling system should have increases in pH and in ammonia nitrogen during the winter when temperature will control the nitrification rate. During the summer, the rate-limiting parameters for the nitrification reactions will

Figure 1. Flow diagram for modifying typical extended-aeration system.



probably be pH, alkalinity, and dissolved oxygen; therefore, the ammonia buildup will depend on adjustments of these parameters. If the system is run in the summer without any adjustments in pH and alkalinity, the pH and the alkalinity will decrease, and the ammonia will increase. The winter pH should approach 8.3 and the summer pH should be between 5.5 and 6.0.

In addition to the satisfactory performance of the biological system, the sand filtration system adequately removed the suspended solids and required only infrequent backwashing.

Sodium fluorescein appeared to be an acceptable dye for coloring the flush water in all respects except for its greenish yellow color. It deteriorates in sunlight and is easily removed by activated carbon. Because blue is normally an appealing color, a blue food coloring such as FDC blue No. 1 may be more acceptable. This color can also be removed by activated carbon.

Evaporation as a means of producing zero discharge from a water-reuse system was evaluated by the study of a typical rest area in Virginia that treats 37 900 L (10 000 gal)/d, recycling 90 to 95 percent of the water, and having a final holding pond with a surface area of 500 m^2 (5380 ft^2). The data compiled indicate that, if holding ponds of the size currently used at rest areas in Virginia are appropriately covered, zero discharge is feasible. In addition to solar evaporation, the application of evaporation technology may be an acceptable means for producing zero discharge.

The Virginia Department of Highways and Transportation has constructed a prototype water-recycling system that is now operating at a rest area on I-81 in Rockbridge County. When compared with other alternatives for treating wastewater and conserving water, this system has an estimated saving of $30 000 annually.

CONCLUSIONS

Rest areas can use extended aeration followed by sand filtration in a scheme such as the one shown in Figure 1 to recycle and reuse water for flushing toilets. The system should be capable of recycling 90 to 95 percent of the water used. Water from water fountains and lavatories can provide the 5 to 10 percent of additional water necessary to ensure a steady-state dissolved-solids composition in the recycled water. The system will have a wastage of 5 to 10 percent of the average daily flow; however, evaporation of equivalent volumes may be a means of producing zero discharge. In certain locations, solar evaporation may be used to produce zero discharge.

The wastewater treatment described is applicable to areas with deficient water supplies and to areas where there are problems with wastewater disposal. It will not meet the needs of all rest areas, but in certain locations it can provide a viable alternative to current practice. The system can be added to existing extended-aeration systems, or it can be incorporated in the design of new facilities.

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The opinions, findings, and conclusions expressed in this paper are those of the authors and are not necessarily those of the Virginia Highway and Transportation Research Council or the Federal Highway Administration.

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Simplified Method for Design of Curb-Opening Inlets

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The purpose of this paper is to expand on and simplify the method of designing curb-opening inlets given in the Hydraulic Engineering Circular 12. A reanalysis of the experimental data has shown that the performance of a curb-opening inlet can be represented by a single dimensionless graph of the interception ratio of the curb-opening inlet (Q_i/Q) as a function of the length of the curb-opening inlet (L_i) divided by the product of the Froude number of the flow at the outer edge of the inlet depression (F_w) and the width of the spread of uniform flow in the street. The unit discharge of the inlet, up to a value of Q_i/Q defined by the cross slope alone, conforms closely to the unit discharge of the same inlet for the sump condition if the effective length of the weir crest and same total head are used in the latter case. Above this value of Q_i/Q, the required length of inlet varies as the 0.4th power of the ratio of L_i to 1.65 $F_w T$, regardless of cross slope. A design method is presented that enables computation, with reasonable confidence, of the required length of inlet for any cross slope, any grade, any width of depression, any spread of flow on the pavement, and any pavement roughness. The results agree well with the experimental data on subcritical and supercritical slopes. The analysis disclosed a number of deficiencies in the experimental data. Recommendations for remedying these deficiencies are given.

The data on curb-opening inlets first reported by Bauer and Woo (1) and their subsequent design charts (2) have been widely reproduced in hydraulic design manuals. Unfortunately, these charts are confined to a maximum longitudinal slope of 4 percent, a fixed manning n-value of 0.016, inlet lengths of 1.5, 3.05 4.6 m (5, 10 and 15 ft), and a range of flow spread up to 3.05 m (10 ft).

The original experimental data for supercritical slopes were reported by Karaki and Haynie (3). The experiments were full-scale and made on longitudinal slopes of 0.01 and 0.04 percent and cross slopes of 0.015 and 0.06 percent. Two surfaces having Manning n-values of approximately 0.01 and 0.016 were used. After initial tests to establish an optimum shape for the depression, tests were run with a depression width of 0.6 m (2 ft) (Figure 1) and widths of flow on the street of 1.5 and 3.1 m (5 and 10 ft). For each configuration, tests were begun with an inlet either 0.75 or 1.5 m (2.5 or 5 ft) long, which was then increased in 1.5-m (5-ft) increments to the length required to intercept all of the flow, or to the maximum discharge available in the apparatus.

The data were quite consistent except for a few sets that departed from the norm. In each run, the flow intercepted and the total flow were measured. This paper presents the results of a reanalysis of the original data. The symbols used are defined in Figures 1 and 2 and below.

- a = vertical distance of depression plane of curb face measured from intersection of normal street surface and curb face (feet),
- d = depth of water of uniform gutter flow at curb face (feet),
- F_W = Froude number of flow depth at distance W from curb face,
- L_i = length of curb-opening inlet (feet),
- L_1 = length of inlet when $Q_i/Q = 1$ on first section of curve for Q_i/Q = function of L_i/F_WT (Figure 2),
- L_2 = length of inlet at point of intersection of first and second sections of Q_i/Q = function of L_i/F_WT,
- L_3 = length of inlet when $Q_i/Q = 1$ on second section of curve for Q_i/Q = function of L_i/F_WT,
- n = roughness coefficient in modified Manning formula for triangular gutter flow (Equation 3),
- Q = approach flow to inlet (feet^3/second),
- Q_i = portion of Q intercepted by inlet (feet^3/second),
- Q_i/Q = interception ratio of curb-opening inlet,
- Q_2 = value of Q_i at L_2 (feet^3/second),
- Q_c = portion of Q carried past inlet = $Q - Q_i$,
- S = lontitudinal slope of pavement,
- S_x = cross slope of pavement,
- W = width of depression of curb-opening inlet (feet),
- Q_{comp} = value of Q_i estimated for composite section,
- T = width of spread of uniform flow in street (feet), and
- Z = $1/S_x$.

[SI units are not given for the variables in this model since it was derived for use with U.S. customary units. This nomenclature is the same as that used in the Hydraulic Engineering Circular (2) except for dropping the subscript on S_0 and adding the symbols used in Figure 2.]

The reanalysis shows that the interception ratios (Q_i/Q) can be represented as dimensionless functions of the parameter L_i/F_WT and the cross slope S_x.

The paper first presents the basic equations defining these functions, demonstrates how these equations were derived, and compares the results to experimental data for subcritical and supercritical slopes. It then compares the performance of these equations to those of a weir of the same dimensions, using the latter relation to verify the reasonableness of the interpolations for cross slopes intermediate to those tested, and hypothesizes the performance of untested inlets on street sections having a composite cross section. Finally, a computation table demonstrating the application of the method to practical design problems is given.

GENERAL PERFORMANCE CHARACTERISTICS OF CURB-OPENING INLETS

The performance of curb-opening inlets can be described by a single dimensionless diagram in which the interception ratio (Q_i/Q) is a function of L_i/F_WT (Figure 2). For lengths of inlet less than L_2, Q_i/Q is directly proportional to L_i/L_1, where L_1 is the intercept at $Q_i/Q = 1$. For lengths of inlet greater than L_2, Q_i/Q is proportional to $(L_i/L_3)^{0.4}$, where $L_3 = 1.65\ F_WT$.

The product F_WT is a measure of the gravity force acting on the flow and L_1, L_2, and L_3 are directly proportional to F_WT. In terms of the dimensions of the street section and the inlet,

$$F_W = (0.262/n)[(T - W)S_x]^{1/6} S^{1/2} \tag{1}$$

The term in brackets is the depth of the flow at the outer edge of the inlet depression.

The discharge Q is usually the independent variable, and the width of the flow upstream from the inlet is

$$T = (Qn/0.56S^{1/2})^{3/8} S_x^{-5/8} \tag{2}$$

which is based on the general equation for flow in a shallow triangular channel (2).

$$Q = 0.56(Z/n)d^{8/3}S^{1/2} \tag{3}$$

ANALYSIS OF EXPERIMENTAL DATA

Karaki and Haynie (3) plotted Q_i/Q as a function of L_i/F_WT, with W/T as a third parameter, to demonstrate the effects of geometric variables and surface roughness, but did not develop conclusions.

The analysis reported here is based on plotting Q_i/QL_i as a function of L_i/F_WT as shown in the lower portions of Figures 3 and 4 for cross slopes of 0.015 and 0.06 respectively. The symbols separate the data for each set of runs; L_i is the sole variable.

Q_i/QL_i is constant up to $L_i/F_WT = 0.4$ for $S_x = 0.015$ and $L_i/F_WT = 0.8$ for $S_x = 0.06$. Each curve then breaks downward with a slope of -0.6. If the constant value of Q_i/QL_i is taken from the plot and $Q_i/Q = 1$, a new value of L_i that is now defined as L_1 is obtained. The upper curves in Figures 3 and 4 are then plotted as L_i/L_1 versus the corresponding value of L_i/F_WT. This collapses all the data into a single line whose equation is

$$Q_i/Q = L_i/L_1 \tag{4}$$

L_1 is the x-intercept at $Q_i/Q = 1$.

At the breakpoint, $Q_i/Q = L_2/L_1 = 0.567$ and 0.749 for $S_x = 0.015$ and $S_x = 0.06$ respectively. The value of L_1/F_WT can be computed algebraically as $(1/0.567)0.4 = 0.705$ for $S_x = 0.015$ and $(1/0.749)0.8 = 1.07$ for $S_x = 0.06$.

The two characteristic lengths (L_1 and L_2) are functions of S_x and W. The length (L_3), at which 100 percent interception is attained, is $1.65\ F_WT$.

For lengths less than L_2, the inlet is performing essentially as a weir. The unit discharge is $Q_i/L_i = Q/L_1$ for inlet lengths up to L_2. Beyond that point, Q_i/QL_i varies as $(L_i/F_WT)^{-0.6}$ (Figures 3 and 4). Therefore, Q_i/L_i for a given configuration is constant for $L_i < L_2$ and decreases rapidly for $L_i > L_2$. This fact is economically significant because the cost of added increments of length is not justified by the rapidly decreasing flow increments.

The equations for L_1 and L_2 are

$$L_1/F_WT = 2.79W^{-1/6}S_x^{0.3} \tag{5}$$

$$L_2/F_wT = 3.67W^{-1/6}S_x^{0.5} \tag{6}$$

The ratio (L_2/L_1) is also the interception ratio (Q_i/Q) at which the two curves intersect and is

$$L_2/L_1 = 1.315S_x^{0.2} \tag{7}$$

For both cross slopes and $L_i > L_2$, the exponent of Q_i/QL_i is -0.6, so that the exponent of Q_i/Q for the line to the right of the breakpoint must be (1 - 0.6) = 0.4. The form of the equation is thus

$$Q_i/Q = (L_i/L_3)^{0.4} \tag{8}$$

L_3 is the value of L_i when $Q_i/Q = 1$, or $L_3 = (Q/Q_i)^{2.5}\,L_i$.

If the values of Q_2/Q and L_2 are substituted into this equation,

$$L_3 = 1.65F_wT \tag{9}$$

for both cross slopes.

Since the intersection of the two lines is not exactly on the 0.8 ordinate line in Figure 4, the coefficients in the above equations were adjusted slightly to satisfy Equation 9. The lack of definition is due to the small number of data points for $L_i > L_2$, which in turn is due to the limitation in discharge capacity of the experimental apparatus.

Not all sets of runs were used in Figures 3 and 4 because certain sets departed from the norm for unex-

Figure 1. Graphical definition of symbols.

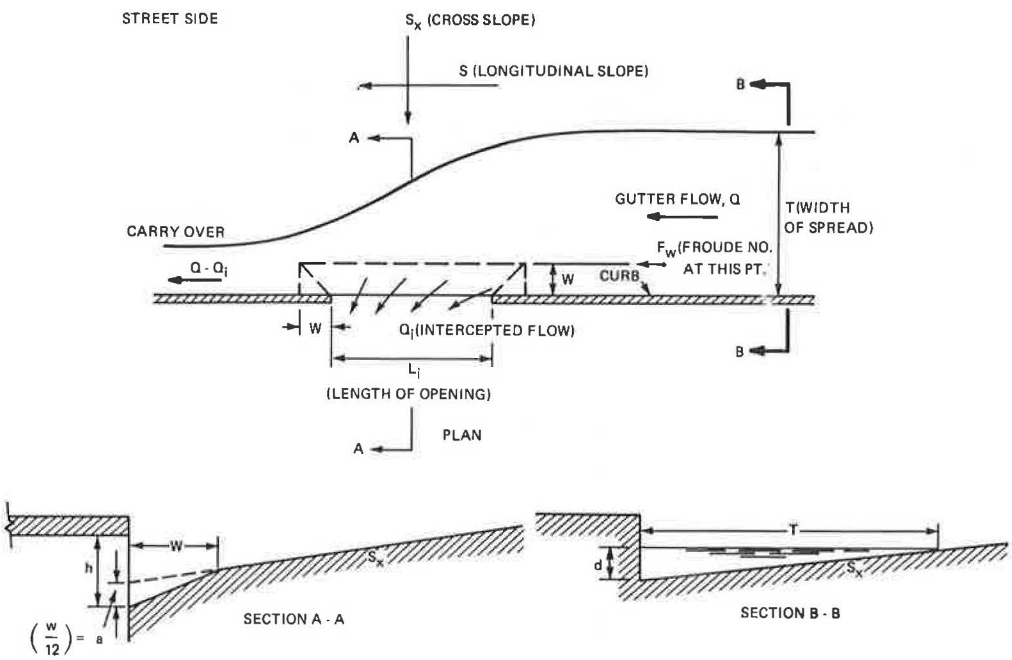
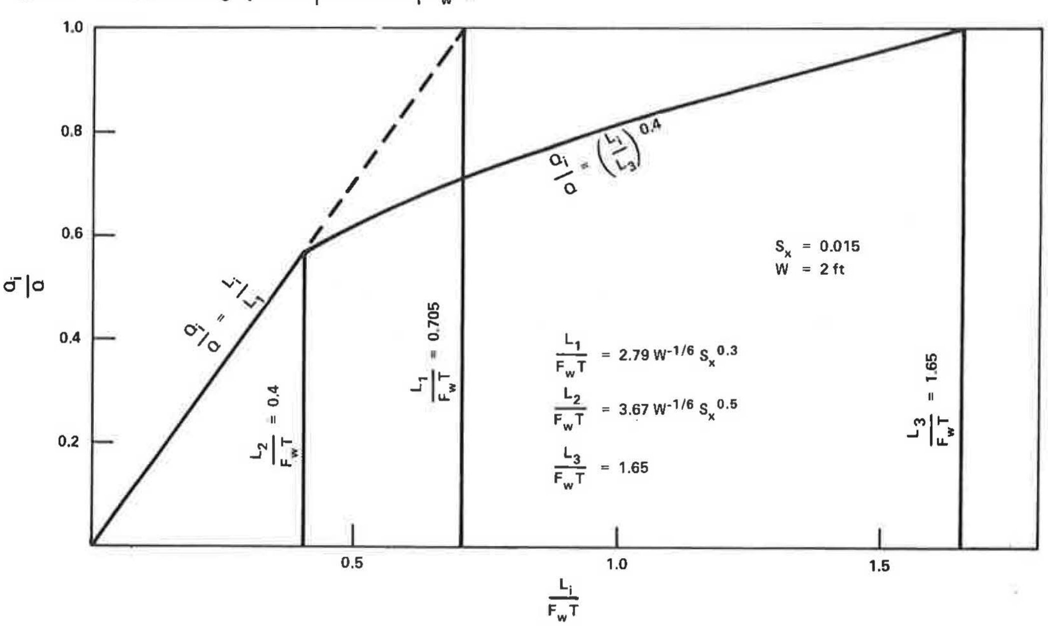


Figure 2. Dimensionless graph of Q_i/Q versus L_i/F_wT.



plained reasons. None of the runs for T = 5 and S_x = 0.015 were usable because there were too few points for $L_t < L_2$. There were only two sets of runs for W = 0.305 m (1 ft) and unfortunately they departed from the norm. To estimate the apparent relationship of W in Equations 5 and 6, the Froude model law was used by taking inlet configurations that were geometrically similar to those of a tested inlet and computing L_1 for inlets of several different ratios and the same W/T. F_W was computed by Equation 1 for the appropriate W in the range from 0.31 to 1.83 m (1 to 6 ft), and F_WT was then computed for each model. L_1 was computed by similitude and divided by the corresponding value of F_WT. This gave a new value of the coefficient in Equation 5, which varied from the initial value in inverse proportion to $W^{1/6}$. This relation was reasonably constant over a range of $0.1 < W/T < 0.3$, but may not be entirely correct and should be verified by additional experiments.

Comparison to Experimental Data

The values of Q_1/Q computed by using Equations 5 and 6 for all runs are plotted against observed values of Q_1/Q in Figures 5 and 6. An enveloping line has been drawn around the sets of data that departed from the norm and were excluded from the analysis. The straight line indicates perfect agreement between computed and observed results.

Figure 7 shows another comparison, that of Q_1 as a function of S where S_x = 0.015, n = 0.016, and W = 0.6 m (2 ft). The experimental runs were for n = approximately 0.095 and are plotted at the value of S that has the same value of F_WT as does n = 0.016. This is acceptable because Q_1/Q is a direct function of F_WT for a given L_1. The agreement is reasonably good up to L_t = 10.7 m (35 ft).

Figure 7 also shows values of Q_1 that were obtained by the use of the charts given in the Hydraulic Engineering Circular (2). A similar graph was plotted for S_x = 0.06, but is not included here. It also showed good agreement with experimental data, but the Q_1 values derived from the charts averaged about 20 percent lower than the computed values.

The experimental data available for subcritical slopes are limited, and until such time as more complete data are available, the equations in this paper may be used.

Change of Q_1 With Longitudinal Slope

For a given L_t, Q_1 increases as the 0.3rd power of S until $Q_1/Q = L_2/L_1$, the breakpoint in Figures 3 and 4. For steeper slopes, Q_1 is constant for a given length as required by Equation 4. This is supported by the experimental data for L_t = 5.

The incremental change in Q_1/L_t can be calculated by differentiating Equation 8.

Figure 3. Experimental data (S_x = 0.015).

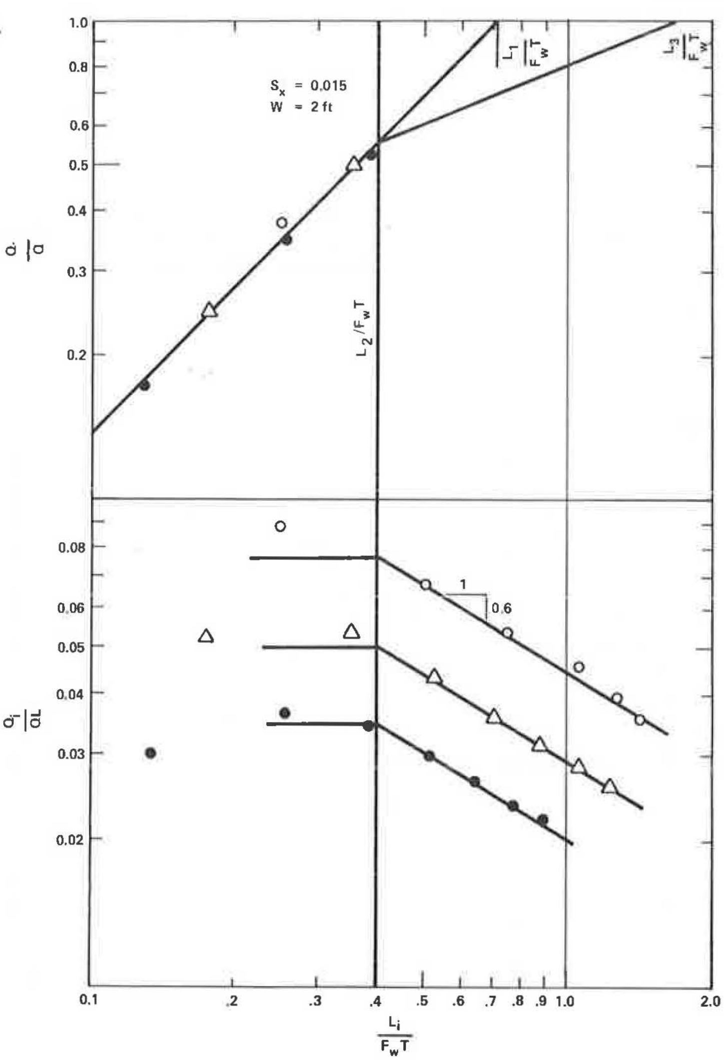
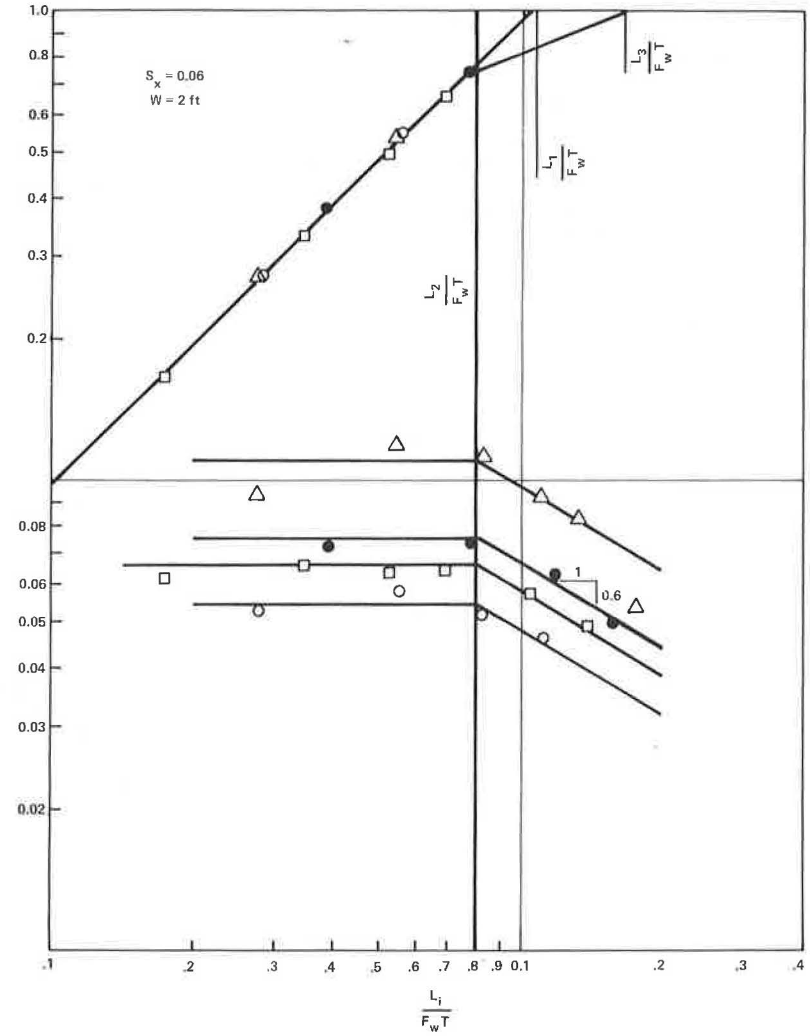


Figure 4. Experimental data ($S_x = 0.06$).



$$dQ_i/dL_i = (Q/L_3^{\ 0.4})0.4L_i^{-0.6} \quad (10)$$

This reduces to $dQ/dL_t = 0.4Q/L_3$ when $L_t = L_3$ and $Q_t/Q = 1$. Thus, the last increment of inlet needed to intercept all of the flow becomes a very small quantity and, if the length of an inlet could be limited to that which would intercept 90 percent of the flow, L_t/L_3 would become $0.9^{2.5} = 0.77$, which means that the inlet could be shortened by 23 percent.

Extrapolation to Steeper Slopes

The design charts (2) are limited to a maximum grade of 4 percent, which was the limit of the experiments. Figure 7 shows that the data can safely be extrapolated to an 11 percent grade. This is because the set of runs on a smooth surface at 4 percent had $F_wT = 39$, and this plots at S = 0.11 for a rough surface having n = 0.016. At some slope, there will be a possibility of the generation of roll waves, which are a function of F_w, but apparently this did not occur within the range of Froude numbers tested.

Inlet on Grade Compared to Same Inlet at Sump

At the point of zero grade (sump), the curb-opening inlet performs according to a modified weir formula (1) that has the following equation when W = 0.6 m (2 ft) (2):

$$Q_i = 1.7(L_i + 1.8W)[d_{max} + (W/12)]^{1.85} \quad (11)$$

in which $d_{max} = S_xT$ and $(L_t + 1.8W)$ is the effective weir length.

In the following analysis, the unit discharge $[Q_t/(L_t + 1.8W)]$ for the weir is compared to the unit discharge (Q_2/L_2) for the inlet on a supercritical slope. Q_2 is the value of Q_t at L_2, the point at which the Q_t curve in Figure 7 levels out. The results are plotted in Figure 8 for the same data sets used in Figures 3 and 4. The line of equality shows that, as an average, Q_2/L_2 is about 0.06 $ft^3/s/ft$ less than that for the inlet at the sump.

Validity of Equations in Intermediate Cross-Slope Range

The relation established above provides a way of checking the probable validity of Equations 4 and 5 for intermediate cross slopes. Computations using W = 0.6 m (2 ft), n = 0.016, S = 0.01, and T = 3.05 m (10 ft) were carried out for cross slopes of 0.03, 0.04 and 0.05. Equations 1, 3, 4, and 5 were combined so that Q_2/L_2 reduces to $Q_2/L_2 = 28.3\ S_x^{1.2}$. The values computed from this equation are plotted against $Q_2/(L_2 + 3.6)$ in Figure 8. These values are slightly higher than those found for the experimental data in the middle range and tend toward divergence beyond $S_x = 0.06$. (Since this value is probably approaching the maximum cross slope, the divergence is not important.) Thus, Equations 4 and 5 can safely be used for interpolation between $S_x = 0.015$ and $S_x = 0.06$.

Composite Cross Section

The street cross section commonly used has a steeper cross slope within the gutter width than in the pavement. Consequently, the equations derived in this paper do not apply strictly.

The composite section is advantageous hydraulically and from the point of view of traffic because it concentrates more of the flow near the curb. Figure 9 shows the capacity of the composite section relative to that of the straight section for the case where the gutter width is 0.6 m (2 ft) and the cross slope is $^1/_{12}$. The computations for these curves are based on the method suggested in the Hydraulic Engineering Circular (2). The differences are negligible for steep cross slopes and large values of T.

No experiments have been run for curb-opening inlets on composite cross sections. However, it is reasonable

Figure 5. Comparison of computed and observed values of Q_i/Q (S_x = 0.015).

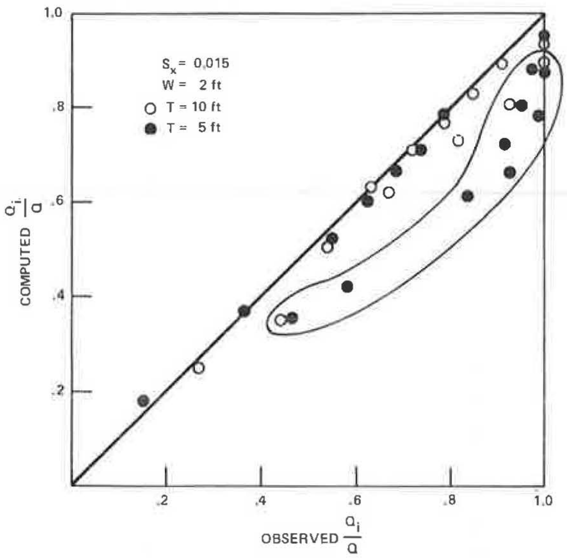


Figure 6. Comparison of computed and observed values of Q_i/Q (S_x = 0.06).

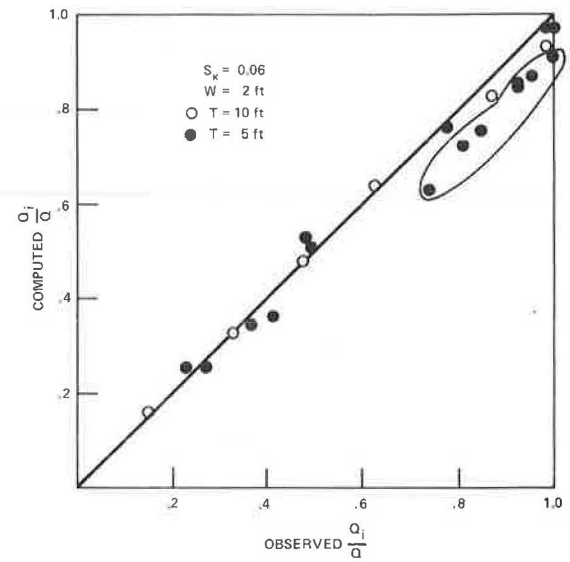
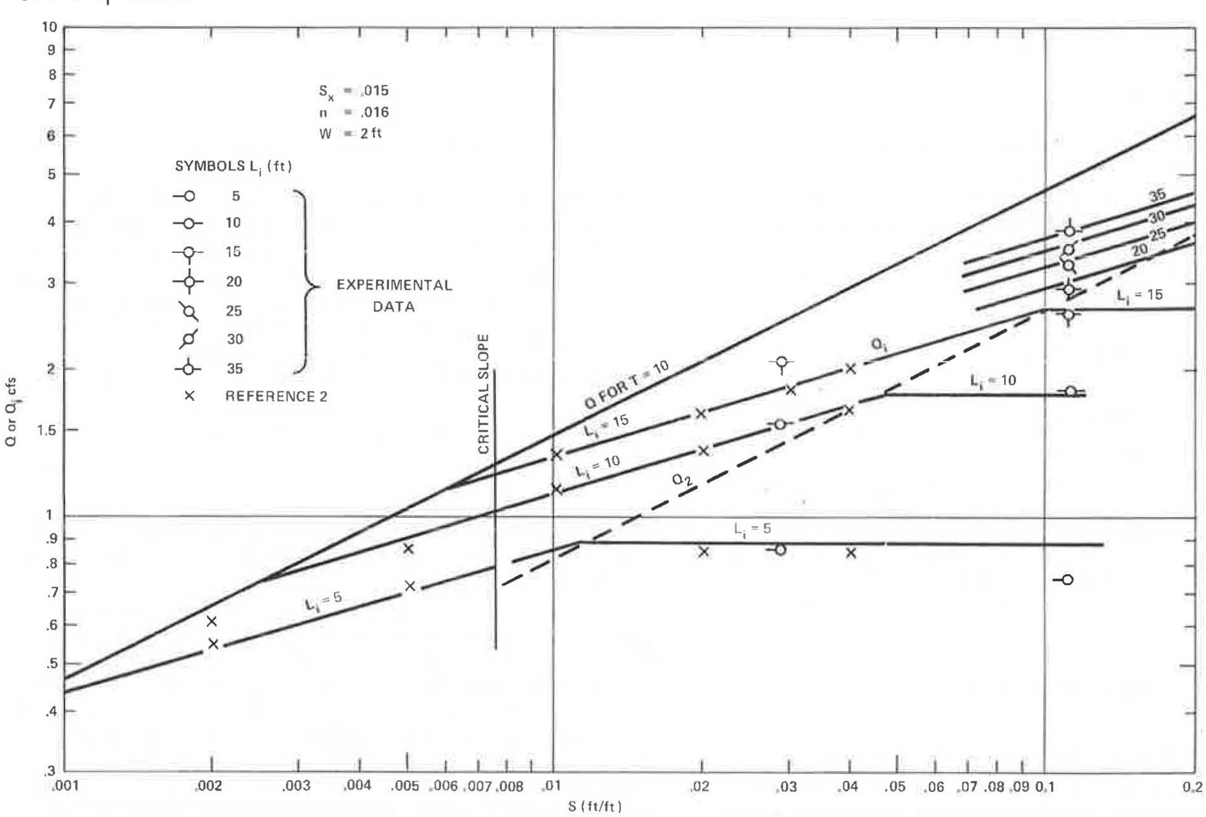
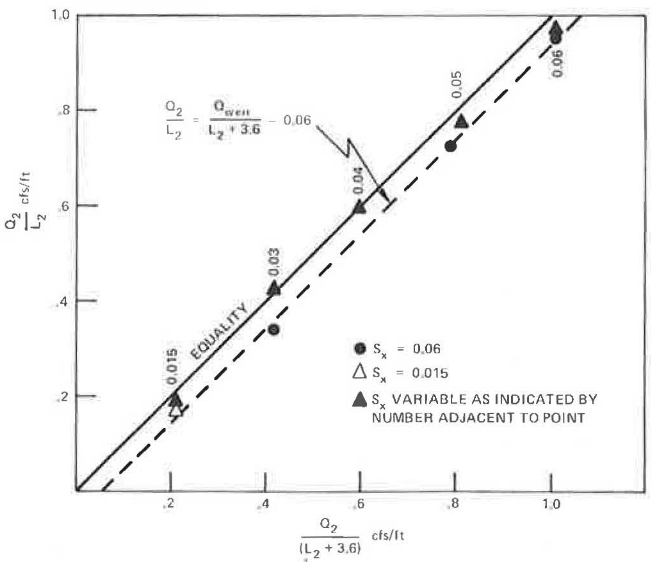


Figure 7. Q_i versus S.



to assume that all of the increment in flow in the deepened gutter will be intercepted if the width of the inlet depression is equal to or greater than the gutter width. Figure 9 shows a way of estimating the increment in flow that will be intercepted.

Figure 8. Comparison of unit discharge of inlet on grade to that of same inlet at sump.



APPLICATION OF SIMPLIFIED METHOD

The application of these equations to the design of curb-opening inlets or to the calculation of the capacity of existing inlets is illustrated in Tables 1 and 2. Fixed values of W = 2 and n = 0.016 are used, which reduce Equations 1, 2, 3, 5, and 6 to the forms shown below.

$$F_w = 16.4[(T - 2)S_x]^{1/6}S^{1/2} \tag{1a}$$

$$T = (Q/35S^{1/2})^{3/8}\,S_x^{-5/8} \tag{2a}$$

$$Q = 35S_x^{-5/3}T^{8/3}S^{1/2} \tag{3a}$$

$$L_1 = 2.49S_x^{0.3}F_w T \tag{5a}$$

$$L_2 = 3.27S_x^{0.5}F_w T \tag{6a}$$

In a design or evaluation, S_x, S, W, and n are generally known. Two other variables must also be known or be selected as design criteria. Usually the runoff rate (Q) at the point of design is known, or T may be given as the criterion for inlet spacing, which will determine Q. The other variable may be Q_i/Q by design criterion or a given L_i. Once the fixed variables are determined, the computations may be programmed for a digital computer, a handheld electronic calculator may be used, or the equations may be graphed for direct solution.

Example 1: The interception capacity of an inlet 2.4 m (8 ft) long is to be determined. Since $L_i < L_2$, Q_i/Q can be calculated by using Equation 4 and is 8/14.7 = 0.54, which makes Q_i = 2.0 ft³/s. The increment that

Figure 9. Ratio of discharge on composite section to discharge on straight section as a function of S_x and T.

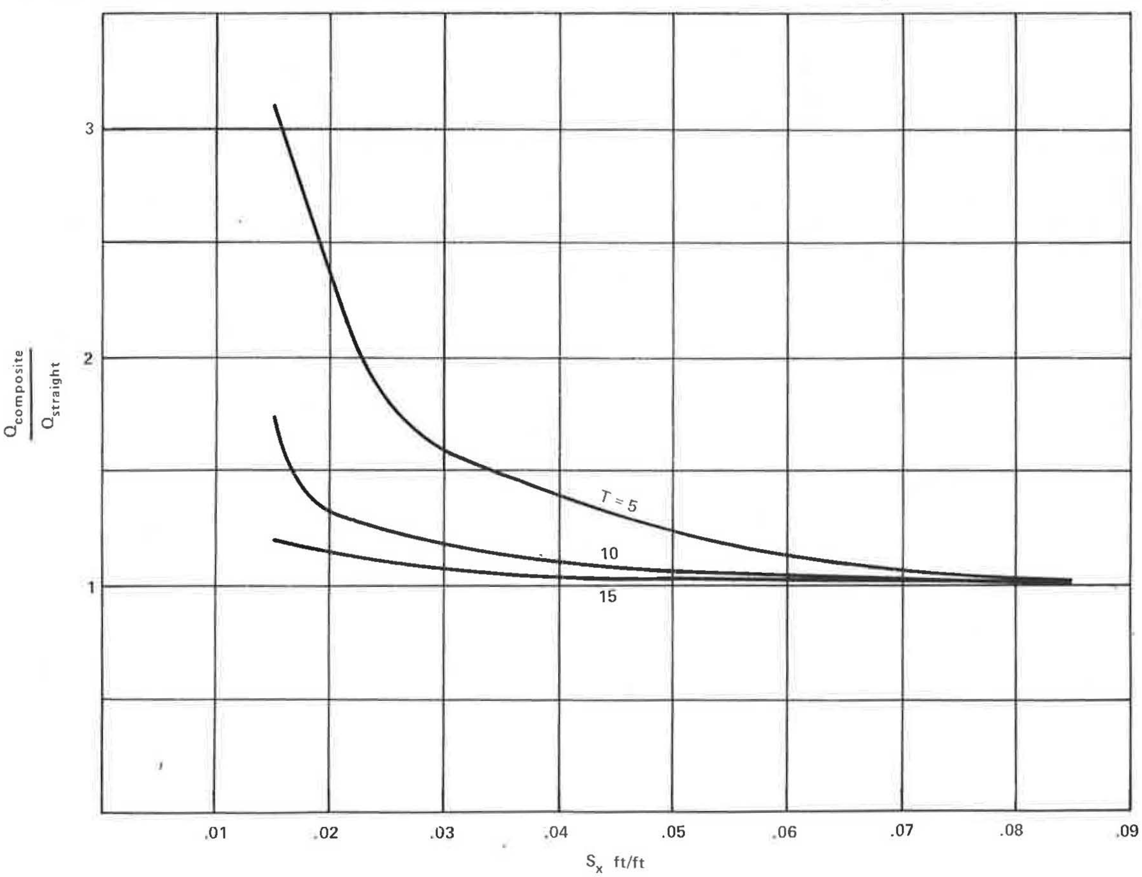


Table 1. Example of computations for design of curb-opening inlets: parameters.

Example	S_x	L_1/F_wT	L_2/F_wT	L_3/F_wT	S	T (ft)	$Q/S^{1/2}$ (ft^3/s)	Q (ft^3/s)	$F_w/S^{1/2}$	F_w	F_w T (ft)
1	0.02	0.770	0.462	1.65	0.025	10	23.6	3.73[a]	12.1	1.91	19.1
2	0.02	0.770	0.462	1.65	0.025	8.6	23.6	2.5[a]	11.7	1.85	15.9
3	0.02	0.770	0.462	1.65	0.025	8.6	23.6	2.5[a]	11.7	1.85	15.9
4	0.02	0.770	0.462	1.65	0.005	9.6	23.6	1.5[a]	12.0	0.847	8.47
5	0.04	0.948	0.654	1.65	0.005	8.0	41.9	3.0[a]	12.9	0.915	7.35

Note: 1 ft = 0.305 m; 1 ft^3/s = 0.028 m^3/s.
[a] Given values.

Table 2. Example of computations for design of curb-opening inlets: results.

Example	L_1 (ft)	L_2 (ft)	L_3 (ft)	L_2/L_1	Q_t/Q	Q_1 (ft^3/s)	Q_2 (ft^3/s)	L_t (ft)	Use L_t (ft)	Q_c (ft^3/s)	Q_{comp} (ft^3/s)
1	14.7	8.8	31.5	0.601	0.54	2.0	2.24	8[a,b]	8	1.7	3.2
2	12.2	7.3	26.2	0.601	0.90[a]	2.0	1.50	20.1[c]	20	0.3	2.7
3	12.2	7.3	26.2	0.601	0.80	2.0	1.50	15[a,c]	15	0.5	2.7
4	6.5	3.9	14.0	0.601	1.0[a]	1.5	1.00	14[c]	14	0	1.9
5	7.0	4.8	12.1	0.690	0.8[a]	2.4	1.80	5.6[c]	6	0.6	2.6

Notes: 1 ft = 0.305 m; 1 ft^3/s = 0.028 m^3/s.
[a] Given values. [b] $Q_i < Q_2$ and $L_i < L_2$. [c] $Q_i > Q_2$ and $L_i > L_2$.

would be added if the approach gutter of width W was sloped $^1/_{12}$ is read from Figure 9 and is 0.31 Q = 0.31(3.73) = 1.2 ft^3/s. As a check of the calculations L_2/L_1 should have the same value as computed by Equation 7.

Example 2: The length of an inlet to intercept 90 percent of the flow is to be calculated. In this case, $Q_t > Q_2$ and Equation 8 is used. This gives $L_t = (0.9)^{2.5}L_3 = 0.768(26.2) = 20.1$ ft. If this is rounded to 20 ft the carry-over discharge (Q_c) will be 0.3 ft^3/s.

Example 3: The capacity of a 4.6-m (15-ft) inlet for the conditions given in example 2 is to be calculated. Here $L_t < L_2$ and Equation 8 again is used. This makes $Q_t/Q = (15/26.2)^{0.4} = 0.80$ and $Q_t = 0.8(2.5) = 2.0$ ft^3/s.

Example 4: A subcritical slope is assumed, which makes $F_w = 0.856$, and 100 percent interception is required for Q = 1.5 ft^3/s. This could be an inlet at the end of a block where no carry-over is to be permitted. In this case $Q_t > Q_2$ and Equation 8 is used: $L_t = 1^{2.5}(14) = 14$ ft. This result could have been obtained more simply by noting that, when $Q_t/Q = 1$, $L_t = L_3$.

Example 5: S_x is 0.04, and 80 percent of the flow is to be intercepted. The required length of inlet is 5.6 ft or a 6-ft standard inlet. (If 100 percent interception is required, the length will be 12 ft = L_3.)

A detailed program for future research on curb-opening inlets is beyond the scope of this paper, but the recognition of certain deficiencies in the data points to the need for evaluation of the following:

1. Performance on subcritical slopes,
2. Performance on supercritical slopes up to about 15 percent,
3. Inlets on street sections having a gutter cross slope that is steeper than that of the pavement and on parabolic cross sections,
4. Effects of the width of depression on inlet performance,
5. Performance on at least one cross slope between 0.015 and 0.06 and possibly up to 0.10 (inlets on superelevated curves), and
6. Effects of guide vanes cast into the inlet-depression surface.

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As chief of the hydraulic research division at the time the experimental research (3) was conducted, I must accept responsibility for deficiencies in the experimental data that show up in this paper. The current study leading to this paper has not been supported by any agency.

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Development and Field Testing of a New Locator for Buried Plastic and Metal Utility Lines

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The research and development program for a new instrument to directly locate buried plastic and metal utility lines is described. Downward-looking radar was used as the location method. The operating instructions for the pipe locator are summarized, and the laboratory and field test results are discussed. The development has proceeded from computer-processed laboratory measurements to a completely self-contained, battery-powered, portable instrument that will soon be produced commercially.

Third-party damage is a major problem for utility companies and contractors and, with the increase in environmental regulations that require utility services to be buried, there has been an increase in the number of third-party damage incidents. A recent survey of third-party damage showed that lack of accurate knowledge of the locations of buried facilities is the main cause of dig ins.

Plastic pipe is widely used by gas and water utilities, primarily because of its ease of installation and freedom from corrosion. It cannot, however, be directly located with a conventional metal-pipe locator. Nonmetallic water lines are often buried without a tracer, but plastic gas lines usually have a metallic wire or foil tracer, which can be located with a metal detector, buried over the pipe. Various procedures are used to bury, find, and follow such tracers, but none are completely successful. In some cases, the contractor fails to install the tracer during construction, and in others, another contractor working in the same area removes the tracer and does not replace it.

In addition to the problem of their inability to locate plastic pipe, the present metal-pipe locators are of limited value in some situations. They do not accurately determine pipe depth and are often confused by nearby metallic objects such as parked cars.

Because of the number of accidents caused by excavation equipment, many local, state, and federal agencies are considering legislation that would severely limit the use of plastic pipe and require hand digging near known pipelines. In Europe, several countries already forbid the use of automated excavation equipment such as trenchers because of the danger of third-party damage.

Thus, an instrument that accurately locates buried plastic pipelines is urgently needed. Such a device should locate the pipeline directly and not depend on a metallic tracer for its operation.

RESEARCH PROGRAM

This paper reports the results of a research effort to develop a portable, easily operated locator for buried utilities. The goal of the program was to develop a self-contained battery-powered instrument that could be used by a semiskilled operator.

Several pipe-sensing methods were investigated. Sonic methods were rejected because of the difficulty of coupling sound into the ground. Magnetic and gravimetric methods were rejected because they are insensitive and too easily confused by other buried objects. Downward-looking or underground radar, which has been investigated for the U.S. Army for locating buried objects, appeared to be the most promising method. The successful development of the plastic-pipe locator required expertise in three areas—downward-looking radar, antenna design, and computer signal processing.

Downward-looking radar is similar to conventional radar in that both emit bursts of electromagnetic energy and listen for echoes reflected from objects in the signal path. The most important difference between the two is that the downward-looking radar signals are transmitted into the ground, rather than into the atmosphere. Since the earth is nearly opaque to conventional radar signals, special broadband impulses are used to penetrate the ground. A special antenna, more properly a transducer, is required to couple the impulses into the ground. For shallow-probing applications, there are many underground objects that can produce echoes, and the echoes are received very soon after transmission of the interrogating signal. The total time from transmission of the pulse to receipt of the echo is less than 100 ns, and impulses of 1 ns or less (compared to normal radar signals of 1 μs or longer) must be used. For this type of interrogating signal, echoes of different objects have different shapes. Figure 1 shows the basic underground radar system.

At the beginning of the research program in 1971, a complete plastic-pipe distribution system was built (Figure 2). This test system is identical to a current commercial installation, except that no tracer wire or foil was installed over the pipes. Several metal-pipe test targets were also installed at known depths, and the original soil was used to fill the trenches.

The initial work was the development and testing of an antenna that could see buried pipelines. To locate a buried object, some characteristic of the object that is different from its surroundings must be sensed. With metallic objects, magnetic properties are usually sensed because soil is nonmagnetic. But plastic pipe is nonmagnetic itself, and some other characteristic of the pipe must be sensed. For radar sensing, this characteristic can be conductivity, permittivity, or permeability. Then as the radar impulse passes through the earth, there will be a change (as when the impulse hits the pipe wall) that causes part of the impulse to be reflected. This method is equally easy for metal and plastic pipes.

A unique antenna was developed to discriminate between pipe and other buried objects. This antenna is much more sensitive to longitudinal objects than to objects having other shapes, which greatly simplifies signal processing. Some typical waveforms for this antenna are shown in Figures 3, 4, and 5. Figure 3 shows a time-domain echo waveform with several important features circled. The first feature, which is always seen, is a portion of the transmitted pulse energy. This feature is used to locate the time reference (zero-depth point). The next feature seen is a random variation that is due to the ground roughness. Finally, the pipe echo is seen, after a time delay that is proportional to the pipe depth.

Figure 4 shows the effect of moving the locator from a spot directly above the pipe. Figure 5 shows the effect of rotating the locator when it is directly above the pipe. (In both figures, the circled portion of the waveform contains the pipe echo.) With this antenna and data-processing technology, the time-domain echo waveform provides information on the presence of a buried pipe, its location, its depth, and its direction.

To an experienced observer, the shape and polarity of the echo also provide information about the identity of the target. The waveforms in Figure 6 show that the echoes of metal and plastic pipes have approximately the same shapes, but opposite polarities. Figure 7 shows the echo of a horizontal 0.3-m (1-ft)-square copper plate buried 0.1 m (4 in) below the earth surface. The double bump feature that characterizes this kind of target is circled. The separation of the two peaks is proportional to the size of the plate. The additional bumps in the echo clearly distinguish it from that of a buried pipe.

The remaining tasks in the research program were (a) to develop a reasonably priced, portable system that will give this information to the operator as clearly and simply as possible and (b) to verify the operation of the system under the wide range of soil types, ground-moisture conditions, and other environmental factors that occur in the real world.

The next phase of development of the plastic-pipe locator involved measurements on the test system in which the data were recorded and computer processed in the laboratory. This system allowed measurements under controlled conditions and, during it, the antenna design was optimized.

Figure 1. Basic underground-radar system.

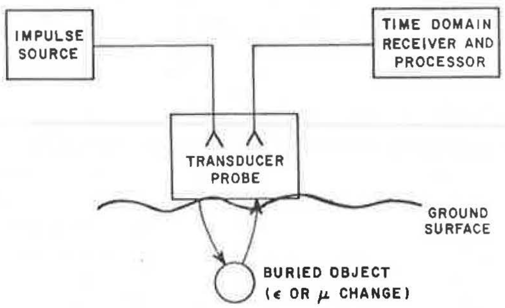


Figure 2. Experimental pipe system.

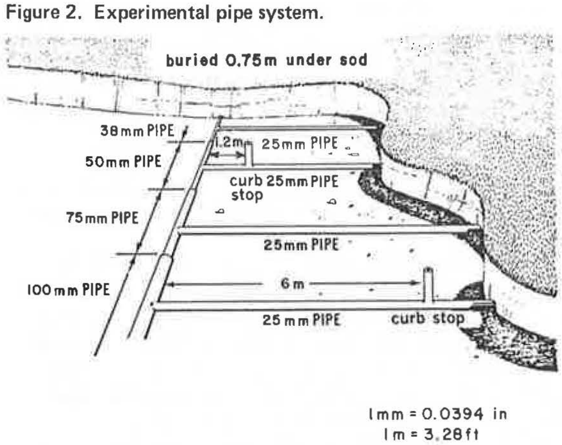


Figure 3. Typical plastic-pipe echo.

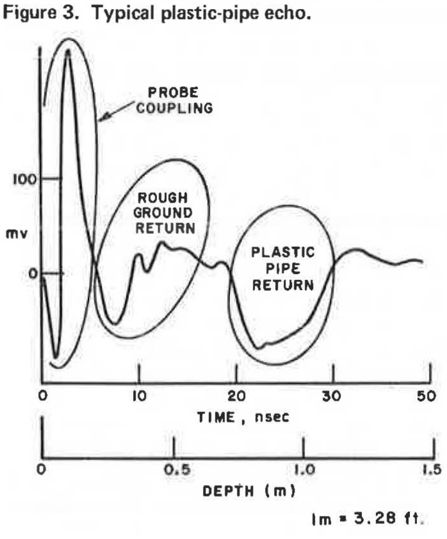


Figure 4. Change in echo as antenna is moved away from target.

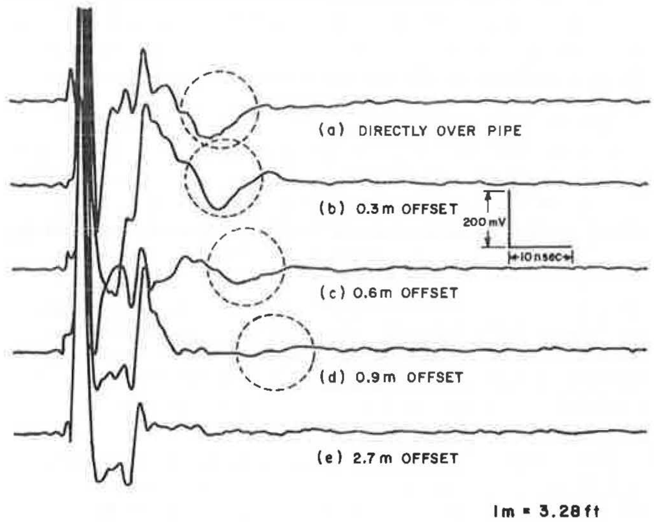
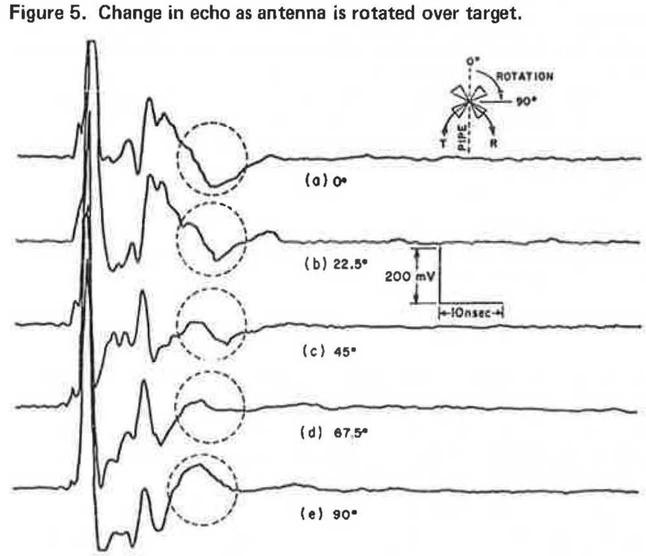


Figure 5. Change in echo as antenna is rotated over target.



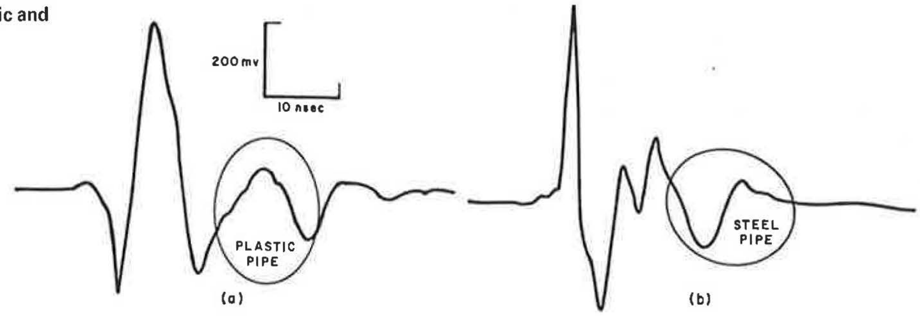


Figure 6. Comparison of plastic and metal-pipe echoes.

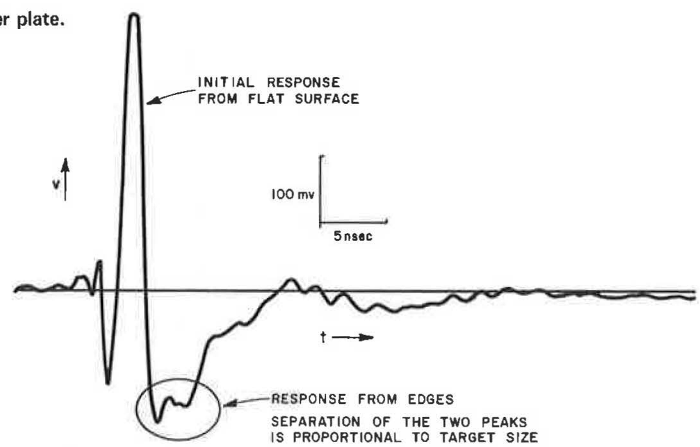


Figure 7. Echo from buried copper plate.

After this the development of the pipe locator included field measurements over buried plastic and metal pipes in the central Ohio area. These data were recorded on magnetic tape for later computer processing. The computer stored all of the test data so that various processing methods, including averaging, filtering, and Fourier transforms, could be tried.

The results of the field measurements showed that both plastic and metal pipes could be located at depths of up to 3 m (10 ft) with an accuracy in depth of better than ± 0.3 m (± 1 ft). The direction could be determined to within $\pm 15°$. Taps (tees) could be located to within ± 0.3 m (± 1 ft). The actual horizontal location of the pipe could be determined to within ± 0.3 m (± 1 ft), depending on its depth. These results exceeded the initial requirements for a plastic-pipe locator.

EXPERIMENTAL PIPE LOCATOR

After suitable antenna and processing systems had been developed, an ac-line-powered, portable pipe locator was constructed. This instrument weighed more than 45 kg (100 lb) and included a visual display of pipeline-echo waveforms for direct interpretation of the data. Trailing an umbilical cord to a cart of power supplies, this self-contained instrument demonstrated that it is possible to locate pipelines without computerized data processing.

Because the ultimate goal of the program was the development of a completely portable, battery-powered instrument, the next object was the construction of a lightweight, plastic-pipe locator. The availability of a commercial, time-domain reflectometer weighing 3.6 kg (8 lb) (including its rechargeable batteries) greatly simplified construction of the receiver portion of the locator. The complete pipe locator, shown in Figure 8, weighs 18 kg (40 lb).

Extensive testing of this instrument showed that it could locate both metal and plastic gas lines accurately. Its accuracy in determining location was verified by digging or by visual clues such as the locations of curb stops and service risers. Its accuracy in determining depth was verified by measuring the depth to the shut-off valve at the curb box. Experience with servicemen showed that a person having ordinary skills could be trained to operate the pipe locator in a few hours.

The data display is an interesting feature of the plastic-pipe locator. Most conventional pipe locators use an audio tone to indicate the presence of a pipe. However, the plastic-pipe locator gives more information about the pipe (such as its depth and whether it is plastic or metal), and a visual display was more appropriate. A cathode-ray-tube display was first considered, but was rejected because these are fragile and have high power consumptions and limited brightness. Also, the signal processing in the pipe locator is digital, and this would have required conversion to analog form for the cathode-ray tube. Instead, a matrix of light-emitting diodes 10 rows high by 16 columns wide was used in the prototype instrument.

PRODUCTION PIPE LOCATOR

The ultimate success of the plastic-pipe locator depends on its wide use by contractors and utilities. With this goal in mind, a commercial firm has been licensed to manufacture and market the instrument. Two prototype production locators are currently being field tested. The commercial instrument is shown in Figure 9.

The original experimental locator was of one-piece construction. Although it worked well, it was difficult to transport and store. The production instrument is of three-piece construction and is much easier to use. The parts of the instrument are the console, which has the controls and display, the battery power supply, and the antenna probe, which weighs only a few kilograms. It has weatherproof connectors so that the parts can be quickly separated for storage. The antenna is watertight and can be put in puddles without damage. Figure 10 shows the control panel and hooded display, which is bright enough to be viewed in direct sunlight. A power cord for operation from a 12-V automobile electrical system is provided, and the rechargeable battery can be changed in the field. The performance characteristics of the instrument are summarized below (1 mm = 0.39 in; 1 m = 3.3 ft; 1 kg = 2.2 lb).

Performance Characteristics	Description
Depth capability	50-mm diameter plastic or metal pipe to 3 m deep
Accuracy	Within 0.15 m of depth and position with digital readout of pipe depth; ability to distinguish closely spaced pipes to within 0.3 m
Mechanical specifications	Three instrument parts; 22 kg total weight; watertight antenna housing; weatherproof instrument housing
Power supply	Self-contained rechargeable battery or 12-V automobile electrical system
Data display	Monolithic gas discharge, 11 rows high by 32 columns long, approximately 50 mm high by 100 mm long, viewable in direct sunlight

The production pipe locator is of solid-state construction for reliability and reduced maintenance. Other than recharging its battery, no periodic maintenance is required. It is designed to operate in all weather conditions and is unaffected by extremes in temperature.

In the production instrument, the display matrix is 11 rows high by 32 columns long. The eleventh row is used as a cursor to aid in determining pipe depth. Instead of light-emitting diodes, a glow-discharge (neon), monolithic matrix is used. It is much brighter and easier to read than the original display.

USING THE PLASTIC-PIPE LOCATOR

The operation of the plastic-pipe locator is quite simple, but some training of the operator is required. This training involves learning to recognize typical pipe echoes and the procedure for determining pipe type and direction.

Figure 8. Experimental pipe locator.

Figure 9. Prototype of production-model pipe locator.

Figure 10. Control panel of pipe locator.

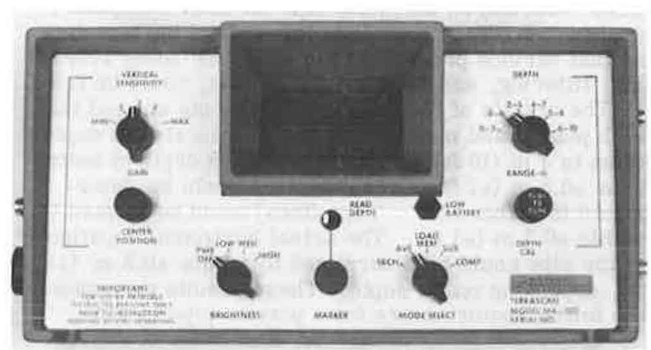
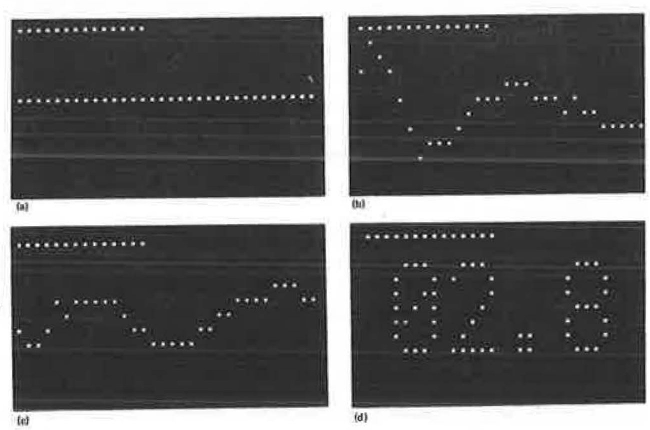


Figure 11. Display showing (a) no target, (b) target, (c) target reversal, and (d) pipe depth.



The operation of the instrument is best learned at a location where there are known buried pipes.

The operator first places the antenna on the ground at a place where there are no buried pipelines to establish a background or clutter level. (Clutter is defined as a signal from anything but a pipe.) The clutter level is less than the return echo from a pipe but varies with soil type, amount of buried rock, and ground cover. When the antenna is not above a pipe, the operator sees a single row of lights across the display as shown in Figure 11a. If the clutter is high, the row may have a slight ripple. The top partial row of lights is the cursor.

To find a pipe, the antenna is placed on the ground and the display read. The locator is then moved in 0.3 or 0.6-m (1 or 2- ft) steps until the pipe is found. As the pipe is approached, the line of light becomes a sinusoidal curve, whose maximum amplitude occurs when the antenna is directly above the pipeline (Figure 11b). To find the direction of the pipe, the antenna is rotated to again find the point of maximum amplitude. This establishes the pipe as being in one of two orthogonal directions, and moving the antenna in either direction establishes the actual direction of the pipe.

Once a pipe is located, it may be followed. If the pipeline has a tee, a null (straight line) is seen over the tee location. If the line changes from plastic to metal or vice versa, the phase of the waveform on the display reverses. This can be seen by comparing Figures 11b and 11c.

The plastic-pipe locator has several display modes. An average mode is provided for use where clutter is high. Averaging greatly reduces the effect of clutter while retaining location accuracy, but slightly reduces locating speed.

A compare mode is provided so that it is possible to store an echo waveform, move the antenna, and then display the stored and the new waveforms alternately. A characteristic of pipe echoes is that, if the antenna is rotated through 90°, the phase of the echo reverses. This feature is useful in distinguishing between clutter and pipe echoes because clutter does not reverse phase.

A useful feature of the production pipe locator is a cursor that is controlled from the operator's panel. As the control is turned, the eleventh row of the display lights up in one-column increments. When the end of the lighted row matches the center of the echo, and a button is pushed, the display will digitally show pipe depth. This is shown in Figure 11d.

TEST PROGRAM

The program for testing the plastic-pipe locator had two goals—to test the instrument in a wide range of field conditions and to obtain the impressions of service persons who will use the instrument. The initial test program concentrated on locating pipelines where the exact pipe location was known or where later excavation could verify it.

A severe test of the instrument took place at a chemical plant where the right-of-way for a buried electrical cable was surveyed. The plant is very old and the locations of most pipelines, including some untraced plastic ones, were in doubt. The consequence of cutting into some of these pipelines would have been to shut the plant down, and some of the lines carried acid. In a total survey time of 9 h, 600 m (2000 ft) of right-of-way were traced, and 69 pipeline crossings were found. Some were documented from existing drawings and the others were found by later excavation.

The environmental conditions encountered in this test were severe. The weather was usually below freezing. At times, the antenna was put down in a puddle of water. The ground cover included crushed limestone, dirt, asphalt paving, unreinforced concrete, and slag. Much of the area was near power lines, railroad tracks, steel tanks, and piping. In one location, the antenna was placed between the rails of a railroad track and a pipeline was located below. Most of the pipelines found were metal, but it would have been impossible to use a conventional metal-pipe locator successfully because of interfering metal objects.

A wide range of soil types and conditions were used in testing the pipe locator. Plastic and metal pipes were successfully located in clay, loam, sand, and fill with large rocks and debris, and through asphalt and unreinforced concrete. Moisture content has little effect on the operation of the pipe locator, and pipes have been found under several inches of snow and through frozen ground.

The plastic-pipe locator is far superior to existing metal-pipe locators in finding metal pipelines and conduits. Metal-pipe locators are subject to interference by above-ground metal objects, such as fences and automobiles, and some units will not operate in the vicinity of overhead power lines. The plastic-pipe locator is completely unaffected by above-ground objects. The ability of the instrument to individually locate pipelines spaced as closely as 0.3 m (1 ft) from each other is very valuable where multiple utilities must be traced.

In actual field tests, the plastic-pipe locator accurately located plastic and metal pipelines more than 3 m (10 ft) deep. It can determine depth and location to within 0.3 m (1 ft) and direction to within 15°. The locator can determine whether the pipeline is plastic or metal. Pipes as small as 25 mm (1 in) in diameter have been located, and plastic pipes 50 mm (2 in) in diameter and 3 m (10 ft) deep have been located.

The pipe locator has been shown to service persons who will use the production instrument. Their comments have been favorable: Their usual question is, When can I have one?

Field testing is continuing, and production is scheduled to begin by the end of 1977.

The use of the plastic-pipe locator is not limited to the location of pipelines. The instrument has been successful in locating buried power, telephone, and data cables, and also sewer lines. It can be used in agriculture to locate and trace field drain tile. In buildings with unreinforced concrete floors, it can be used to locate electrical conduits and water and sewer lines embedded in the floor. Although it cannot be used to locate objects through reinforced concrete, it can be used to detect the reinforcing rods.

Thus, downward-looking radar offers a solution to the problem of locating buried utilities, regardless of their composition. The unit sees plastic pipe, clay drain tile, and concrete storm sewers as well as metal pipes, cables, and conduits. It provides more information and is more reliable than previous detectors, even on metal pipes.

OTHER APPLICATIONS

This system should also be evaluated in a larger context. It is one example of the capability of underground radar for sensing buried man-made objects and natural features. Other research has already established that underground radar can define geological features to depths on the order of 30 m (100 ft), can find the water table, and is valuable in archaeological research. A theoretical study has predicted that sensing to great depths is feasible. Another study is concerned with the location of objects a short distance ahead of digging apparatuses. With continued research and development, more applications of underground radar will be found, thus providing inexpensive, practical information about "what's down there" to a large group of scientists, engineers, and industrial users.

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Design Procedure for Uncased Natural-Gas Pipeline Crossings of Roads and Highways

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A method for designing uncased natural-gas pipeline crossings of roads and highways is presented. The procedures used are not new, but are adaptations of techniques that have been thoroughly tested and validated. The method involves a combined calculation of the internal hoop stress in the pipe that results from the operating pressure of the pipeline and the external stresses that result from dead load, live load, and impact loading. The internal stress is calculated by the Barlow formula, and the external stresses are calculated by the Spangler formula, which incorporates the Marston theory for the calculation of dead load and the Boussinesq point-load theory for live load. A brief resumé of the history of the use of cased pipeline at highway crossings is given to explain the reasons for using it in the past. Advances in the technology of steel making, pipe manufacture, pipeline construction, nondestructive inspection, pressure testing, cathodic protection, and maintenance inspection are listed to support the increased use of uncased pipeline. The improved cathodic protection of the carrier pipe that is available in uncased crossings is given as the primary justification for their use.

This paper presents a method for the design of uncased, natural-gas pipeline crossings of roads and highways. The design criteria used are equal to or exceed the specifications of the federal pipeline-safety standards (8).

BACKGROUND

History of Cased Pipeline at Highway Crossings

The practice of encasing pipeline crossings of highways dates back to the beginning of natural-gas pipeline construction and was undoubtedly used with other types of pipelines earlier. Early pipelines were constructed of cast iron or low-strength steel, and their sections were joined by screwed connections, mechanical couplings, or bell and spigot joints. The pipe was not coated or cathodically protected to prevent corrosion and, as a result of joint failure and corrosion, leaks that required repair or replacement of pipe sections developed (3, 4). The use of casings at road crossings provided a relatively simple and economical way to repair and replace pipes under roads without affecting the surface use of the roads.

Although welding has become a common method of joining pipe sections and coatings have been developed to protect the pipe from exposure to the factors that cause corrosion, the use of casings at highway crossings has continued for several reasons. Among these are the lack of integrity of the circumferential welds produced by using the oxyacetylene or bare-electrode, manual-arc processes, lack of sophisticated welding inspection techniques, and inadequate cathodic protection techniques.

Justification for Uncased Pipeline at Highway Crossings

Over the past 20 years, progress in all areas of pipeline technology has resulted in a pipeline network that has one of the best safety records of any form of transportation. Technological progress that has affected pipeline safety includes

1. The development of high-strength steels with improved ductility and notch toughness;
2. Improved processes for manufacturing steel plate for pipe, which results in fewer internal flaws in the pipe wall;
3. Advanced welding techniques for making the longitudinal seam in pipe joints;
4. Modern inspection techniques, including ultrasonic and radiographic procedures, for quality control of pipe in the manufacturing process;
5. Standardized shipping procedures that reduce shipping-related damage;
6. New coating materials and application techniques that result in better bonding and improved protection of the pipe surface from corrosive environments;
7. Improved welding techniques and materials for joining pipe;
8. The use of radiographic techniques for on-site inspection of pipe welding during construction;
9. Strength testing of pipeline segments following construction to at least 90 percent of yield;
10. Installation of cathodic protection systems that virtually eliminate pipe corrosion and the resulting leakage; and
11. Detailed inspection and monitoring procedures for the operation of pipelines.

Because of these and many other improvements in pipeline design, construction, and testing and operating procedures, the requirement for encasing pipeline crossings of highways is today greatly reduced. Most authorities on pipeline operations now recognize that encasing pipelines severely reduces the cathodic protection of the encased carrier pipe. Thus, encasing, which was once a simple and economical means for maintaining, repairing, and replacing pipeline sections under roadways, is now itself a major maintenance problem. More and more agencies and organizations concerned with pipeline safety, including the National Transportation Safety Board (NTSB) (10), the U.S. Department of Transportation Office of Pipeline Safety Operations, the National Association of Railroad and Utility Commissioners (NARUC), the American Petroleum Institute (API), and the American Society of Mechanical Engineers (ASME), are recommending that the use of cased pipeline at highway crossings be discontinued.

Development of Design Criteria

Although the movement toward the use of uncased pipeline at highway crossings is relatively recent, the development of acceptable design criteria for uncased crossings is at least 15 years old. The design procedure presented in this paper is not new, but rather is adapted from several methods developed primarily under the sponsorship of the American Society of Civil Engineers (ASCE) Research Council on Pipeline Crossings of Railroads and Highways. In 1955, the first revision of the American Standards Association code for pressure piping (13) included design criteria for natural-gas pipelines that were based on the population density adjacent to the pipeline and for uncased pipeline crossings of roads, highways, and railroads. To provide consistency in the design criteria, the criteria established for uncased pipeline crossings in each population-density classification were equivalent to those for cased pipeline crossings for the next higher population-density classification. This resulted in a 10 to 12 percent decrease in the operating-stress level and a 20 to 25 percent increase in safety factor over the adjacent pipe sections. The validity of this design criteria has been thoroughly documented (6). The design criteria are included in the present federal pipeline safety standards.

On the basis of the ASCE studies and independent research, Spangler (2) developed the design procedure that is often referred to as the Iowa formula (3, 4, 5). This has become the most widely accepted procedure for the design of uncased pipeline highway crossings and is used by the API (7).

Unfortunately, the use of cased pipeline crossings has acquired the status of infallibility that accompanies age and experience. As a result, there is great reluctance among highway engineers, designers, and administrators to accept uncased pipeline crossings of highways in spite of the mass of test data that supports present design criteria. Very few states permit uncased pipeline crossings under any conditions, and those that do generally impose stringent restrictions in factors of pipe diameter and operating pressure or stress level or both. A summary of present state policies on casing is given in Policies for Accommodation of Utilities on Highway Rights-of-Way (11).

Approved Design Procedure

The design procedure for uncased crossings presented here uses the Barlow formula for the calculation of internal pipeline stress and the Spangler method for the calculation of external stresses, including dead load, live load, and impact loading. The Barlow formula for determining the pipe-hoop stress that results from internal loading is taken from the federal pipeline safety standards (8) and includes a design factor known as the class location that limits the level of internal stress on the basis of the population density adjacent to the pipeline. A complete description of the recognized class locations and the applicable design factors can be found in the federal pipeline safety standards (8).

The Spangler method (1, 2) for the calculation of external stress loading incorporates the Marston theory for the calculation of dead load and the Boussinesq point-load theory (1, 3) for the calculation of live load, including impact loading. The Spangler formula includes several design parameters that may be varied to satisfy the overall design concept of the approving authority. These parameters include the load coefficient, the wheel load, the impact factor, and the bending and deflection parameters.

A brief description of three of these factors, the load coefficient, the bending parameter, and the deflection parameter, and the range of variability of each is appropriate at this point. The load coefficient is a factor in Marston's calculation for loads on pipes in trenches. It is a function of the ratio of the height of backfill or earth above the pipe to the width of the ditch or the diameter of the bored hole. It is also a function of the internal friction of the soil backfill and the coefficient of friction between the backfill and the sides of the ditch. The original formula recognized five different classes of soil. The factor generally accepted today is the soil class that Marston labeled ordinary maximum for clay (thoroughly wet). However, higher and lower values of this factor are available and may be used where conditions warrant. Values of this factor are given in Table 1.

The bending and deflection parameters are derived from Spangler's work (1, 2) and are dependent on the distribution of load over the top half of the pipe and the resultant distribution of the bottom reaction. The load distribution over the top half of the pipe may be considered as uniform, but the bottom reaction depends largely on the extent to which the pipe settles into and is supported by the soil in the bottom of the trench. In bored installations, in which the bored hole normally exceeds the pipe diameter by 5.1 cm (2 in) or less, the bottom reaction is considered to occur over an arc of up to 120°. In open-trench installations, in which the width of the ditch may exceed the pipe diameter by 0.3 m (1 ft) or more, the bottom reaction is generally assumed to occur over an arc of 30 to 60°. Here again, the factors have a considerable range of variation that depends on the conditions at the particular installation. This latitude of design should readily satisfy the requirements of the approving authority. Values of deflection and bending parameters are shown below (12).

Width of Uniform Reaction (°)	Parameters	
	Deflection	Bending
0	0.110	0.294
30	0.108	0.235
60	0.103	0.189
90	0.096	0.157
120	0.089	0.138
150	0.085	0.128
180	0.083	0.125

The following parameters were used in the development of the design procedure presented here.

1. Class-location factor: By agreement, the class-location design factor was taken as 0.50, the design factor specified for class location 3, for all installations in class locations 1, 2, and 3. Since uncased crossings in class locations 2 and 3 are required to have a design factor of 0.50 under the federal pipeline safety standards, the cost impact of using this factor in class 1 locations also will be minimal. This design factor limits the maximum allowable operating pressure of the pipeline to a pressure that will produce a hoop stress of 50 percent of the specified minimum yield strength. A class-location design factor of 0.40 was used for instal-

Table 1. Safe working values of C_d for calculation of loads on pipes in trenches.

H/B[a]	C_d Minimum Possible Without Cohesion[b]	Maximum for Ordinary Sand[c]	Completely Saturated Topsoil	Ordinary Maximum for Clay (thoroughly wet)[d]	Extreme Maximum for Clay (completely saturated)[e]
0.5	0.455	0.461	0.464	0.469	0.474
1.0	0.830	0.852	0.864	0.881	0.898
1.5	1.140	1.183	1.208	1.242	1.278
2.0	1.395	1.464	1.504	1.560	1.618
2.5	1.606	1.702	1.764	1.838	1.923
3.0	1.780	1.904	1.978	2.083	2.196
3.5	1.923	2.075	2.167	2.298	2.441
4.0	3.041	2.221	2.329	2.487	2.660
4.5	2.136	2.344	2.469	2.650	2.856
5.0	2.219	2.448	2.590	2.798	3.032
5.5	2.286	2.537	2.693	2.926	3.190
6.0	2.340	2.612	2.782	3.038	3.331
6.5	2.386	2.675	2.859	3.137	3.458
7.0	2.423	2.729	2.925	3.223	3.571
7.5	2.454	2.775	2.982	3.299	3.673
8.0	2.479	2.814	3.031	3.366	3.764
8.5	2.500	2.847	3.073	3.424	3.845
9.0	2.518	2.875	3.109	3.476	3.918
9.5	2.532	2.898	3.141	3.521	3.983
10.0	2.543	2.918	3.167	3.560	4.042
11.0	2.561	2.950	3.210	3.626	4.141
12.0	2.573	2.972	3.242	3.676	4.221
13.0	2.581	2.989	3.266	3.715	4.285
14.0	2.587	3.000	3.283	3.745	4.336
15.0	2.591	3.009	3.296	3.768	4.378
Very Great	2.599	3.030	3.333	3.846	4.545

[a] Height of fill above top of pipe to breadth of ditch a little below the top of the pipe.
[b] These values give the loads generally imposed by granular filling materials before tamping or settling.
[c] Use these values as safe for all ordinary cases of sand filling.
[d] Use these values as safe for all ordinary cases of clay filling.
[e] Use these values only for extremely unfavorable conditions.

lations in class location 4 (multistory buildings).

2. Load coefficient: The values for ordinary maximum for clay (thoroughly wet) were taken from Marston's tables.

3. Wheel load: A value of 9072 kg (20 000 lb) was assumed.

4. Impact factor: A value of 1.5 (i.e., a nonrigid pavement) was assumed.

5. Bending and deflection parameters: The bending and deflection parameters were taken as the values for 0° arc (point loading) and for 60° arc for bored and for open-trench installations respectively.

COMPARISON OF DESIGN PROCEDURE WITH OTHER DESIGN CRITERIA

The design for uncased pipeline crossings at highways that results from the use of this procedure will be quite conservative. For comparison, the API recommended practice (7) uses design parameters of a 6800-kg (15 000-lb) wheel load and a 120° bottom-reaction arc for the deflection and bending parameters for bored installations, and the federal pipeline safety standards (8) require the use of a class-location design factor for the next higher class location.

IMPLEMENTATION OF DESIGN PROCEDURE

The installation of uncased pipeline at highway crossings is not a common procedure, and the design and installation practices have not yet been standardized. At present, API is considering updating their recommended practice (7), revising it to include hydrocarbon-gas pipelines, and developing a standard procedure that would be acceptable to all of the states and to the railroads. It is unlikely that any company will adopt a standardized design procedure until it is determined whether the results of this activity are generally accepted and approved.

Because one of the major reasons for using uncased pipeline at crossings is improved cathodic protection for the carrier pipe, the protection of the pipe-coating material during installation is an important concern. There are various procedures available for this protection. The most effective procedure presently available for use in long bored installations is cement coating. This is a field-applied coating and is quite costly. Thus, for long bored installations, particularly those using large diameter pipe, the economics at present favor the use of casing with a heavy petroleum or wax filler in the void space.

Cost Comparison of Cased Versus Uncased Pipeline

A significant factor in the use of uncased pipeline for highway crossings is the reduced installation cost. Uncased pipeline eliminates the need for the additional casing pipe, casing seals, casing insulators between the casing and the carrier pipe, casing vents, and, in some instances, casing filler. The installation of an uncased crossing should cost about 25 percent less than that of a cased crossing (or 40 percent less if casing filler is required).

The saving in operating cost is equally significant. The federal pipeline safety standards require that all pipelines be patrolled and that their cathodic protection be monitored on a periodic basis. Cased crossings must be checked as a part of this monitoring procedure to determine whether a short has occurred in the cathodic-protection system between the carrier pipe and the casing. Shorts may result from physical contact between the carrier pipe and the casing because of movement or settling of the carrier pipe or as a result of the failure of the casing end seals, which permits groundwater to enter the casing and provides a contact path between the carrier pipe and the casing. The long-term performance of the available seals has been poor. If a short occurs, the pipe must be excavated to clear the short or to install casing filler to protect the carrier pipe from environmental factors that would cause, or accelerate, corrosion. The cost of clearing casing shorts may vary from several thousand dollars to several hundred thousand dollars, depending on the depth of cover and the conditions encountered in the excavation. Such expenditures could be eliminated by the use of uncased pipeline.

PIPELINE DESIGN PROCEDURE FOR UNCASED HIGHWAY CROSSINGS

Basic Design Formula

This procedure provides a means for determining the combined stress exerted on an uncased pipeline at a road crossing. The combined stress (for the purposes of this procedure) is considered to be the sum of the stress due to internal pressure and the stress created by external loading (soil and vehicular). The combined stress is determined as follows:

$$S_T = S_I + S_E = (PD/2t) + (0.024\,92 K_b WEDt)/(Et^3 + 3K_z PD^3) \quad (1)$$

$$W = C_d \delta B_D^2 + (3LDI/2\pi H^2) \quad (2)$$

where

S_T = total combined stress (kilopascals),
S_I = hoop stress due to internal pressure (kilopascals),
S_E = hoop stress due to external loading (kilopascals),
P = internal pipeline pressure (kilopascals) (which may not exceed the pressure determined by Barlow's formula using design factors of 0.50 in class 1, 2, and 3 locations and 0.40 in class 4 locations),
D = outside pipe diameter (meters),
t = nominal wall thickness (meters),
K_b = bending parameter [for bored installations at 0° = 0.294, and for open-trench installations at 60° = 0.189 (see text table)],
W = total external load (kilograms per linear meter) of pipe (includes soil dead load and vehicular live load),
E = modulus of elasticity of steel [206.8 GPa (30 000 000 lb/in²)],
K_z = deflection parameter [for bored installations at 0° = 0.110, and for open-trench installations at 60° = 0.103 (See text table)],
C_d = load coefficient (Table 1),
δ = unit weight of soil [use 1922 kg/m³ (120 lb/ft³) unless the unit weight of the highway subsoil material is known],
B_D = width of pipe trench or diameter of bored hole (meters),
L = wheel load = 9072 kg (20 000 lb),
I = impact factor (use 1.5 for nonrigid pavement and 1.0 for rigid pavement), and
H = height of soil over pipe (meters).

Sample Calculations

Bored Installation

Assume the following conditions: (a) pipe diameter = 0.508 m (20 in), (b) thickness of pipe wall = 0.0103 m (0.406 in), (c) pipe grade = 5LX-42 [specified minimum yield = 289.590 MPa (42 000 lbf/in²)], (d) maximum operating pressure = 5.516 MPa (800 lbf/in²), (e) class 1 location, (f) minimum cover depth = 1.52 m (5 ft), (g) acceptable wheel load = 9072 kg (20 000 lb), (h) unit weight of soil = 1922 kg/m³ (120 lb/ft³), and (i) impact factor = 1.5. Determine the total combined stress at maximum internal pressure and at zero internal pressure.

1. Calculate the internal stress at maximum internal pressure :

$$S_I = PD/2t = 5516 \times 0.508 \times 10^2/2 \times 1.03$$
$$= 136.025\ \text{MPa} \quad (19\,704\ \text{lbf/in}^2) \quad (1a)$$

2. Calculate the stress due to external load:

$$S_E = 29.42 \times 10^{-3} K_b WEDt/(Et^3 + 3K_z PD^3) \quad (1b)$$

where

K_b = 0.294,
K_z = 0.110,
B_D = 0.56 m (1.82 ft) [bore 0.051 m (2 in) larger than D],
H = 1.52 m (5 ft),
H/B_D = 2.72, and
$W = 1.951 \times 1922 \times 0.56^2 + (3 \times 9072 \times 0.508 \times 1.5)/(2 \times 3.142 \times 1.52^2)$
$= 1176.19 + 1428.68$
$= 2604.87$ kg/m (144.76 lb/in),

which gives $S_E = 29.42 \times 10^{-3} \times 0.294 \times 2604.87 \times 206.8 \times 10^6 \times 0.506 \times 1.03 \times 10^{-2}/\{[206.8 \times 10^6 \times (1.03 \times 10^{-2})^3] + (3 \times 0.110 \times 5516 \times 0.508)^3\} = 52.475$ MPa (7550 lbf/in²).

3. $S_T = 136.025 + 52.475 = 188.500$ MPa (27 254 lbf/in²) and the specified minimum yield = 188.500/289.590 = 65 percent.

Open Trench Installation

Assume the following conditions: (a) pipe diameter = 0.61 m (24 in), (b) pipe wall thickness = 0.0127 m (0.500 in), (c) pipe grade = 5LX-60 [specified minimum yield = 413.700 MPa (60 000 lbf/in²)], (d) maximum operating pressure = 8.274 MPa (1200 lbf/in²), (e) class 1 location, (f) minimum cover depth = 1.22 m (4 ft), (g) acceptable wheel load = 9072 kg (20 000 lb), (h) unit weight of soil = 1922 kg/m³ (120 lb/ft³), and (i) impact factor = 1.5. Determine the total combined stress at maximum internal pressure and at zero internal pressure.

1. Calculate the internal stress at maximum internal pressure:

$$S_I = PD/2t = 8274 \times 0.61/2 \times 1.27 \times 10^{-2}$$
$$= 198.706\ \text{MPa} \quad (28\,800\ \text{lbf/in}^2) \quad (1a)$$

2. Calculate the stress due to external load:

$$S_E = 29.42 \times 10^{-3} K_b WEDt/(Et^3 + 3K_z PD^3) \quad (1b)$$

where

K_b = 0.189,
K_z = 0.103,
B_D = 0.91 m (3 ft) (based on standard bucket width),
H = 1.22 m (4 ft),
H/B_D = 1.33,
C_d = 1.12 (from Table 1), and
$W = 1.12 \times 1922 \times 0.91^2 + (3 \times 9072 \times 0.61 \times 1.5)/(2 \times 3.142 \times 1.22^2)$
$= 1782.6 + 2662.8$
$= 4445.4$ kg/m (249.7 lb/in),

which gives $S_E = 29.42 \times 10^{-3} \times 0.189 \times 4445.4 \times 206.8 \times$

$10^6 \times 0.61 \times 1.27 \times 10^{-2}/[(206.8 \times 10^6 \times 0.500^3) + (3 \times 0.103 \times 8274 \times 0.61^3)] = 39.446$ MPa (5742 lbf/in^2).

3. $S_T = 198.706 + 39.446 = 238.152$ MPa (34 542 lbf/in^2) and the specified minimum yield = 238.152/413.700 = 57.6 percent.

CONCLUSIONS

It appears obvious to many pipeline operators that the use of uncased pipeline at highway crossings is preferable to the use of cased pipeline in the present conditions of pipeline design, construction, testing, and operating technology. There are proven design procedures and criteria that will assure the operating safety of uncased crossings. Cathodic protection of uncased carrier pipe is much greater than that of cased pipe, which significantly reduces a major cause of pipeline leaks. Uncased crossings have definite economic advantages to both highway departments in terms of initial costs and pipeline operators in terms of initial and operating costs. The use of uncased crossings has been recommended by the NTSB and is being favorably considered for recommendation by NARUC. Technologically outmoded objections against the use of uncased pipeline at highway crossings should be put aside, and procedures for the approval of the use of uncased pipeline should be inplemented.

ACKNOWLEDGMENT

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Coordinating Utility Relocations as a Function of State Highway Agencies

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Coordinating the relocation of utility facilities from the construction area for new highways and accommodating them on existing rights-of-way is an important consideration of state highway agencies. Data on the various divisions, bureaus, departments, sections, and units of the highway agencies that have the responsibility for this particular area of work were obtained from a questionnaire submitted to all 50 states and the District of Columbia. The results were tabulated and analyzed on the basis of 45 replies. It was concluded that all utility-related functions, such as preliminary engineering, estimates, liaison, coordination, plan development, and review and approval, for both highway projects and new utility installations should be referred to one central office or one individual in each district office for those agencies that are so structured.

The utility-relocation function has various concepts, levels of responsibility, and locations in different state highway agencies. Some agencies have a central office that is responsible for the required liaison with utility companies throughout the entire state. Others have divided the function on a geographical or political basis.

THE PROBLEM

All state highway agencies engage in projects that necessitate the adjustment or relocation of utility facilities. Liaison is the necessary coordination that allows the work to be performed and is the relation between the agency and the public utilities and between the divisions within the framework of the organization and the Federal Highway Administration (FHWA). Liaison between the utilities and the consulting engineers employed by the state during the highway design stage is also necessary. Liaison can be defined as a form of appreciation; that is, appreciation of the views and problems of others and taking the necessary steps toward making an overall plan that resolves or compromises differences.

In many instances, there is insufficient coordination between highway planning and utility planning. The utilities must provide service to meet the needs of changing patterns of land use and transportation. If economic efficiency is to be realized, utility impacts must be considered during the design stage of the highway planning process.

To avoid unnecessary delays and costs in the construction and maintenance of highway improvements and to protect existing facilities, utility companies should be advised sufficiently in advance as to the effects of such construction on existing or proposed facilities so that they will have time to design the required adjustments, budget the required money, procure the necessary materials, supplies, and equipment, and schedule and perform the work.

One major problem encountered in clearing utilities from a construction area is the coordination of highway-contractor activities and utility-company work schedules. Labor difficulties, material shortages, and difficult construction also complicate the situation. The wish by utilities to use the highway right-of-way is desirable, so long as the capacity, safety, and appearance of the highway can be preserved. To ensure these, such use and occupancy must be authorized and reasonably regulated by each agency. The dual interests of the states and the utilities in the matter of multiple use of highway rights-of-way have been discussed by the American Association of State Highway and Transportation Officials (AASHTO) (1), and most agencies have adopted specific policies and procedures for this purpose. A summary of these policies and procedures has been given in another report (2), and the FHWA policy on accommodation of utilities is contained in the Federal-Aid Highway Program Manual (3).

Within the various state agencies, the existing concepts, regulations, policies, and responsibilities are not clearly defined, and control over the coordination of utility relocations is usually fragmented.

TREND

Certain states, FHWA, and the Transportation Research Board (TRB) consider the utility-relocation function to be one of great importance. In 11 states, this function is considered to be of sufficient importance for the listing of a representative in the AASHTO Directory of Member Departments. In FHWA, this function has branch status, and TRB has a Committee on Utilities. The American Public Works Association has a National Utility Location and Coordination Council.

FHWA has established policies and procedures for adjusting and relocating utility facilities on federal-aid highway projects and prescribes the extent of federal-aid participation in costs incurred by utility companies in such relocations. It also provides for an alternate simplified procedure for processing utility relocations or adjustments that may be implemented at the election of a state.

By requiring the states to submit written policies and procedures for the administration and processing of federal-aid utility adjustments and requiring each state to develop a utility-accommodation policy, FHWA has brought additional importance to the utility-relocation function. Prior to this, some states had no formal policies, and others had conflicting policies.

QUESTIONNAIRE AND ANALYSIS

The coordination of utility relocations as a function of the state highway agency was studied by analysis of the replies to a questionnaire that was submitted to the highway departments of all 50 states and the District of Columbia. The cover letter specifically directed the questionnaire to the person in charge of utility relocations and adjustments. The results tabulated and the analysis given are based on 45 replies. The following subjects were evaluated.

1. Do the same or different offices handle liaison for utility relocations caused by highway projects and for new utility installations on existing rights-of-way?
2. What states have an FHWA-approved utility-accommodation policy and utility-relocation-procedure manual or both?
3. How many states use or plan to use the alternate procedure provided by FHWA for federal-aid projects?
4. What states use a master utility or blanket agreement with utility companies?
5. On which utility relocations is federal aid requested?
6. At what level and in which division does the utility-relocation function best belong?
7. With what other divisions in the agency are the most contacts made?
8. What is the purpose of type of contact made with other divisions?
9. What is the basis for the organizational arrangement?
10. Have any states recently changed the location of the utility-relocation function?

(Division as used in this paper is defined as a subunit, such as design, construction, maintenance, legal, or finance, of the highway agency.)

Titles

The various titles of persons in charge of utility relocations include utilities engineer, chief of utilities bureau, chief of utilities section, development engineer, right-of-way engineer, right-of-way agent, manager, supervisor, administrator, and others. The distribution of titles is shown in Figure 1. Seven agencies have separate bureaus or divisions to handle the total utility-relocation function. The distribution of titles of persons in charge of processing permits for new utility installations is shown in Figure 2. The most commonly used are maintenance or permit engineer or utility engineer. In 19 agencies, the same individual is in charge of both permits for new utility installations on existing highway rights-of-way and highway-related relocations.

Division Contacts

The distribution of divisions in charge of utility relocations is shown in Figure 3. The right-of-way division is in charge in 16 states, the design division in 13, and the utilities division in 7. Figure 4 shows the distribution of divisions in charge of permits for new utility installations on existing highway rights-of-way. Maintenance divisions most frequently have this function.

Policies and Procedures

All of the agencies surveyed have a utility-accommodation policy, and 42 of them have received FHWA approval. Thirty-five agencies have prepared utility-relocation procedure manuals and 28 of these have received FHWA approval. The data collected from the questionnaire have been updated by the Washington office of FHWA.

The FHWA-approved alternate procedure is used by 11 states, and 5 more plan to use it. Several agencies were undecided. In most states, the reason for not using the alternate procedure was the fear that a later audit or review would result in nonparticipation for certain items over which there might be a difference of opinion.

Some form of a master or blanket utility agreement is used to reduce repetitious paper work by 21 agencies.

Federal aid was requested for most utility-relocation expenses by all but two of the agencies responding. The number of agencies requesting federal aid for various types of utilities is shown in Figure 5.

All but one agency indicated that utility companies were permitted to use consultants to design highway-related relocations, but seven agencies did not have a formalized procedure for this purpose.

Utility-Relocation Function as a Separate Division

To the question of whether the utility-relocation function should be at the division level, 19 agencies responded yes. Some of the reasons are given below:

1. Putting all utility functions together would increase efficiency and provide better management control.
2. The utility-relocation function is a major activity with great responsibilities and varied and complex problems.
3. The complexity of this function requires greater and more uniform controls to improve overall utility operations.
4. Because utility relocation affects many areas of other divisions, there would be better coordination if there were a broad intradivisional relationship that gave better staff and line relations.
5. Being at division level would ensure that the utility-relocation function was included in all planning stages.
6. The utility-relocation function involves all stages of highway projects, and the procedures and inspections used are specialized and do not relate to other divisions.
7. The utility-relocation function is usually independent of other divisions.
8. The utility-relocation function should be combined with the railroad-relocation function and be all inclusive—before, during, and after construction.

Twenty-five agencies responded negatively to the idea of upgrading the function to a division level. One agency did not answer this question. Of the 25, several of these were departments of transportation, some of whom evidently misinterpreted the question as implying that the utility function would be on the same level as, for example, the division of highways, rather than be a subunit of the highway division.

The reasons given for not upgrading the function to division level are summarized below.

Figure 1. Distribution of titles: person in charge of utility-relocation function.

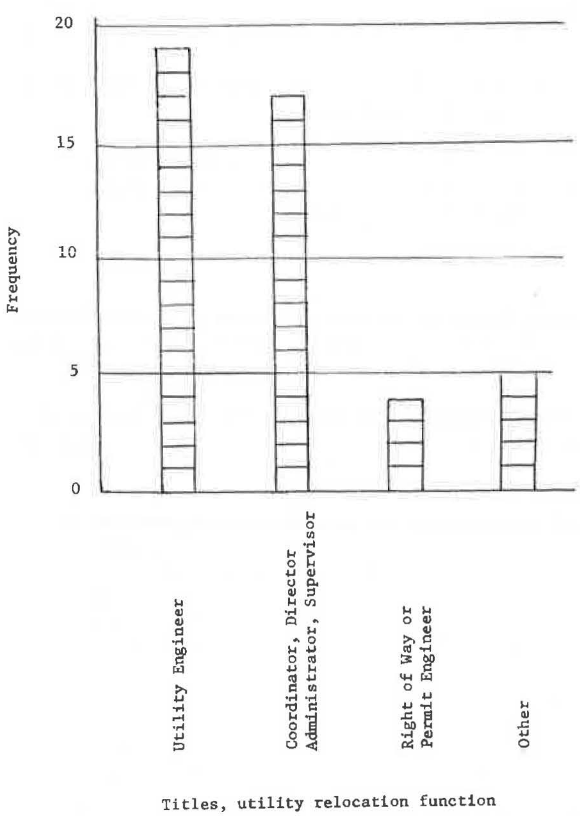
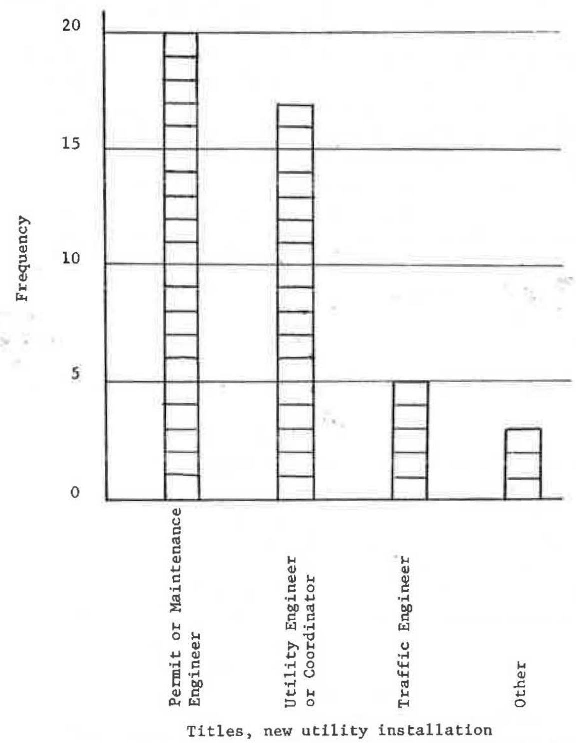


Figure 2. Distribution of titles: person in charge of permits for new utility installations.



1. Benefits would be minimal.
2. The coordination of the function is better if it is in, for example, the right-of-way division or the design division.
3. Utility relocation is a support, not a regulating or operating, function.
4. The present situation is satisfactory (this was the most commonly given reason).

The replies to the question, "If the utility function were not at a division level, in which division should it be?", are shown in Figure 6. The 14 replies that recommended the right-of-way division gave the following reasons:

1. Mutual benefits would accrue from joint use of the right-of-way.
2. The right-of-way division, by its nature, crosses intradivisional lines, and this would be conducive to better communications.

Figure 3. Distribution of divisions in charge of utility relocations.

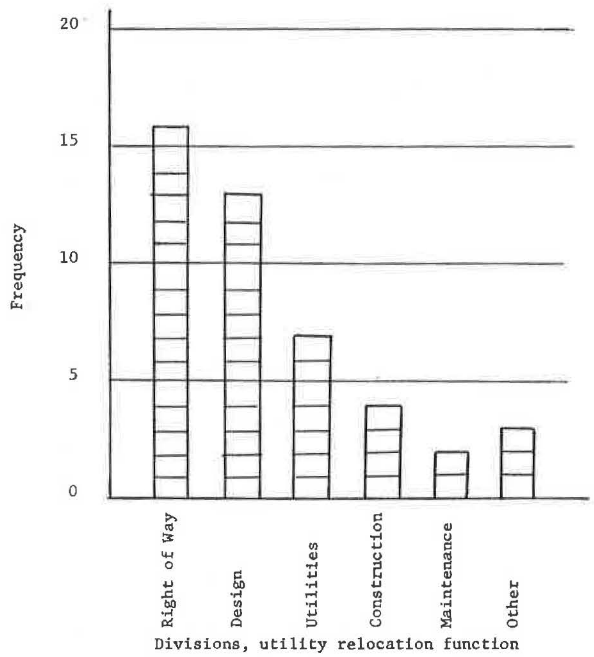


Figure 4. Distribution of divisions in charge of permits for new utility installations.

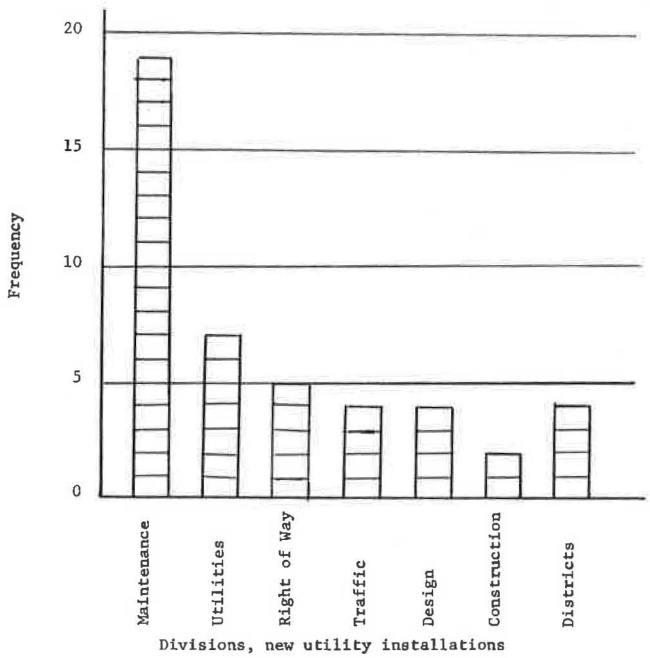


Figure 5. Number of agencies requesting federal aid for various types of utilities.

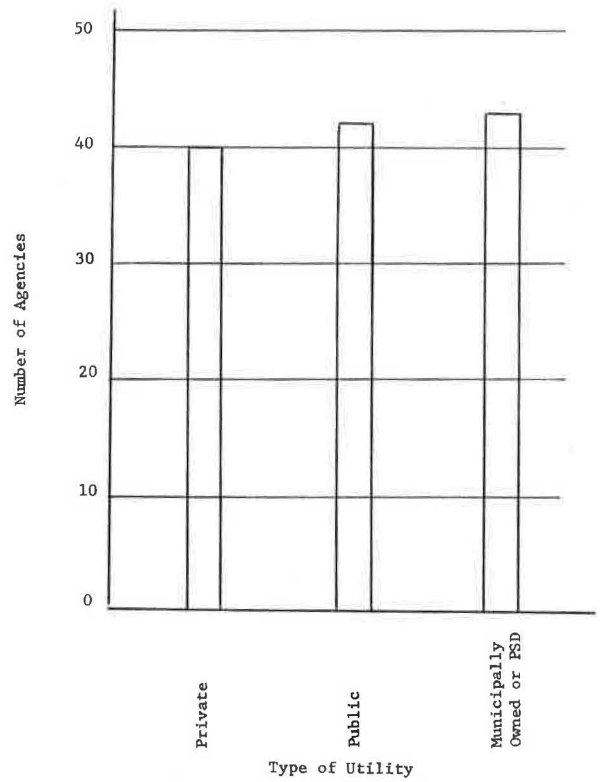


Figure 6. Distribution of recommendations for divisions to best handle utility-relocation function.

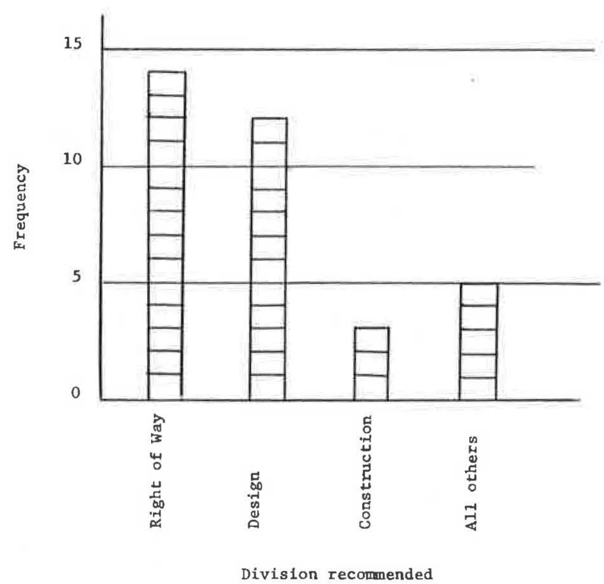
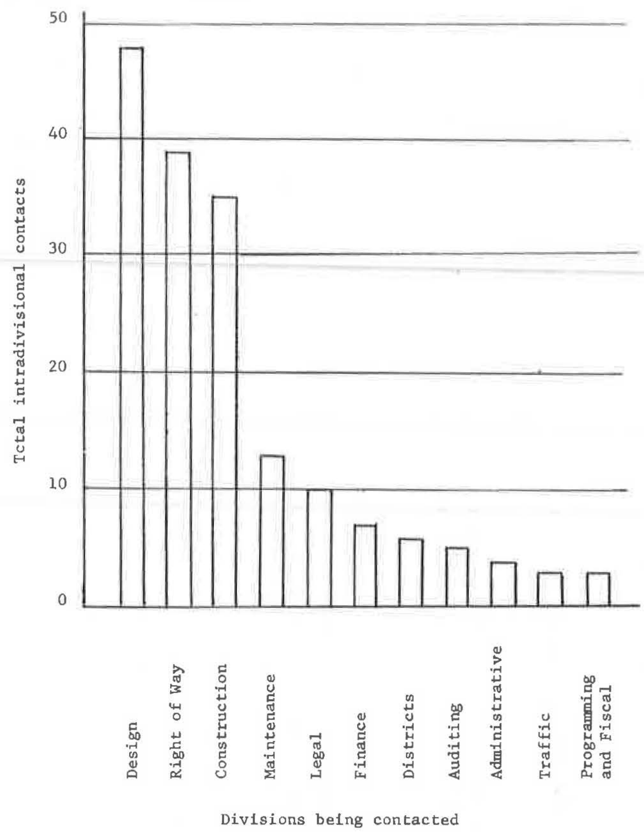


Figure 7. Distribution of intradivisional contacts concerning utility-relocation function.



3. Certain items of relocation are normally considered a part of acquisition, and the cost of appraisals could be similarly considered.
4. The right-of-way division is the most efficient in associating and negotiating with utilities because many utilities are represented in the American Right-of-Way Association.
5. Relocating utilities is a part of clearing the right-of-way.

Design and plans-development divisions had 12 recommendations. Some reasons given were that

1. Closer contact with the development division can affect changes and reduce costs by minimizing utility relocations,
2. Better liaison between plans and utility cost estimates could be maintained,
3. A closer working relation with plan development would be realized, and
4. Correlation of development priorities and anticipation of the work load would be facilitated.

Construction divisions had three recommendations: The main reason given was that a closer, more direct line of communications could be developed at the district level.

Divisions that received single recommendations were engineering services, road operations, traffic, information and liaison, and facilities development.

Intradivisional Contacts

The grouping of involved divisions shown in Figure 7 was derived by combining the division in which the utility-relocation function is now located with the list of its most frequently contacted divisions. The closest contacts were those with the design, the right-of-way, and the construction divisions, and second closest contacts were those with the maintenance, legal, and auditing or finance divisions. There was much less involvement with district offices and traffic-operations divisions.

Location of Function

In four states there is a legal basis for locating the utility-relocation function in a specific division, and in three states there is a partial legal basis for doing so. In all other cases the primary reason for the location of the function is organizational in nature. In one state, Texas, utility relocations are defined by law to be a right-of-way cost and expense, which requires the utility section to be in the right-of-way division.

During the last 5 years, 13 agencies have changed the location of the utility-relocation function. These moves are summarized below.

Division Moved From	Division Moved To	No. of States
Right-of-way	Design	5
Right-of-way	Construction	1
Right-of-way	Utilities	1
Design	Right-of-way	2
Construction	Right-of-way	1
Construction	Design	1
General services	Design (and back again)	1
Utilities	Maintenance	1

The frequency of changes suggests that trial and error attempts are being made to locate the optimum position for this function.

SUMMARY AND CONCLUSIONS

Because of geographical, political, and organizational variations, it is impossible to recommend a specific location for the utility-relocation function that is appropriate for every agency. However, a number of factors can be considered.

1. The early involvement of utilities in the project planning process is desirable to determine the optimum final solution and best overall plan.
2. A reduction in the number of offices with which utilities must coordinate is desirable.
3. A reduction of the number of interdepartmental review contacts would reduce processing time, which would reduce delay and costs and increase lead time for utility relocations.
4. A single source of informational memorandums and regulations will limit conflicting requirements, which will put installation processes and procedures on a more uniform basis.
5. One well-organized utility group can achieve better working conditions, such as flow of communications, coordination, effort, and working relationship, than can a fragmented group working independently.
6. A well-organized utility group can review and process relocations on a more consistent basis, which provides better public relations.
7. More efficient processing review of utility proposals can reduce internal cost by minimizing duplication of effort.
8. More efficient processing of utility proposals will enable faster highway construction work, which will cause fewer traffic conflicts, delays, accidents, and detours.
9. Better coordination of work schedules will result in

fewer pavement cuts on both new and old projects.

The purpose of the questionnaire was to gather data about and identify common traits of the utility-relocation function in various agencies in order to identify the optimum location for this function. All but one state, Hawaii, had a specific unit established for the purpose of utility coordination, and in 19 agencies, one office handled the function for relocations required by both highway projects and new installations.

In most agencies, the right-of-way or the design division is considered the most appropriate location for the function if there is no separate utilities division. Utility relocation and accommodation are closely related to design details and joint uses of the highway right-of-way.

All of the states now have a statewide utility-accommodation policy, and 35 states have prepared a relocation procedure manual. Approximately 50 percent of them have some form of master agreement, and 25 percent use or plan to use the FHWA-approved alternate procedure to provide more lead time and reduce processing time.

The recommendations to make the utility-relocation function a division were outnumbered by the recommendations to not do so. However, the reasons for separating the utility function into a separate division were numerous and convincing. A separate utilities division could incorporate many of the desirable factors listed above.

This study is primarily a state-of-the-art finding and can be used as a basis for other studies, such as that of combining railroad relocations and utility relocations into one section.

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Standard Color Markings for Underground Facilities

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The one-number-to-call system is the latest and most effective tool in the continuing campaign to prevent damage to underground facilities by excavating equipment. Another project that should help to reduce such damage is the use of standard colors for the stakes that are used to mark the locations of underground facilities. In such a system, each type of utility is assigned a color for marking its facilities. (Unless the colors are standardized, utilities that serve more than one state, county, or area would need to stock all colors of markers, which could lead to errors in staking.) Contractors and highway maintenance workers would recognize the uniform colors and be able to identify the type of facility.

More and more utility facilities are being placed below ground every day, and considerable attention has been directed to the danger of damage to them during subsequent excavations. This damage can be caused by earth-moving or excavating equipment and other construction or digging activities, and identifying the responsibility for its prevention is complex. Contractors maintain that if engineers would accurately locate the utility lines on their construction plans, damage could be avoided.

But, before looking for a solution, consider the source of the problem: About 20 utilities, such as water, sanitary sewers, gas, electric power, telephone, telegraph, cable television, street lighting, traffic-signal cables, police-signal cables, fire-signal cables, steam lines, and drainage systems, can be found beneath the streets and highways. Each year, various corporations spend large sums of money to locate and mark their below-ground facilities to help prevent accidental damage. They have had some success, but more information is needed. Where does most of the damage occur? The problem appears to be most serious in areas that are growing rapidly and are highly populated. Naturally, construction activity increases in growing areas and, if they are densely populated, they will have a higher concentration of utilities.

There have been many attempts to identify the general group that is responsible for most of the dig-ins. One study found that private contractors constructing streets and highways, residences, industrial and commercial buildings, and sewer and drainage systems were responsible for 75 to 80 percent of the damage to buried gas systems. Another study blamed 78 percent on other utilities (including their contractors) or landscaping and fencing contractors (1). A recent survey showed that 25 percent of those interviewed considered the underground-damage situation very serious or critical. This is a reflection of the high cost of repairing damages and also of its severe impact on public opinion. The public normally is not aware of the reason why service is interrupted, and most utilities do not consider it a good policy to identify the specific individual or company who caused the situation. However, this policy is beginning to change, especially with regard to chronic offenders who are careless with underground facilities.

The one-number-to-call concept is being implemented in a number of locations. This is a system in which an excavator planning to dig in a given area can, with one telephone call, advise all participating utilities of his plans. The receiver of this information records it and then transmits it to all participating owners of underground facilities in the area, usually by teletype or telephone. The participating owners then mark or stake the locations of their facilities in the field or advise the excavator that they have no facilities in the area. To be effective, all owners of underground facilities in an area covered by such a system must be included in that system.

Another project that should help reduce dig-ins is the use of standard color markings for the stakes that show the locations of underground facilities. In such a system, each type of utility is assigned a color to mark its facilities. One of the basic reasons for a standard color marking is administrative. Unless colors are standardized, utilities that serve more than one state, county, or region are required to stock all colors of markers, which can lead to errors in staking. Standard colors enable contractors working in different states and counties to recognize and identify the type of underground facility. Utilities and contractors today usually operate in more than one area. Some states have already passed legislation specifying particular color codes.

LEGAL REQUIREMENTS

The National Transportation Safety Board has recommended that the American Public Works Association standardize colors that could be used for temporary marking and staking for the identification of underground facilities, and has urged local chapters to support the adoption and use of these standard colors (1).

The U.S. Department of Labor Occupational Safety and Health Administration has said (2) that

prior to opening an excavation, effort shall be made to determine whether underground installations; i.e., sewer, telephone, water, electric and such, will be encountered, and if so, where such underground installations are located. When the excavation approaches the estimated location of such an installation, the exact location shall be determined, and when it is uncovered, proper supports shall be provided for the consisting installation. Utility companies shall be contracted and advised of proposed work prior to the start of actual excavation.

The New York industrial code states the following (3):

53-3.6 Staking, marking, or other designation. (a) Every underground facility in or within 4.6 m (15 ft) of a proposed excavation or demolition work area shall be staked, marked, or otherwise designated by the operator in accordance with the provisions of subpart 53-4 of this rule. Every excavator shall be familiar with such provisions, especially those relating to size and depth indications, color coding, centerline or offset staking or marking, and the location of underground facilities by designations other than staking or marking. (b) Whenever the excavator determines that a review of the staking, marking, or other designation is necessary or that additional information is required before he commences the excavation or demolition work, he shall so notify the operators. 53-3.7 After commencement of excavation, the excavator shall be responsible for protecting and preserving the staking, marking, or other designation until no longer required for proper and safe excavation or demolition work at or near the underground facility. 53-3.8 Where any underground facility has been staked, marked, or otherwise designated by the operator within a proposed work area, the excavator shall verify the exact type, size, direction of run, and depth of such underground facility or its encasement before he commences the proposed excavation work.

53-4.7 Uniform color code required: The following uniform color code shall be utilized on staking and marking used to designate the location of underground facilities:

Color	Utility or Type of Product
Yellow	Gas, oil, petroleum products, steam, compressed air, compressed gases, and all other hazardous liquid or gaseous materials except water
Red	Electric power lines or conduits
Orange	Communication lines or cables, including but not limited to telephone, telegraph, fire signals, cable television, civil defense, data systems, electronic controls, and other instrumentation
Blue	Water
Green	Storm and sanitary sewers including force mains and other nonhazardous materials
Purple	Radioactive materials

In Michigan a public utility served with a notice of excavation is required (4) to "establish the precise location of the underground facilities in advance of construction." The approximate location of the underground facility must be marked with stakes or other physical means that follow a prescribed color code (4).

The following legislation has been enacted in Wisconsin (5).

182.0175(2)(e) Every person owning transmission facilities shall, upon receipt of notice under paragraph (a)3, mark in a reasonable manner the locations of transmission facilities in the field so as to enable the person engaged in excavation or demolition to locate the transmission facilities without endangering the security of such facilities. The marking of facilities shall be accomplished within 3 working days after receipt of the notice, except if notice is given more than 10 days before the excavation or demolition is scheduled to begin; marking need not be accomplished more than 3 working days before excavation or demolition is scheduled to begin. If the approximate location of an underground transmission facility is marked with stakes or other physical means, the public utility shall follow the color coding prescribed herein.

Color	Utility or Type of Product
Safety red	Electric power distribution and transmission
Safety red	Municipal electric systems
High-visibility safety yellow	Natural gas distribution and transmission
High-visibility safety yellow	Oil distribution and transmission
Safety-alert orange	Telephone and telegraph systems
Safety-alert orange	Cable television
Safety-alert orange	Police and fire communications
Safety-precaution blue	Water systems
Safety green	Sewer systems

The following resolution was passed by the St. Petersburg City Council (6).

Whereas, utilities in street rights-of-way within the city are being placed underground in increasing numbers; and, whereas, the St. Petersburg Utilities Coordination Group has recommended that all utilities placed underground can, when necessary, be color-coded above ground to facilitate their identification prior to the commencement of construction work in an area known to contain underground utilities; now, therefore, be it resolved by the city council of the city of St. Petersburg, Florida: That the following color codes are hereby designated and assigned to the utilities listed herein for the purpose of facilitating the identification of these utilities in street rights-of-way when the pipe, conduits, drains, cables, or wires of said utilities are placed underground:

Color	Type of Utility
Orange	Telephone
Green	Cable television
Red	Fire alarm
Black	Traffic signals
Yellow	Power
Gray	Sanitary sewer and storm drainage
Brown	Nonpotable
White	Gas
Blue	Water

Be it further resolved: That upon the request of application to the city for a permit to excavate ground that is likely to contain underground

Table 1. Edison Electric Institute questions and replies.

Question	Reply
When requested, does your company temporarily mark the location of your underground facilities?	Yes (86); no (2)
If yes, does your company use a specific color?	Yes (61); no (27)
What color is used?	Red (31); yellow (15); blue (3); orange (5); yellow for distribution and red for transmission (1); yellow in one state and red in another (1); red or orange stakes (yellow on payment) (1); red flags or orange spray paint (1); blue stake with red top (1); red in one area and green in another (1); red for circuit route and yellow for submersible equipment (1)
Does your company participate in a one-number-to-call system?	Yes (39); no (49)
If yes, has each utility been assigned a color code?	Yes (26); no (13)
What color is used to denote electric facilities?	In use—red (23), blue (2), orange (1), no specific color (9); proposed—red and yellow (1); yellow (1); red (1)
Has the state(s) in which your company operates enacted or is the state(s) in which your company operates in the process of enacting legislation providing for notices to public utilities by persons excavating or discharging explosives near underground facilities?	Yes (51); no (37)
If yes, list name of state.	Enacted—Arizona, Connecticut, Colorado, Georgia, Illinois, Indiana, Kentucky, Maryland, Michigan, New York, Pennsylvania, Rhode Island, Wisconsin; pending—Alabama, Arkansas, Delaware, Florida, Georgia, Illinois, New York, Ohio, Utah, Virginia
Does this legislation include color coding for marking location of underground facilities?	Yes (18); no (32)
If legislation includes color coding, what specific color is used for electrical facilities?	Red (4); blue (1); no reply (31)
Would safety red be acceptable to your company if proposed as an identifying color temporarily marking the location of your electrical underground facilities?	Yes (79); no (9)
If answer is no, please state reason.	Use blue; prefer yellow or orange; other utilities in state use red
If answer is no, please list preferred color.	Orange (3); yellow (5); blue with red top (1)

utilities, all utility companies shall be notified of the need for aboveground color-coded identification of the locations of their underground lines.

The highway-facilities committee of the New Jersey Utilities Association is preparing a uniform color code for designating the location of underground facilities. These markings will mark the location of underground facilities with a centerline location and by type of underground facility and follow the following guidelines.

1. Where centerline stakes or marks indicate the size of the underground facility, the facility shall be assumed to lie within a strip of land equal to the width of the facility plus 1.2 m (4 ft), with the centerline of such strip of land at the stakes or marks.
2. Where centerline stakes or marks do not indicate the size of the underground facility, such facility shall be assumed to lie within the strip of land 1.2 m (4 ft) in width, with the centerline of such strip of land at the stakes or marks.

The following standard color code shall be utilized and prominently displayed on any stake, marking, or other designation for an underground facility.

Color	Utility or Type of Product
Yellow	Gas, oil, petroleum products, steam, compressed air, compressed gases, and all other hazardous liquid or gaseous materials except water
Red	Electrical
Orange	Communications: telephone, telegraph, traffic signals, fire signals, cable television, civil defense, data systems, electronic controls, and other instrumentation or controls
White	Water
Green	Storm and sanitary sewers and all other nonhazardous materials

QUESTIONNAIRE

The Edison Electric Institute Transmission and Distribution Committee has surveyed 88 utility companies about their policies on the use of an identifying color for temporary markings to denote the locations of underground electrical facilities. The questions asked and the replies received are summarized in Table 1.

The uniform color code that is most commonly used in staking and marking the location of underground facilities is summarized below.

Color	Utility or Type of Product
Safety red	Electric power distribution and transmission
Safety red	Municipal electric systems
High-visibility safety yellow	Gas distribution and transmission
High-visibility safety yellow	Oil distribution and transmission
High-visibility safety yellow	Dangerous materials, product lines, and steam lines
Safety-alert orange	Telephone and telegraph systems
Safety-alert orange	Police and fire communications
Safety-alert orange	Cable television
Safety-precaution blue	Water systems
Safety-precaution blue	Slurry pipelines
Safety green	Sewer systems

The ideal way to make a standard color-code program effective would be through state legislation that makes it mandatory. By having standard color markings, excavators would know not only whose facilities were where, but also which utilities had not completed their location and marking work. For a marking system to be effective, it must

1. Follow a uniform system and
2. Use standard methods so that marking devices, such as color and stakes, will be readily recognizable and so that any stakes used will be different from survey stakes or other marking devices.

This color code is suggested as a model or guide toward uniformity and could be used by all utilities, agencies, contractors, and excavators.

CONCLUSIONS

Many utilities are interested in a uniform color code for marking the locations of underground facilities because this would simplify their administrative and logistic problems. From the point of view of highway departments, a color code would make the underground plant within the confines of the highway right-of-way easily identifiable by maintenance crews.

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Computerized Mapping and Record Systems for Utilities

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In the telephone company, the outside-plant-location record is the permanent inventory of all outside plant in service. It serves as a primary document and tool for engineers designing additions, deletions, and changes in the plant. The use of this record is described briefly. The drafting effort required to maintain this large data base is discussed, and the difficulties in maintaining it by manual methods are emphasized. Explorations into applications of computer-graphics technology are reviewed. The implementation of a pilot, interactive-graphics minicomputer system that completely automates all drafting of outside-plant-location records and engineering work orders is described. This system is an on-line system that replaces paper records by electronically stored records that can be accessed and retrieved by a computer in real time at interactive-graphics terminals. It is used to demonstrate how future drafting and record drawing can be performed on displays at cathode-ray-tube terminals. It is cited as an example of a specialized computer-assisted, map-based record system. The advent of this new technology is a result of recent developments in computer hardware, computer systems, and digital graphic systems and has led to many new proposals for joint-use systems that have a common, fundamental map base. These are described in terms of their impact on land-ownership records. The efforts of the American Public Works Association to develop standards for this field are reviewed and the aims and purposes are described.

In the telephone company, the outside-plant-location record is the permanent inventory of all outside plant in service. It serves as the primary document and tool for engineers making additions, deletions, and changes in the plant.

Managing this program requires about 10 000 engineering and 10 000 equivalent-engineering, clerical people and an expenditure of about $100 million/year for drafting and allied areas. The record keeping associated with this is a complicated job that has not changed basically in 40 years.

A result of this highly labor-intensive activity is that, even with good management, there are cycles and phases when the records are not current. The error rates rise where it is necessary to keep duplicate records. The master copies deteriorate with time whether they are on paper or mylar, and it is difficult to enforce uniformity of record-keeping methods. Furthermore, when records are kept on paper, they are not continuous, and there is a major problem in ensuring that all appropriate sheets are updated when changes are made. Because the engineering work is designed on the basis of these records and there are times when the actual field conditions are different, delays in construction or the restoration of the physical plant can occur, and an inefficient use of personnel and equipment can result. As a result, some engineers maintain private copies of the record at their own level of requirements.

The number of people and the expense that goes into this work have been increasing in recent years. Figure 1 outlines the work flow for the keeping of records of construction work orders.

In the manually kept system, there are at least three redrawings of the appropriate symbology. Each retranscription increases the probability of error, and there is a lengthening lead time when the work load is heavy.

COMPUTER-ASSISTED DRAFTING

A new technology called interactive graphics has emerged in recent years. This uses a combination of devices that are linked together and operated by a computer program. A typical installation has a computer including its software, various peripheral devices, and the interactive-graphics terminals. The computer electronically updates and processes data. It must be fed software, that is, a set of encoded programs of instructions to control its operations. A variety of input and output devices and peripherals can be coupled to the computer: A disk unit can serve as a means of on-line magnetic storage of data, a plotter can transform this data into paper drawings (magnetic tapes can also serve as storage devices for the data and are more readily transportable), and a digitizer can convert information on a paper map or record into an electronically stored record in usable computer language. The graphics terminals are devices to access and interact with the electronically stored record (the data base). These cathode-ray-tube (CRT) terminals can have various characteristics in these systems. They can be local or remote. They can provide only the capability to view the data base, or they can be interactive, i.e., change the data base. They can operate with or without digitizers. The terminal has rapid access to the data base and can provide a visual display of any portion of this data on the CRT. It is also the device through which inquiry and interaction can take place by means of a keyboard and such equipment as a data tablet, a light pen, a thumb wheel, or a stylus. Figure 2 shows a large terminal and its various components.

Compare one of these interactive-graphics systems (Figure 3) with the outside-plant, engineering-record system shown in Figure 1. Once an item is drawn at the interactive terminal, it is stored in the computer either as a part of the data base or as a potential part of the data base that is part of a work order. All kinds of information, both textual and graphic, that a draftsman supplies can be entered from the interactive terminal. At any time, a paper copy of the display on the CRT can be obtained. A pilot installation of this kind of system has been installed at one telephone company. It is a modest experimental effort that will cover approximately 78 km^2 (30 $miles^2$) of fully developed urban area that is served by five telephone wire centers. After a full evaluation, this pilot installation of a larger outside-plant, engineering drafting system will be developed.

This mode of operation has several advantages over the manual mode. The master records on paper and film are eliminated. There is only one copy of the record, and it is in the magnetically stored data base. This method promises reductions in clerical personnel by the elimination of multiple work operations. Individual items need be drawn only once, which means that the drafting of construction work orders and the preposting and final posting of these plant changes on the record data base are accomplished quickly.

Since there are fewer retranscriptions, errors should be reduced. There are obvious economic benefits. Drafting with these systems is estimated to be twice as fast as manual drafting. But, there are more important benefits, such as improved readiness to serve. If records are more accurate and up-to-date, less time is required to search them. Faster repair of failures of large cable is one example of how an accurate up-to-date record can improve service.

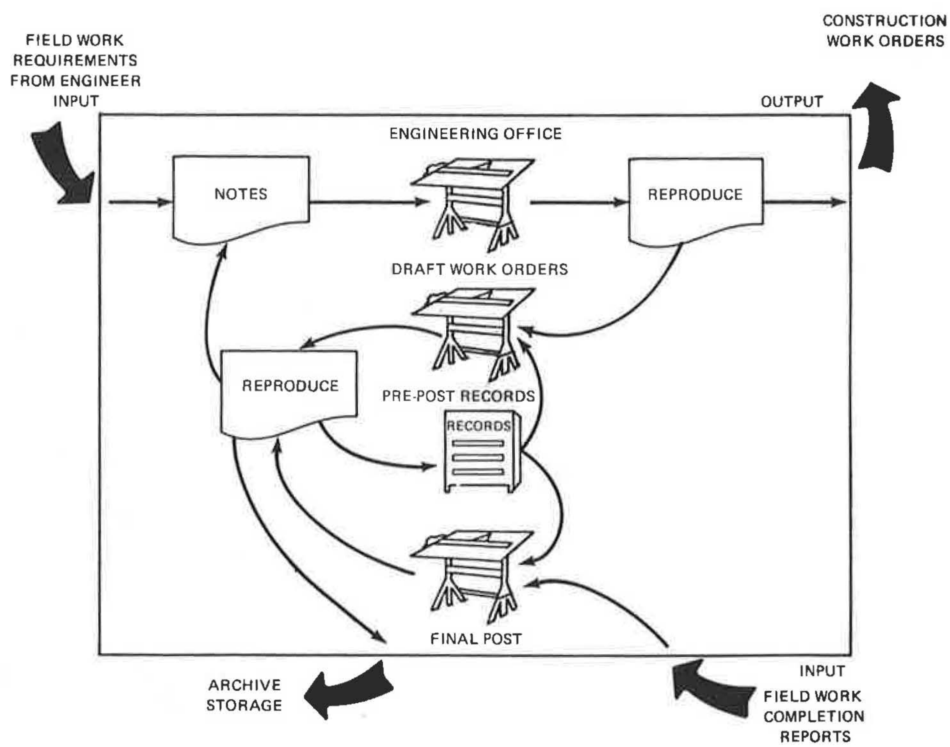
The telephone system is not unique in recognizing the need for improvement in present record-keeping procedures and the impact that this new technology of computer-aided digital graphics can have on operations. It is only typical of many utilities, as this next wave of future shock hits.

In attempts to apply this technology, the major start-up cost is that of the conversion of the existing records to the digitized electronic data base. For example, the records of a typical urban telephone wire center are estimated to require about 6 million bytes when stored in a digitized graphics format. This conversion cost cannot be absorbed in a short period of time by any regulated industry, whether it be telephone, gas, electricity, or a sewer network.

LAND USE RECORDS

Let us consider another application of digital computer graphics that has been developed in recent years. The most widespread application of computer-assisted graphics to geographically based record systems is in the area of land-ownership records. These are map-based records of land parcels, and the related records of ownership, tenure, assessment value, assessment parameters, tax rates, tax-billing data, and such. There are a variety of these systems, most of them quite new, but all of them designed for the primary purpose of supporting tax and ownership records (2). They are all map-based. That is, the data are treated as various overlays to a fundamental map record that is usually restricted to street and property outlines, although occasionally the system incorporates vertical elevation data in the form

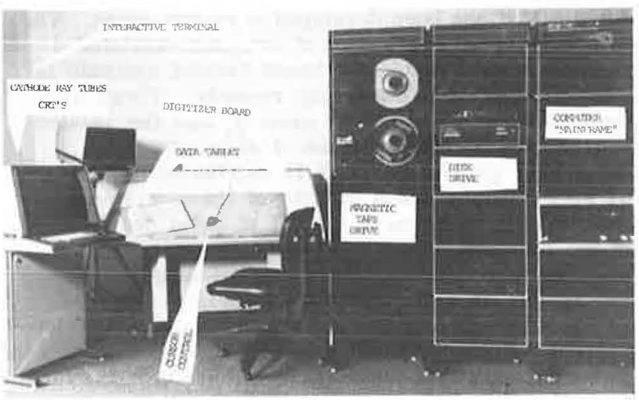
Figure 1. Paper flow without computer.



of contour overlays, as shown in Figure 4. These systems require the same types of equipment and software and provide similar benefits to their users as do the utility-record systems. They also suffer from the same economic problem: The bulk of the cost is that of the initial effort to digitize the map and its information overlays. Thereafter, interactive maintenance is economical because of the reduced drafting cost. However, there is one particular benefit that has pressed developments for land use applications. Consider the experience of the Canadian National Capital Commission. In one small semirural community, the tax rolls were increased almost 20 percent through the effort to overlay against a continuous map, when unassessed properties were discovered and added. This benefit accrues in almost any comprehensive review of a manual record. One of the greatest supervisory difficulties in managing a manual record-keeping system is that of ensuring that record changes are continued from sheet to sheet in the paper operation. This difficulty does not occur in a map-based computer-stored record because the viewer can scan the data base in much the same way that a movie camera can pan a scene.

All of the systems that have been installed have faced this burden of conversion, and several have proposed a community effort. The same map base can be as valuable for utility location, if utility records are overlaid in the data base, as it is for ownership and tax-record location. Once a community has been mapped on a computer, there are many other data about that community that are more readily analyzed or grouped or processed. Examples are demographic data, fire and police-protection coverage, and solid-waste collection. There have been attempts in the past to develop a community of interest and divide the cost of initiating a fundamental map. This is sometimes the point at which a utility becomes interested. At other times, field people have seen opportunities and voluntarily participated in joint computer-assisted mapping projects. None of these have realized the anticipated benefits because none of them have fully addressed optimum user needs in terms of cost. To achieve the savings inherently possible in community projects, there should be some reference standards to allow better value judgments of the anticipated benefits.

Figure 2. Basic interactive-graphics design system.



Note: M and S Computing, Inc., Huntsville, Alabama.

STANDARDIZATION

For many years, utility companies have worked together to solve a common problem, that of the accidental damage to underground facilities by digging operations. The most effective remedy is found in the one-number-to-call systems. Within a region, anyone preparing to dig can call one telephone number for information and assistance in locating underground plant. Over 80 of these systems have been established across the country, many of them under the aegis of the Utility Location and Coordination Council (ULCC) of the American Public Works Association (APWA). These systems require a geographically referenced record. Map-based records are used, and the operation is most efficient if common-base maps are used for all records.

In 1975, ULCC instituted a task force to investigate the impact of computer-assisted mapping systems and the possible benefits of one-number-to-call operations.

Figure 3. Simplified paper flow with computer.

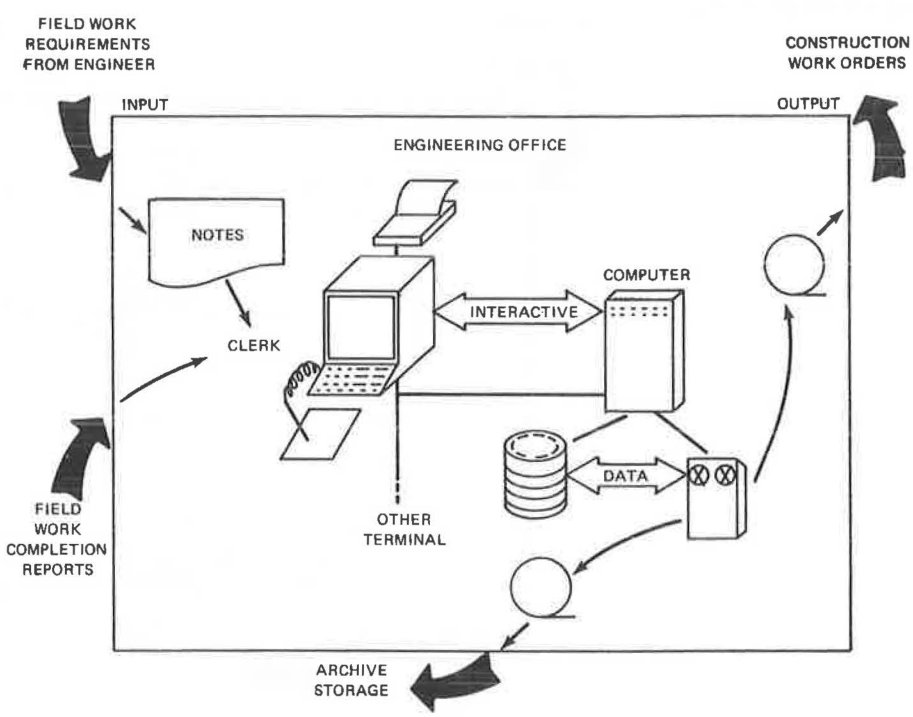
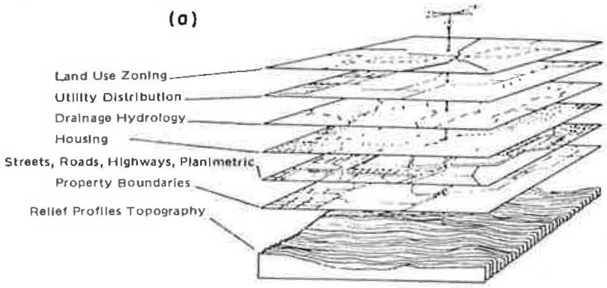
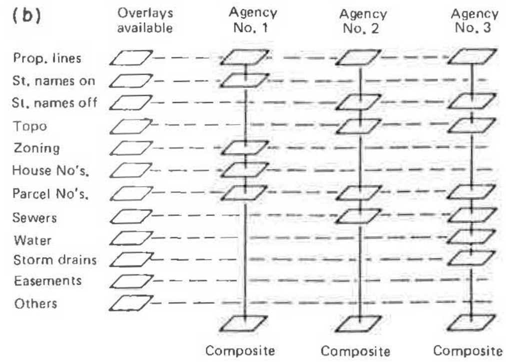


Figure 4. Possible data overlays and search strategies.



The Data Base information is collected and stored in such a way that it may be accessed by subject.



Maps and reports may be computer generated to contain only the information required by the user. Each user has different needs.

Note: Hoskins-Western-Sonderegger, Lincoln.

The report of this task force contained a single recommendation (3). This was that, under the auspices of APWA, there should be a single test-bed installation to establish a joint-use, map-based, computer-assisted system. The project was named Computer-Assisted Mapping and Records-Activities Systems (CAMRAS). The reason given for this single finding was the results of the investigation of existing and proposed systems, which had found that individual choices were made on a one-shot basis and that it was impossible to go back and reconstruct the significant factors leading to choices and decisions. Where local political factors or interdepartmental procedures had been overriding, these could not be subtracted from the conclusions reached. Thus, it was not possible from the experience of present users to construct a basis for technical and procedural standards that would assist future users and planners to evaluate the performance of their own systems and allow some common basis for automatic exchange of data between special-purpose systems and joint-user systems. These are factors that have been generally overlooked in the press of the industry to make sales and of users to improve their record-keeping functions and exploit this new technology. The task-force findings were not critical, but it was felt that there should be some institutional leadership of this new development so that the different systems will have some probability of compatibility. It is also desirable to avoid the wasteful duplication of effort that occurs when individual agencies each evaluate the maze of claims, counterclaims, and experiences of many different existing systems. The existing systems are, in almost all cases, mutually incompatible in the possibility of exchanging or substituting data from one to the other.

Why was government or industry not suggested as the leader here? There are several answers. The existing systems and technology appear to be based on minicomputer capacities that center around local communities or portions of urban centers. There is no national influence by these communities except through an institutional association. On the other hand, the federal government programs are on a national scale and do not provide the fine-grain detail that community systems require. Individual private industry likewise cannot introduce standards except internally, and vendors of all kinds—hardware, software, and services—are making proprietary developments from conclusions based on the experiences of one or two customers at a time. They are, therefore, not in a position to take a broad view to optimize a market. These facts support the conclusion that the leadership should be an institutional responsibility. The potential areas in which standards and procurement specifications or both should evolve from the experience of the test bed are listed below.

1. Operational system standards: (a) data base definition (smallest physical length graphically distinguishable in stored data) and (b) data base content—fundamental records and map content (basic building modules and layers); features and fixtures (e.g., reference grid and its coordinates); facilities, i.e., telephone-utility plant within rights-of-way (visible, subsurface, and vertical location); data base structure [overlay assignments and overlay names (codes), viewing composites and auxiliary views (projections), and capacity]; inputs (data base maintenance) and outputs {hard media, i.e., quality [size, scales, registration error, status (prefile, file, under construction, as-built, and such), notation, and timing rules, overlay content, edge overlap, and reference indexing] and magnetic media, i.e., interface specification (headers; record content, format, and field description; and code)}; hardware performance (computer main frame, peripherals and adjunct memory, interactive terminals, and operator's documentation); software performance [response time (real time and hard copy), operational procedures (e.g., batch versus real time), reliability (recovery-restoration-protection), machine-independent, data-security provisions, overlap provisions, macro commands, high-level source language, and access to source code]; personnel subsystem (skill qualifications and training documentation); and procurement.

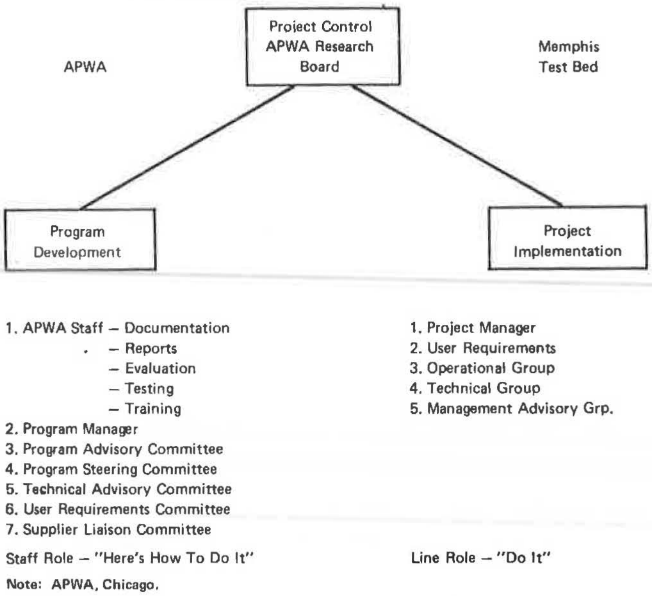
2. Conversion (manual to automated) system standards (i.e., initial digitization): (a) outputs final—all data base specifications above apply; (b) facility map(s)—source qualifications and encoding (record reliability); and (c) procurement—translation of operational system standards for impact.

3. Photogrammetry standards: (a) map base—composition and size, definition, and distortion tolerances and (b) procurement—translation of operational systems and conversion standards for impact.

4. Aerial photography: (a) synchronization of ground-reference survey including monuments, facility targeting, and flyover and (b) procurement—translation of operational system, conversion, and photogrammetry standards for altitude, area, and picture quality.

The test-bed operation will be managed as an individual community operation with an advisory staff of participating members of APWA and the support, for standards development, of the APWA staff (Figure 5). This effort should lead to standards that will assist

Figure 5. CAMRAS organization.



utilities and public-works agencies in their participation in one-number-to-call utility-location systems, to procurement guidelines for future users, and to guidelines for the establishment of a joint-user system.

Therefore, to the advantages of computerized mapping, we now add the benefits of standardization. With standardization, we add the capability for computer-to-computer exchange of data, which implies compatibility with private records systems. Vendor performance is clarified because the entire procurement process is simplified. Procurement documents that have clear performance standards will focus vendors' developmental activities, and a better evaluation can be used to justify vendor selection. Perhaps in summary of all of the above: For all future users, the experimental risk is reduced.

Governments at all levels profit from the existence of widely accepted standards. One small example is that the fine-grain, ground-control networks of local municipalities can be more readily referenced to a national network through computer-controlled conversion systems. Those who are interested in land records, including surveyors, particularly benefit by standards for reference and recording. Conveyors, agents, and legal representatives are more assured by standard descriptive systems. Insurors are more certain of the permanent existence of parcel descriptions and parcel-adjacency references. Therefore, the courts benefit because there is a reference method in the computerized standard-recording procedure that can be compared to the methods of the case in hand. Buyers and sellers of land are more readily assured of the conveyance records. Permit agencies and recorders of rights-of-way, contractors, and designers and engineers are more assured that their records are mutually compatible with those of others whom they may affect (and who may affect them). Finally, utilities are more assured that the locations of their systems are reliably referenced. However, with the daily installation of new systems, the ability of those already involved to adjust to a reference standard is constantly being reduced. With each new system that is installed, it becomes more difficult to promulgate a generally accepted benchmark standard. AWPA is unique in that the needs for and benefits from these standards cross the full spectrum of its membership.

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Eliminating Vehicle Rollovers on Turned-Down Guardrail Terminals

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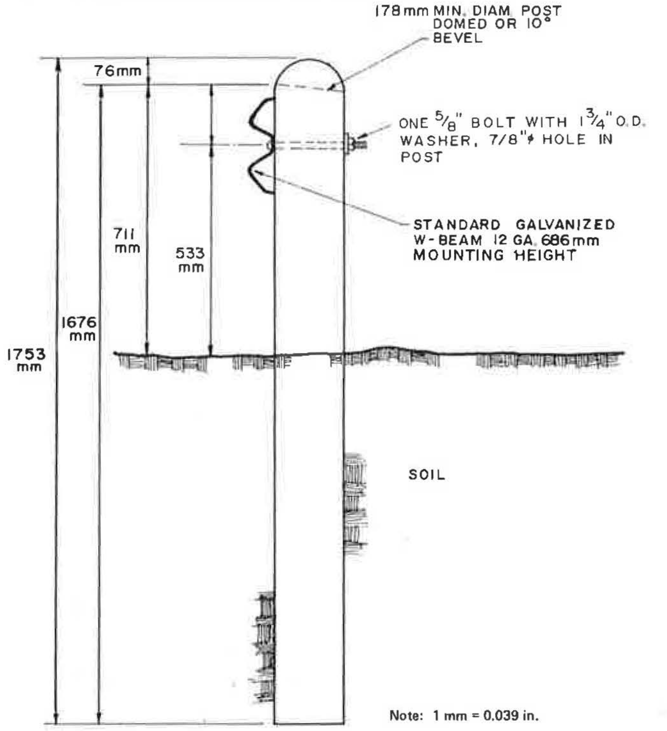
A relatively simple method has been found to modify the turned-down ends of highway guradrails to eliminate or minimize the probability that a vehicle impacting them will ramp and roll over. To modify the standard guardrail, the 5/8-in diameter bolts are removed from the first five posts. With these bolts removed, the rail will drop to the ground if the turned-down terminal section is struck by a vehicle, which eliminates ramping of the vehicle. To hold the rail at its proper height [69 cm (27 in) in Texas] before and during a vehicle impact along the length of need, backup plates are bolted to the first five posts. The action of this modified guardrail terminal is simple. When a vehicle tire or bumper pushes down on the turned-down terminal, the rail drops from the first five posts, which allows the vehicle to pass over the rail. If the vehicle bumper impacts the rail on the length of need and pushes it laterally against the backup plates on the posts, the rail is held at its proper height and the vehicle is redirected. The test program included the four crash tests for longitudinal barrier terminals. All of the tests were successful, and no vehicles rolled over.

The steel flex-beam W-beam guardrail is used extensively on highways. In the late 1950s and early 1960s, the dangers of guardrail ends became apparent after

Figure 1. Standard bolted guardrail-to-post connection.



spectacular accidents in which guardrail ends pierced and ran through vehicles. The remedy for this has been to turn down and bury the ends of the guardrail. This simple treatment eliminates the vehicle-piercing and impalement accident and, at the same time, anchors the guardrail so that it has the tensile strength necessary for effective vehicle redirection.

However, in the late 1960s, the California Division of Highways and the Southwest Research Institute conducted several crash tests (1, 2) on turned-down guardrail terminals and found that these ends can launch an impacting vehicle and cause it to roll over. Because of these crash tests, safer end treatments have been sought, and several alternatives have been developed (3), but even these have had certain deficiencies.

Because Texas has thousands of turned-down guardrail terminals, engineers at the Texas Transportation Institute and in the Texas State Department of Highway and Public Transportation have been seeking a relatively simple method to modify these terminals to eliminate or minimize the probability that a vehicle impacting on them will ramp and roll over. A relatively simple solution has been found.

MODIFIED TURNED-DOWN TERMINAL

The standard guardrail in Texas is made of 10 or 12-gauge steel flex beam and mounted 69 cm (27 in) high. It is fastened with $^5/_8$-in diameter steel bolts to either wood or steel posts, and blockouts for the rail are optional. In some of the older installations, there is an intermediate post at the midspan of the 7.6-m (25-ft) turned-down section and, in many installations, two 3.8-m (12.5-ft) post spacings are used at the beginning of the length of need.

The design chosen for modification and evaluation was a non-blocked-out guardrail mounted on 18-cm (7-in) diameter wood posts. This design, which is the most commonly used in existing installations, offers the greatest potential for cost-effective improvements. The standard bolted connection used in this design is shown in Figure 1.

The modifications of this design were designed to prevent the launching and rolling over of a vehicle that can result from its impact with the turned-down section. A number of modifications were proposed and analyzed. The design chosen for full-scale testing and evaluation is essentially that shown in Figure 2. The guardrail-to-post connection for the first five posts was modified as shown in Figure 3. A standard W-section backup plate 0.3 m (1 ft) long is fastened to the post with a standard $^5/_8$-in dimaeter bolt, but the continuous rail element is not connected by this bolt. The rail element nests in the backup plate and is lightly held in place by a clip made of 0.32 by 1.9-cm ($^1/_8$ by $^3/_4$-in), mild-steel strap 20 cm (8 in) long. This weak connection allows the rail to be depressed downward under a small vertical load.

With this construction, the rail will drop to the ground if the turned-down terminal is struck by a vehicle. This action eliminates the undesirable situation of the vehicle ramping and rolling over. The backup plates hold the rail at the proper height [69 cm (27 in) in Texas] before and during vehicle impacts along the length of need. These plates are 30 cm (12 in) long for posts 1 through 4 and 15 cm (6 in) long at post 5 where the first standard lap splice occurs. At post 1, the standard lap splice is modified by reversing the splice bolts and placing the nuts on the outside of the rail.

The action of this modified guardrail terminal is excitingly simple. When a vehicle tire or bumper pushes down on the turned-down terminal, the rail quickly drops from the first 5 posts, which allows the vehicle to pass over it without the violent ramping effect of a rigidly turned-down end. If the vehicle bumper impacts the rail at the length of need (or any other high point) and pushes it laterally against the backup plates on the posts, the rail is held at the proper height and the vehicle is redirected. The backup plate resists the downward force component of the turned-down terminal.

CRASH TEST RESULTS

Five full-scale, vehicle crash tests of the modified turned-down guardrail terminals were made between July 30 and August 24, 1976. The test conditions are summarized below (1 kg = 2.2 lb).

Test	Vehicle Mass (kg)	Impact Point
1	1024	Midpoint of turned-down terminal section
2	2068	Beginning of turned-down terminal section
3	2068	On length of need
4	1021	End of turned-down terminal section
5	2068	End of turned-down terminal section

The data taken from high-speed film are given in Table 1, and selected frames from the film are shown in Figures 4 through 11. The data taken from accelerometer measurements made with a 100-Hz, low-pass, maximum flat filter and the vehicle-damage classifications are given in Table 2.

Test 1

The guardrail installation evaluated in this test was a variation of the final design described above. In this installation, posts 2 and 4 (Figure 2) were omitted, and 15-cm (6-in) long backup plates were used on posts 1, 3, 5, 6, and 7. The remainder of the rail was installed as shown in Figure 1. This installation is shown in Figure 12.

Figure 2. Modified standard guardrail with turned-down terminal: bolts removed from posts 1 through 5.

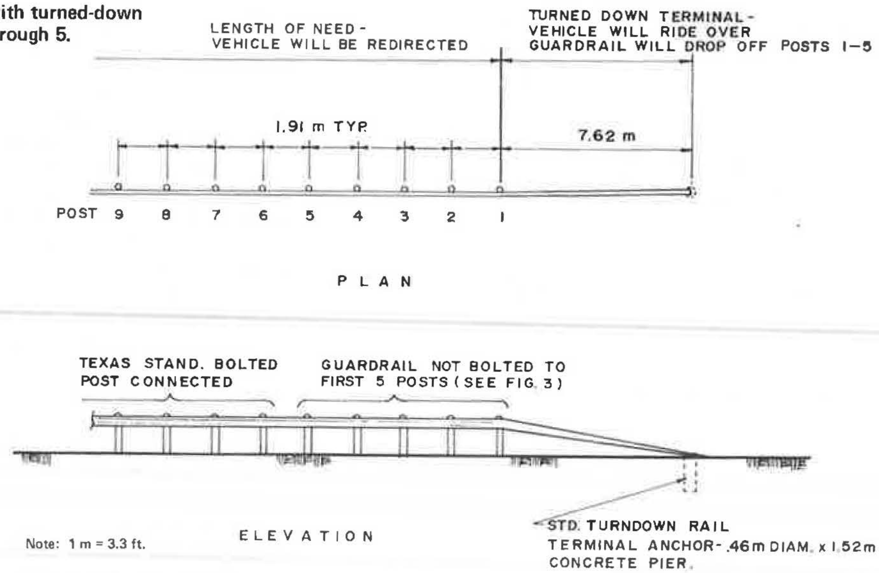
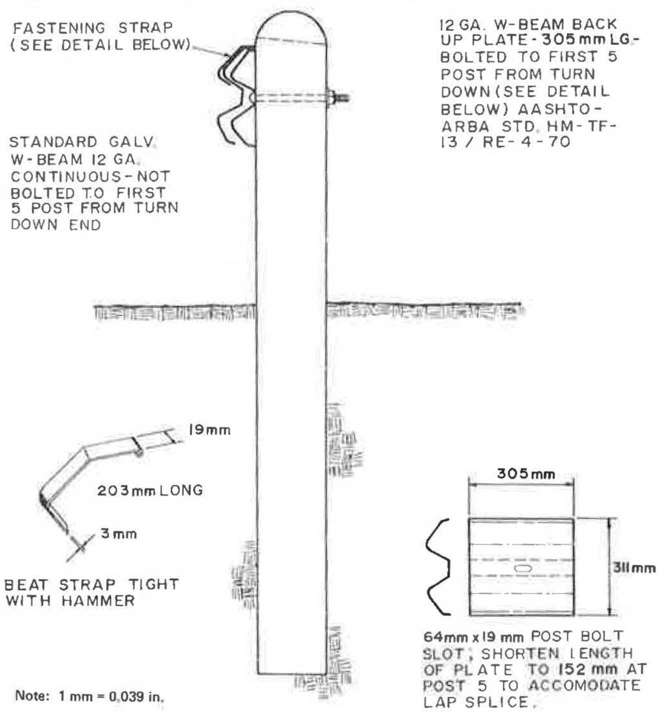


Figure 3. Modified guardrail-to-post connection.



In this test, a 1034-kg (2280-lb) 1971-model automobile impacted the turned-down terminal section of the guardrail at an angle of 17.5° and a speed of 101.7 km/h (63.2 mph). The point of impact was midway between the end anchor and the beginning of the length of need. On impact, the right front wheel of the test vehicle mounted the turned-down section. As the vehicle continued forward, the W-section disengaged from the backup plates and was pushed down. The vehicle rode over the rail, impacted the first post (breaking it near ground level), and continued upright on its path for about 100 m (330 ft) behind the guardrail. After crossing the guardrail, the vehicle was airborne for a short distance and then exhibited oscillatory roll motion to a maximum displacement of about 29°. The vehicle did not roll over, and the performances of the turned-down terminal section and the vehicle were considered good. The critical roll angle of such an automobile is about 53.4°. The damage to the vehicle and the guardrail is shown in Figures 13 and 14 respectively. One post and two 7.6-m (25-ft) pieces of W-section of the guardrail had to be replaced.

Test 2

The guardrail installation evaluated in this test was identical to that used in test 1, except that 0.32 by 1.9-cm ($^1/_8$ by $^3/_4$-in), mild-steel straps 20 cm (8 in) long were added at the guardrail-to-post connections having backup plates. This installation is shown in Figure 15.

In this test, a 2068-kg (4560-lb) 1970-model automobile impacted the guardrail at an angle of 27.5° and a speed of 88.8 km/h (55.2 mph) at a point 30 cm (1 ft) upstream of the beginning of the length of need. The behavior of the guardrail was similar to that in test 1 in that the rail was depressed and the vehicle rode over it. There was some partial redirection (yaw displacement) of the vehicle during its interaction with the rail. The vehicle was partially airborne after leaving the rail and exhibited oscillatory roll motion to a maximum displacement of approximately 45°. The critical roll angle for such a heavy automobile is about 60°. The vehicle did not roll over, and the performance of the turned-down terminal section was considered acceptable because the actual point of impact was 30 cm (1 ft) upstream of the beginning of the length of need. At this location of the impact point, redirection is not a necessary requirement. The damage to the vehicle and guardrail is shown in Figures 16 and 17 respectively. The first post of the guardrail was displaced laterally, and the second post was displaced and fractured. It was necessary to replace both posts and two pieces of W-section.

The original objective of this test had been to impact the guardrail along the length of need and obtain a redirection of the vehicle. The vehicle, however, pushed down the rail and rode over it without rolling over. There are two apparent reasons for this: (a) The right front bumper of the vehicle actually impacted the rail 30 cm (1 ft) upstream of post 1 on the terminal section and not on the length of need and (b) the rail was only 61 cm (24 in) high at post 1 because of the repairs after test 1 (Figure 14) and, consequently, the bumper of the

Table 1. Results of film data of crash tests.

Test	Speed (km/h)			Angle From Rail Line (°)		Time (s)		Barrier Displacement (m)			Distance to Parallel (m)		Avg Deceleration, Displacement (*g*)		
	Initial	Parallel	Final (departure)	Impact	Departure	To Parallel	Of Contact	Dynamic	Residual	Stopping Distance	Longitudinal	Lateral	Longitudinal	Lateral	Total
1	101.7	–	–	17.5	–	–	0.383	KGD[a]	–	–	–	–	–	–	–
2	88.8	–	–	27.5	–	–	0.447	KGD[a]	–	–	–	–	–	–	–
3	94.4	59.0	58.1	25	17.5	0.298	0.685	0.76	0.70	–	6.46	1.86	2.3	2.4	4.1
4	47.9	–	–	3.5	–	–	0.893	KGD[a]	–	–	–	–	–	–	–
5	89.0	–	–	5.5	–	–	4.450	KGD[a]	–	57.3	–	–	0.54	–	–

Note: 1 km/h = 0.6 mph; 1 m = 3.3 ft.
[a]KGD = knocked guardrail down.

Figure 4. Sequential photographs of test 1 (side view).

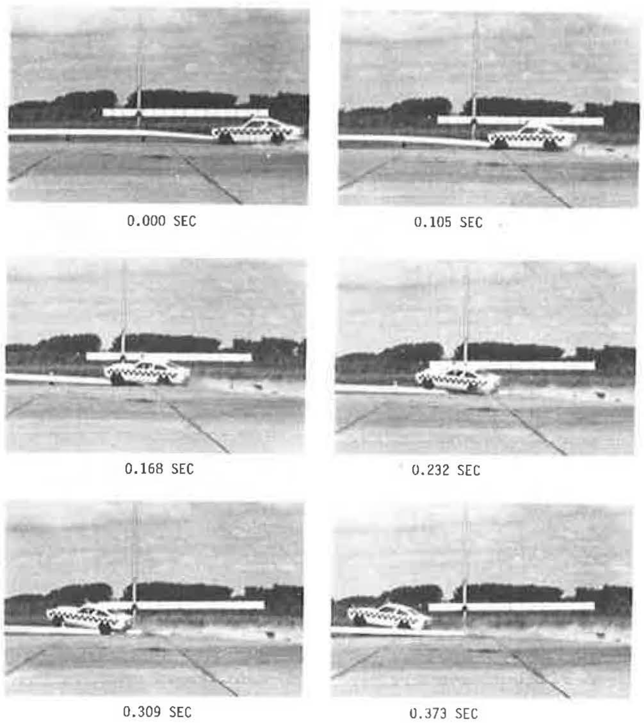
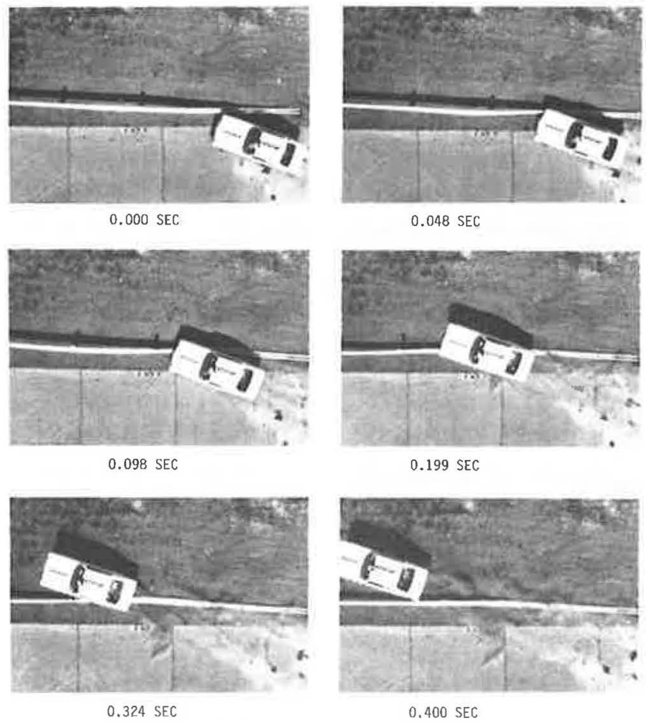


Figure 5. Sequential photographs of test 1 (overhead view).



automobile was above the terminal and pushed it down. Several modifications were made in the installation and in the conduct of test 3 to eliminate these problems. Test 2 was still considered a success in that the vehicle struck the terminal section, pushed it down, and rode over it without rolling over.

Test 3

As a result of the behavior of the guardrail and the vehicle in test 2, several changes were made in the guardrail design and in the test procedure.

1. The point of impact of the vehicle was moved 30 cm (1 ft) downstream from post 1 into the length of need.
2. In the repair of the guardrail and terminal section, care was taken to ensure that the rail was 69 cm (27 in) high at post 1. During installation, the end piece of rail was bolted to post 1 and pretwisted through an angle of slightly more than 180° to put a permanent 90° twist in it. This gave a neater fit and closer dimensional tolerance after the bolt was removed from post 1 and the backup plates were installed. The installation at post 1 is shown in Figure 18.
3. The length of the backup plates was increased from 15 cm (6 in) to 30 cm (12 in) and posts 2 and 4 were added to make the guardrail post spacing uniformly 1.9 m (6.25 ft). This strengthened and stabilized the guardrail and increased vehicle redirection when the rail was impacted on the length of need.

These slight modifications should not affect the results of test 1 in which the vehicle engaged the terminal section and pushed it down and the guardrail rotated away from the posts and backup plates.

The installation shown in Figure 19 was impacted by a 2037-kg (4490-lb) automobile at an angle of 25° and a speed of 94.4 km/h (58.7 mph). The point of impact was 30 cm (1 ft) downstream of the beginning of the length of need. The guardrail contained and redirected the vehicle without adverse pocketing and snagging, and therefore its performance was good. The vehicle left the rail at an angle of 17.5° and a speed of 58.1 km/h (36 mph). Damage to the front wheel caused the vehicle to follow a curved path and return to the guardrail with another impact at a point approximately 61 m (200 ft) downstream. During the redirection, there was some interaction between the front wheel of the vehicle and the guardrail posts, but there was no snagging effect. The damage to the vehicle and guardrail is shown in Figures 20 and 21 respectively. The rail remained nested in the backup plates and at the intended height. Post 3 was broken off at ground level and post 2 and 4 were bent back. The repairs to the guardrail consisted of replacing one post and one 7.6-m (25-ft) section of flex beam. This test was considered very successful.

Figure 6. Sequential photographs of test 2 (side view).

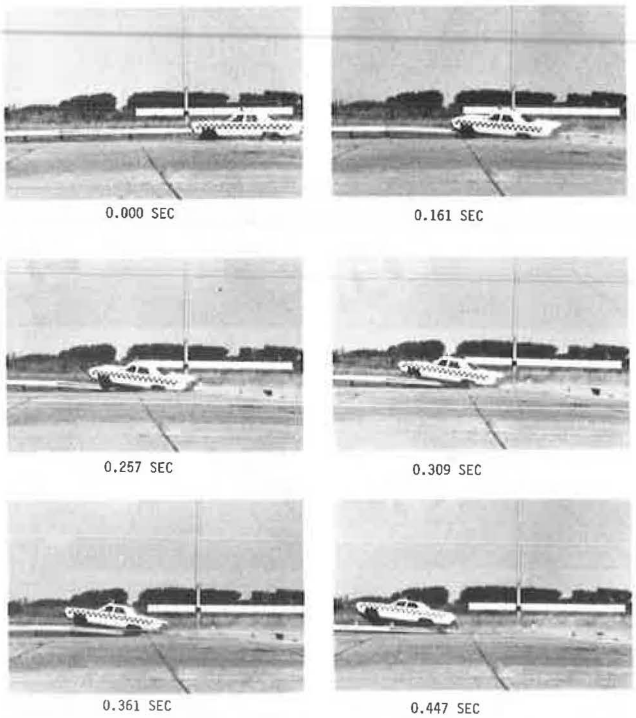
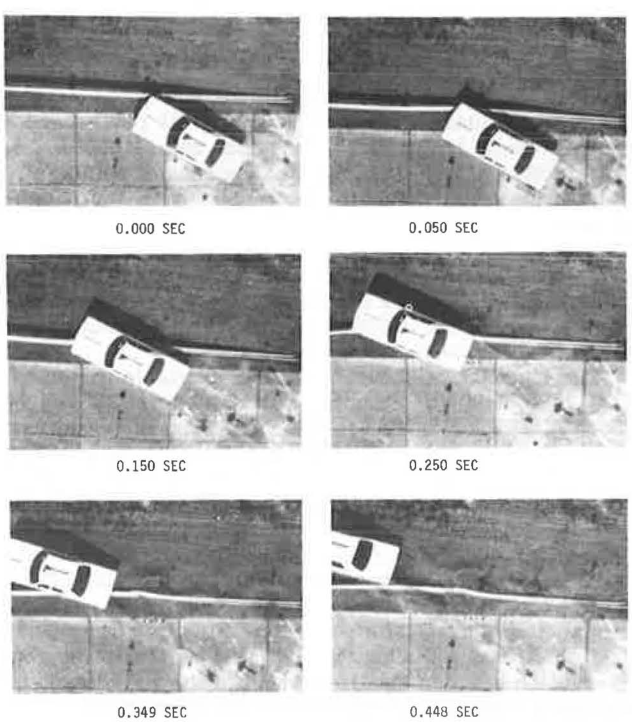


Figure 7. Sequential photographs of test 2 (overhead view).



Test 4

The guardrail installation for this test was identical to that used in test 3.

Test 4 was essentially a head-on test of the terminal section and a small vehicle. The 1021-kg (2250-lb) 1971-model automobile impacted the terminal section at a negative angle of 3.5° and a speed of 47.5 km/h (29.5

Figure 8. Sequential photographs of test 3 (overhead view).

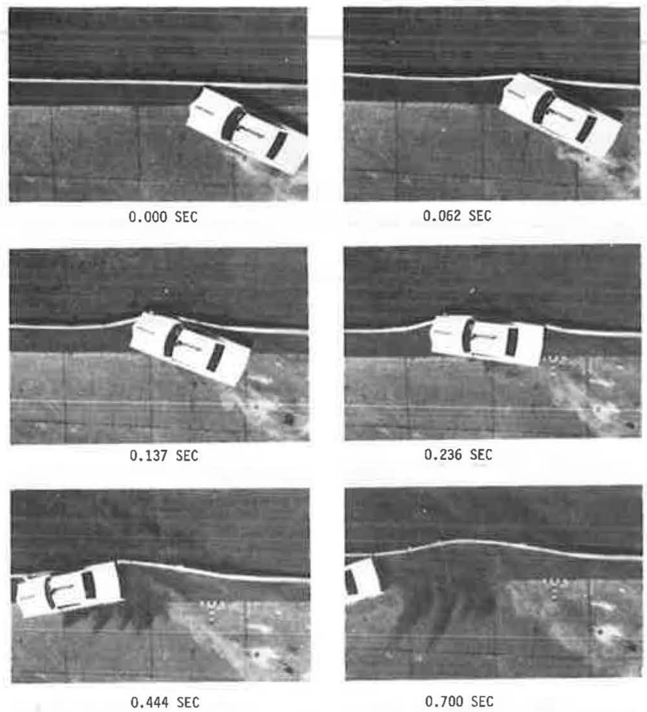


Figure 9. Sequential photographs of test 4 (side view).

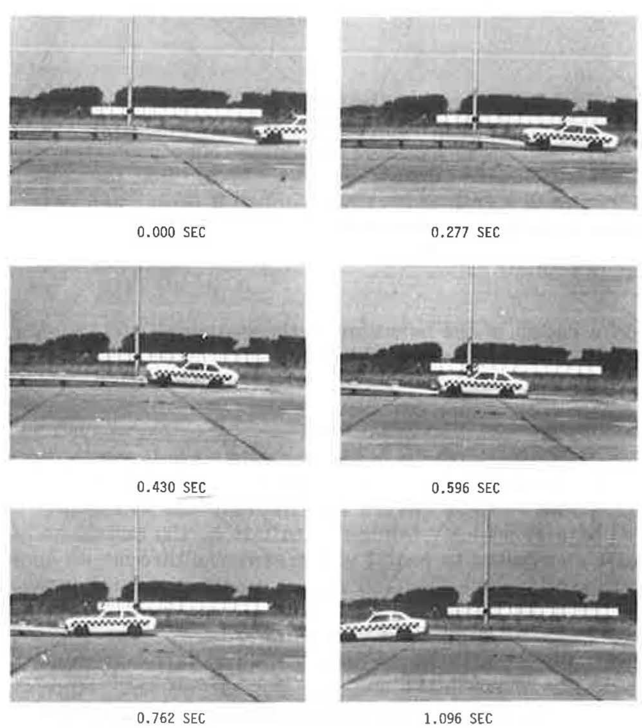


Figure 10. Sequential photographs of test 4 (overhead view).

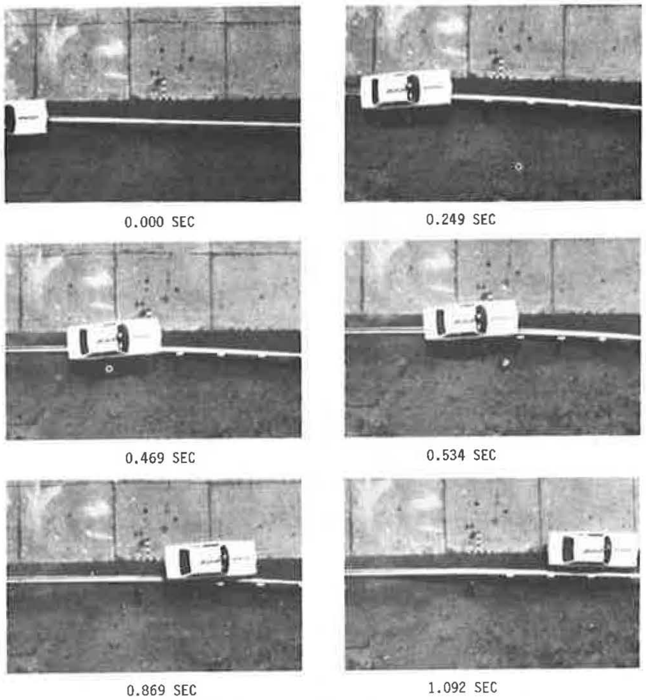


Figure 11. Sequential photographs of test 5 (side view).

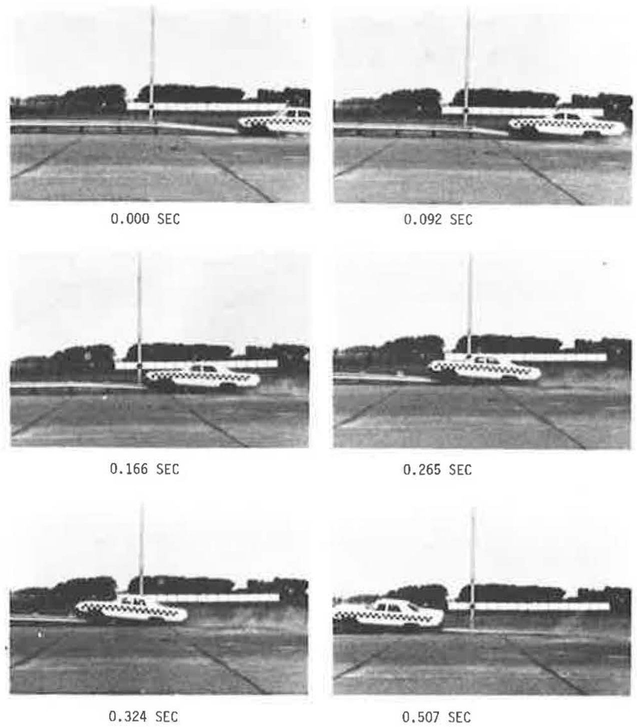


Table 2. Results of accelerometer data and vehicle-damage classifications of crash tests.

	Max Avg 0.050-s Deceleration (g)		Avg Deceleration Over Contact Time (g)		Peak Deceleration (g)		Vehicle-Damage Classification		
Test	Longitudinal	Lateral	Longitudinal	Lateral	Longitudinal	Lateral	TAD[a]	SAE[b]	Remarks
1	2.1	1.8	0.75	0.65	10.0	11.0	FC-2	12FECW1	Rode over terminal section; no rollover
2	2.1	1.5	0.65	0.55	5.7	6.9	FR-1	01RYMS1	Rode over terminal section and rail; no rollover
3	5.1	7.9	1.2	2.2	14.1	21.1	FRQ-5	01RDEE2	Smooth redirection
4	1.8	1.0	0.2	0.2	4.5	4.4	RF-1	01FFEE1	Rode over terminal section; no rollover
5	3.0	1.2	0.57	nil	5.3	5.5	FC-3	12FCEN8	Straddled rail for 57 m (188 ft) before stopping

[a]TAD = traffic-accident data project. [b]SAE = Society of Automotive Engineers.

Figure 12. Terminal before test 1.

Figure 13. Vehicle after test 1.

mph). On contact with the turned-down terminal section, the vehicle began to ride up. The rail disengaged from the backup plates and was depressed. The right front corner of the bumper of the vehicle impacted the first post and split it vertically. The vehicle continued forward, rode over the rail, returned to the roadway side of the guardrail, and finally came to rest against the rail. The position of the vehicle and the damage to the guardrail are shown in Figure 22.

The performance of the rail in this test was very good. The maximum average 0.050-s longitudinal deceleration was less than 1.8 g and all peak values were less than 4.5 g. The damage to the vehicle is shown by Figure 23. The repairs to the rail consisted of replacing one post and one backup plate. (Before this test, it had been anticipated that the vehicle would remain astraddle of the rail and knock down several posts, but this did not happen.)

Figure 14. Guardrail and terminal after test 1.

Figure 15. Guardrail and terminal before test 2.

Figure 16. Vehicle after test 2.

Figure 17. Guardrail and terminal after test 2.

Figure 18. Post 1 with backup plate and metal clips before test 3.

Figure 19. Guardrail and terminal before test 3.

Figure 20. Vehicle after test 3.

Figure 21. Guardrail after test 3.

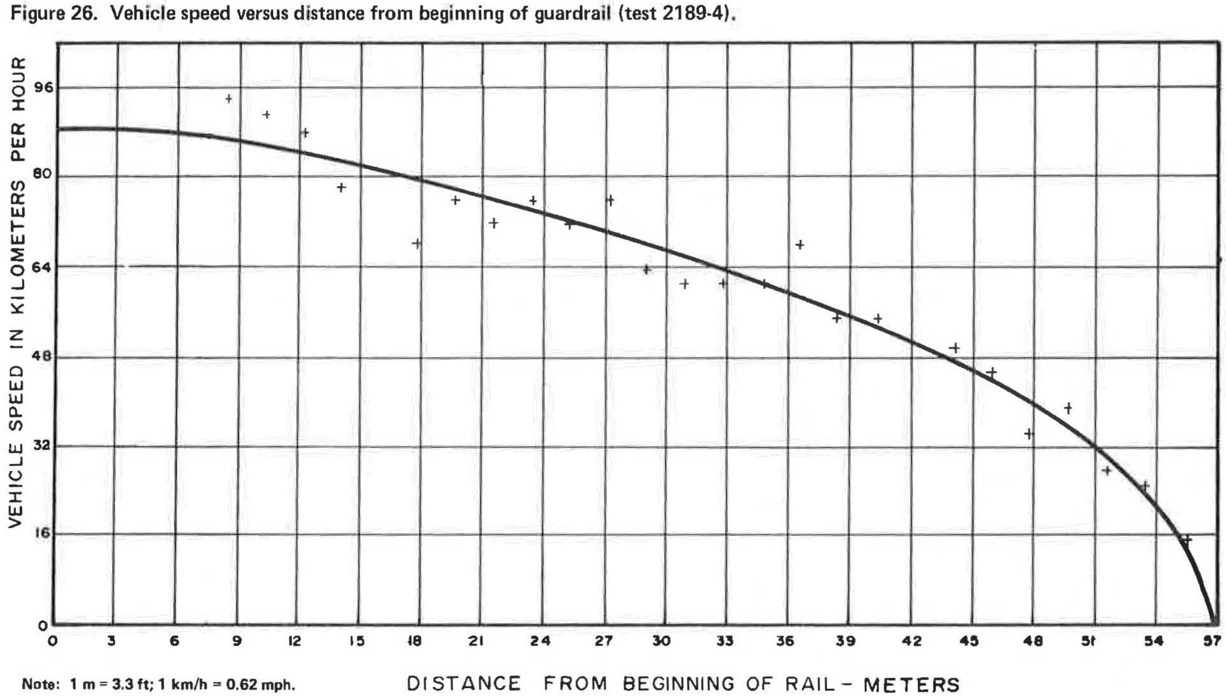
Figure 22. Position of vehicle and damage to guardrail after test 4.

Figure 23. Vehicle after test 4.

Figure 24. Position of vehicle and damage to guardrail and terminal after test 5.

Figure 25. Vehicle after test 5.

Figure 26. Vehicle speed versus distance from beginning of guardrail (test 2189-4).



Note: 1 m = 3.3 ft; 1 km/h = 0.62 mph.

Test 5

The guardrail installation for this test was identical to that used for tests 3 and 4.

In this (essentially head-on) test, a 2068-kg (4560-lb) 1970-model automobile impacted the turned-down terminal section at an angle of 5.5° and a speed of 89.0 km/h (55.3 mph). On impact, the vehicle depressed the rail in a manner similar to that of the vehicle in test 4. It then continued astraddle of the rail, exhibiting a low-amplitude, oscillatory pitching and rolling motion, and eventually stopped on the top of the rail approximately 57 m (188 ft) from the end anchor. The position of the vehicle and the damage to the guardrail are shown in Figure 24. Twenty-six posts were split, broken, or bent over. The maximum average 0.050-s longitudinal deceleration was approximately 3 g and the peak values were all below 5.5 g. The extensive damage to the undercarriage of the vehicle is shown in Figure 25. The repairs to the guardrail consisted of replacing 26 posts and eight 7.6-m (25-ft) sections of rail.

In some installations, e.g., bridge abutments and other fixed obstacles, the approach length of guardrail may be less than the 57 m (188 ft) traveled by this test vehicle. If a vehicle became captive at the end of the rail, it could impact the obstacle from which it was being protected. To obtain some indication of the potential severity of such impacts, a curve of velocity versus distance traveled by the test vehicle was developed from the documentary movie film and is given in Figure 26. To avoid the possibility of such head-on impacts, guardrail terminals should be flared away from the roadway.

CONCLUSIONS

A relatively simple method of modifying the turned-down ends of guardrail terminals has been developed. This method should eliminate or greatly minimize the probability that a vehicle impacting them will ramp and roll over. The hardware used in the design are either standard guardrail components or items that are readily available commercially.

Successful crash tests were conducted as described in the NCHRP Recommended Procedures (4). In three of the tests, the vehicle impacted the modified terminal section, depressed the rail, and rode over it without rolling over.

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Design of Barrel Trailer for Maximum Collision Protection

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This report describes a Texas type, steel-barrel trailer developed by the Highway Wayside-Equipment Research Office and the Equipment Office of the Ontario Ministry of Transportation and Communications. When attached to a sign truck, the trailer provides maximum crash protection for occupants of impacting automobiles in rear collisions at impact speeds of up to 100 km/h (60 mph). This means that restrained occupants will survive such collisions without serious injuries. (Crash protection is expected to be somewhat less for angular impacts.) The trailer can be towed at traveling speed and backed up at slow speed on a closed traffic lane. For full protection of a working crew, the trailer should be attached to the kind of heavy sign truck that is presently used in maintenance operations. Although the trailer is an extra piece of equipment and requires special driver skill in backing, it is recommended for use on high-speed highways with high traffic volumes, expressways, or freeways. The trailer reduces impact severity considerably and is more effective than nontrailer attachments at impact speeds of 80 km/h (50 mph) or less. The first prototype tried on the road has been involved in two collisions. In both instances, the impact attenuation and redirectional capabilities of the steel barrels were sufficient to prevent injuries. The connections between the barrel modules, which were originally welded, now consist of bolts and hard rubber spacers and are still being developed.

In September 1974, a car traveling at an estimated speed of 130 km/h (80 mph) struck the rear of a sign truck that was protecting a night crew who were making illumination measurements. The driver of the car was killed instantly, and the truck was severely damaged (Figure 1).

Although there were warning systems in operation and the driver was exceeding the legal speed limit, the case nevertheless dramatically illustrates the need for greater protection from such collisions for the driving public. The solid backs and rigid bumpers of the trucks now in use are road hazards of the greatest severity. Moreover, in the following year, from December 1974 to November 1975, there were 34 collisions with Ministry of Transportation and Communications sign trucks in three major districts in Ontario (Toronto, Ottawa, and Hamilton).

Because of these accidents, the Highway Wayside Equipment Research Office and the Equipment Office have initiated the development of a protective trailer for sign trucks. Prototypes have been designed and built, and their operations tested. The attachment of a barrel trailer can reduce the probability that a straight or angular rear or side collision will be fatal or result in serious injuries from about 0.7 to 0.33 (1).

DESCRIPTION OF TRAILER

Figure 2 shows the first prototype trailer in operation. Details of the design are shown in Figures 3 and 4. The trailer is made of 18-gauge paint drums, 0.9 m (3 ft) high and 0.6 m (2 ft) in diameter, with 355-mm (14-in) holes on their tops and bottoms. The barrels are tied together with adjustable steel cables and welded together by means of flat steel connectors that arch over the rims (but this design is now being changed by the substitution of bolted connections).

A cable suspension, which was added before major use, is supported on top of the barrels by a stirrup.

The trailer has the following operational features.

1. It can be pulled on smooth or moderately rough roads at a normal traveling speed without touching the

Figure 1. Damage to truck after fatal accident.

Figure 2. Operational test of crash trailer.

Figure 3. Design features of first trailer.

Figure 4. Cable suspension of barrels on first trailer.

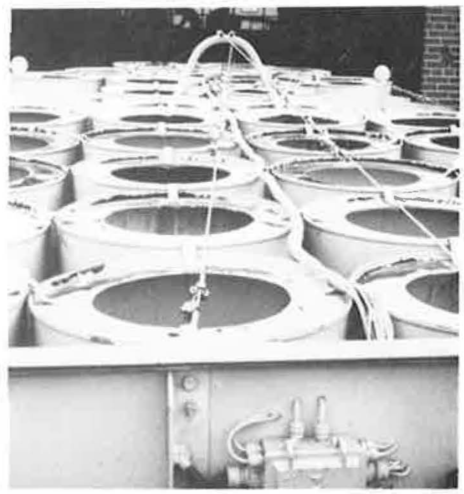
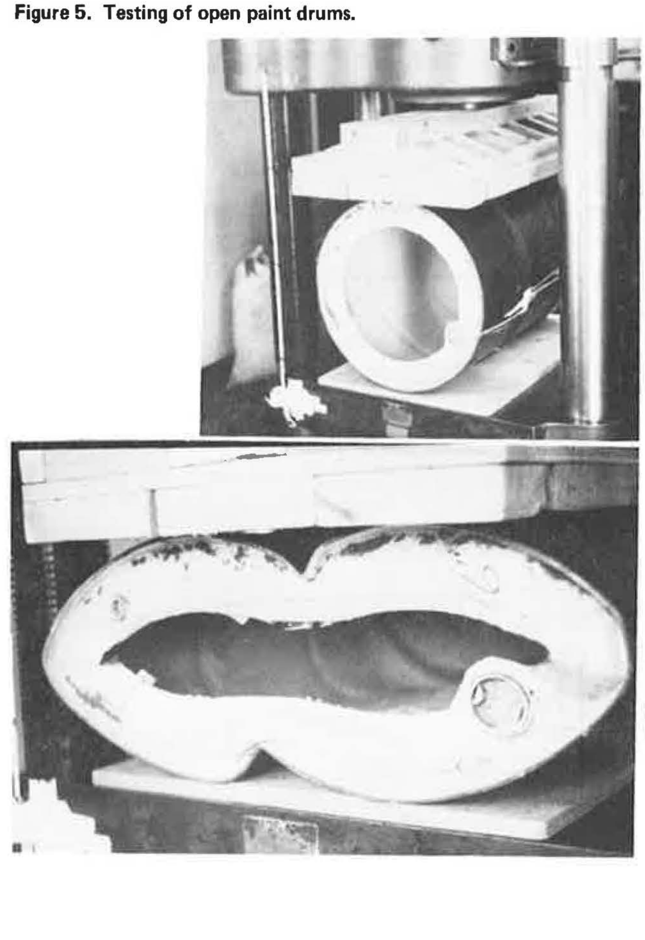


Figure 5. Testing of open paint drums.



ground if the distance of the barrels from the ground is 200 mm (8 in) or more when the trailer is standing on a horizontal plane surface.

2. It can be backed up easily by a skilled driver at a speed appropriate for setting markers.

The connection to the truck during the backing up operation is flexible, and the angular-impact behavior of the design in this situation cannot be predicted without field testing.

DESIGN FOR IMPACT ATTENUATION

The trailer can be connected more rigidly for nondriving or stationary conditions by tautly engaging the chains. In this situation, the impact behavior can be predicted fairly accurately if the crash properties are known and the hitch connections are sufficiently strong. Thus, design calculations are presented for a straight rear impact.

Crushing Force of Barrels

Although there are data for crush forces of steel drums (2, 3), crush testing for static forces or a slow loading rate was performed. The testing procedure is shown in Figure 5. Typical crushing forces and deformations of barrels are shown in Figure 6. The results of actual

Figure 6. Typical force deformation of steel drum.

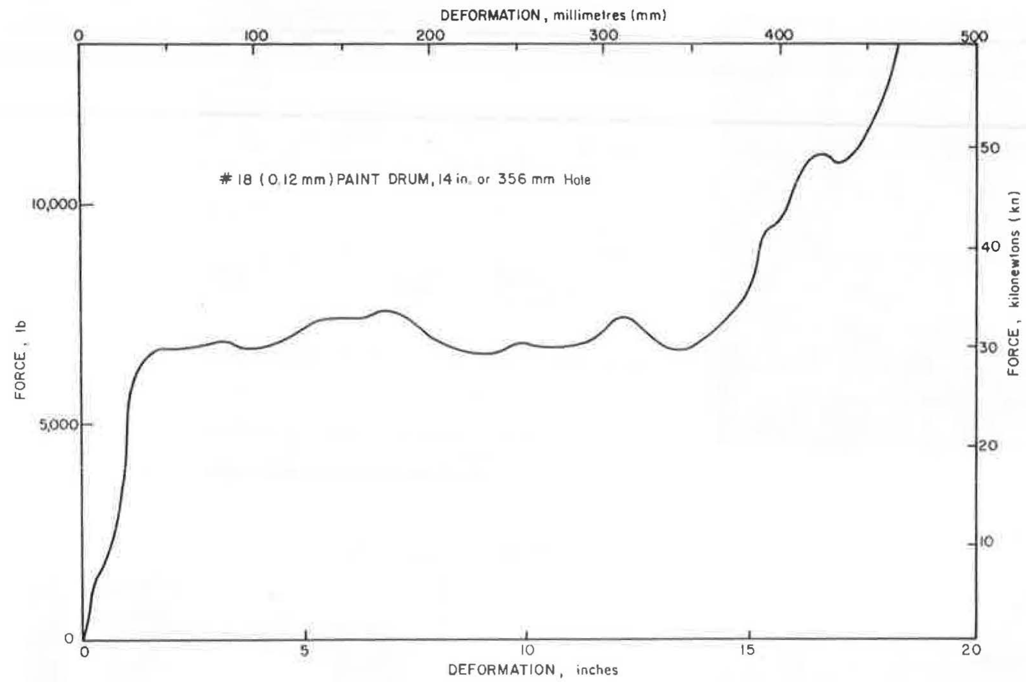
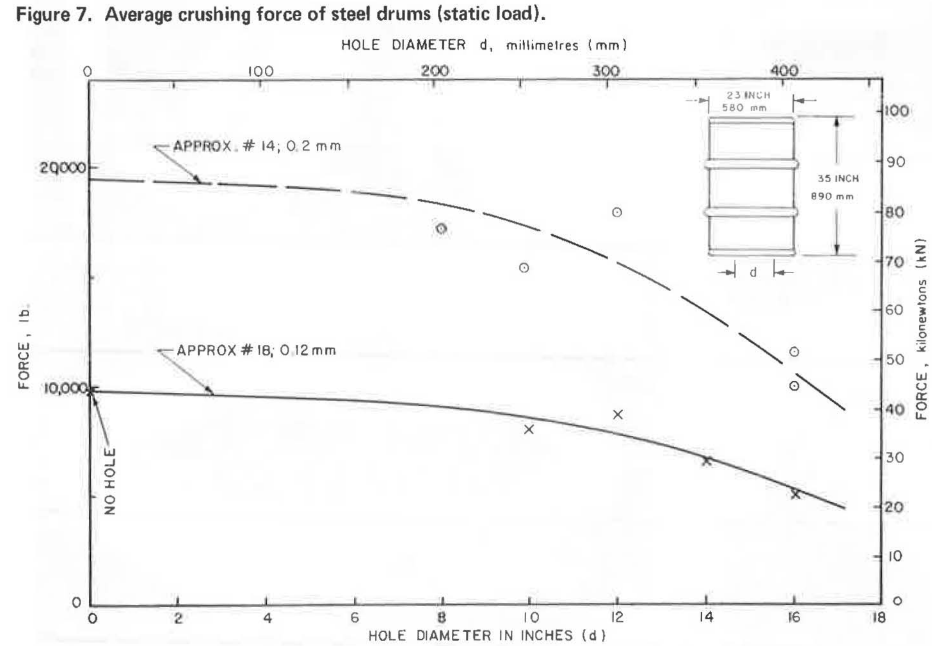


Figure 7. Average crushing force of steel drums (static load).



tests are plotted in Figure 7 (the wall thickness of the barrels used in these tests could be measured only approximately). The design criteria are given below (1 km/h = 0.6 mph and 1 kg = 2.2 lb).

Property	Criterion
Impact speed, km/h	100
Impact angle, °	0
Range of automobile weights, kg	800 to 2000

Dynamic Equations

The following equations, which may be used for the design, are derived from the physical laws of momentum and energy conservation:

$$v = mV/(m + M) \tag{1}$$

$$s = v^2/2gf \times (m + M)/M' \tag{2}$$

$$e = ½mV^2 - (m + M)v^2 \tag{3}$$

where

V = impact velocity of automobile,

v = velocity of total mass after impact,
M = mass of truck and trailer,
M′ = mass of truck only (weight on braking wheels),
m = mass of automobile,
s = skidding distance after impact,
g = acceleration of gravity [9.81 m/s^2 (32.2 ft/s^2)],
f = coefficient of friction (assumed to be 0.7), and
e = energy absorbed by crushing material.

These equations are independent of the crushing material and its attenuation mechanics. Their solutions

Figure 8. Velocity after impact.

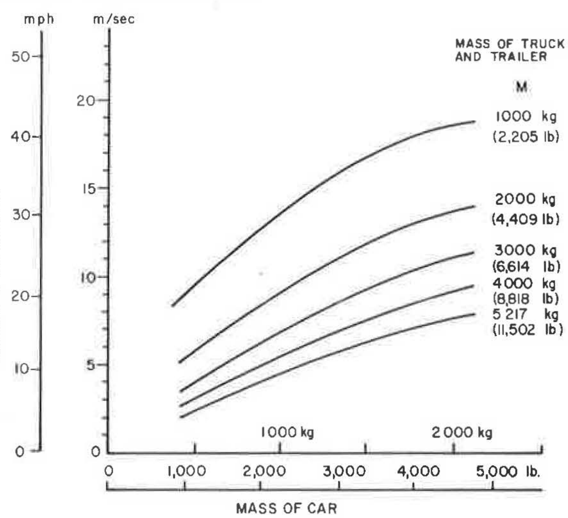


Figure 9. Skidding distance after impact.

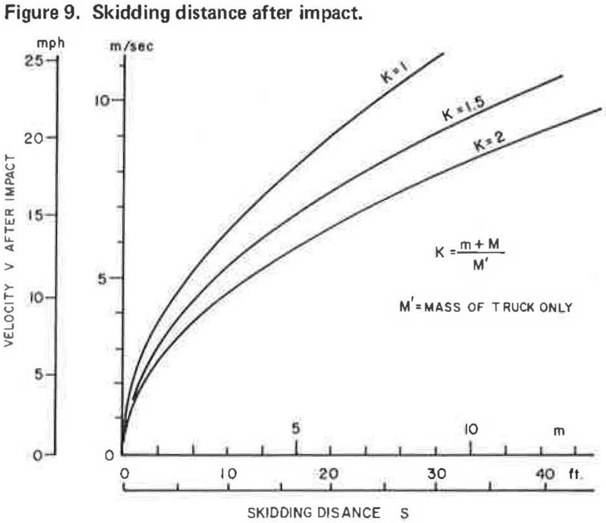


Figure 10. Dissipated impact energy.

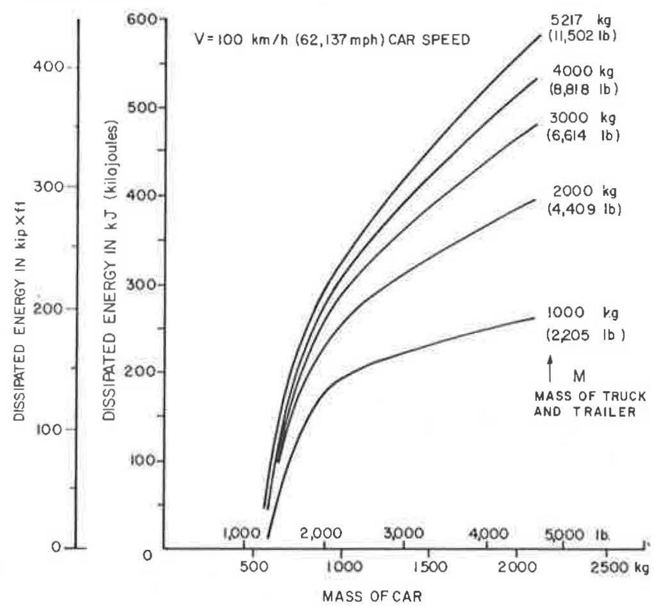


Figure 11. Average automobile deceleration.

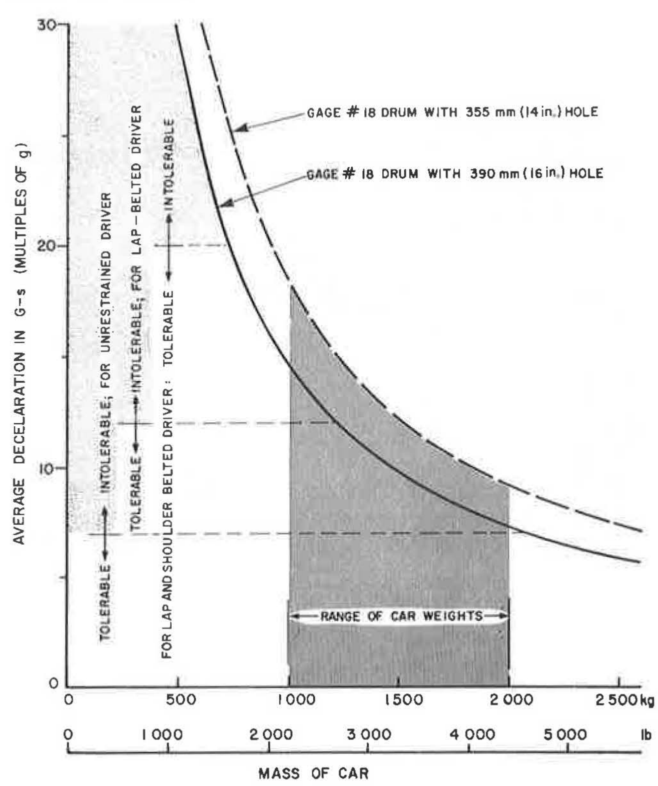


Figure 12. Design layout and angular impact force components.

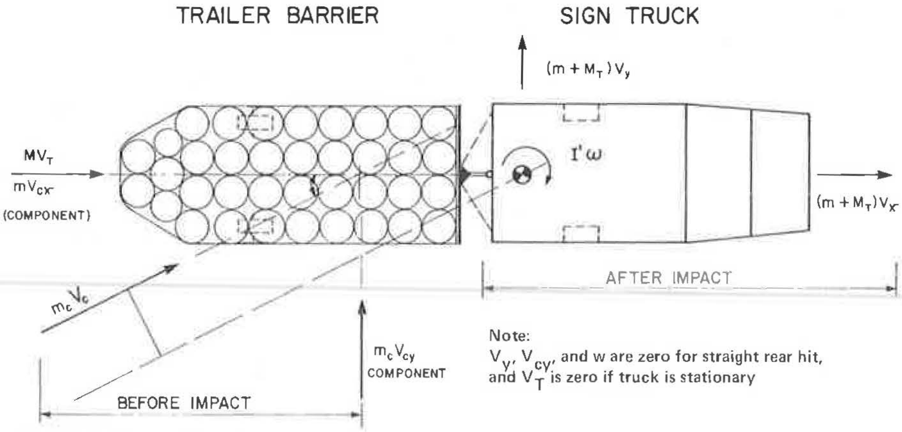
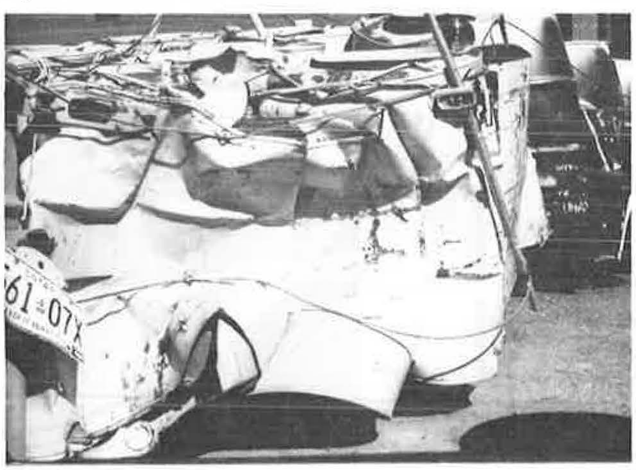


Figure 13. Damaged crash trailer after glancing-blow collision with automobile.

Figure 14. Wrecked crash trailer after angular rear collision with truck.



are plotted in Figures 8, 9, and 10 for an impact speed of 100 km/h (62.14 mph).

Crush Design

We can assume that the energy (e) is dissipated by crushing the steel drums. In this design there are nine rows of drums with four drums in each row.

For a drum made of 0.12-mm (18-gauge) steel with 405-mm (16-in) holes in its top and bottom, the static average crushing force for 65 percent of the total diameter [381 mm (15 in)] is approximately 24 kn (5400 lb). A dynamic factor of 1.5 must also be applied (2, 3). Thus, the energy-absorbing capacity for one drum is at least

$$E_1 = 1.5 \times 0.381 \times 24\ 000 = 13\ 716\ \text{J} \tag{4}$$

and, for a row of four drums,

$$4E_1 = (m \times G \times g) \times D = (\text{dynamic force} \times \text{distance}) \tag{4a}$$

where G = multiple of g for average deceleration and D = crushing distance or stroke of one drum. Therefore, we have $G = 4E_1/mgD = [(4 \times 13\ 716)/(9.81 \times 0.381)] \times 1/m$ or $G_{16} = 14\ 680/m$. The corresponding expression for drums with 355-mm (14-in) holes is $G_{14} = 18\ 350/m$; i.e., an energy absorption of 17 145 J/drum.

These estimated average decelerations are plotted in Figure 11 for a wide range of automobile weights.

What number of drums should be used in the design? For a total mass (M) of 5217 kg (11 500 lb), the dissipation energy is 555 000 J for a 2000-kg (4400-lb) automobile (from Figure 10). The number of drums (n) is then $n_{16} = (555\ 000/13\ 716) = 40.4 = 40$ and $n_{14} =$

Figure 15. Modified crash trailer.

(555 000/17 145) = 32.4 = 32, where n_{16} and n_{14} are the numbers of drums having 405-mm (16-in) and 355-mm (14-in) holes respectively. The final design has 37 drums with 405-mm (16-in) holes. This design permits tolerable average decelerations for lap and shoulder-belted drivers in the range of automobile weights indicated in Figure 11, and drivers of heavier automobiles would need only lap belts to have the same degree of protection that lap and shoulder-belted drivers have in smaller automobiles. The design layout is shown in Figure 12.

The average crushing force and therefore the average deceleration are the same for both low and high impact speeds. Fewer barrels are crushed by lighter automobiles. The truck skids a longer distance after impacts from heavier vehicles (Figure 9).

Angular Impact

Lack of knowledge about the parameters of the system (such as the mass moment of inertia of the truck and trailer combination) makes it impossible to evaluate the angular-impact behavior of the design. But, it can be anticipated that less energy will be absorbed by crushing as energy is dissipated by the friction that will accompany rotation or spinning of the system. The average deceleration will depend on the impact angle and the point of contact.

COLLISIONS WITH THE FIRST PROTOTYPE TRAILER

The first prototype, as shown in Figures 2, 3, 4, and 13, was used in the summer of 1975 to protect the operation of a low-speed paint striper. Within 3 months, there were two collisions.

The first collision occurred on the Queen Elizabeth Way, eastbound, at 5:25 a.m. on June 22, 1975. The driver of the sign truck gave the following report. The truck was traveling at a speed of 10 to 13 km/h (6 to 8 mph) in the center lane behind the striper, which was painting the two broken lines simultaneously. The trailer was hit a glancing blow by a compact automobile having an estimated speed of 105 to 113 km/h (65 to 70 mph). Only one barrel and its attached light fixtures were damaged. The automobile, which had several occupants, was redirected, crossed two lanes, and drove on without stopping. If the trailer had not been attached, the automobile would probably have crashed into the right rear corner of the truck. This means that considerable accident costs and possibly human lives were saved in this accident. The damage to the trailer is shown in Figure 13.

The second collision occurred in the right-hand lane of Highway 400, northbound, at 1:20 p.m. on July 14, 1975. The sign truck with the trailer attached was traveling at a speed of about 16 km/h (10 mph) behind the striper. The trailer was hit at a slight angle by an empty 2700-kg (3-ton) truck, traveling at about 100 km/h (60 mph). The driver of the impacting truck applied the brakes before the crash and thus the truck impact speed was probably reduced. The barrels on the right-hand side of the trailer were fully or partially crushed, and the trailer was pushed under the sign truck. The damage to the trailer was beyond repair (Figure 14). The driver of the impacting truck was uninjured although he would probably have suffered injuries if his truck had hit the right-hand corner of the sign truck.

FURTHER DEVELOPMENTS

The trailer design was modified after an evaluation of the accident performance of the first prototype. The modifications are shown in Figure 15. A cable suspension supported by a stirrup was mounted on top of the barrels to increase stability during rough driving operations, and a bolted connection that uses steel washers and hard rubber spacers between the barrels is being developed.

At the present stage of development of the trailer, several questions remain unanswered:

1. What are the exact barrier-resistance characteristics for a straight rear impact?
2. What is the behavior of the trailer at an angular impact of 15 or 25°?
3. What is the performance of bolted connections between barrels?

Testing under controlled conditions and supplementary computer simulations will be necessary before these questions can be answered. This topic has been discussed by Jung and Billings (4, 5).

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Upgrading Safety Performance by Retrofitting Bridge-Railing Systems

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The inadequate performance of many current bridge-railing systems in vehicle collisions has resulted in a large number of injuries and fatal accidents. An analysis of representative bridge-railing designs submitted by 44 highway agencies in 1974 showed that most did not fully conform to the 1973 American Association of State Highway Officials specifications. Most of these bridge-railing installations have been in service for more than 10 years and were designed to less restrictive requirements. An alternative to replacing inadequate installations with conforming systems is to upgrade the existing installation with a modification or retrofit design. To reduce the number of potential retrofit designs, existing bridge-railing systems can be grouped in four categories according to their profile geometries and features. Each category has its own constraints for retrofit modification (i.e., curbs, parapets, and such), but a properly conceived retrofit design for a category can be adapted to any bridge-railing system in that category. About 82 percent of the existing systems reported in the survey can be placed in Southwest Research Institute categories II and III. Five retrofit designs for these categories were developed and evaluated by a 22-vehicle crash-test program. These five designs are judged suitable for carefully monitored in-service use to upgrade the safety performance of substandard systems.

Olson (1) has shown that about one-third of all fatal accidents on freeways during the period of 1965 to 1967 involved a vehicle that ran off the road and hit a fixed object and that about 22 percent of those fixed objects were the barrier-railing systems of bridges. A bridge-railing system includes the approach guardrail, the transition, and the railing itself. Hosea (2) has reported that an element of a bridge-railing system was the first object struck in 18 percent of the fatal accidents at fixed objects on completed sections of the Interstate highway system in 1968. An analysis of accidents at bridge railings by location has shown that 73 percent of the vehicles impacted the approach guardrail and bridge end and 27 percent collided with the bridge railing (1). The performance of the barrier systems in these accidents was judged inadequate because 16 percent of the vehicles either vaulted or penetrated the installation and 52 percent pocketed or snagged.

In 1967, the American Association of State Highway Officials (AASHO) published a study of design and operational practices related to highway safety (3) that emphasized the need for a structurally sound transition between guardrails on bridge approaches and the bridge railings themselves, and in several states, intensive programs to upgrade existing bridge-railing systems were initiated. Accident statistics from California (4), shown in Table 1, indicate that those programs have been most successful. The rate of fatal accidents on freeways per 100 million vehicle kilometers of travel (MVKT) for bridge railing decreased in the period from 1965 to 1973 by about 50 percent and even more significantly in 1974 and 1975 when the 24.6-m/s (55-mph) speed limit was set. This improvement in highway safety, which has been confirmed by observations in other states, may be partially attributed to improvements in vehicle crashworthiness and occupant restraint systems. Nevertheless, the California statistics indicate that upgrading bridge-railing systems has helped to reduce the number of fatal accidents.

An approach that promises further reductions in highway fatalities is that of upgrading substandard bridge-railing systems with cost-effective devices that can be easily retrofitted to existing bridges. Such devices should be readily adaptable to a majority of existing bridges and quickly and economically adjustable in the field.

ANALYSIS OF EXISTING BRIDGE-RAILING SYSTEMS

A letter survey of 51 highway agencies was carried out in September 1974. The purpose of the survey was to determine (a) the types of bridge-railing systems that are in current use, (b) the types of approach guardrails that are in use, and (c) the accident experience of different bridge-railing designs. A total of 44 state highway agencies responded to at least a part of the survey. These responses reported on 3940 km (2450 miles) of bridge railing, which is about 15 to 20 percent of the estimated total [20 600 to 24 780 km (12 800 to 15 400 miles)] in the United States. This is considered a significant and meaningful sample.

From the information assembled in this survey, 14 representative designs for bridge-railing systems were selected and analyzed with respect to their conformance to the 1973 AASHO specifications (5) and their predicted (or observed) performance during vehicle crash tests. The selection of the 14 examples was based on two factors: (a) a minimum of 56 km (35 miles) of reported installation and (b) sufficient design information. The combined length of the 14 examples is 2665 km (1656 miles) or 67 percent of the total reported in the survey.

All of these systems were designed before the adoption of the 1973 specifications and should not be expected to conform. However, these specifications can be used as a safety-performance reference. The analysis showed that only 1 design fully conformed, 1 design could be considered marginally conforming, and 12 out of the 14 designs did not conform with one or more provisions of the specifications. On the basis of length, 68 percent of the systems analyzed did not conform, and 27 percent conformed marginally. A more detailed analysis of existing systems is given in the program report (6).

CATEGORIZATION OF BRIDGE-RAILING DESIGNS

It is estimated that there are more than 200 unique bridge-railing designs in use on the approximately half million bridges in the United States. To develop and evaluate safety modifications for each of these designs would require a formidable expenditure of effort and most certainly would not be cost-effective.

An alternative approach is to place existing bridge-railing designs, regardless of their safety-performance capability, in one of the four Southwest Research Institute (SwRI) categories that are illustrated in Figure 1 and described below.

Category	Description
I	Metal rail and metal posts mounted flush to bridge deck; curb or walk (if present) does not project more than 0.15 m (6 in) above bridge deck

Category	Description
II	Concrete baluster rail or concrete parapet with up to four metal rails; no curb or walk that projects into traffic lane beyond face of rail
IIIN	Concrete parapet with walk less than 0.061 m (24 in) wide and curb; one to three rails may be attached to parapet
IIIW	Concrete parapet with walk 0.61 m (24 in) or more wide and curb; one to three metal rails may be attached on top of parapet
IV	Concrete safety shape with or without one or two metal rails

The definitions of these categories are based on the geometries of the bridge-railing profiles and are specifically delineated so that all bridge railings within a category that lack adequate safety-performance capability will be amenable to a common retrofit design. The fact that a specific bridge railing is exactly like or only remotely similar to one of the four categories is not intended to imply its level of safety performance. The purpose of the categorization is to reduce the number of retrofit designs so that a design developed for a given category will be applicable to all of the bridge railings in that category.

CONSTRAINTS ON RETROFIT DESIGNS

After a review of bridge-railing design drawings and interviews with bridge designers, the following constraints were added to the general bridge-railing service requirements for the retrofit designs.

1. A majority of the bridges of interest are narrow. Although pavements and shoulders have often been widened in recent years, narrow bridges have been retained because of the expense and technical difficulty required for their modification. The high rates of accidents at bridge ends have been attributed to both the narrow roadways on bridges and the funnel effect of the transition from a wide highway to a narrow bridge. Consequently, a large number of bridges that may require retrofitted bridge railings cannot afford further reduction of the bridge-deck width by encroachment of the bridge-railing modification and, thus, bridge-railing retrofit designs must attempt to maintain present bridge-deck widths.
2. Curbs and walks extending out from a bridge railing are only marginally effective in redirecting errant vehicles. Furthermore, vehicles impacting curbs are caused to jump and may strike the backup structure in an unpredictable attitude. The newest design standards minimize the use of curbs in front of longitudinal traffic barriers for this reason. However, the curbs are an integral part of the structure of many bridges and cannot be removed without major reconstruction. Thus, retrofit designs should accommodate the existing curb conditions.

RETROFIT DESIGNS

In developing the retrofit designs a majority of the effort was directed toward categories II and III for the following reasons:

1. Category I represents fewer than 14 percent of the existing bridge-railing systems. Moreover, these designs can often be upgraded by simple procedures such as the replacement of a rail or may be economically replaced by a conforming design. (While the least common, some category I barriers may be the most hazardous and thus justify a high priority for upgrading if exposure is high.)
2. About 82 percent of the existing bridge-railing systems are either category II (42 percent) or category III (40 percent).
3. Category IV represents fewer than 6 percent of existing bridge-railing systems, and most category IV designs already satisfy the 1973 AASHO requirements and perform satisfactorily during vehicle crash tests.

In this program, a bridge-railing system is defined as including (a) the approach guardrail and terminal, (b) the bridge railing, and (c) the transition from the approach guardrail to the bridge railing, and it is important for safety performance in collisions that these elements behave in an integral manner. Hence, the entire bridge-railing system should be considered during the design phase, although the emphasis in this research was directed toward the bridge-railing and transition elements, and less attention was given to the approach guardrail and terminal. The transition elements in the retrofit designs include a portion of the approach guardrail, but no upstream guardrail terminals were evaluated in the test phase. Suitable terminals have been developed and evaluated in other programs (7).

One retrofit design was developed for the category II bridge railing. The primary element of this R(II)-1 design is a new railing element known as the tubular thrie beam, which is fabricated by welding two thrie (or triple corrugated) beams together as shown in Figure 2a. The beam is joined to and blocked out from the concrete baluster rail by 0.15-m (6-in) diameter collapsing tubes spaced at 2.5-m (8.3-ft) intervals; there was no attempt to space the tube supports to the concrete post in the test system.

Four retrofit designs were developed for the category IIIN bridge railing, and one was developed for the category IIIW bridge railing. A sectional view of the R(IIIN)-1 design is shown in Figure 2b. The essential features of the design are the tubular thrie beam that is blocked out from the concrete parapet by brackets spaced at 2.5-m (8.3-ft) intervals. The bracket is made of a relatively stiff 15 by 15 by 4.8-mm (6 by 6 by 3/16-in) transverse section (TS) and a 0.15-m (6-in) diameter tube section with a 3.2-mm (1/8-in) wall. The traffic face of the tubular thrie beam aligns with the curb and does not encroach on the bridge deck. Obviously, the retrofit design eliminates use of the walkway. The collapsing tubes provide 0.15 m (6 in) of dynamic deflection, which attenuates the lateral redirection forces on the car.

The R(IIIN)-2 design shown in Figure 2c has two 0.18-m (6.87-in) deep, semielliptical aluminum rails at mounting heights of 0.38 and 0.68 m (15 and 27 in). The rails are blocked out from the concrete parapet with rigid brackets spaced at 2.4-m (8-ft) intervals. The traffic face of the rail is about on the curb line; for walks of other widths, the bracket geometry would require adjustment to maintain the approximate alignment of the rails and curb. The brackets are attached to the category IIIN system by six 19-mm (3/4-in) diameter anchor bolts embedded 0.15 m (6 in) deep into the concrete parapet and walk. The transition from the approach guardrail to the bridge railing is effected by a stepped reduction in spacing of the soil-mounted posts from 2.4 to 0.9 m (8 to 3 ft) and the use of posts with soil-bearing plates.

The R(IIIN)-3 design shown in Figure 2d has beams of 20.7-MPa (3000-lb/in^2), 28-d compressive strength concrete cast at the test site. The reinforcing consists of No. 4 bars in each corner that are continuous for the length of the 8.50-m (27.9-ft) beam, and the tie bars are No. 4 size and spaced 0.66 m (26 in) apart along the beam. The 25-mm (1-in) diameter anchor bolts are cast into the rear side of the beams, 76 mm (3 in) from the base, on typical 2.4-m (8.0-ft)

Table 1. Fatal accidents involving bridge railings on California freeways.

Statistic	1965	1966	1967	1968	1969	1970	1971	1972	1973	1974[a]	1975[a]
Accidents at bridge railings	23	26	29	29	30	17	15	27	25	9	5
Accidents at bridge end posts at gore[b]	3	10	5	3	5	6	2	8	5	6	3
Accidents at guardrail at fixed objects[c]	14	16	14	11	13	7	9	13	16	12	5
Total possible bridge-railing accidents	40	52	48	43	48	30	26	48	46	27	13
Travel MVKT	37 000	41 785	46 450	54 244	59 498	63 500	68 452	74 444	79 308	78 753	84 421
Total freeway completed by end of year, km	2 877	3 313	3 839	4 188	4 402	4 846	5 139	5 514	5 754	5 910	5 997
Rate of fatal accidents per bridge railing (per 100 MVKT)	0.17	0.20	0.17	0.13	0.13	0.08	0.06	0.10	0.09	0.06	0.02

Note: 1 km = 0.62 mile.

[a]Maximum speed limit = 24.6 m/s (55 mph).

[b]This item is actually not a bridge-railing problem that can be solved by a retrofit design but rather is a head on impact situation that is best handled by a crash cushion; it is included to permit comparison to Olson's data (1).

[c]Estimated to be 39 percent of the guardrail adjacent to fixed objects.

Figure 1. SwRI categories for classification of bridge-railing systems.

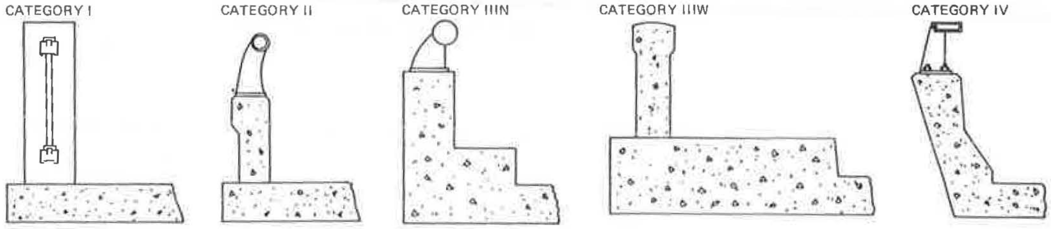
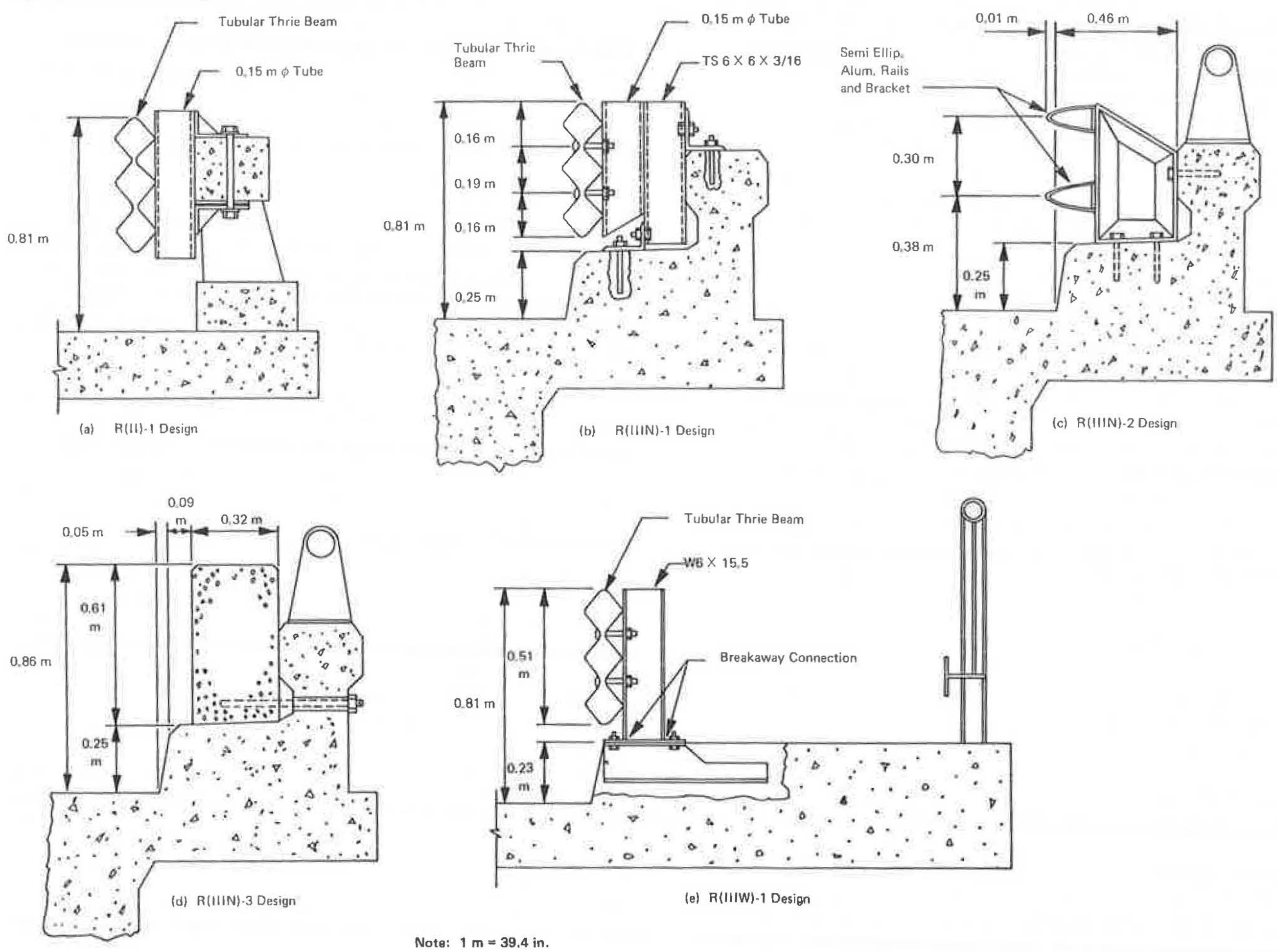


Figure 2. Retrofit designs.



Note: 1 m = 39.4 in.

Figure 3. Test installations.

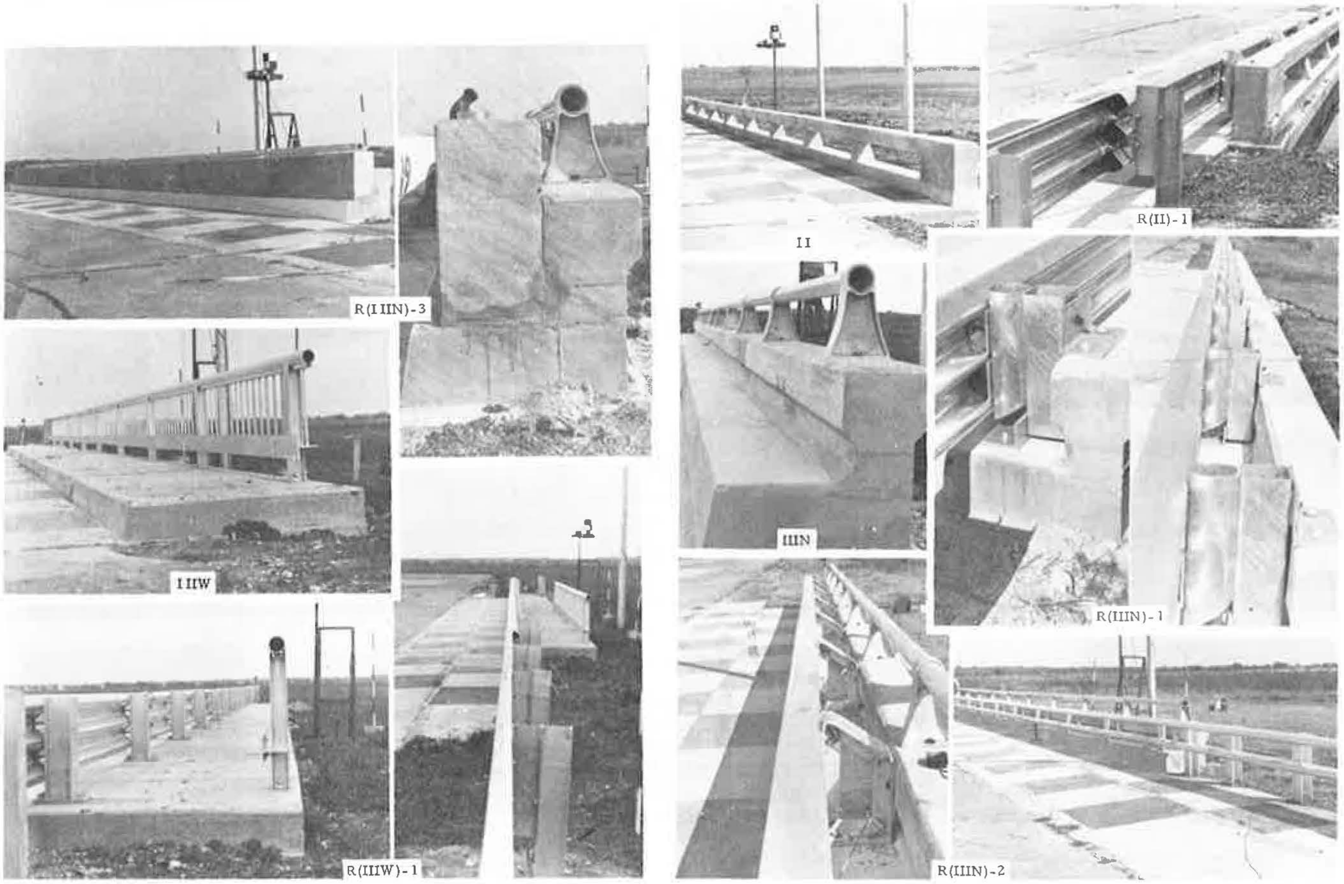


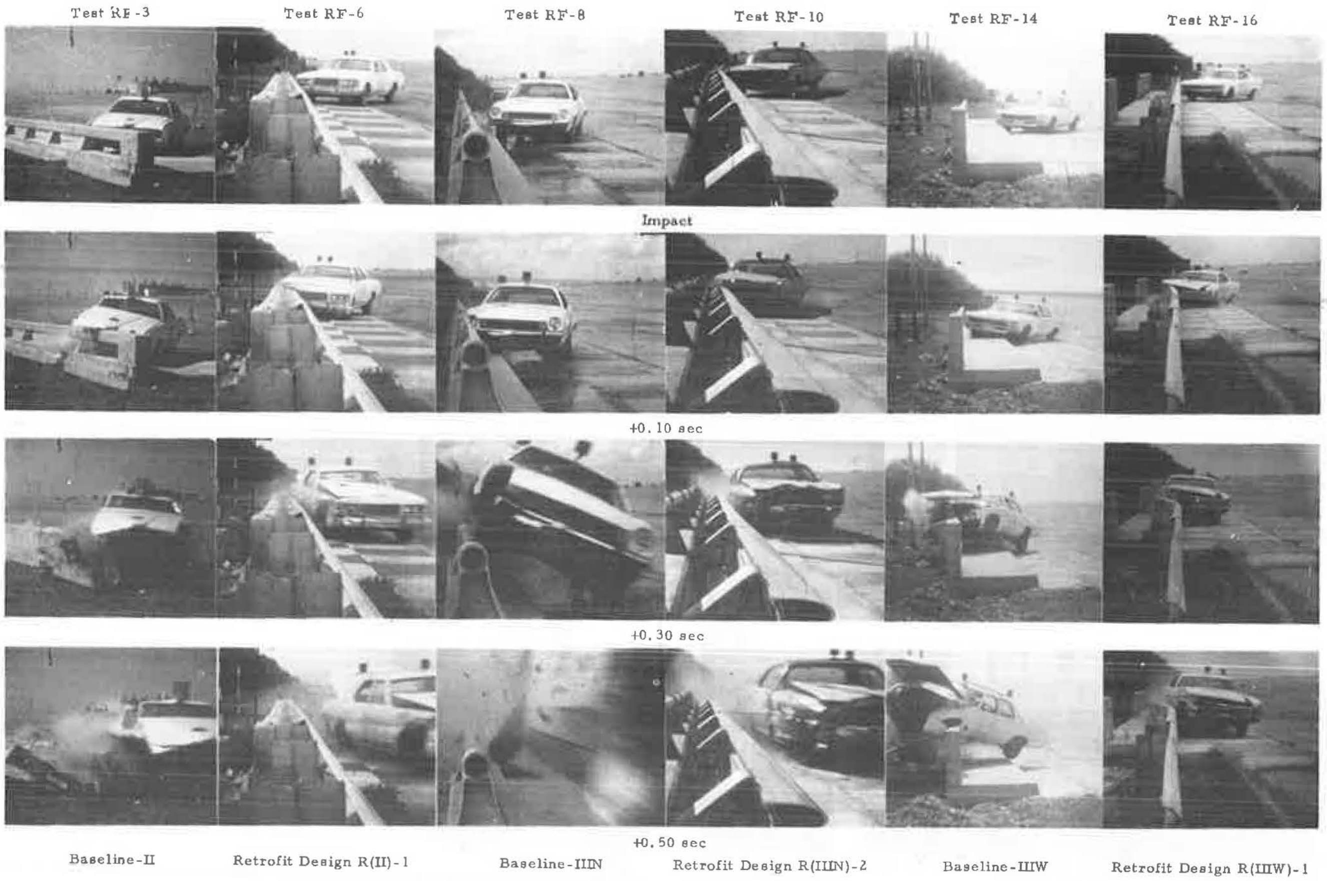
Table 2. Summary of full-scale crash-test results.

Test	Barrier Description	Vehicle Weight (kg)	Impact Speed (m/s)	Impact Angle (°)	Vehicle Exit Conditions: Speed (m/s)	Vehicle Exit Conditions: Angle (°)	Vehicle Acceleration (max over 50-ms duration) (g): Longitudinal	Lateral	Resultant	Max Permanent Rail Deflection (mm)	Remarks
3	II	2041	27.0	30.0	13.7	+2.0	-10.7[a]	-12.3[a]	16.3[a]	0	Vehicle damage severe; barrier damage considerable
4	II	966	25.5	15.5	20.6	-5.8	-4.1[a]	-7.1[a]	8.2[a]	0	Vehicle frame damage considerable
5	R(II)-1	1021	25.9	17.1	22.3	-6.8	-4.1[b]	-8.4[b]	9.1[b]	13	Vehicle smoothly redirected and drivable after test
6	R(II)-1	2041	27.1	25.0	24.4	-6.0	-5.9[b]	-11.7[b]	13.1[b]	127	Vehicle smoothly redirected and drivable after test
7[b]	R(II)-1	2132	26.9	25.0	21.0	-10.0	-4.0[b]	-7.5[b]	7.6[b]	549	Vehicle smoothly redirected and drivable after test
8	IIIN	971	28.0	15.9	24.2	+7.4	-3.6[a]	-5.3[a]	5.9[a]	0	Vehicle mounted walk during redirection; frame damage considerable
9	IIIN	2142	26.1	29.4	17.0	-11.2	-9.6[a]	-4.2[a]	10.4[a]	0	Vehicle mounted walk during redirection; frame damage considerable
1	R(IIIN)-1	971	28.4	16.8	24.9	0	-3.7[a]	-6.1[a]	6.3[a]	27	Vehicle smoothly redirected and drivable after test
2	R(IIIN)-1	1950	29.8	23.9	23.5	-2.9	-8.2[a]	-8.2[a]	11.0[a]	152	Vehicle smoothly redirected; two tubes on barrier collapsed
13[b]	R(IIIN)-1	2041	29.1	26.1	19.6	-0.7	-7.2[b]	-8.7[b]	10.7[b]	457	Vehicle damage severe; barrier damage considerable
10	R(IIIN)-2	1974	26.8	21.7	21.2	-3.5	-4.2[a]	-5.9[a]	7.2[a]	0	Vehicle smoothly redirected; frame damage considerable
11	R(IIIN)-2	930	28.8	15.1	25.0	-9.0	-3.8[a]	-5.9[a]	6.6[a]	0	Vehicle smoothly redirected; sheet-metal damage minor
12[b]	R(IIIN)-2	2006	35.6	19.9	23.8	-8.1	-5.9[a]	-8.2[a]	9.8[a]	366	Vehicle frame damage extensive
18	R(IIIN)-3	2006	29.2	7.5	27.8	+4.8	-6.5[a]	-5.6[a]	7.9[a]	0	Vehicle smoothly redirected and drivable after test
19	R(IIIN)-3	2006	27.1	11.5	24.8	+2.5	-4.4[b]	-4.9[b]	6.3[b]	0	Vehicle smoothly redirected and drivable after test
20	R(IIIN)-3	971	24.8	9.2	20.9	-12.0	-4.3[b]	-8.3[b]	9.4[b]	0	Vehicle smoothly redirected and drivable after test
21	R(IIIN)-3	971	19.7	16.2	16.8	-1.6	-5.2[b]	-6.3[b]	8.1[b]	0	Vehicle smoothly redirected; sheet-metal damage minor
22	R(IIIN)-3	971	27.7	18.3	24.5	+0.7	-8.2[b]	-16.1[b]	17.7[b]	0	Vehicle smoothly redirected; frame damage considerable
14	IIIW	1944	27.7	25.3	n/a	n/a	-4.5[a]	-3.3[a]	4.8[a]	n/a	Vehicle penetrated barrier; damage extremely severe
15	R(IIIW)-1	957	29.0	13.5	26.1	-5.7	-1.8[a]	-5.5[a]	5.8[a]	12	Vehicle smoothly redirected and drivable after test
16	R(IIIW)-1	1967	28.0	25.8	22.7	-3.9	-4.4[a]	-6.2[a]	7.6[a]	61	Vehicle smoothly redirected; frame damage considerable
17[b]	R(IIIW)-1	2021	21.5	28.6	13.9	-4.8	-4.6[a]	-4.3[a]	6.3[a]	155	Vehicle damage severe; barrier damage minor

Note: 1 kg = 2.205 lb; 1 m/s = 2.237 mph; 1 mm = 0.039 in.

[a]Obtained from high-speed cine data. [b]Obtained from accelerometer data. [c]Bridge railing transition test.

Figure 4. Dynamic performance of baseline and typical retrofit designs.



spacings, and project horizontally 0.30 m (12 in). After a 28-d field cure, the beams are lifted into place on the 0.41-m (16-in) wide walk and set in a leveling grout. During the setting of the beams, the anchor bolts are inserted through 38-mm (1.5-in) drilled holes in the parapet and locked into place.

A sectional view of the R(IIIW)-1 design is shown in Figure 2e. A tubular thrie beam at a mounting height of 0.81 m (32 in) is supported on 15 by 22-cm (6 by 8.5-in) W-posts spaced 2.54 m (8.33 ft) apart. To provide lateral flexibility during vehicle collisions, the post-to-baseplate weldments are portioned to break at a loading of 4536 kg (10 000 lb). The foundation beam distributes the impact forces and moments so that no damage is sustained by the concrete during impact, and repairs after an impact will consist of replacing one or more posts.

TEST PROGRAM

A program of 22 full-scale crash tests with vehicles ranging from 930 to 2142 kg (2050 to 4722 lb) was conducted to examine the dynamic performance of the five retrofit designs in terms of the stated performance goals. The test installations are shown in Figure 3. Visual comparisons of the dynamic performances of baseline and retrofitted systems are shown in Figure 4, and the results of the test series are summarized in Table 2.

A detailed description of the bridge-railing designs and the test procedures and results have been discussed by Michie and others (6).

SUMMARY

1. Although recent accident statistics indicate that the safety performance of bridge-railing systems has improved as new designs have been introduced and old installations upgraded, as late as 1968 bridge-railing systems did not perform satisfactorily with respect to today's standards in more than 68 percent of the fatal accidents at bridge railings. An appraisal, according to the 1973 AASHO bridge specifications, of 14 specific bridge-railing designs (representing 67 percent of those surveyed) showed that only 1 system conformed, 1 system marginally conformed, and 12 did not conform with one or more provisions of the specifications. On the basis of lengths of the systems, 68 percent of the existing installations did not conform, and 27 percent of them marginally conformed.

All 14 systems had been designed prior to 1973 to conform to earlier specifications, and the nonconforming deficiencies, which were minor in several instances, were expected.

2. Inadequate safety performance of the approach-guardrail segment can be attributed to inadequate design or improper layout and installation or both. Most states have now adopted improved designs for approach guardrail, but this trend is too recent to be significantly reflected in the field.

3. Although there are more than 200 unique bridge-railing systems in service, these systems can be grouped into four categories, within which all variations are amenable to a common retrofit design. Categories II (parapet but no curb) and III (parapet and curb with as many as four metal rails) represent about 82 percent of all bridge-railing systems in service.

4. Five bridge-railing safety-improvement modifications (for categories II and III systems) have been developed and evaluated. These modifications are judged suitable for carefully monitored in-service use.

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Evaluation of Concrete Safety Shapes by Crash Tests With Heavy Vehicles

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Three crash tests were made to evaluate concrete median barriers at speeds of approximately 70 and 90 km/h (45 and 55 mph) and angles of approximately 7 and 16° with an 18 000-kg (40 000-lb) intercity bus. The 61.0-m (200-ft) long installation was cast in place and reinforced with one number 4 bar placed 150 mm (6 in) below the barrier top. The freestanding barrier was restrained by a 25-mm (1-in) layer of asphalt placed at the installation bottom on the side opposite the impact. The results of the program include the following: The safety shape performed well at the lower angle impacts with no barrier distress or translation. The severe test [an impact speed of 85.1 km/h (52.9 mph) and an impact angle of 16°] showed that the concrete safety shape with minimum reinforcement and foundation restraint can redirect large vehicles at high impact speeds and angles. In the severe test, the rear-end impact during redirection was the principal cause of the extensive barrier damage and displacement.

The concrete safety shape is a widely used traffic barrier. Although it was originally used as a median barrier, it is also used on structures and roadway shoulders. This paper is taken from the report (4) of a study of safety shapes that was sponsored by 21 transportation agencies and administered by the Federal Highway Administration Office of Research. One part of this program was a crash-test evaluation that used an 18 000-kg (40 000-lb) intercity bus impacting a concrete median barrier (CMB) under various conditions.

BACKGROUND

In 1971, 36 states used concrete safety shapes to some extent (1). Of these 36 states, 19 specified the shape first used by New Jersey, which is denoted as MB5 by Michie and Bronstad (2).

In this program, a survey of 25 agencies provided information about CMB accident cases. The following observations were made:

1. The performances of various shapes are comparable except in the prevention of vehicle rollovers, for which the MB5 shape has a definite advantage.
2. A number 4 bar placed 152 mm (6 in) below the top of the barrier is the most common reinforcement used in CMB construction.
3. The CMB is effective in containing and redirecting large vehicles. Only two of the 49 heavy-vehicle accidents reported resulted in penetration of the barrier.
4. The barrier failures that occur are due primarily to heavy-vehicle impacts.

Full-scale, heavy-vehicle crash tests of the CMB have been extremely limited (3). In this program, a series of tests was used to evaluate the performance of a lightly reinforced MB5 barrier with minimal foundation restraint when impacted by an 18 000-kg (40 000-lb) intercity bus. The cast-in-place installation, as shown in Figure 1, was 61.0 m (200 ft) long and was reinforced with one number 4 bar placed 152 mm (6 in) below the barrier top. Restraint of the freestanding barrier was provided by a 25-mm (1-in) layer of asphalt, 1.2 m (4 ft) wide, on the bottom of the installation on the side opposite the impact.

TEST PROCEDURES

The crash tests were performed with a vehicle controlled by linear actuators attached to the steering linkage, and the linear actuators were remotely controlled through a hard line by the operator in the chase vehicle. The vehicle ignition and brakes were remotely controlled through a tether line that also carried the signals from strain-gauge accelerometers, which were mounted to the vehicle floor pan, 0.30 m (12 in) aft of the front axle

Figure 1. MB5 barrier.

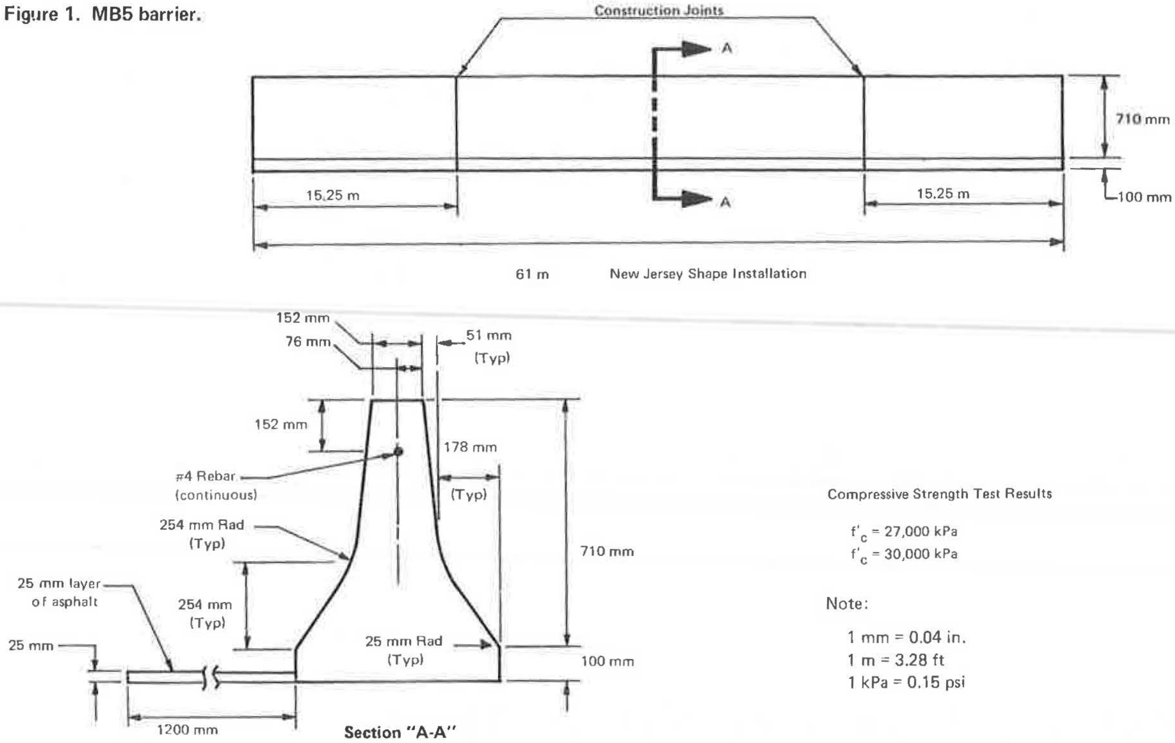
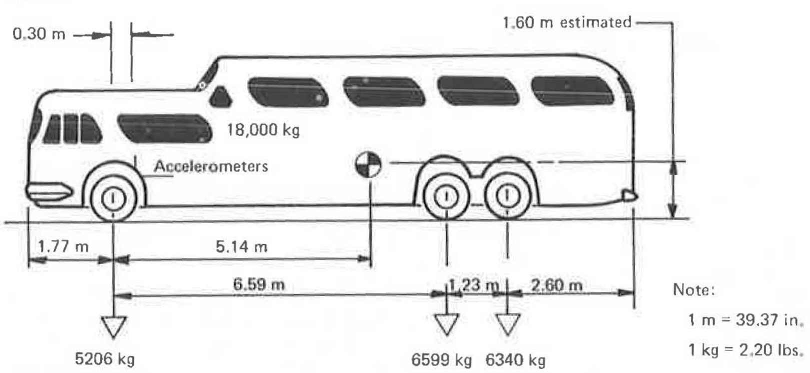


Figure 2. CMB test vehicle.



on the longitudinal centerline (Figure 2). The data were derived from two sources: micromotion analysis of high-speed film and accelerometers.

The data were taken from the film by using a motion analyzer and processed by the Southwest Research Institute Data IV motion-analysis computer program. The strain-gauge accelerometer data were recorded at 1.5 m/s (60 in/s) on magnetic tape and replayed through Society of Automotive Engineers J211 class 60 specification filters; the signals were recorded on oscillograph charts or directly processed by using analog-to-digital conversion.

TEST RESULTS

The results of the test series with an 18 000-km (40 000-lb), 1955-model bus are summarized below (1 km/h = 0.62 mph and 1 m = 3.3 ft).

	Test		
Item Measured	CMB-21	CMB-22	CMB-23
Impact speed, km/h	67.1	83.0	85.1
Impact angle, °	11.5	6.6	16.0
Max roll angle toward barrier,°	8	9	24
Max 50-ms avg vehicle accelerations			
Longitudinal, *g*	-0.9	-0.9	-0.8
Lateral, *g*	-0.7	-0.8	-1.0
Maximum barrier translation, m	0	0	0.8

Further details of the test program have been given by Bronstad and others (4).

Test CMB-21

The impact conditions were a speed of 67.1 km/h (41.7 mph) and an angle of 11.5°. As shown in Figure 3, the vehicle impacted the barrier 12.4 m (40.8 ft) from the upstream end of the system and was smoothly redirected with a maximum roll angle of 8° toward the barrier and a total barrier-contact length of 7.9 m (25.9 ft). The maximum 50-ms average vehicle accelerations, which were obtained from high-speed film, were -0.9 *g* (longitudinal) and -0.7 *g* (lateral). The barrier damage (Figure 4) consisted of gouging and scraping of the concrete

Figure 3. Sequential photographs of tests CMB-21 and CMB-22.

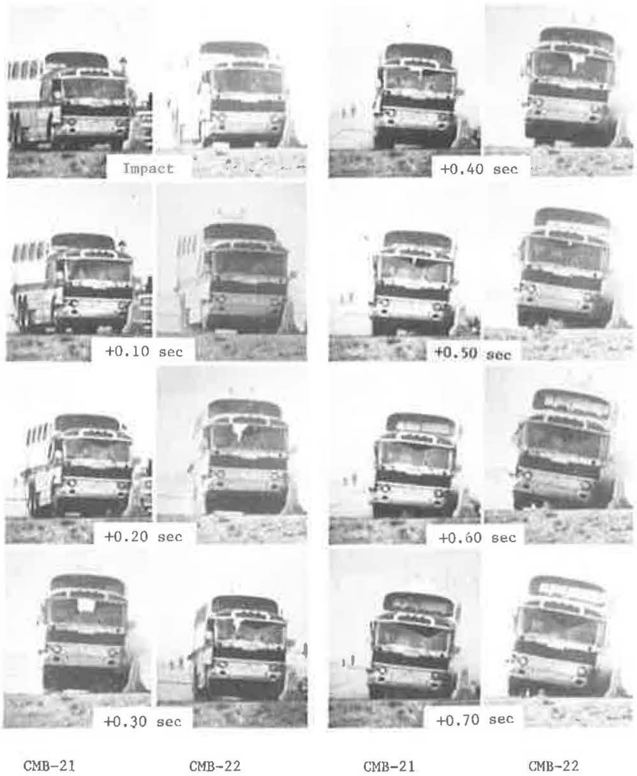


Figure 4. Damage after test CMB-21.

surface by contact with the rim; there was no translation of the barrier. The vehicle, which was drivable after the test, sustained minor front-bumper and sheet-metal damage, which caused some wheel-well intrusion on the left front corner.

Test CMB-22

The minor repairs necessary after test CMB-21 were made before this test. The impact conditions were a speed of 83.0 km/h (51.6 mph) and an angle of 6.6°. As shown in Figure 3, the vehicle impacted the barrier 23.5 m (77.1 ft) from the upstream end of the system and was smoothly redirected with a maximum roll angle

Figure 5. Damage after test CMB-22.

of 9° toward the barrier. The right front tire of the bus was airborne for approximately 0.3 s. The vehicle was redirected with a total barrier-contact length of 8.5 m (28.0 ft). The maximum 50-ms average vehicle accelerations, which were obtained from high-speed film, were -0.9 *g* (longitudinal) and -0.8 *g* (lateral). The damages to the barrier and the vehicle were similar to those in test CMB-21 (Figure 5). The vehicle was drivable after the test.

Test CMB-23

A leak in the oil system and the wheel-well damage were repaired before this test. The impact conditions were a speed of 85.1 km/h (52.9 mph) and an angle of 16°. As shown in Figure 6, the vehicle impacted the barrier 23.5 m (77.0 ft) from the upstream end of the system and was redirected with a maximum roll angle of 24° toward the barrier. The maximum 50-ms average vehicle accelerations, which were obtained from high-speed film, were -0.8 *g* (longitudinal) and -1.0 *g* (lateral). There was a local failure of the barrier and subsequent asphalt displacement because of the frontal contact; the resulting displacement was not measurable, but was estimated to be between 0 and 100 mm (0 and 4 in). The significant barrier and foundation failure was caused by the secondary impact. The maximum deflection of the barrier was 0.8 m (2.6 ft). The vehicle damage was extensive: There was major damage to the left front quadrant of the vehicle around the fender and the left side radiator and radiator door. These damages are shown in Figure 7.

DISCUSSION OF RESULTS

The following conclusions can be made from the results of the crash-test program.

1. The severe test (16°) illustrates that a standard MB5 barrier with minimum reinforcement and foundation

Figure 6. Sequential photographs of test CMB-23: front view and rear view.

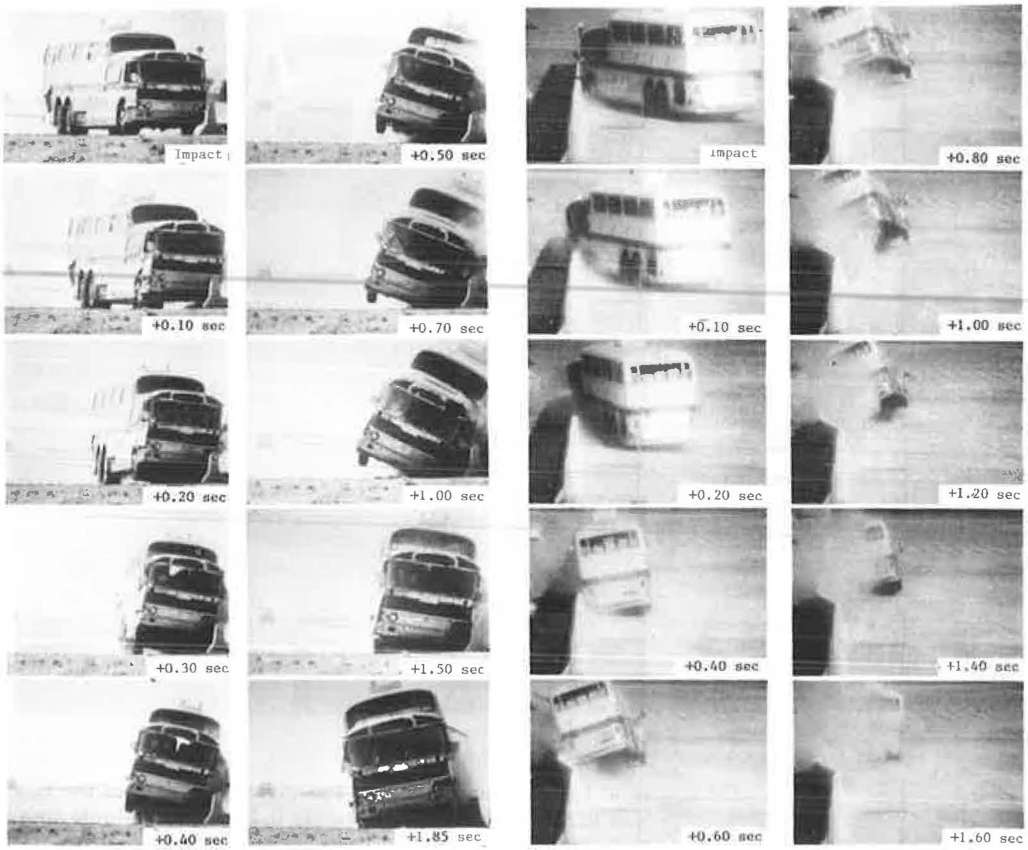
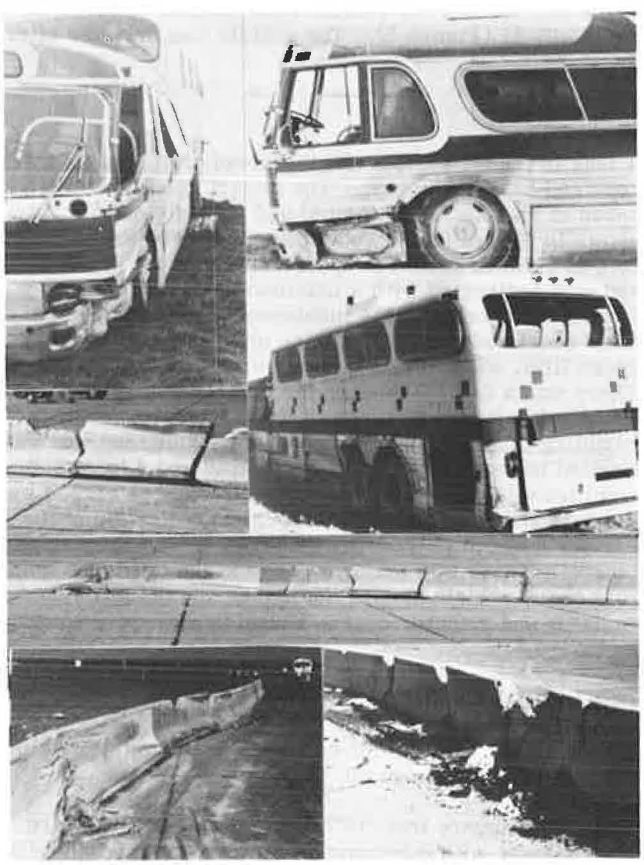


Figure 7. Damage after test CMB-23.



restraint can redirect large vehicles at high impact speeds and angles. Since the redirection of the bus occurred (i.e., the bus rotated through an angle larger than the impact angle) before the significant barrier damage and displacement, this redirection would have occurred regardless of the damage and deflection. The possibility of the bus rolling over the barrier cannot be completely dismissed, but is considered unlikely.

2. In the severe test (16°), the local fracture of the concrete occurred during the initial contact with the bus front at a load that was high enough to force the impacting left wheel against the back of the wheel well. However, although this initial contact produced local barrier failure and nominal displacement, the rear-end contact as the bus was redirected was the principal cause of the extensive barrier damage and displacement.

3. Even if a more rigid barrier had been used, it is still probable that at least local failure of the barrier wall would have occurred during the rear-end contact.

4. Barrier failure could have been changed by a foundation of sufficient embedment to produce rolling of the barrier rather than lateral translation, but this is more undesirable because it produces ramping.

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Control of Outdoor Advertising: The Georgia Experience

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Federal legislation to control outdoor advertising on Interstate highways began with the passage of the bonus act in 1958. This act granted additional Interstate-system construction funds to states enacting appropriate controls. In 1965, the broader Highway Beautification Act, which withholds funds from all states failing to adopt acceptable legislation, was passed. Georgia responded to both laws and passed outdoor-advertising control acts that were typical of those in most states. An analysis of the Georgia experience in controlling billboards is the focus of this study. It is concluded that the legislation has failed to achieve its stated objectives. Loopholes in the act have permitted extensive billboard construction. The federal insistence on the use of eminent domain, rather than police power, to remove nonconforming signs and the meager appropriations for this purpose have meant that few signs have actually been removed. Recommendations are made to more effectively control billboard proliferation and to provide signs to give the motorist information that is more compatible with protection of the visual environment.

The preamble to the Highway Beautification Act of 1965 declared:

> The erection and maintenance of outdoor advertising signs, displays, and devices in areas adjacent to the Interstate system and the primary system should be controlled to protect the public investment in such highways, to promote the safety and recreational value of public travel, and to preserve natural beauty.

How effective has this act actually been in achieving these goals? This paper examines its practical effects in a typical state—Georgia—and attempts to answer this question and to formulate recommendations that will more effectively accomplish its stated objectives.

HISTORY OF OUTDOOR-ADVERTISING CONTROL REGULATIONS IN GEORGIA

In Georgia, attempts to pass legislation controlling outdoor advertising have been lengthy and frustrating. The first was in response to the bonus act of 1958, the federal carrot that offered additional interstate construction funds to any state adopting billboard controls. This law was struck down by the Georgia Supreme Court. With the passage of the Highway Beautification Act of 1965, Congress exchanged positive for negative incentives: A state failing to pass acceptable controls lost 10 percent of its federal-aid highway funds. However, Georgia then adopted additional legislation to avoid this federal stick.

Bonus Law and the Georgia Law of 1964

Soon after construction of the Interstate highway system began in 1956, strong concerns were expressed over the need to curb the spread of outdoor advertising along this 67 200-km (42 000-mile) national network (1). In 1958, Congress passed the so-called bonus act to encourage individual states to develop control measures and provide a degree of national uniformity should they decide to do so. Any state entering into an agreement with the federal government to control advertising along the Interstate highways that was consistent with national policy would receive a bonus of 0.5 percent of the construction cost of the highway project. The act provided for control of outdoor advertising within 210 m (660 ft) of the Interstate right-of-way. It permitted four classes of signs within the controlled area: (a) directional or other official signs; (b) on-premise signs; (c) signs within 19.2 km (12 miles) of an advertised activity; and (d) signs in the specific interest of the traveling public, i.e., signs containing information about places operated by the government, natural phenomena, historic sites, and locations of eating, lodging, camping, and vehicle services.

In 1959, an amendment was adopted that prevented controls from applying to those segments of the Interstate system that traversed (a) areas that had been zoned commercial or industrial within the boundaries of incorporated municipalities as such boundaries existed on September 21, 1959, and (b) other areas in which the state had clearly established land use as industrial or commercial as of September 21, 1959.

The 1958 act did not specify the methods to be used by a state for sign removal or acquisition of advertising rights. Any state that effected control by purchase or condemnation was declared eligible for federal reimbursement on a 90:10 ratio, provided that such cost did not exceed 5 percent of the cost of the Interstate right-of-way within the project. Twenty-five states entered into bonus agreements before the expiration of the program on June 30, 1965; of these, three used the power of eminent domain to eliminate nonconforming signs, seven combined compensation for certain existing signs with police-power controls, and the remainder elected to use police-power measures only.

Georgia was the only state in the Southeast to enact a bonus law. In 1964, the Legislature passed Act 585, Section 1128, to "control the erection and maintenance of outdoor-advertising signs, displays and devices adjacent to the National System of Interstate and Defense Highways in Georgia."

The standards enumerated by the Georgia law were similar to those established by the bonus act, limiting control to within 210 m (660 ft) of the right-of-way, and permitting, within this limit, the following four classes of signs:

1. Directional and other official signs;
2. On-premise signs advertising the sale or lease of, or activities being conducted on, the property on which the signs were located (only one such sign was allowed to be visible to traffic proceeding in any one direction, or to be located more than 15 m (50 ft) from the advertised activity);
3. Signs within 19.2 km (12 miles) of the advertised activity; and
4. Signs in the specific interest of the traveling public (trade names were permitted on these only if they identified vehicle services, equipment, parts, fuels, oils, or lubricants offered for sale at places deemed of interest to the public).

Classes 3 and 4 signs were restricted to an area of 14 m^2 (150 ft^2), all signs were required to be more than 305 m (1000 ft) apart, and billboards were not allowed within 3.2 km (2 miles) of an interchange. The act did not provide for acquisition of signs under eminent domain, but required the removal of nonconforming signs after a 27-month amortization period.

In 1966, Georgia became the only state with a bonus agreement to have its outdoor-advertising control law overturned by the courts. In the case of Branch versus State Highway Department [222 Geor. 770, 771-2. 152 S.E. 2d 372, 374 (1966)], the Georgia Supreme Court declared the law unconstitutional because of the lack of a provision for payment of just compensation. Chief Justice Duckworth, in his ruling, stated,

> The enactment of this so-called outdoor-advertising control act was purely a legislative exercise in futility. Its sole purpose is to dictate, control, and limit uses of private property for public purpose, without a semblance of provision for first paying for such taking or damaging.

Cunningham (2) has commented that the flavor of the Georgia court's opinion can be gathered from the following excerpt:

> We believe that this matter is important enough to justify the following observations. Private property is the antithesis of Socialism or Communism. Indeed, it is an insuperable barrier to the establishment of either collective system of government. Too often, as in this case, the desire of the average citizen to secure the blessings of a good thing like beautification of our highways, and their safety, blinds them to a consideration of the property owner's rights to be saved from harm by even the government. The thoughtless, the irresponsible, and the misguided will likely say that this court has blocked the effort to beautify and render our highways safer. But the actual truth is that we have only protected constitutional rights by condemning the unconstitutional method to attain such desirable ends, and to emphasize that there is a perfect constitutional way which must be employed for that purpose....

and has concluded that the opinion lacks any analysis of the problem of nonconforming uses and is singularly unpersuasive.

The court's only argument was with the failure of the state to acquire billboards under eminent domain rather than by police power, but the entire act was overturned because the General Assembly had neglected to include a severability clause. Such a clause would have upheld the rest of the law in the event of one section or clause being judged unconstitutional. This decision cost Georgia not only the additional funds it would have received under its bonus agreement with the Secretary of Transportation, but also the cost of removing billboards that were erected between the invalidation of the 1964 act and the effective date of the 1971 act.

1965 Highway Beautification Act and the Georgia Response

Some revision of the Georgia act would have been required even without the unfavorable court ruling, to allow the state to comply with the Highway Beautification Act of 1965, which contained major differences from the 1958 bonus law. The clearest distinctions were the added condition that any state that failed to provide for the effective control of outdoor advertising within the 210-m (660-ft) limit would lose 10 percent of its federal-aid highway funds and the extension of outdoor-advertising controls to the federal-aid primary system. Compensation payments for sign elimination were made mandatory rather than discretionary, and the federal share of such compensation was set at 75 percent. A significant feature of the act was that all commercial and industrial zones were now recognized as exclusions, which eliminated the 1959 cutoff date. The act also provided for an unzoned commercial or industrial area, to be defined by agreement between the states and the Secretary of Transportation. All permitted signs, including directional and other official signs, were made subject to size, lighting, and space requirements. On-premise signs were exempted from all controls.

Georgia had taken immediate steps to comply with the Highway Beautification Act of 1965 by passing a constitutional amendment, ratified in 1966, that allows the state to acquire billboards and junkyards under the power of eminent domain. After the ratification of the amendment, the General Assembly passed a new Outdoor-Advertising Control Act (Vol. 1, No. 271, Secs. 1, 3; 423) in 1967. The law encompassed the Interstate and primary systems and maintained the 210-m (660-ft) control zone. The following signs were permitted within the limit:

1. Directional and other official signs,
2. On-premise signs,
3. Signs located in areas that are zoned commercial or industrial under the authority of law, and
4. Signs in a business area adjacent to an incorporated municipality (unless in conflict with local zoning laws).

There were no stipulations concerning the maximum number of signs permitted per unit distance, nor were any minimum size regulations established, except that of customary use in the outdoor-advertising industry within the state. The act did, however, contain several spacing requirements; e.g.,

1. In a business area located inside the limits of a municipality, no sign shall be within 46 m (150 ft) of another on the same side of the highway unless separated by a structure or roadway;
2. In a business area on the primary system or

within the approaches to a municipality, no sign shall be within 91 m (300 ft) of another on the same side of the highway;

3. In a business area on the Interstate system, no sign shall be within 152 m (500 ft) of another on the same side of the highway; and

4. All signs in an unzoned area, i.e., an area occupied by one or more commercial or industrial activities, shall be within 1067 m (3500 ft) of the boundary line of the property on which the activity is located and may be on either side of the road, on the Interstate system. On the primary system, the distance is limited to 640 m (2100 ft).

These spacing requirements seem stringent at first glance, but a typical example changes this impression: One small country store located on a secondary road and adjacent to an Interstate highway can result in the permitting of billboards within 1067 m (3500 ft) on each side of the property and on both sides of the highway; this constitutes a potential 28 billboard sites and there are no regulations as to the maximum size or number per site. A more readily perceived result was the opening of rural areas of the Interstate system to billboards. Since most interchanges have at least one commercial establishment, this allows sign boards for at least 1 km (0.62 mile) on each side of the interchanges.

The 1967 law also provided for the acquisition of signs through compensation, authorizing the State Highway Department to exercise the power of eminent domain. This power was upheld in Burnham versus State Highway Department [224 Geor. 543, 163 S.E. 2d 698 (1968)], which ruled that the 1966 constitutional amendment allowing payment of compensation for junkyards was constitutional, and thereby simultaneously established the validity of using eminent domain for the control of billboards.

The 1966 amendment was again the subject of litigation in National Advertising Company versus State Highway Department [230 Geor. 119, 195 S.E. 2d 895 (1973)], in which the court upheld the constitutionality of the statute providing that no outdoor advertising shall be erected or maintained within 210 m (660 ft) of the nearest edge of the right-of-way of the Interstate or primary highway system. This decision was based on the police power of the state to zone property for future use, as distinguished from taking or damaging in respect to use already in existence; no compensation payments are required. According to Justice Jordan,

As we view the present case, what is really involved is nothing more than the exercise of specific police powers as clearly authorized by the 1966 amendment to the Constitution of Georgia . . . not with respect to eminent domain and payment of just and adequate compensation before taking private property for public purposes, but with respect only to the exercise of zoning powers, to prevent in the future . . . the erection and maintenance of certain advertising devices within a certain distance of the right-of-way of certain highways in certain areas.

Although the 1967 Georgia act was upheld by the courts, it failed to meet the requirements of the federal government. The Secretary of Transportation declared that the Georgia law did not comply with the directives of the Highway Beautification Act of 1965, and threatened to withhold 10 percent of Georgia's federal-aid highway funds. This warning prompted the state to once again revise its billboard-control regulations and resulted in the passage of the Outdoor-Advertising Control Law of 1971 (Code 95A, Art. IV, Secs 913-934).

The primary differences between the 1967 and the 1971 laws were the adoption of maximum size and minimum spacing requirements and a more stringent definition of what an unzoned commercial and industrial area is. The permissible distance for the establishment of signs in these areas was reduced from 1067 m (3500 ft) on either side of the property line of a commercial or industrial use to 183 m (600 ft) on either side of such a structure and, on the Interstate system, signs were permitted only on the same side of the road as the business activity.

The obvious deficiency that marked all of the Georgia billboard legislation up to this point was unaltered in the 1973 act: This is the arbitrary 210-m (660-ft) control zone, which gave rise to the phenomena of the jumbo sign located just outside the control limit. There are now over 250 of these jumbo signs along the Interstate highways in Georgia, and the cost of their removal is estimated to be in excess of $3 million. A 1974 amendment to the federal Highway Beautification Act of 1965 has attempted to remedy this situation by changing the arbitrary 210-m (660-ft) limit to that of signs visible from the roadway, and the Georgia law has been amended to conform to the new federal standards. However, because of the compensation feature, it was necessary to pass a constitutional amendment, and during the period between the passage of the legislation and the ratification of the constitutional amendment by the voters, outdoor advertising firms continued to erect jumbos.

CONTROL OF OUTDOOR ADVERTISING IN GEORGIA IN PRACTICE

How effective has the Georgia Outdoor-Advertising Control Law been in controlling the erection of new billboards and in removing billboards that are nonconforming? Unfortunately, the law has been ineffective in both respects.

Control of New Billboards

Size and Spacing Requirements

The 1968 amendments to the Highway Beautification Act directed the Secretary of Transportation to accept size and spacing limitations that were customary in the state. Thirty-two states, including Georgia, adopted a maximum size limitation of 112 m^2 (1200 ft^2). To put this in perspective, the industry's standard poster panel and the largest painted bulletin normally used on the Interstate system have areas of 38 and 63 m^2 (300 and 672 ft^2) respectively.

Customary spacing in Georgia, as in almost all states, was defined as every 152 m (500 ft) on the Interstate system, 91 m (300 ft) on the primary system, and 30.5 m (100 ft) on the primary system within municipalities. The numbers of signs and sign faces that this spacing permits per kilometer of highway are summarized below (1 km = 0.62 mile).

Category of Highway	Signs	Sign Faces
Interstate in commercial or industrial zone	13	26
Primary outside of municipality	22	44
Primary within municipality	66	132

This means that, if each of the signs on a primary highway within a municipality is the maximum allowable size [112 m^2 (1200 ft^2)/site], the total area of the sign faces will equal approximately 2 football fields/km of roadway.

Commercial and Industrial Zones

Since it is obvious that the size and spacing requirements constitute virtually no control of outdoor advertising, the designation of commercial and industrial areas becomes all important. Under the 1959 amendments to the bonus law, billboards were not controlled in areas that were designated commercial and industrial zones of municipalities as of September 21, 1959. The Highway Beautification Act of 1965 extended this exclusion of controls (except for spacing and size limitations) to include all commercial and industrial areas, either zoned or unzoned. In the 1968 amendments the Secretary of Transportation was also directed to accept state and local determination of zoning for this purpose.

Local zoning authorities do not often consider that providing an uncluttered view for the Interstate motorist is of great importance. The real or imagined benefits to be derived by local businesses from billboard advertising usually have much greater priorities. In practice, many local communities, particularly rural counties, attempt to circumvent the intent of the highway-beautification law by zoning long stretches of highways within their borders as commercial and industrial.

Anticipatory Commercial and Industrial Zones

Under generally accepted land use planning techniques, lands are zoned not only for the present, but also for anticipated development. This practice results in allowing billboards in vast areas of rural countryside. For example, on Interstate 85 in Gwinett County near Atlanta, an industrial zone extends approximately 6.5 km (4 miles) beyond the currently developed industrial area. On this length of highway, there are 51 billboard sites, only 4 of which would satisfy the unzoned-commercial-zone criteria by being actually located near an industrial land use.

False Commercial and Industrial Zones

Other counties have also designated areas as commercial and industrial zones primarily to allow billboards. The most flagrant example in Georgia may be that of largely rural Columbia County near Augusta, which has zoned its entire 42 km (26 miles) of Interstate 20 as industrial. Other rural counties have zoned large areas on either side of interchanges as commercial zones, even when commercial activity is confined almost exclusively to the areas immediately adjacent to the interchange. For example, Jackson County, which contains 32 km (20 miles) of Interstate 85, has zoned 19 km (12 miles), 2.4 (1.5) on each side of 4 interchanges, as commercial.

Other counties have designated as agricultural and commercial zones areas along Interstate highways in which billboards are almost the only permitted commercial use. Morgan County has designated an agricultural and commercial Interstate zoning district that allows agriculture, residences, agricultural-related businesses, motels and trailer parks, restaurants, gasoline stations, and billboards. Glynn County has a forest and agriculture district that permits billboards.

Thus far, the Georgia Transportation Board has not accepted these and similar examples as true zoning for the purpose of the Outdoor-Advertising Control Act. If this type of zoning is accepted by the board, however, and there is considerable political pressure to do so, then any effective control of billboards in the rural areas of Georgia will be lost.

The main reason that the Georgia Transportation Board has been reluctant to accept false commercial and industrial zoning is the lack of acceptance of such zoning by the U.S. Department of Transportation. The 1968 amendments to the Highway Beautification Act (3) stated that

> The states shall have full authority under their own zoning laws to zone areas for commercial or industrial purposes, and the actions of the states in this regard will be accepted for the purpose of this act.

Nevertheless, the Federal Highway Administration has taken the position (4) that zoning

> which is not a part of comprehensive zoning and is created primarily to permit outdoor advertising structures, is not recognized as zoning for outdoor advertising control purposes.

They further state that

> A zone in which limited commercial or industrial activities are permitted as an incident to other primary land uses is not considered to be a commercial or industrial zone for outdoor advertising control purposes.

This position has been tested and upheld in the courts [State of South Dakota versus Volpe, U.S.D.C., S.D., Civ. 72-4024 (1973)].

Unzoned Commercial and Industrial Areas

The other kind of area in which billboards may be legally permitted even though the predominant land use is not truly commercial or industrial is that of the unzoned commercial and industrial area. Since even the most obscure commercial or industrial use can serve to designate such an area, this section of the law definitely allows billboards to be legally erected in predominantly rural areas. For example, small family businesses that happen to back up to an Interstate highway, and that often cannot even be seen from the highway, can permit the erection of several large billboards. Ironically, junkyards in rural areas that are screened and controlled under the Highway Beautification Act have also served as the justification for permitting billboards.

The lengths to which some advertising companies will go to use this loophole are fascinating. On Interstate 75 between Atlanta and Chattanooga, a property owner erected a shed having an area of approximately 14 m^2 (150 ft^2) in a rural area and affixed a small sign designating it as a warehouse. A 63-m^2 (672-ft^2) billboard was then erected next to this warehouse, and the outdoor-advertising firm then applied for a permit based on the area being an unzoned industrial area. The Georgia Department of Transportation refused to accept this obvious maneuver, however, and eventually the sign was removed.

Removal of Nonconforming Billboards

The effectiveness of the Highway Beautification Act in removing nonconforming Georgia billboards is summarized in a report issued by the Georgia Department of Transportation (5). From the inception of the outdoor-advertising control program, only 431 signs had been acquired at a total cost of $494 430, of which $377 152 were federal funds. Over 20 000 nonconforming signboards remained on Georgia's Interstate and federal-aid primary systems. An additional $3.5 million of federal highway-beautification funds have been allocated to the state for sign removal, however, and careful concentration of these and other funds could restore the scenic quality of significant stretches of highways.

CONCLUSIONS AND RECOMMENDATIONS

The Georgia outdoor-advertising control law meets all the requirements of the Highway Beautification Act of 1965 as amended, and the Georgia Department of Transportation is now administering the law diligently and vigorously. Even so, the declared goals of the act are definitely not being achieved in Georgia nor in the overwhelming majority of the states.

Recommended Legislative Revisions

The present definition of commercial and industrial areas does not effectively limit billboards to placement in true commercial and industrial areas where they are a compatible activity. This is particularly true in unzoned commercial and industrial areas.

One way to close this obvious loophole in the law would be to require a dual test before permitting billboards in areas that are zoned commercial and industrial. Under these revised criteria, not only would the billboard have to be in an area zoned as commercial and industrial, but it would also have to be located near substantial development of this type. This revision would largely eliminate the problem of false or anticipatory zoning. Similarly, the criteria for an unzoned commercial and industrial area should be revised to limit billboards to unzoned areas that are truly industrial and commercial in character and not designated as such merely on the basis of a single incompatible land use.

There should also be a limit to the number of signs allowed per unit distance within commercial and industrial zones, and the size and spacing requirements should be revised to recognize that there are different types of highways and areas. For example, a large billboard that would not be out of place in a heavily industrialized area along a freeway would be inappropriate in a small commercial area along a conventional two-lane highway or city street, but the present law makes no distinction between the two kinds of areas. The adoption of these modest recommendations would not result in the complete elimination of billboard clutter, but would eliminate many of the present abuses and tend to confine billboards to areas where they are a reasonably compatible land use.

Need for Informational Signing

Under the provisions of the Highway Beautification Act, all nonconforming signs were to be removed by July 1, 1970. However, the amendment of the act that requires payment of just compensation and the meager Congressional appropriations for highway-beautification purposes have combined to postpone this final-removal date indefinitely. In Georgia, only 2 percent of the nonconforming signs have been removed thus far.

The funds available to Georgia for sign removal can be used best by concentrating on the Interstate system in rural areas. Since many motorists desire information about available services, these removals should be combined with a system of informational signs on the right-of-way. This type of sign is specifically provided for in the Highway Beautification Act and has already received extensive testing, including that at an interchange on Interstate 95 in Georgia (3). Although this program is opposed by the outdoor-advertising companies, it can adequately meet the need of the motorist for information in a manner that will not destroy the natural beauty of the rural countryside.

Congress also seems to be very concerned about providing information for motorists. The Federal-Aid Highway Act of 1976 amends the beautification law to specifically permit information signs within the right-of-way of primary highways and to permit the retention, in specific areas, of nonconforming billboards that give directional information, if a state demonstrates that removal would work a substantial economic hardship in such defined area. The practical effect of this amendment may possibly be to destroy the billboard-control program unless alternative motorist-information services are provided.

On-Premise Signs

The Highway Beautification Act makes no attempt to control on-premise signs, the Congress feeling that this is more properly a function of local government. Few local governments have met this challenge, however, and the sizes and heights of on-premise signs, particularly those on high-rise pylons at Interstate-highway interchanges, have steadily increased.

Ironically, this inaction on the part of local government is partially due to the highway-beautification program. The main problem was the question of the use of police power to control signs and require their removal through amortization. Fortunately, in the recent case of City of Doraville versus Turner Communications [236 Geor. 385, 223 S.E. 2d 798 (1976)], the Georgia Supreme Court appears to have joined a growing body of legal opinion cited by Dobrow (6), when it ruled that

1. Municipalities have the right to enact and enforce sign-control ordinances,
2. These ordinances can be more restrictive than the state outdoor-advertising control act, and
3. The two-year amortization period in the Doraville ordinance is reasonable and does not result in a taking of private property.

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