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A REVIEW AND EVALUATION OF BRIDGE BARRIER DESIGN AND EXPERIMENT PROCEDURES

BY

COLIN B. BROWN

Final Report to the Sponsors: Division of Highways, Department of Public Works, State of California, and the Bureau of Public Roads.

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THE DEPARTMENT OF PUBLIC WORKS
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PREFACE

This report is concerned with the present state of knowledge of the loads on and response of bridge barriers, which, when organized, suggests possible design criteria and methods for comparative evaluations of various material and configurational types. These objects meant that the library study was not restricted to actual information on bridge barriers, but also encompassed related fields such as traffic engineering and missile testing. The bibliography includes all the publications directly referred to in the report together with other pertinent work. It is arranged alphabetically under the name of the senior author and anonymous notes by various public authorities are grouped at its beginning. In the text references are indicated by the authors' names and the year of publication, i.e. (Grime and Newby, 1961), (-------, 1964.a).

The report is largely based on the library search conducted by Mr. J. J. Reilly during the summer of 1964 whilst completing his graduate studies at the University of California at Berkeley.

INTRODUCTION

Barriers along the edge of roadways may be regarded as being of two types; either they serve as warning devices only or they serve as a positive device which ensures that an impinging vehicle neither breaks through the barrier nor is reflected back across its own roadway. Certainly the features of the first type of barrier may be incorporated in the second type with the result that the opague road edging restricts the transverse view. On bridges it is often considered advisable to have a safe edge barrier which clearly demonstrates the boundary of the roadway and yet allows appreciation of the view. Such a compromise is seldom attained. Here interest is focussed on a positive barrier structure intended to confine a vehicle striking it to the bridge and to return the vehicle to a course on the roadway parallel and adjacent to the rail. The reasons for these requirements arise purely from our social ideas of safety, inasmuch as a vehicle falling from an overpass onto the highway, crashing through a central reservation or being returned across its own roadway could result in serious accidents involving innocent traffic. In this respect no regard is evident concerning the safety of the impinging vehicle but a secondary purpose of the barrier is to minimize damage to this vehicle and its occupants. Thus an effective barrier must be strong enough to withstand fracture, stiff enough to maintain a correct configuration to guide a vehicle properly back into the road, and together with the vehicle possess resilience and damping capacity to help protect the vehicle occupants. These requirements are seldom met in a barrier and priority must be given to protecting innocent travellers.

Full-scale tests on barriers have not been solely concerned with those used on the edge of bridges but have had the purpose of giving information on the response and strength of all types of highway barriers. From this information must be extracted those facets applicable to bridges. Clearly, the lateral deflections of a struck barrier must be more restrictive for a bridge or canyon road than for a normal highway. Because of this the use of that class of barrier which slows and eventually stops a vehicle over a considerable lateral distance is seldom possible. In particular the types of energy absorbing barriers consisting of multiflora rose hedges (Skelton, 1958; Zurcher, 1960) and breakable bollards (Pedersen, Mathewson and Severy, 1958), although satisfying our secondary requirement, are not suitable from the primary viewpoint of a bridge engineer. Bridge barriers are usually of metals, timber or reinforced concrete. The loads of impact may be transmitted by bending of members between posts or by membrane strains occasioned by lateral deformations and reacted by longitudinal tie-backs. In addition, all or part of a barrier may consist of a wall which cantilevers from the pavement level. In all these cases, the load has first to be resisted by the superstructure elements and then transmitted by post, tie-back or wall to the main bridge. Under these circumstances a barrier may be considered as structural elements, with inter-connecting parts, which eventually transmit the loads to an adequate foundation. From this viewpoint the parts of a barrier may be idealized in the usual manner of structural design and the professional questions concerning strength, stability, service and safety posed. An additional feature associated with the barrier which effects the nature of the loading but not the barrier capacity is the kerb. The location of this on the highway side of the barrier may cause the vehicle to move vertically and thus effect the impact height. This important part of the impact loading has been investigated and the part played by kerb height, location and shape determined.

The loads impinging on a bridge barrier require definition of the characteristics of the striking vehicle. Essentially the vehicle velocity vector, the location of its center of gravity, its weight, its resilience and the contact height relative to the barrier must be known. In addition, the design loading is not only a function of the vehicle but also of the barrier rigidity, strength and resilience. Already mentioned has been the effect of the kerb on the height of impact, the remaining features require traffic information and some reasonable analytical treatment of it in order to postulate some type of design loading.

These descriptions of the barriers and the loading variables indicate the type of information necessary to evolve a reasonable design method for bridge barriers. The response characteristics of a barrier to loading must be understood and be predictable analytically. The load characteristics must be understood and sufficient data should be available to make a decision on a specification for design loading. In fact neither the response nor the loading is at the moment clearly comprehended and it is still necessary to resort to full-scale testing in order to compare the response of various barrier types and to propound static forces for design loading. This report is divided into three parts which consider

a) the response of barriers
b) the loading on barriers
c) suggestions concerning future steps to improve the understanding of a) and b).

The first part deals with the evidence obtained through model and full-scale tests, analytical work and experiments on the elements of the barrier. The second part considers bounds on the angle of attack of an impinging vehicle, vehicle characteristics and velocities. In the final part the suggestions make use of what is known and attempt to suggest improvements in test methods and analytical approaches.

BARRIER RESPONSE

An understanding of the response of a bridge barrier to an impacting load may be obtained by considering the work done with models and with full-scale tests together with any analytical work that is available. Essentially, the results of experimental and analytical model studies, to be fruitful, must be corroborated by full-scale information. In a like manner the model development must be guided by existing field information. In this part the three methods of considering barrier response are dealt with under separate headings but the interdependence of the sections will be evident and these are summarized in the discussions.

MODEL TEST

The most complete descriptions of model tests are given by Ayre and Abrams, 1954, 1955, Ayre and Hilger, 1956, and Ayre, Abrams and Hilger, 1955. Takahashi, 1960, 1961 also carried out some model experiments which were purely preliminary to his effort to determine the maximum collision speed of two vehicle types impacting at 20° on a barrier for a prescribed allowable lateral deflection to occur. The work of Ayre et.al. was not extended to full-scale tests as originally planned but the results of the model experiments are of great interest in as much as many rail variables are introduced over a wide range of conditions.

Design of Experiments. In order to scale the experiments proper similitude relationships are developed from the three non-dimensional equations of motion of the impacting vehicle onto a cable guardrail which accepts only axial strain. These relationships are for time scale and velocity scale in terms of the length scale,

$$\left(\frac{t_p}{t_m}\right)^2 = \left(\frac{V_p}{V_m}\right)^2 = \frac{\ell_p}{\ell_m}.$$

From the stiffness scale a measure of force and hence the weight scale is obtained,

$$\frac{(A \cdot E)_p}{(A \cdot E)_m} = \frac{f_p}{f_m} = \frac{W_p}{W_m}.$$

Thus an arbitrary selection of the length scale results in a translational velocity and time scale. Similarly the selection of the model cable stiffness determines the weight of the model vehicle. The above relationships are for a cable type of guardrail and the report largely involves only this type. In the tests on rails which bend as beams the same length and force scales were used and the bending stiffness scale resulted from considering a beam loaded at its center. Hence

$$\frac{(EI)_p}{(EI)_m} = \frac{f_p}{f_m} \cdot \left(\frac{\ell_p}{\ell_m}\right)^2.$$

Even though the experiments were conducted in a horizontal plane it was still necessary for the model vehicle to have its vertical location of the center of gravity and its radii of gyration about the horizontal axes correspond to the real vehicle in the manner defined by the length scale. This would be evident from the essential variables of the loading on the barrier discussed in the Introduction.

The experiments at Johns Hopkins University were designed to illustrate the effects of the following in a cable like barrier:

1) speed of impact
2) angle of impact
3) ground friction with vehicle
4) post spacing
5) point of impact relative to posts
6) initial cable tension
7) rigid or yielding posts
8) rigid or yielding tie-back anchorages
9) cable flexibility (use of tension spring)

In the more limited tests on beam like barriers only the speed and angle of impact were varied. The measured quantities in the experiments associated with these variables were:

i) vehicle path and velocity

ii) dynamic increase in cable tension.

Clearly knowledge of the impact and reflected vehicle velocity vector from (i) leads to an estimate of the energy dissipation in the crash.

Apparatus. In order to determine the effects of the variables listed the vehicle velocity vector must be established throughout the whole motion. A suitable way of carrying out this is to photograph the rail region during the whole impact. This requires a stroboscopic light source which illuminates the head of the vehicle and thus plots on the exposed negative the vehicle course. Knowledge of the flash frequency allows measurement on the negative of the distance covered between flashes and hence the local velocity.

The vehicle must be designed to allow weight redistribution, varying wheel friction and certain material yielding features at the head. In addition it would be preferrable to have some form of driving and braking force system. The work reported at Johns Hopkins provided weight redistribution and variable wheel friction facilities only. The first point was covered by using a plywood model frame with duraluminum weight carried on it. Friction differences were provided by using different rolling surfaces for the ball bearing wheels. Thus using glass $\mu = 0{\cdot}18$, masonite $\mu = 0{\cdot}33$ and rubber $\mu = 0{\cdot}70$ and wide ranges of road conditions were simulated.

The dynamic effects on the barrier itself may be measured by direct photography at impact to show the dimensional changes in the configuration and by strain gage readings on the cable. These readings can be displayed on a recording oscillograph after amplification.

Experimental Procedures. The nine variables listed above were incorporated in the Johns Hopkins tests as follows.

1) V_0 = 10, 20, 30, 40, 50 m.p.h.

2) α = 15, $22\frac{1}{2}$, 30°

3) μ = 0·18, 0·33, 0·70

4) d = 16, 24 ft.

5) at center and 3/4 d from post on reflection side

6) $T_0 = 0, 4, 8, 12$ k and slack.

The final three variables were allowed for by mechanical devices which ensured that the energy dissipation in impact was confined to that connected with the cable-vehicle friction.

Results. Although most of the work on models has been on cable like barriers the success of the investigations in displaying the interrelationship of design variables and also giving quantitative information would indicate that similar, valuable results for beam barriers could be cheaply obtained. Some of the conclusions from these cable tests are presented because they may be equally valid for bridge beam barriers. Indeed, the experimenters thought that many of the cable results were applicable to beam barriers.

The reflection angle of impinging vehicles was not completely predictable. For all but the highest speeds this angle is greatly reduced as the friction coefficient between wheels and road is increased. When the posts yield substantially then the reflection angle decreases. In a similar manner, the exit velocity decreases with increasing road friction, pi, and with post yielding. In general, as expected, the lowest final velocity was associated with the structural arrangement with the largest number of energy absorbing components. The few tests carried out on beam type barriers indicate that both the angle of reflection and final speed are larger than those for the cable type. This would appear to be due to the smaller energy dissipation on the beam-vehicle contact region. The spacing of the posts, d, has little effect on the final speed in cable barriers but does have a great effect on the angle of reflection (although this effect is not predictable). Additional information obtained about the exit velocity vector is that the velocity decreases as the impact angle increases in a roughly linear manner and that the reflection angle increases with impact angle and velocity. As the distance from the first post ahead of the impact point increases the final velocity increases. The influence of the post to impact point position has an erratic but influential effect on the reflection angle. Finally, any increase in initial tension in the cable causes decrease in the reflection angle. These effects of the variables of the model test on the final or exit velocity vector are summarized in Table 1.

Variable	Effect On Vehicle Exit Velocity Vector	
	Reflection Angle Decreases	Final Velocity Decreases
Wheel-road friction coefficient increases	X	X
Posts yield	X	X
Impact angle increases		X
Impact angle decreases	X	
Impact velocity decreases	X	
Increase in initial cable tension	X	
Distance to first post ahead of impact decreases		X

Table 1 - Effect on Vehicle Exit Velocity Vector by Variables in a Cable Type Model Barrier.

Information concerning the response of the cable barrier was also obtained from the model tests. In this case the dynamic uni-axial strains in the cable were of importance and easily measurable. The maximum dynamic increment of cable tension tends to increase more or less linearly with the increase of the impact velocity and non-linearly with increase of impact angle. As an example of this last point, an angle increase from 15° to $22\frac{1}{2}^{\circ}$ resulted in a doubling of cable tension increment, whereas an increase from $22\frac{1}{2}^{\circ}$ to 30° had little effect on the tension increment. The initial tension in the cable and the post spacing had little effect upon the dynamic increment of cable tension but the position of impact relative to the first post ahead on the reflection side has importance. As this distance is increased then an increase in the increment of cable tension occurs. However, the closer the impact point to the post the greater the likelihood of a second impact. The post itself should be designed to be offset from the cable in order that "snagging" of the impinging vehicle is avoided. From a comparative viewpoint model tests show that a maximum increase of cable tension during the collision must be associated with yielding posts and minimum cable flexibility (absence of a tension spring between tie-back and cable). The presence of a tension spring minimizes the tension increase in the cable.

The secondary purpose of a bridge barrier is to protect the occupants of the crashing vehicle. A useful feature in this respect is the energy dissipation of the vehicle and barrier structure over the impact period. In model tests the knowledge of the impact and reflecting velocities allows a determination of the energy dissipated in impact. In addition, information concerning the parts of the structure and arrangements of these parts for optimum damping can be obtained. The energy dissipation in friction between the cable and the vehicle can be estimated by finding the coefficient of friction and the mean normal force on the deflected cable during the collision. The product of this mean, the coefficient of friction and the sliding distance gives an estimate of the energy dissipated. An estimate of the dissipation between tires and roadway can be made using the known coefficient of friction, the wheel reactions and the vehicle rotation and translation during impact. Finally, the damping of mechanical systems such as rotating posts, shifting tie-backs and vehicle crumbling can be approximated in models. Seldom will a true balance between that computed from the impact and reflection velocities and that determined from the individual structural parts be possible. However, some valuable information concerning this phenomenon may be obtained. Ayre et.al., using some of the methods outlined did not get a proper balance, but, they were able to show that the vast majority of the damping is concerned with the friction between vehicle and cable, and that this will be greatest in the least rigid guardrail system. Additionally they were able to show that the energy loss increases with an increasing impact angle and with increasing impact velocity.

Discussion. The lengthy review of model tests given is not justified by the use that has been made of them up to the present time. It has been the intention here to show the type of information that can be obtained from such tests which should be helpful in design and comparative evaluation of different types of barriers. In model tests considerable idealization occurs and yet it is because of this that the critical variables can be isolated. With full-scale tests the number of parameters occurring may ensure that basic phenomena are obscured because of the extensive coupling. Further the statistic approach possible in models cannot be obtained with the limited number of impacts with real vehicles. This is not meant to suggest that further development of bridge barriers should be controlled by model work, but it is believed that a competent program would involve a combination of field and laboratory experimentation. Under these circumstances, the general features in the response of the barrier and vehicle can be pin-pointed by inexpensive model work, where the whole effort can be controlled and the features confirmed by many repetitions of the event. The full-scale field tests could be used to demonstrate particular characteristics indicated in the model work and also the field work must be used for events which are not illustrated in the laboratory. For instance, stiff barrier systems with high velocity impact, which are the subject of the State of California full-scale tests, could not be easily modelled in the laboratory. Similarly, the energy dissipation from material damping and joint damping can never be successfully emulated by models. These features must be investigated fundamentally by full-scale laboratory tests, studies in tribo-physics and field crash tests. With a competent-model program the manner of full-scale field tests can be controlled and the combined results should lead to the development of an analysis which properly describes the interesting phenomena. It is also possible that model tests could illustrate conditions which could improve bridge barriers. The insight which the work of Ayre and his colleagues at Johns Hopkins gives into the response of cable like barriers should indicate the usefulness of a similar program on beam barriers.

FULL SCALE TESTS

As suggested previously, full scale crash experiments should be made in conjunction with model tests. This type of program has not been undertaken except by Takahashi in his work on maximum speeds to cause a given deflection in a barrier. At the Cornell Aeronautical Laboratory full scale tests have been used as a check as well as a help in providing an approach to the analysis of impacted barriers (-------, 1963). Most other full scale tests were designed either to answer some important questions concerning configuration, kerb effects and vehicle performance or to establish the adequacy of a manufacturer's finished product. The number of tests conducted is legion yet comparison of fundamental response information is often difficult because of the different test environments. However, comparative information is found in the reports of the tests carried out by the State of California over a period of many years. (Beaton, 1956; -----, 1957; -----, 1959; -----, 1960; Nordlin, Field and Hackett, 1964). These tests are subsequently referred to by the letter C and the year of the report (C, 1957). As well as these state reports the California work has been reported in the technical literature. (Beaton and Field, 1960; Beaton, Field, Moskowitz, 1962; Moskowitz, Schaefer, 1960). The tests on aluminum rails have been carried out by Lehigh University. (-----, 1963a; -----, 1963b). In Canada eight types of guide rails were tested with regard to an elevated highway in Montreal. (Hénault, 1963). Jehu, 1964, studied the problems of kerbs and fences in Great Britain (-----, 1964) and provided an interesting review of tests on American, Belgium, Danish and German rails. The Danish kerb barrier (Dansk Auto Vaern, D.A.V., -----, 1958) is widely used in Europe, The Soviet Union and Japan and was apparently developed by a Danish builder and his wife with simple plaster models constructed in the kitchen. (Boesgaard, 1952). Takahashi, 1960 tested these barriers in Japan and similar tests occurred in Sweden. (-----, 1955). Further recent tests of barriers in this country have been carried out by Cichowski, Skeels and Hawkins, 1961; Lundstrom and Skeels, 1959. An excellent review of full scale testing of various vehicle crash barriers is given by Irving, 1962.

Full scale tests on crash barriers have been made since the early twenties. A full coverage of the work from 1924 to 1941 is given in the Report of the Committee on Highway Guards, 1941. In that period two publications are of note, Barnett, 1939, together with the accompanying discussion is of interest and Slack, 1934, provides a graphic account of not only full scale tests but their application, together with laboratory tests on members, to the design problem. The concept of full scale testing has changed little from the early days. Essentially a vehicle is moved into a barrier and the response of barrier and vehicle is noted. Vehicles used include trucks and buses, but most of the tests are carried out with standard sedan passenger cars. The method of control and the instrumentation have become more sophisticated in recent years. These improvements have allowed the impact velocity vector to become more severe. Slack writes of speeds of impact of 15 to 20 m.p.h. with impact angles of 20° (some tests at 40°). In the C,1964 tests velocities of 75 m.p.h. were accomplished with impact angles of 25°. However Brickman, in the discussion of Barnett's 1939 paper, mentions tests on cable barriers at 20° impact angle and speeds of 41 to 85 m.p.h. Full results are only given for an impact of 41.7 m.p.h. and the remainder calculated in terms of the kinetic energy at impact. In this report the results of contemporary experiments only will be reviewed.

As before stated, it is not always easy to compare one set of tests with another. Varying test environments do not help but in addition the objectives of the tests, and hence the data obtained and the care in controlling all variables, often differ. In order to get this into perspective it is first necessary to examine these objectives before going on to the design of experiments, apparatus and experimental procedures, and then results. Here an attempt is made to synthesize the important recent tests carried out in this manner.

Objectives of Tests. These fall into three basic groups:

a) determination of barrier capacity and response
b) determination of the impact velocity vector
c) determination of the damage to cars and occupants.

The tests by the Cornell Aeronautical Laboratories combined the first two objectives in an attempt to check the assumptions made in their analysis and also to give insight into the selection of assumptions. The tests on aluminum guard and bridge rails were intended to demonstrate that barriers of this material fulfilled conditions in A.A.S.H.O., B.P.R. and revised A.A.S.H.O. specifications.* The bridge rail tests in addition expose the competency of these specifications and add material of basic interest. The tests by Beaton, Field and Moskowitz, 1962, resulted from the findings of a

* It should be noted that some confusion exists when the photographs and text in the "Dynamics Tests of Aluminum Guard Rails" 1963 are compared.

one year study of cable and beam barriers used on freeways and concerned specific information which was found to be of interest in these observations. The tests were all on cable barriers observed, with various modifications. The remaining work reported was for the definite purpose of gleaning particular information to fulfill the three objectives given above. To keep these tests in perspective the objects of the series of five tests over a period of eight years by the State of California are given in Table 2.

Test	Objective	Details
C.1957	b	11 kerb designs of 6", 9", and 12" heights tested in 1954 4 most adequate designs of 9", 10", 11", and 12" heights tested in 1955 to determine dynamic lift of kerb.
	a	5 barriers tested in 1955 to determine 1) The minimum barrier height and 2) The maximum distance that a rail could be set back from the kerb without the latter providing a dynamic lift.
C.1956	a,b	1955 tests to determine dynamic lift characteristics. (See C.1957a)
C.1959	a	Behaviour and response of 3 types of median barriers.
	c	Effects of the 3 barrier types on test cars and dummy occupants.
C.1960	a	Behaviour of 3 types of guard rail a) Concrete balustrade b) Concrete rail and parapet c) Steel rail on posts.
C.1964	a	Behaviour of 2 types of rigid bridge rails and a modified design.
	b	Effects of kerb on dynamic lift and effects of rail height.

Table 2. Objectives of Five State of California Full Scale Barrier Tests

The tests of Wutzier, 1960 on D.A.V. kerbs served purely as manufacturers' demonstrations. The tests by Takahashi on D.A.V. barriers dealt with determining maximum speeds of impact to induce a definite lateral rail deflection. The Swedish tests on the D.A.V. barriers were to establish the barrier performance and capacity for 2 ton light trucks and 20 ton trucks. The Canadian tests (Hénault, 1963) were intended to facilitate design of guide rails at a particular location. The work compared various guide rails under the three headings given above.

Design of Experiments. The type of variables involved in a test has been outlined in the report on model experiments. Essentially, in full scale tests a similar set of variables have to be controlled. In the model work this control was quite possible but in the full scale tests much more ingenuity is demanded. Having in mind the variables in the model tests and the objects of these tests it is possible to arrange components to be controlled in full scale tests under four headings:

1) vehicle variables
2) barrier variables
3) kerb variables
4) road variables.

Under the first heading would be included such items as the velocity vector, braking and driving forces, vehicle weight and weight distribution. For the barrier conditions considerations of configuration, material properties, structural action and initial conditions in the barrier structure may be important. The kerb effects include not only the vertical position of the vehicle at impact but also the velocity and direction of the vehicle at that time. Model tests by Ayre et.al. indicate that the coefficient of friction between vehicle wheels and the road surface are material in the response of both the impacting vehicle and the barrier. It would seem that the road design and construction could incorporate pertinent variables to be considered in the design of full scale crash tests. Additional to these considerations certain questions concerning the interaction between the vehicle and the barrier may have to be faced. These would involve the energy dissipation in the impact, which would depend upon the friction between vehicle and barrier together with the irrecoverable deformations of the two. Only in full scale tests can these features be properly considered and little help from models or mathematical analysis can be expected until the phenomena are better understood.

The design of experiments to isolate the interesting variables and to keep constant other effects is difficult in full scale tests. In the first place the statement of the interesting features is not always clear. Under these circumstances it is not surprising that many tests have been designed only to determine whether a particular barrier is safe under certain conditions of loading and what conditions do cause failure. These ad hoc tests do give definite results as far as immediate performance is concerned but they do little to help in improving barriers and their design. Undoubtedly, such tests may be essential to compare barriers and for acceptance purposes. Tests with a more basic engineering intention can often benefit from this information but it would appear that the main source of understanding must come through a combination of full scale and model tests with some supporting analytical work. The tests by the State of California are notable for the way that important variables are investigated independently. The tests by the Cornell Aeronautical Laboratory, with the limited intention of justifying assumptions made in the analysis and verifying the analysis, are also clearly designed. In Japan, Takahashi chose to determine a critical velocity for a particular vehicle and barrier. In these three groups of tests the technical objectives were well defined, resulting in conscious efforts to isolate the interesting variables. The success of these tests would appear to justify the time spent on the design of the experiments.

Apparatus. The objectives and design of the tests determines the requirements of the apparatus. The instrumentation is concerned with operating the vehicle, describing the vehicle path and velocity, impact force, strains and deflections in the barrier.

The vehicles in the State of California 1964 tests had the following basic controls:

brakes . . . on-off

ignition . . . on-off
steering . . . incremental left and right.

The brake lines were connected to a gas system which was actuated by a solenoid-valve. By "pulsing" this system the car was brought to a normal halt. The Lehigh University tests used a spring loaded mechanism, applied once, to halt the car. Ignition was turned off in the Californian tests when the brakes were applied. The ignition was wired into a remote control panel where it was actuated. Any loss of signal or failure of the transmitting apparatus automatically applied the brakes and hence cut the ignition. The steering was controlled by a 2 H.P. electric motor mounted on the passenger side floor board and connected to the steering wheel by means of a belt and pulley system. Signals to a pulser actuated the motor in increments varying from 1/8" to 1" per pulse. The pulse rate was variable from 2 to 20 per second and was set after a trial run. In the Lehigh tests steering was accomplished by means of a 6 volt motor geared to the steering wheel. Hénault, 1963 used a system of guide rails to steer the vehicle in a curved path at the barrier.

Control of California tests was by radio equipment in a truck which followed the test car at a distance of 200 ft. The control equipment in the test car was located in the trunk and an audio instrument was set up so that the sounds made by the test car were transmitted to the control operator in the truck. This allowed better correlation between his reactions and the test car movements. The Cornell tests used the same vehicle apparatus as the State of California set-up described.

Vehicle velocity vector--at least in the horizontal plane--was determined by cameras mounted high over the impact region. The ground distances were marked on a surface grid and velocities were determined from photographs and knowledge of the number of frames exposed per second. Impact information was obtained by the use of documentary cameras.

Conditions in the impacting vehicle and the dummy occupants required extensive and ingenious instrumentation in the California tests. Accelerometers were placed on the vehicle and on the anthropometric dummy. The vehicle accelerometers measured both longitudinal and transverse accelerations although only average values were possible because the gages were mounted directly onto the car frame and were exposed to the natural vibrations of the frame at impact. Dynamic strains in the barrier were measured by strain-gages and mechanical stylus gages were used to produce deflection curves, both for horizontal and vertical motion, of the barriers under impact. A significant feature of all test instrumentations studied concerns the correlation of the data in the time sense. In California this was accomplished by the vehicle tripping a series of flash bulbs which were recorded on each camera. These signals were marked on the oscillograph records of the barrier strains and these and any brake action were established on the accelerometer records.

Experimental Procedures. These depend entirely on the objective of the test involved; here the procedures will be considered with respect to the impacting vehicle, barrier and environmental conditions such as the roadside kerb.

In all tests the vehicle was directed, at some known velocity vector, into a selected portion of the barrier. Nearly all tests were conducted with passenger cars normally found on the highway. The weight of such vehicles was between 3,000 and 4,000 lbs. Tests with trucks and single-deck buses weighing 20,000 lb. and 17,500 lb. have been made in Sweden on D.A.V. rail and in California on a cable and chair link barrier. Vehicles have been despatched at the barrier by towing, under free motion down a ramp and also under their own driving unit in order to obtain the required input energy. The use of ramps for high speeds is largely a prohibitive matter; for instance a 50 ft. high ramp results in a terminal velocity of 38 m.p.h. In order to increase the input energy heavier vehicles must be employed and the nature of the response for heavy vehicles at low speeds is not necessarily the same as passenger cars at high speeds. In order to remedy this situation the types of remote control systems indicated in the previous section have been developed. Under these conditions, with adequate acceleration and deceleration distances, impact conditions within the speed potential of the vehicle can be obtained. A method utilized in England employs a towing vehicle which disengages itself from the impact vehicle before collision with the barrier. Speeds of 50 m.p.h. are obtained with this method. Tests in general have employed impact velocities of about

65 m.p.h. and impact angles below 20°. As indicated previously more extreme impact angles and velocities have been employed (40° and 85 m.p.h.). That part of the velocity vector associated with the angle relative to the road surface is a special feature which is dealt with at length later.

Barriers tested have only been specifically designed for bridges in a few cases. The corrugated metal beam barrier mounted on steel or concrete posts tested by Beaton and Field, 1960 (C.1960) was considered to be of this type. Zurcher, 1961 used a beam barrier with a Δ cross-section connected to box steel posts set in the concrete deck. The connections involved spring brackets. These have not been tested except for any impacts which have occured on those installed on one bridge in Switzerland. Other bridge railings installed, but not tested in the experimental sense, include the Swedish model with posts at 6'-6" centers of mild steel with $3\frac{1}{4}$" x 1" mild steel strips running on each side of the posts for the length of the bridge at 3'-7" above the road. This provides a continuous tension member and Rinkert, in a preliminary report, claims the rail is adequate to resist cars without excessive damage and bus impacts normal to the rail at 31 m.p.h. Many of the designs for bridge barriers would appear to have been envisaged with an eye to elegance rather than use; (Franck, 1961). This cannot be said of an Austrian system which includes an 8" kerb on top of which is a 12" D.A.V. barrier supported behind by a cantilever from the bridge deck; 22" behind this barrier is a post and rail type barrier similar to the Swedish type already described. Beaton and Field, 1960, reported full scale tests on conventional reinforced-concrete parapet designs, with top rail and balustrading. The C.1964 report includes five tests on two basic geometric designsand a trial design bridge railing. These designs included a parapet surmounted by posts and horizontal pipe rails. The Lehigh tests on an extensive range of 10 designs gave much interesting and sometimes comparative information.

The limited tests on actual bridge barriers emphasizes the necessity of considering the information available for work on normal highway edge and median barriers. These involve cable, link fence, beam barriers and solid parapet barriers (D.A.V.). The first three types include posts as supporting members and the spanning members (either subjected to membrane or bending strains). In all types the problem of the reaction transmitted to the ground is different from that in a bridge. In the experiment the location of impact relative to the highway and posts is an important parameter.

The main environmental point concerns the presence and design of kerbs. The experiments included geometric variations of kerb height, undercutting and distance of the kerb edge to the barrier. These geometric factors were thought to have bearing on the velocity vector of a vehicle at impact, especially the angle of the vector normal to the road surface. Physically, the material of the kerbs was varied to establish the rebound and energy dissipation characteristics. In this respect kerb facings of steel and concrete were tested. Other environmental factors indicated by model tests, such as road to wheel friction, have not been isolated in full scale tests. The experimental procedures are summarized in Table 3.

Item		Variable
Vehicle		Weight. Velocity. Horizontal and Vertical Incidence Angle. Location of Impact.
Barrier -	Parapet type	Solid or open (configuration).
	Cable type	Spacing of cables. Initial tension. Post spacing.
	Beam type	Spacing of beams. Post spacing.
	All types	Material of Construction. Shape. Height. Connection detail. Method of load transmittal to main structure or ground.
Kerb		Geometric: height, width, shape. Material: concrete or steel facing.

Table 3. Variables Considered in Full-Scale Barrier Tests

The variables discussed and outlined in Table 3 have to be correlated with the usages of bridge barriers. For instance the connection of barrier posts or parapets to the main structure is an important detail for which information on posts reacting directly to the ground is not valid. Similarly, much of the work on barriers has been concerned with damage to the vehicle and occupants; in bridge barriers this point is of secondary importance from the design viewpoint.

Results and Discussion. In considering the results of tests it is necessary to first determine the effects of kerbs on the velocity vector of the vehicle impinging on the barrier. Beaton, Field and Moskowitz, 1962 mention that cars have been thrown 8 to 10 ft. into the air and cartwheeled in accidents. Clearly protection against such eventualities is unreasonable and here we will concentrate on the effects of kerbs and the barrier itself in altering the velocity vector. On the Santa Ana freeway a car jumped a 30" barrier because it struck the kerb on leaving the pavement. With a kerb closer to the barrier the impinging vehicle could be rising at impact. Palmer, 1962 notes that whenever a kerb is struck before the barrier the vehicle impinges at an unfavorable angle and the drivers loose control because they are thrown about. The ability of a vehicle to climb a kerb depends upon the kerb shape, facing material and roadway to tire friction coefficient, as well as on the vehicle dimensions, shape and weight distribution. Beaton and Peterson found that the reduction in the friction between the kerb and tire effectively helped to redirect vehicles back into the highway, usually completely out of control. The Road Research Laboratory showed that the ability of vehicles to climb a kerb was much less when the roadway was wet and the road surface-tire friction low (Jehu, 1964). Both of these low friction successes in avoiding climbing did little to decelerate the vehicle. Work by Beaton and Peterson found that an undercut kerb was necessary to minimize climbing and maximize the vehicle deceleration. It has been suggested that it may not always be advisable for kerbs to prevent climbing. Vehicles at contact with kerbs may tend to roll over and instead of being resisted by the barrier face actually may use the top of the barrier as a fulcrum in the rotation. Kerbs which cause the contacting wheels to rise provide a downward reaction on the non-contacting wheels and prevent this rolling. A Dutch kerb 12" high with a $15\frac{1}{2}$" radius concave profile is claimed to carry out this object.

The most comprehensive set of quantitative tests on kerb effects has been carried out by State of California officials and their results will be summarized. The C.1953 work was on eleven types of kerbs each with heights of 6", 9" and 12" with test cars directed at $7\frac{1}{2}^{\circ}$, 15°, 20° and 30°. Angles between 15° and 20° seemed to be critical in as much as a vehicle deflected at 15° would mount the kerb at 20°. From these tests the four most promising designs were selected and tested with heights of 9", 10", 11" and 12". All 9" kerbs were mounted at 30 m.p.h. at angles of incidence of 15° and 20°. For heights greater than 9" increased speeds were necessary and for flatter angles speeds of over 60 m.p.h. were necessary to mount undercut kerbs with steel pipe face and with 3" steel angle corner cover. Mounting cars behave as follows:

a) The wheel deforms with little change in car elevation;

b) The wheel recovers and helps in the dynamic jump of the car.

Once a vehicle has mounted a kerb the length and the height of the dynamic jump are independent of the shape and material of the kerb. The adjustment of kerb parameters is only of importance for maximizing the number of deflected vehicles but is of no importance to a vehicle which will mount the kerb and strike the barrier. This means that attempts to improve the action of kerbs in deflecting and decelerating vehicles may be carried out independently of barrier design considerations provided the height of incipient mounting is not reduced.

Consideration of the mechanics of the action of vehicle mounting a rubbing kerb results in two questions of importance for bridge barriers:

i) The minimum height of the barrier;

ii) The maximum distance of a barrier from the kerb to ensure a minimizing of dynamic jump.

The intention is to maintain the velocity vector of the vehicle as close to horizontal as possible and yet have the advantage of the deflecting capacity of the rubbing kerb. The conclusions of C.1957 are given here as they supersede the findings of C.1956. The conclusions are four-fold:

a) rails lower than 21" high do not present an effective barrier to impinging automobiles at high speeds and incidence angles (45-60 m.p.h. and 20-30°). The average height of a car center of gravity is 21" to 23",

b) barriers with no kerbs should be built to a minimum of 27" high,

c) if a rubbing kerb extends more than 5" from the face of the barrier railing then the kerb will act as a fulcrum and induce a dynamic lift,

d) to account for (c) the 27" barrier height recommended in (b) should be increased by 5" for each 12" proudness of the edge of the kerb beyond the barrier. This rate of increase is greater than most of the plots of dynamic jump versus closeness to the barrier curve in C.1957 and a maximum height of barrier of 48" is proposed which implies that the kerb is more than 48" proud, at which distance the vehicle trajectory would have generally passed its highest point and be descending. The cases where the dynamic jump had a greater gradient than 5/12 are in Table 4. In view of the evidence that dynamic jumps are flatter than 5/12 and the absence of a trend in the

Kerb Type	Height	Impact Angle	Impact Speed
A. Concrete with undercut	9"	7.5°	60 m.p.h.
and bullnose upper corner	11"	20°	30 m.p.h.
B. As A but bullnose made	9"	7.5°	60 m.p.h.
by 3" leg steel angle plate	9"	15°	20 m.p.h.
	11"	15°	45 m.p.h.

Table 4. Kerbs With Reported Dynamic Jump Greater Than 5/12 (C.1955 and C.1957)

results of Table 4 it may be concluded that the recommendations of the State of California investigators are reasonable. The mention of the coincidence of minimum barrier height and vehicle center of gravity in (a) was first made by Barnett, 1939. He also noted the general decrease of wheel diameters with time and the standardization of bumper height at 17" by the S.A.E. This is important because one variable of vehicle type is avoided in full scale tests. It is unfortunate that the case cited by Beaton, Field and Moskowitz, 1962, of a dynamic jump of 30" made no mention of the kerb type and dimensions.

Although limited in numbers, actual full scale tests on bridge barriers provide much useful information. The results of the C.1960 and C.1964 are of particular interest. Those reported in 1960 employed three types of barriers each of which was tested by a car impacting at 60 m.p.h. and 30° angle. Only one of these designs was effective and this was subjected to a 17,000 lb. bus at 40 m.p.h. and 30° angle. The kinetic energy of the bus was about twice that of the car. The tests were not considered to be typical of normal use but were thought to be representative of a more severe, but more practical, type of oblique accident with a bridge rail. Firstly a semi-rigid barrier was tested consisting of 12 ga. corrugated metal guard rail, fastened to 6", 15.5 lb H posts at 75" centers. The posts were secured to the fascia of the concrete deck by 2-$1\frac{1}{4}$" bolts and 2-3/4" bolts. These anchorages were inadequate, failed on impact and trapped the vehicle in the resulting pocket. Two rigid rails were tested and a definition of required behaviour was given as to

> "...retain the vehicle on the structure, reflect the vehicle at a low angle so that it continues to travel in the general direction of the traffic flow, and must not impart a rolling or twisting movement to the vehicle, at least not to the degree that would result in the car overturning in the travelled lanes."

The first rail tested did not meet these requirements. Vertical posts were demolished and a gap large enough for a vehicle to pass through was established; 14' of the railing was destroyed. The test was considered a failure. The barrier was of the balustrade type with a 9" high kerb and 21" projection, giving the vehicle a 9" rise where it struck the rail. The height of the rail and kerb was 37" $>$(27" + $\frac{5}{12}$ .21"). The rigid barrier design consisted of a 21" high concrete parapet carrying a concrete rail supported on concrete posts at 48" centers.* A concrete rubbing strip projected from the parapet. The car test was repeated twice; the second time the impact was at an expansion joint. On each occasion the vehicles were badly damaged and the barrier largely unscathed. The bus impact resulted in the vehicle rising high enough in the collision for a part of the frame to slide along the parapet wall shearing off the posts.

These described tests led to the development of the three barrier types reported in C.1964. The tests reported in 1964 were more severe than the previous experiments and involved speeds of 75 m.p.h. at 25° with a 4,300 lb car. The barriers all consisted of a concrete parapet with a pipe rail carried by posts surmounting it and all posts fixed to it. The variables were parapet height and kerb proudness. The top of the rail was in each case 15" above the top of the concrete parapet. Essentially the tests showed that if,

a) the height of the parapet is below the center of gravity of the impinging vehicle

and, b) the deflection of the railing is large, possibly due to post failure,

then there will be a tendency for the vehicle to rise and subsequently vault the barrier. However, if the parapet is below the center of gravity of the vehicle but the rail is stiff, then there will be no tendency for a vehicle to climb the barrier.

The first test was on a barrier with a 10" kerb projecting 24" from the face of a 28" high concrete parapet. The 15" post and rail system of aluminum made a total height of 43" above the roadway. The kerb did not induce a dynamic lift, (although this result may not be extrapolated to smaller incidence angles) and the headlight-mudguard assemblage struck about 10" below the rail top and was torn off by a post. The trajectory of the vehicle was not corrected in the manner previously specified because of the inadequate strength of the rail-post assembly. Test two was on the same components as the first

* The word concrete when applied to structural elements such as parapets, rails and posts is meant to signify that the members are reinforced.

except the parapet height was 21" (total height 36") and the kerb reduced to a rubbing strip (4" proud, 10" high). The car rose 12", sheared off 3 posts although it did not pass over the barrier. It should be noted that the vehicle rise was caused by the front bumper riding up onto the top of the parapet before the wheel contacted the kerb. This is apparent in the film sequence (Exhibit 7). In the third test steel posts and pipes were used in the same set up as in test two. This steel assembly was overdesigned so that a comparison with test two was possible. Test three was completely successful from the strength viewpoint although kinematically the frame appeared to attempt to ride up onto the parapet but was held down by the rail. In test four the same set-up as in test two was employed except that the parapet height was increased to 28" and the total height to 43". The test was completely successful, the vehicle was redirected smoothly with no tendency to climb the parapet, and the rail suffered no structural damage. In the last respect it is difficult to determine the load on the rail as the mudguard passed under it and was ripped off by a post. The fifth test was with the same set-up as in test two except the aluminum railing was supported by malleable iron posts. As in test two, with a 21" high parapet the vehicle tended to rise up but in five the car was redirected so that its side rather than the top of the hood impinged onto the rail. All these tests employed parapets which were found adequate from the previous work by the State of California investigations. The barriers tested in C.1964 are designated California Type 1, in tests 2, 3, 5; Type 2 in test 1; and Modified Type 1 in test 4.

The Lehigh University carried out eighteen tests on aluminum bridge rails. The rails incorporated ten different designs to the specifications of A.A.S.H.O., Revised A.A.S.H.O., State of California and B.P.R. The tests were designed for the impacting vehicle to be at a speed of 60 m.p.h. and an impact angle of 25°. Variations from these figures were included. Test details are given in Table 5.

Test	Barrier	Speed	Angle	Result
1	Extruded Posts 8'cc, 3 rails, 10" kerb, Height 44", A.A.S.H.O.	60 mph	25°	Failure. Connection between posts and baseplate broke. All rails fractured at one place.
2	As 1 except welded plate post, Height $52\frac{1}{2}$", 1962, B.P.R.	50	25	Posts: no damage. Rails: $\frac{1}{4}$" lateral permanent set in bottom rail.
3	As 2	62	25	Posts: baseplate and anchorage failure. Rails: Permanent set laterally and vertically. Toprail $2\frac{1}{4}$", $\frac{1}{4}$". Middle rail $3\frac{1}{4}$", 3/4". Bottom rail 3 3/4", 1 3/4".
4	As 1 except height 51 5/8",1962,B.P.R.	65	25	Posts: no damage. Rails: slight damage to ends.
5	As 4	56	25	Posts: no damage. Bottom rail: 2" lateral set, 1 1/8" upward set
6	10" kerb, 21" parapet*, Pipe rail and cast post at 8'cc on parapet top, Height 38",1962,BPR	60	25	Posts: deformation at contact face. Rails: no damage.
7	As 6 except two pipe rail, Height 49 5/8", 3/4" bolt at rail post connection	64	30	Posts: no damage. Rails: bottom 1 3/4" lateral set.
8	As 7 except $\frac{1}{2}$" toggle bolt	56	35	Posts: no damage. Rails: bottom 3/4" lateral set

9	As 7 except no parapet	60	25	Posts: slight buckling concrete cracked under post Rails: Bottom 2 5/8" lateral set Top 1 1/2 lateral set
10	21" parapet. Extruded posts at 6'-6"cc. Single formed rail, Height 33 7/8" Revised A.A.S.H.O.	62	25	No damage.
11	As 10	55	25	Posts: no damage. Rails: slight deformation.
12	As 10 except two rails, 10" parapet-kerb, Height 26"	67	25	Posts: Flange to base plate connection broken. Rails: 3" lateral set.
13	As 12	62	26	Posts: damage negligible. Rails: lower lateral set 1" vertical set 5/8", Upper lateral set 5/8".
14	As 12. Rail splice included at impact point	58	27	No damage.
15	As 10 except no kerb or parapet, 3 rails, Height 32", Posts at 8'cc.	62	25	Posts: slight damage. Rails: $\frac{1}{4}$ lateral set.
16	As 12	58	27	Posts: no damage. Rails: slight set.
17	California No. 1 (no kerb)	50	28	No damage
18	California No. 1 (no kerb)	62	25	Posts: no damage. Rail: 1 3/4" lateral set 1/4" vertical set

Table 5. Summary of Aluminum Bridge Rail Tests by Lehigh University. 1963
* Parapet height includes kerb.

From these results it can be stated that two barriers failed by a break at the base plate-post connection. One other design successfully deflected the car but the same connection was broken. The presence of a kerb meant that the bottom rail accepted considerable vertical force. Whether permanent set occurred in the bottom rail depended upon the beam strength of the member. The California Type 1 showed a vertical set with an impact velocity of 62 m.p.h. but no set at 50 m.p.h. The parapet had no kerb. The signs of dynamic lift are difficult to determine from the test sequence photographs but information on the vertical set gives some indications on this matter. For tests 7, 8 and 9 the post loads were determined by strain gage and scratch gage information. The results are in Table 6.

Test	7	8	9
Contact Point relative to post	$1\frac{1}{2}$' before	3' after	$1\frac{1}{2}$' before
Angle	30°	35°	25°
Speed	64 m.p.h.	56 m.p.h.	60 m.p.h.
Max. post load			
Strain gage	38,500 lb	38,000 lb	41,500 lb+
Scratch gage	30,500 lb	30,000 lb	Damaged

Table 6. Loads on Posts

The tests by Hénault in Montreal were essentially on bridge barriers for use on elevated highways. Thirteen tests were carried out on eight different barriers. Impact was intended to be at 55 m.p.h. at an angle of 20° on two pipe rails connected to posts and fixed to the original kerb. Additional features such as a small concrete wall and various spring mounted metallic devices, were put in front of the rails and their effects examined. The important conclusions were as follows:

a) that the concrete wall was the strongest barrier

b) that the vehicle rebound was inversely proportional to the strength of the barrier

c) that the projection of the kerb had a profound effect on the kinematics of the vehicle

d) that the barrier should be struck by body work and wheels concurrently to ensure its best performance. The practical recommendation was for the kerb to be surmounted by a small concrete wall (12" high, 6" across the top and 14" across the base) behind which is the two rail barrier. The feature different from the California barrier is that the rail is not on the top of the parapet.

The full-scale tests on guard rails as opposed to bridge barriers are often informative. It may be thought that the necessity of using rigid types of barriers rather than flexible cables and link fences on bridges may result in more severe collisions. Encouragement may be read into the work of Beaton, Field and Moskowitz, 1962, who found that although full-scale controlled tests did indicate less severe effects with cables, the weight of statistical data on actual usage over a one year period showed that the collisions on both types are of about the same severity. In one respect the beam barriers involve more vehicles in as much as 58% of the collisions concerned two cars whereas the figure with cable barriers was 39%. It was found that the lower cable could provide the conditions for a dynamic lift with low incidence angles. Removal of the cable obviated the problem although a small sports car was reported to have passed beneath a cable barrier. Truck collisions on beam barriers resulted in the destruction of the barrier in the impact region but the trucks penetrated only a few feet beyond. This type of restraint, although adequate on highways, does not conform to the requirements of a bridge barrier.

The use of concrete guard rails is more common in Europe and Japan than this country. One effect of these was reported in an unpublished comment by Tresidder, 1958 which indicated that these barriers on dangerous bends resulted in drivers keeping well away from the barrier and near to the center of the road. On 19 sites personal injuries were reduced by 40% but serious and fatal accidents increased. The effect of a rigid barrier on bridges may tend to crowd the center lanes and this in itself could be a hazard. In this respect, Hénault, 1963, found that the uglier the barrier the further drivers will keep from it. In general D.A.V. rails, or their like, reflect cars at speeds of under 35 m.p.h. and 20° incidence angles, are destroyed by trucks, and overturn vehicles moving quicker than 35 m.p.h. This overturning is towards the barrier and on a bridge may result in the vehicle using a parapet rail as fulcrum and vaulting over.

Takahashi, 1961, noted that beam barriers do not perform as flexural members but enjoy large normal strains. This membrane action results in the posts leaning towards the impact point. Clearly, short lengths of barriers would have to be reacted by a reduced number of posts, whereas, long barriers would have the longitudinal base shear on the posts distributed over far more members. Cichowski, Skeels and Hawkins, 1961 suggested that in order to avoid longitudinal collapse on impact the installation should be 100 ft. long for a 4,000 lb car at 35 m.p.h. and 20°. For 65 m.p.h. the length should be increased to 250 ft. In bridge barriers such lengths may not be easily obtained and yet the same membrane strains would be beneficial. For restricted lengths some end anchorage device would induce longitudinal resistance and hence membrane strains. It must be pointed out that the membrane strains will be increased when the posts are flexible, (either as material or due to the fixing) with short, rigid posts on top of parapets the effect would be much smaller. Cichowski et.al. and Takahashi demonstrated the increase in load capacity of beams and the reduced deflection of beams respectively, as the spacing is decreased. Cichowski et.al. found that by reducing spans by 50% the buckling loads of beams were doubled, Takahaski found that increasing the post spacing by 25% resulted in six times the lateral deflection for the same static load. This last effect is of importance in bridge structures where retention of an impacting vehicle is of interest.

Pocketing is concerned with a vehicle being restrained from longitudinal motion by the barrier. Large deflections resulting from post failure is one way this may occur but the tests in California indicated the tendency of vehicle parts to catch between the parapet top and the underside of the rail. This effect is made more noticeable if initial contact is at a post. Beaton and Field, 1960, suggested that spring brackets between posts and beams facilitated pocketing.

These brackets are successful at low speeds in reducing the impact damage but have no apparent effect at high speeds. The large deformations associated with pocketing in beam barriers suggests that the action will not occur in well designed bridge barriers. However, attention to detail in construction can vitiate any mechanical snags.

Sixty full scale tests were carried out by Cichowski, Skeels and Hawkins in 1961. Reference has already been made to some results and conclusions; additional points relative to bridge barriers will be summarized here.

a) that high membrane and bending strength are advisable. The membrane strength may be provided by adequate longitudinal tie-back in short lengths of railing.

b) a typical impact will destroy four to eight concrete posts. This is not undesirable on a highway but may lead to a vehicle leaving a bridge.

c) improvement of design of barrier termination points required.

d) indicates the different type of rail required to contain heavy vehicles.

Two additional forms of tests must be mentioned. Due to the writing of specifications in terms of static loads a form of proof testing involving static tests on elements and also impact tests on elements have been carried out. Those performed by the aluminum industry appear to be the most complete in this respect. The objectives included a relation between static and dynamic tests for various shapes and the setting up of acceptance criteria. The steel industry tests give a complete picture of the static strength of post and beam members. In addition Takahashi and the Oregon State Highway Department have carried out static tests on barrier assemblages. The second form of test has been carried out by the Cornell Aeronautical Laboratory and have been carefully designed to check and indicate assumptions in an analytical theory. This work is reported in the next section.

ANALYTICAL MODELS

Only one serious piece of analytical work has been attempted and this for highway barrier rather than for bridge barriers. This was by the Cornell Aeronautical Laboratory for the State of New York, Department of Public Works. This work will be referred to by CAL. Ayre, Abrams and Hilger developed differential equations for three degrees of freedom allowed the model vehicle. These degrees of freedom are x in the direction of incidence, y orthogonal to x and Θ the rotation about the origin. The origin was located at the center of gravity of the vehicle at first point contact with the barrier and was considered inertial. The equations obtained were

$$\frac{W}{g} \cdot \ddot{x} = D.\cos\Theta - (L_1 + L_2).\sin\Theta + R.\cos\phi$$

$$\frac{W}{g} \cdot \ddot{y} = D.\sin\Theta - (L_1 + L_2).\cos\Theta + R.\sin\phi$$

$$\frac{W}{g}.p^2. \ddot{\Theta} = -(L_1\ell_1 + L_2\ell_2) + R.r$$

where p is the radius of gyration of the vehicle with respect to a vertical axis through the center of gravity, R the resultant force exerted by the guardrail on the vehicle, ϕ the direction of R relative to x, r the lever arm of R relative to the vehicle center of gravity, L_1 and L_2 transverse friction at rear and front wheels, ℓ_1 and ℓ_2 distance of center of gravity from rear and front axles and D the longitudinal driving or braking force. The experimenters suggest that a step by step numerical solution of these equations would be possible but prefer to non-dimensionalize them and then obtain similitude relationships for model tests already discussed.

The CAL analytical work also uses a step by step numerical approach but the considerable refinement of the formulation will be summarized here.

Test Program. Seven tests were carried out, the first two were only partially successful due to pocketing of the vehicles between posts. The third test was successful and indicated various parameter modifications. These first three tests were on New York Standard Rails.

Test one involved a 4-3/4" cable system, each cable of ultimate strength of 23k. The cables were at 4" centers and the barrier height was 26" above the pavement. The cables were supported by 6 B8.5 posts through spring steel brackets giving an offset of 6". The posts were 10' apart and sunk for 3'-3" of their 5'-3" length. Test two was on a 100' section of structural 10 ga. sheet of 'W' type rail mounted on 6B8.5 posts at 12'-6" centers. Posts were 5'-6" long and driven 3'-3" into the ground. No end anchorages were used. Test three was on a similar rail to that of test two except the posts were at 6'-3" centers and supported rails on both sides, as in a median barrier. The 14' long rails were connected to the posts by bolting through sections of 6B8.5 sections. Tests four and five were on barriers developed by CAL and consisted of 10" x $5\frac{1}{4}$" box rail supported at 4' centers by $2\frac{1}{4}$" x $2\frac{1}{4}$" H, 4.1 posts with U shaped saddle containing the box with no constraints. The box had $\frac{1}{2}$" clearance on each side of the saddle. The posts were concreted into 39" deep holes. End anchorages to simulate a long structure were used in four. Tests six and seven were on the same structure as four and five except the box was 10" x $2\frac{1}{4}$" (1/10 of the I of 4 and 5) and $\frac{1}{4}$" clearance was allowed. To provide longitudinal strains the box was attached by cable to a tie back and became a tension box.

The results of the tests on these barriers are given in Table 7.

Test	1	2	3	4	5	6	7	Units
Impact Angle	34	19	16	18	24	25	20	degrees
Exit Angle	*	*	9	7	7	*	9	degrees
Impact Velocity	41	54	67	58	52	60	55	m.p.h.
Exit Velocity	--	--	48	41	47	*	37	m.p.h.
Loss of K.E.	100	100	50	50	18	100	55	%
Contact Length with Barrier	*	*	18	17	14	*	24	ft.
Barrier max. deflection	144	72	18	13	10	138	30	in.
Barrier permanent set	--	--	13	2	2	--	13	in.

Table 7. Results of CAL Full Scale Tests

* Pocketed.

These results are not intended to be comparative but do provide the required information on assumptions, parametric values and checks of the theory.

Formulation of Mathematical Models. The object set forth was to properly describe barrier response relationships and then employ these to describe the vehicle path during the whole impact. With this in mind the two parts can be attacked separately. It was found that complete generality of barrier response was too complex for solution and therefore the response of each barrier type was described separately. In the case of the vehicle path, the already described barrier load-deflection characteristics were used as input. An important claim of the investigators is that the procedure can be used for optimum barrier studies. The computational procedures involved were as follows:

1) vehicle center of gravity position and direction were extrapolated ahead for a given time increment(constant acceleration assumed)

2) from this a barrier deflection was arrived at with a corresponding barrier reaction force. This was employed to compute the vehicle acceleration at the end of the time increment, (linear acceleration assumed)

3) using the previous linear variation a second extrapolation of vehicle position and direction was made and compared with (1). The average was taken if a third extrapolation was necessary. Repetition occurred until set limits on the positional agreement were obtained from successive extrapolations.

The effects of vehicle deformation were accounted for in the initial collision by a second iterative scheme to obtain a force balance between simulated vehicle forces and barrier forces. This extrapolation procedure had to be repeated for each change in the contact point because a change of this point on the vehicle, relative to the center of gravity, affects both barrier deflection and hence load position relative to the posts. The effects of barrier deformation were treated differently for two types of barrier models, namely,

a) those that depended on the load position

b) those that were to all intents independent of load position.

The load position referred to was relative to the posts. In the type (a) polynominal expansions were used to the mid-span amplitude of the rail for various numbers of eliminated posts. In the type (b) tabular load-deflection characteristics were set up for entry to the part (2) above.

The complexity of the mathematical approach must be compared with the three equations of Ayre et.al. Even these equations are not immediately tractable and require an iterative procedure for solution together with a decision on parameter values. In the case of the CAL work the inclusion of an extensive range of variables further complicates the solution.

Assumptions. The assumptions only involved variables associated with the barrier and vehicle. Other factors mentioned in the objectives and test design of the full scale test section were not taken up. With regard to the barrier, the following assumptions were made:

1) point contact of vehicle on barrier.

2) barrier inertia neglected.

3) rail and post bending moments fully plastic and remain constant at the value given by the dynamic yield stress. Membrane stresses reduce plastic moment in a polynominal manner.

4) plastic hinge moves with point of vehicle application.

5) force-deflection, stress-strain relationship linearly elastic, perfectly plastic. Force-deflection relationships for bending assumed common for all points of contact.

6) linear unloading force-deflection curve. Slope of curve adjusted to release all the elastic energy stored in loading, when the barrier is finally unloaded.

7) zero bending moments in beams further than three spans from the contact point and zero tension in rails five spans away.

8) no post failure due to excessive lateral deflection, no longitudinal element failure due to excessive strain.

The following assumptions with regard to the vehicle were made:

1) a rigid body with three degrees of freedom in a horizontal plane (as Ayre et.al.). Loads impressed at

a) the corner of fender contact

b) the center of front axle laterally

c) the center of rear axle laterally.

The contact point (a) shifts as the magnitude of the force results in body collapse and deformation. Load is transferred step-wise from (b) to (c) when deflection of barrier at (c) attains equality to that at (b);

2) tire loads are given by forces (b) and (c). Front tire direction relative to vehicle is unchanged in the collision. The lateral forces vary linearly with the angle between the plane of the tire circle and direction of the vehicle up to slip at 10°. Then, in the manner of the Coulomb friction phenomena, the force remains constant.

Assumptions regarding a combination of barrier and vehicle effects are:

1) post impact and elimination are not caused by secondary collisions;

2) the frictional force between the vehicle and the barrier are of the Coulomb type and proportional to the lateral force;

3) no energy dissipation due to structural collapse during a secondary collision;

4) at the impact of a post by vehicle, a constant force is in effect along the barrier center line until post failure, specified by a limiting longitudinal deflection. Elastic part of the post deflection ignored;

5) definitions of pocketing and snagging in the mathematical model.

Evaluation of the Procedures. The interaction between variables ensures that the validity of the assumptions and the effect of other parameters could not be properly determined. Under these circumstances the investigators chose to enter the analyses with estimated parameters for barrier and vehicle which were varied during the solution in order that agreement with full scale test results was obtained. These adjusted values can always lead to good correlation but in some cases the parameter values were not realistic. In general it must be said that this was a first attempt at a complete analysis of a very complex problem. It will provide insight for future work where such insight must involve the clear understanding of reaction of separate variables. It is the difficulty of such a separation which made parameter adjustments to fit test data of structural response necessary.

Two assumptions were found to be particularly unrealistic. The simplifications used for secondary or rear end collisions were a source of discrepancy between observed and evaluated heading angles after a secondary collision. The vehicle-barrier friction force was found to be more complicated than assumed.

The response of barriers with deflections independent of load position were found to be more successfully predicted than where dependence on the impact position existed.

BARRIER LOADING

Most of the full scale testing on barriers has employed passenger cars which were typical of those found on the national highways. In addition some tests have been carried out with heavy trucks and buses which have usually resulted in the collapse of the barrier. An implication of this is that the barriers are only required to structurally restrain 4,000 lb vehicles and that they act as a boundary indicator for other, heavier vehicles. If this is the intention of the specification writers and designers then some success is apparently being obtained by existing barriers. It is the intention here to enquire into the consequences of such restrictions on the adequacy of barriers. To do this it is necessary to investigate the number of barrier accidents involving vehicles of all types and the velocity vectors of such vehicles. In 1939 Barnett looked into this matter and although his quantitative results are not valid today many of the qualitative conclusions are. Essentially Barnett considered first the distribution by weight of the vehicles on the highways and then the distribution of velocities amongst these vehicles: The weight analysis showed at least 80% of the vehicles below 6,000 lb. The mean car velocities were greater than trucks but in the case of buses no such differential existed. It was demonstrated that a barrier which was adequate for an impacting car would be equally adequate in resisting a vehicle 50% heavier at 80% of the velocity. Stephenson, 1957 in his work on vehicle distributions on bridges obtained more contemporary information on the distribution of vehicle weights. In general there is extensive information available from State authorities on such weight distribution and some on the vehicle velocities. This data, although helpful, is not truly relevant to the barrier problem. The information required concerns precise data on weight and velocity vector for those vehicles which have actually struck barriers. An examination of this information, organized in a statistically satisfactory manner, would lead to understandable and reasonable design parameters. In this respect the reporting on traffic and accident data is not too helpful.

It would be necessary to have a proper assemblage of weights and velocities, together with an extrapolation of these into the future, in order to make a real decision on design loads. The extrapolation itself may not be difficult because of the use of adequate law enforcement on most highways to keep the speeds of vehicles within the allowed limit, because of configurational design of highways to definite vehicle speeds and because of the common bridge design loadings employed in the country. These three factors should ensure that mean and variance of velocities and upper limits on loads alter little in the years ahead. Hénault examined the accidents on the Montreal Boulevard and found that the main dependents appeared to be driver's age, road conditions and time of day. The conclusions are summarized in Table 8 for the accidents involving actual breaking through of the highway barrier.

Variable	% of barrier failures
Drivers under 25 years	50%
Drivers under 30 years	75%
Damp or slippery pavement	58%
Time between 1 a.m. and 4 a.m.	44%
Time between 7 a.m. and 12 a.m.	28%
Time between 2 p.m. and 10 p.m.	28%

Table 8. Accident Variables on Montreal Boulevard

These figures suggest that immaturity and carelessness are important co-factors and knowledge of the percentage of drivers in various age groups in the future should be of value in extrapolating existing data. In addition the legal requirements for heavy vehicle operator licences should minimize carelessness in the handling of trucks and buses. It would seem reasonable to anticipate that surface conditions will not deteriorate in the future and with regard to the time element, it would be difficult to see reason why the percentage of night driving should change. Only by understanding the human as well as engineering variables can satisfactory extrapolation of information into the future be made.

In this section the occurrence, weight, velocity and incidence angle information for impacting vehicles on barriers are considered under separate headings.

Incidence of Barrier Impact. Before seeking information concerning the incidence of barrier collisions it is as well to obtain some idea of the vehicle population and annual mileage in the U.S.A. Petty and Michael, 1962, reported that in 1960 there were 73 m. motor vehicles registered, 12 m. trucks and buses (17%), 61 m. passenger cars (83%). The former vehicles travelled 130 b. miles in the year and the latter 590 b. miles. Some idea of the mileage of roadway and bridges involved may be gleaned from the work completed in California in 1963. 2742 miles of multi-lane divided highway were opened to traffic, 1352 miles to full freeway standards, 761 miles to expressway standards. In the year 61 bridges were completed. The total bridges in the State at the time were 6,271 made up as in Table 9. These two hundred miles of bridges must be compared with the total of over 150,000 miles of roads of all classes.

Type	Number	Length (ft.)
Concrete: arch	232	36,818
girder	2,158	491,294
slab	2,414	145,885
Masonary arch	33	962
	4,837	674,959
		(127.8 miles)
Steel: arch	5	1,400
plate girder	385	171,930

stringer	277	60,504
deck truss	30	33,166
pony truss	29	10,524
through truss	63	115,999
suspension	2	15,097
Composite multi-plate and arch	77	1,482
	868	410,102
		(77.7 miles)
Timber: arch	1	59
stringer	585	42,473
deck truss	6	1,576
through truss	1	79
	593	44,187
		(8.4 miles)
Total Bridges	6,271	1,129,248
		(213.8 miles)

Table 9. Bridges in California State Highway System Dec. 31, 1963.

In California in the same year 131,520 accidents involving injuries and fatalities occurred and a total number of 306,522 accidents reported. Of this last figure 10,730 involved striking fixed objects, under which heading bridge barriers would belong, and 2,598 of these did not occur at intersections and therefore could have been on a bridge. On freeways the number of fixed object collisions was 0.00747 per million vehicle miles compared to 0.0307 for vehicles which ran off the freeway.

Greene, 1962, attempted to obtain complete national information on vehicle collisions with fixed objects. Unfortunately only 19 States replied and only New Mexico, Ohio and Connecticut had complete information. Some comparative information is given in Table 10. These results would indicate, first the low number of collisions with fixed objects compared to the figures for total accidents, and secondly, the small proportion of these fixed object collisions involving bridges. All of these figures discussed suggest that about

State	Road Types	Object Struck	Number of Collisions		
			Divided Highways	Other	Total
Connecticut (1955-59)	Rural and Urban	All	3,153	12,488	14,641
		Guide rail	1,517	4,383	5,910
		Bridge	212	297	509
Ohio (1959)	Rural	All	54	236	290
		Guide rail		31	31
		Bridge	22	104	126
New Mexico (1955-59)	Urban	All	492	1,703	2,195
		Guide rail	3	21	24
		Bridge	7	52	59
Total of returns (percentage form)		Guide rail	Rural 44%, Urban 29.1%		
		Bridge	Rural 3%, Urban 4.3%		

Table 10. Accidents with Fixed Objects

0.12% of highway mileage involves bridges and that about 0.11% of all accidents involve collision with bridges.

These figures are not given with full confidence but are intended to convey some impression of the small incidence of bridge barrier collisions, and that this incidence is of the same order of magnitude as the proportion of bridge mileage in a highway system.

Vehicle Weight. The type of vehicles involved in accidents are given by the California Highway Patrol report for 1963. Table 11 presents these figures and the proportion of each type of vehicle in use. The sum of the accident proportions leaves some doubt about these figures,

Vehicle Type	Proportion in Accidents	Proportion in Use
Cars, taxis, etc.	0.945	0.88
Buses	0.006	*
Trucks, pick-ups, etc.	0.096	0.12

Table 11. Proportions of Vehicle Uses and Vehicle Accidents in California, 1963.

* No information available

but in general it may be said that the involvement of cars in accidents is in excess to their proportion on the road. However, another way of considering accident figures is to compare them with the mileage covered by each class of vehicle. This is done by Crosby, 1959 from data on the New Jersey Turnpike for the period 1952-57. This is tabulated in Table 12. It should be remarked that the accident rate

Vehicle Type	Accidents	Use	Vehicle Mileage
Cars, Single tire trucks	82.2%	87.29%	88.29%
Buses	1.30%	1.98%	1.51%
Dual tire trucks	16.50%	10.73%	10.20%

Table 12. New Jersey Turnpike Data 1952-57.

on the turnpike is less than half of the national rate. However, these figures may be compared to freeway usage, from which it would appear that trucks are considerably more susceptible to accidents than their usage implies. Buses, on the other hand, are little involved.

From the above it may be suggested that bridge barriers may be subjected to vehicles impacting more or less in proportion to the distribution of traffic types except for the increased likelihood of truck impact on freeways.

<u>Vehicle Velocity</u>. Again it is possible to compare the normal usage vehicle speed to the impact speed. For this purpose the work of Webb's "Speed Survey," 1957, in California and the accident statistics of the California Highway Patrol report, 1963 are used. The results are tabulated in Table 13. From this information it can be seen that data

Vehicle Type	Driving Speeds		Impact Speeds*	
	Average	Critical**	Average	Critical
Cars, taxi, etc.	55.6	64.9	23.5	40
Buses	55.3	63.9	16.5	30
Trucks	49	57	25	45

Table 13. Comparison of Normal Driving and Impact Speeds

* Computed from the data of Table 14.

** 80% of vehicles at speeds below the critical speed.

collected for normal driving speeds is no indicator of impact velocities. Table 14 shows the range of reported impact velocities at accidents in California during 1963. Interest here is on the upper end of these figures which emphasises the likelihood of passenger cars striking a 'barrier' at high speeds, especially when compared to buses, and also when compared to trucks.

Vehicle Type	Speed								
	0	1-10	11-20	21-30	31-40	41-50	51-60	61-70	>71mph
Cars,taxis,etc.	16.3%	13.6%	15.1%	21.7%	13.7%	8.1%	6.8%	3.7%	1%
Buses	11.5%	32.8%	22.5%	17.2%	7.6%	5.7%	1.8%	1.1%	1/1243
Trucks	11.4%	15.5%	16.0%	19.4%	15.1%	13.6%	6.5%	2.2%	0.3%

Table 14. Reported Impact Velocities at Accidents in California in 1963.

<u>Vehicle Direction</u>. In the previous discussion on barrier response it was demonstrated that the impact direction relative to the pavement depends upon conditions of configuration of kerb and barriers. Here we consider the direction of an impacting vehicle relative to the longitudinal barrier and in the plane of the pavement.

Two ways exist of approaching this problem. First, reported information at accidents can be reconstructed so that impact angles can be deduced. Second, bounds on the impact angle can be set from analytical considerations. Hénault used both approaches but found that accident report figures were in no way conclusive. It would seem that a curved bridge in plan would allow a larger impact angle than a straight bridge. In this respect the California Highway Patrol reports 73.7% of vehicles leaving the highway do so on curves. Unfortunately, no figures appear to exist on the proportion of curved bridges and the proportion of impacts on such curved bridges.

The angle of impact used in the State of California full scale tests, namely 30°, was selected in the light of several car-bridge rail collisions reported (C.1959, C.1960). However, most impacts in that State are reported at between 15° and 20°. Hutchinson of the University of Illinois mentions that over 80% of the vehicles which have left a section of Route 66 deviated less than 10° from the highway line at departure. In addition he presents a cumulative curve to illustrate the distribution of vehicle departure angles. Irving, 1962, comments that,

"There is a widely expressed view that the majority of vehicle-barrier collisions occur at angles of less than 20°--a figure quoted as being arrived at as a result of the consideration of the relevant accident statistics without further substantiation."

It is the conclusion here that this state of affairs has not improved in the meantime except for Hutchinson's work on Route 66.

Making use of road configuration, vehicle speed, position at loss of correct path, and road-wheel friction, authors have proposed formulas to describe the incidence angle. In 1939 Barnett used the expression

$$\alpha = \cos^{-1} (1 - \frac{d}{R})$$

for the impact angle α in terms of the highway curve radius, R, and the distance between the vehicle line before deviation and the barrier, d. The same expression was obtained in Swedish work in 1955. The final impact angle depends on the variables. Barnett obtained 16° and the Swedes 25°. Hénault considering d = 28' and the vehicle velocity at deviation as 55 m.p.h. found an impact angle of 28° at Montreal. Mention in this work is made of vehicle-pavement friction, but it is not clear how it enters the computations. In 1939, Brickman's interesting discussion of Barnett's paper considers the condition of a vehicle making a turn without skidding. Such a turn would have a minimum radius of $\frac{V^2}{g\mu}$ which when substituted for R above results in an impact angle

$$\alpha = \cos^{-1} (1 - \frac{g\mu d}{V^2}).$$

V is the velocity at deviation, μ the tire-road friction. Some approximate values of α from Brickman's analysis are given in Table 15. The effects of tire-road friction are clearly brought out by these results together with reduction of incidence angle with increase in speed. To these values of α must be added the angle between the tangents to the road curve between first deviation and impact in the case of impact on bends in the highway. With skidding the angle α must be reduced.

Speed (V)	μ	d	α
50 m.p.h.	0.9	28'	32°
60 m.p.h.	0.9	28'	26°
65 m.p.h.	0.9	28'	24°
50 m.p.h.	0.6	28'	25°
60 m.p.h.	0.6	28'	21°
65 m.p.h.	0.6	28'	20°
50 m.p.h.	0.3	28'	18°
60 m.p.h.	0.3	28'	15°
65 m.p.h.	0.3	28'	14°

Table 15. Impact Incidence Angles from Brickman's Analysis.

The figures from Brickman's analysis suggest that the road surface should be smooth to minimize the impact angle. It would seem reasonable to presume a figure of $\mu = 0.3$ for normal conditions, where the impact angle is below the often quoted figure of 20°. With wet or icy conditions the angle of impact will decrease, but, no doubt, the incidence of accidents will increase.

CONCLUSIONS AND RECOMMENDATIONS

In the course of this review various suggestions concerning future work on bridge barriers have been made. These need not be repeated here. Conclusions from the review fall into two parts; first what has been learnt and second, what else requires investigation and how may this be carried out. On the first part it may be said that the effect of kerbs has been understood and methods for design of configurations provided. In addition various "safe" and "unsafe" barriers have been examined and many of the host of variables isolated. Model tests and similitude testing have been little used on beam-like barriers and yet the extensive information provided by the former in the case of cable barriers gives considerable encouragement with respect to their use on bridge barriers. Full scale tests have been extensively made and information as to reasonable configurations and some knowledge of structural response has been obtained. In the analytical field it is felt that little progress will be possible until the response and loading are better understood.

In making recommendations it is believed that the proper viewpoint to take is that of the designer. It is he who should determine what information is required and who should analyse the information when it is obtained. The information will be of a statistical type for loadings and be reduced data for response of structures. In much of the work reported it would appear that the efforts of testing groups have been largely divorced from the real needs of the designer. Naturally this is not always so, as instanced by the work on kerb effects in California, the effects of post spacing in Japan and at General Motors and by the post loadings determined by Lehigh University. However, the positive design information provided by these particular tests indicate the value of real communications between the design office and testing field.

The type of information a designer requires may be considered under the headings of loadings, parameters affecting the barrier response and the functioning of the structural elements. In fact, a complete understanding of the last two headings is far from available and it would seem reasonable, in the first place, to have effective static loading requirements for design purposes. The use of static design must be backed up by adequate proof and acceptability tests for differing materials and arrangements of structural elements. Again these tests need not be the impact of real vehicles on made up sections but be made on equivalent apparatus in a more controlled environment. The equivalence requires some understanding of parameters affecting barrier response mentioned above, and can only be obtained after extensive full scale and associated laboratory testing. However, the use of full scale testing may then be clearly understood to fall into two parts,

a) to determine general characteristics of barrier response

b) to be used as a scale for the proper development of equivalent, inexpensive proof testing.

The previous section discussing loading was a deliberate manipulation of available data to get some idea of the likelihood of various types of vehicles hitting a bridge barrier at definite speeds and angles of impact. The information directly available does not give immediate figures on the subject but by conjecture the likelihood of various types of collisions can be obtained. Only with data of actual collisions on bridge barriers can a true idea of design loading be found. This loading cannot be that of the worst impact that could occur but must be specified in a statistical sense so that a known percentage of all impacts will be contained. The social satisfaction expressed by more resistant railings may involve expenses completely out of line with this gain. Such a decision must be made from a complete set of statistical information of direct barrier impacts and not inferred from manipulation of possibly dependent data as carried out in this work. The decision on loading must be made by a specification authority and it should be in two parts:

a) a statement of various vehicle weights, impact angles and speeds that a barrier should be designed for,

b) static loads equivalent to (a).

The first part must be carried out as a problem in decision theory the second part requires experimentation with model and full scale barriers.

The results of the section on loading, although not claiming to be complete, do indicate the vehicle type impact probability and the speed of impact. From this it is thought that the speeds used in tests are too high. Undoubtedly they do represent the effects of heavier vehicles at lower velocities but it is quite unclear what these vehicles and speeds are. In order for the tests to have relevance they must involve the intended design vehicles, speeds and impact angles. Only with a picture of the response of barriers to these loadings can equivalent, static design loads be postulated. The selection of static design loading may be attempted in one of two ways:

a) specify an equivalent horizontal loading to be applied anywhere at a definite level.

b) specify a loading which varies in intensity according to post spacing, application position, longitudinal barrier restraint and other parameters exposed by response tests.

In the case (a) the allowable strength and deformation values of the barrier must be adjusted having in mind the parameters of (b). In the case (b) uniform values for various element strength and deformation may be used. Both cases take into account the main parameters that affect the barriers; in the first case the effect is dealt with by varying the loading and in the second by varying the allowable response. Both methods involve fictitious quantities and neither is preferable but the designer who has not the opportunity to consider all the evidence must not be waylaid into thinking he is dealing with real loads. Only by understanding the real reasons for the variables in loading or response can he hope to produce a reasonable design.

Two other points must be mentioned about loadings. The work on impact angles would appear to be fairly complete and it is possible to determine bounds on them. The information concerning the high incidence of high speed impacts in the early hours should be considered as an extreme load in as much that the probability of such a vehicle causing a multi-vehicle accident if it falls onto the under freeway should be examined. An approach to this already exists through the consideration of the possibility of collisions at various times by vehicles crossing an unbarricaded median into the opposite traffic lanes. It is believed that design of barriers for these high impact speeds, when the probability of serious additional accidents is low, would be uneconomical.

With definite design loading the designer will then require knowledge of barrier response. In particular the structural action of the various elements is of importance. First we will consider the variation of response with changes in configuration. In this respect the action of kerbs has already been described and in general it would seem that kerbs are avoided as much as possible. The effect of cantilever concrete walls surmounted by post and rail systems may be considered in two classes. First, the psychological effect of such a massive barrier may cause neglect of the roadway near the wall with subsequent dangers on the center lanes. Already evidence to this effect exists but the full nature of the problem has yet to be investigated. Second, the shape and height of the walls should be considered. The height should be sufficient such that "snagging" is avoided and the shape may allow for the safe redirection of much impinging traffic. As mentioned in the main body of the work, concave surfaces have already been successfully used for such direction and overturning control. In the case of post and rail barriers configuration concerns rail and post spacing, set-back of post from rail, and post and rail shape. Here the configuration will not tend to have harmful psychological effects (except that confidence in the non-severity of impact may lead to increased impact incidence), and thought must be concentrated on the ability of each barrier configuration to properly redirect and contain vehicles without "snagging" and other restrictive actions. Some evidence exists on this matter and a reasonable program of model tests should provide conclusive results.

Decisions on acceptable configurations lead to questions of the structural effects exhibited by barriers. For configuration design real vehicle behaviour must be considered and in discussing structural effects it is necessary that real vehicle impacts be studied in order that idealizations of response for specified design loading can be made. In the case of a parapet wall the response is largely that of an unyielding barrier with local spalling. These external effects need to be augmented by knowledge of dynamic strains in the steel and concrete together with the position of maximum strains relative to impact. Some work on this is outlined in C.1964 report but no data is provided. With comparative knowledge of these then the response of various forms of wall thickness and reinforcing can be evaluated. In the case of beam barriers the relative importance of membrane and bending strain energies must be decided for various arrangements of longitudinal constraint. The dynamic strains must be examined and the time-displacement pattern understood. The last effects have already been considered at length from photographs of full scale impacts but complete gaging of members has seldom been carried out.

Coupled with such full scale tests, model experiments could be successfully employed to answer the question of the relative strain intensities for various structural arrangements. Knowledge of this response will put the designer in a position to deal with impact effects in a static analysis. From the strength viewpoint, knowing the dynamic straining rate and the capacity of material in this respect (from laboratory tests), adequate allowable strengths may be prescribed. From the displacement viewpoint knowledge of the manner of plastic deformation will lead to static plastic analysis which will provide adequate indication of barrier deformation response. Clearly both barrier strength and displacement capabilities will have to be properly specified and knowledge of the actual dynamic effects is essential for successful quasi-static analysis and design.

The approach suggested here consists of deciding on an equivalent static load system to be applied to a barrier and then, by static analysis design the barrier for adequate strength and displacement resistance. The loading must be obtained from statistical data, the design should be based on static methods but incorporate strength and stiffness capacities known from dynamic information, and properly reflect the true response characteristics with respect to bending, shear and axial strains. It is realized that such a simplified approach will lead to shapes, proportions and details which must be proof tested. A successful test program could be built up based on laboratory tests on elements and joints followed by simulated impact on composite barriers performed in the laboratory and then a limited number of field impact tests.

These proof tests can properly be used to examine the design methods and capacity of a few barrier systems. However, with adequate knowledge on this subject tests to compare optional systems must be developed. These test. should not be field vehicle impacts but closely controlled experiments with reuseable impact vehicles at definite velocity vectors. The first part of the test program in which checks on design procedures are carried out bears resemblance to the suggestions of National Casting Company Report (----, 1963). This report outlines an endeavour which is much more extensive than the first part proposed here inasmuch as the report incorporates design and comparative testing in one program. It is believed that if the design approach is tested and checked first and then the comparative proof testing developed, a more economical solution will be obtained. The comparative proof test vehicle might be based on the adjusted automobile developed at the University of Minnesota (Ryan and BeVier, 1960) and the G. M. sled together with the sled vehicles described in the various Stapp Conferences. Such a vehicle should incorporate similar energy dissipation to that in normal impacting vehicles, but this can be obtained by a hydraulic dash pot system rather than a crumbling mechanism.

Little detail has accompanied these suggestions because it is believed that a progressive approach is necessary in the design and testing techniques. This can be best developed without the benefit of inflexible detail but within the bounds of clearly stated objectives. The objectives stated here will lead to reasonable and empirically checked design methods for understood loadings and to comparative similitude tests for subsequently introduced designs. In order that these objectives are clearly available they are grouped in Table 16.

Objective	Treatment	Specification
Loading on barrier	Examination of bridge barrier impact statistics. Determination of equivalent static load.	Equivalent static load
Response of barrier	Effect of configuration a) rigid barrier b) flexible barrier from model and full scale tests.	Heights, rail and post spacing
	Origin of dynamic strains and magnitude. Model test to indicate effects of bending and axial strains. Full scale tests to determine magnitudes and durations.	Analysis statement for combination of bending and axial strains.
Structural strength and stiffness	Yield and displacements at strain rates of actual barrier. Laboratory work.	Allowable strength and displacement data.
Examination of actual designs	1) Full scale tests to determine adequacy of a few designs. 2) Comparative proof test in laboratory environment.	Restraints on proof test performance.

Comparisons with new designs	Adequacy with respect to design and satisfaction of comparative proof stress requirements.	Statement of equivalence

Table 16. Proposed Barrier Development Program

It should be noted that the proposal virtually calls for two types of barrier specifications. One for rigid parapet barriers and the other for beam and post type flexible barriers. Clearly the two will have different performance requirements.

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