# TRAFFIC SAFETY ON BRIDGES

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## ABSTRACT

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Although there has been considerable research carried out on Traffic Safety on Highways, as a whole, very little emphasis has been placed on Traffic Safety on Bridges in particular. The purpose of this Report is to explore existing devices and ways which are being used on Bridge Structures for the safety of traffic, how these methods and criterion can be improved, and what means should be adopted in the future to improve the traffic safety on bridges.

ACKNOWLEDGEMENTS

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TABLE OF CONTENTS

## LIST OF FIGURES

# LIST OF TABLES

# INTRODUCTION

In recent years there has been more and more emphasis on the safety of traffic on bridge and overpass structures. The bridge structures are designed to provide safe crossings over rivers, valleys, railway lines and other traffic obstacles. Bridge structures generally serve their purpose well, but most surveys concerning traffic flow on bridges have shown that many bridges, by nature of their structure create hazards to the traffic. The object of this report is to discuss such problems and shed light on deficiencies in the planning and design of bridges. This is important as it is closely related to the safety of traffic.

The main emphasis of this report has been put on physical planning and structural aspects of bridges. Preventive measures are also discussed as they help in reducing accidents and thus improving the flow of traffic.

This report, for objective discussion of the problems, is being limited to "Highway" bridge structures and their related components which influence the safety of traffic directly; in particular, bridge superstructure is the main topic of discussion.

The major discussion in this report on the traffic safety on bridges, is due to the fact that the bridge itself is only one factor among many that may cause accidents and jeopardize traffic safety. The discussion of some other factors such as inexperience or impaired drivers or faulty vehicles, is obviously beyond the scope of this Report, however, it is suffice to say that other sources of trouble could be minimized through proper training and strict observation of the traffic regulations.

Finally, this report explores the possibilities of employing innovations from other technical fields and to apply them as preventive measures on a bridge structure. These techniques, such as "Buffering System" and "Dragnet Vehicle Arresting System" are discussed to show how they could be used in minimizing accident severity on future bridges.

# 2. BRIDGE PLANNING

## 2.1. General

The big increase in traffic volume and speed of vehicles in recent years has resulted in a large number of accidents; therefore, the need arose to give more attention to reduce the hazards which may be causing accidents on approaches to bridges or on bridge structures. Although a great deal of research related to highway safety design has been conducted, relatively little research work has been done on traffic safety on bridges.

Bridge engineers have only in recent years, begun to apply new techniques and devices to improve the traffic safety on bridges. The emphasis in the past has been on the vehicle with little mention of bridge structure other than brief comments in police reports and newspaper articles such as "the vehicle was flipped by the guardrail, or car travelled too fast on a curved bridge". This type of description provided little useful information towards geometric or structural design improvement of bridges. For a more detailed examination of environmental factors contributing to the increasing number of accidents on bridges, engineers were forced to review their planning approach. Now they are not only expected to design new, safe bridge structures, but also to improve the conditions on the existing bridges to achieve better safety records; therefore, it has become the designer's prime obligation not only to provide safety features on existing bridges, but also to plan and implement new devices to build safer crossings in the future.

2.2. Roadway on Bridges

The safe crossing record of a particular bridge or overpass has a definite realtion with its planning and engineering design. Roadways on bridges require proper width, gentle side slopes, proper drainage provision, and a skid resistant surface. For example, a narrow bridge on a relatively wider highway system may cause accidents due to its incompatibility with the highway. From a safety point of view, functional bridges should not make their structures too apparent or cause a restrictive feeling to the driver. The change in roadway surface or width at the bridge crossing should be such that there is no significant change in the drivers' behaviour. Any unexpected change in the vehicle's speed or direction, may result in discomfort or even in an accident.

2.3. Roadway Clearances

Generally, wide bridges are relatively safer and give a sense of freedom. Parapet walls, guardrails, and light poles placed too close to the pavement may be hazardous and cause drivers to shy away and make abrupt moves.

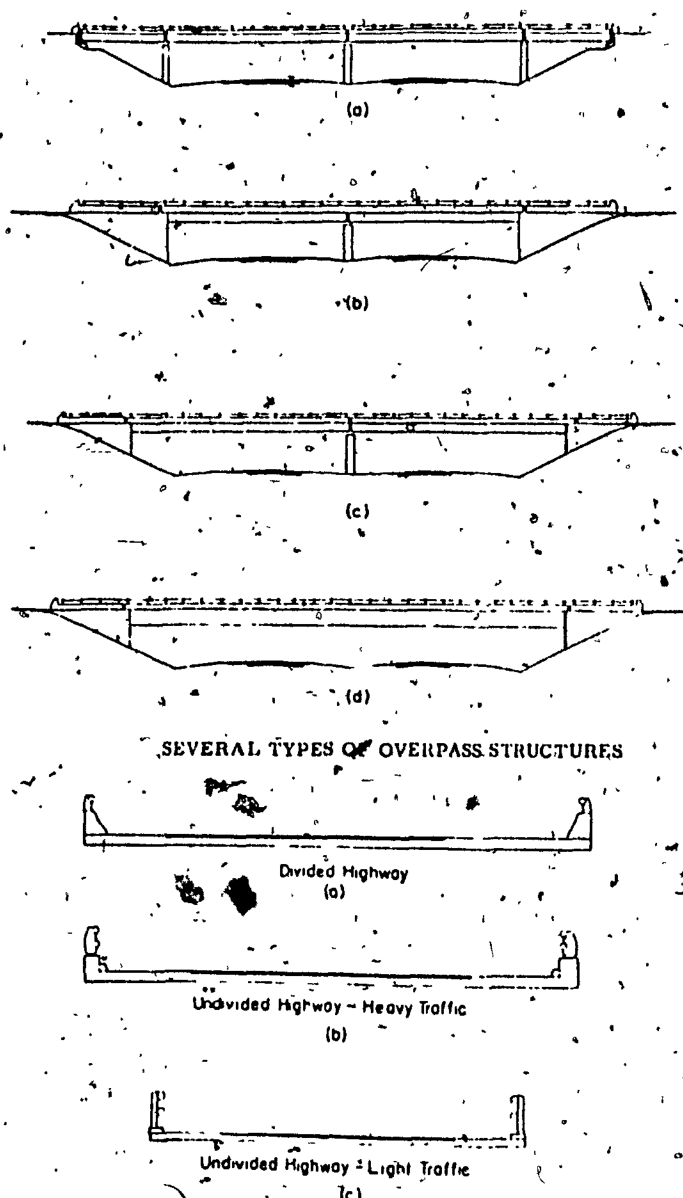



FIG. 1. BRIDGE ROADWAY ELEVATIONS AND CROSS-SECTIONS [1]

The clear width of the roadway on bridges and overpasses should be as wide as the approaching pavement and shoulders. In the case of long bridges, where large overall costs do not allow this, some compromise may be necessary. Reduction in width of long span bridges may also be possible considering the drivers' tendency to be alert and drive more carefully. On the other hand, short bridges are relatively less expensive and in this case there is no justification to sacrifice safety by keeping the roadway narrow in relation to the approaching roadway.

2.4. Sidewalks

Provision often must be made for the pedestrian on the crossings, particularly on long structures that span over wide highways, watercourses, or railroads close to the urban areas. Sidewalks should only be provided on a bridge where there is no other separate pedestrian overpass available, otherwise it could contribute to congestion and effect the safety of both vehicles and pedestrians.

2.5. Shoulders, Curbs

Where there are no curbs on the approaches, the curb to curb width on the overpass structure normally is a continuous pavement wider than that of the travelled way on the approaches. This additional width of pavement would need special treatment to reduce hazards and provide proper surface to the travelling vehicles. On long bridges the curbs are usually located within the shoulder width of the highway. The approach end of this curb should be extended on the approach shoulder and gradually curved away from the travelled way to or beyond the shoulder line. This is both to increase the visibility and to warn or deflect an out of position vehicle back into the travelled way. On the far side of the structure the curb should be curved away, and not terminated abruptly.

## 2.6. Horizontal Alignment

Many bridges are being built with curved alignment to match the layout and the geometry of the highway system. In the design of a curved bridge, it is necessary to establish the proper relation between vehicle speed and curvature, and also their joint relations with superelevation.

### 2.6.a. Side Skidding

Side skidding on curved bridges depends upon a number of elements among which the more important are: the speed of the vehicle, condition of the roadway surface, and the type and condition of the vehicle tires. In general, the maximum side friction factor for good dry pavement ranges from about 0.5 at low speed to approximately 0.35 at high speeds. The coefficient of friction that can be utilized with safety by the majority of drivers becomes quite important for design. The design should be based on safe structures in operation with reasonable maintenance. One of the criterion in providing maximum side friction factor can be the point at which the centrifugal force causes the driver travelling at a high speed to recognize a feeling of discomfort and instinctively acts to slow down.

2.6.b. Geometry

One important element of horizontal alignment design is the clear distance on curves without sight obstructions, such as large sign boards or high quadrails. Design to provide adequate sight distance may require adjustments in the normal highway cross section or change in alignment. Considering design speed and proper minimum sight distance as controls, the actual condition should be checked to provide adequate sight distance for future bridges.

There are design controls recognized in practice which are important for the achievement of the safe and smooth flow of traffic over the curved bridge structures.[1] For example, excessive curvature or poor combination of curvatures can contribute toward accidents, limit capacity, cause losses in time and increase operating cost. Therefore caution should be exercised in the use of compound circular curves. Any abrupt change in alignment near the highway and bridge junction should be avoided. Such a change makes it difficult for a driver to keep within his own lane. To avoid the appearance of inconsistency, horizontal alignment should be co-ordinated carefully with the general highway profile.

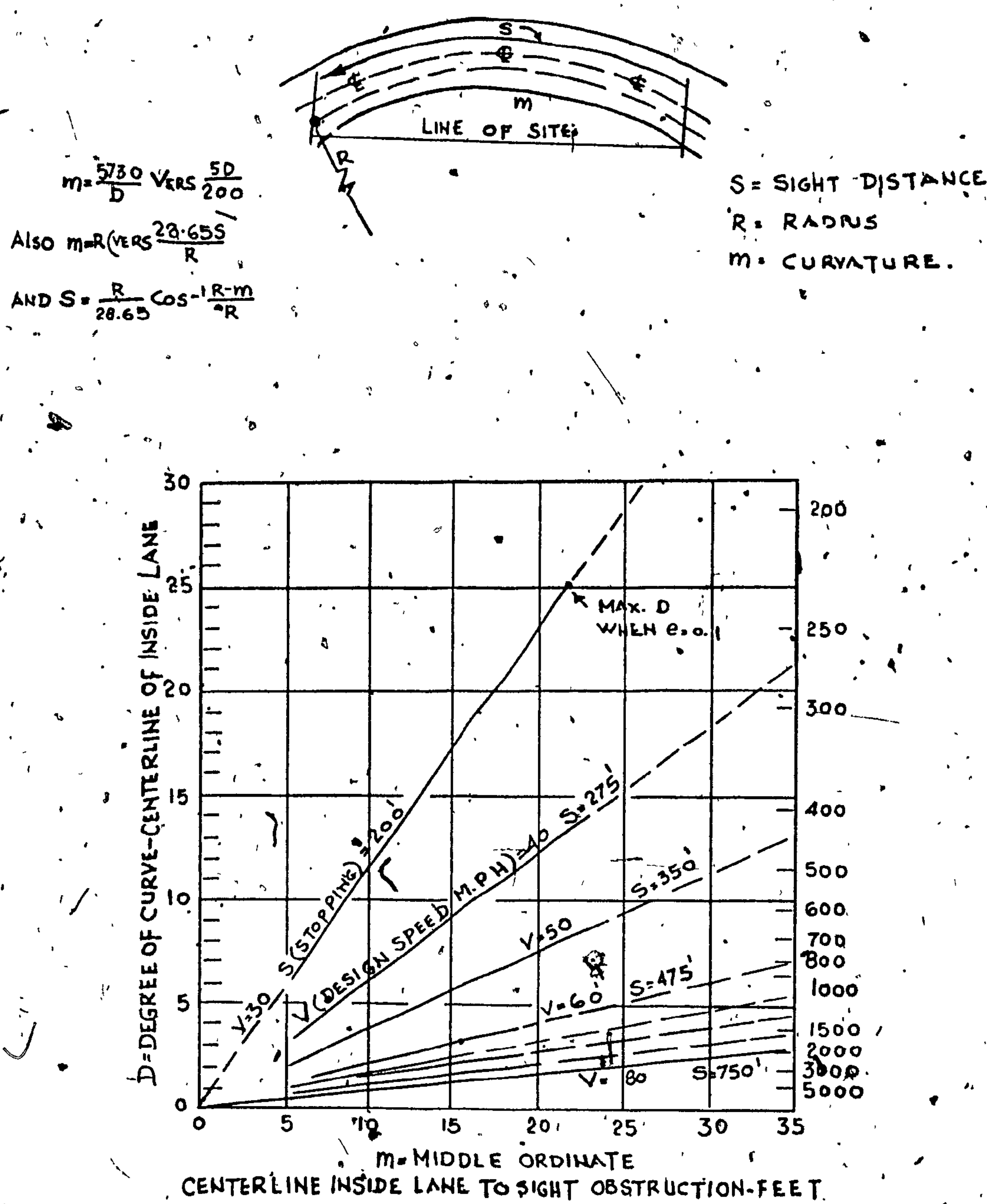



FIG. 2 HORIZONTAL CURVE DESIGN [1]

2.7. Vertical Alignment

It is important to consider grades and their impact on the safety of traffic, especially crossings over navigation channels requiring higher clearances. This will call for proper vertical alignment to ensure smooth and safe traffic operations.

2.7.a. Bridge Profile

Statistically, vertical curves have been prime collision locations due to lack of sight distance. Even on modern bridges, with separated traffic lanes, vertical curves cause high numbers of collisions. Often, the effect of grades on truck speeds is much more pronounced than on passenger cars. Slow truck movements on ramps can be annoying to other fast moving vehicles, which may be unable to pass, due to the restricted width of lanes on approach ramps. Bridge approaches, therefore should be designed so that the speed of the trucks will not be reduced to cause an intolerable condition for the other vehicles.

2.7.b. Approaches

Traffic passing over a bridge should be provided with the same degree of utility and safety as on the approaching highway. The presence of the structure itself should not be augmented by low geometric standards that may result in low safety records.

The alignment and profile of the highway near the bridge should be relatively flat with high visibility. The

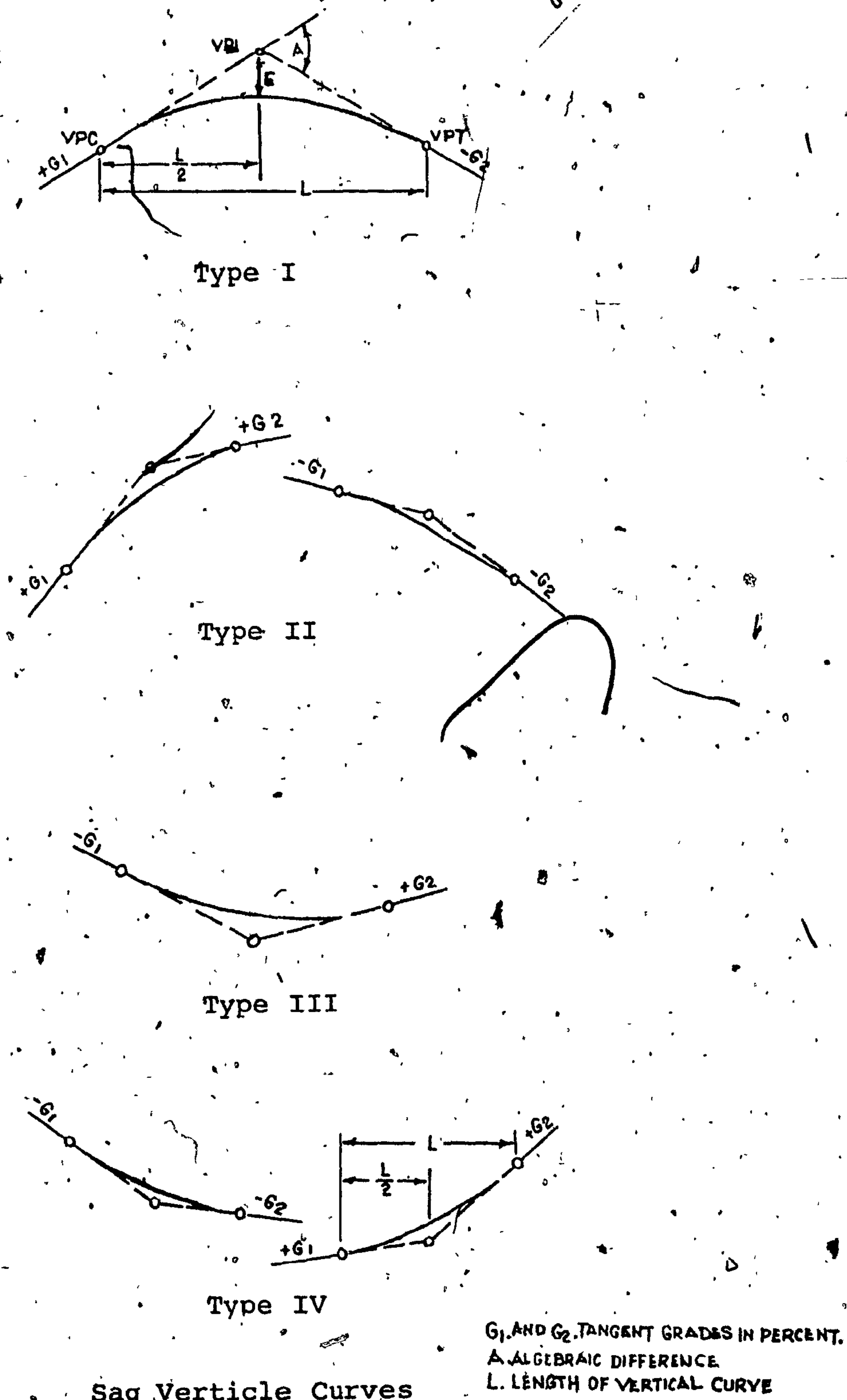



FIG. 3. TYPE OF VERTICAL CURVES [1]

general control for horizontal and vertical alignment and their combination, should be adhered to closely. In particular any sharp horizontal or vertical curves should be avoided. A horizontal curvature that begins at, or near a pronounced crest or sag should be avoided. The gradients which may slow down commercial vehicles or which may be difficult to negotiate when icy, should be avoided. Reduction in vehicle speed by long upgrades encourages passing and this manoeuvre can be hazardous in the vicinity of bridge structures.

2.8. Underpasses

Design of the vertical and horizontal alignments of bridges should be the same as that at any other location on the highway. At the present time, the horizontal sight distance limitations on piers and abutments at curves on existing bridges usually present a more difficult problem than that of vertical limitations. A curvature of the maximum degree for a given design speed, and the normal lateral clearance, at piers and abutments of underpasses should provide adequate stopping sight distance.

Similarily, on bridges with sharp horizontal curvatures for the design speed, sight deficiencies result from the usual offset to bridge rails. If sufficiently flat curvatures cannot be utilized, the clearances between roadways and abutments, piers or rails should be increased as necessary to obtain the proper sight distance.

An increase in construction of structures over existing highways, in recent years is changing the appearance of safety of traffic. These overpasses built mainly to eliminate grade crossings of two roadways, not only eliminate traffic lights, congestions, but also eliminate accidents and considerably improve the traffic safety on both roadways.

As a result of the publication of "Highway Design Guidelines"[2], designers across North America have taken particular notice of potential hazards. This publication specifically favoured the construction of two span overpasses instead of four span structures that have often been built in the past. Two span superstructure is supported by a single centre pier and two abutments. This arrangement of removing the two piers close to the roadway eliminated the danger to traffic. This publication also recommended that the centre pier should stand in a median width of approximately 60 to 80 feet, as opposed to a width of about 40 feet, previously used.

The need for safer underpasses is indicated by recent studies into various aspects of highway safety. According to figures released by the California Division of Highways in 1966[6]. traffic accidents occured most frequently in that State when a driver lost control, ran off the highway and hit a fixed object. Abutments or piers were the most commonly hit objects.

Besides providing greater safety, the change from four span to two span bridge construction produced aesthetically more appealing bridge structure. The clean lines of a two-span overpass, supported by a single pier presents a much more pleasing appearance. This gained importance at the time when the public awareness of environmental beauty and safety of the users was getting more attention.

However, the cost of the superstructure with single span will increase as a result of doubling the span length. The main girders will be heavier and deeper requiring an increase in distance between the upper and lower roadway. Where the median is very narrow it will be necessary to use a structure without a pier and accept the increased costs, but wherever the median is wide enough to permit a safe pier, considerable saving can be obtained. Additional safety can also be achieved by placing a length of guardrail along the piers.

## 2.9. Overpasses

It may be necessary sometimes to consider several alternative layouts to determine if the main road should be carried over or under the structure. At any particular location, the governing condition could be either topographical considerations or alignment and grade line control of one highway. The second case will become important where a major road is so predominant in design that it overweighs topographic and crossroad control. Traffic on the major road may be such a type, and of sufficient volume, that humps or dips in its general grade may endanger the safety of traffic and increase the rate of accidents.

A new roadway crossing on an existing highway should be given preference to go over the structure as a wider view is available from above, and drivers will have minimum feeling of restriction or confinement.

On the other hand, where turning traffic is significant, the ramp profiles will be best fitted if the major road is at the lower level. The ramp grades then assist turning traffic to decelerate as they leave the major highway. Merging traffic from ramps will have the opportunity to accelerate as they approach.

When there is no pronounced advantage to the selection of either an underpass or an overpass, the location which will provide the better sight distance on the major road, for safe passing, should be chosen.

Where a new highway crosses an existing road carrying a large volume of traffic, an overcrossing by the new highway will eliminate disturbances and hazards to the existing road.

For a smooth flow of traffic and safe operations at any bridge structure, a uniform design criteria should be developed to a practical and adequate level. Severe grades and sharp curves are hazardous and are also uncomfortable to drive on.

Efforts should be made to provide adequate clearances for various parts of the bridge system, depending on the volume and the type of traffic.

## 2.10. Use of Models

In recent years different types of models are being used in the planning and designing of bridges. Usually the cost of a model is small compared to the overall cost of the bridge structure. Thus, for a small expense and working time benefits obtained through having a scale model as a tool with which to work are worthwhile. Any faulty design or possible traffic hazard can be spotted quickly on a model, which may go unnoticed on the paper plans and profiles.

Decisions can be made as how the bridge will fit to the site and its related structures such as ramps, approach roads, etc. Models can also highlight the comparative features of various types of bridges, overpass approaches, pier dimensions, end spans and skewed layouts proposed for a particular location.

From a safety point of view, it is important that drivers of vehicles should not be confronted with a view of a "structural forest" of turning roadways, massive piers or abutments to confuse him while he is making a decision of passing either under or over a structure. Again, the use of a model provides a ready means to avoid such undesireable features. Discussions held around a model are more productive when each team member has a clear view of the object under discussion.

FIG. 4(a) MODEL OF A COMPLEX INTERCHANGE [4]

FIG. 4(b) OVERPASS MODEL [4]

The modern trend in bridge design is towards the safety of the flow of traffic. A model can quickly reveal locations and details where drivers' safety is questionable. Models do not replace the plans, profile and cross-section, rather they complement traditional methods by presenting these three views simultaneously. Therefore, they provide an opportunity to introduce any changes or improvements which could be possible.

It is observed that the full potential of models, as a design tool, is not yet fully recognized. Other possible uses of the model to investigate locations of signs and lighting devices, also need further study before the importance of models in these fields can fully be appreciated. It is hoped that in future, bridge planners will make the full use of bridge models, and apply their findings in improving the safety of future bridges.

# 3. BRIDGE STRUCTURES

## 3.1 General

Bridge structures in general, and superstructures in particular, are responsible for the safety of traffic passing over them. Bridge structures are designed to carry traffic loads, as well as all other live and dead loads. The majority of bridge structures do fulfill their tasks, but there have been numerous occasions in the past, when bridge structures have failed to withstand these loads and collapsed, causing heavy losses of property and human lives.

If failure of a bridge structure is defined as a result of collapse, there are few failures, but if non-conformity with design expectations is the criteria, [3] then there are many failures. There may be a single and simple explanation for a failure but usually it is a combination of conditions such as mistakes, oversights, misunderstandings, ignorance and incompetence, on part of either designers or builders.

Bridge failures in recent years have obliged Engineers to assess their design assumptions, construction techniques, and material reliability through strict quality control. Continuous pressure for greater economy, both in design and construction had often resulted in safety being reduced to a minimum.

There is no doubt that with advanced and sophisticated design methods, application of computers and utilization of high strength materials, more economical bridge structures can be built. But the main consideration should always be the safety of traffic, which cannot be sacrificed as a price for progress and economy.

3.2. Concrete Bridges

Progress in the manufacture of high strength reinforced concrete has created a fresh thinking in the design philosophy of bridge structures. Working stress design methods can restrict the efficient use of high strength steel in concrete. This is probably one of the reasons for the wider use of prestressed concrete structures where such a use is possible even within the limitation of working stress design. This development has resulted in the production of better bridges. Materials have created a necessity for a revision in the maximum values of admissible stresses for concrete and steel of different qualities and grades with respect to the factor of safety. Bond characteristics between steel and concrete have also been improved further by the use of deformed bars, but bond efficiency for quantative measurement point of view is still a matter of research.

In recent years prestressed box section has become a standard structural member for all slender long span bridge structures. These box type bridges, because of their considerable torsional strength, allow themselves to be used in skew and curved bridges. Diaphrams are often used but the number and size of diaphrams in a hollow girder should be limited to the minimum required for structural safety. These diaphrams, in addition to adding dead load, also cause difficulties in construction which could cause poor workmanship and contribute towards production of unsafe bridge structures.

3.3. Fatigue and Stress in Concrete Bridges

The amount of information on the fatigue properties of reinforced concrete bridge beams is limited, but it indicates that the possibility of fatigue failure in normal service is remote, since the stress range is likely to be well below the endurance limits for both steel and concrete. [9] Some tests conducted at PCA labs however, indicate a need for further study when high strength steel is to be used. On a tentative basis, it may be said that fatigue will not be factor in normal concrete bridge. The favourable behaviour of prestressed concrete with regard to dynamic loading and its freedom from harmful cracks makes it a highly desirable bridge building material.

While it is important to induce correct prestress in the concrete when curved cables are used, it is not easy to control the prestressing force at the centre of the girder within predicted error limits. When the dead load to live load ratio becomes large, as in long span bridges, a small amount of prestressing error can cause appreciable error in concrete properties. Variation in section area of cable, variation of Young's modulus of cable, along with the error in measurement of elongation and in the reading of the pressure gauge, all collectively can cause a grave error. Therefore this aspect should be adequately investigated and considered both during design and construction. Concrete bridge structures, although they cannot yet be designed to extend over such great spans as modern steel bridges and especially the suspension bridges, have not entered the region where aerodynamic forces will be important.

Deflections caused by live loads are unlikely to be a problem, if vibrations caused by such loads are not objectionable.* The dead load deflection can be eliminated by provision of camber. If relatively large spans to thickness ratio are used from the standpoint of strength, it may be desirable to place a suitable limit on deflection or slab thickness. At present, such restrictions seem unnecessary as the "Limit State Design" calls for a check on the loss of stiffness resulting from creep, skrinkage, plasticity and cracking. In addition, a check on local damage may also become necessary.

## 3.4. Steel Bridges

On December 15, 1967, a 40 year old eye bar chain suspension bridge over the Ohio River collapsed without warning. At the time of the collapse, the bridge was being used by peak traffic and the sudden bridge failure plunged a large number of vehicles into the river below, causing heavy losses.

This suspension bridge, known as "Pleasant Bridge" had a different design compared to other suspension bridges. Each cable consisted of a pair of eye bars, instead of wire stands, and was connected by pins at 50 foot intervals. The eye bar cables formed the top chord of the stiffening truss over a portion of the 700 feet of centre span and 380 feet of end spans, and the base of each tower was resting on rocker shoes to permit the entire tower to rotate slightly under unbalanced loads.

The eye bar steel had significant properties; the material used was heat treated carbon steel with yield strength of 105 ksi, elastic limit of 75 ksi and working stress of 50 ksi. The initial failure which was attributed to the collapse was the crack that developed in the eye bar head due to the stress concentration caused by friction. (Certain ultra high strength steel with yield stress above 80 ksi are suspected[4] to accelerate cracking while under tensile stress and under certain environmental conditions. The effect is called "stress corrosion cracking").

As the nominal stress in the shank of the eye bar approached the allowable stress of 50 ksi, the actual stress at the edge of the hole was estimated at about 85 ksi (well above the specified 75 ksi mark). Both analytical and experimental evidence indicated a stress concentration factor at the edge of the hole where the crack started. Higher stresses may have existed in localized areas as indicated by a study of residual stresses created by heat treatment and loading history. Thus, at the critical point, the steel was likely stressed well beyond yield, and localized cold working probably occured, increasing brittleness susceptibility to stress-corrosion and corrosion fatigue.

Metallurgical examination of the cracked surface with the scanning election microscope and other techniques indicated that the mechanism of crack growth was definately associated with a corrosion process. It could not be established, however, whether this was stress corrosion as defined above or was a process of corrosion fatigue in which the cract propogation is caused by repeated loadings producing a range of stress in the presence of a corrosive element.

### 3.5. Aerodynamic Loads on Steel Bridges

Tacoma Narrows bridge was one of the spectacular failures due to wind action in 1940, after only 4 months in use. However, it was not the first bridge to become the victim of aerodynamic instability. Several cable suspension bridges without the diagonal stays, used by Roebling in Brooklyn, required stiffening and correction to overcome oscillations. [8]

The Tacoma Bridge design was checked for wind loads with a model. These tests showed some vibratory oscillation and lateral deflection. However, no torsional oscillation under strong wind loads was predicted. Before opening the bridge to traffic, the diagonal cables were added to connect the stiffening girders to the cables at midspan, and dash-pot buffers separated the towers from the floor system, all to absorb longitudinal vibration. A month before the failure, back cables were added between the anchorage and the side spans. None of these protections could prevent the bridge from going into "cork screw" motion which caused the failure of the superstructure. The maximum wind velocity at the time of collapse was only 42 MPH, much lower than the designed wind loads.

Photographic record of the torsional oscillation of the Tacoma Narrows Bridge provided more exact date for aerodynamic investigation of the structure than any other report. The main span was 2800 ft. with two side spans of 1100 ft. each. The superstructure was 39 ft. wide with stiffening trusses only 8 ft. deep. The bridge was designed at maximum efficiency of materials to meet the tight budget. The two lane bridge was too slender for such a large span. The ratio of span to truss depth, a measure of stiffness, was 350, almost double the 168 ratio in the Golden Gate Bridge, which also had experienced some oscillation under cross wind loads.

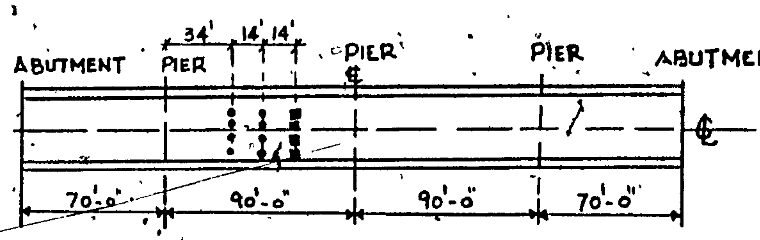
## 3.6. Investigation Work

### 3.6.a. Ultimate Capacity of Bridge Structures

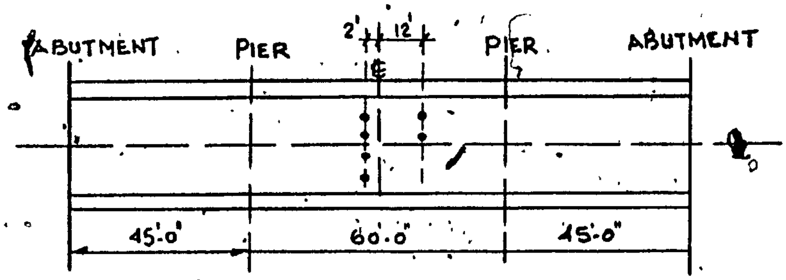
Four deck girder Highway bridges in Tennessee valley located in an area to be flooded as part of a TVA reservoir, were tested to failure in 1970. The ultimate load for each bridge was measured. Also the load causing the first permanent crack was computed and compared with the measured load. These "Life Size" test models provided an excellent opportunity to test and compare the results of design loads with actual loads which caused permanent damage. Each bridge failed in flexural mode. Composite action was lost in the prestressed concrete bridge prior to flexural failure, with a resulting reduced load capacity. The loads based on AASHO specification gave a lower bound to the actual ultimate load for each bridge.[10]

The present approach in the specification of American Association of State Highway Officials is towards the use of "Load Factor Design" for deck girder bridges. This design assumption is based on the prediction of the ultimate capacity of the individual bridge girders along with the amount of overload that would cause first permanent set and fatigue considerations.

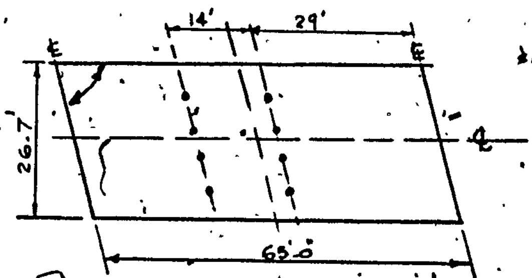
This research project was a perfect setting to assess, through tests on typical highway bridges, the accuracy with which the bridge designer is able to predict the ultimate bridge capacity and load causing permanent damage.



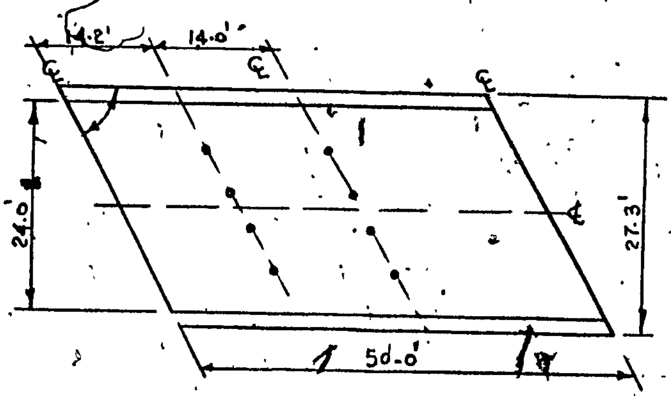

BRIDGE 1.




BRIDGE 2



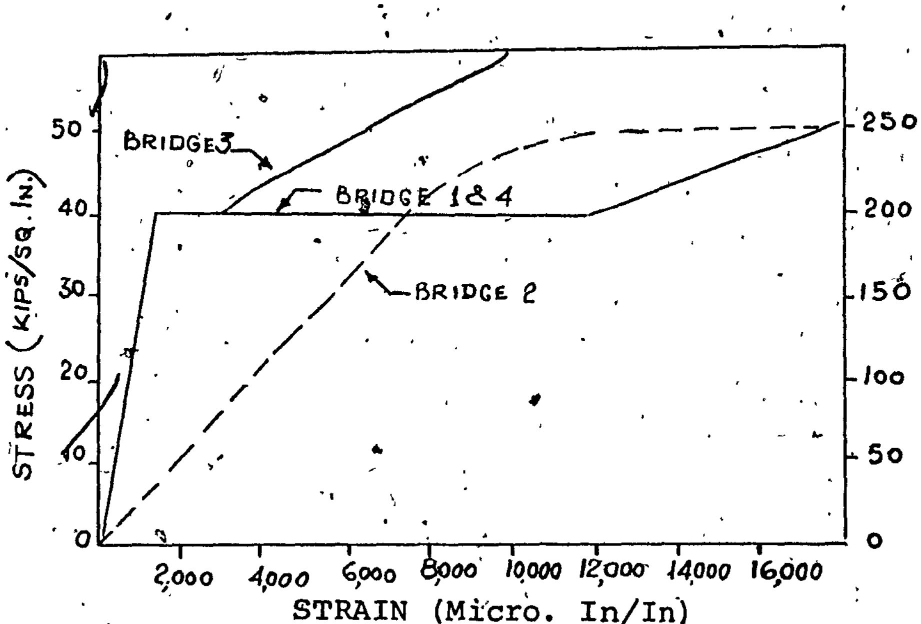

BRIDGE 3.




BRIDGE 4.

• LOAD POINT
■ 4 KIP. LOAD

FIG. 5. POSITION OF LOADS USED IN TESTS [10]




STRESS-STRAIN CURVES FOR STEEL

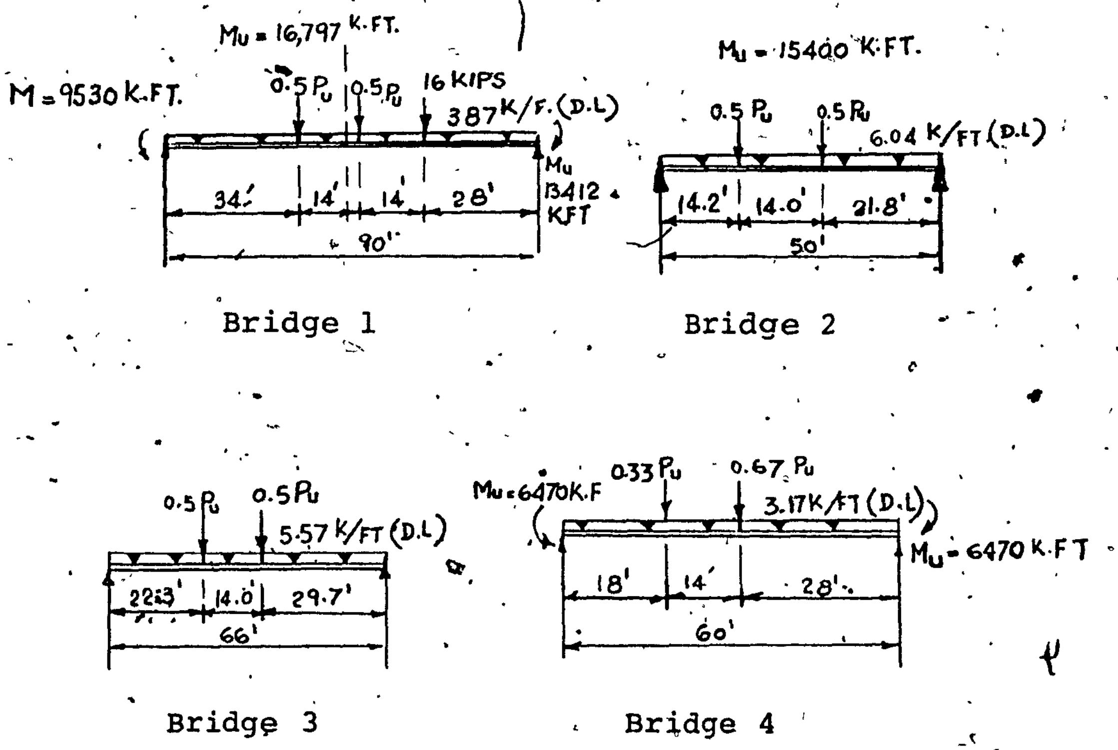



FIG. 6. LOCATION OF APPLIED LOADS AND MAGNITUDES OF CALCULATED ULTIMATE MOMENTS. [10]

A description of all 4 bridges is given in table 1. Each of the bridges were 2 lane deck girder bridges with 4 longitudinal girders. From a testing viewpoint bridges 1 and 4 were the most useful of the four bridges. Bridge #1 was a flat sag curve, otherwise identical to bridge #4. Bridge #2 composed of AASHO type 3 precast, prestressed sections, was also of recent design which was a widely used type. Although bridge #3 was not of recent design, its reinforced concrete T-Beam construction was representative of a number of bridges at present in use throughout North America.

The ultimate load carrying capacity of a bridge subjected to flexural loading depends not only on its flexural capacity but also on the position of the applied loads. For the tests, the loads were placed in such a way as to simulate a HS loading in the position resulting in maximum positive moment near the centre of a span. The loads were assumed for calculation purposes to have a uniform lateral distribution. In determination of the theoretical ultimate capacity, each bridge was assumed to act as a unit.

The 1971 interim specification of AASHO was used as a basis for calculations of the ultimate capacity of bridges 1, 2 and 4. These specifications do not provide for the determination of ultimate capacity of reinforced concrete bridges such as bridge #3. Therefore the values of ultimate capacity for bridge #3 was calculated by using the

TABLE 1. DESCRIPTION OF BRIDGES [10]

| NO. | GENERAL DESCRIPTION | SPAN | GIRDER SPACING | SKEW (DEG) | LOCATION | DESIGN LOADING |
|---|---|---|---|---|---|---|
| 1. | 4 span continuous 36" steel rolled beam. composite in positive moment regions. | 70'<br>90'<br>70' | 8'- 4" | 90 | Tenn. 130 over Elk River | 4-20,1963 |
| 2. | Simple span composite with AASHO type 3 precast prestressed concrete beams | 66' | 8'-10" | 70 | Tenn. 130 over boiling Fork Creek | HS-20 1963 |
| 3. | Simple span reinforced concrete T-beam monolithic construction | 30' | 6'-10" | 60 | US-41A over Elk River | H-15, 1938 |
| 4. | 3 span continuous non-composite 27" steel rolled beams | 45'<br>60' | 7'- 4" | 90 | Manford Rd. over Elk River | H-15 1956 |

TABLE 2. MEASURED AND COMPUTED RESULTS [10]

| NO. | Av. Ulitmate Comp. Strength (p.s.i.) | Ultimate load Measured (kips) | Theoretical (kips) | AASH (kips) | Centreline Defl. (in.) | Load Causing First Permanent Set Measured (kips) | Computed (kips) | Permanent Defl. (in.) |
|---|---|---|---|---|---|---|---|---|
| 1. | 5800 | 1250 | 1270 | 930 | 22.8 | 620 | 714 | 0.25 |
| 2. | 5500 | 1140 | 1267 | 1100 | 9.5 | 660 | 759 | 0.12 |
| 3. | 8500 | 1580 | 1465 | 844 | 7.3 | -- | -- | -- |
| 4. | 5600 | 640 | 695 | 388 | 26.4 | 500 | 376 | 0.00 |

general method presented for determination of flexural capacity in the ACI code.

3.6.b. Failure Mode

All bridges except bridge #2 failed in flexural mode. Each bridge behaved in a ductile manner.

Bridge #1

The behaviour of this bridge was almost linearly elastic up to yielding point, at the section under the applied loads nearest to the centre of the span. As the load was increased, there was considerable rotation at this section and in turn, considerable deflection. Shortly after yielding began and the loads were increased further, the bridge lifted off the abutment nearest the applied load, thus making it impossible to develop more moment at the first pier. The bridge continued to experience increasingly large deflections for each load increment until, after a very large deflection, yielding occured and a plastic hinge formed at a section near the centre pier at the end of the "cover plates" on the side of the pier away from the loaded span. Shortly after this hinge formed a secondary compression failure of one of the curbs occured at the section of maximum positive moment, and the test was ended.

Bridge #2

This bridge acted in a predicted way up to an approximate load of 950 K but there was a considerable amount of "dishing" of the bridge at this point and interior girders were deflected more than the outside girders. The outcome of this dishing was a tendency for the bridge deck to separate from the interior precast girders.

At a load of approximately 950 kips, this separation occured and immediately composite action of the interior girders was lost. The behaviour of the bridge was radically changed. Almost immediately there was a crushing of the extreme top fibre of the interior precast sections at the sections of maximum moment. This crushing and accompanying rotation resulted in a redistribution of moments at the section and an increase in the moment in the exterior girders. As the load was increased further the interior girders failed in shear and the test was terminated.

Bridge #3

This bridge designed for the equivalent of an H-15 loading behaved elastically up to very high loads, and it was not obvious enough when yielding first occured. The reason for the absence of a clearly defined yield load is related to the stress - strain curve for the steel bars in all members at a cross section. The strain in the most highly stressed bars would increase to the strain hardening region while other bars were reaching yield.

Bridge #4

The load deflection curve for this bridge closely resembles that for a typical intermediate grade structural steel. The stiffness of the bridge up to yield was considerably greater than predicted for a new composite bridge because partial composite action existed up to yield. Failure of the bridge was initiated by yielding at the section of maximum positive moment. After this occurance, there followed considerable rotation of the resulting plastic hinge and very large deflection with only a nominal increase in load. Then plastic hinges formed near the 2 piers on the sides away from the loaded centre span, and further deflection took place with a reduction on load capacity.

3.6.c. Code Applications

The ultimate loads predicted by the specifications were in all cases less than those measured for bridges 1 and 4. The reason for the relatively low value predicted by AASHO specifications is the fact that no redistribution of moments at ultimate load was considered. The ultimate load as calculated by using AASHO specifications for bridge #3 was much lower than the measured value is most likely due to the fact that the maximum steel stress was taken as that yield. Actually, because of the short yield plateau for the steel and the fact that a low percentage of steel was used, the steel stress at ultimate was much above yield.

One important conclusion was noted with respect to AASHO specifications; that it recommends much lower values compared to the actual ultimate capacity of each of the 4 bridges listed. However, in this experimental work only conventional type bridges were tested. While new types of sophisticated and more complex bridges are being built, research work is needed to test actual size bridges against their design assumption and see how materials would behave under actual loads and stresses.

3.6.d. Investigation Conclusions

For all bridges except bridge #2, the theoretical method described earlier predicted within a low error the ultimate capacity of each bridge. Value predicted for bridge #2 was significantly higher than the measured value because of the loss of composite action in the interior girders. The mode of failure for each bridge, again with an exception of bridge #2, was the same as that predicted.

## 3.7. New Design Techniques

Box girder steel bridge design was made famous by the English consulting firm of Freeman, Fox and Partners when they used it for Severn Bridge several years ago.

Westgate Bridge (Melborne, Australia) a steel box girder bridge collapsed on October 15, 1970.

The bridge also designed by Freeman Fox and Partmenrs, had a cable stayed box girder with a 1,102 ft. main span flanked by 472 ft. of suspended side span and 367 ft. unsuspended side span. Half width steel box girder sections had been installed in place when failure occured. According to the report, on the day of the collapse, workmen had removed a number of steel bolts from a tranverse splice in the upper flange of the girder to correct a buckle. This buckle had resulted from placing a concrete block on the upper flange to correct a difference in camber between the two halves of the span.

The Royal Commission listed inadequate supervision of inexperienced queries and responsibility for removal of the bolt on the day of the accident as errors by Freeman Fox. The Commission pointed out that if the safety factor had been adequate, the span would not have buckled under the weight of the concrete block, placed to correct the camber difference. Had this buckle not occured, there would have been no need to take bolts out of the bridge on the morning of the collapse.

<u>Milford Haven Bridge</u>, also a welded box girder bridge designed by Freeman Fox is being built over Cleddan River on the western most tip of Wales. Failure of highly stressed diaphram plate of this bridge structure has been blamed for the fatal collapse of the 252 ft. long span.

Calculations made after the collapse showed that the diaphram would begin to buckle when the working reaction at the pier, supporting the contilevered span reached 500 long tons, and that it would fail under a reaction of between 900 and 1300 long tons. At the time of the collapse, the reaction was 963 tons, just inside the failure range.

Unlike the Severn Bridge, the Milford Haven Superstructure is unstayed and depended on depth of girder for stability.

The investigation report pointed out three "unforseen hazards":

1. The diaphram could have been as much as 3/4" out of flat, making it susceptible to local buckling.

2. Bearing on the Pier was out of the line with the neutral axis, and may have imposed a bending moment on the diaphram.

3. Some bolts that were supposed to be loose were tight and under movement of a load may have torn longitudinal stiffeners loose from the bolt on flange making it unstable in compression.

Not enough is known yet about the exact behaviour of thin walled welded box girder bridge structures. As a result of these bridge failures, strict design requirements were imposed in Britain by the Morrison Committee. Model studies are being conducted to verify the recommendations.

The new rules will emphasize web and diaphram where most stresses occur under various loading conditions and temperatures. The Morrison Committee reported that some existing bridges could be unstable under steady winds and that a minor change in deck shape could make a substantial difference. It was also recommended that the weld size should be kept to a minimum to reduce the heat input that causes lock-in stresses. It was discouvered that flange width ratio of 55 used in the girder, became critical with respect to residual stresses.

# 4. PREVENTIVE MEASURES

## 4.1. General

More and more attention is being given to improve techniques and devices which help to reduce the number of accidents, and provide a safer movement of traffic over bridge structures. Bridge structures in the past were designed and built for a much smaller volumne of traffic moving at a much lower speed. In recent years, however, heavier and faster moving vehicles have created problems of a bigger nature. This chapter mainly deals with items and devices which if properly designed and used will definitely help to improve the traffic safety on bridges.

## 4.2. Lighting

Inadequate lighting on bridge structures will affect the safety of traffic at night considerably. Statistics have proved that the night time accident rate is significantly higher than during the daylight hours, which to a large degree, may be attributed to visibility. [6] There is evidence that in urban areas, fixed sources of light on overpasses tends to reduce accidents. Lighting bridges in rural areas is also desirable but the need for it is much less than on bridges near urban areas. Most modern bridges in rural areas are designed as deck type structures with horizontal and vertical alignments of fairly unrestrictive type so that the vehicle's headlights have the opportunity for maximum utilization.

However, bridges either in urban or rural areas need to be lighted depending upon the layout and traffic volume involved. Bridges with horizontal curves have a problem of the opposite type, as the headlights of vehicles on the other side create hinderance due to a variety of vehicle directions and turning movements; the installation of metalic light screen can eliminate this hazard.

At bridges, it is also desirable and sometimes necessary to provide a fixed source of lighting. Drivers should be able to see not only the roadway ahead, but the entire turning roadway area. Also they should be able to see other vehicles which may influence their own behaviour. Without lighting there may be considerable hesitation and uncertainty on the part of drivers, and this very fact could create some confusion, and contribute to potential traffic hazards.

Where a small bridge is provided with fixed source of lighting, it is desirable that the intensity of light be diminished gradually as the distance from the lighted area increases. This gives adequate time to the drivers of vehicles leaving the structures to adjust themselves to the darkness beyond. This will eliminate the blind interval experienced upon leaving a brightly lighted area. Since eye accomodation for change in lighting requires a long distance, it may be desirable to use low light intensities for short bridges.

Studies have shown that in order to minimize[5] the effect of glare, bridge lights should be mounted at least 30 ft. high. Lighting uniformity can be furthur improved by mounting them at greater heights. Lighting poles should be placed clear of shoulders, and with a barrier curb at the pavement edge, not too close to the edge of the pavement. On bridges in rural area, poles should not be used on the median, unless the roadway is of sufficient width.

Consideration should also be given to the use of reflectorizing devices on the bridge component such as curbs, piers, abutments and rails. The greater the volume of traffic, particularly the turning traffic, the more important and effective these reflectors will be in reducing the number of accidents.

## 4.3. Bridge Surface

Most bridge pavement or surfaces have skid resistant properties when first placed, but loose effectiveness due to wear. Engineers have been studying the vehicle skidding problem particularly resistance to skidding offered by pavement surfaces, for several decades.[16] In fact, initial papers written on the subject 30 to 40 years ago include many present day ideas. Today, as a result of ever increasing traffic volume and speeds, the skidding problem is rapidly gaining significance, Several studies,[18] which have analyzed accident records, indicate that the incidence of total accidents as well as accidents directly involving vehicle skidding increases significantly with decreasing friction co-efficient between pavement and the tire because stopping distance and cornering capabilities are direct functions of friction co-efficient. A high value of friction co-efficient is highly desirable for the roadway surfaces of curved bridges.

There are certain factors which primarily affect the wear-resistant qualities of concrete deck and reduce friction. These factors can be grouped under the following four headings:

a. type of surface finish
b. type of fine aggregate
c. mix proportions
d. construction practices.

Tests[16] have shown that concrete pavement composed of fine aggregates with high siliceous particle content is suited to withstand polishing and wearing effects over a long period of time. This siliceous particle content is very important, and a high percentage should be used.

The range of co-efficient of friction values found on Portland Cement Concrete can be sufficient to render some concrete pavement slippery when wet. Mix proportions have been found to have a bearing of friction properties. Control of water cement ratio, sand content, and curing practices are particularly important. A majority of concrete pavement contains polished lime stone, river gravel and a suitable finish which will provide a surface which will wear slowly.

It is generally accepted that there are two kinds [12] of pavement textures or roughness which contribute to friction resistance; these are (i) Macrotexture, which is a result of factors such as the roughness built into pavement surface; (ii) Microtexture, which on the other hand, is the roughness inherent in the aggregate itself which prohibits or prevents its polishing under traffic.

Macrotexture, needed to ensure minimum drop of skid resistance with speed,can be effectively provided only by a proper surface finish. Microtexture, needed to ensure high skid resistance, also is solely a function of the surface finish. Thus the use of a proper surface finishing for concrete pavement cannot be over emphasized.

Asphalt pavement, on the other hand, can deteriorate due to a variety of cracking types or other degradation under traffic loads and environmental factors. There has been very little work done on "aging gradient", although it has been suspected for some years that the asphalt at the top of the layer can be significantly harder than at some depth, depending on how long the pavement has been in service.

Cracking of bituminous surface layer will occur when the induced stresses, either externally applied or internally developed, exceeds the tensile strength of the material. The externally applied stresses can occur due to traffic, while internally developed stresses may be associated with temperature changes.

In order to prevent some of the potential hazards associated with pavement cracking on bridges adequate thickness of surfacing materials should be used to support the traffic load.

## 4.4. Guard Rails

Railings were provided on early highway bridges for pedestrians and slow moving vehicles. Collisions were uncommon, impact forces were small, and replacement costs were minor. However, over the years with high speed vehicles, and a greater volume of traffic, problems have become a major concern. Already, many bridge railings constructed a few years ago, are no longer considered structurally sound if subjected to the greater impact forces. Through past experiences, engineers have developed a service criteria to apply on bridge railing systems, besides other items, this criteria also takes into consideration the following:

a. Vehicle parameters, such a physical dimensions, weight and speed.

b. Bridge roadway characteristics, such as width and type of surface.

c. Railing type and performance when subject to vehicle impact.

An examination of the statistics for a period of three years revealed that:[5]

a. Approximately 33% of fatal accidents on bridges involved fixed objects.

b. Approximately 22% of the fixed object fatal accidents involved bridge barrier railing systems.

c. Certain sections of bridge barrier railings constitute a hazardous condition to an out of control vehicle.

At the present time, the hazardous condition of major concern and also responsible for approximately 56% of fatal accidents, is the end of the bridge railing as indicated in Table No. 1, of this, approximately 36% involved bridge ends not protected by an approach guardrail, see Fig. 2a where as approximately 20% involved bridge ends protected by an approach guardrail, Fig. 2b.

The experience indicates that narrow bridges, which have a width less than the approach road and shoulder width present a clear hazard to approaching vehicles. Present design practices however, recommend that shoulder width[3] on bridges within interstate highway systems, should be wide enough, so that in case of an emergency, the driver can maneouver his vehicle. In the United States of America, some states construct approach guardrails adjacent to bridge ends, flared guardrails, and in some cases twisting and tying the approach end of guardrail to a concrete anchor.

It has been demonstrated by researchers conducting full scale dynamic tests that[15] the approach guardrail and bridge rail juncture must possess structural compatibility, as well as alignment compatibility. Therefore, the strength and alignment transition from a semiflexible approach guardrail to a more rigid structural system (bridge railing) must be provided. Current design practice to achieve a strength transition adjacent to the bridge railing has resulted in decreased guardrail post spacing, larger size posts, adequate bolting of the approach guardrail to the bridge structure, and anchoring of the guardrail end.

In the event, when an out of control vehicle is redirected by an approach guardrail, a secondary collision (ususally at a large impact angle) with the bridge railing may result in a fatal accident.

The ultimate point of fatal accidents in this case is merely relocated. Certainly the elimination of the hazardous bridge rail end is necessary, but it must be kept in mind that this may not completely solve the problem. For example, it has been observed that accident frequency may increase as the length of a guardrail increases.

A safe railing system should restrain a colliding vehicle and prevent a vehicle from vaulting, minimize deceleration to a level which will be tolerable to the vehicle driver, and smoothly redirect a colliding vehicle. Penetration of a barrier railing, which constituted approximately 15% of the fatal accidents most probably can be eliminated by adequate guardrail design.

Vaulting of a barrier railing can be eliminated by proper attention to providing adequate railing height in addition to structural strength, and the elimination of discontinuities such as curbs and sidewalks.

Engineers of the State of New York, Department of Public Works,[22] have developed guardrails which are capable of being displaced laterally under collision forces, these produce less severe impact conditions than rigid barrier railings. Therefore, it is apparent that further reduction in impact force can be achieved by providing structures capable of greater lateral displacement of the barrier subjected to collision forces.

## 4.5. Snow and Ice Control

Engineers have, for a long time, realized the safety problems related to snow and ice on bridge surfaces. Highway maintenance organizations and snow removal equipment usually are designed primarily to accommodate general roadway problems, and there has been little attention given to bridges having problems in connection with snow and ice control on their surfaces.

From a physical point of view, adequate snow removal and storage facilities at the proper place is important for safety of the traffic. Bridge shoulders must be wide enough to accommodate temporary snow storage. The excessive snow from overpass decks to the travelled way below can create another problem which should be considered in the design of the bridge parapet and rails. The phenomenon of the bridge deck becoming slippery before the pavement of adjoining roadway is a very important factor that influences snow removal and ice control on the bridges. Melted snow and ice on the bridge decks may also result in a hazard due to a sudden drop in the temperature.

Signs, railings, and other bridge components obstruct the wind under certain conditions, causing drifting across the pavement. Swirling and drifting snow can also obscure drivers' vision. While selecting the bridge elements, the effect of drifting can be minimized by considering the direction and the velocity of the prevailing winds.

Signs, curbs and traffic markings on bridge surfaces can be buried with blown or plowed snow. Under certain weather conditions, signs become obscured by blowing, sticking snow and can confuse motorists, especially those who are unfamiliar with the area.

Sign posts or guardrails located close to the edge of pavement of shoulder may interfere with efficient plow operations particularly when wing plows are used in clean-up operations. A safety hazard is created when the plow must pull out into traffic to plow around an obstacle near the shoulder.

AASHO[1] recommend that roads having the greater volume of traffic be designed to go over the roadway with less traffic. This, besides other reasons, is due to the possibility of drifting snow on the lower road. The bridge going over will generally be less subject to drifting, depending on wind velocity and direction.

Drainage can also present serious safety hazards to traffic during periods of sudden changes in temperatures. Drains should be designed and located so that no obstruction to plowing is created during winter months. When located in the travelled way, the drains should be flush with the pavement so that plow blades will ride over and clean all frozen snow, ice or debris from the grate. Weepholes and spouts on overpasses should be located so as not to cause slippery or icy roadways underneath.

## 4.6. Wind Forces on Mountain Bridges

Vehicular accidents caused by high velocity crosswinds are known to occur in North America and other parts of the world. Dangerous conditions often develop in mountaineous terrain where motor vehicles are suddenly exposed on bridge crossings after emerging from the protection of earth cuts or embankments.

An experimental wind screen was installed on a two lane high elevation freeway bridge across a mountain ravine between San Diego and Centro, California.[17] The screen extends 8ft. above the 2ft. high concrete parapet wall giving a total height of 10ft. above the pavement.

The wind screen is made of chain link mesh fencing fabricated from 9 gauge galvanized steel wire, all of the apertures are filled with crimped alluminum slats vertically placed. The spaces between the slats yield a porosity of about 20%.

Before the wind screen was installed, the winds sometimes reaches velocities that could overturn large truck trailers and often produced forces that impaired drivers' control in smaller vehicles. This condition made it necessary to restrict truck traffic during the period of high winds. The Bridge Department of California initially furnished a wind screen made of chain link mesh fencing and aluminum slats for all but the topmost portion where horizontal layers were considered as an upward deflecting mechanism. This idea was eventually discarded, instead the air turbulance that developed at the face of the wind screen was to be relied on to provide any extra lift required. This later proved to be a sound and economical decision, the higher the wind speed, the greater the lift. During the high winds, the extra lift varied from 2 to 3 ft. above the top edge of the screen.

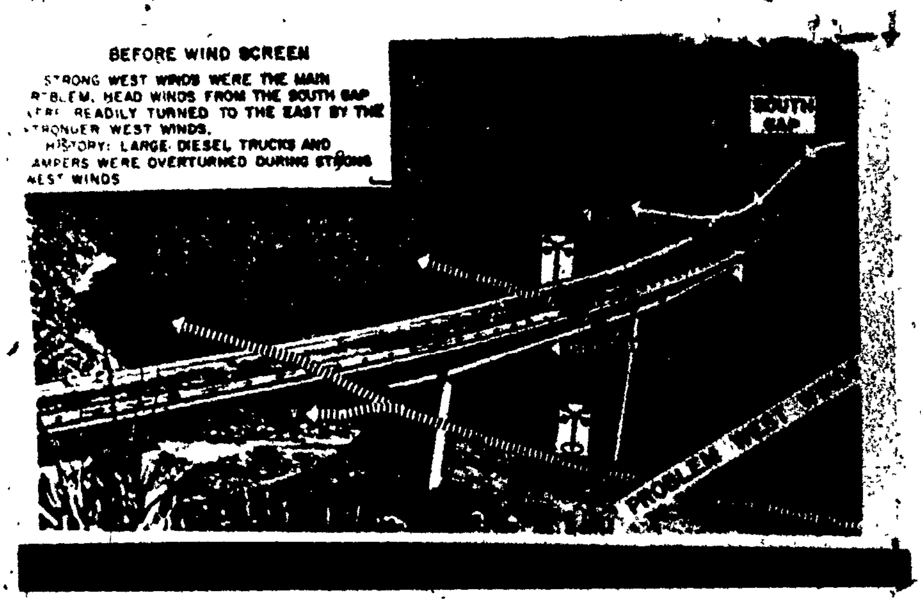




FIG. 7. WIND EFFECT BEFORE AND AFTER INSTALLATION OF WIND SCREEN. [17]

Figures 7a. and 7b. show Devil Canyon bridge before and after the erection of the wind screen. Tests were conducted across the bridge, with a small wind flag mounted on a long pole and later with a 15 ft. pole having alternate short and long ribbons spaced 1 ft. apart. These wind flag-poles were quite valuable in disclosing that cross-winds were lifted because of turbulance procuded on the windward face of the screen. The amount of wind lift varied with the wind speed thus producing a higher zone of protection when it was most needed for tall trucks during strong wind gusts.

Since the wind screen was installed, the head wind did not prove hazardous during the trial period of 24 months. Large vehicles no longer were restricted from travelling because of winds. Some velocity head winds found a protected path along the bridge behind the screen but have not been a problem.

The accident reducing possibilities offered by wind screens appear to have more advantages than may have been recognized. The greatest need usually exists wherever recurrent or persistent cross-winds lead to inadequate drivers' control or the overturning of vehicles. These roadway structures cut straight through undulating topography where protection alternates with exposure. The slated chain-link mesh fence seems to be one of the most economical ways to reduce accidents on bridges or elevated structures in mountainous terrain where high winds pose danger to traffic.

# 5. BRIDGES OF TOMORROW

## SAFETY DEVICES

### 5.1 General

The need for research to improve the vehicle safety on bridges hardly needs more emphasis. Planners and persons responsible for the maintenance of existing bridge structures are well aware of the losses due to accidents.

It is generally agreed that accidents on bridges are mainly caused due to the fault of either one element or the interaction of three main elements known as driver, vehicle, and the transportation media, the bridge structure. Bridge engineers, as part of the safety program, can fulfill their duty and contribute to the safety cause by providing more functional and safer bridge structures.

Research work on safety is being carried out in various Government departments and University laboratories individually, and the need exists to communicate and exchange information on a large scale to bring the research finding into practical use at an early date. This will not only help to save more lives, but also reduce the duplication of research work at various institutions.

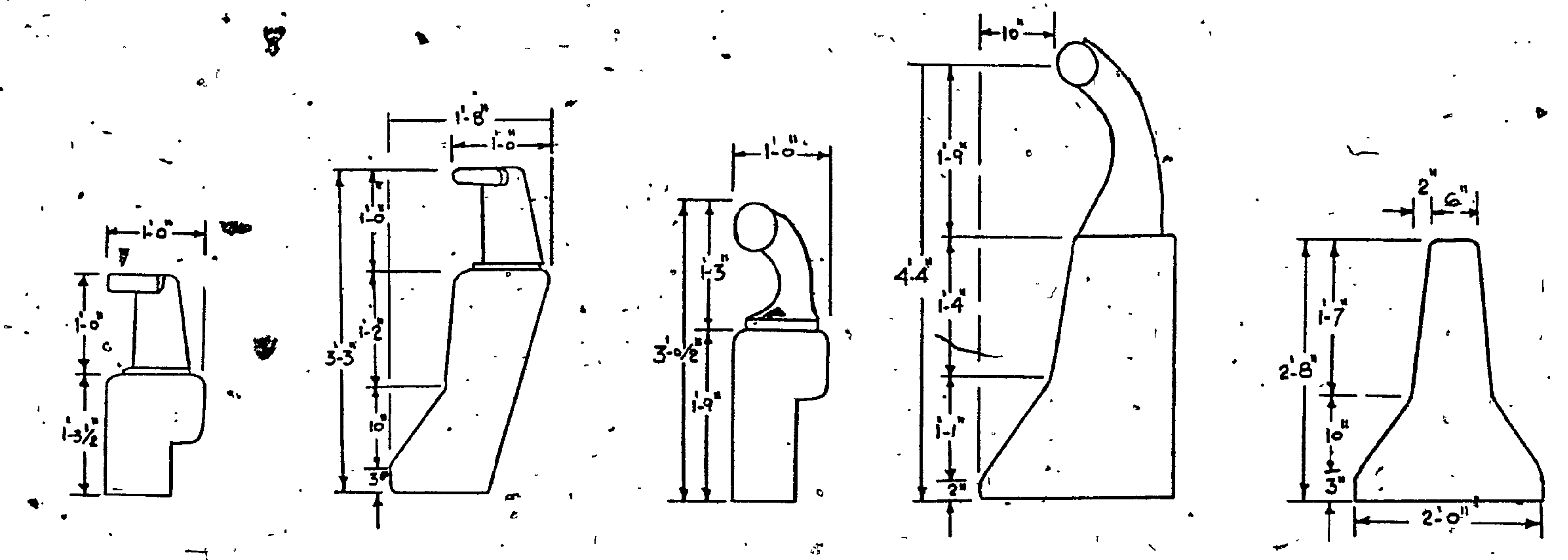
One area which particularly needs more investigation is to restrain out of control vehicles on bridge structures. In case of collision, these protective or restraining devices should keep the damage to minimum level. If these barriers on sides or at median are not strong enough, vehicles will break them and hit the oncoming traffic or plunge down the bridge. On the other hand, if these barriers are too strong, impact forces will cause considerable damage to the vehicle and its occupants. There is a need to find devices which will restrain or guide the traffic back to its travelling lane in case a vehicle should hit the bridge curb or guardrail. This chapter will discuss briefly the devices which can be used to reduce the impact forces, should a car go out of control.

## 5.2 Improved Guardrails

During a collision between an out of control vehicle and a bridge barrier railing system in which the vehicle is redirected, the vehicle is subjected simultaneously to lateral and longitudinal deceleration.

A considerable amount of research has been conducted to establish the limit of human tolerance to these deceleration forces. After reviewing the related literature,[15] [22] [33] it is concluded that a rapid deceleration at the center of gravity of a vehicle for a period of time is capable of producing serious injuries to an unrestrained occupant of a vehicle. The National Safety Council of USA developed a rational damage scale which employs photographs of damaged vehicles. A set of photographs of damaged vehicles rated from 1 (minor damage) to 7 (major damage) was employed by accident investigators at the scene of each accident. The data represented accidents involving 951 vehicles in which 184 injuries and 7 fatalities occured. Through the analysis of these results, it is demonstrated that the cars in which most injuries occurred were vehicles rated seven point on the vehicle damage rating scale. This conclusion proves clearly that the degree of injuries has a direct relation to the damage done to a vehicle after a collision with barrier guardrails on a bridge structure.

The primary objective of research work should be to establish a criteria for a bridge rail system to serve as a basis for design. This criteria may also serve as a guide for examining existing rail design and as a basis for preparing future recommendations for improved safety. Therefore, design criteria, besides other considerations, may include the following points:




TYPE-9 RAILING (a) | TYPE 20 RAILING (b) | TYPE I RAILING (c) | GENERAL MOTOR RAILING (d) | NEW JERSEY - MEDIAN (e)

FIG. 8 BRIDGE BARRIER RAILINGS [20]

a. A bridge rail system must smoothly redirect a colliding vehicle.

b. A bridge rail system must remain intact after a collision.

c. A bridge rail system must laterally restrain the crashing vehicle.

d. A bridge rail must minimize vehicle deceleration.

e. A bridge rail must have a compatible approach rail to prevent collision with the end of the bridge.

f. A bridge rail must permit adequate visibility.

g. Safety must have priority over other design considerations.

Type "20" Bridge Barrier Rail. This guardrail has been developed by the Bridge Department of California Division of Highways. This consists of a single steel rail mounted 12 inches above the top of a 27 inch high contoured concrete parapet wall (Fig. 8). The test carried out at a speed of 45 to 66 mph. and at impact angles of 7, 15, and 25 degrees; the results showed that this system will retain and redirect a 4,900 pound passenger vehicle colliding at speeds up to 65 mph and at angles of 7 to 25 degrees with the barrier. It was observed that vehicle damage increased as the angle if impact enlarged.

A test by the California Division of Highways [31] showed definite reduction in damages suffered by the vehicle striking this type of barrier rail. Thus, the contoured traffic face definitely minimizes the collision severity at shallow angles of impact. The Type "20" Bridge Barrier, however, does not offer any advantage over rigid bridge barrier rails, when impacted at greater approach angles.

## 5.3 Improved Barrier Curbs

Generally, on highways and bridges, curbs are used to control rainfall drainage, deter vehicles from leaving the roadway, delineate the road edge and as an aid in orderly roadside development. This discussion is directed to the safety effect curbs have in redirecting strayed vehicles back to the roadway. The curbs under consideration are the type commonly called barrier curbs. These curbs are relatively high and steep-faced, are designed to prevent vehicles from leaving the pavement, and are between 6 to 12 inches high. The upper corner is often rounded or chamfered to discourage the wheel rim from biting into the curb face.

The use of the curb as a device for redirecting vehicles has been considered by designers since the early days of bridge construction; however, this use has never been universal. Particularly, in recent years, low barrier curbs (12 inches in height or less) have not been used for redirection. It is suggested that drivers tend to veer away from structures having a formidable appearance, but evidence [29] indicates that drivers soon become accustomed to such structures, and after a reasonable period, use the full width of the roadway. In any case, curbs placed a short distance beyond the traffic pavement edge cause little reduction in effective lane width.

In terms of redirection effectiveness, a curb whose height is below the centre of mass of a vehicle will cause overturning immediately upon impact. Therefore, a barrier curb of limited height can never be expected to redirect a vehicle over the full range of operation impact conditions. On the other hand, the range of conditions under which a curb can be effective in redirection of a vehicle, and damage resulting from striking the curb will be far less than that which would result from striking a guardrail.

The first published research on curb mounting and redirection was carried out in 1953 by the California Division of Highways. [30] The research consisted of full scale impact tests on eleven (11) curb cross sections. The two 9 inch curbs, V and VI-M were found to be most efficient in redirection.

As a result of these tests, a second series of barrier curb tests was undertaken in 1955.[31] Four basic cross sections were tested, shims were used to achieve a desired thickness. Conclusions indicated that an efficient barrier curb should be at least 10 inches high, to under cut, and have a moderately smooth surface texture. A very smooth surface tends to redirect a vehicle back into traffic at a relatively high angle. The upper corner should be rounded so as to reduce the tendency of the wheel rim to grab onto the curb top.

The Canadian test[32] conducted as part of a larger study to determine the efficiency of the curb in combination with various guardrails as a redirection system. The cross section is shown in Fig. 8b. In an interesting series of tests conducted in England[33] the redirective character of the "Trief Curb" was found to conform to the equation:

$$V \sin X = \text{Constant} = K \quad \ldots\ldots\ldots\ldots\ldots (1)$$

where:

$V$ = Impact Velocity

$X$ = Impact Angle

In effect, whenever the component of vehicle velocity normal to the curb was larger than the fixed value, the vehicle would mount; below the value, redirection would occur.

The influence of tire-curb friction on mounting was confirmed in those tests. Mounting velocity at a 15 degree impact angle increased from 12 to 20 when the tire and curb were wet.

The development of efficient barrier curbs and the assessment of their effectiveness seem well worth the effort. [24] It is evident that a carefully designed barrier curb can be effective in redirecting vehicles back to the roadway. In addition, vehicle damage will be substantially less when a vehicle impacts with a curb than when it runs into a guardrail.

## 5.4 Curb - Guardrail Combination

Combination of curb-guardrail represents a marriage of two effective devices to improve the safety of vehicles on bridges. The utility of such a combination seems obvious; vehicles striking the curb at small angles are redirected with little damage, but for vehicles climbing the curb, the guardrail acts as a positive secondary retainer. In the United States of America, a number of different curb-guardrail combinations are installed at numerous locations along the highways and bridges. The variety of system is the result of design changes and improvements in both curb and guardrail standards.

The primary question relative to using a combined curb-guardrail system, is whether the curb contributes adversely to the redirective performance of the guardrail.[35] In a single case of vaulting, a 17,500 pound city bus struck a concrete bridge rail, broke through the rail, and came to rest straddling the rail. It was uncertain whether the structural failure of the rail contributed to the final position of the bus, or whether no vaulting would have occured if the rail had remained intact. Despite inconclusive results, there was no proof that vaulting could have occured as a result of curb-guardrail dynamic interaction.

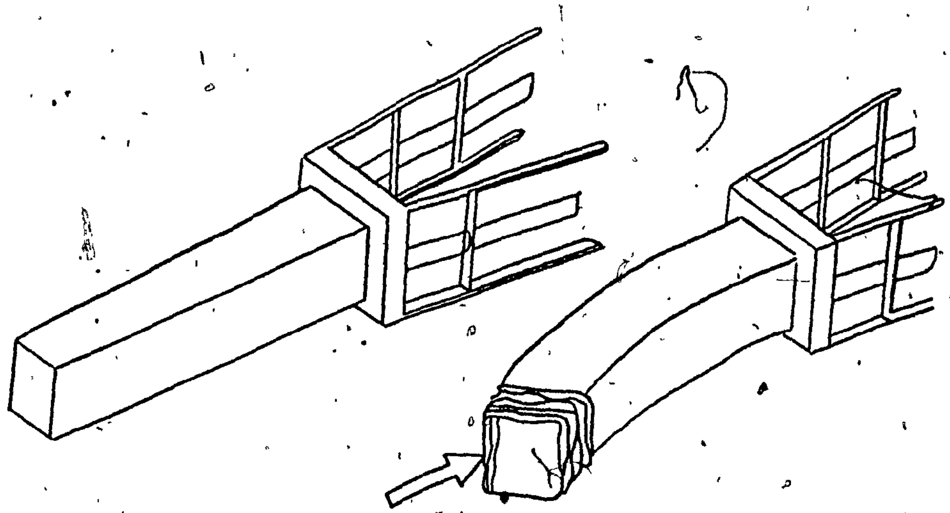
Curb-guardrail combinations of various varieties have been tested by several organizations. Dynamic data has shown the tendency for vehicles to bound into the air after a curb impact. The possibility therefore exists for a vehicle to receive a jump impulse from the curb and vault over the adjacent guardrail. During a study of various types of curb-guardrail combinations,[35] no conclusive evidence of a vaulting problem has been identified as the result of curb-guardrail combinations under study. Therefore, although intution suggests that vaulting is a potential problem, this has not proved to be the case.

However, care should be taken to ensure that the height and setback of guardrails installed behind curbs satisfy the prescribed minimum.

A recently developed barrier curb appears to be an efficient redirective device.[36] Seventy percent of the vehicles striking this curb in urban traffic conditions can be expected to be redirected. Vehicle damage in these encounters can be expected to be modest, and far less than what can be expected if the vehicle were to strike a guardrail. It is recommended that further research be initiated to better define the cross section for a barrier curb optimize for redirection. It is also recommended that total redirective performance of this curb be optimized in combination with guardrail.

## 5.5 Buffering System

A current awareness of traffic safety on highways in general and on bridges in particular makes energy-absorbing devices very attractive. Among other patents,[23] NASA holds the patent on energy-absorbing devices. The system is somewhat like a structural column, Fig. 9, that must absorb axial loads in reducing the speed of a vehicle, and withstand the bending and shear forces in redirecting the vehicle. It must also absorb shear



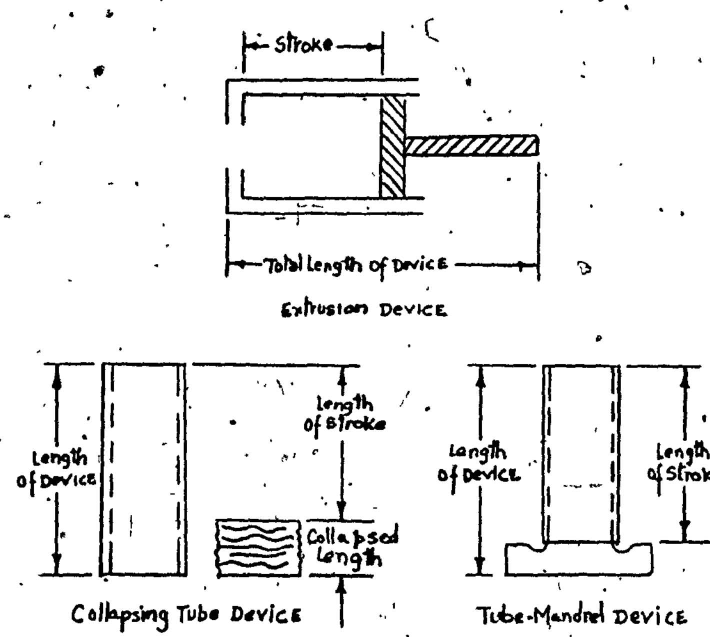



FIG. 9. IDEAL BUFFERING SYSTEM [23]

and bending on the vertical plane to compensate for accentric loading due to differences in the relative heights of the mass centre of the vehicle and the impact absorbing system. Therefore, the action of the vehicle during an impact is largely a function of the reacting forces generated by the system, and axial deceleration is governed by the collapse of load absorbing column of this system.

Functional buffering systems for out-of-control vehicles should be designed and properly tested with performance-design criteria, have been established. To define such criteria, three major points need to be clarified for proper shock absorbing system design:

a. Range and probability of vehicle dynamic conditions prior to impact.
b. Relationship between the time-deceleration history of the vehicle and resulting injury to vehicle occupant.
c. Action of the vehicle during and after impact.

With performance criteria available, it will be possible to evaluate objectively, the effectiveness of available buffering systems, and then these systems can be termed either effective or ineffective depending on whether they can satisfy the existing performance criteria.

In practice, a complete buffering system is a fairly complex arrangement of energy-absorbing units and tie-down, interconnecting and load-distributing elements. However, most of the available energy absorbing devices can be engineered to operate over a wide range as vehicle impact absorbing and damage reducing devices.

In the initial stage of a study at the University of Denver, National Aeronautical Space Administration had 53 different devices available for application in bridge and highway safety programs. These devices were checked for their service reliability, reusability and material efficiency. As vehicle impacts can occur over a wide range of positions and directions, the proper function of these devices under a restricted range of loading directions, required the proper and adequate support and load-directing structures. Experimental studies still being conducted on these devices. The properties of the selected energy absorbing bumper will be used to develop effective safety devices for highway bridges. It is hoped that research work will produce useful results and in the near future could provide us with many effective devices to reduce the severity of accidents and cost of replacing damaged highway structures.

## 5.6 Corrugated Metal Buffer

A new type of energy-absorbing buffer is being developed in the lab[37] which is mainly made of corrugated metal elements that deform plastically on impact and absorbs the energy of the impacting vehicle. The buffer has a parabolic shape to form a gradual transition between an energy-absorbing buffer for frontal impact, and an energy-absorbing guardrail for side impacts. The model buffer was[33] found to perform well in a variety of situations including head-on, angled, and glancing impacts. The following performance criteria was used in a scale testing at Denver Research Institute[34]:

a. The force-displacement curve should be such that a range of vehicles can be stopped without excessive loads being imposed on the lighter vehicle or excessive stoppage distance being required for the larger vehicles.
b. The buffer mass activated at impact should be small compared to the weight of the impacting vehicle.
c. Buffer deformation and motion should be localized to the immediate area of the impacting vehicle.
d. The buffer should not produce significant angular acceleration until the vehicle has been entrapped.
e. The lateral stiffness of the buffer should be increased greatly towards the base.

The objective of this test program was to develop a simple, inexpensive buffer satisfying the above mentioned criteria and capable of performing well in a broad spectrum of impact situations.

The idea of using these parabolic corrugated metal arches was conceived to apply them parallel to the surface of roadways so that they would form a guardrail for a glancing side impact and would deform to absorb the energy of a vehicle impacting the nose.

Three types of metal arch buffers were studied in the course of project. Type 1. consists entirely of two or more parabolic corrugated metal arches, type 2. consists of metal arches and barrels, and type 3. consists of corrugated metal arches and corrugated metal stiffening elements.

The corrugated metal arch buffer, although, simple in design, performed very well in a wide variety of scale-model impacting situations. The type 3. buffer has demonstrated an overall promising performance, which may soon qualify it to be used on highway bridges. In addition, barriers could be used at locations where lateral space is limited.

## 5.7 "Dragnet" Vehicle Arresting System

Vehicles crashing into bridge guardrails or abutments have forced research engineers to find new methods and devices to reduce the crash impact, and damages. This con-

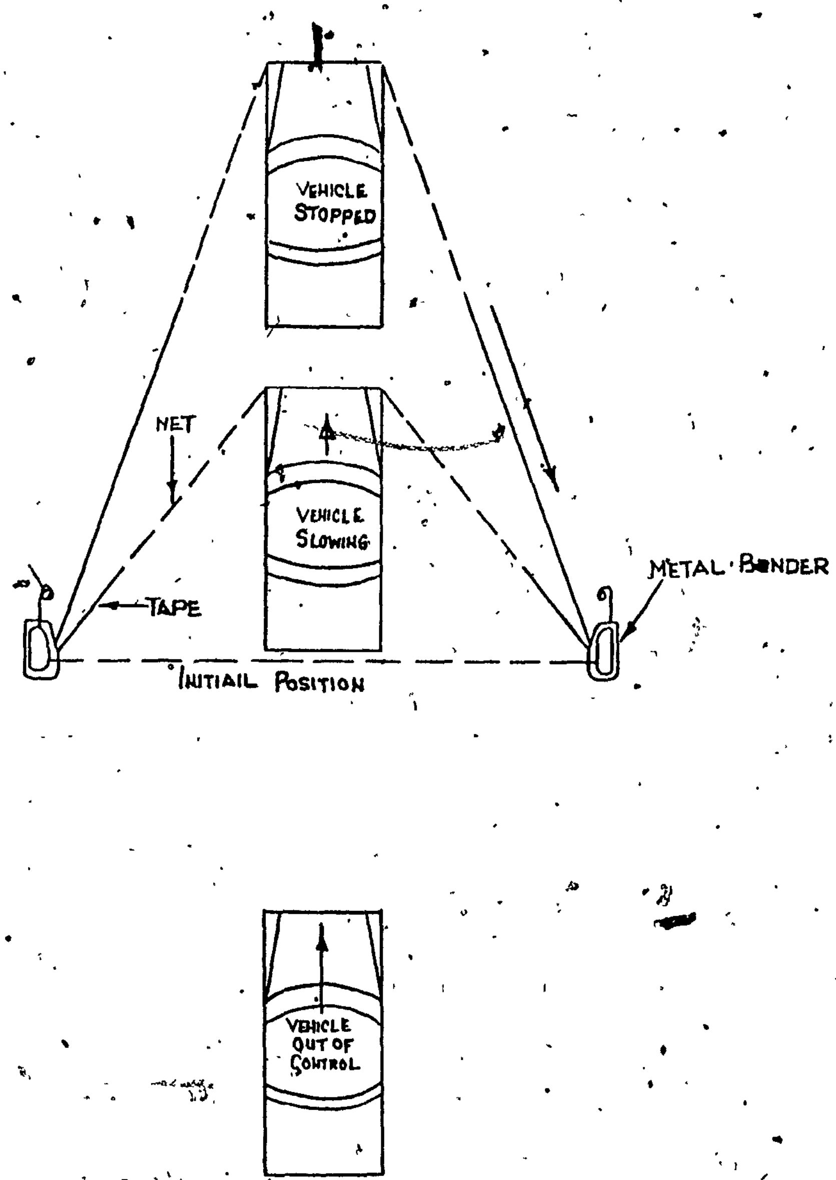



FIG. 10. IDEALISED FUNCTION OF DRAGNET ARRESTING SYSTEM [24]

tributed significantly to the development of new concepts to increase the traffic safety, and at the same time determine the applicability of new safety devices.

One such device already existing and being used in other fields is the "Dragnet" Vehicle Arresting System by [24] Van Zelm Associates, Inc., of Providence, Rhode Island, U. S. A. The system, which consists of a steel net, fig. 10, attached at each end to metal energy absorbing devices, has been used on "drag strip" raceways and in improved forms, for aircraft arrestments, but so far not used widely on suitable locations on highway bridge structures. This may be due to the lack of independent testing of the system for this application.

The system had been subjected to six full-size automobile crash tests by the Texas Transportation Institute [24] during the period from December 1967 to November, 1968. The main object of these tests was to evaluate the performance of the dragnet system in stopping an out of control vehicle over a relatively short distance with acceptable deceleration levels.

The energy absorbed by the metal benders ranged from 50 percent to 96 percent. There are several reasons for this difference. In most impacts there is some gravitational potential energy gain caused by the tendency of the net to pull the vehicle down in front and the tendency for the rear end to rise. This results in an increase in the elevation of the vehicle's centre of gravity. In the case of impact with an angle, there may be a significant amount of horizontal rotational energy present. Also present may be transverse rotational energy which is defined in the same way as the horizontal rotational energy except that the mass moment of inertia and angular velocity are about the longitudinal vehicle axis. Other energy expenditures may be accounted for by the axial strain energy that goes into the cable and tapes of the dragnet system, the vehicle deformation and frictional losses of the vehicle with the pavement.

The "Van Zelm Dragnet" Vehicle Arresting System proved excellent in four out of the six tests. Deceleration levels were reduced in rigid guardrail impact. These tests also showed that fairly accurate prediction of vehicle stopping distance and deceleration can be obtained.

It was recommended by the body conducting these tests, that the dragnet system is an effective, practical and economical system for safely stopping vehicles that are out of control. It was also recommended that the dragnet system can be applied to a median on two-way traffic bridge to reduce collision impacts of traffic travelling in opposite directions. Application of the dragnet system can be extremely helpful on long bridges over rivers or water bodies or elevated long structures, where the incidence of vehicles plunging may be feared. However, this system may have its limitations; long trucks or truck trailers cannot be expected to decelerate and stop within the same reasonable distance as passenger vehicles.

## 6. SUMMARY AND CONCLUSIONS

The statistics of accidents on bridge structures demand systematic and carefully planned research work. Fortunately, the research work carried out at the present time is an unpresedented level, but still, major thrust of research work is aimed towards highway traffic safety, and there are very few programs being carried out with specific safety consideration on bridge structures. However, there is still considerable need for research work in this particular area. There is need for additional work to improve and develop more rational bridge design approachs. This should also incorporate the methods of estimating bridge rail deformation under specified load conditions as a function of the structural and material properties. The amount of vehicular traffic, design speed, auto dimensions and weight and similar information must be based on statistical studies. Developed designs should be tested on a full scale to determine adequacy of material, structural system and behaviour of vehicle in relation to impact.

Through the exploratory work in this thesis, the writer has concluded that there is no one single factor, but rather the effect of many factors which contribute toward the accidents over bridge structures. It is also observed that it will be rather difficult to eliminate accidents completely, but through research, the number of accidents can definitely be reduced. It is also unrealistic to expect any single breakthrough to ensure traffic safety as a whole, but instead, individual projects and observations will produce facts and devices which will, some day in the near future, provide a solution to present day safety problems on bridges. With ever increasing traffic volume, and accident rate, there will always be need and pressure to build safer bridge structures which will ensure the passage of traffic. It will be a continuous process of research work to study all the physical, structural and other ralated aspects and be able to recommend uniform criteria for planning and design of bridges for future traffic demands.

One area which should dominate the research program is the need to involve the personnel responsible for the operation of traffic on highways and bridges. Most operational personnel in bridge safety programs seem to be concerned only with the apparent accidents causing problems and rather neglecting other apparently minor factors which contribute towards the accidents. Involvement of personnel responsible for the traffic operation on bridges can expose many more factors influencing the vehicle movements on bridges. However, the main theme should be a team work research and not purely academic oriented research carried out by highly sophisticated experts.

At the meeting of The Highway Research Board Committee on highway safety in 1964, one speaker described a very simple finding, which undoubtedly could

[26]

save many lives on bridges - a 5¢ washer inserted behind the head of a bolt and nut to hold guardrails in place. He showed this example to emphasize the need for systematic procedures for implementing such findings.

Such discoveries will continue, possibly by means of systematic research. Such investigation work is needed in order to create a climate of understanding of accidents and therefore recognize and implement the remedial measures or devices to improve the safety records on bridges.

It is felt that design methods for bridges must be kept under constant review, particular attention being paid to the broad philosophy and concept of design, as well as to details. The rapid developments of modern times and the increasing number, size, scale and scope of work will make bridge design much more complex than at present. It will be necessary in future to design more causiously, taking into consideration all possible and foreseeable conditions as precisely as possible.

The value of knowledge gained through actual field experience cannot be over-estimated. Such knowledge is essential for establishing service-ability conditions of bridge structures.

The reasons and responsibilities of bridge failures usually are discovered after a long investigation. The lesson is learned at a considerable cost to everyone involved, which could have been avoided, if the design technique, construction method and exact behaviour of material would have been understood well

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