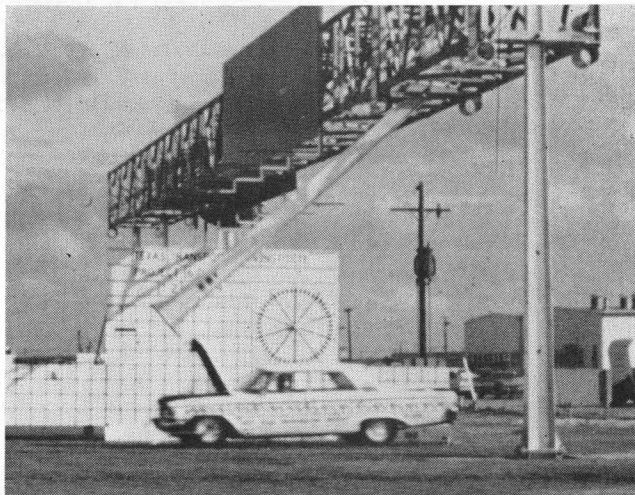# SUMMARY REPORT
OVERHEAD SIGN BRIDGE RESEARCH
PROJECT HPR-2(107)

190-Tex- 13-4

 # 464

SAFETY PROVISIONS FOR SUPPORT STRUCTURES
ON OVERHEAD SIGN BRIDGES

A Cooperative Research Study
Sponsored by the Highway Departments of

Alabama	Kentucky	New Mexico
Arkansas	Louisiana	Rhode Island
Connecticut	Maryland	South Dakota
District of Columbia	Michigan	Tennessee
Florida	Mississippi	Texas
Georgia	Montana	West Virginia
Hawaii	Nebraska	Wyoming
	New Hampshire	

In Cooperation with the
U. S. Department of Transportation, Federal Highway Administration

Overhead Sign Bridge Research Project HPR-2(107)

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Foreword

This summary report contains a brief description of information developed under Research Project HPR-2(107), entitled "Safety Provisions for Support Structures on Overhead Sign Bridges," which was sponsored jointly by twenty-two highway departments and the U. S. Department of Transportation, Federal Highway Administration.

TECHNICAL MEMORANDUM NUMBER	AUTHOR(S)	TITLE
605-1 (Volume 1 of 6)	E. R. Post C. P. Garner, Jr. R. M. Olson	DESIGN, ANALYSIS AND CONSTRUCTION
605-2 (Volume 2 of 6)	J. E. Martinez J. J. Jumper F. Y. Baskurt	MATHEMATICAL SIMULATION AND CORRELATION
605-3 (Volume 3 of 6)	R. H. Gunderson A. Cetiner	A STUDY OF BUCKLING STRESS FORMULAS
605-4 (Volume 4 of 6)	D. L. Ivey C. E. Buth R. M. Olson T. J. Hirsch	TESTING PROGRAM
605-5 (Volume 5 of 6)	A. J. Stocker	FABRICATION AND CONSTRUCTION
605-6 (Volume 6 of 6)	R. M. Olson D. L. Ivey J. E. Martinez E. R. Post R. H. Gunderson	SUPPLEMENTARY STUDIES

In the following paragraphs each of these Technical Memoranda have been briefly described and the reader of this summary report is referred to the Final Report on the project for more specific information.

Acknowledgements

Introduction

High-speed highways require the placement of roadside and overhead directional signs for efficient control of traffic. Fixed supports of sign structures constitute a hazardous condition to the occupants of an errant vehicle. Accident information compiled by the California Division of Highways' Traffic Department during the years of 1965-7 indicated that approximately 7 percent of 640 single vehicle fixed-object freeway fatal accidents involved steel sign supports.[1]* Eliminating fixed sign supports from gore areas has proven effective in reducing accident frequency and severity. Operational experience has further shown that, wherever practical, signs should be mounted on overcrossing bridge structures or that the supports of signs should be: (a) located 20 to 30 feet from the roadway shoulder, or (b) placed behind guardrail, or (c) provided with break-away devices.

The field performance of break-away roadside sign supports, conceived and designed by Hawkins[2] and developed and tested by the Texas Transportation Institute,[3,5] is summed up in the following two paragraphs presented in the publication Texas Highways.[4]

> "The first breakaway signs were installed on Texas highways in 1965 and in less than $3\frac{1}{2}$ years, 171 accidents were documented with only one reported injury and one fatality. The Bureau of Public Roads made the breakaway design mandatory for all signs on federal aid highway construction throughout the nation after the design proved successful in Texas.
>
> As research continues on the breakaway overhead sign bridge, some 80,000 breakaway signs already are in place along the 69,000 miles of state-maintained highways in Texas. More are being added daily with some half-million signs due for conversion to breakaways."

In 1965, and subsequent to the promising results obtained from tests on prototype break-away roadside sign supports, Hawkins[5] began an investigation to extend the break-away concept to the larger supports of OVERHEAD SIGN BRIDGE STRUCTURES (OSB). A preliminary design by Hawkins of an OSB with four break-away supports showed that the concept warranted further consideration. The safety characteristics of the OSB break-away concept was the primary objective of this research study.

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*Superscript numbers refer to items listed under References.

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Design, Analysis, and Construction

Technical Memorandum 605-1

by

E. R. Post, C. P. Garner, Jr., and R. M. Olson

General Design Considerations

The PROTOTYPE OSB with four break-away support columns, on which full scale head-on and angle tests were conducted, is shown in Figure 1. Except for minor modifications, the PROTOTYPE OSB was essentially the same as the preliminary design of Hawkins.[5]

The PROTOTYPE STRUCTURE was designed to represent a very large OSB structure which might be constructed on the Interstate Highway System. The PROTOTYPE OSB has an overall length of 140 feet. The truss is 6 feet in width and depth. The 100 foot middle section of the truss is of sufficient length to span a four-lane divided highway. The OSB is structurally adequate to resist dead loads and a 100 mph wind load with all four columns in place; whereas, when one of the four columns is temporarily displaced by a colliding vehicle, the OSB is structurally adequate to resist dead loads and a wind load of 50 mph. The study conducted by Hawkins[5] has shown that the sign area of 325 sq. ft. located above the lanes of travel in each direction on the PROTOTYPE OSB would exceed the sign area of 95 percent of the installations on Texas highways, thus 650 sq. ft. represents a reasonable maximum sign installation. To satisfy these design conditions, massive break-away columns,
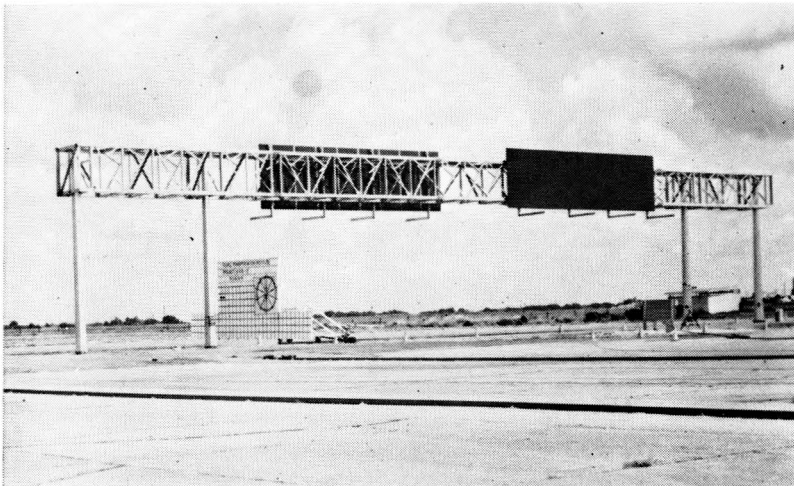
Figure 1. Prototype OSB with four break-away supports.

each weighing approximately 1,500 pounds, were needed, and four 1¾ inch high strength bolts were used at the break-away base connection. The OSB truss was designed stiffer than would actually be needed under field conditions.

The PROTOTYPE OSB break-away columns, shown in Figure 1, have an over-all height of approximately 26½ feet above the slip base. To reduce the mass and inertial effects of the break-away columns during a collision incident, and hence, minimize the vehicle damage and decelerations, the columns were: (a) fabricated from a 100,000 psi Heat-Treated Constructional Alloy Steel (ASTM 514), and (b) tapered in both the flanges and web. The wedge tapered columns are also pleasing in appearance. Each break-away column was designed to clear a colliding vehicle as it rotates about a 1-7/16 inch diameter stainless steel pin connection, subsequent to the release of the break-away base connection and the fracturing of four ½ inch A307 bolts in the upper connection. A close-up view of the column connections is shown in the photographs of Figure 2.

The PROTOTYPE OSB was designed in accordance with 1968 AASHO Specifications for the Design and Construction of Structural Supports for Highway Signs[6] with the exceptions of: (a) the break-away column devices, and (b) the lateral buckling requirements of sections 6:a(3) and 6:a(7). The lateral buckling requirements of AASHO are applicable to that portion of a beam or column subjected to pure bending. Sign supports
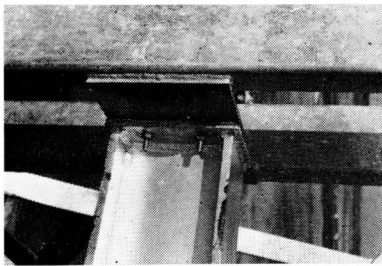
Upper Connection
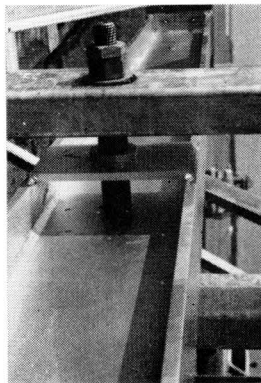
Pin Connection
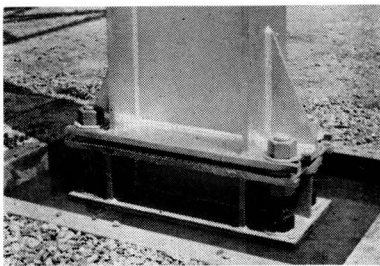
Slip Base

Figure 2. OSB column connections.

are essentially cantilevers and in this project, the columns were analyzed as such. The tapered break-away wedge columns were designed on the basis of information from research studies performed at Columbia University by Krefeld.[7]

In addition to being strong enough to withstand static loading conditions, the OSB columns under dynamic loading conditions must: (a) have break-away connections weak enough to produce tolerable decelerations on the colliding errant vehicle, and (b) have a strong enough pin connection to assure that the break-away connections will actuate and the column will rotate clear of the vehicle.

Three other important design features, which became apparent during this study deserve mention. Steel pipe sections were fastened to the lower chord members of the truss to distribute the impact forces and minimize damage as a column, following a collision incident, strikes the truss. Secondly, two horizontal angles were fastened at approximately mid-depth of the truss and on each side of the column, to guide the column during an angle collision so that the column will not snag on and damage the vertical truss members. Finally, a thin sheet metal "keeper plate" was placed between the slip base plates of the column and the stub post, as shown in Figure 2, to eliminate the possibility of the breakaway columns walking off their foundation stub posts under vibrations set up by wind and traffic.

Possible Locations of OSB Having Two or More Break-Away Column Supports

It is necessary to insure that an OSB will not collapse onto the roadway when an out-of-control vehicle collides with one break-away support. Therefore, to prevent the collapse of an

Figure 3. Structures with one fixed and two break-away supports.

OSB, a minimum of three column supports must be provided. At present it is recommended that in such installations one support be fixed.

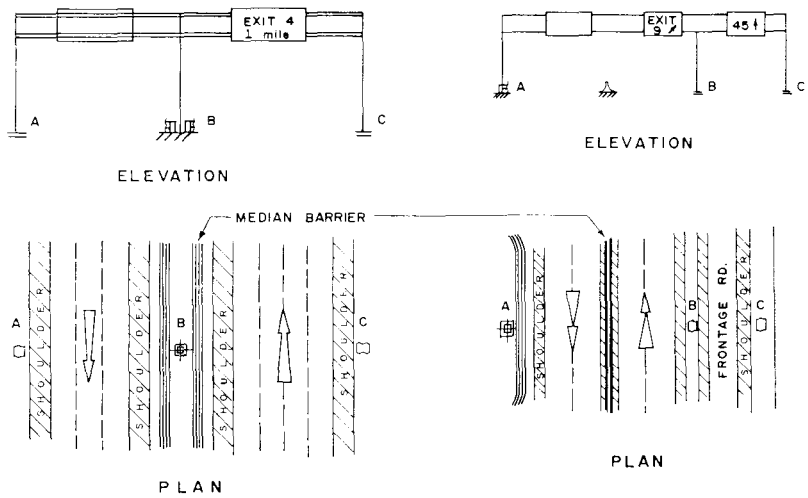Figure 3 shows examples of structures with two break-away supports and one fixed support. These sketches illustrate a condition which exists on many freeways which have a traffic barrier separating opposing traffic lanes of traffic.

Another example condition exists where the fixed support would be constructed on a right of way line, and the break-away supports would be located near the shoulders of a divided highway having a wide median. Michie⁵ indicates that median barriers are not warranted when the distance between pavement edges of opposing lanes of travel exceeds 40 feet.

These examples can be extended to a structure having four supports, one of which may be fixed because of roadway conditions. This discussion illustrates the point that the stability of a structure having break-away supports can be enhanced if one or more supports are fixed.

Mathematical Simulation and Correlation

Technical Memorandum 605-2

by

J. E. Martinez, J. J. Jumper, and F. Y. Baskurt

Mathematical Model

The Prototype OSB shown in Figure 1 is comprised of approximately 400 members. The solution for the response of this structure subjected to some loading conditions requires solving a large number of simultaneous equations; and, even with the facilities of an IBM 360/65 computer having a core capacity of 100K words, it is usually necessary to employ outside storage facilities in order to perform the static analysis of the structure. An elastic dynamic analysis considering the entire OSB not only adds to the information storage difficulties, but makes the computer cost to solve the problem prohibitive. The reason for this is that the solution must be carried out numerically and the response must be followed for a considerable length of time. Consequently, in this study it was decided to employ a rigid body model to describe the dynamic behavior of the break-away column support.

This model assumed the supporting column to be a rigid body having only an angular degree of freedom and being hinged at the truss connection and idealized the colliding vehicle as a single-degree-of-freedom spring-mass system. This idealized system along with the forces that are taken to act on it is shown in Figure 4.
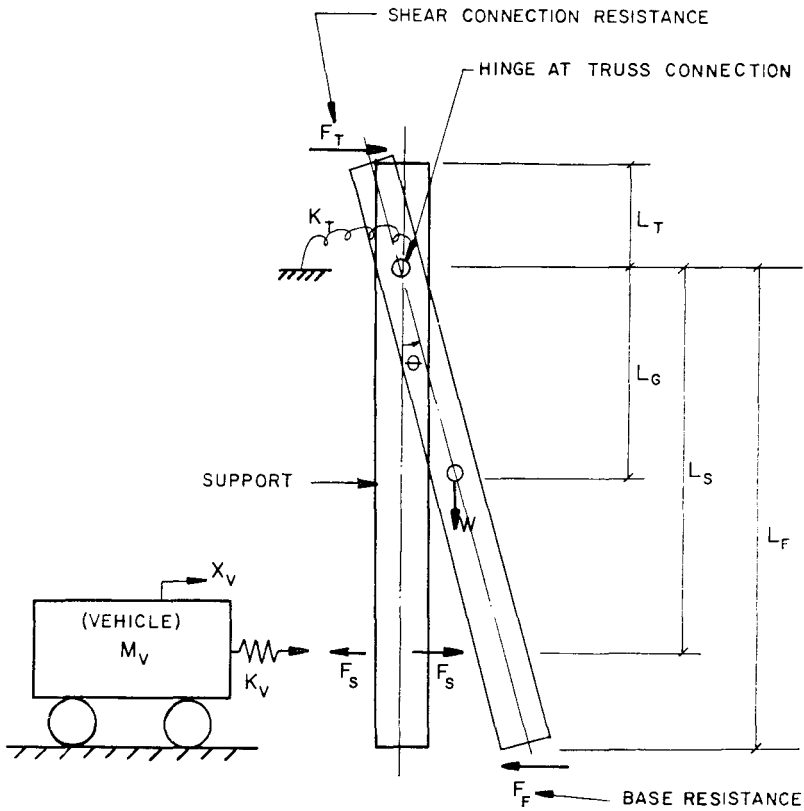
Figure 4. Vehicle and support idealization.

The forces F_F and F_T in Figure 4 represent shear resistances offered by the base connection and the upper connection to the truss, respectively, whereas the force F_S represents the vehicular impact force.

Correlation with Test Results

In order to validate the mathematical model of the break-away column and vehicle, the results obtained from the model simulation were compared with the results obtained from seven full-scale crash tests as shown in Table 1. As evident in Table 1, a reasonably good agreement exists between the results of the mathematical model simulation and crash tests. Such a comparison is remarkable when one considers the simplicity of the model and the difficulties involved in acquiring and reducing data obtained from crash tests. One of the difficulties in data reduction was estimating from the high-speed film the time at which contact was lost between the break-away column and colliding test vehicle. For the angle collisions, a portion of the difference between the model and test data could be due to the fact that the model was developed for head-on impacts only.

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Table 1

COMPARISON OF MODEL SIMULATION AND CRASH TEST DATA

Test Number	Year, Make and Type of Vehicle	Vehicle Weight (lbs)	Impact Angle (deg)	Impact Velocity (mph)	Change in Vehicle Velocity (mph) Test	Model	Time Column & Veh. Were in Contact (sec) Test	Model	Average Long. Vehicle Deceleration (G's) Test	Model	Maximum Column Penetration (in) Test	Model	Kinetic Energy Imparted To Truss and Pipe Distrib. (kip-ft) Model
605-A	1963 Ford	3950	0	25.7	5.4	5.9	0.091	0.082	2.9	2.9	12	12	3.1
605-B	1959 Simca	2100	0	44.0	14.7	16.2	0.080*	0.139	8.9	5.3	21	26	19.1
605-C	1963 Ford	4090	0	46.5	8.9	9.3	0.080	0.099	5.5	4.3	15	20	38.5
605-D	1962 Cadillac	4880	0	54.0	9.0	9.1	0.080	0.087	4.2	4.7	18	19	62.9‡
605-E	1963 Ford	3920	15	28.6	7.2	7.0	0.085	0.095	4.1	3.4	10	14	6.5
605-F	1959 Borgwood	2350	15	52.0	14.3	16.3	0.062	0.074	10.7	10.1	20	18	37.0†
605-G	1962 Ford	3950	15	50.1	10.2	10.4	0.059	0.093	7.9	5.1	19	21	48.9†
605-H**	1961 Cadillac	5150	0	75.3	11.5	11.7	0.066	0.083	8.1	6.4	25	27	142.1†
605-I**	1962 Cadillac	5170	15	72.0	11.2	11.2	0.068	0.083	7.7	6.2	32	26	128.2

*Time during which break-away components were activated. Vehicle snagged lower end of support post, was lifted and pulled to a stop, wedged between support post and the ground.

**Tests H and I reported in TM 605-6; "Supplemental Studies."

†Tests on exterior support. Bolts in upper connection of adjacent interior support fractured.

‡It was predicted from a film analysis that the kinetic energy imparted to truss and pipe distributors was approximately 49.8 kip-ft.

The model simulation should be of value to engineers who desire to analyze other proposed OSB structures with break-away supports.

A parameter study was conducted using the mathematical model to determine the dynamic response of a vehicle and break-away support for a variety of conditions. The general conclusions obtained from the parameter study and observations of full-scale crash tests are summarized as follows:

1. The application of the break-away concept to the supports of an overhead sign bridge is feasible.

2. The prototype truss is structurally adequate to withstand the torsional loads imparted to it by the rotating break-away support, and the OSB structure as a whole remained stable under the impact forces.

3. Vehicle velocity changes and deceleration increase as the break-away base and upper shear connection resistances increase.

4. Vehicle velocity changes, deceleration, and damage increase as the column support weight increases.

5. Small size passenger vehicles are subjected to higher velocity changes, deceleration, and damage than that for larger size passenger vehicles.

Buckling Stress Formulas for Overhead Sign Bridge Supports

Technical Memorandum 605-3

by
R. H. Gunderson, A. Cetiner, and R. M. Olson

The supports of overhead sign bridges are essentially cantilever beams. The 1968 AASHO Specifications[6] require, however, that the critical lateral buckling stress be limited by formulas based on theoretical and experimental investigations of a simple beam subjected to loads which produce pure bending. This portion of the study examined the AASHO formula to determine its applicability to overhead sign bridge supports.

Rigorous adherence to the present AASHO buckling requirement results in uneconomical open sections for overhead sign bridge supports which are fixed at the base, elastically restrained by the truss at the top, and are subjected to wind and dead loads which produce lateral, transverse, axial, and moment forces. The same conditions exist on roadside signs with two or more supports.

A suitable alternate to the AASHO Specifications for critical buckling stress has been developed in Technical Memorandum 605-3 and shown to agree with available test results of Krefeld.[7]

a. THEORETICAL SUPPORT CONDITIONS — AASHO,AISC

b. TEST MODEL SUPPORT CONDITIONS (4)

c. OVERHEAD SIGN BRIDGE SUPPORT CONDITIONS (TYPICAL)
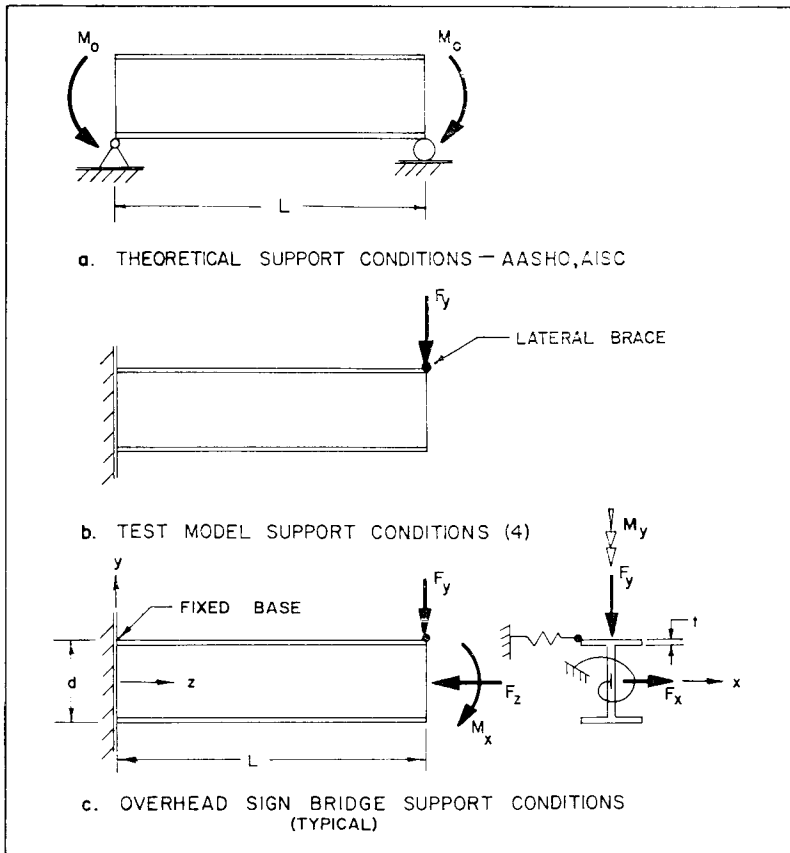
Figure 5. Support conditions and loadings.

Theoretical critical buckling stress formulas have been derived by deVries[10] and Winter[11] for cases of lateral buckling of simple beams loaded by end moments, and by Timoshenko[12] for buckling of cantilever beams loaded by transverse end forces and straight slender columns under concentric and eccentric axial forces. An overhead sign bridge support is subjected to a combination of the above loads, and the end conditions do not correspond to any of the above cases as shown in Figure 5.

PROPOSED ALLOWABLE CRITICAL BUCKLING STRESS FORMULA

$$F_b = \frac{1}{F.S.} \left[\frac{R}{I_x} \left(\frac{2d}{L} \right) \left(\frac{K_x}{K_y} \right)^2 \frac{\sqrt{EI_yGK}}{\left(1 - \frac{d}{2L} \sqrt{\frac{EI_y}{GK}} \right)} \right] \quad (1)$$

Where, R, is a stress reduction factor for tapered columns de-fined by Krefeld[7], and, K, is the torsional rigidity. The effects of end support conditions are included in the effective length factors, K_x and K_y. ($K_y{}^2$ is a simplification of K_yK_z).

As required in Section 6(a) (4) of AASHO[6], overhead sign bridge supports shall be proportioned in accordance with the following interaction formula.

$$\frac{f_a}{F_a} \;+\; \frac{f_b}{F_b} \;+\; \left(\frac{f_v}{F_v}\right)^2 \;\leq\; 1.0 \qquad (2)$$

where, the actual bending stress, f_b, is determined for bending about the two principal axes of the support cross section.

The allowable unit compression stress, F_b, in WF and I supports subject to lateral buckling can be determined by use of the proposed formula, Equation (1), or one of its simplified versions presented in Technical Memorandum 605-3.

Comparison of Critical Lateral Buckling Stress Formulas

A comparison of several forms of critical lateral buckling stress formulas are shown in Figure 6. In each case, it is as-sumed that the column section is prismatic and the bending stress is below the yield point. The test results of Krefeld[7] are for relatively light sections with high moment of inertia ratios of I_x/I_y and the tension flange of the cantilever beam was lateral-ly restrained at the point of application of the transverse load as shown in Figure 5b. The tests of Krefeld[7] closely approxi-mate the end condition existing for a typical overhead sign
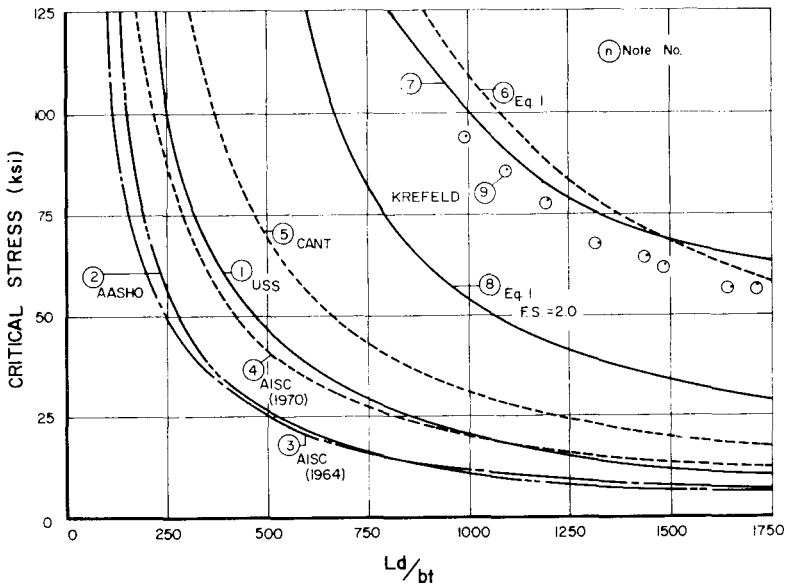

Figure 6. Comparison of critical lateral buckling stresses vs. Ld/bt from various theoretical formulas and test results (pris-matic supports . . . R = 1).

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bridge support as shown in Figure 5c. As evident in Figure 6, the proposed critical lateral buckling stress formula, Equation (1), compares closely with the buckling stresses determined from the tests by Krefeld.[7] However, Equation (1) allows the engineer more latitude to account for different end conditions through the effective length factors, K_x and K_y, which the Krefeld empirical formulas do not contain.

Using a factor of safety of 2.0 the critical buckling stress predicted by the proposed formula, Equation (1), is approximately 4.8 times the stress determined by the current AASHO Specifications.[6] If the critical buckling stress is the limiting factor in the design, the use of the proposed formula could result in a significant savings in material. This is also of great importance in the design of an overhead sign bridge break-away support because the weight of the support must be kept to a minimum in order to limit vehicle deceleration and hence, prevent injury to the vehicle occupants.

Note	Description
1.	Lateral buckling of simple beam in pure bending. No F.S., USS Design Manual[15] Formula 20 (see also References 10, 11, 14). Figure 5a.
2.	USS Formula 20, F.S. = 1.8, AASHO Specifications.[6] Figure 5a.
3.	Allowable stress formula based on lateral buckling. F.S. ≧ 1.67, AISC Steel Manual,[13] Sixth Edition, 1964. Figure 5a.
4.	Allowable stress formula based on lateral buckling modified to include effects of end moments; AISC Steel Manual,[13] Seventh Edition, 1970. F.S. = 1.67.
5.	Lateral buckling of cantilever beams with transverse end load by Timoshenko,[12] No F.S. Figure 5b.
6.	Lateral buckling of restrained cantilever; includes effect of end conditions. No F.S. Proposed stress formula; Equation (1). Figure 5c.
7.	Lateral buckling of restrained cantilever; modified Equation (1) in terms of Ld/bt. No F.S. Figure 5c.
8.	Proposed critical stress formula with F.S. = 2.0. Equation (1). Figure 5c.
9.	Test Data, Krefeld,[7] et al. Figure 5c.

Table 2

SUPPORT CONDITIONS

Displacement	Sign Bridge Support	Test Model
Transverse Translation, y-direction	Unrestrained	Unrestrained
Rotation, x-axis	Unrestrained	Unrestrained
Rotation, y-axis	Elastically Restrained (restrained in many cases)	Unrestrained
Lateral Translation, x-direction	Elastically Restrained	Tension Flange Restrained
Torsional Rotation, z-axis	Elastically Restrained (rigid if firmly attached)	Elastically Restrained (due to lateral brace)

In comparing the end conditions for a typical overhead sign bridge support (Figure 5c) and the test model (Figure 5b) as shown in Table 2, it is apparent that the test model restraints are much less rigid. Therefore, it appears that the proposed formula, which was developed for the test model, will have a factor of safety larger than 2.0 when applied to an actual design.

# Testing Program

Technical Memorandum 605-4

by

### D. L. Ivey, C. E. Buth, R. M. Olson, and T. J. Hirsch

A testing program, consisting of several laboratory static tests and eight full scale crash tests, was conducted to determine: (a) the feasibility of a Prototype OSB with break-away supports from a viewpoint of safety and structural integrity, and (b) the validity of a mathematical model simulation technique.

The first laboratory test was performed to obtain some insight on how the break-away support connections would function under a static load simulating a head-on vehicle collision. In addition to showing that the break-away connections would function satisfactorily, this laboratory test provided information on the energy expended by: (a) the slip base friction due to an applied bolt torque of 50 lb-ft and controlled column axial load (theoretically estimated to be 8 kips in each of the four supports), (b) the 20 ga. sheet metal keeper plate, (c) the four fractured bolts in the upper break-away connection, and (d) the rotational and translational displacements of the support and truss. Other laboratory tests were conducted later in the program to study in greater depth the parameters which affect the frictional resistance of the lower break-away support connection. Data obtained from these laboratory tests are used as input into the mathematical model.
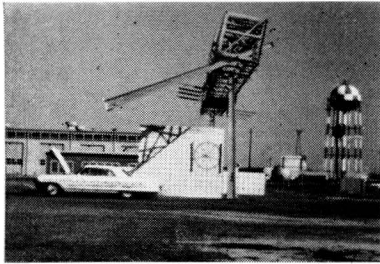
The dynamic behavior of the Prototype OSB break-away support and truss during a head-on 54 mph passenger vehicle collision is shown in Figure 7. As clearly visible, the rotation of the truss was negligible until the rotating support struck and crushed two of the four pipe load distributors mounted on the lower chord members of the truss. In addition to distributing the load and expending a portion of the rotational kinetic energy of the support, the pipes are effective in preventing severe damage of the truss. The horizontal angles located at mid-depth of the truss, as visible in Figure 7, were shown in a later 15 degree angle collision to be effective in preventing severe damage to the truss as the rotating support, twisting about its pin connection, was guided through the truss—it is conceivable that the support would have snagged on the top of the colliding vehicle had the horizontal angles not been used and the support
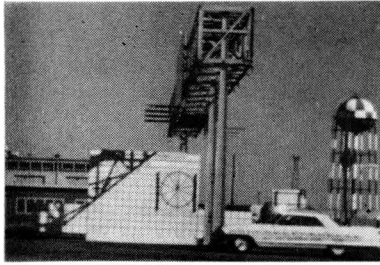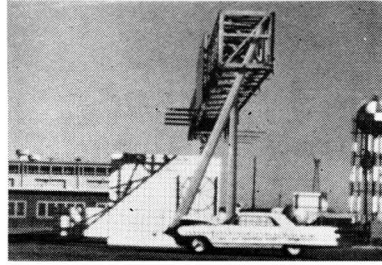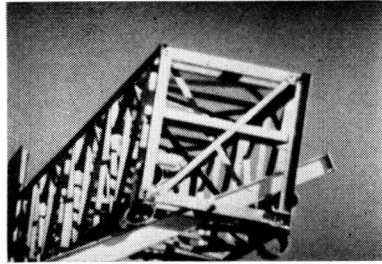
t = 0.010 sec   t = 0.109 sec

t = 0.453 sec   t = 0.453 sec

Figure 7.   Dynamic behavior of the prototype structure (test 605-D).

snagged on the truss; therefore, the horizontal angles serve, like the pipe distributors, a multiple purpose.

A comparison of the results obtained from a high-speed film analysis with the results obtained from a mathematical model simulation was presented earlier in Table 1. The good comparison clearly indicates that the mathematical model can be used with a high degree of confidence to analyze other proposed OSB structures with break-away supports.

A preliminary low-speed 21 mph full scale crash test not shown in Table 1 was conducted following the first laboratory test to primarily confirm that the upper break-away connection would perform satisfactorily under dynamic loading conditions.

During the preliminary crash test the break-away support did not contact the truss; however, during subsequent tests conducted at speeds exceeding 21 mph the support did strike the truss. The amount of rotational kinetic energy imparted to the truss shown in Table 1 was predicted by the mathematical model. Static tests were run on an exterior support to evaluate the torsional stiffness characteristics of the truss and remaining supports, and to evaluate the performance of the pipe distributors (discussed earlier). These tests showed that the bolts in

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Test 605-A	Test 605-G
Impact Speed 25.7 mph	Impact Speed 50.1 mph
Penetration 12 inches	Penetration 19 inches

Figure 8. Typical vehicle damage.

the upper connection of an adjacent interior support would fracture when the truss had expended about 14 kip-ft of energy, and that the truss would need to expend about 145 kip-ft of energy in order to fracture the bolts in both of the interior upper support connections. In three crash tests 605-D, F, and G, an exterior support was struck, and the bolts in the adjacent interior upper support connection were fractured.

The damage sustained by a standard size automobile increases as the impact speed increases, as shown in Figure 8. The automobiles in the two tests were similar models and had the same weight; but the impact speed in Test 605-G was nearly twice that in Test 605-A, and the support penetrated into the vehicle a much greater distance in the high speed collision. The increase in vehicle damage can be explained by noting that a break-away support is accelerated more by the high speed impact, and the resulting inertial force (product of mass-moment-of-inertia and angular acceleration) increases as the speed of impact increases, other conditions being unchanged.

Fabrication and Construction

Technical Memorandum 605-5

by

A. J. Stocker

Contract

A contract for the foundations, complete OSB structure in place, and sign brackets was awarded to Riverside Industries, Incorporated of Ft. Worth, Texas on April 29, 1969, with sixty (60) days required for completion of the project.

Cost of the entire prototype OSB structure including foundations and erection was $18,500. An additional $424 was spent

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for sign brackets and $1,300 for fabrication and erection of the two information signs; the total cost was $20,224.

Fabrication

Only minor changes and substitutions were made relative to the original working plans and specifications. The fabricator initially chose to galvanize the complete OSB structure but because of the development of a warping problem in the first break-away support during welding, it was requested that painting of the supports be allowed. This request was granted because maximum interpass temperature was 300°F for welding the supports and galvanizing is done at 850°F. The possibility of additional warpage during galvanizing was the reason for the request.

The original intention was to use ASTM A-514 Type F steel for the break-away supports and the welding procedure was written on this basis. When it was found that Type F plate had a 60-day delivery, the fabricator's request to use Type A was granted and the welding procedure rewritten and approved.

It was found that the established procedure for alternating welding beads was not followed in the shop on the first support. Adherence to this procedure on the remaining three supports eliminated the camber problem. Camber in the first support fabricated was removed satisfactorily by using a 200-ton jack.

Slower welding speeds and two passes instead of one were required in welding the A-514 steel, as compared with A-36. The fabricator felt that only their most qualified craftsmen should do the job. Both of these factors increased the cost. Table 3 shows a comparison between the welding of A-514 and A-36 steels. The table is included since the prototype overhead sign bridge was fabricated from high strength steel while many contemporary sign bridges are fabricated from A-36 steel.

Table 3

COMPARISON OF WELDING PROCEDURES FOR A-514 AND A-36 STEELS

	A-514	A-36
Welding Speed	13"-15"/min	18"-22"/min
Amperes	200 amps	400 amps
Rod Deposited	2' to 3'/min	3' to 4'/min
Rod Size	1/8"	3/16"
Number of Passes	2	1

Foundations

The four identical foundations consisted of two 30-inch drilled concrete shafts with 60-in. bells joined by a 36-in. by 26-in. beam. The drilled concrete shafts were 9 feet deep.

The foundations have two bolt patterns instead of one required for the single break-away column support in this program. Since many existing overhead sign bridge structures have two-column supports, it was decided to construct the foundations so that a two-column support could be tested at some future date, if desired.

Erection

The 140-ft. truss was delivered to the job site in three sections; the two end sections were 60 feet and the middle section was 20 feet in length. The truss was assembled on the ground and the four break-away supports installed while the truss was on its side. The total OSB assembly was raised into position with two 35-ton truck cranes.

Supplementary Studies

Technical Memorandum 605-6

by

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Additional work was requested by the sponsors after completion of the original contract: (a) additional full-scale crash tests, (b) static laboratory tests on break-away bases and other structural components, (c) parameter studies by mathematical simulation and (d) alternate concepts for support columns.

Two additional full-scale crash tests were conducted; the first car (5,150 lbs) struck an outside support head-on at 75.3 mph and the second car (5,170 lbs) struck an inside support at 72.0 mph at a 15 degree angle. These tests were labeled H and I, respectively, and results are included in Table 1. Comparisons between test results and the computer model were good. A summary of the crash test results on the entire project is given.

During Test H, the rotating support struck the truss with sufficient force to shear off the upper bolts and the truss rotated about the upper pin connections as shown in Figure 9. The structure was not visibly damaged during this test and the truss was repositioned for the next test.

Nine additional laboratory tests were conducted using a column stub and base. It was concluded that variables associated with bolts, nuts and washers caused as much as 100% variation in the peak force required to slip the base and the angle at which the load is applied has very little influence on the value of the peak base force. A crane was used to apply a torque to the truss after Test H and it was found that the truss was less stiff than in a previous torsion test, presumably due to
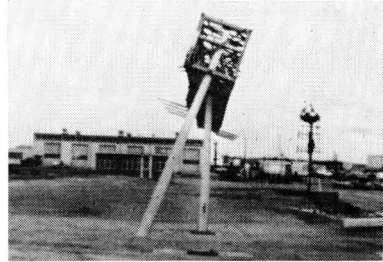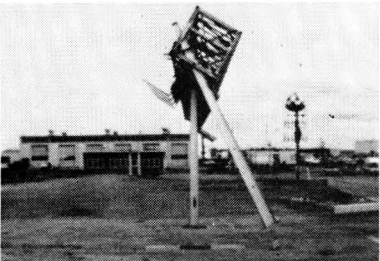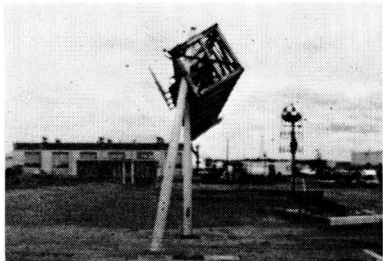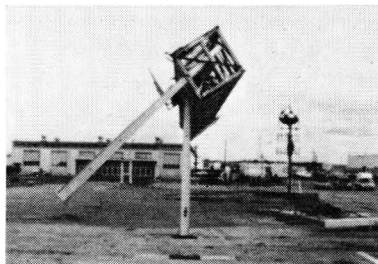
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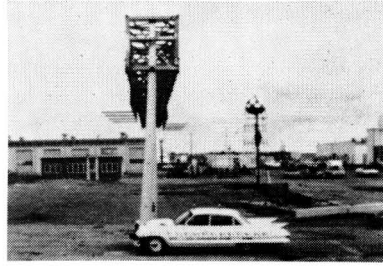
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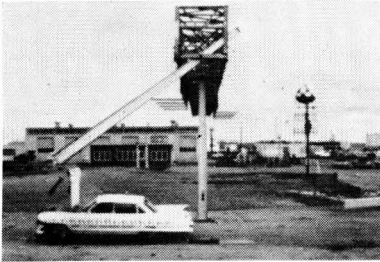
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Figure 9. Sequential photographs of test H.

bolt slip and member deformations in the truss which occurred during the testing program.

Additional parameter studies were conducted to determine (a) behavior of selected vehicles; (b) structural response when support conditions (such as base shear force) and vehicle parameters (vehicle spring constant) were varied, and (c) effect of varying support weight. The study indicated that lighter supports produced lower vehicle velocity and momentum changes, lower average decelerations, and caused less energy to be imparted to the truss by the support as it swings following impact. Also, varying the base connection resistance up to 20 kips does not significantly affect vehicle behavior nor support response.

The mathematical simulation correlated very satisfactorily with test results and provided reliable information to fill the gaps in the test results.

Alternative concepts for the support columns were suggested by various individuals during the course of this study. These concepts are discussed in the appendix to TM 605-6. Some of these demonstrate a good knowledge of the problem and present ingenious methods of improving the prototype column support.

The authors are grateful for the interest shown in this project by many of the highway engineers and for the constructive comments of the reviewers.

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