HIGHWAY RESEARCH BOARD
Special Report 10

Investigation of Wind Forces On Highway Bridges

PRESENTED AT THE

Thirty-Second Annual Meeting

January 13-16, 1953

1953
Washington, D.C.

National Academy of Sciences—
National Research Council

publication 272

HIGHWAY RESEARCH BOARD

1953

2101 Constitution Avenue, Washington 25, D. C.

HIGHWAY RESEARCH BOARD

Special Report 10

Investigation of Wind Forces On Highway Bridges

PRESENTED AT THE

Thirty-Second Annual Meeting

January 13-16, 1953

1953

Washington, D.C.

DEPARTMENT OF DESIGN

C. N. CONNER, *Chairman*

Principal Highway Engineer, Bureau of Public Roads

Committee on Bridges

G. S. PAXSON, *Chairman*

Bridge Engineer, Oregon State Highway Commission, Salem, Oregon

ARCHIBALD, RAYMOND, *Bureau of Public Roads, 2161 Flood Building, 870 Market Street, San Francisco 2, California*

ERICKSON, E. L., *Bureau of Public Roads, U. S. Department of Commerce, Washington 25, D. C.*

FOSTER, GEORGE M., *Chief Deputy Commissioner, Michigan State Highway Department, Lansing 13, Michigan*

GUNTER, T. B. JR., *Bridge Engineer, North Carolina State Highway and Public Works Commission, Raleigh, North Carolina*

HOGAN, JOHN T., *Consulting Structural Engineer, Portland Cement Association, 347 Madison Avenue, New York, N. Y.*

QUADE, M. N., *Consulting Engineer, 51 Broadway, New York 6, N. Y.*

ROWE, R. ROBINSON, *Supervising Bridge Engineer, California Division of Highways, Box 1499, Sacramento 7, California*

SIESS, C. P., *Department of Civil Engineering, University of Illinois, Urbana, Illinois*

Investigation of Wind Forces on Highway Bridges

G. S. VINCENT, *Principal Highway Bridge Engineer, Bureau of Public Roads*

The models tested represented a 100-ft. pony-truss span with 22-ft. roadway to a 1:16 scale and two 250-ft. through high trusses of light design with 24-ft. and 36-ft. roadway, respectively, to a 1:24 scale. There were also 1:24 scale models of deck plate girder bridges 9 ft. deep having two, four, and six girders and 28-ft., 34-ft., 48-ft., and 52-ft. roadways, some with and some without 5-ft. cantilevered sidewalks.

Force and moment coefficients were determined at a wind velocity of 100 mph. after testing each model at velocities down to 25 mph. to establish the absence of Reynolds number or scale effects.

On the basis of these tests the specified wind load of 50 lb. per sq. ft. on 1½ times the area of the bridge in elevation is adequate for the transverse forces on trusses without live load in a 100-mph. wind and a considerably lighter loading per square foot is indicated by the pony truss and deck girder models. In localities exposed to extreme winds, perhaps heavier loading should be used.

The ratio of maximum longitudinal to maximum horizontal force was 0.60 and 0.50 for the through trusses with 36-ft. and 24-ft. roadway, respectively, 0.25 for the pony truss and less than this for the deck girder spans. These indications tend to support the provisions of Art. 3.2.14 of the 1953 American Association of State Highway Officials bridge specifications specifying a longitudinal wind force of 50 percent and 25 percent of the lateral force for trusses and girders, respectively.

The tests showed that the wind forces generally increase with increased width of bridge whether or not additional supporting members are used but no clear-cut nor consistent criterion for this increase is evident.

An upward angle of only 5 deg. materially increases most of the wind forces and upward angles of 5 deg. and 10 deg. revealed strong vertical forces and overturning moments not provided for in the specifications. Winds of this nature occurring in gusts or near bluffs or other features of the terrain may have contributed to the failure of some bridges in the past. The report presents an attempt to set up a criterion for these lifting and overturning forces.

The tests leave unanswered questions such as the effects of through girders, deck trusses and open grid floors. It is desirable to investigate further the wide spread in wind loading between through trusses and pony trusses.

● SPECIFICATIONS stipulating the wind forces for which bridges shall be designed have been developed over an extended period. They have conformed, in general, to formulas proposed from time to time on the basis of tests on flat plates of various sizes and shapes. They have probably been influenced also by historic failures of bridges due to wind action. The requirements are probably conservative for most bridges, although they have undoubtedly been exceeded at times on some bridges or portions of bridges. There is good reason to suspect that they do not provide adequately for force components which may occur under certain wind conditions arising from gusts or features of the terrain, such as bluffs, or the shape of the structure. On the other hand, certain assumptions, such as the use of 70 percent of the specified transverse wind force as a longitudinal force to be resisted by towers and substructures have been considered excessive. Usually this has only a moderate effect on a bridge design, but in 1950, in two states, bridge plans were developed using several continuous spans and the requirements for resisting the longitudinal wind force at the single anchor pier increased the cost some $20,000 above the cost of a design based on what was considered a more realistic loading.

The Bridge Committee of the Department of Design of the Highway Research Board considered this situation and proposed a program of model tests to verify this and other provisions concerning the wind loading. A cooperative project with these objectives was subsequently set up with funds contributed by the highway departments of Colorado, District of Columbia, Florida, Georgia, Hawaii, Kentucky, Louisiana, Maine, Maryland, Massachusetts, Missouri, New Jersey, Ohio, Oregon, Pennsylvania, Virginia, Washington, and Wisconsin. The Bureau of Public Roads contributed engineering and consulting services. Following several conferences of the interested parties, the models were designed and detailed by the Bridge Branch of the Bureau of Public Roads, built under

contract by an experienced builder of aerodynamic models and tested by the Bureau of Aeronautics of the Navy Department in its 8- by 10-ft. wind tunnel at the David Taylor Model Basin.

Nomenclature

Pitch angle or angle of attack (upward +)	θ
Skew angle or "yaw"	ψ
True pitch angle $\sin \alpha = \sin \theta \cos \psi$	α
Coefficient of pitching moment (about longitudinal axis)	C_m
Coefficient of yawing moment (about vertical axis)	C_n
Span length, ft.	b
Characteristic length, ft., width of bridge deck ±	c
Moment center below deck, inches	d
Yawing moment, ft.-lb., $N = C_n qAB$	N
Drag (transverse force), lb.	D

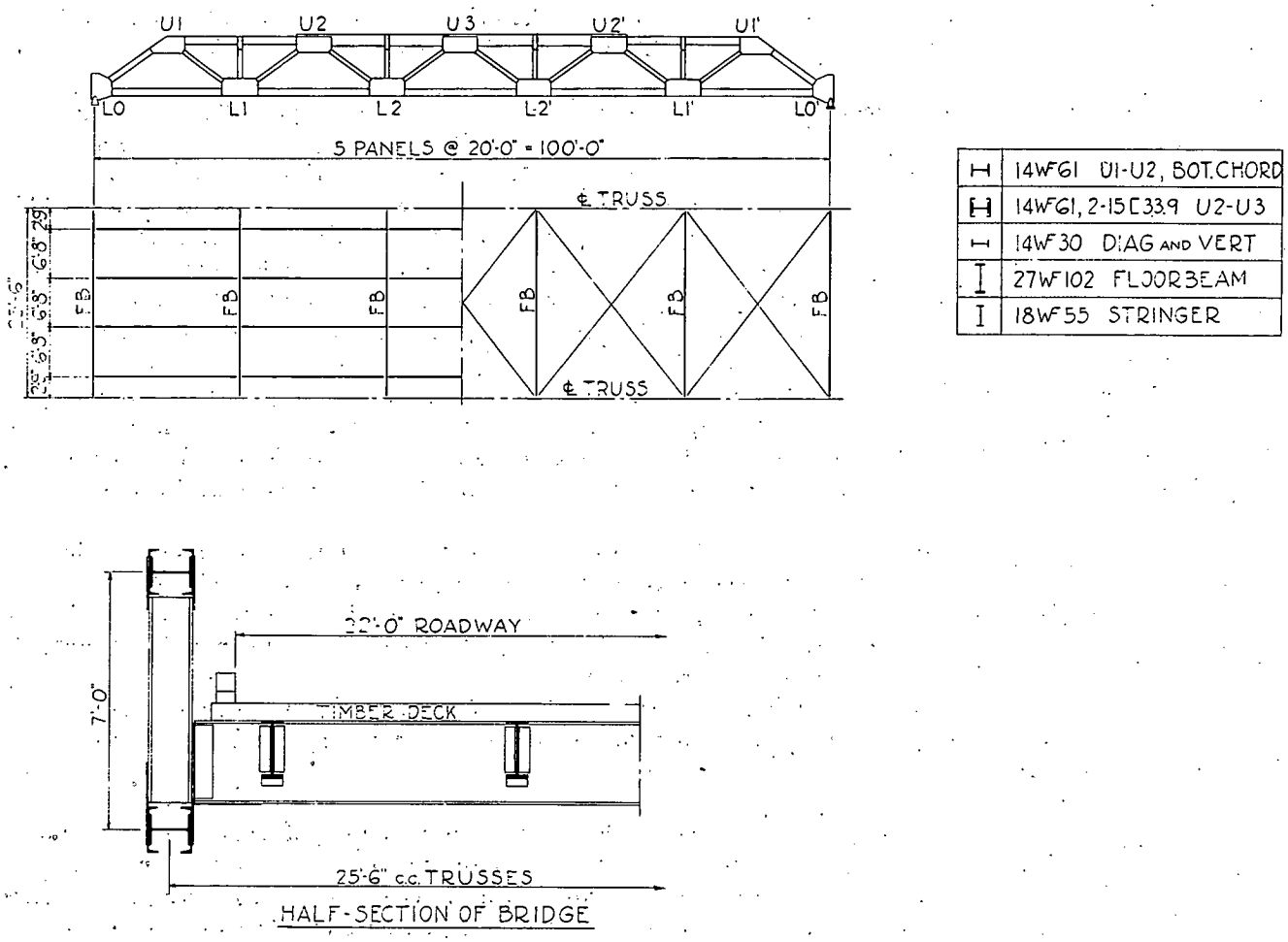


Figure 1. Prototype of pony-truss model.

True skew angle $\tan \beta = \tan \psi \sec \theta$	β
Wind velocity, feet per second	V
Area, sq. ft. (seen in side elevation)	A
Air mass density, slugs per cu. ft.	ρ
Coefficient of force	C
Dynamic pressure, lb. per sq. ft., $\frac{\rho V^2}{2} = q$	q
Coefficient of transverse force component	C_D
Coefficient of vertical force component	C_N
Coefficient of longitudinal force component	C_Y
Coefficient of rolling moment (about transverse axis)	C_r
Coefficient of uplift at windward shoes	C_L
Coefficient of resultant force	C_R
Transverse distance between shoes, ft.	c'
Vertical distance from reference point to bottom chord, ft.	d'
Coefficient of vertical force at windward quarter point	C_v
C_v referred to area in plane	C'_v

Program of Tests

The program provided for testing through trusses, pony trusses, and deck-plate girders with solid decks

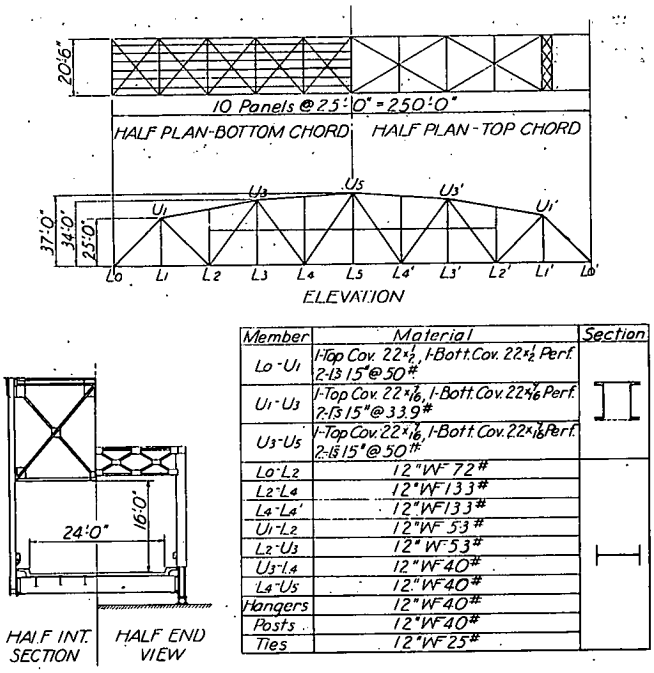


Figure 2. Prototype of through-truss models.

of conventional design, consideration being given to later testing of through girders, deck trusses, and open-type decks.

For the pony truss the Army Engineers' standard 100-ft. span with 22-ft. timber deck and H-beam truss members was selected because a very well detailed 1:16 scale model was available (see Figure 1).

For the through truss a fairly light design was selected, because the unit wind pressure is generally greater when the individual members are slender, and it was intended that the tests should be representative of the most severe conditions likely to be encountered. A 250-ft. span with 24-ft. roadway built on the Alaska Highway was selected. In order to study the effect of additional width, the same design was used with a 36-ft. roadway with a slight increase in floorbeam depth and necessary modifications in the bracing (see Figure 2).

The basic deck-girder design was taken from a 200-ft. span of the Chesapeake Bay Bridge with girders 9 ft. deep (see Fig. 3). It was desired to investigate to some extent the effects of width, the number of girders and the amount of overhang outside of the girders. Typical arrangements, were selected as follows: (1) four girders, 28-ft. roadway and 8-ft. sidewalks with sidewalks cantilevered beyond the girders; (2) four girders, 28-ft. roadway and 2-ft. wide curbs, with girders approximately under curb line; (3) six girders, 48-ft. roadway and 7-ft. sidewalks with sidewalks cantilevered beyond girders; (4) six girders, 48-ft. roadway and 2-ft. wide curbs with girders approximately under curb line; and (5) two girders, 28-ft. roadway and 2-ft. wide curbs.

Figure 6 shows dimensions of the 1:24-scale girder models with the alternate sidewalk arrangements. These two widths of overhang represent reasonable limits of the relative overhang used in bridge design. The two-girder model was obtained by removing the two interior girders of the four-girder model.

Models

Steel members of the pony-truss span were accurately represented by aluminum plates and extruded sections and connected by miniature bolts. The deck was of laminated wood cut to scale but was replaced for the tests by a ¼-in. aluminum plate to stiffen the model for mounting on the single spindle in the wind tunnel. The principal dimensions of the model are shown in Figure 4.

Figure 5 shows the 1:24-scale models as designed to represent the 250-ft. through truss with 24-ft. and 36-ft. roadways. However, this would have been too long to clear the wind-tunnel walls in the skewed position and the models were actually made slightly shorter by reducing the end panels to two thirds of their proper length, as may be detected in the photographs (Figure 9 and 11). Structural steel members were made from 24-gauge galvanized iron, bent to form angles, channels, and I beams and assembled by spot welding. Plywood (¼-in.) was used for the decks and plywood strips formed the curbs. The rail was modeled by ¼-in. wire mesh of 0.05-in. wire giving a solidity ratio (solid area over total area) of about 0.36. The middle floor beam and some of the stringers were cut to admit the steel plate of the mounting spindle which was bolted to the bottom of the floor. Principal dimensions and typical details are shown

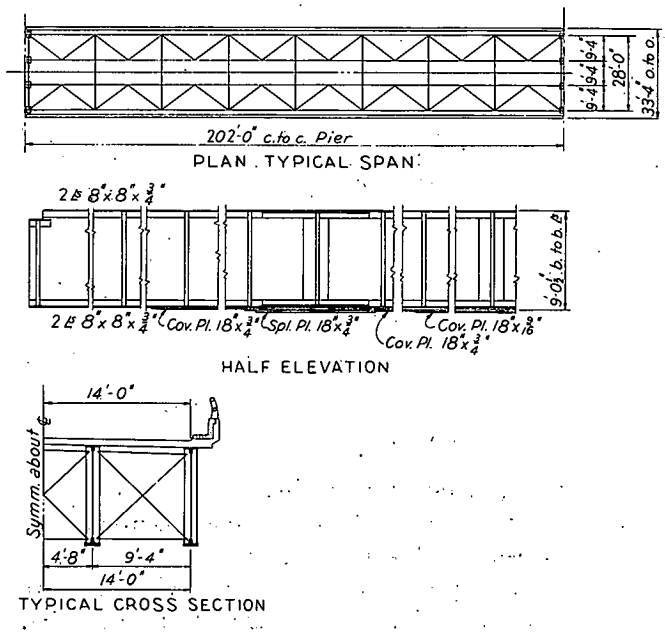


Figure 3. Prototype of deck-girder models.

in Figure 5. The models for the span were didvided into three separate parts. The middle portion or "live" model, 78 in. long was to be mounted on the spindle so that all components of wind force and moment acting on it, equivalent to the middle 156 ft. of prototype, could be measured. The two end portions, "dummy" models of the same construction, were provided for separate mounting in correct alignment and grade but not in contact with the live model. Their function was to insure the correct wind flow over all parts of the live model corresponding to the wind flow over the corresponding portion of the prototype.

The deck-girder models also were formed of galvanized iron, fabricated to represent the structural steel to scale and with plywood floors, sidewalks and curbs and wire mesh rails. It was necessary to use ¾-in. plywood for the floors to provide the stiffness required to sustain the wind forces on the relatively large girder areas at 100 mph. This departure from true scale was not considered significant as it represented extra material between the girders immedi-

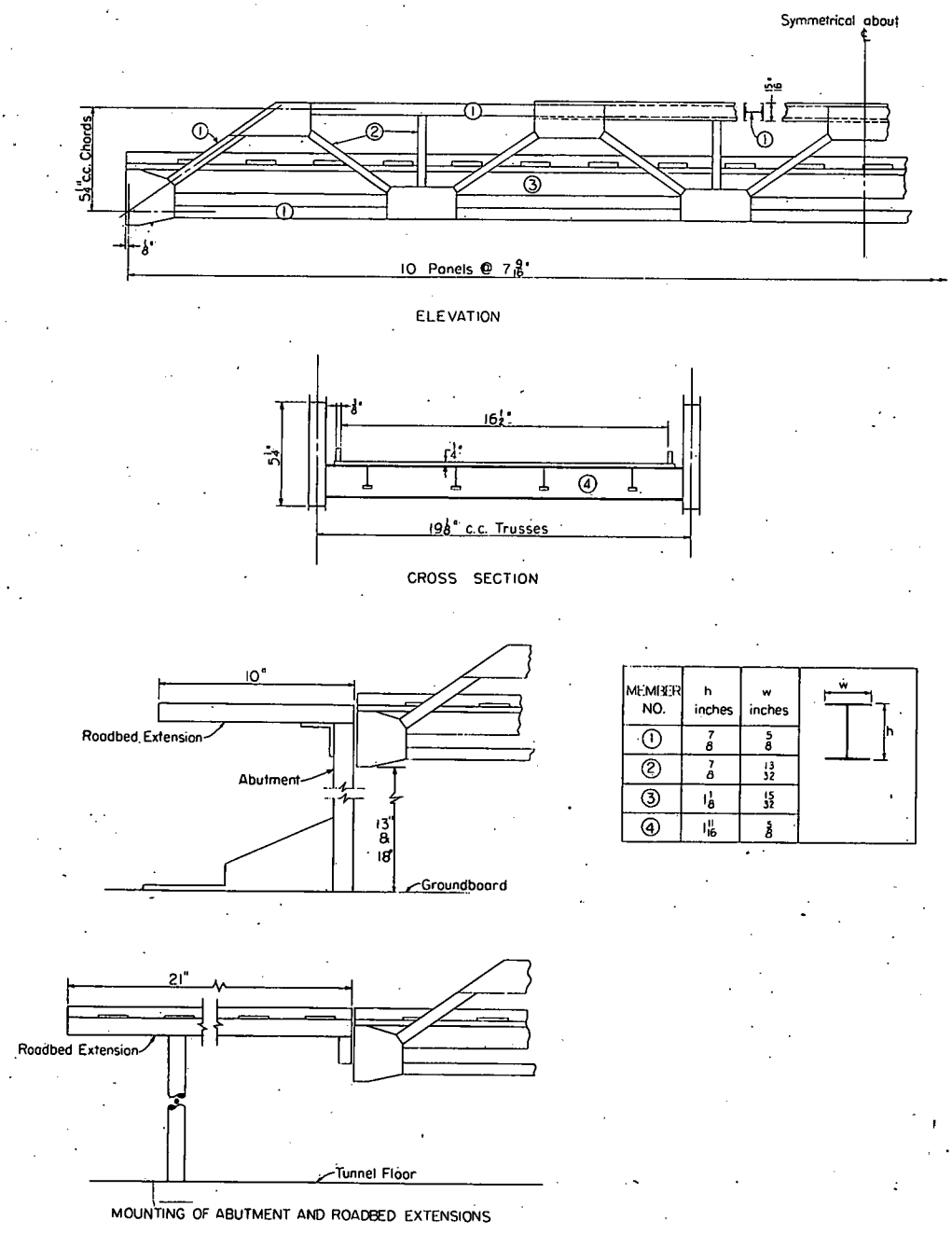


Figure 4. Sketch of pony-truss model.

ately under the floor where it could not affect the wind flow. The excess depth was beveled outside the girders representing with only small distortion a typical filleted cantilever construction. Only two models were built, with four and six girders, respectively—the sidewalks and wide curbs for each model being interchangeable. These models also were cut into live and dummy parts (see Fig. 6).

Tests at David Taylor Model Basin

Figures 7 to 13 inclusive are photographs of the various models mounted in the wind tunnel. The spindle supporting the live model was in turn supported by the reactions to six weighing machines beneath the tunnel. From the weighted reactions the three components of force and of moment acting at any selected point on a live model could be computed. A warning light indicated any contact of the spindle or live model with other parts of the tunnel or the dummy model which occasionally occurred because of deflection or vibration, whereupon the test run was interrupted and the condition corrected. Signal lights for each weighing machine showed while automatic devices operated to compensate for elastic yielding and restore the model supports to normal position. Pressing a button while all lights were off recorded each scale reading on a paper roll.

Because of topography and temperature effects, wind streams are often not horizontal. The angle from the horizontal is, in aeronautical language, called "pitch" or "angle of attack." Pitch angles are designated in this report by the letter θ.

The lateral tilt of the model to set the pitch angle or vertical angle of attack, of the wind was fixed by inserting suitable wedges between the model floor and the steel plate of the spindle head to which the model was bolted. All models were tested at pitch angles, θ, from -5 deg. (downward) to $+10$ deg. (upward) with some tests at -10 and $+15$ deg. Skewed angles are designated by the letter ψ.

Any desired angle of skew, called "yaw" in aeronautical practice, could be set in a few seconds by rotating the spindle with an electric motor and worm drive which also actuated a set of dials showing the angular position of the spindle. It was necessary to shift the dummy extensions whose supports were attached by screws to the tunnel floor. The dummies were made long enough to produce the correct flow of wind across the live models at all angles of skew to be tested, and the live models were as long as they

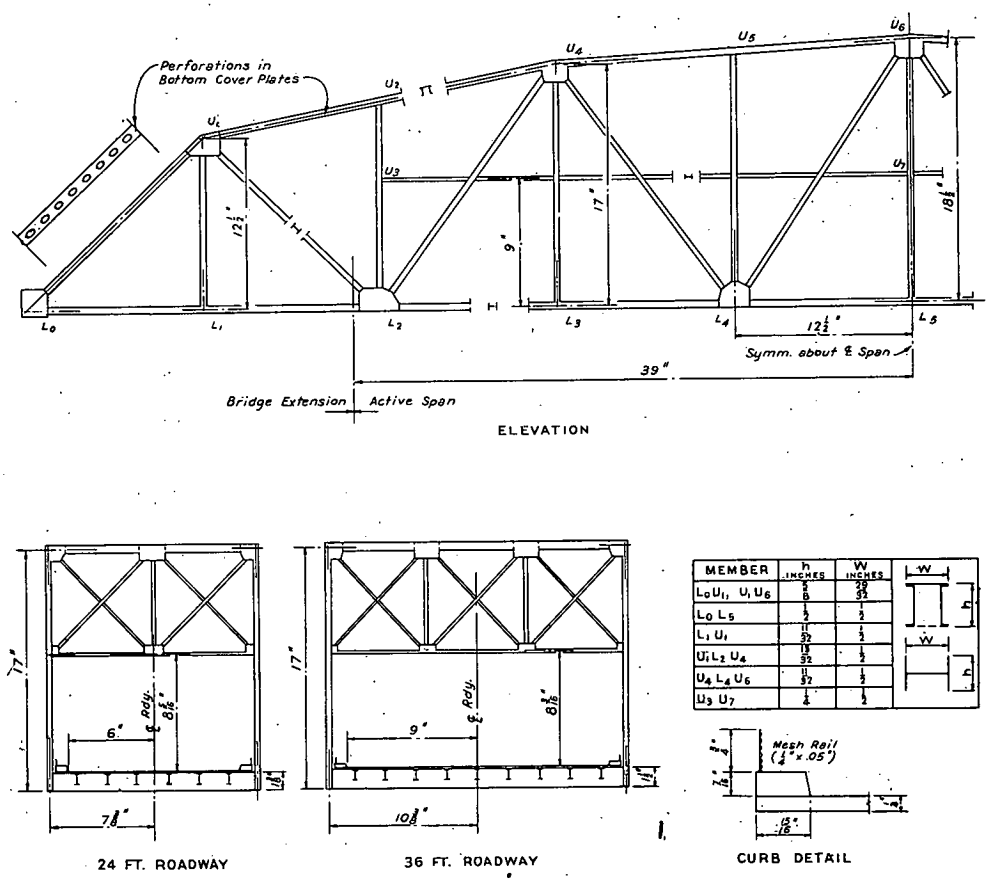


Figure 5. Sketch of through-truss models.

could be without bringing the dummies against the tunnel walls in the skewed positions. All models were tested at each 10-deg. interval of skew or yaw, ψ, from 0 to 40 deg. and at 60 deg. Some were tested at 75 deg.

It should be noted that angles θ and ψ are not the

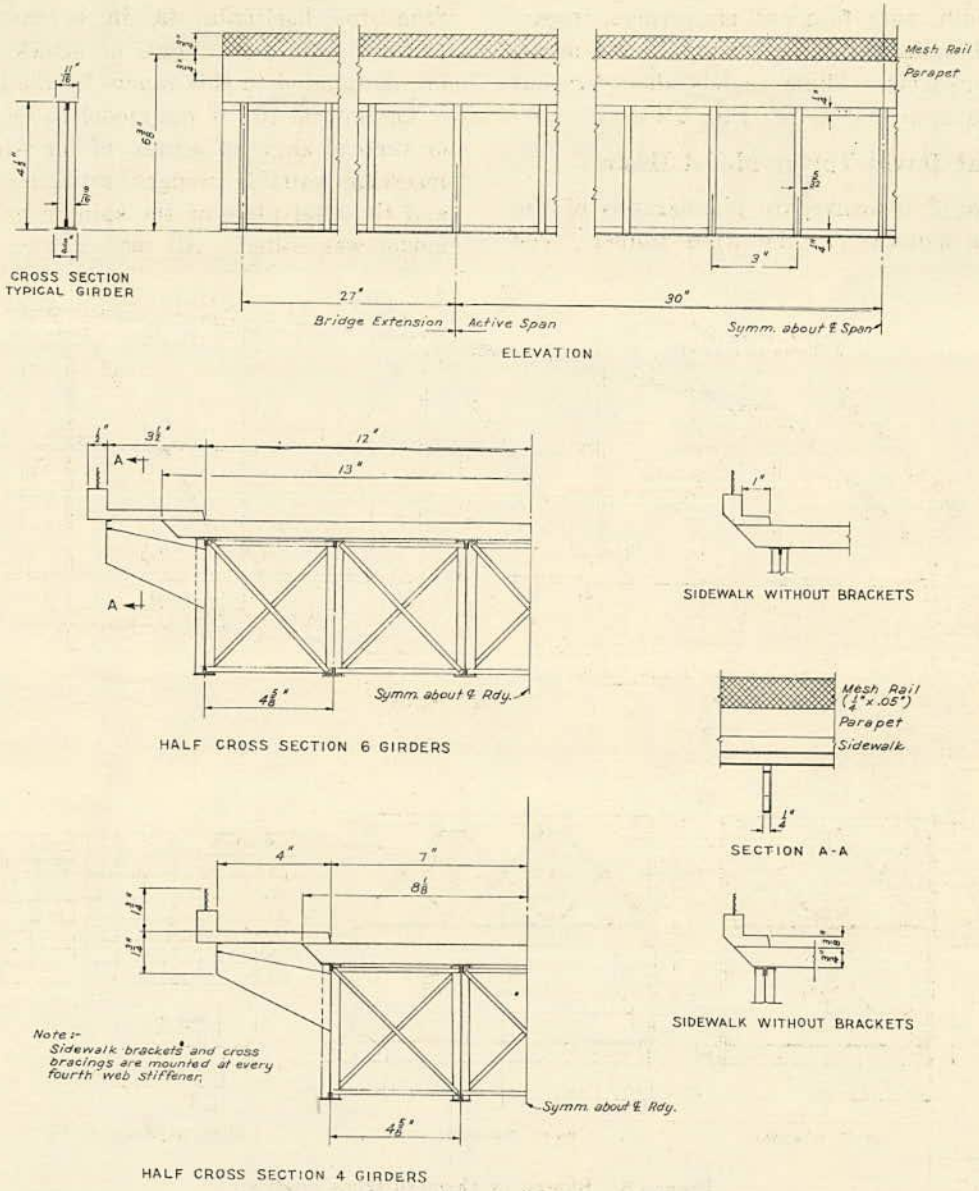


Figure 6. Sketch of deck-girder methods.

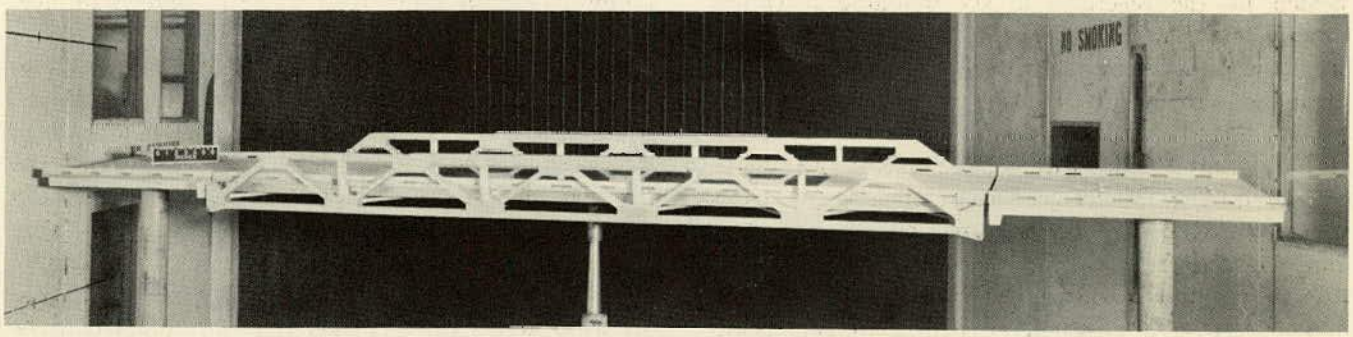


Figure 7. Photograph of pony-truss model with dummy extensions.

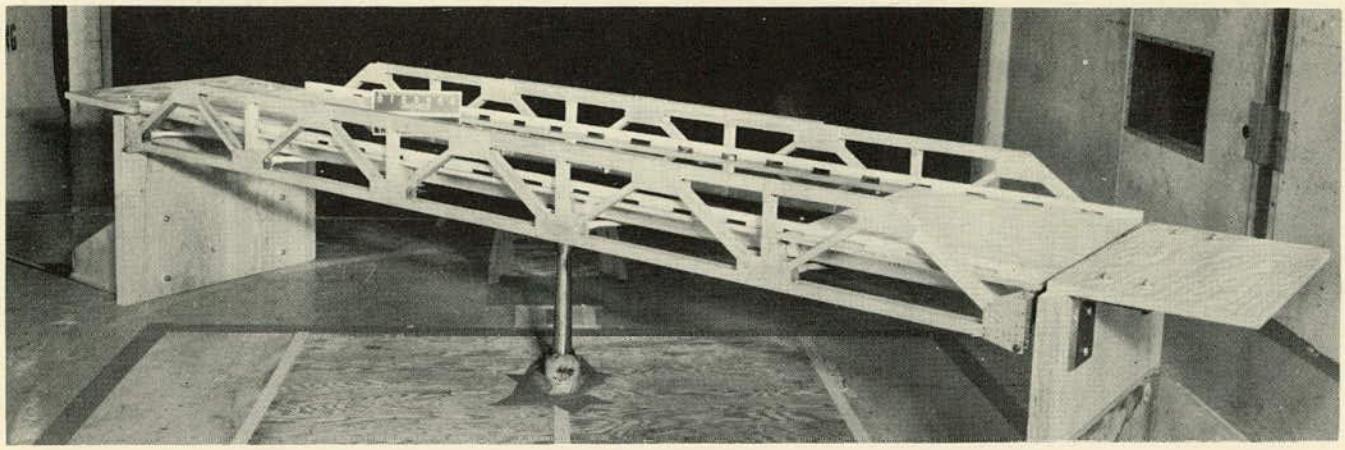


Figure 8. Photograph of pony-truss model with abutments and ground board.

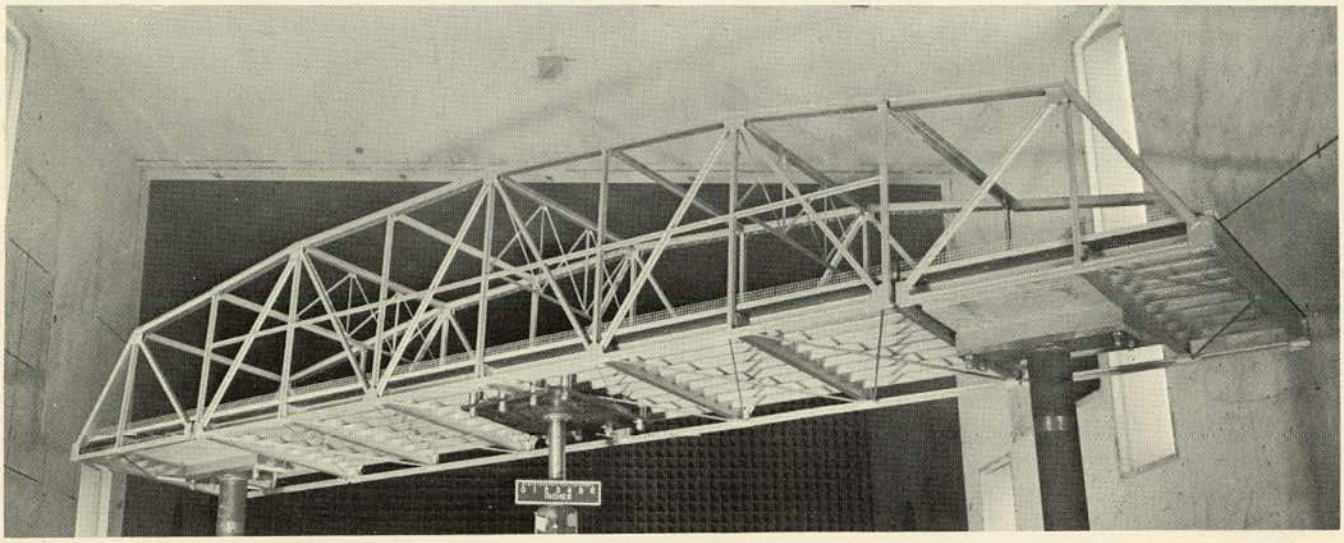


Figure 9. Photograph of through-truss model with 36-ft. roadway.

same as the true pitch angles and skew, except where $\psi=0$ deg. The model is tilted laterally by the angle θ, but as the wind approaches from increasing skew the effective angle of attack with reference to the wind becomes less, and is equal to zero at a 90-deg. skew.

If α designates the true pitch angle, and β the true skew angle, then the following equation can be written:

$$\sin \alpha = \sin \theta \cos \psi$$
$$\tan \beta = \tan \psi \sec \theta$$

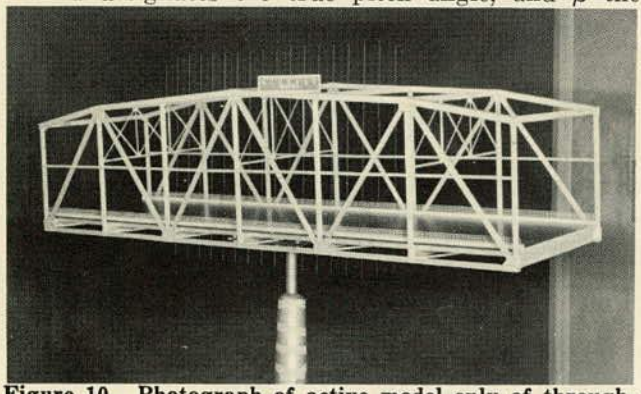


Figure 10. Photograph of active model only of through-truss with 36-ft. roadway.

The difference between ψ and the true angle of skew, β, is of no consequence, but the true pitch angle may be considerably less than θ as indicated by the following tabulation for θ equals 10 deg.

Yaw Angle ψ	Pitch Angle α
0°	10.00°
15°	9.66°
30°	8.65°
45°	7.05°
60°	4.98°
75°	2.58°

For reference, all of the data from these tests are worked on the bases of ψ and θ, but it should be kept

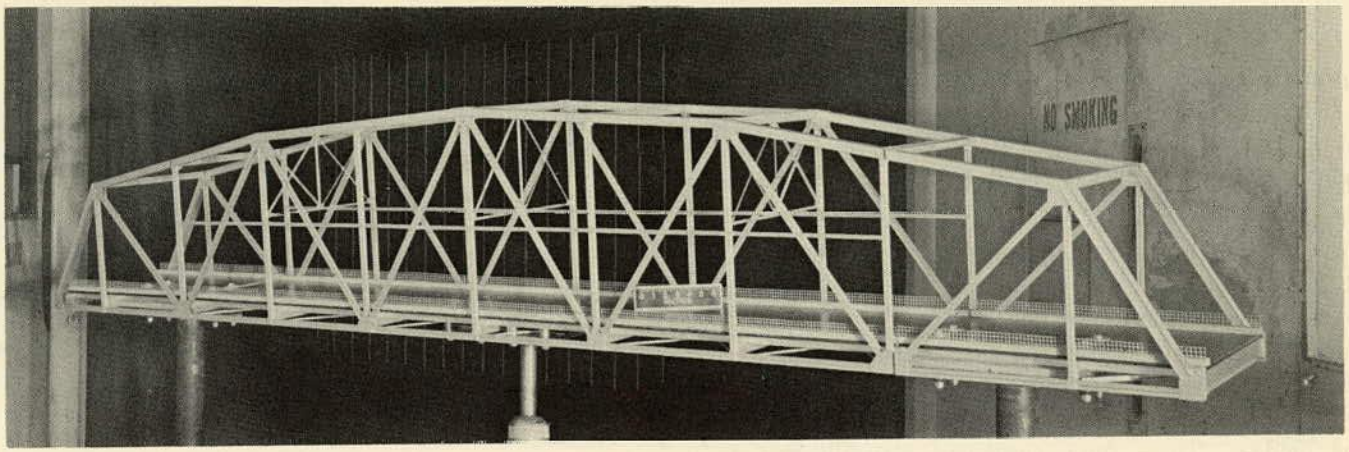


Figure 11. Photograph of through-truss model with 24-ft. roadway.

in mind when interpreting data for tests at a considerable skew angle, that the actual pitch angle, α, is much less than θ. There is some compensation in the fact that with a wind approaching at a sharp skew a structure which is located at a normal elevation above the water, there is less possibility of a large effective pitch angle occurring over the entire span than is true when the wind approaches at right angles to the bridge.

All principal tests were run with the dummies in place, but a few runs were made without them, most of these on the pony truss model, for purposes of comparison only. Wind forces on a span thus exposed and without end supports are quite different from those on a span as a normal component of a bridge, even for a wind at right angles to the bridge axis.

The pony-truss model was also tested with the equivalent of idealized abutments in place and with a plywood sheet or ground board representing flat ground or water 17 ft. and 24 ft. (prototype) below the bottom chord in order to get data on the relative effect of the change in wind pattern occasioned by such obstructions.

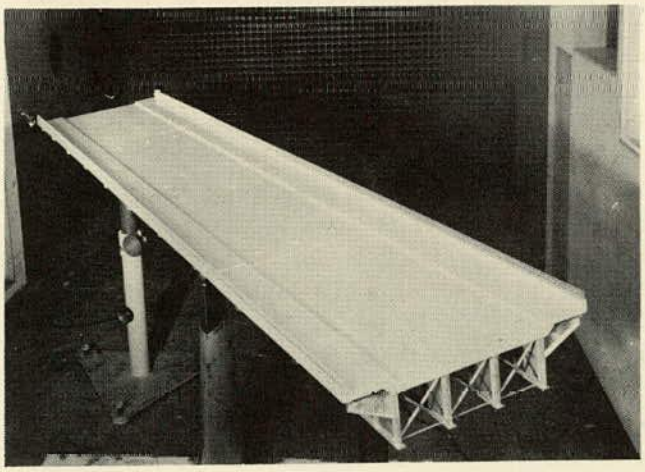


Figure 12. Photograph of four-girder model with wide sidewalk.

The tests for record were run at a wind velocity of 100 mph. but these were preceded, for each model, by test runs at 25 and 60 mph. to detect any Reynolds number of scale effect. The measured forces were found to be proportional to the square of the velocity indicating the absence of such effect and reasonably establishing the reliability of the model tests as an indication of the wind forces on the prototype at any velocity. (A brief discussion of the significance of the Reynolds number, an important parameter associated with fluid mechanics, is given in Appendix B.

Interpretation of Coefficients

The test data were recorded in the convenient coefficient form used in aeronautical practice: The wind force acting on an object may be expressed:

$$\text{Force} = C\rho\frac{V^2}{2}A \qquad (1)$$

in which ρ is the mass density of air (0.00238 slugs per cu. ft. at sea level and 15 deg. C.).

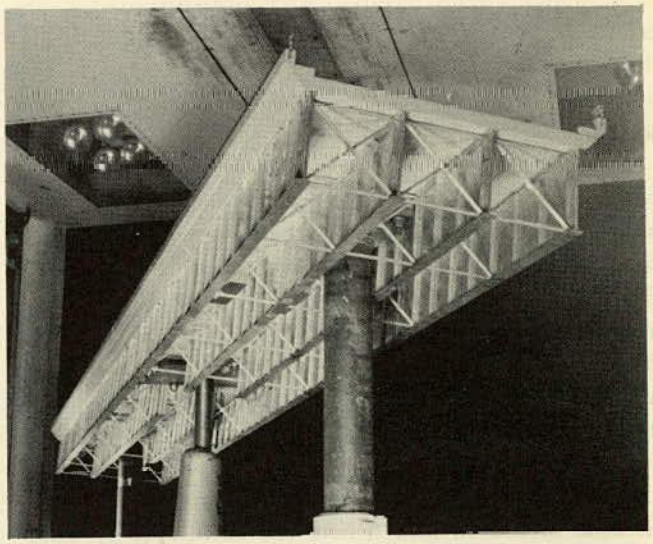


Figure 13. Photograph of four-girder model with narrow sidewalk.

V is the wind velocity in feet per second.

A is the area of the body exposed in the wind in square feet.

C is an experimental coefficient.

The force components in different directions differ, of course, and to distinguish them subscripts are used for the coefficient, C. Thus, C_D, is the coefficient for "drag," that is, the total horizontal transverse force at right angles to the longitudinal axis of the bridge. C_N is the coefficient for the "normal" or total vertical force acting on the bridge and C_Y is the coefficient for the "side" force as referred to the wind which is the total force acting parallel to the longitudinal axis of the bridge. These force coefficients are illustrated graphically in Figure 14. Their numerical values were determined by measuring the corresponding total forces and dividing these by $\frac{\rho V^2}{2}$.

If Equation 1 is divided by A it will express the average pressure per square foot of area in elevation. The term $C\rho V^2/2$ will also express the unit pressure acting normal to the surface at any point on the object if the proper experimentally determined value of C is used. V is always the velocity of the approaching wind, but the numerical value of C varies from point to point across the surface, depending on the modified velocity and direction of the wind at the particular point as explained in the text in connection with Figure 20. There is usually some point or line on the windward or leading edge and sometimes on the leeward or trailing edge where the wind stream divides and where the air is stationary over a very small area. This is called a "stagnation point." At such a point $C = +1$ and the pressure is equal to $\rho V^2/2$ which is the maximum positive unit pressure which can be developed in bringing the windstream to a complete stop locally. This pressure, $\rho V^2/2$, is called the "dynamic pressure" and designated by the abbreviation, q. Its value for a velocity of 100 mph. in air at sea level and 15 C. is 25.58 lb. per sq. ft.

The wind, when diverted by the object, crosses some parts of the surface at velocities greater than that of the approaching windstream, resulting in a reduction in pressure over those parts in accordance with Bernoulli's Law. Over such areas there is, relatively, a suction, and C is negative. Negative values of C range up to -3 or -4 over sharply curved surfaces and may be several times as large over very small areas at sharp edges or corners.

The total force in a given direction on the body is the resultant of all unit pressure components acting in that direction, and hence, the coefficient of total force may exceed $+1$ (it usually does for some force components).

Figure 14 shows a sketch of an idealized flat model in a wind that approaches at a skew angle, ψ, and a deviation from the horizontal of $+\theta$ (plus indicating an upward wind). It shows also and defines the coefficients for the total force components, adapted from aerodynamic practice. C_D, C_N and C_Y are for the transverse, vertical, and longitudinal force components respectively.

The moments of the wind forces about the transverse, longitudinal, and vertical axes can be represented as the products of the dynamic pressure, q, the area, A, as seen in elevation and experimental coefficients, multiplied in each case by a distance, to yield moment.

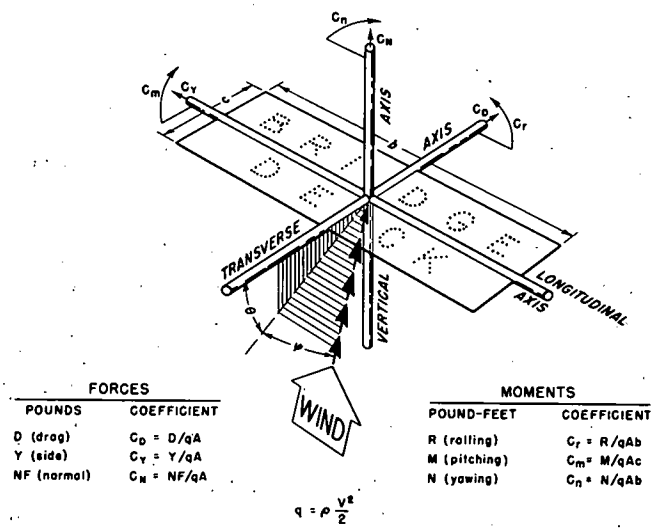


Figure 14. Notations and diagrams of force and moment components.

The area represented by A is selected for convenience in reference to the forces considered. In all of these tests and for all forces and moments it was the area as seen in side elevation. Thus, a glance at the coefficients gives the relative strengths of the various force components; this is an advantage which would be lost if each were computed for a different area, as for example, if the vertical force coefficient, C_N, were computed for the area in plan, which would otherwise be a logical procedure.

C_Dq is the average unit pressure of the lateral or transverse force over the area in elevation. If tests indicated C_D to be 2.93 for a particular model the unit pressure in a 100-mph. wind would be $2.93 \times 25.58 = 75$ lb. per sq. ft. which is 50 lb. per sq. ft. on 1½ times the area.

The forces measured by the scales in these tests were those parallel to the three axes of the wind tunnel and the moments were determined about these axes. The force and moment coefficients computed from these measurements were converted by the

Bureau of Aeronautics personnel to the coefficients referred to the axes of the model as shown in Figure 14 by use of the proper trigonometric relationships.

TABLE 1

MODEL DIMENSIONS

Model	Area in Elevation A *	Elevation center of gravity above centerline bottom chord or back of lower flange	Span b	Characteristic length c	Distance of moment center below top of deck d
	sq. ft.	ft.	ft.	ft.	in.
Pony truss	2.0833	0.218	6.312	1.458	0.25
Through truss 24-ft. roadway	2.03	0.509	6.302	1.156	1.40
Through truss 36-ft. roadway	2.03	0.509	6.302	1.656	1.40
Deck girder 4-girder	2.6562	0.277	5.000	1.3541	1.40
Deck girder 6-girder	2.6562	0.277	5.000	2.1666	1.40

* Neglecting area of wire-mesh handrail.

For each model the elevation of the reference point for moments with respect to the top of the deck was indicated. Graphs were prepared indicating the various coefficients plotted against yaw for different angles of attack. These graphs are reproduced in Appendix C. They were used to develop the curves and figures which follow.

Table 1 shows the areas and controlling dimensions used by the Bureau of Aeronautics in computing the coefficients and which must be employed in using the data to compute forces, moments, and reactions. The areas given do not include anything for the wire-mesh railing. An independent check gave areas differing from those shown by 2 percent or less. When the solid area of the rail was added the total areas exceeded those tabulated for the pony truss, through truss and girder by 8, 9, and 5 percent respectively. Thus, the plotted coefficients are moderately conservative, if the rail is taken into account in computing the area.

Presentation of Data

Table 2 shows in a general way the wind forces which affect different elements in the design of bridges and the correlation of these with the six force and moment coefficients. As a first step in analyzing the data a study was made of the effects of the supplemental factors, C_n and C_r.

From Figure 14 the yawing moment about the vertical axis of symmetry is:

$$N = C_n q A b \qquad (2)$$

If this moment is resisted by reactions at the two ends (as for the pony truss model) each end reaction will be $N/b = C_n q A$. Since the drag D is resisted equally by the two end supports the total horizontal reaction at one end is:

$$(C_D/2 \pm C_n) A q \qquad (3)$$

for one end or half this amount per shoe. This illustrates the convenience of selecting the dimension, b, in computing C_n.

It was found that C_n increased the horizontal reaction of one end by 0 percent to about 10 percent but this was not controlling because: (1) its effect was negligible at 0-deg. skew at which C_D was a maximum and (2) at those skew angles where C_n contributed appreciably to the end reaction the combined effects of C_n and C_D was less than that of C_D alone at 0-deg. skew.

Similar analysis shows that when the rolling moment is significant the vertical reaction at one end is $(C_N/2 \pm C_r) A q$. For the pony truss span C_r ranged up to more than 10 percent of $C_N/2$ but was negligible for the condition (skew angle and pitch) which gave maximum uplift. For the girders and through trusses its effect was greater for some conditions but it should be noted that for these the active or live models were relatively shorter which possibly tends to increase the value of the coefficient, and it is hardly to be expected that C_r would contribute greatly to the lift reaction for such a span length, for example, as the combined lengths of the dummy and live models.

Both C_n and C_r were small except in a skewed wind. Generally, C_r was positive tending to increase the uplift at the windward end. C_n was predominantly negative for the girder models, increasing the transverse wind reaction at the windward end but usually the reverse for the pony truss span.

Altogether, the supplemental factors, C_n and C_r, were not important, but it is conceivable that they may be for some bridges justifying a conservative attitude in considering the major wind forces.

TABLE 2

Wind effect	Bridge elements affected	Applicable test data	
		Primary	Supplemental
Transverse force	Laterals Portals Sway Frames Shoes (Lateral Force) Bents (Shear)	C_D	C_n (Tends to load one end reaction with more than 50% of total transverse force)
Longitudinal force	Fixed Shoes Bents Piers Foundations	C_Y	
Uplift and overturning moment	Bents Piers Shoes Anchors Foundations	C_N, C_D, C_m	C_r (Tends to increase uplift at one end)

Effect of Abutments and Ground Board

The tests with the ground board and abutment were made only on the pony truss model and in a horizontal wind. The relative influence of these conditions must be judged by comparison with the horizontal wind tests of this model with dummy extensions but without abutments and ground board.

Transverse Force

From Figures B and D (appendix) it is seen that the maximum value of C_D with the abutments occurred at $\psi = 20$ deg. and exceeded the maximum value without abutments, (which occurred at $\psi = 0$ deg.) by 7 percent. The effect of C_n is small for both of these conditions.

Longitudinal Force

The same figures show that C_Y was reduced slightly by the presence of the abutment and ground board.

Uplift

The dimension, c, used in determining C_m is not quite equal to the distance center of center of trusses but may be assumed to be for the purposes of determining approximate relative effects. On this assumption the uplift at one shoe may be expressed:

$$\frac{C_L}{2} = \frac{C_N}{4} + \frac{C_m}{2} + \frac{C_r}{2} \qquad (4)\ *$$

Curves showing the maximum uplift at one shoe computed by the above formula are plotted against skew angle in Figure 15 for a horizontal wind with and without the abutments and ground board. Both moments C_m and C_r were materially increased by the proximity of these obstructions and the uplift was substantially increased. However, greater values were obtained in upward angled winds without abutments or ground board, as shown also in Figure 15. Estimating the probable effect of the abutments at upward angles of 5 and 10 deg. presents much uncertainty in the absence of tests for these conditions. Certainly the percent excess of Curve A over Curve B is not significant since the uplift at $\psi = 0$ deg. and $\theta = 0$ deg. is small. It seems probable that even the numerical spread between these curves would not persist at upward angles of 5 or 10 deg. and there is reduced possibility of the larger upward angle of attack occurring over an entire span when the structure is near the water or ground surface. On the other hand, C_r which contributes most to the uplift at large skew angles, must be due more to the abutment than to the ground board and may be developed also by the piers of a high bridge.

Altogether, it seems advisable to allow for a substantial increase in the uplift over and above the values indicated by the tests made without abutments.

* Equation 4 is adequate for this comparison but a more accurate expression would substitute for C_m a moment coefficent referred to the longitudinal axis at the elevation of the shoes. See Equations 5 and 6.

Resultant Wind Forces

Figures 16 to 19 inclusive show, for different wind conditions, the resultant wind forces acting in a plane perpendicular to the longitudinal axis, plotted to scale on cross section sketches of the models. The force coefficients for the through-truss models are greater than for the others and have been plotted to a smaller scale as indicated.

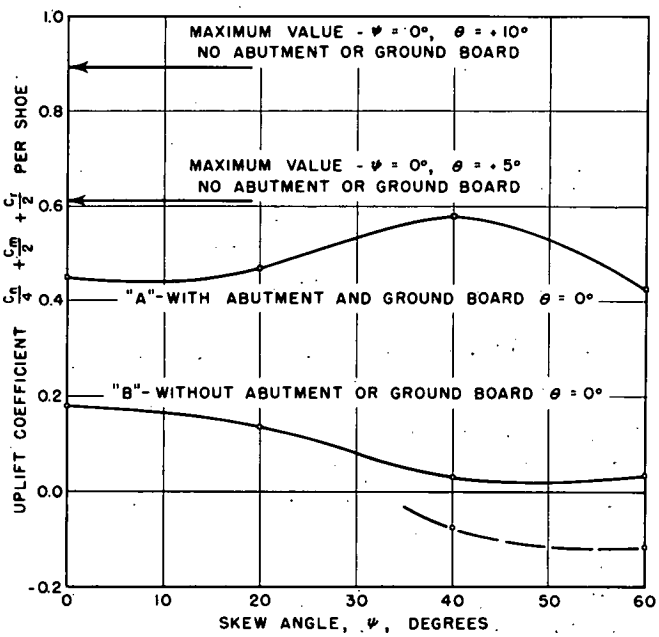


Figure 15. Graph of uplift as affected by abutments and ground board.

Several characteristics common to all of these diagrams may be noted: (1) even in a horizontal wind the resultant often has a considerable vertical component and winds of small vertical angle, 5 and 10 deg., impose very large vertical components, often greater than the horizontal component; (2) a small vertical inclination of the wind sometimes increases the horizontal component of the force; (3) the resultant often strikes well above the center of gravity of the area in elevation; and (4) a small change in inclination of the wind may cause a marked change in the magnitude, location and direction of the force.

Figure 19 for the six-girder model illustrates these points in a striking way. Resultant forces high above the model and, in some cases, nearly horizontal are inexplicable unless it is realized that there can be strong, nearly equal upward and downward forces producing a couple or overturning moment.

Engineers of the Bureau of Aeronautics who supervised these tests noted the wide variations for the girder models and made supplemental tests to verify or explain them. A small idealized model of about

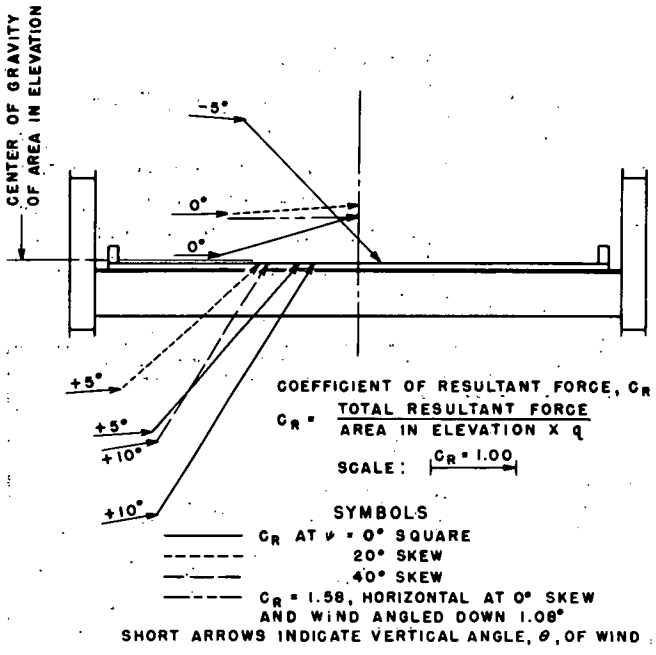


Figure 16. Sketch of resultant wind force on pony-truss model.

the windward two thirds of the section was made and tested in a smoke-facility tunnel in which the direction of flow of the various laminae of the windstream was shown by fine smoke streamers. The indications of these tests are shown in Figure 20. The leeward sidewalk was omitted, as it was thought that the windward features exerted the controlling influence on the wind flow. Also, the interior girders were omitted, since the wind is guided more by an envelope of the over-all shape than by features shielded by other parts. The validity of this latter statement is shown

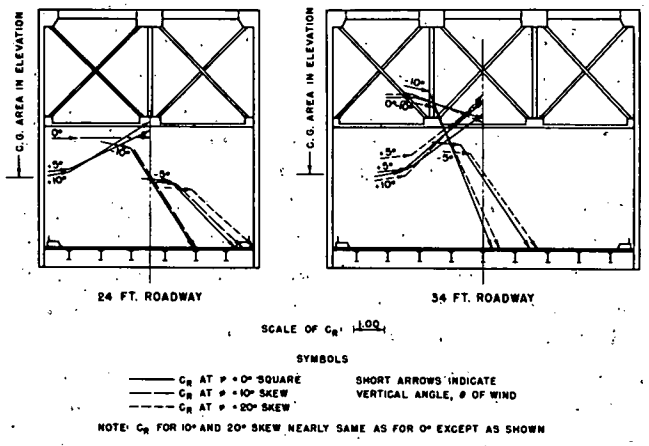


Figure 17. Sketch of resultant wind force on through-truss models.

by the small change resulting from the removal of the interior girders of the four-girder model in the static pressure tests (see Fig. 18).

The significance of the streamlines can be judged quite well by keeping two facts in mind: (1) the force required to deflect the wind stream must act, in general, toward the center of curvature, and the reaction against the model is, of course, the reverse; (2) the difference in pressure between two points in

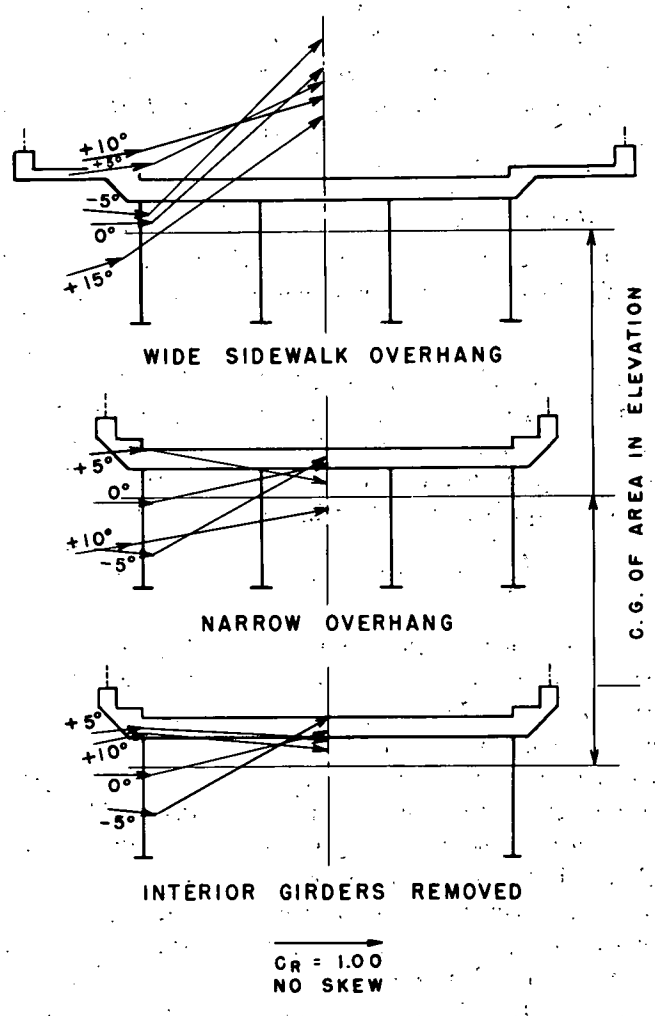


Figure 18. Sketch of resultant wind force on four-girder models.

the wind stream is inversely proportional to the squares of the velocities (Bernoulli's Law).

It is clear, then, that much of the drag, or horizontal force, arises from the pressure against the windward girder and that this pressure must also act upward against the bottom of the overhang producing a strong lifting force and overturning moment. The curvature of the stream lines crossing both the top and the bottom surfaces suggests a reduction below normal pressure on both areas, and detailed pressure tests showed these to be areas of negative pressure. The net vertical force, the difference between total forces on the

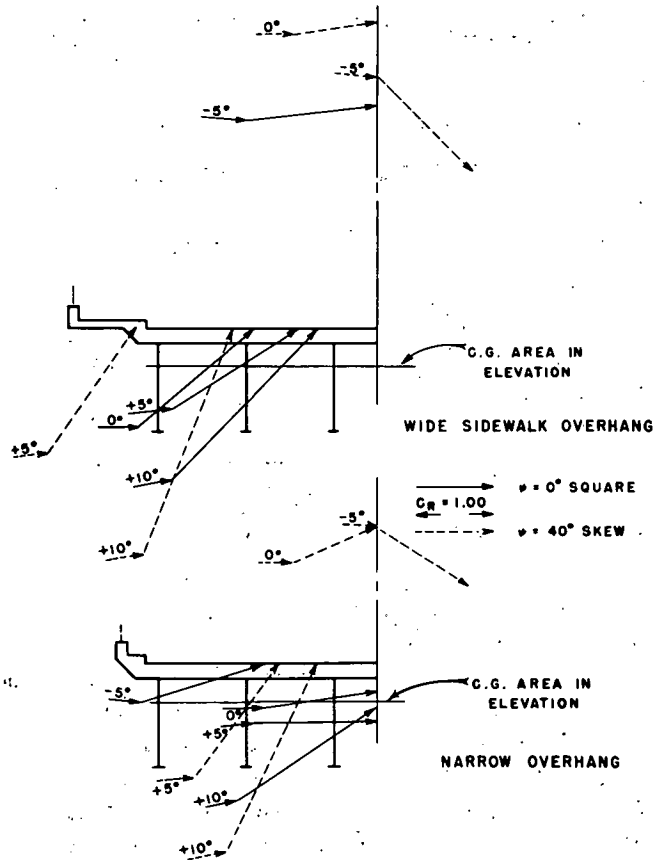


Figure 19. Sketch of resultant wind force on six-girder models.

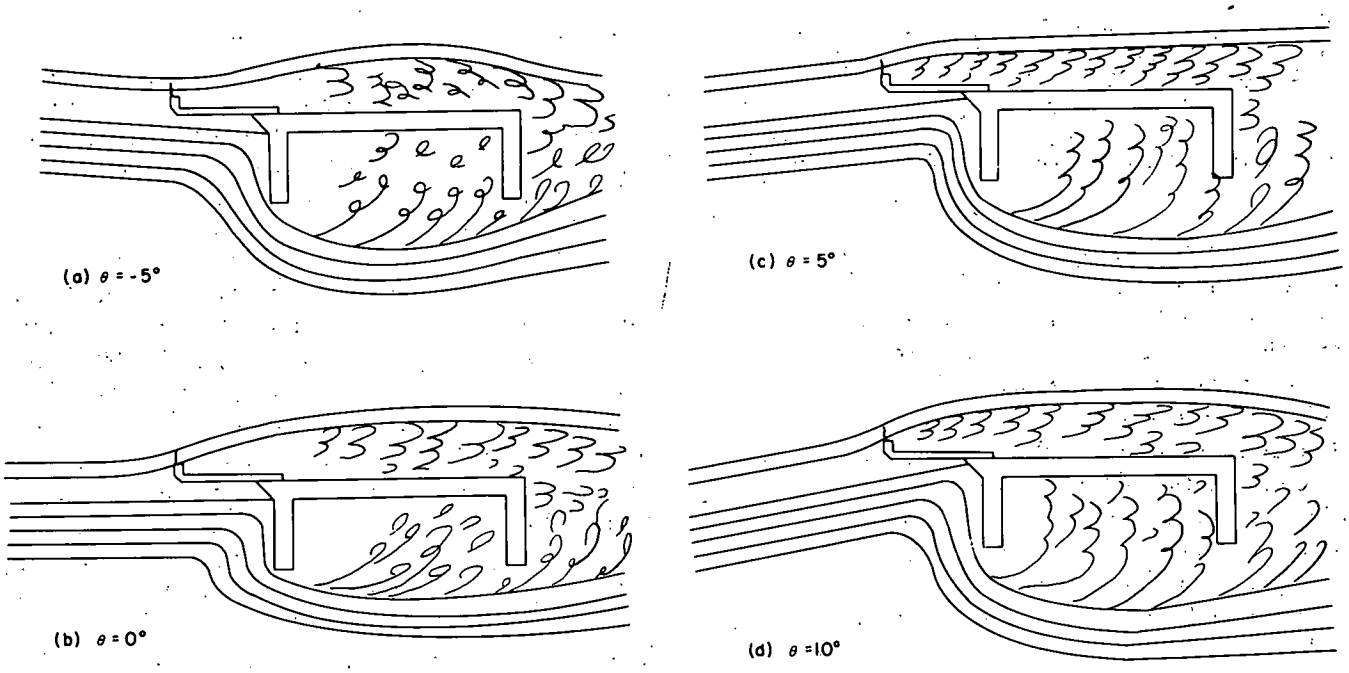


Figure 20. Sketch of stream lines around girder section.

two surfaces, depends upon the relative velocities crossing them and is very sensitive to changes in the vertical angle of attack. The position of the area of greatest negative pressure is likewise affected, influencing the strength of the couple which results from the action of the nonconcurrent vertical forces.

It has been found that the wind force on a single structural shape such as an angle may be doubled or completely reversed by a change of about 20 deg. in the direction of the wind. More complex angular sections show this tendency but to a smaller degree.*

The aeronautical engineers stated that with a wider section the flowlines would be expected to turn nearly parallel to the deck over the leeward portion restoring normal and more nearly balanced vertical forces. They saw evidence of this in the results of tests at the larger skew angles with their longer wind paths across the structure.

Maximum Transverse Force

Figure 21 shows curves for C_D, the coefficient of transverse force, plotted against skew angle, ψ, for all models. These are generally shown for a pitch angle (angle of attack), θ of +10 deg. The somewhat lower values for $\theta = 0$ deg. are shown for some models for comparison. These curves generally show the maximum force at $\psi = 0$ deg., but some individual tests show the greatest values at about 15 deg. of skew.

Most striking is the wide spread between the curves for the through truss models and the group of curves for the pony truss and deck girders. This spread was noted to a less extent in the results of static tests on modifications of models for the Tacoma Narrows

* "Drag Coefficients for Structures Studied in Wind Tunnel Model Tests" by W. Watters Pagon, Engineering News-Record, October 11, 1934.

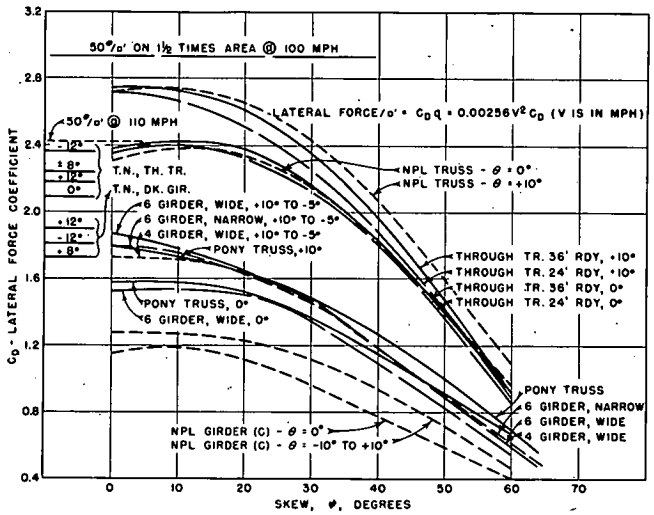


Figure 21. Graph of lateral-force coefficients.

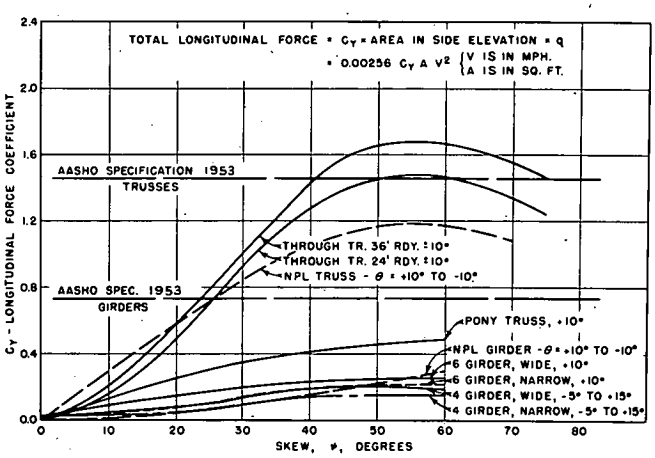


Figure 22. Graph of longitudinal-force coefficients.

Bridge.* Values for some of these at 0 deg. of skew are plotted in Figure 21. The truss members for that model were relatively heavier than those of the through truss models tested on this project.

Also shown are curves derived from static wind-tunnel tests made at the National Physical Laboratory (England) on a 1:30 scale model of the suspended structure of the proposed Severn River suspension bridge in Great Britain. The Severn Bridge section has deck trusses 27 ft. 6 in. deep and 78 ft. center to center, and carries two 24-ft. roadways, 9-ft. cycle tracks, and 6-ft. sidewalks. The cycle tracks and sidewalks are cantilevered outside of the trusses. An open reservation about 8 ft. wide separates the two roadways and open reservations about 10 ft. wide

* "Aerodynamic Stability of Suspension Bridges with Special Reference to the Tacoma Narrows Bridge Part I, Investigations Prior to October, 1941" by F. B. Farquharson, University of Washington, Bulletin No. 116, Part I, 1950, Tables XVI-XXI inclusive.

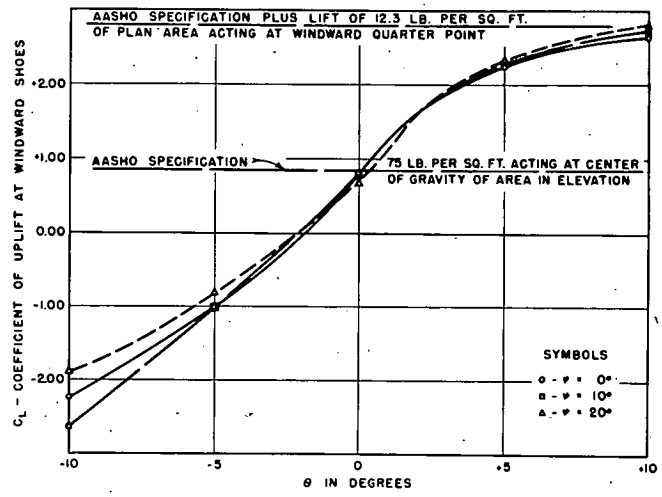


Figure 23. Graph of uplift coefficients for through truss, 36-ft. roadway.

separate the roadways from the cycle tracks. There are eight lines of open type railing.*

The National Physical Laboratory also determined the static loadings on a series of through girder arrangements. Curves are shown for the arrangement which most nearly corresponded with one of the models of the present series (two girders spaced at twice their depth).** The lower drag of this model as compared with the deck girders probably is due largely to the absence of the cantilevered overhanging deck.

Although supported by independent tests, a drag coefficient for through trusses 50 percent greater than that for the deck girders and pony truss may be reluctantly accepted, or doubted by some, unless some rational explanation can be found. There are several known factors which bear on this:

Reynolds Number (See also Appendix B)

The drag coefficient for a circular cylinder drops rapidly with increasing values of R up to 1,800 and again in the range between 200,000 and 300,000. Flat plates show some evidence of this drop in the lower range of Reynolds number but do not appear to show it in the upper range. If there were a scale effect it would tend to cause greater coefficients for the smaller truss members as compared with the deck girders. However, Table A shows that R was well above the critical value of 1,800 for even the smaller members

* "A Summarized Account of the Severn Bridge Aerodynamic Investigation' by R. A. Frazer and C. Scruton, Her Majesty's Stationery Office, York House Kingsway, London W.C.2 (price 25 shillings).

"An Experimental Investigation of the Aerodynamic Stability of Suspension Bridges with Special Reference to the Proposed Severn Bridge" by C. Scruton, Proc. Inst. Civil Engineers. Vol. I, Part 1, March 1952.

** "Tests on Plate-Girder Bridges in the Duplex Wind Tunnel" by D. H. Williams, H. L. Nixon and W. C. Skelton. Report NPL /Aero / 216, to be published. (Data used by permission.)

in these tests. There is a remote possibility of a scale effect between the models and their prototypes.

Aspect Ratio (Length over Width)

The drag on a square plate with flat side normal to the wind is about 55 percent of that on a plate having an infinite aspect ratio and for a ratio of 17.8 it is about 70 percent. (Short plates permit some of the air to escape around their ends thus reducing the total resistance).* For the models tested the aspect ratios of various elements are approximately as follows, including the effect of the dummy extensions and including the effective extension of the length of any member by gusset plates, etc.

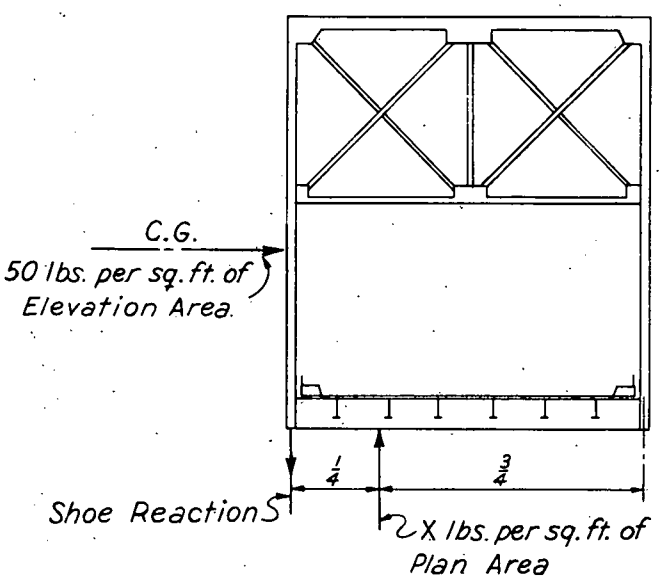


Figure 23(a). Sketch illustrating computation of equivalent vertical pressure.

Pony truss deck	6.3	ft./0.17	ft. =	37
bottom chord	6.3	ft./0.05	ft. =	126
diagonal	0.83	ft./0.035	ft. =	24
Girder models deck	9.6	ft./0.53	ft. =	18
Through truss deck	9.65	ft./0.115	ft. =	84
top chord (to hips)	8.4	ft./0.055	ft. =	153
bottom chord	9.65	ft./0.04	ft. =	240
web diaphragms	1.8	ft./0.03	ft. =	60

The data available are too incomplete to support calculations of the relative drag to be expected on the basis of aspect ratio. The comparisons suggest that the pony-truss model should have distinctly greater drag than the deck girders, which was not the case. They suggest that the drag of the through truss might be expected to exceed that of the pony truss by 10 or 15 percent, whereas the excess was 50 percent.

Aspect ratio probably accounts for a small part of the observed difference.

Relative Exposure

A sharp difference between the models lies in the proportion of the area in elevation which is exposed again to the windstream as in the leeward truss and handrail. The deck girders have very little of such double exposure and the pony trusses only a little more. Tests show that a leeward duplication of a body has a drag of its own although in certain positions its presence reduces the drag measured on the windward one.* The slight increase in total drag when a model was rotated to a small skew angle showed the effect of a reduction in the shielding effect or, more precisely, the tendency of the windward and leeward members to discharge separate vortex trails instead of being enveloped in the same trail.

This effect of duplicate exposure is also shown on the Tacoma Narrows models which were tested with solid decks and with the decks 72 and 100 percent open. Opening the decks increased the drag 10 to 25 percent for the truss models, 0 to 10 percent for the half through girders, and had little effect on the deck girders. A part of this increase might be attributed to the turbulence caused by the wind blowing through the mesh floor, except for the fact that the drag usually was greater with the fully open deck than with the mesh (see University of Washington Bulletin 116-1, Figures 62 to 66 inclusive). The small effect of opening the floor of the girder spans was due to the large wake of the leading girder enveloping the remainder of the structure even at pitch angles of ± 8 deg. so that there was less tendency for the wind to flow through the floor and thus strike the leeward girders than in the case of the truss models.

Turbulence

Increased drag due to the conditions described in the preceding paragraph may be attributed to the increased turbulence of the wind passing a network of obstacles. It is a principle of aerodynamics that the drag on an object increases with the turbulence caused by that object, that is, with the amount of energy dissipation in the windstream.

It appears that the greater drag of the through truss models was due principally to the duplicate exposure of many members and to the greater turbulence, with perhaps a small increase due to the greater aspect ratios.

* For a reasonable assumed representation of the effect of aspect ratio on C_D. See "Aerodynamic Stability of Suspension Bridges with Special Reference to the Tacoma Narrows Bridge Part III", University of Washington, Bulletin No. 116-3, Figure 141 and Table XXII.

* Drag Coefficients for Structures Studied in Wind Tunnel Model Tests" by W. Watters Pagon, Engineering News-Record, October 11, 1934.

The objection may be offered that if the greater drag were contributed by the upper members of the truss models the resultant pressure should have been raised above the center of gravity to a greater degree than on the girder models which was not the case. The answer is that the couple due to nonconcurrent vertical forces was a major factor in determining the position of the resultant force as previously discussed.

Maximum Longitudinal Force

Figure 22 shows the longitudinal force coefficient, C_Y plotted against the skew angle, ψ. Some curves are composites to represent the highest values at any angle of attack from -5 to $+15$ deg. Data plotted from the National Physical Laboratory tests also are shown. Here the wide spread between the through trusses and all the others is even more evident than in the case of the transverse or lateral forces. The force coefficients corresponding to the 1953 AASHO Specification at 100 mph. are plotted on this figure, that is 50 percent and 25 percent of the specified lateral loading, applying respectively to trusses and girders.* It is seen that the upper value is exceeded only by the test results for the wide through truss span which correspond to about 57 percent of the specified lateral loading. Not only the girders but also the pony truss showed longitudinal forces well below 25 percent of the specified lateral force when interpreted for a 100-mph. wind.

Maximum Overturning Effect

The analysis of the lift component and the pitching or overturning moment is much less straightforward than that for transverse and lateral force. The test data show more variation and less consistency with respect to the pitch and skew angles. Small changes in either angle may reverse the direction of the vertical force. The truss models show some consistency, but changes on the girder models seem capricious. This, of course, is due to the sensitive balance between forces acting on the upper and lower surfaces, as previously discussed.

Not only are the actual forces difficult to reduce to a system, but their practical effect on a bridge as viewed from the standpoint of design loading depends much upon the type of structure. For a high, narrow truss span, a critical danger of overturning may be present due to a high line of action of the resultant horizontal pressure or to uplift, often eccentric, acting on the deck, or to both causes. In this case, the critical feature is the uplift and moment acting at the elevation of the shoes, which may develop a net uplift at the windward shoes. Low superstructures, in no danger of overturning or uplift on their own immediate supports, may develop large overturning moments in high, slender bents supporting them. In this case, the most-severe condition results from the moment of the horizontal forces about the base, combined with any upward or downward wind force acting on the deck.

Several effects of uplift and pitching moment were studied, but the most-consistent results were obtained by computing the uplift at the windward shoes. This was arbitrarily taken at the elevation of the center of the bottom chord or the back of the lower flange. The results are plotted against pitch angle θ in Figures 23 to 29 inclusive. The coefficient of uplift, C_L, was computed for the entire model or span, consistent with all the coefficients shown on Figure 14. The actual uplift at one shoe (assuming two shoes at each end) is $C_L\, q\, A/2$, A being the total area of the span in elevation.

$$C_L = C_N/2 + C_M c/c' \qquad (5)$$

in which C_M is the coefficient for moment about a longitudinal axis of the bridge at the elevation of the bottom chord or flange, c, is the characteristic dimension for moment (see Fig. 14 and Table 1), and c' is the transverse distance between shoes, that is, the distance center to center of trusses or of outside girders. The moment about the reference point used in reporting the test results is $C_m\, cqA$ and the moment referred to the elevation of the bottom chord or flange is $(C_m\, c + C_D\, d')\, qA$ in which d' is the vertical distance from the reference point to the bottom chord. The coefficient for this moment then is:

$$C_M = C_m + C_D \frac{d'}{c} \qquad (6)$$

C_L was computed by Equations 5 and 6 for all models for all angles of pitch and for those skew angles showing the larger positive (uplifting) forces.

For the through truss with 36-ft. roadway (Fig. 23) the maximum value of C_L is about 2.8. For a velocity of 100 mph. this indicates an uplift at one windward shoe equivalent to ($2.8 \times 25.58 = 71.6$) lb. per sq. ft. times half of the area in elevation. For this model the area in plan was 5.36 times the area in elevation. Hence the maximum uplift coefficient referred to plan area (here designated C'_L) is $2.8/5.36 = 0.522$ and the uplift force on one shoe is equal to $0.522 \times 25.58 = 13.4$ lb. per sq. ft. times half of the plan area. Since a fourth of the dead load falls on one shoe, a dead load of 26.8 lb. per sq. ft. of plan area would suffice to prevent actual uplift.

The uplift on the windward shoes calculated from the specifications for a transverse wind pressure of 50 lb. per sq. ft. on 1½ times the area in elevation acting at the center of gravity of the area was com-

* See Appendix A which gives Article 3. 2. 14 of these specifications.

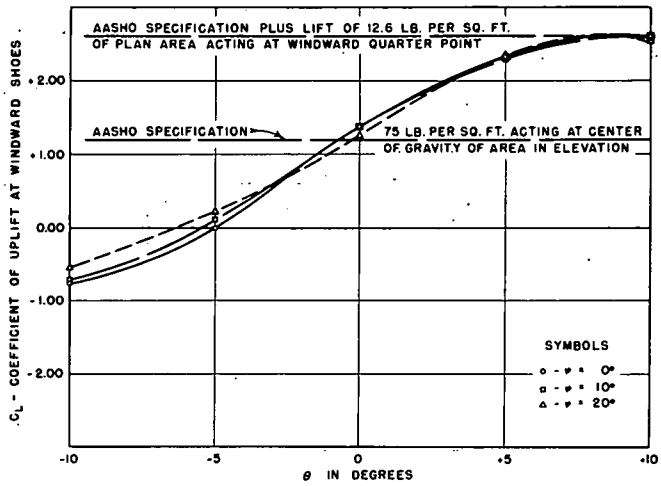


Figure 24. Graph of uplift coefficients for through truss, 24-ft. roadway.

puted for each model. Considered for a 100-mph. wind, this unit pressure corresponds to a coefficient $C_D = \frac{50 \times 1.5}{25.58} = 2.93$ as previously computed. The center of gravity of the area of the through truss models was 0.509 ft. above the center of the bottom chord (Table 1) and the coefficient of uplift (both windward shoes) due to the specified load is therefore:

$$C_L = \frac{2.93 \times 0.509'}{1.728'} = 0.863$$

1.728 ft. being the distance center to center of trusses. C_L is plotted on Figure 23 and corresponding values for the other models are plotted on the succeeding figures. It is evident that the actual uplift as plotted on these figures for each model far exceeds that computed from the specifications.

It has been indicated earlier in this report that moments and the position of the resultant force are governed largely by the vertical forces. It was thought that the simplest equivalent representation of their action would be a uniform pressure over the area in plan acting at some assumed eccentricity. For horizontal flat plates and a number of other sections in a horizontal wind, the resultant vertical force acts at about the windward quarter point and a windward eccentricity of a fourth of the deck width was therefore assumed for these studies. The computations for the through truss with 36-ft. roadway are given to illustrate the analysis.

Maximum C_L to be accounted for	2.80
C_L due to AASHO specified wind loading	0.86
C_L to be accounted for by vertical loading	1.94

Width, center to center of trusses, 1.728 ft.

If the unknown vertical wind force at the windward quarter point is $C_v\ q\ A$ its reaction at the windward shoes is $C_v\ q\ A \times 3/4$. Equating this to the vertical reaction to be accounted for:

$$\frac{3}{4} C_v\ q\ A = 1.94\ q\ A$$

$$\text{or } C_v = \frac{4}{3} \times 1.94 = 2.59$$

This agrees quite well with $C_N = +2.65$ as measured at $\psi = 20$ deg. and $\theta = +10$ deg., which was the condition yielding the highest value of C_L, 2.80, in Figure 23. Furthermore, it may be observed in Figure 17 that the resultant wind force for this model at these angles intersects a horizontal line through the center of gravity of the elevation area nearly midway between the centerline and the windward truss.

C_v referred to the plan area may be computed as $C'_v = \frac{2.59}{5.36} = 0.482$ which for a 100-mph. wind is $0.482 \times 25.58 = 12.3$ lb. per sq. ft.

It is interesting to note (Fig. 24) that the maximum uplift for the through truss with 24-ft. roadway can be accounted for by the AASHO specified horizontal wind loading plus practically the same vertical pressure (12.6 lb. per sq. ft.) acting at the windward quarter point of the area in plan. The computed value of C_v was 1.87 which compares quite well with the measured values of C_N of +1.57, +1.65 and +1.75 at $\theta = +10$ deg. and $\psi = 0$, 10 and 20 deg. respectively.

In a similar study of the uplift on the pony-truss model, it was recognized that the actual horizontal transverse force was about 70 percent of that measured on the through trusses and it was anticipated that a more realistic conception of the relative wind forces would be obtained by combining the unknown equivalent vertical force with only 70 percent of the

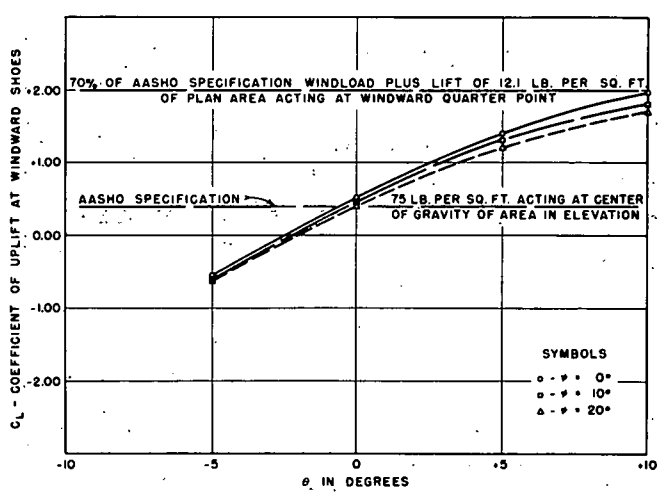


Figure 25. Graph of uplift coefficients for pony truss, 22-ft. roadway.

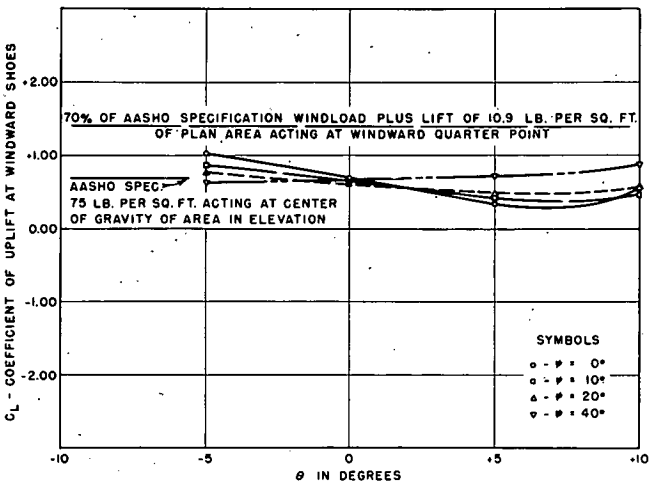


Figure 26. Graph of uplift coefficients for four girders, narrow overhang.

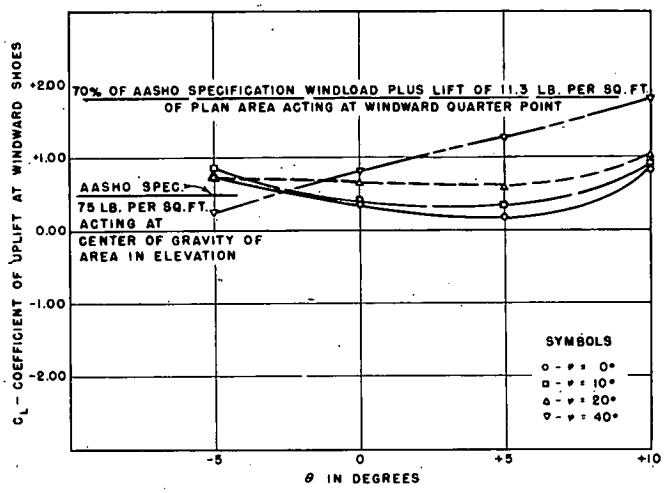


Figure 28. Graph of uplift coefficients for six girders, narrow overhang.

AASHO wind loading acting horizontally at the center of gravity. This resulted in a vertical loading of 12.1 lb. per sq. ft. of plan area and a coefficient, C_v of +2.29 (C_N for $\theta = 10$ deg. and $\psi = 0$ deg. was 2.96).

The approximate maximum values of C_L for the girder models at pitch angles from −10 to +10 deg. were only roughly obtained by extrapolation from the plotted curves. These curves (Figs. 26-29 inclusive) show net uplift at the windward shoes even for downward winds of 10 deg. and the smallest uplifts generally occur at an upward angles of about 5 deg. It was not anticipated that the data for the girder models could be approximated by any such simple equivalent vertical loading as was possible for the truss model, but the same method and assumed line of action of the vertical force were used. The resulting equivalent upward pressure are 10.9, 16.3, 11.3, and 13.8 lb. per sq. ft., which are surprisingly close to the values of 12.1, 12.3 and 12.6 lb. per sq. ft. for the truss models.

From the earlier discussion of Figure 15, it may be assumed that the presence of an abutment or pier might increase C_L for one shoe by as much as 0.2, or 0.4 for two shoes. Calculations indicate that this would increase the above unit lifting pressures by 2.5 to 3.5 lb. per sq. ft. Thus, a value of 15 lb. per sq. ft. would suffice for all but one girder model (four girders with wide overhang) and 20 lb. per sq. ft. would be a conservative outside value for all models.

Chester Bridge

The Mississippi River bridge at Chester, Illinois, which was overturned by a wind storm on July 29, 1944, affords an interesting comparison with these model indications. It consisted of two continuous 670-ft. spans with 24-ft. roadway and narrow sidewalks with trusses 28.5 ft., center to center, having

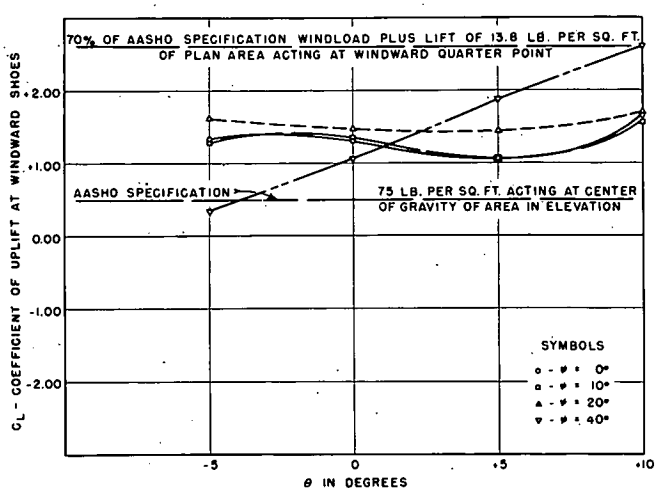


Figure 27. Graph of uplift coefficients for four girders, wide overhang.

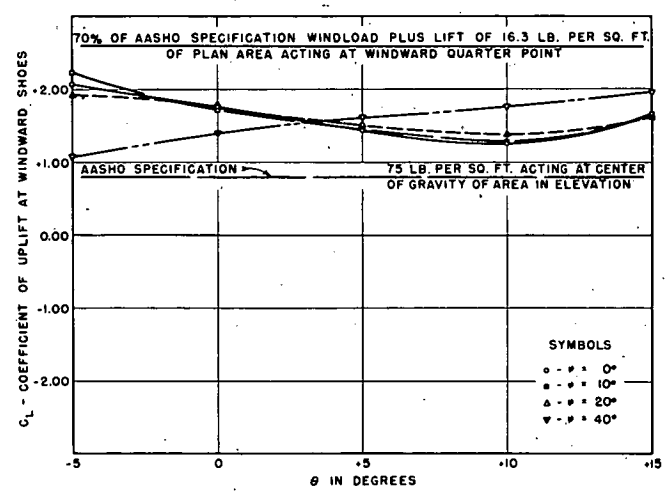


Figure 29. Graph of uplift coefficients for six girders, wide overhang.

depths of 60 ft. at the hips, 100 ft. over the middle pier, and 70 ft. throughout most of the span. It had approximately the same width but twice the height represented by the 24-ft. roadway truss model. John I. Parcel, in *Engineering News-Record* for August 3, 1944, indicated that a horizontal wind pressure of 110 lb. per sq. ft. on 1½ times the area in elevation would have been necessary to overturn the span and stated the dead load as 4,640 lb. per linear ft. of bridge. Using these figures, the area in elevation per foot and its lever arm about the shoes were approximated, and it has been calculated that, with the AASHO wind loading of 50 lb. per sq. ft. on 1½ times the area in elevation, it would be necessary to add an uplift of about 60 lb. per sq. ft. of plan area acting at the windward quarter point in order to overturn the span. This greatly exceeds the figure of 20 lb. per sq. ft. conservatively derived above from these tests for a 100-mph. wind angled upward 10 deg. and supports the suggestion that tornadic action must have been a factor in the destruction of the Chester Bridge. The reports state that the Missouri end (west) of the bridge was lifted off its supports without injury to the fingered expansion joints and complete overturning followed. The actual wind forces acting on a small part of the bridge must have reached several times the average equivalent figures used above.

Another illustration of the effect of a tornado occurred on June 16, 1928, during the construction of a highway bridge over the Red River east of Altus, Oklahoma. Of twenty-three 80-ft. spans, the eight westerly spans, which were bolted up or partly riveted but had no anchor bolts nor decks, were thrown off their piers. Some spans which were anchored but had no floors were shifted slightly. Possibly they were out of the path of maximum effect. There was evidence of strong wind action on one of the decked spans which did not move; a pier form plate was sheared when blown against the handrailing.

It is evident that lifting forces in a tornado far exceed the lift developed in these tests at 100 mph. and an upward angle of 10 deg.

Downward Reaction Due to Overturning

The compressive reactions to the leeward shoes of the truss models can be roughly approximated by adding to the AASHO specified wind loading a wind pressure of about 12 lb. per sq. ft. of plan area acting downward at the leeward quarter point. The actual vertical force was about twice this but acted near the centerline.

For the girder spans the dominant vertical action was upward and the compressive shoe reactions were relatively small.

Summary and Conclusions

1. The results of these model tests agree very well with the provisions of the 1953 AASHO Bridge Specifications as to maximum transverse and longitudinal wind forces for a velocity of 100 mph. with the wind approaching at not more than 10 deg. from the horizontal. The allowance for additional trusses, possibly over 50 percent of the added exposed area, seems advisable, although such allowance is not needed for deck girders.

2. The indicated tendency to overturning and uplift at the windward shoes within the same range of wind conditions can be approximated conservatively by the specified lateral wind force acting at the center of gravity of the area in elevation supplemented by a lifting force of 20 lb. per sq. ft. of plan area assumed to act midway between the centerline and the windward truss (or deck edge in the case of cantilevered deck construction).

3. The maximum downward reaction to the leeward shoes can be conservatively approximated by the overturning effect of the specified lateral force, plus a downward force of 20 lb. per sq. ft. of plan area acting at the leeward quarter point of the exposed area.

4. These tests and the others cited showed the maximum wind loads on girder and pony-truss spans to be less than 70 percent of the maximum on through truss spans. However, it would seem inadvisable to consider reducing the specified loading for girders and pony trusses until these indications are confirmed by more comprehensive tests, including tests on through girders, deck trusses, open-grid decks, etc.

5. There are probably locations where the design should provide for wind velocities greater than 100 mph. If security against tornadoes is to be provided, considerably greater wind loads, particularly uplift, should be specified.

Acknowledgement

The wind-tunnel tests were planned and the pony-truss model was tested under the supervision of Commander C. W. Stirling, head of the Aeronautics Laboratory, and the tests on the other models were completed under Captain W. C. Fortune. J. N. Fresh, head of the Subsonic Division, and A. W. Anderson and E. T. Burgan, project engineers, assisted in planning the program to obtain the desired information and directed the testing, computing, and preparation of data. F. N. Wray, of the Highway Research Board, facilitated the work by his careful attention to correlating the efforts of the various agencies and individuals engaged. Raymond Archibald, chairman

of the Bridge Committee of the American Association of State Highway Officials, contributed much to shaping up the project and getting it underway. The generous cooperation of the Engineer Board of the Department of the Army in making its excellent pony-truss model available for the first tests is appreciated. The cordial relations of all those participating in the work have lightened the work and brought quick solution of the various problems encountered.

This report is based upon information presented in a report submitted by the Aerodynamics Laboratory to the Highway Research Board which is concurrently being published under the designation "Wind Tunnel Investigation of the Force and Moment Coefficients on Several Types of Bridge Models" by Arnold W. Anderson and Elmer T. Burgan, David Taylor Model Basin Report No. 858, Aero No. 842, Bureau of Aeronautics, Department of the Navy.

APPENDIX A

Article 3.2.14 of the 1953 AASHO Specifications.

3.2.14.—Wind Loads.

The following lateral forces shall be applied to all structures. They shall be considered to act horizontally in any direction (see Art. 3.4.1 for design stresses used under the various combinations of loads and forces):

1. A transverse wind force on the structure applied as a moving horizontal load of 50 pounds per square foot on 1½ times the area of the structure as seen in elevation, including the floor system and railings and on one-half of the area of all trusses or through girders in excess of two in the span. In no case shall the total of this force be less than 300 pounds per linear foot of bridge.

The intensity of this force may be reduced 70 per cent (to 15 pounds per square foot) when included in the combination of forces designated as Group III of Art. 3.4.1.

2. A lateral force of 200 pounds per linear foot due to the action of wind against a moving live load, and applied 6 feet above the roadway. (This force is designated "W. L." in Art. 3.4.1.) Where a reinforced concrete floor slab or a steel grid floor is effectively attached to its supports it may be assumed to transmit this lateral force to the ends of those supports.

3. The total assumed lateral force shall not be less than 300 pounds per linear foot in the plane of the loaded chord and 150 pounds per linear foot in the plane of the unloaded chord on truss spans, and not less than 300 pounds per linear foot on girder spans.

4. In calculating the uplift in posts and anchorage of viaduct towers due to the foregoing lateral forces, the decks shall be considered as loaded on the leeward traffic lane with a uniform vertical load of 400 pounds per linear foot of lane, against which load the lateral force of paragraph (2) above shall be applied. These loads shall be applied only in case they increase the net uplift.

5. A longitudinal wind force shall be assumed, equal to the following percentage of the total lateral or transverse wind forces on the structure:

For through or deck trusses.....50% of lateral
For through or deck girders or beams.....25% of lateral

6. The wind forces specified above may be reduced if there are permanent features of terrain which clearly act to reduce the possible wind pressures on exposed surfaces.

APPENDIX B

Reynolds Number

The Reynolds number, R, is the ratio of the inertial force to the viscous force which a fluid stream exerts on a body.

Inertial force is the product of dynamic pressure and area and may be expressed: $\frac{\rho V^2 L^2}{2}$, ρ being the mass density, V, the velocity and L a linear dimension of the body.

Viscous force is the viscous drag or friction between the fluid and the body. It is proportional to the coefficient of viscosity, μ, and to velocity and area, and inversely proportional to the distance between the body and an element of the fluid and may be expressed: $\frac{V L^2 \mu}{L}$. The ratio of these is: $R = \frac{\rho V L}{\mu} = \frac{V L}{\nu}$ as $\nu = \frac{\mu}{\rho}$ is known as the kinematic viscosity. For air at 15 C. and sea level, $\rho = 0.002378$ slugs per cu. ft. (lb. sec.^{2}/ft.^{4}), $\mu = 0.373 \times 10^{-6}$ lb. sec./ft.^{2} and $R = 6380\ VL$, in which V is in ft. per sec. and L is in ft. L is generally taken as the dimension of the body parallel to the direction of the wind although this is impracticable for a flat plate normal to the wind in which case its width is used. In comparing a model and its prototype corresponding dimensions are used and the Reynolds number for the model is that for the prototype reduced by the scale factors of the linear dimensions and of velocity. The Reynolds numbers of two elements of the same structure in a wind stream are proportional, say, to their widths parallel to the direction of the wind.

In general, a large Reynolds number simply means that the viscous force is neglegible compared to the inertial force and that force is truly proportional to velocity squared, that is, $F = \frac{C \rho V^2 A}{2}$, A being the area of the body and C being constant for variations in velocity. C may be computed from measured force and velocity as:

$$C = \frac{2 F}{A \rho V^2}. \qquad \text{(A)}$$

If R is small the viscous force may not be neglegible, and a significant part of the force may be proportional to the ve

locity. In this case if measured data for different velocities are substituted in Equation A different values for C will be obtained revealing the "scale effect" or Reynolds number effect between the two tests. It is customarily compared with respect to the drag coefficient, $C_D = \frac{\text{Drag force} \times 2}{\rho V^2 A}$. The change in C_D is not gradual but occurs rather abruptly between certain values of R. For a circular cylinder it falls quite rapidly with increasing R up to about $R = 1{,}800$, then changes little up to $R = 200{,}000$ to 300,000 in which interval there is another abrupt drop. Above 300,000 it seems to be fairly constant.

C_D is greater for a flat plate normal to the wind than for a circular cylinder but its possible variation with R is not so well known. There is some indication that it also has a critical value at about $R = 1{,}800$.

If tests of a model at two different velocities show the same value for C_D there is no scale effect on that section between the corresponding values of R.

Table A shows the Reynolds numbers for the models discussed in this report and their prototypes computed for various members and for the three test velocities. It should be noted that Reynolds numbers for both models and prototypes are well above the critical value of 1,800 and in the range where the drag coefficient is practically constant.

TABLE A

REYNOLDS NUMBER

$R = 6380\ VL$

Model and Member	Reynolds Number, R in Thousands				
	L Ft.	Model at V = 25 mph. 37 fps.	Model at V = 60 mph. 88 fps.	Model at V = 100 mph. 147 fps.	Prototype at 100 mph. 147 fps.
Pony truss, 1:16 scale					
deck depth	0.169	38	95	159	2,540
chord width	.078	18	44	73	1,168
web width	.073	17	41	68	1,088
deck width	1.460	345	820	1,370	21,900
Through truss, 1:24 scale					
deck depth	0.115	27	65	108	2,590
top chord width	.075	18	42	70	1,680
main web member width	.035	8	20	33	792
lateral angle	0.014	3	8	13	312
deck width 24-ft. roadway	1.156	273	649	1,084	26,000
36-ft. roadway	1.656	389	930	1,554	37,300
Girder span, 1:24 scale					
overall depth except rail	0.53	125	298	497	11,950
overall width, narrow roadway	1.42	335	797	1,332	32,000
overall width, wide roadway	2.67	630	1,500	2,502	60,000
			63 mph. 92 fps.	77 mph. 103 fps.	
Tacoma Narrows, 1:20 scale, U. of W. Bul. 116-1 Tables XVI-XXI					
Through Girder Depth	0.401		235	263	7,520
Width	2.04		1,200	1,340	38,100
Through Truss Deck Depth	0.112		66	74	2,100
Deck Width	2.00		1,173	1,314	37,500
Chord Depth	0.110		65	72	2,060

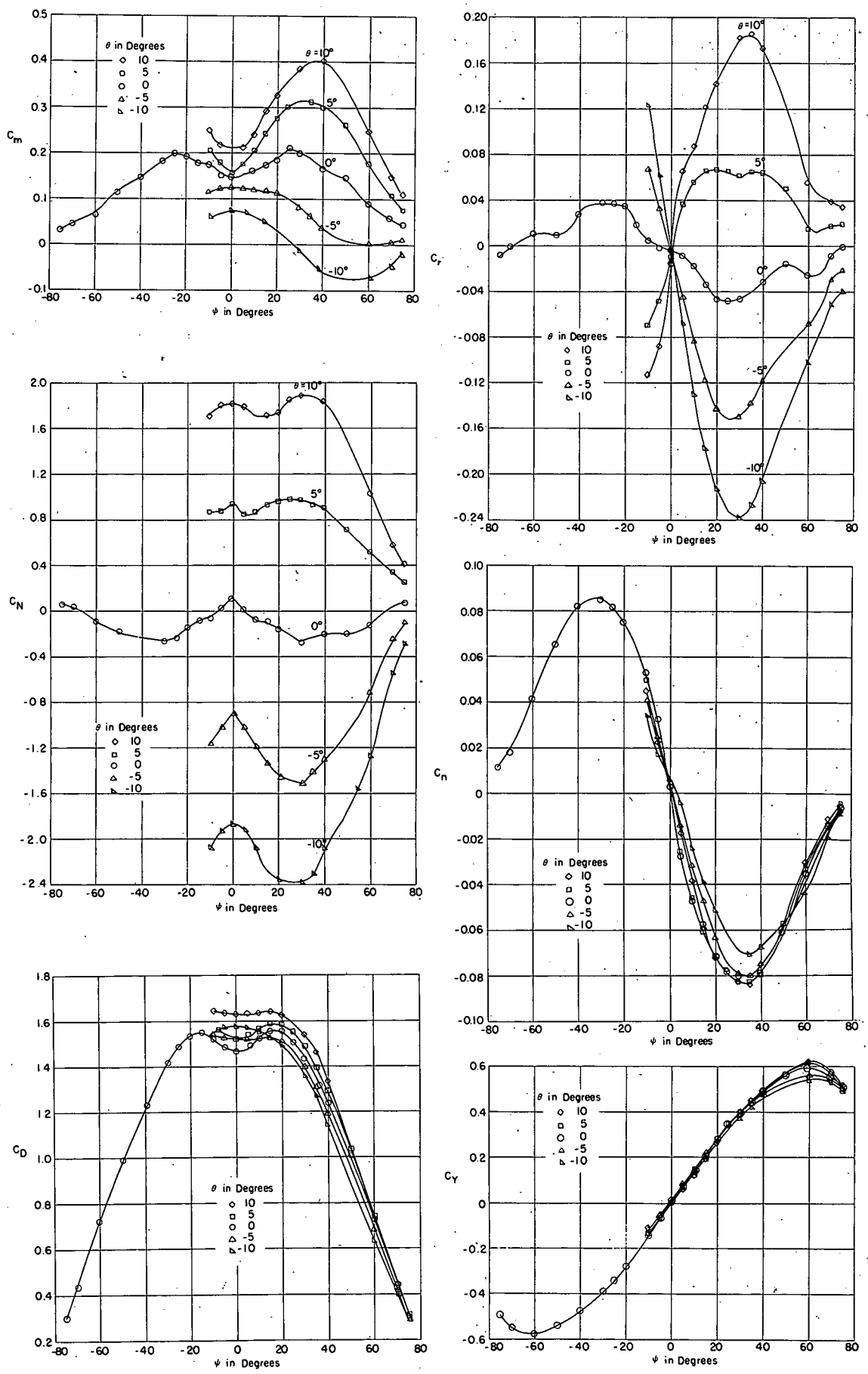


Figure A. Aerodynamic characteristics in yaw of a pony-truss model without roadbed extension.

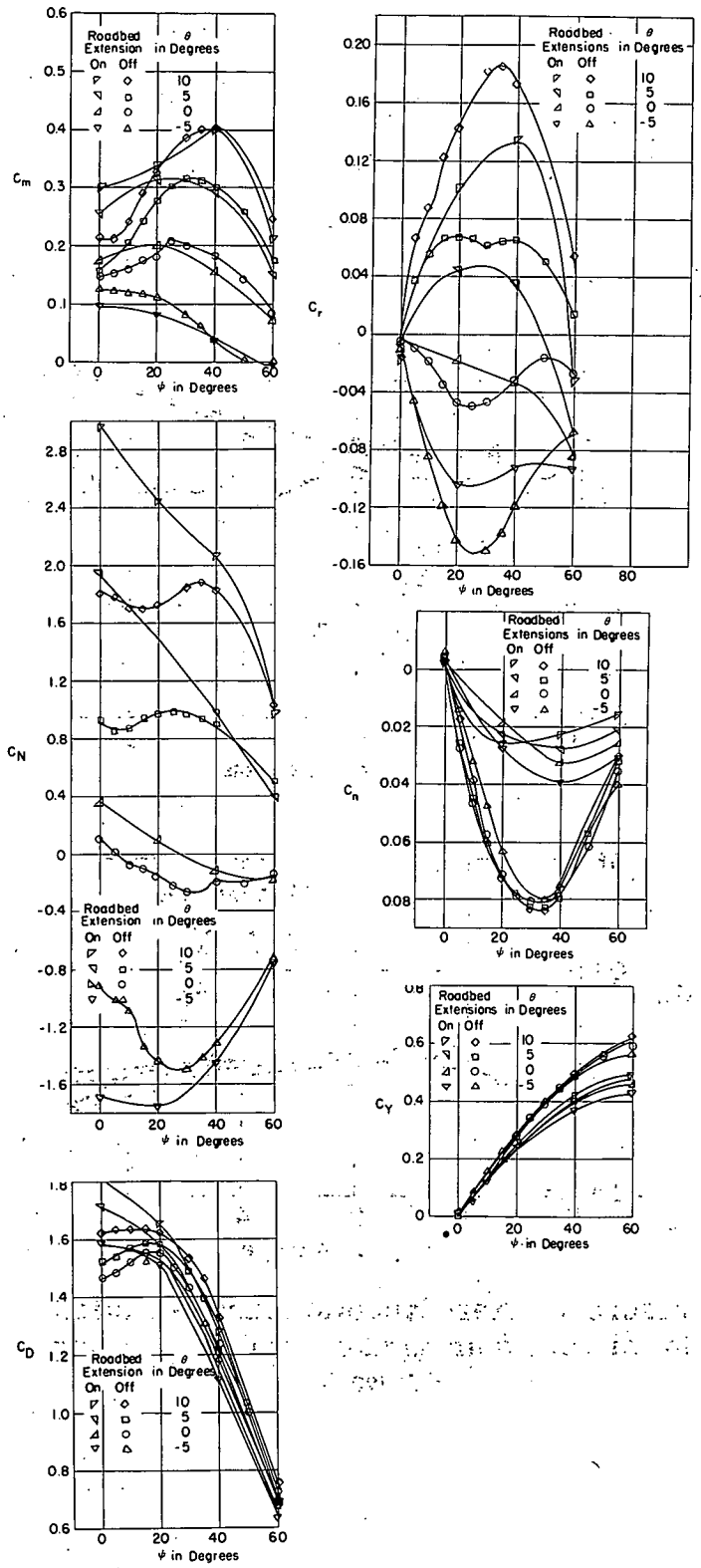


Figure B. Aerodynamic characteristics in yaw of a pony-truss model with and without roadbed extensions.

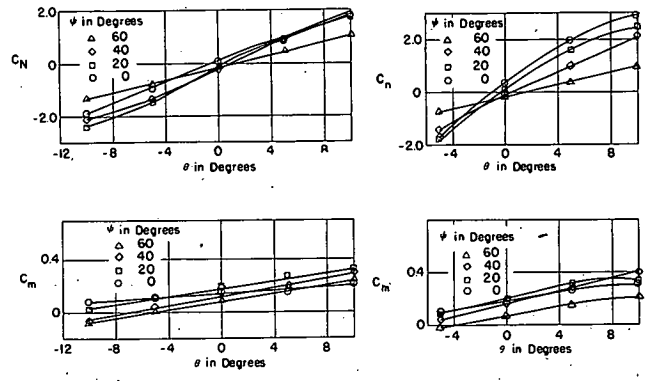


Figure C. Aerodynamic characteristics in pitch, pony-truss model with and without roadbed extensions.

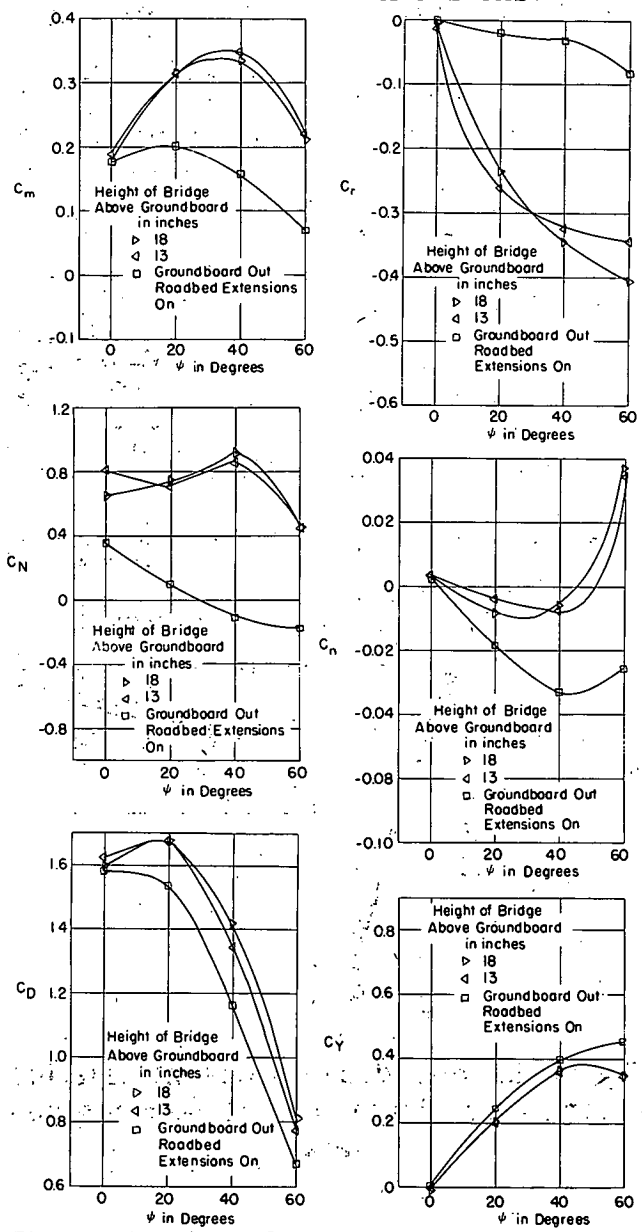


Figure D. Aerodynamic characteristics in yaw of a pony-truss model with ground board.

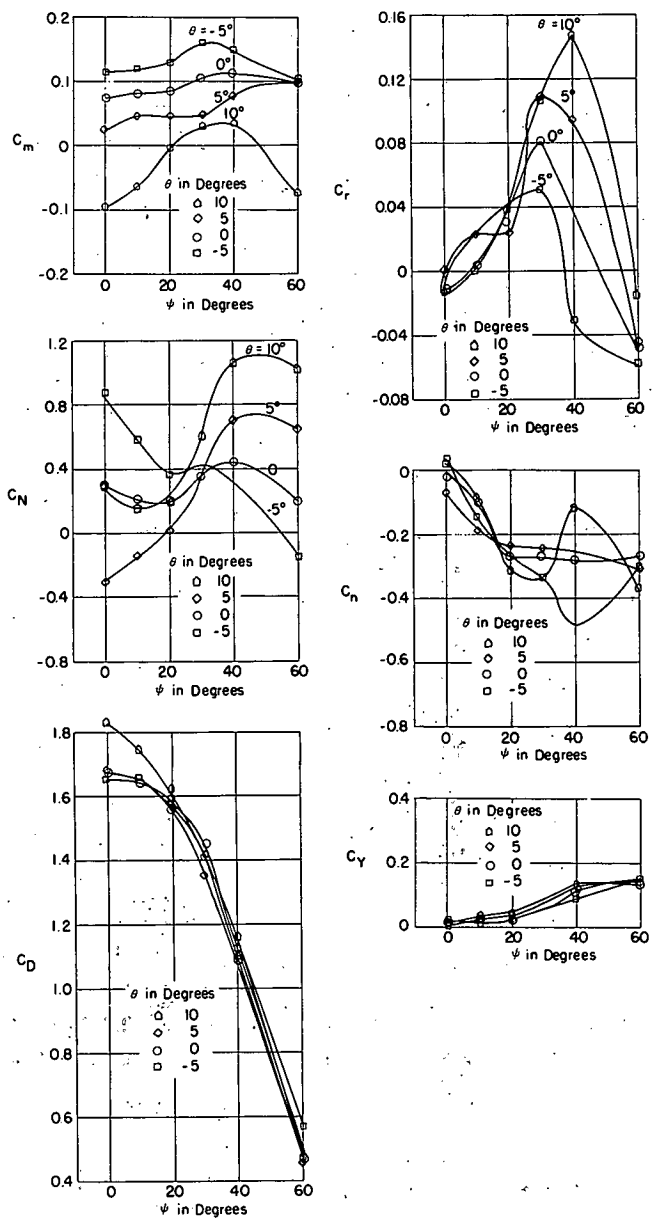


Figure E. Aerodynamic characteristics in yaw, four-girder model, narrow sidewalk.

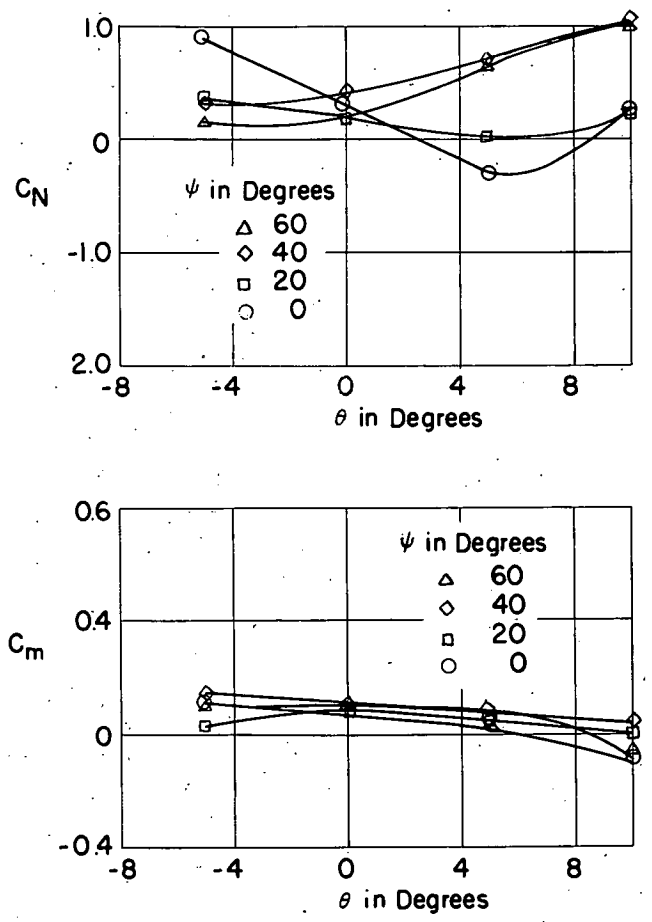


Figure F. Aerodynamic characteristics in pitch, four-girder model, narrow sidewalk.

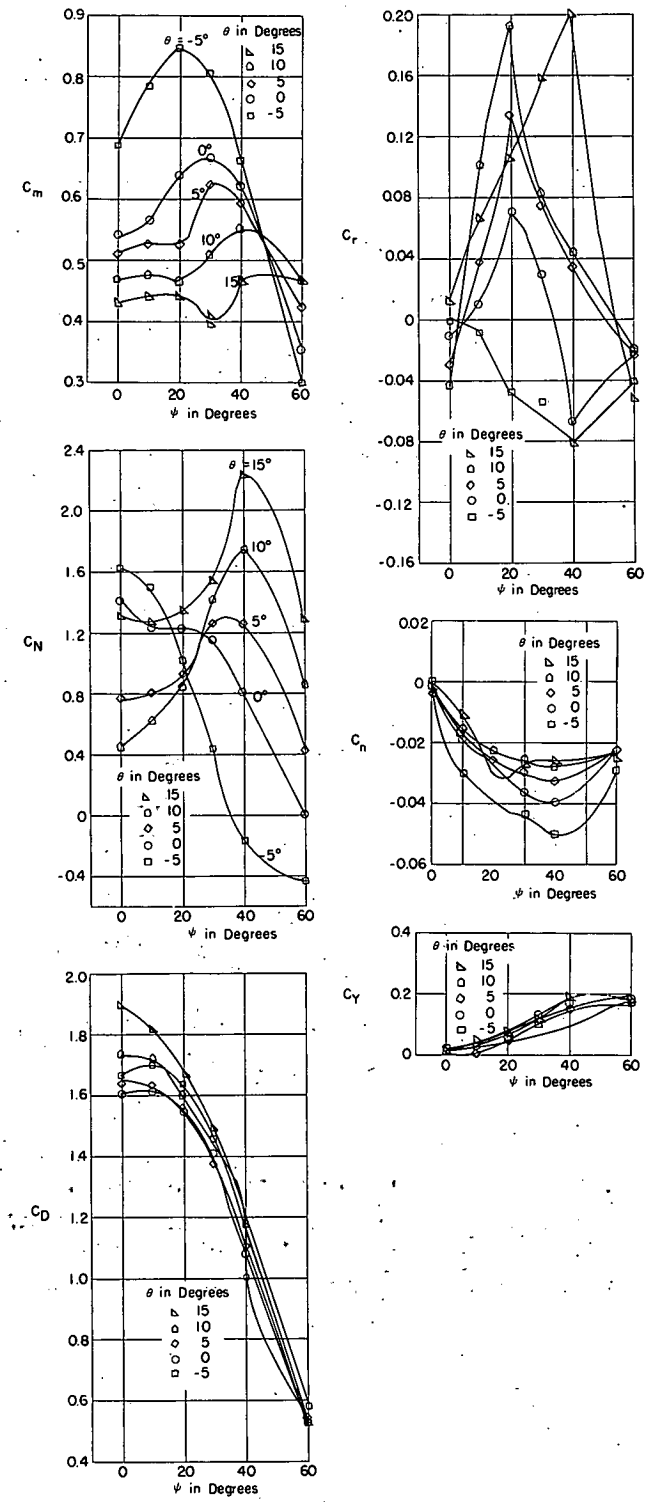


Figure G. Aerodynamic characteristics in yaw, four-girder model, wide sidewalk.

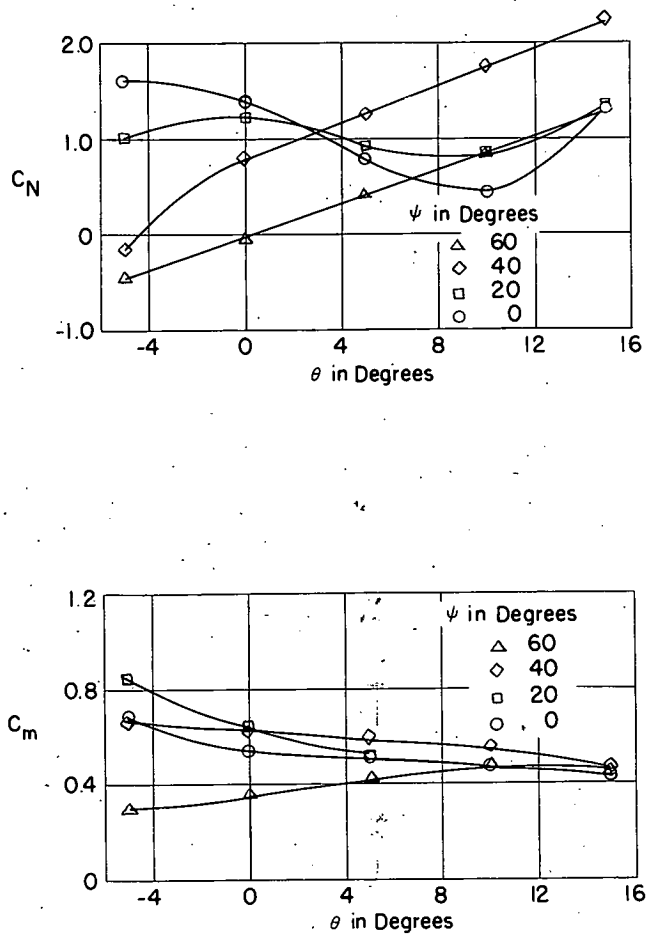


Figure H. Aerodynamic characteristics in pitch, four-girder model, wide sidewalk.

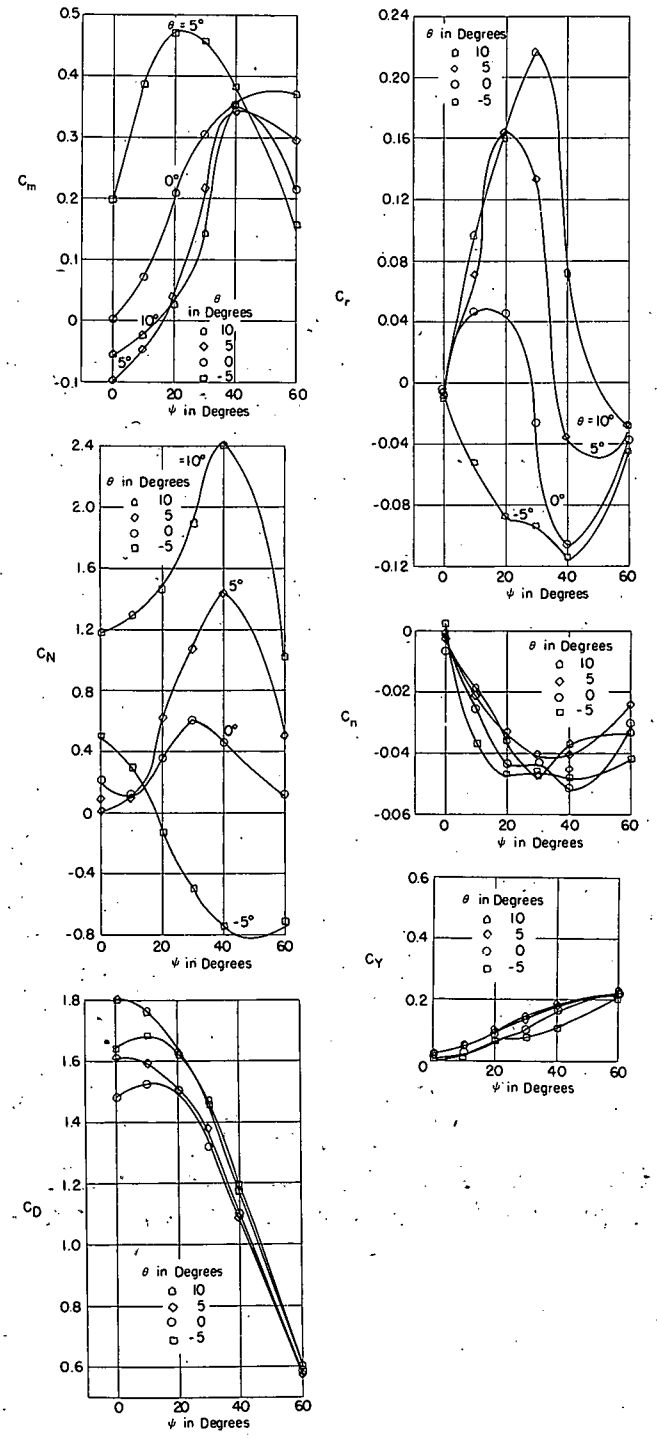


Figure I. Aerodynamic characteristics in yaw, six-girder model, narrow sidewalk.

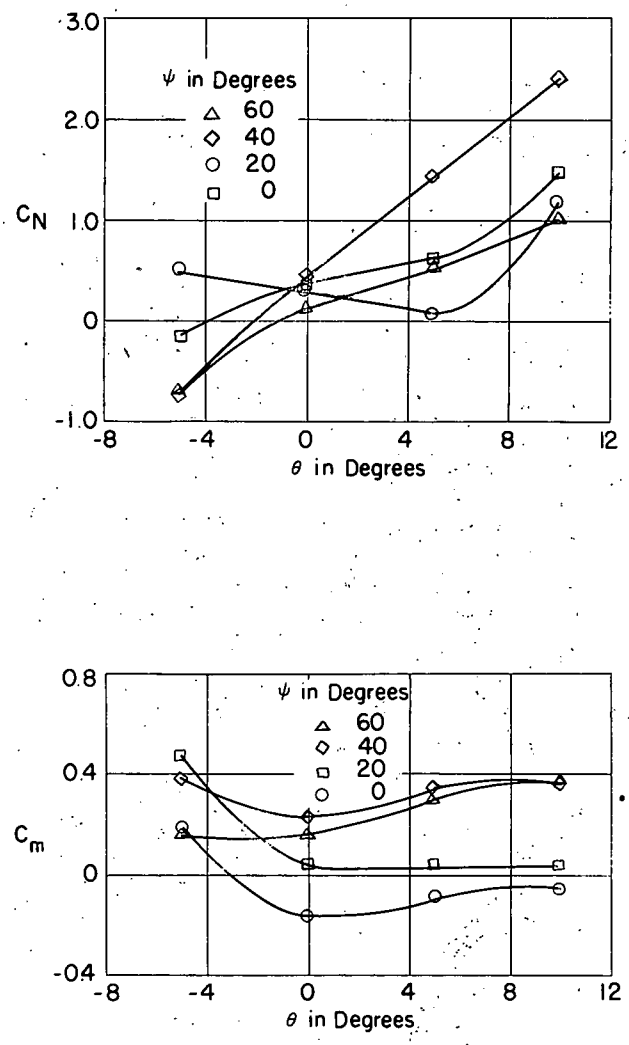
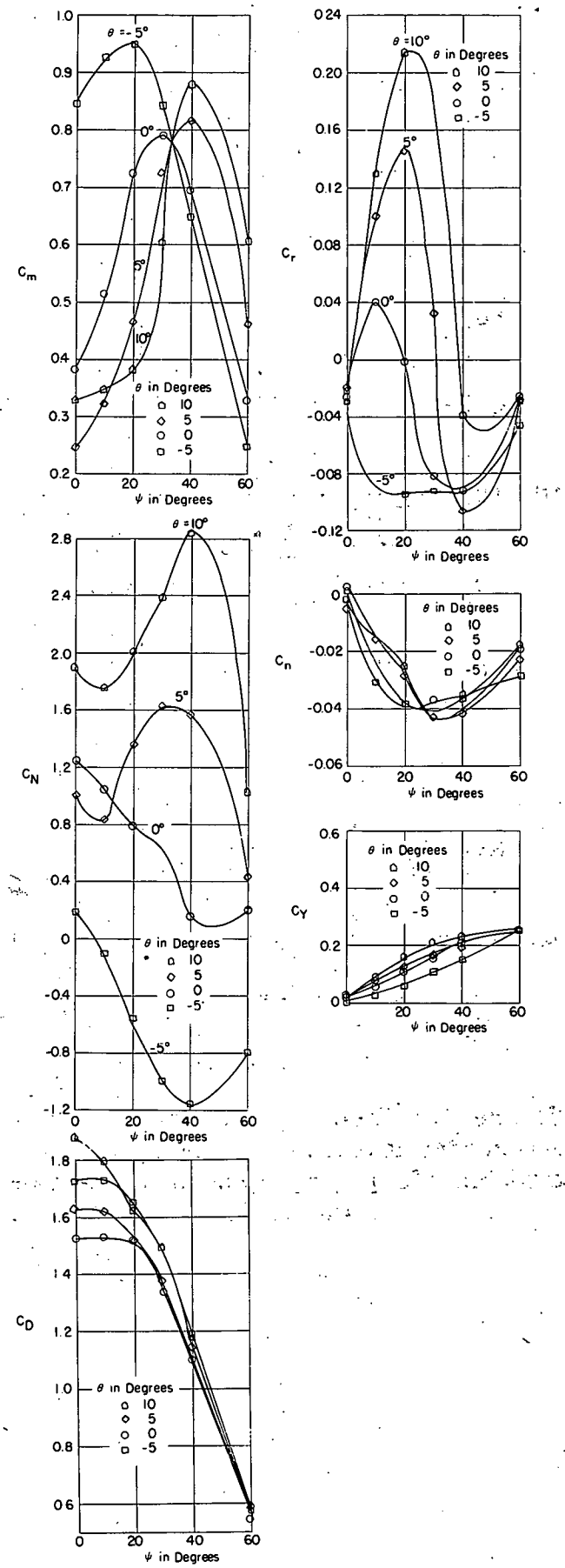


Figure J. Aerodynamic characteristics in pitch, six-girder model, narrow sidewalk.



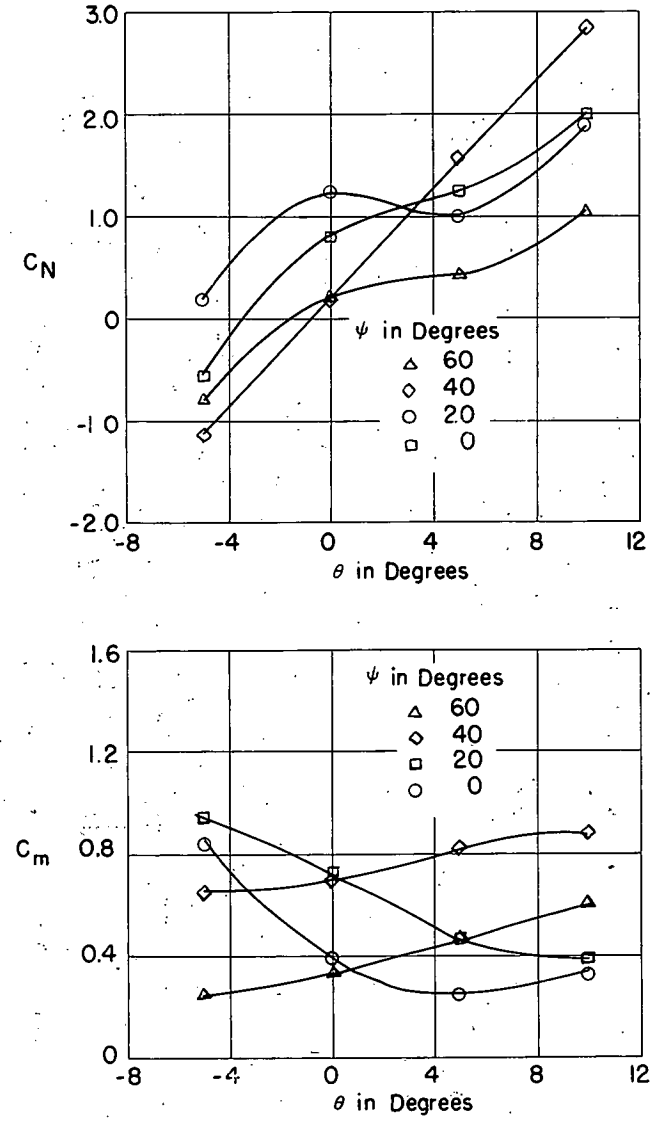


Figure L. Aerodynamic characteristics in pitch, six-girder model, wide sidewalk.

Figure K. Aerodynamic characteristics in yaw, six-girder model, wide sidewalk.

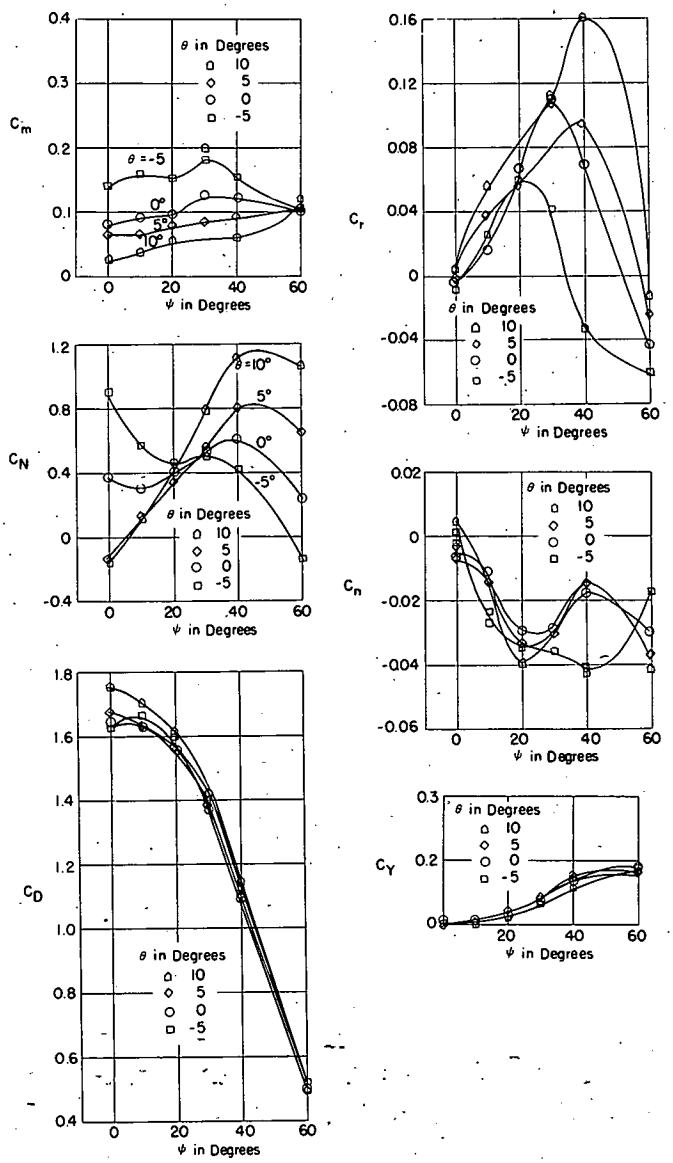


Figure M. Effect of removing interior girders, four-girder model with narrow sidewalks.

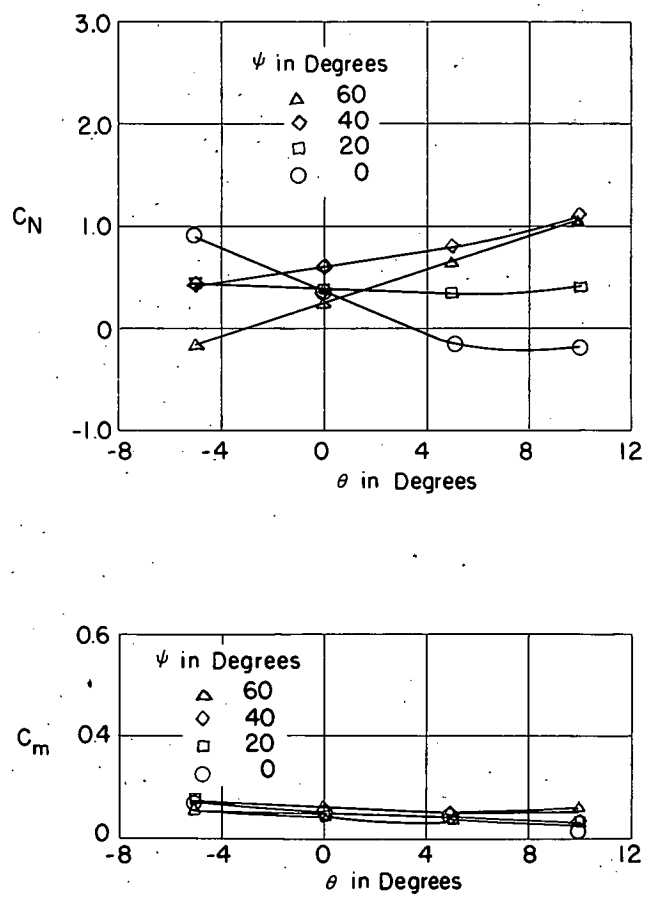


Figure N. Effect on aerodynamic characteristics in pitch of removing interior girders, four-girder model, narrow sidewalk.

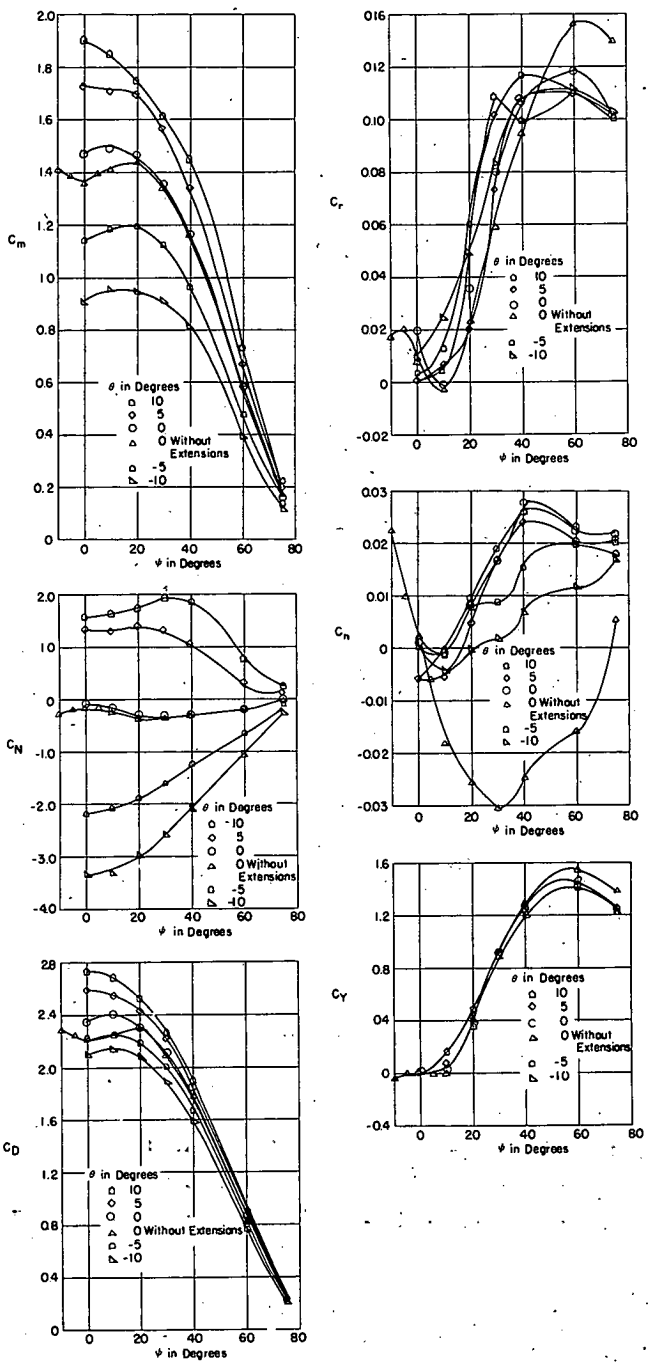


Figure O. Aerodynamic characteristics in yaw, through-truss model, 24-ft. roadway.

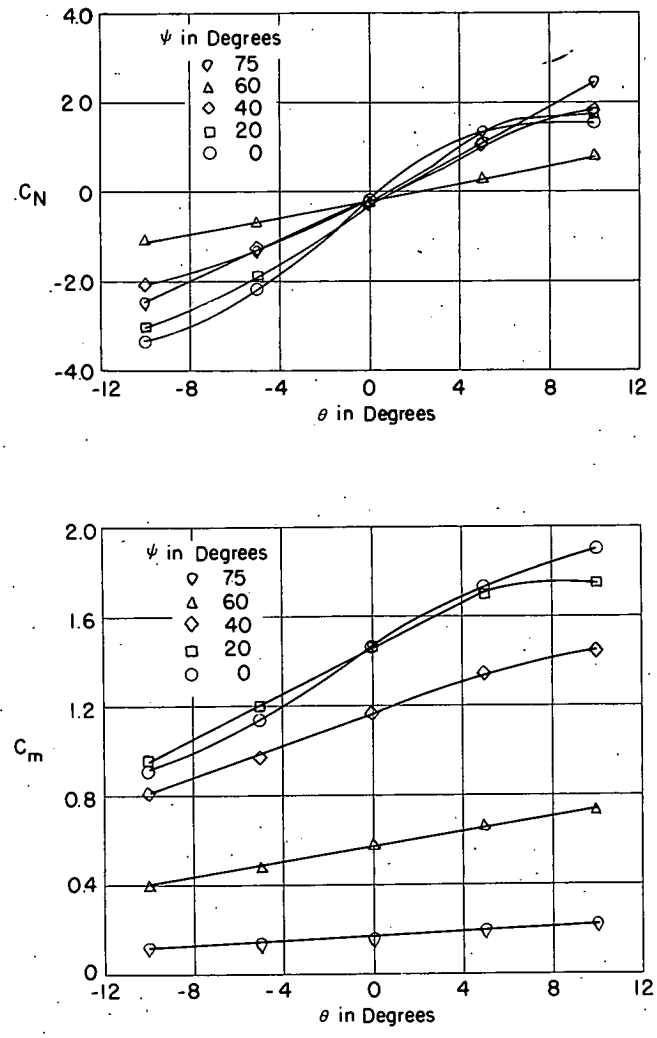


Figure P. Aerodynamic characteristics in pitch, through-truss model, 24-ft. roadway.

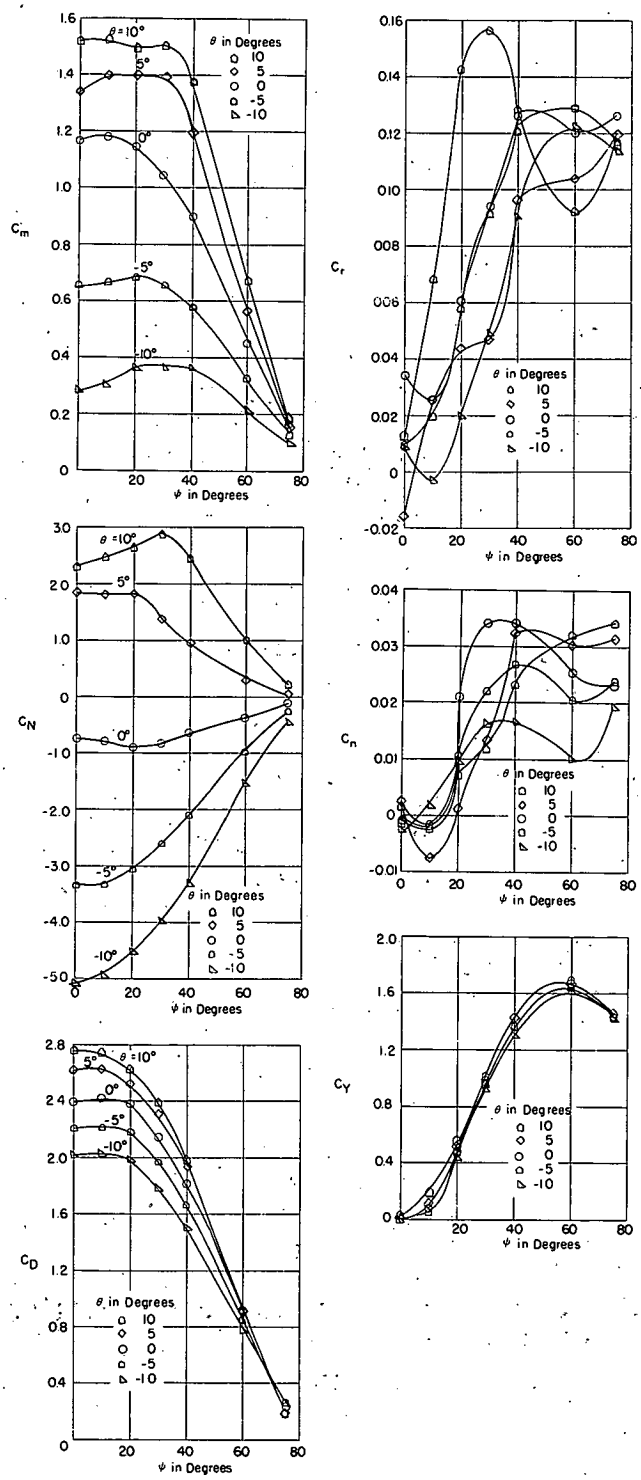


Figure Q. Aerodynamic characteristics in yaw, through-truss model, 36-ft. roadway.

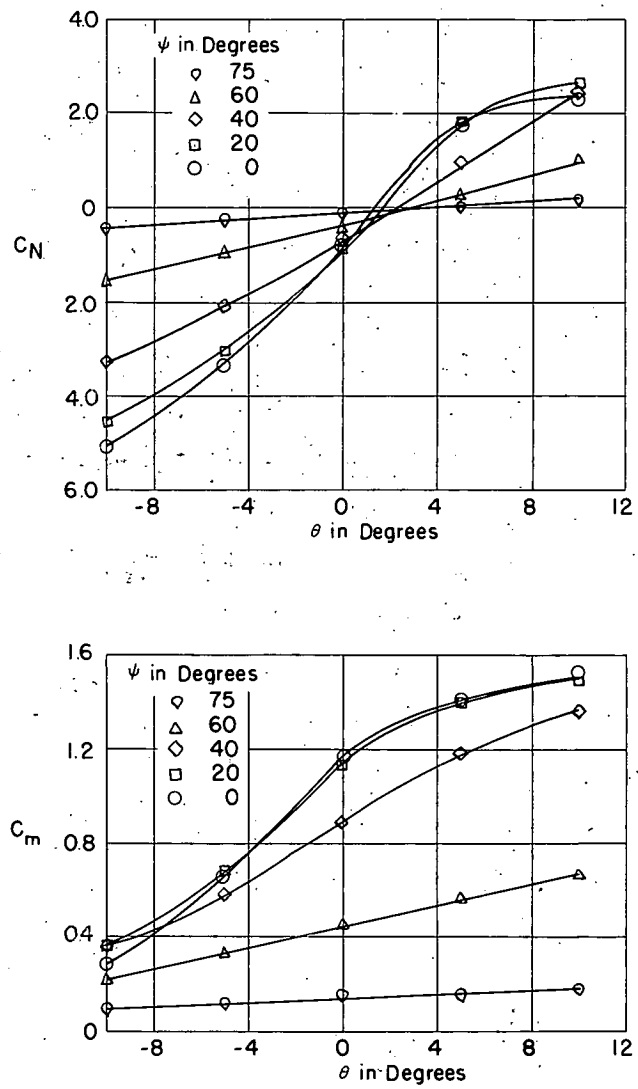


Figure R. Aerodynamic characteristics in pitch, through-truss model, 36-ft. roadway.

The Highway Research Board is organized under the auspices of the Division of Engineering and Industrial Research of the National Research Council to provide a clearinghouse for highway research activities and information. The National Research Council is the operating agency of the National Academy of Sciences, a private organization of eminent American scientists chartered in 1863 (under a special act of Congress) to "investigate, examine, experiment, and report on any subject of science or art."