FIELD MONITORING AND EVALUATION FOR SIGN SUPPORT STRUCTURES SUBJECT TO DYNAMIC LOADS

March 2003

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JHR 03-291 Project 00-5

This research was sponsored by the Joint Highway Research Advisory Council (JHRAC) of the University of Connecticut and the Connecticut Department of Transportation and was carried out through the Connecticut Transportation Institute at the University of Connecticut.

Technical Report Documentation Page

1. Report No.	2. Government Accession No.	3. Recipient's Catalog No.
JHR 03-291	N/A	N/A

4. Title and Subtitle Field Monitoring and Evaluation for Sign Support Structures Subject to Dynamic Loads	5. Report Date March 2003 6. Performing Organization Code N/A
7. Author(s) Michael DelGrego and John T. DeWolf	8. Performing Organization Report No. JHR 03-291
9. Performing Organization Name and Address University of Connecticut Connecticut Transportation Institute Storrs, CT 06269-5202	10. Work Unit No. (TRAIS) N/A 11. Contract or Grant No. N/A
12. Sponsoring Agency Name and Address Connecticut Department of Transportation 280 West Street Rocky Hill, CT 06067-0207	13. Type of Report and Period Covered Final 14. Sponsoring Agency Code N/A

15. Supplementary Notes

N/A

16. Abstract

Recent changes in the sign support specification have resulted in an increase in design wind pressures. As a result, some existing overhead bridge highway sign structures supported by two vertical trusses are no longer adequate, even though they have performed acceptably over the years. As a result of the new design provisions, a program was begun to reinforce the sign supports, which involved adding stiffeners to the vertical truss chords. An initial review of the, then, current design procedures, using estimated effective length factors, indicated that use of a more rigorous stability analysis could show that many of the existing vertical trusses had sufficient strength to meet the new wind loading without the expensive field modifications. This could be achieved through more accurate calculations of the effective lengths for the vertical truss chords. The study reported herein was undertaken to use the new stability software to study existing sign supports, review alternative approaches for strengthening the trusses when the improved stability analysis is not sufficient, and revise the overall design approach and design software used by ConnDOT. The results presented in this study can be used in both the review of existing sign structures and the design of new sign structures.

17. Key Words	18. Distribution Statement
Buckling, Highway Signs, Sign Structures, Stability, Steel Design, Trusses, Wind Loading	No restrictions. This document is available to the public through the National Technical Information Service Springfield, Virginia 22161

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price
Unclassified	Unclassified	50	N/A

Form DOT F 1700.7 (8-72) Reproduction of completed page authorized

SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimetres	mm
ft	feet	0.305	metres	m
yd	yards	0.914	metres	m
mi	miles	1.61	kilometres	km
		AREA		
in^2	square inches	645.2	millimetres squared	mm^2
ft^2	square feet	0.093	metres squared	m^2
yd^2	square yards	0.836	metres squared	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	kilometres squared	km^2
		VOLUME		
fl oz	fluid ounces	29.57	mililitres	mL
gal	gallons	3.785	Litres	L
ft^3	cubic feet	0.028	metres cubed	m^3
yd^3	cubic yards	0.765	metres cubed	m^3

NOTE: Volumes greater than 1000 L shall be shown in m^3

Symbol	When You Know	Multiply By	To Find	Symbol
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams	Mg
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5(F-32)/9	Celcius temperature	°C

* SI is the symbol for the International System of Measurement

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimetres	0.039	inches	in
m	metres	3.28	feet	ft
m	metres	1.09	yards	yd
km	kilometres	0.621	miles	mi
		AREA		
mm^2	millimetres squared	0.0016	square inches	in^2
m^2	metres squared	10.764	square feet	ft^2
ha	hectares	2.47	acres	ac
km^2	kilometres squared	0.386	square miles	mi^2
		VOLUME		
mL	millilitres	0.034	fluid ounces	fl oz
L	litres	0.264	gallons	gal
m^3	metres cubed	35.315	cubic feet	ft^3
m^3	metres cubed	1.308	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.205	pounds	lb
Mg	megagrams	1.102	short tons (2000 lb)	T
		TEMPERATURE (exact)		
°C	Celcius temperature	1.8C+32	Fahrenheit temperature	°F

°F 32 98.6 212 °F
-40 0 40 80 120 180 200
-40 -20 0 20 40 60 80 100
°C 37 °C

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Introduction

In 2001, AASHTO released a new edition of their Standard Specifications for Structural Supports for Highway Signs, Luminaires and Traffic Signals (1) (referred to hereafter as Support Specifications). This has required that states, including Connecticut, check the performance of existing sign supports and design new supports based on the updated standards of the Support Specifications. Most estimated wind speeds, especially along coastlines, have increased creating larger wind loads and higher stresses in the sign supports.

Due to this increase in wind speed, the Connecticut Department of Transportation required an investigation on all sign structures in the state to determine the adequacies of the supports. This report covers the overhead bridge structures supported on both ends by vertical trusses. An example of a typical truss supported sign structure is in Figure 1. The structure consists of a horizontal, three-dimensional box truss (Figure 2), which spans the highway and is supported on both ends by two vertical, two-dimensional, trusses (Figure 3).

The horizontal box truss is fabricated into multiple segments for ease of transporting, and connected together on the site. Each segment of the horizontal truss is made from round, tubular members with the chords typically having a larger cross-section than the diagonals. At both ends of the interior segments a plate is welded to each of the four chords, which allows it to be connected to the segment next to it (Figures 4 and 5). At the outer ends of the exterior segments, the four chords are typically connected to the vertical truss using U-bolts. (Fig. 6) One of the U-bolts is shown in Figure 7. The horizontal truss is usually made from aluminum because of its low weight.

The vertical support trusses consist of two main vertical chords with diagonals connecting them as shown in Figure 3. The in-plane direction of the two-dimensional trusses is parallel to the highway so that it braces the structure as the wind load pushes against the highway sign panels. Like the horizontal box truss, the chords are much larger than the diagonals and carry most of the forces. At the bottom of the vertical chords, base plates (as shown in Figure 8) are welded to the vertical truss chords. Four anchor bolts are used and the resulting connection is assumed fixed for moments. All the members in the vertical trusses are made out of steel because of its strength, stiffness, and ductility.

In an overhead bridge structure, the most critical stresses occur at the bases of the vertical chords. Calculating the actual stresses in these members is complicated, though, because the vertical truss supports are indeterminant. Prior analyses on the vertical truss supports were based on assumptions and simplifications, which possibly resulted in an over-designed support structure. However, when the same conservative design method was used to reanalyze the structures according to the updated Support Specifications with its increased wind loads, the vertical truss chords appeared to be overstressed. Gray, Wang, Hamilton and Puckett (2) have reported on signs that have collapsed. Following the revisions in the Support Specifications that required larger wind loads, the State of

Connecticut began a program to stiffen the critical members and hence, reduce the actual stresses. A typical stiffened vertical truss chord is shown in Figure 9.

Retrofitting was typically achieved by welding two steel stiffeners to the four vertical truss chords, increasing the moment of inertia of the chords about the weak axis for flexure and stability in the out-of-plane direction. However, the welding was performed on the site, which can lead to a reduced weld quality, compared to welding in the shop, and it is usually more expensive.

Studies of loads on sign supports are limited. Kaczinski, Dexter and Van Dien (3), Cook, Bloomquist and Kalajian (4), and Gray, Wang, Hamilton and Puckett (2) looked at fatigue problems. Cook, Bloomquist and Agosta (5), Johns and Dexter (6) and Cook, Bloomquist and Kalajian (4) have studied the influence of truck induced wind loads. DeWolf and Yang (7) at the University of Connecticut applied a system stability analysis to the trusses of interest in this investigation.

This research was initially undertaken to develop a more accurate stability analysis, correcting the simplified assumptions, so that the full capacity of the vertical truss support is obtained. The second part, reported herein, has involved a full review of the design process using the software developed in the first part by DeWolf and Yang (7). The work has led to a revised design procedure, based on the updated Support Specifications. The results of this research will reduce the need for the costly retrofitting.

Stability Behavior

Buckling is a major concern when it comes to the design of the tubular, vertical truss compression members. Fouad, Calvert and Nunez (8) have noted that the strength of steel tubes used in sign supports is one of the many areas in need of research. For the overhead truss supports, two modes of buckling can occur: in-plane buckling and out-of-plane buckling. For in-plane buckling of the vertical truss support, the chords are braced by the diagonals, which, reduces the effective length and raises the critical load. However, the chords are not braced in the out-of-plane direction, and hence, out-of-plane buckling normally governs in the design. There are different approaches that can be used to determine the effective length factors for design, depending on the assumptions made in the stability analysis.

Previous design practice, using simplified stability assumptions, is based on constant chord axial forces from both the gravity and wind loads. The gravity load results in axial compression forces in the support columns, and it is essentially uniform along the full height (there is a slight variation due to the gravity load resulting from the vertical truss self-weight). The wind however, since it is applied horizontally, results in a force that varies along the chord lengths, with the greatest forces occurring in the lower part of the chord. The assumption of constant axial forces can lead to conservative designs.

To account for the variable axial force, a system buckling analysis was derived using an eigenvalue analysis of the entire structural support. The method is based on a formulation of the geometric stiffness matrix with assumed displacement functions, developed by Hartz (9). The approach requires that the member be divided into multiple elements to achieve acceptable results as shown by Yang (10). This method then produces a stability analysis that can be used to determine effective length factors, K, based on the critical element in the truss support. The approach was used by Yang and DeWolf (7) to determine the critical effective lengths for both in-plane and out-of-plane buckling.

In-plane Buckling

In previous designs, the effective length factor of the vertical truss chords for in-plane buckling was based on the individual elements between the diagonals. The value of K is assumed equal to 1.0, representing the condition when the ends of each segment of the vertical chords are pinned and prevented from sidesway. Even when the diagonals are actually fixed to the chords, a pinned connection may be assumed. The moment of inertia of the diagonals is much smaller than the moment of inertia of the vertical chords and thus, the diagonals do not supply significant rotational resistance to the chord.

It is not correct to look only at the individual chord elements and not at the vertical truss support as a whole. Most of the axial stress in the vertical chords is a result of the horizontal wind load acting on the face of the sign panels. The applicable loads acting on a typical vertical truss are shown in Figure 10. Table 1 compares the stresses due to both wind load and dead load. The horizontal wind load is transferred to the vertical supports for the in-plane direction of the trusses. Due to the wind force, the vertical truss acts like a cantilevered beam with maximum moment at the base and zero moment at the top where the concentrated wind load is applied. This creates a compression force in the rear vertical chord, and a tension force in the front vertical chord. The axial force is a result of the moment from the wind acting on the vertical truss and is largest at the base and zero at the top, varying in between the two. This phenomenon reduces the effective length factor of the chords because the segments under less stress, near the top of the vertical truss, brace the segments near the bottom that are more highly stressed. Thus, assuming an effective length factor equal to 1.0 is conservative.

Out-of-plane Buckling

For out-of plane buckling, the effective length factor, K, was previously assumed equal to 1.0. This K value was used assuming the base of the vertical chords is fixed against rotation and translation and the top of the vertical chords is only fixed against rotation, but allowed to sway. The connection at the top of the vertical truss chords has been assumed as rotationally fixed because the horizontal truss attached to the top of the vertical chords has a much larger moment of inertia than the supporting truss bending about the weak axis, thus preventing any joint rotation. Again, assuming a uniform axial load in the chords is very conservative because the axial forces in the vertical chords are primarily due to the wind load. Like in-plane buckling, the effective length factors can be reduced with a more accurate stability analysis.

Additionally, in order to obtain a connection at the top that is restrained against rotation requires that the overhead box truss be connected so that it resists rotation. A review of typical signs in Connecticut has shown that U-bolts have been used for the connection between the supporting vertical truss and the horizontal box truss. A photo of a typical connection between the vertical supporting truss and the horizontal three-dimensional truss is shown in Figure 6. Figure 7 shows one of the four connections using the U-Bolt. Experience has shown that U-bolts cannot guarantee a fixed connection during the full life of the structure because of the effects of relaxation in steel. This can lead to slippage in the connection at the top of the support allowing some rotation. Thus, the current connection detail does not reliably provide full moment transfer. This shows that the previous assumption that K is equal to 1.0 may be unconservative. Modifying the connection so that slippage is prevented would result in higher stability strength.

AASHTO Design Provisions

Below is a description of the provisions from the Support Specifications that apply to overhead bridge sign structures.

Loads (Support Specifications Section 3)

The Support Specifications Section 3.4 specifies four different load combinations to account for dead load, ice load, wind load, and fatigue. They are:

(I) Dead Load only
(II) Dead Load + Wind Load
(III) Dead Load + Ice Load + ½(Wind Load)
(IV) Fatigue

Load combination (III) allows for the actual wind pressure to be reduced by 50%, but the (Wind Load) cannot be taken less than 25 psf. Also load combinations, (II) and (III), allow an overstress of 33%. Load Combination (IV), Fatigue, applies only to cantilever-type sign structures. Since the signs analyzed in this research were the overhead bridge sign structures, fatigue does govern.

Group II and Group III both have two load cases, as described below, to take into account wind gusts from any direction. To satisfy these circumstances, the Support Specifications Section 3.9.3 recommends applying a normal and a transverse component of wind simultaneously. The normal component shall be applied in the direction perpendicular to the face of the sign panels and the transverse component shall be applied in the direction parallel to the face of the sign panels.

Load Case 1: 1.0×(Wind Load) for the normal component and 0.2×(Wind Load) for the transverse component.

Load Case 2: 0.6×(Wind Load) for the normal component and 0.3×(Wind Load) for the transverse component.

For both cases, (Wind Load) shall be calculated as the load acting in the direction perpendicular to the face of the sign panels.

Dead Load Provisions (Support Specifications Section 3.5)

The dead loads included in all calculations shall be any load permanently attached to the structure and any temporary load applied during maintenance. These include weight from the signs, horizontal truss, and vertical trusses. The Connecticut Department of Transportation recommends a flat panel sign weight of 3 psf for normal signs of interest in this study and 12 psf for Variable Message Signs (VMS). The full dead load shall be applied for load combinations: (I), (II), and (III).

Ice Load Provisions (Support Specifications Section 3.7)

An ice load of 3 psf shall be used in all areas of Connecticut. The ice load applied to the sign structure assumes 0.60 inches of ice, weighing 60 psf, and it may accumulate on the exposed surface areas of all members. However, the ice shall only be considered on one face of each sign panel due to the vertical orientation of the signs. Ice loads only apply to load combination (III).

Wind Load Provisions (Support Specifications 3.8)

The Support Specification editions up to 1994 used a different equation than the 2001 edition for estimating wind pressure. Below is a comparison of the two equations.

The old Support Specification equation for the wind pressure was:

$$P_z = 0.00256(1.3V)^2 C_d C_h \qquad \text{(psf)}$$

where:

V = Fastest-mile design wind speed from the isotach map
C_d = Drag Coefficient
C_h = Coefficient for height measured above ground
The Support Specification 2001 edition equation is:

$$P_z = 0.00256\, K_z G V^2 I_r C_d \qquad \text{(psf)} \qquad \text{(Support Specifications Eq. 3-1)}$$

where:

V = 3-second-gust wind speed from isotach map
C_d = Drag coefficient
K_z = Coefficient for height measured above ground

I_r = Wind importance factor
G = Gust effect factor, determined from an equation

The Wind Importance Factor equals 1.0 when a recurrence interval of 50 years is chosen. This corresponds to the recurrence interval used for the isotach map in the 2001 edition. Rearranging the 2001 equation gives:

$$P_z = 0.00256 \ G \, V^2 I_r C_d K_z$$

Comparing the past and present equations, assuming C_d is the same in both equations, K_z equals C_h, I_r = 1.0, and G = 1.14, as determined from the Support Specifications, shows that the difference is in the wind speed portion. The old equation used $(1.3\ V)^2$ and the new equation uses $(1.14\ V^2)$, with different specified wind speed values, V.

The design wind speed, V, in the past editions of the code, was the fastest-mile wind speed. This speed is the peak wind speed averaged for 1 mile of wind passing at a point. In the 2001 edition, the wind speed, V, is the 3-second-gust wind speed, which is the average wind speed measured over an interval of three seconds.

According to the 2001 edition of the code, the 3-second-gust wind speed is approximately 22% faster than the fastest-mile wind speed. Using this fact and inserting (1.22 V) into the past equation will produce the same exact wind pressure as inserting (1.0 V) into the Support Specifications 2001 equation. This change in the equation for wind pressure along with an increase in wind speeds led to new wind speed maps, based on the 3-second-gust wind speed.

In the previous editions of the code, the fastest-mile wind speed for Connecticut was 80 mph. The Support Specifications 2001 map now shows a 3-Second-Gust wind speed of 120 mph along the coast and 110 mph for the inland portions of Connecticut. Inserting 80 mph into the old wind pressure equation and 110 mph and 120 mph into the Support Specifications 2001 equation shows an increase between 27% and 51% in wind pressure, depending upon the location of the sign structure.

<u>Allowable Stresses</u>

Almost all of the supports in Connecticut are made from steel and aluminum, but the Support Specifications also allows for members to be made from wood or fiber-reinforced composites. The aluminum and steel design guidelines in the Support Specifications both have very similar approaches for determining the allowable stresses. The allowable stresses are related to the member’s slenderness ratio.

For aluminum, there are two slenderness ratio limits that divide members into three categories. If the slenderness of a member is smaller than the lower limit, it is defined as compact. These types of members do not buckle until after its full cross-section has yielded. If the slenderness of the member is larger than the upper limit, it is defined as slender. A slender member is one that will buckle before the yield stress has been reached and, therefore, will buckle elastically. If the slenderness is between the two limits, the member is defined as non-compact. A non-compact member buckles after a portion of the cross-section has yielded, and full cross-sectional yielding will not be reached. Once a member has been defined as compact, non-compact, or slender, the allowable stresses for bending, shear, and axial compression can then be calculated.

Determining the allowable bending stress in steel is similar to the process for aluminum. However, shear and axial compression only have one slenderness ratio limit for steel members, which separates members into two categories. The limit will determine whether a member will buckle elastically or buckle while in its inelastic range.

CSR Equations

The actual stresses are then compared to the allowable stress. The Support Specifications design requirements for the combination of wind and gravity load in the vertical truss chords are based on interaction equations. The approach involves combining the effects of axial load, moment, and shear to determine values of *CSR*, combined stress ratio. The design is acceptable if all applicable *CSR* values are equal to or smaller than 1.0. There are three equations given for determining the *CSR* values. The first two apply where the axial load is large and the third applies when it is small. The three equations are as follows:

$$\frac{f_a}{0.6F_y} + \frac{f_b}{F_b} + \left(\frac{f_v}{F_v}\right)^2 \leq 1.0 \qquad \text{(Support Specifications Eq. 5-17)}$$

$$\frac{f_a}{F_a} + \frac{f_b}{\left(1 - \frac{f_a}{F_e'}\right)F_b} + \left(\frac{f_v}{F_v}\right)^2 \leq 1.0 \qquad \text{(Support Specifications Eq. 5-18)}$$

$$\frac{f_a}{F_a} + \frac{f_b}{F_b} + \left(\frac{f_v}{F_v}\right)^2 \leq 1.0 \qquad \text{(Support Specifications Eq. 5-19)}$$

where:
f_a = Actual axial stress (ksi)
F_a = Allowable axial stress (ksi)
f_b = Actual bending stress (ksi)
F_b = Allowable bending stress (ksi)
f_v = Actual shear stress (ksi)
F_v = Allowable shear stress (ksi)
F_e' = Euler buckling stress (ksi)

Design Process

Most designs involve a trial and error process. Thus, it requires the designer assume member sizes, find the actual stresses in each member, and then compare the actual stresses to the allowable stresses. This process must be repeated until all the member sizes are sufficient. The best way to perform iterations is to develop a program that will do basic calculations. This study has involved modifying and updating the design approach. The formal design process is shown in the spreadsheet previously used in Connecticut, both to meet the new code provisions and to make use of the stability analysis developed in the first part of this study (7). The updated spreadsheet is shown in Appendix A.

The spreadsheet design has the ability to analyze vertical truss supports for new sign structures by inputting a trial cross-section. This process will find the most efficient sized members that will satisfy all design requirements and also reduce the cost that could result from a potential over-design. The design spreadsheet requires the input of all the dimensions including the cross-sectional properties. It then calculates the CSR values based on the above equations.

The design process can also be used to analyze existing signs that were designed according to the old Support Specifications wind loads. Using the actual dimensions and inputting the necessary data into the program for the existing structure, the CSR values are calculated and displayed at the top of the spreadsheet. This can be used to determine if an existing sign structure needs to be strengthened. If a trial cross-section does not satisfy the CSR equations, many options are available to increase the overall structural capacity, as discussed in the next section.

Increasing the Structural Capacity

The most effective ways of strengthening the structure are to make alterations as follows. One option is to modify the connections at the top of the vertical truss supports to be able transfer moment. Another is to increase the size of the vertical truss diagonals. A third approach is to increase the size of the vertical truss chords. All three of these suggestions should be taken into consideration before making a final decision because ways of minimizing the amount of steel and the cost may not always be obvious. Each option is discussed in the following sections.

Modifying Connection at Top of Vertical Truss Supports

One method of increasing the vertical truss chord's capacity is to modify the connection with the horizontal truss. As shown in Table 2, the moments will vary due to dead and wind loads depending on the type of connection that is used at the top of the vertical truss supports. In the following, both the moment resistant and pinned cases are discussed separately, noting the beneficial design aspects for each.

If the connection between the vertical truss and horizontal truss is pinned, the horizontal truss is assumed as a simply supported beam, transferring only the vertical reactions from gravity load to the vertical truss supports. The moment due to the gravity load from the signs and horizontal truss will not be transferred to the base of the vertical truss. The moment at the base will only be a result of horizontal wind components.

However, the effective length factor will be fairly large because the tops of the vertical truss chords are free to rotate and sway. Also, the vertical truss support acts like a cantilever in the out-of-plane direction. The transverse component of wind will only be resisted at the base of the vertical truss support. This will cause large moments at the base due to wind, and possibly require a large cross-section.

If the connection between the vertical truss and horizontal box truss is capable of transferring full moment, the gravity load from the horizontal truss supporting the signs will result in a vertical reaction and a moment in the vertical truss support. The vertical reaction and moment must be transferred to the base through the vertical truss chords, increasing the moment at the base. However, because the vertical chords are resisted from rotation at the top and bottom, the effective length factors are decreased. Also, because the connections at the top and bottom of the vertical truss chords is rigid, the moment from the transverse component of wind (acting in the out-of-plane direction) will be reduced at the base by about 50% with the other half being taken by the top connection.

Increase Size of Vertical Truss Diagonals

Another approach to strengthen the vertical truss supports is to increase the buckling strength of the elements. Ultimate failure will occur by buckling of the vertical chords, which is directly related to the effective length. Decreasing the effective length of the chord allows them to carry a larger load. This can be accomplished by increasing the size of the diagonal members, which helps brace the vertical truss supports against sway. As shown by DeWolf and Yang (7), doubling the moment of inertia of the tubular diagonals will decrease the effective length by about 31% and by tripling the moment of inertia the effective length will decrease by about 44%. However, increasing the diagonal sizes only reduces the effective length factor for the in-plane direction. When the out-of-plane direction governs in the design, which is more common, then increasing the size of the diagonals is unproductive.

Increase Size of Vertical Truss Chords

Since changing the connection at the top of the supports so that they transfer moment and/or increasing the size of the diagonals may not be adequate, an alternative is to increase the size of the vertical truss chords. This works because the governing CSR equations are based on the forces at the bottom of the vertical chords. Changing the size of the chords impacts the slenderness ratio of the member and directly affects the results of the CSR equations. The process is trial and error, but normally only a few tries are needed to determine the most efficient cross-section.

Summary

In review, the benefit of using a moment resisting connection at the top of the support is that the effective length factor for the vertical truss support as well as the moment at the base from the transverse component of wind are both reduced. If a pinned connection is used instead, the effective length factor will be much larger. However, with a pinned connection, there will be no additional moment at the base due to the gravity loads acting on the horizontal truss. Thus, the choice of connection depends on the actual moment at the base of the vertical truss chords due to the gravity loads on the horizontal truss. The moment is affected by, both, the magnitude of the gravity load and the length of the span. In other words, very large signs near mid-span can greatly increase the moment due to its weight and the ice loading on the large surface area, and longer spans can significantly increase the moments on the supporting truss when a rigid connection is used. This can have a negative effect on the structural capacity.

The new design procedure developed in this investigation incorporates either a pinned or moment resistant connection at the top of the vertical truss supports. The effective length factors for both cases are available from the stability analysis developed by DeWolf and Yang (7). The results for the software are manually input into the spreadsheet.

Design Example

The sign structure used to discuss the behavior and demonstrate how modifications can be made to meet the new Support Specifications for existing signs is shown in Figure 11, and the vertical truss support is shown in Figure 12. This sign is typical of those used in Connecticut. The chords are made from 10-inch tubes with a wall thickness of 0.365 inches, and the diagonals are made from 3.5-inch tubes with a wall thickness of 0.188 inches. The sign structure was sized to meet the old Support Specification requirements, using the lower wind pressures.

In order to use the stability software to calculate the effective length factors of the vertical truss chords, the loads applied to the vertical support must be determined. These loads are determined by inputting the known dimensions and properties of the existing sign structure, excluding the effective length factors, and applying the equations in Appendix B. Once the loads have been calculated, these values can be input into the stability software. After successfully running the stability program developed by Yang(7), effective length factors can then be manually input into the design spreadsheet. If the stability software is not used to calculate the effective length factors, the values must be approximated.

Effective Length Factors Using the System Stability Analysis

The advantages of using the system stability approach to determine effective length factors, K, are shown in Table 3. This table is based on the sign shown in Figures 11 and 12, varying the supporting chord sizes, using the available 8-inch and 10-inch tubes. The K values shown for the chords are based on using the full column length to obtain an effective length.

The results for the diagonals are not shown in this table. The use of the system buckling analysis for in-plane behavior has shown that the actual K values for the diagonals are 1.0 if the diagonals are pinned to the chords, as expected. The values decrease to approximately 0.5 for diagonals rigidly connected to the chords. This is because the chords are typically much larger than the diagonals. Thus, this is approximately the same as having a rigid connection at the ends of the diagonals. This is discussed in more detail in the report by DeWolf and Yang (7).

For in-plane behavior, Table 3 gives K values for the chords that are based on the full chord length. The normal design approach has been to use a K value of 1.0 with the largest length between diagonals. A direct comparison between the two K values is then not correct. The research has shown however that the effective length, equal to K times the actual length, obtained from the system stability analysis is often larger than the value previously used in the normal design approach. This is because there is some sidesway. The result is that the normal assumptions used in the design of these columns can produce an unconservative design for in-plane behavior. Fortunately, as has been demonstrated by DeWolf and Yang (7), the out-of-plane behavior governs for design, and thus the structure is not unconservative.

As shown in Table 3 for out-of-plane behavior, the values of the effective length factor, K, computed for the vertical truss chords are considerably smaller than the values of 2.0 used when the top is pinned and 1.0 when the top is rigidly connected to the horizontal truss. The effective length factors are reduced by as much as 28 percent when the tops are pinned to the horizontal truss and as much as 13 percent when the tops are rigidly connected to the horizontal truss. Since the out-of-plane behavior generally governs, the improvement in the design strength is significant. This demonstrates the benefit of including the chord's variable axial force in the stability considerations.

Table 3 also shows that designing the connections between the vertical truss and horizontal truss so they are able to transfer moment substantially lowers the effective length, and hence, increases the column stability strength. This requires that the connections between the horizontal truss and the vertical support trusses have sufficient moment capacity so the trusses remain at right angles with respect to each other. Since the lower and upper chords in the horizontal truss are both connected to the vertical support trusses, this can be achieved by connecting the chord elements to the vertical truss so that there is no slippage during the life of the sign.

Comparing the CSR Values with and without the System Stability Analysis

The design example given in Figures 11 and 12 is now used to show the benefits of using the system stability analysis to determine more realistic effective length factors, K. The chord size is based on the governing lower truss chord segment, where the axial force from the wind is largest. The basic design requires that the applicable *CSR* values be equal to or smaller than one. Table 4 shows the governing K values and the maximum *CSR* value for the different design cases. The first four cases are based on having pinned connections between the supporting truss and the horizontal truss. The first three cases are based on the normally assumed K value of 2.0, i.e. with a pinned connection at the top of the supporting truss. The fourth case uses the system stability approach to determine a more realistic K value. The fifth case is based on using a moment-resistant connection at the top of the supporting truss.

Case 1 - Original Design, Old Support Specifications for Wind Load

The wind pressure that is applied to the sign faces determined from the 1994 Specification is 25.8 psf for the coastal areas in Connecticut. As is shown, the chord size results in a maximum CSR value of 0.97, and the design is satisfactory.

Case 2 - Original Design, New Support Specifications for Wind Load

The wind pressure applied to the sign faces, based on the new requirements for the coastal areas in Connecticut is 39.0 psf. The maximum *CSR* value is now 2.00, and as expected, the design is now unacceptable.

Case 3 - Original Design with Added Stiffener, New Support Specifications for Wind Load

The approach that has been used in Connecticut to meet the larger required wind pressure has been to weld stiffeners to the chords, as shown in Figure 8. These are typically 1 inch by 2-inch bar elements. The prime cost in attaching these is the due to the extensive labor. The *CSR* value is now 0.71, and the sign is more than adequate for the new Support Specification.

Case 4 – Original Design, New Support Specifications for Wind Load with System Stability Approach and Pinned Connection at the Top of the Vertical Supporting Trusses

The system stability approach results in a K value of 1.44 for the pinned case, as opposed to the normally assumed value of 2.00. As is shown, the CSR value is now 1.01. Accepting a value that is approximately 1 percent above the maximum, the original sign now meets the new specification without the need for stiffening.

Case 5 – Original Design, New Support Specifications for Wind Load with System Stability Approach and Moment-Resistant Connection at Top of Vertical Supporting Trusses

The largest resulting *CSR* value of 0.99 shows the sign is slightly over-designed. Review of the detailed calculations, as shown in Table 5, shows that the increase in the moment from the dead load due to the change in the joint rigidity approximately balances out the benefits from modifying the connection. DeWolf and Yang (7) have shown that for other signs, the benefits of using moment resistant joints at the top of the support trusses can significantly increase the capacity.

Conclusions and Recommendations

Changes in the Support Specifications has resulted in increased wind speeds and wind forces acting on sign supports. In Connecticut, this has led to field modifications involving the expensive stiffening of the vertical truss chords. This study has looked at ways to avoid the altercations.

The initial part of this investigation explored the stability assumptions. Previously, approximations in the effect length factor led to conservative designs. The result was that vertical truss supports appeared to have a lower structural capacity. Using the system stability analysis developed by DeWolf and Yang (7) has led to significantly increased capacities. The result is that many existing signs should satisfy the updated Support Specifications without modifications.

This study has produced a design approach, based on the existing approach, for determining the capacity of truss sign supports. The existing spreadsheet approach has been modified and updated for use in design. It is based on using the previously developed system stability analysis to calculate the effective length factors for the vertical truss chords.

The study has also explored alternatives for stiffening the vertical truss supports when the use of the stability analysis developed in the initial part of this investigation is not provide capacity. The following are ways that strengthening may be accomplished:

- Stiffeners can be welded to the vertical truss chords to reduce the stresses
- The connection at the top of the vertical trusses can be modified so as to resist moment. This could reduce the effective length factors and increase the structural capacity.

The design approach developed in this study can also be used in the design of new vertical truss supports. Using the design spreadsheet developed in this study, the combine stress ratio (CSR) values can be calculated for trial member sizes based on estimated effective length factors. Once the CSR values are adequate, the stability analysis developed by DeWolf and Yang (7) can then be used to do a full analysis calculating the actual effective length factors and input them into the spreadsheet. If the governing CSR value, calculated during the full analysis, is equal to 1.0, then the member sizes are adequate. Otherwise the process must be repeated using new member sizes. If a member is not adequate, the following options should be considered:

- Modify the connection at top of vertical truss supports. This will reduce the effective length factor for the vertical chords, but will also attract more moment at the base.
- Change the diagonal sizes in the vertical truss. Increasing the diagonals increases the bracing effect for the vertical chords, reducing the effective length.

- Change the vertical truss chord sizes. This will reduce the actual stress at the base and reduce the slenderness of the chords.

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APPENDIX A - OVERHEAD TRUSS SIGN SUPPORT POST ANALYSIS

Data Input Section

Note: At cell K332 and K333 there is a switch for retrofitting the posts.

Struct. No	Type	Town	Rte & Dir.	Mile Pt	CE CSR	DOT CSR	Date
21112	42III	Norwich	2 & 32	36.8	6.81	1.01	12/6/02

Span Length	**125.00** ft	Unit Weight =	Truss	**169** lbs/cf	
Number of Truss Sections	**4**		Posts	**490** lbs/cf	
				Aluminum	169 lbs/cf
Height of Truss Above Ground	**25** ft			Steel	490 lbs/cf

Wind Velocity	**120** mph
Wind Importance Factor (Ir)	1
Velocity Conversion Factor (Cv) =	1
Gust Effect Factor (G)	1.14

HORIZONTAL TRUSS

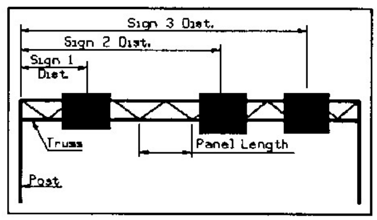
Horizontal Truss Dimensions

Height	**6.660** ft
Width	**6.660** ft

ICE LOAD = 3 psf
(AASHTO Luminaires, Section 3.7, pg. 3-4)

Exterior Section Length	**31.76** ft
Interior Section Length	**32.71** ft
Panel Length	**12.80** ft
Dia. Exterior Section	**6.000** in
Exterior Sec. Thickness	**0.250** in
Dia. Interior Section	**6.000** in
Interior Sec. Thickness	**0.250** in

(If the left and right section lengths are different for the exterior and interior sections respectively then average the values.)



Diagonals — Enter 0 for tube, 1 for angle

Vertical			Horizontal			Interior		
	Type =	0		Type =	1		Type =	0
Tube Dia	(in.)	3.500	Angle Leg	Height (in.)	3.500	Tube Dia	(in.)	3.500
	Enter 0 Here	0.000		Width (in.)	2.500		Enter 0 Here	0.000
	Thick (in.)	0.156		Thick (in.)	0.156		Thick (in.)	0.156
Vertical	Area (si)	1.639	Horizontal	Area (si)	0.912	Interior	Area (si)	1.639
	Length (ft)	9.237		Length (ft)	9.237		Length (ft)	9.419

SIGNS

Number of Signs **4**

Calculated Values

Dimension From Left Vertical Truss to Center of Sign		Distance between signs from left to right when facing the inventory route. (ft)		Dist. from left vertical truss to center of exposed truss	
Sign No.	Distance	L1 =	19.50	D1 =	9.75
		L2 =	12.50	D2 =	36.75
1	**25.00** ft	L3 =	11.00	D3 =	62.50
2	**50.00** ft	L4 =	12.50	D4 =	88.25
3	**75.00** ft	L5 =	19.50	D5 =	115.25
4	**100.00** ft	L6 =	0.00	D6 =	0.00
	0.00	L7 =	0.00	D7 =	0.00
	0.00				

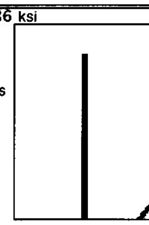
Dimensions of Signs

Sign no.		Height	Width	Enter 1 for VMS, 0 for regular sign.
1		**11.00** ft	**11.00** ft	0
	Crown	**0.00** ft	**0.00** ft	
2		**14.00** ft	**14.00** ft	0
	Crown	**0.00** ft	**0.00** ft	
3		**14.00** ft	**14.00** ft	0
	Crown	**0.00** ft	**0.00** ft	

4		11.00 ft	11.00 ft	0
	Crown	0.00 ft	0.00 ft	
		0.00	0.00	0
		0.00	0.00	
		0.00	0.00	0
		0.00	0.00	

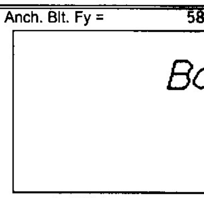
Vertical Truss Chords' Dimensions — Post Fy = 36 ksi

			Left Vertical Truss	Right Vertical Truss	
		Diameter	10.75 in	10.75 in	
		Thickness	0.365 in	0.365 in	
V		* Total Ht.	29 ft	29 ft	* Total Ht.= 1' above top of truss
E	T	c-c Post Spacing	5.25 ft	5.25 ft	
R	R	Panel Spacing	10.5 ft	10.5 ft	
T	U	Bracing Diameter	3.5 in.	3.5 in.	
I	S	Bracing Thickness	0.188 in.	0.188 in.	
C	S	Bracing Length	6.82 ft.	6.82 ft.	
A		Bracing Area	1.96 si	1.96 si	
L		Connection at top of Chord	1 (Enter 1 for pinned, or 2 for fixed)		
		K Longitudinal	0.84		
		K Transverse	1.44		



Anch. Blt. Fy = 58 ksi

B	P	"B" Longitudinal Dim. of BP	12.76 in	
A	L	"D" Transverse Dim. of BP	12.76 in	
S	A	CL Post to CL of Anch. Blt.	4.44 in	
E	T	Diameter of Anchor Bolt	1.5 in	
	E	Tensile Stress Area	1.41 si	(AISC 8th ed p4-141)



COMPUTATIONS

Dead Load of Horizontal Truss

Exterior Section				Interior Section			
Main Chords		4.52	si	Main Chords		4.52	si
	4 chords * Unit Wt.=	21.20	lbs/lf		4 chords * Unit Wt.=	21.20	lbs/lf
Diagonals	Hor & Ver	7.36	si/Ext. section	Diagonals	Hor & Ver	7.36	si/Int. section
		8.64	lbs/lf			8.64	lbs/lf
	Interior	2.41	si/Ext. section		Interior	2.41	si/Int. section
		2.83	lbs/lf			2.83	lbs/lf
	Ext. Truss D.L. =	33.65	lbs/lf	Int. Truss D.L.=		33.65	lbs/lf

Dead Load of Signs

Sign No.	Area	Weight
1	121.00	363.00
2	196.00	588.00
3	196.00	588.00
4	121.00	363.00
	0.00	0.00
	0.00	0.00

Dead Load of Vertical Truss Chords

Left Support Chord	11.91	si/chord
Bracing	22.70	lbs/chord
Wt of one post =	1197.82	lbs
Right Support Chord	11.91	si/chord
Bracing	22.70	lbs/chord
Wt of one post =	1197.82	lbs

D.L. Vertical Reactions at Top of Vertical Trusses from Horizontal Truss and Signs

	Left		Right
(Truss)	2169.48 lbs		2169.48 lbs
(Signs)	951.00 lbs		951.00 lbs
(Total for the support)	3120.48 lbs	(Total for the support)	3120.48 lbs

D.L Vertical Reactions at Base of Vertical Truss Chords from Horizontal Truss, Signs and Selfweight

	Left	Right
(Near)	2758.06 lbs	2758.06 lbs
(Far)	2758.06 lbs	2758.06 lbs

Ice Load on Horizontal Truss

Main Chords		1.57	ft^2/lf post
	4 chords =	18.85	lbs/lf
Diagonals	Vertical	2.64	ft^2/lf panel
	Horizontal	2.89	ft^2/lf panel
	Interior	2.88	ft^2/lf panel
	Diagonal ice load	25.22	lbs/lf
Truss Ice Load =		44.07	lbs/lf

Ice Load on Diagonals of Vertical Trusses

	Left Support	
Circumference	11.00	in.
Area	1.19	ft^2/lf chord
Weight	51.78	lbs/chord
	Right Support	
Circumference	11.00	in.
Area	1.19	ft^2/lf chord
Weight	51.78	lbs/chord

Ice Load on Signs

Sign No.	Area	Weight
1	121.00	363.00
2	196.00	588.00
3	196.00	588.00
4	121.00	363.00
	0.00	0.00
	0.00	0.00

Ice Load on Vertical Trusses

Left Support	2.81	ft^2/lf chord
Total weight including diagonals	296.63	lbs./chord
Right Support	2.81	ft^2/lf chord
Total weight including diagonals	296.63	lbs./chord

Vertical Reactions at Top of Vertical Truss due to Ice Load

	Left		Right
(Truss)	2754.61 lbs		2754.61 lbs
(Signs)	951.00 lbs		951.00 lbs
sum =	3705.61 lbs	sum =	3705.61 lbs

Vertical Reactions at Base of Vertical Truss Chords Due to Ice Load

	Left	Right
(Near)	2149.44 lbs	2149.44 lbs

(Far)	2149.44 lbs	2149.44 lbs

Wind Loads on Horizontal Truss - Section 3.8 with applied wind velocity

Height and Exposure Factor (Kz) =	0.95		
Wind Drag Coef (Cd) =	0.63		
Wind Pressure =	25.01 psf	Wind Load =	71.07 lbs/ft
Pressure for Group III Load Combination =	12.50 psf	Wind Load =	35.53 lbs/ft

Wind Loads on Signs - Section 3.8 with applied wind velocity

Height and Exposure Factor (Kz) = 0.95

Sign No.	L/W Ratio	Drag Coef (Cd)	Pressure	Group III Load Combination Pressure
1	1	1.12	39.03 psf	19.51 psf
2	1	1.12	39.03 psf	19.51 psf
3	1	1.12	39.03 psf	19.51 psf
4	1	1.12	39.03 psf	19.51 psf
			0.00	0.00
			0.00	0.00

Wind Loads on Vertical Truss Chords - Section 3.8 with applied wind velocity

	Left Post	Right Post
Height and Exposure Factor (Kz) =	1.00	1.00
Drag Coef (Cd) =	0.45	0.45
Wind Pressure =	18.91 psf	18.91 psf
Pressure for Group III Load Combination =	12.50 psf	12.50 psf

Longitudinal Reactions at Base of Vertical Truss Chords due to Wind Load

	Left	Right	Group III Load Combination Left	Group III Load Combination Right
(Horizontal Truss)	2664.96 lbs	2664.96 lbs	1332.48 lbs	1332.48 lbs
(Signs)	12371.81 lbs	12371.81 lbs	6185.90 lbs	6185.90 lbs
(One Vertical Chord)	491.29 lbs	491.29 lbs	324.74 lbs	324.74 lbs

Transverse Reactions at Base of Vertical Truss Chords due to Wind Load

	Left	Right
Wind Load Reaction	3007.35 lbs	3007.35 lbs
Reaction for Group III Load Combination	1503.68 lbs	1503.68 lbs

P1=right reaction
P2=left reaction
Distribution of loads due to differential post stiffness.
P2=(PL1^3/I1)/(L2^3/I2+L1^3/I1)
P=P1+P2

Reactions at Base of Controlling Vertical Truss Chord

	Load Case I Left	Load Case I Right	Load Case II Left	Load Case II Right
(Vertical DL)	2758 lbs	2758 lbs	2758 lbs	2758 lbs
Max Vertical Due to Wind	62478 lbs	62478 lbs	37487 lbs	37487 lbs

My (Bending Transv - Dead)	0 f-lb	0 f-lb	0 f-lb	0 f-lb
My (Bending Transv - Wind)	32801 f-lb	32801 f-lb	49201 f-lb	49201 f-lb

Group III Load Combination	Load Case I		Load Case II	
	Left	Right	Left	Right
Max Vertical Due to Wind	31917 lbs	31917 lbs	19150 lbs	19150 lbs
My (Bending Transv - Wind)	16515 f-lb	16358 f-lb	24773 f-lb	24537 f-lb

Section Properties of Vertical Chords	Left	Right
(Area)	11.908 si	11.908 si
(Sx)	649.57 ci	649.57 ci
(Ix)	23953.06 qi	23953.06 qi
(Sy)	29.867 ci	29.867 ci
(Iy)	160.54 qi	160.536 qi
(Mid-thickness radius)	5.193 in	5.193 in

Computed Stresses at Critical Vertical Truss Chord

GROUP I (Dead Load Only)	fa	(fb)y
Left Truss Chord	0.23 ksi	0.00 ksi
Right Truss Chord	0.23 ksi	0.00 ksi

Group II (Dead Load + Wind)

Load Case I	fa	(fb)y	fv
Left Truss Chord	5.48 ksi	13.18 ksi	0.69 ksi
Right Truss Chord	5.48 ksi	13.18 ksi	0.69 ksi

Load Case II	fa	(fb)y	fv
Left Truss Chord	3.38 ksi	19.77 ksi	0.45 ksi
Right Truss Chord	3.38 ksi	19.77 ksi	0.29 ksi

Group III (Dead Load + Ice + 1/2 Wind)

Load Case I	fa	(fb)y	fv
Left Truss Chord	3.09 ksi	6.64 ksi	0.35 ksi
Right Truss Chord	3.09 ksi	6.57 ksi	0.35 ksi

Load Case II

	fa		(fb)y		fv	
Left Truss Chord	1.38	ksi	9.95	ksi	0.23	ksi
Right Truss Chord	1.38	ksi	9.86	ksi	0.23	ksi

Allowable Stresses

	Cc =	126.10			Retrofit Left	0	enter 1 for yes,0 no
				1.33% overstress	Retrofit right	0	
Axial	Fa (left) =	12.92	ksi	17.19 ksi			
	Fa (right) =	12.92	ksi	17.19 ksi	K =	1.44	
Bending	Fby (left)=	23.76	ksi	31.60 ksi	Transverse		
	Fby (right)=	23.76	ksi	31.60 ksi	Left KL/r =	100.43	
					Right KL/r =	100.43	
Shear	Fv (left)=	11.88	ksi	15.80 ksi	Number of Stiffeners left post =	1	
	Fv (right)=	11.88	ksi	15.80 ksi	mber of Stiffeners on Right post =	1	
Euler	Fe' (Left)=	14.80					
Buckling	Fe' (Right)=	14.80					

Combined Stress Ratio of Vertical Truss Chords (AASHTO Luminaires Specifications, Section 5.12.2.1, pg. 5-16)

GROUP I (Dead Load)

	CSR Eq. 5-17	CSR Eq. 5-18	CSR Eq. 5-19	Governing CSR
Left Truss Chord	N/A	N/A	0.02	0.02
Right Truss Chord	N/A	N/A	0.02	0.02

Group II (Dead Load + Wind)

Load Case I

	CSR Eq. 5-17	CSR Eq. 5-18	CSR Eq. 5-19	Governing CSR
Left Truss Chord	0.67	0.98	N/A	0.98
Right Truss Chord	0.67	0.98	N/A	0.98

Load Case II

	CSR Eq. 5-17	CSR Eq. 5-18	CSR Eq. 5-19	Governing CSR
Left Truss Chord	0.78	1.01	N/A	1.01
Right Truss Chord	0.78	1.01	N/A	1.01

Group III (Dead Load + Ice + 1/2 Wind)

Load Case I	CSR Eq. 5-17	CSR Eq. 5-18	CSR Eq. 5-19	Governing CSR
Left Truss Chord	0.35	0.45	N/A	0.45
Right Truss Chord	0.35	0.44	N/A	0.44

Load Case II	CSR Eq. 5-17	CSR Eq. 5-18	CSR Eq. 5-19	Governing CSR
Left Truss Chord	N/A	N/A	0.40	0.40

Right Truss Chord	N/A	N/A	0.39	0.39

Anchor Bolt Check - (Assume Grout Under Baseplate Is Not Effective)

Group II (Dead Load + Wind)

Combination I	fv	ft	Combined Stress Ratio	Anchor Blt Check
Left Post	1.41 ksi	10.83 ksi	0.08	O.K.
Right Post	1.41 ksi	10.90 ksi	0.08	O.K.

Combination II	fv	ft	Combined Stress Ratio	Anchor Blt Check
Left Post	0.93 ksi	18.19 ksi	0.22	O.K.
Right Post	0.93 ksi	18.19 ksi	0.22	O.K.

Allowable Stresses

Fv= 0.3Fy*1.33	23.142 ksi
Ft= 0.5Fy*1.33	38.6 ksi

VALUES REQUIRED FOR STABILITY ANALYSIS

	Left Vertical Truss	Right Vertical Truss
wd =	0.00141 kip/in	0.00141 kip/in
wv =	-0.00344 kip/in	-0.00344 kip/in
w =	7.52 kips	7.52 kips
Pp =	-1560.24 kips	-1560.24 kips

APPENDIX B – DIRECTIONS FOR USING SYSTEM STABILITY ANALYSIS

The system buckling analysis is based on an eigenvalue analysis of the entire structural system first developed by Hartz (9). The method is approximate and requires that the member be divided into multiple elements to achieve acceptable results. This method, as applied in this investigation, produces effective length factors, K, based on the critical element in the truss support. This is one of the lower two truss chord elements, depending on the direction of the wind loading. The software for steel frame stability analysis used in this study is applicable to both in-plane and out-of-plane buckling. It provides for consideration of continuity, different load combinations, and diagonal members that are either pinned to the vertical chord member or rigidly attached to the chord member. The approach is described in more detail in a report by DeWolf and Yang (7).

Using the system stability analysis to calculate the effective length factors for the vertical truss chords requires input from the user. The dimensions of the vertical truss must be input, along with the estimated wind and gravity loads. This portion of the paper explains which values are associated with a given variable for dimensions and loads and guides the user through calculations to retrieve the loads needed by the stability software. Figure 13 shows the variables applied to the structure.

Variables

Dimensions

A_{col} – The cross-sectional area of one vertical truss chord (in^2)
I_{col} – The moment of inertia of one vertical truss chord (in^4)
L_{col} – The height of a vertical truss chord on one support. The height shall be taken as the distance from the base plate to a point just below the bottom chord of the horizontal truss (in)
aa – The vertical elevation of one vertical truss diagonal. This distance represents the height between connections of one diagonal (in)
A_{brace} – The cross-sectional area of one vertical truss diagonal (in^2)
I_{brace} – The moment of inertia of one vertical truss chord (in^4)
Toph – The height of the horizontal box truss. The height is measure from the center of the bottom chord to the center of the top chord in the horizontal box truss (in)
Span – The distance center-to-center of the vertical truss chords (in)

Loads

w_d – Distributed wind loading along one vertical truss chord. (k/in)
w_v – Distributed gravity load of one column (k/in)
w – The concentrated wind load from the top sign box. The wind load must be divided by two before inputting into stability software. (kips)
Pp – The concentrated gravity load from the horizontal truss applied to each

vertical truss chord. Total load from horizontal truss must be divided by two before inputting into stability software. (kips)

The loads are calculated as follows:

$$w_d = P_{vc} \times \left(\frac{1\,\text{ft}^2}{144\,\text{in}^2} \right) \times \left(\frac{1\,\text{kip}}{1000\,\text{lbs}} \right) \times d_{vc} \quad \text{(k/in)}$$

where:

P_{vc} = Pressure due to wind acting on one vertical truss chord (psf)
d_{vc} = Diameter of one vertical truss chord (in)

$$w_v = \frac{-\left(\frac{W_{vd}}{2} + W_{vc} \right) \times \left(\frac{1\,\text{kip}}{1000\,\text{lbs}} \right)}{(h_{vc}) \times \left(\frac{12\,\text{in}}{1\,\text{ft}} \right)} \quad \text{(k/in)}$$

where:

W_{vd} = Total weight of the diagonals on one vertical truss (kips)
W_{vc} = Total weight of a chord on one vertical truss (kips)
h_{vc} = Height of one vertical truss chord (ft)

$$w = \frac{(R_{tr} + R_s) \left(\frac{1\,\text{kip}}{1000\,\text{lbs}} \right)}{2\,\text{chords}} \quad \text{(kips)}$$

where:

R_{tr} = Horizontal reaction at top of vertical truss due to wind acting on the horizontal truss (lbs)
R_s = Horizontal reaction at top of vertical truss due to wind acting on the signs (lbs)

$$Pp = \frac{-(W_{tr} + W_s)}{2\,\text{chords}} \times \left(\frac{1\,\text{kip}}{1000\,\text{lbs}} \right) \quad \text{(kips)}$$

where:

W_{tr} = Weight of horizontal truss acting at top of vertical truss (lbs)
W_s = Weight of signs acting at top of vertical truss (lbs)

Table 1 Axial Stresses in the Vertical Truss Chords due to Dead Load and Wind Load with Wind Fully Applied Perpendicular to the Face of the Sign Panel for Vertical Truss Shown in Figure 13

	Axial Compression Stress (ksi)		
	Due to Wind Load	Due to Dead Load	Dead and Wind
Top of Vertical Truss Chord	0	0.23	0.23
Base of Vertical Truss Chord	5.25	0.23	5.48

Table 2 Out-of-Plane Bending Moments at the base of the Vertical Truss Chords with Dead and Wind Load Fully Applied

Connection at Top of Vertical Truss	Bending Moments in Out-of-Plane Direction (ft-kips)		
	Due to Wind Load	Due to Dead Load	Dead and Wind
Pinned	32.8	0	32.8
Fixed	16.5	34.4	50.9

Table 3 Effective Length Factors, K, Determined With System Stability Analysis

Cross-Sectional Dimensions of Chords In Vertical Truss		K For Out-of-Plane Behavior (Rigid Top Connection)	K For Out-of-Plane Behavior (Pinned Top Connection)	K For In-Plane Behavior
Diameter (inch)	Thickness (inch)			
8.625	0.322	0.87	1.44	0.56
8.625	0.500	0.88	1.44	0.68
8.625	0.875	0.88	1.44	0.84
10.750	0.365	0.88	1.44	0.84
10.750	0.500	0.88	1.45	0.97

Table 4 Design Example Comparisons

Cases	Connection at Top Between Horizontal Truss and Supporting Truss	Wind Pressure	Chord Stiffeners	K values	K out-of-plane	CSR
1	Pinned	Old	No	Assumed	2.00	0.97
2	Pinned	New	No	Assumed	2.00	2.00
3	Pinned	New	Yes	Assumed	2.00	0.71
4	Pinned	New	No	Exact	1.44	1.01
5	Fixed	New	No	Exact	0.88	0.99

Table 5 Influence of Connection Between Horizontal and Vertical Truss on the Capacity

	Connection Between Horizontal and Vertical Trusses	
	Pinned	Fixed
Moment due to Dead Load (kips)	0	34.4
Moment due to Wind Load (kips)	32.8	16.5
K (Out-of-plane)	1.44	0.88
f_b – Actual bending stress (ksi)	13.2	20.4
F_b – Allowable bending stress (ksi)	31.6	31.6
f_a – Actual axial stress (ksi)	5.48	5.48
F_b – Allowable axial stress (ksi)	17.2	23.0
F_e' – Euler buckling stress (ksi)	14.8	39.6
$\left(1-\frac{f_a}{F_e'}\right)$	0.63	0.86
$\frac{f_b}{\left(1-\frac{f_a}{F_e'}\right)F_b}$	0.66	0.75
$\frac{f_a}{F_a}$	0.32	0.24
Governing CSR	1.01	0.99

Figure 1 Typical Overhead Truss-Supported Sign Structure

Figure 2   Typical Horizontal Box Truss

Figure 3   Typical Truss Support

Figure 4   Horizontal Truss Segment Showing Connection Plates

Figure 5   Typical Connection Plate on End of Horizontal Truss Segment

Figure 6 Connection Between Horizontal Box Truss and Vertical Supporting Truss

Figure 7 Detail Showing Typical U-Bolt Used in Connection Between Horizontal Truss and Vertical Supporting Truss

Figure 8   Base Plate Welded to Bottom of Vertical Truss Chord

Figure 9 Connection Detail for Reinforced Truss, Showing Stiffener

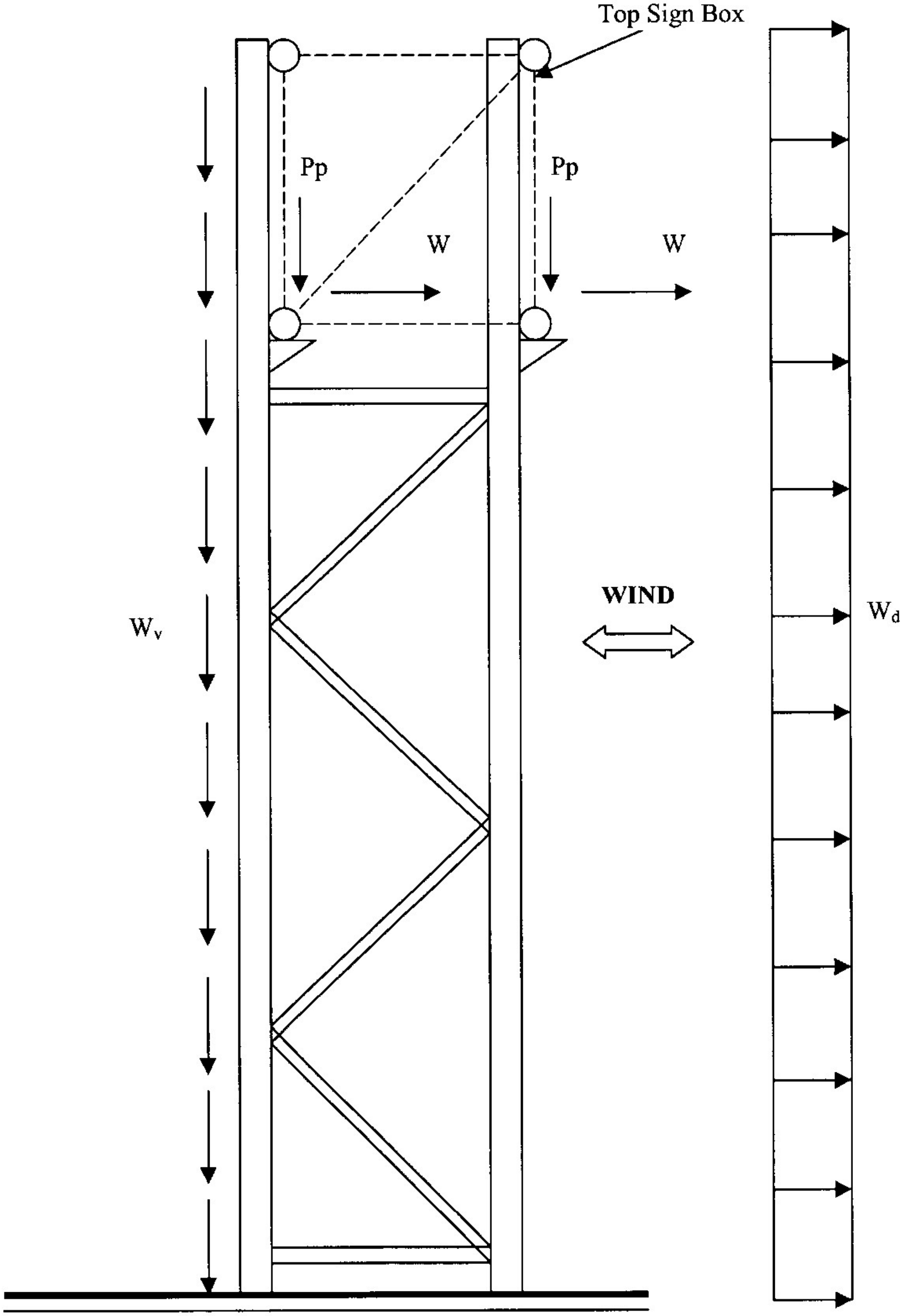


Figure 10   Drawing of Vertical Truss Support Showing all Applied Loads

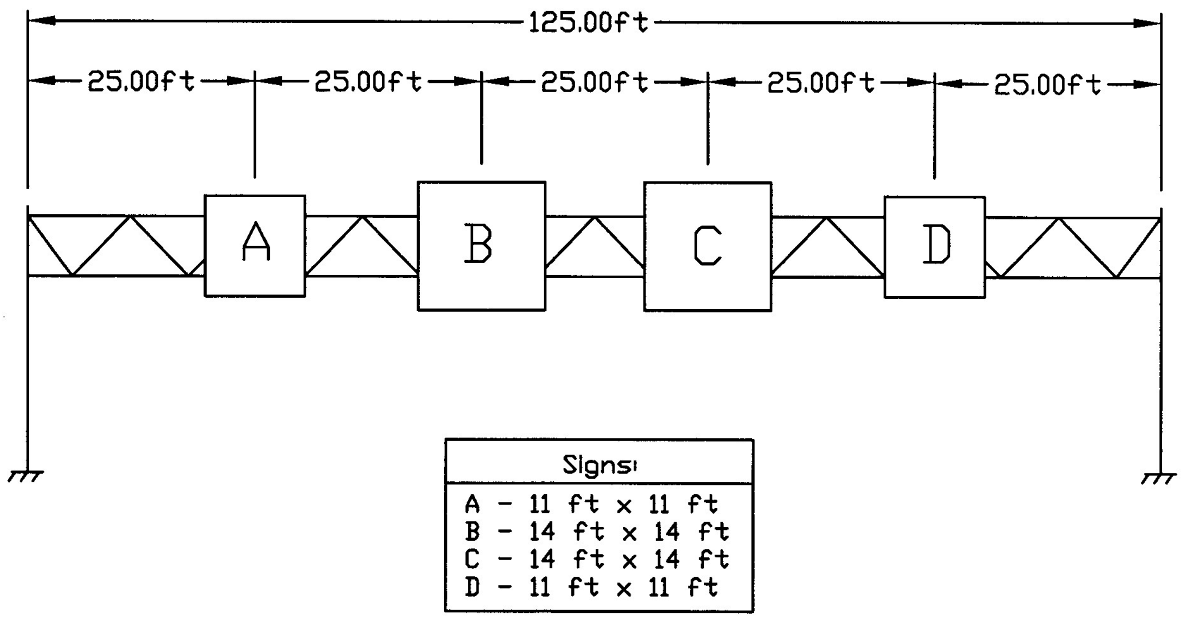


Figure 11 Design Example Sign

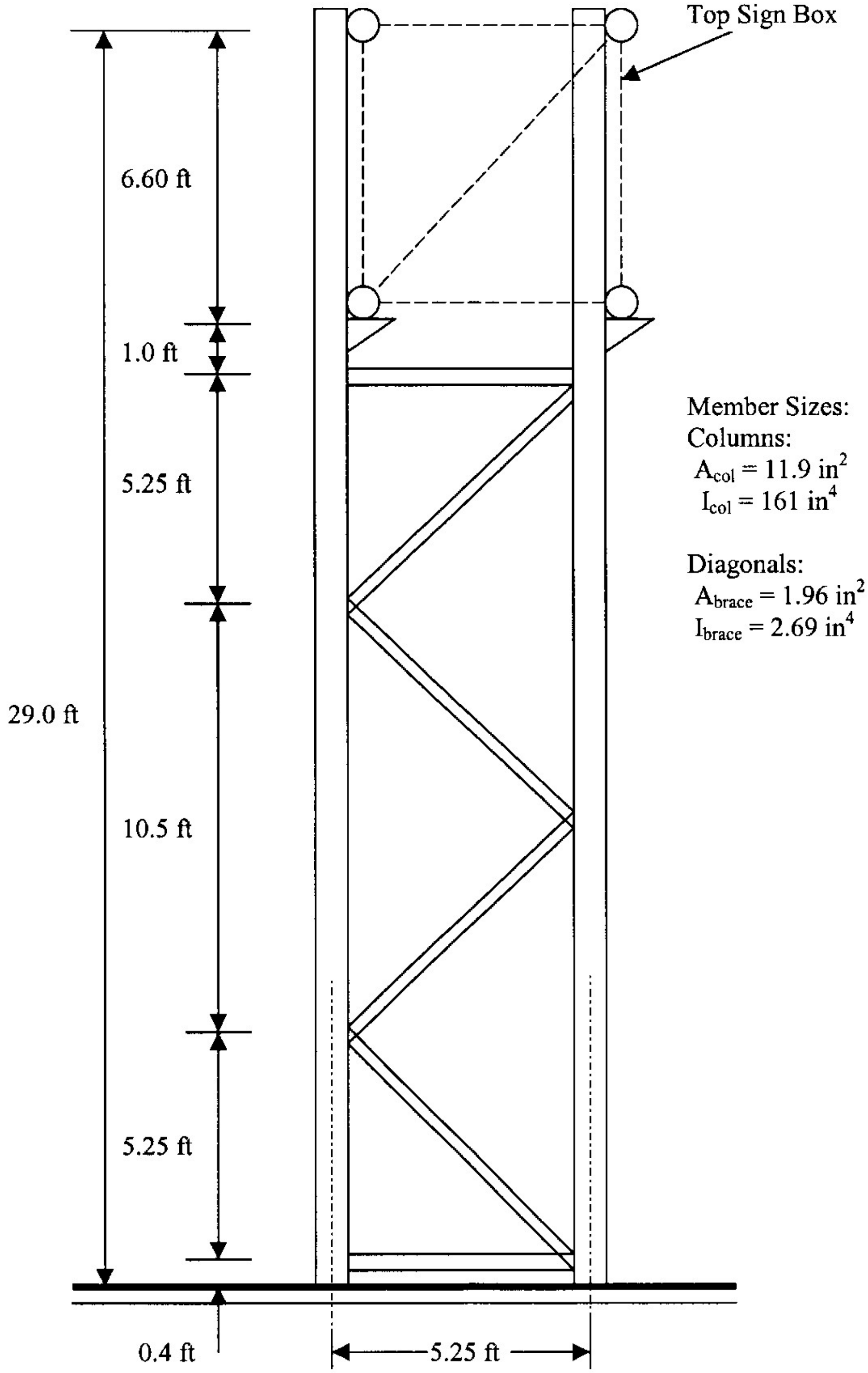


Figure 12   Design Example Truss Support

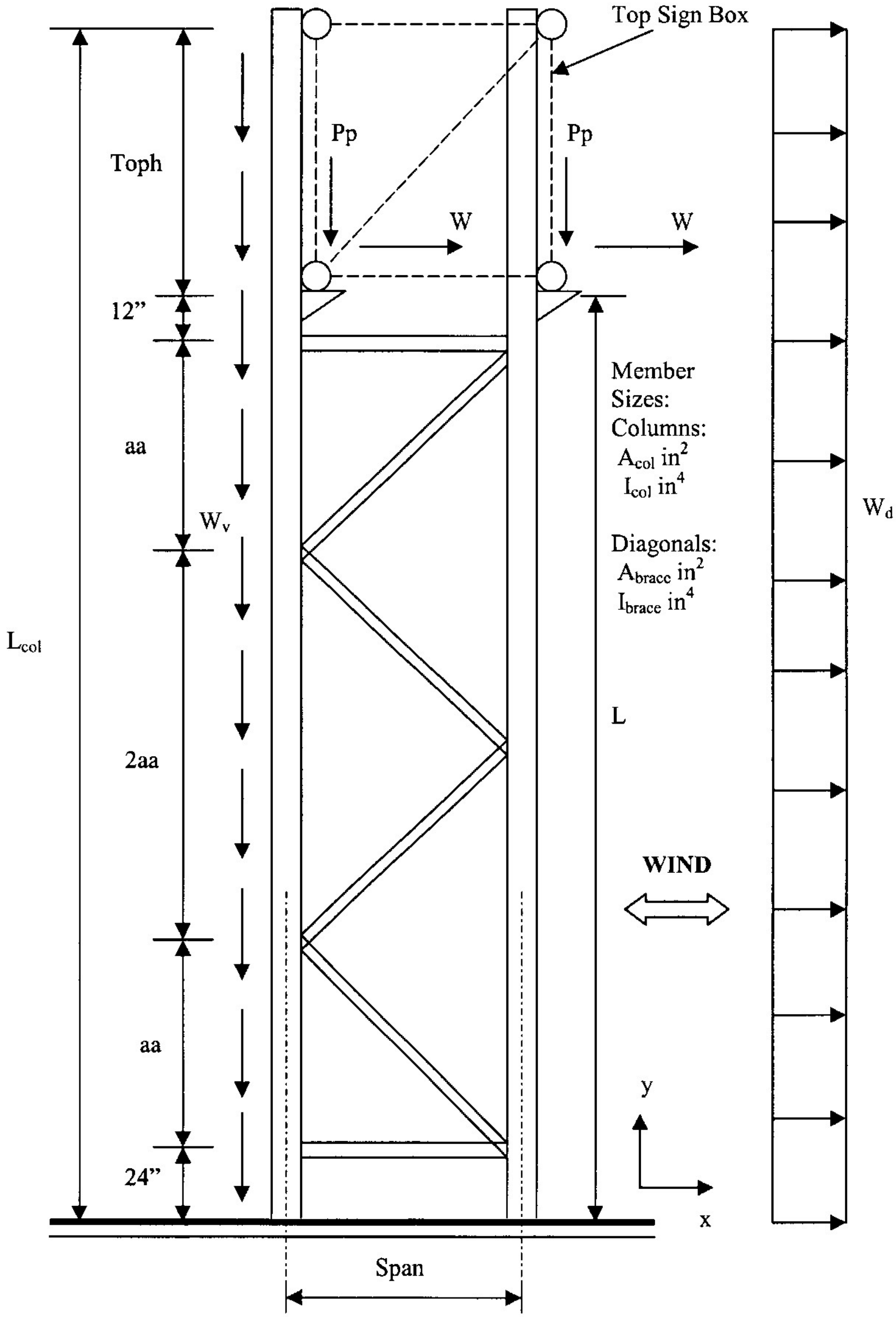


Figure 13 Typical Vertical Support with Variables used for Stability Analysis