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Stability analysis of truss type highway sign support structures

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Abstract. The design of truss type sign support structures is based on the guidelines provided by the American Association of State Highway and Transportation Officials Standard Specifications for Highway Signs, Luminaries and Traffic Signals and the American Institute of Steel Construction Design Specifications. Using these specifications, the column design strength is normally determined using the effective length approach. This approach does not always accurately address all issues associated with frame stability, including the actual end conditions of the individual members, variations of the loads in the members, and the resulting sidesway buckling for truss type sign support structures. This paper provides insight into the problems with the simplified design approach for determining the effective lengths and discusses different approaches for overcoming these simplifications. A system buckling approach, also known as a rational buckling analysis, is used in this study to determine improved predictions for design strength of truss type sign support structures.

Keywords: buckling; sign supports; stability; steel design; trusses; wind loading.

1. Introduction

The design of truss type sign support structures is governed by guidelines provided by the American Association of State Highway and Transportation Officials Standard Specifications for Highway Signs, Luminaries and Traffic Signals (AASHTO 1994) and the American Institute of Steel Constructions Load and Resistance Factor Design (AISC LRFD 1994). The resulting column design strength is normally calculated based on assumptions using the effective length approach. This

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approach does not directly address all issues associated with the determination of the buckling strength of the member. In situations where the behavior of a frame is sensitive to stability effects, the simplified approach can lead to conservative or unconservative estimates of the stability strength.

The current design practice for sign support truss structures includes consideration of the individual members only. The end conditions of the individual member are simplified as either pinned or fixed. The axial load in continuous chord members is normally assumed as constant over the full length. This does not accurately address the variation of the axial load that exists due to the wind loads. This results in an increase in the stability strength since not all elements have the maximum axial force. This is common in many truss configurations when a single chord member is used. The sidesway of the truss in the plane direction is not considered. The influence of the joint rigidity on the bending stiffness for the members is not always accurately modeled, but is instead accounted for with approximate effective length factors.

To properly determine the buckling strength of individual members in sign support structures, the analysis should be focused on the in-plane stability of the overall structural system rather than the in-plane stability of individual members. The sidesway effect should be embedded in the system buckling analysis procedure, along with the load variations and correct determinations of joint rigidities. In the out-of-plane direction for the truss systems, the variation in the load along the chord length also should be included in the evaluation.

There have been recent reports on sign support structures that have collapsed (Cook, *et al*. 1997, Gray, *et al*. 1999, Hartnagel, *et al.* 1999, Kashar, *et al.* 1999). Alampalli looked at the design wind loads (Alampalli 1997). Cook, *et al*. (1997) and Johns and Dexter (1999) studied truck-induced gust wind. Kaczinski (1998), Cook, *et al*. (1999) and Gray, *et al*. (1999) have studied fatigue problems caused by truck-induced vibration. The proposed new sign support specification (Fouad, *et al*. 1998) has recognized that the behavior and strengths of steel tubes used in sign supports is one of the many areas in need of further research work. However, there has not been any research to address the stability problems due to the wind loading. Since there has been an increase in the wind design load, required by the new edition of AASHTO design specification (AASHTO 1998), it is even more important that the stability issue be reviewed. A study of some of the existing signs in Connecticut using the new design specification led to the conclusion that they were not adequate to meet the new requirements. Based on safety considerations, the decision was made to reinforce these signs, even though new specifications do not require retrofitting of existing structures that have been performing well. The goal of this study has been to review the stability of these signs and develop more accurate estimates the stability strength. The replacement of the conservative assumptions on buckling capacities with more realistic estimations should provide a margin that then allows many of the current signs to satisfy the new design wind loads.

This paper reviews the AASHTO guidelines for truss type sign supports and the approaches used for the stability analysis. A system stability analysis is presented to better determine the actual design strength for truss type highway sign support structures. The procedure used is similar to the work done in frame structures by White, *et al.* (1997a). Design recommendations for sign support structures are suggested based on the analytical results in this study.

2. System buckling approach for truss sign supports

There are different design procedures for determining frame stability strength. The isolated subassembly approach is based on consideration of individual elements, with assumptions on end

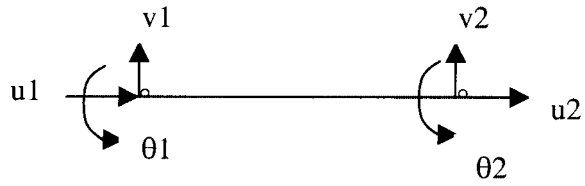

Fig. 1 Stability element with degrees of freedom

conditions. This is typically done with an alignment chart (ASCE 1997, AISC LRFD 1994). The story-buckling approach is based on considerations that the sidesway buckling is a story phenomenon. Both the isolated subassembly approach and the story buckling approach are acceptable for orthogonal building frames (White and Hajjar 1997a). These two approaches include both sidesway and the influence of the stiffness on the end conditions. However, these approaches are not applicable to truss type structures. In these, the diagonals interact with other truss components, and it is not possible to isolate stories. Thus, buckling of the entire truss system is not equivalent to the sidesway buckling of a building frame.

A system buckling approach, also known as a rational buckling analysis, is the most general procedure, with only limited assumptions needed. It has been employed to develop a unified approach for design of orthogonal steel frames (White and Hajjar 1997a). It is also used successfully to study the accuracy and simplicity of different stability design approaches in orthogonal steel frames subject sidesway (White and Hajjar 1997b). The full structural system buckling analysis is the basis of the approach developed in this study.

The system buckling analysis is based on an eigenvalue analysis of the entire structural system. The approach is based on the analysis developed by Hartz (Hartz 1965, Chajes 1974, Chen, *et al*. 1987). The global stiffness matrix is obtained by assembly of element stiffness matrices, which are developed analytically based on finite element interpolations for the displacements. The polynomial displacement function developed by Hartz is used. This approximation requires that the member must be divided into multiple elements to achieve sufficient accuracy. It has been demonstrated in this study that only two or three elements are normally sufficient for each member. Nevertheless, to be safe, each element was divided into five separate elements. The element stiffness matrices were based on an element with six degrees-of-freedom, as shown in Fig. 1. The resulting stiffness matrix includes the first-order elastic stiffness matrix and the geometric stiffness matrix. The full approach is given by DeWolf and Yang (2000).

The method gives a system critical load for the full truss. This is then used to determine an equivalent effective length factor for each truss member. Since the system buckling approach assumes that all members simultaneously reach their individual buckling capacities, the approach can produce overly-conservative designs for members with low axial loads. This is shown in the high effective length factors in the lesser loaded members. Thus, the results are best used for the most critical members, i.e., those with the higher axial loads.

The approach for steel frame stability analysis developed in this study is applicable to both in-plane and out-of-plane buckling. In addition, the system buckling approach can provide for consideration of diagonal members that are either pinned to the vertical column members or rigidly attached to the column members.

3. Design example

A design example for truss type highway sign support structure is presented to illustrate the stability analysis procedure. The assumptions made in the design approach and the design results are presented. The analytical effective length factors are then compared with that of current design practice. The effective length factors can be used with either the AASHTO sign specification or the AISC LRFD guidelines. The general concepts and approach are applicable to either allowable stress design or limit states design.

A typical truss type sign support structure is shown in Fig. 2. It consists of three main parts, the

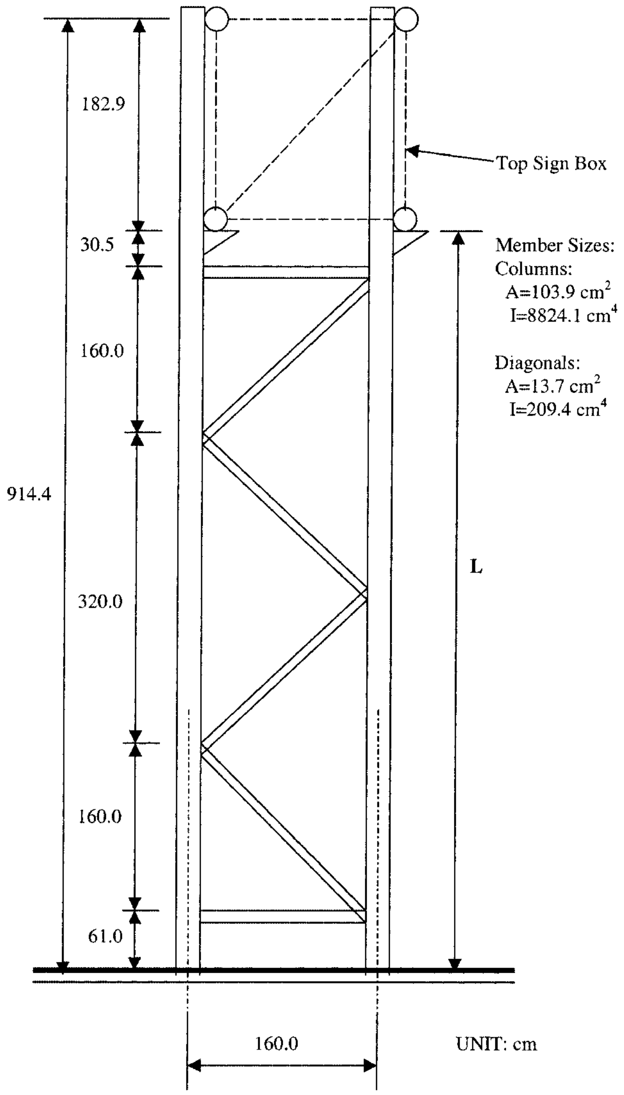

Fig. 2 Truss sign support structure

support columns, diagonal members, and top truss box formed by the sign supporting structure in the perpendicular direction. The ends of the diagonals can be either rigidly connected to the support columns or pinned to the support columns. The truss considered in this design example is based on the trusses used in Connecticut, which have fixed bases. The loads considered in the design are gravity loads (in the vertical direction) and wind loads (in both horizontal directions, in the plane of the truss and perpendicular to the truss). The member sizes are also listed in Fig. 2.

The load combinations needed for design are given by American Association of State Highway and Transportation Officials (AASHTO 1994). Two cases are considered for wind load: (1) 100% wind load normal to the sign panel in the plane of the truss, plus 20% of that load in the perpendicular direction. (2) 60% wind load normal to the sign panel in the plane of the truss, plus 30% of that load in the perpendicular direction. These two loading combinations account for the wind effects in the different directions. The following results are based on these wind loads plus the gravity load, for the structure configuration shown in Fig. 2.

3.1. Current design practice

In the current design approach, the stability behavior is focused on individual members. The effective length factors are based on assumptions on the joint rigidities and sidesway. As a result, simplifications are made on the frame's overall buckling behavior.

The vertical support columns are assumed as fixed at the base, and either pinned or rigidly connected to the top truss box. In the following, the stability behavior is based on the column length L shown in Fig. 2, i.e., the length from the base to the bottom of the sign box. For in-plane behavior, it is not correct to use $K = 1.0$ and a column length equal to the distance between the diagonals. The trusses are slender and thus there is some sidesway. Also, since the diagonals are much smaller than the vertical columns, the joints are not equivalent to pinned joints as normally assumed for trusses. A conservative assumption is to assume that $K = 1.0$, based on the total length L between the base and the top truss box. This does not account for the variation in the axial forces along the members. For out-of-plane behavior, a conservative assumption is to use an effective length factor $K = 2.0$, based on the assumption that there is sidesway in this direction.

The ends of the diagonal members are assumed as rigidly connected to the support columns. For the ideal case with both ends fixed, an effective length factor $K = 0.65$ is recommended in AISC (AISC 1994). Often a value of $K = 0.85$ is assumed in design, allowing for some rotations.

3.2. System buckling analysis approach

3.2.1. In-plane buckling behavior

To study the effects of joint continuity on the structure stability, two models were studied. In the first model, the ends of the diagonals were assumed as rigidly connected to the columns. In the second model, these ends were assumed as pinned to the columns. In both, the columns were continuous over the full height. In the followings, the K values are given for the column subject to compression from the wind loading.

The results for both loading combinations are also shown in Table 1. When the diagonals are rigidly connected to the columns, the effective length factor for the vertical column subject to loading Case (I) is $K = 0.62$, based on the columns length L shown in Fig. 2. For loading case (II),

Table 1 Comparisons of current design practice and system buckling approach results for in-plane behavior

Member	Loading Case in Transverse Directions	Effective Length Factor K		
		Current Design Practice	System Buckling Approach	
			Diagonal Rigidly Connected to the Columns	Diagonals Pinned to the Columns
Columns	(I) 1.0DL+1.0W+0.2W	1.0	0.62	0.63
Columns	(II) 1.0DL+0.6W +0.3W	1.0	0.60	0.61
Diagonals	(I) 1.0DL+1.0W+0.2W	0.85	0.50	1.0
Diagonals	(II) 1.0DL+0.6W +0.3W	0.85	0.50	1.0

the effective length factor is $K = 0.60$. Therefore, case (I) is critical, and the following discussions are based on this loading case. The design assumption that $K = 1.0$ is clearly conservative, and thus the system buckling analysis produces a significant reduction in the effective length factors for the columns. This means that the actual buckling strengths of these column members are larger than those assumed with current design practice. This is primarily due to the fact that the axial load varies along the truss.

There are small differences in the column effective lengths when the diagonals are rigidly connected or pinned to the columns. This is because the stiffnesses of the diagonals are small in relation to the column stiffnesses. When the diagonals are assumed as rigidly connected to the columns, the effective length factor for left support column is $K = 0.60$, that for the right column is $K = 0.62$. These K values are based on wind loading producing compression in the column considered. The difference in the effective length factors for the left and right support columns is due to the locations of the connections of diagonals. When the diagonals are assumed as pinned to the columns, the effective length factor for left support column is $K = 0.61$, that for the right column is $K = 0.63$. Thus, the results show that there is only a slight difference in the effective length factor for the columns when the diagonals are pinned to the columns or rigidly connected to the columns, $K = 0.63$ and $K = 0.62$, respectively. These almost identical effective length factors for the two models imply that the joint continuities between diagonals and columns do not significantly affect the overall buckling strength of the structure. The buckled shape for the truss sign support where the ends of diagonals are rigidly connected to the support columns is shown in Fig. 3.

The effective length factor for the diagonals is $K = 0.50$ when the diagonals are rigidly attached to the columns. This value of 0.5 indicates that the ends are equivalent to fixed ends. This is not surprising considering the large difference in the stiffness of the columns and diagonals. When the diagonals are assumed as pinned to the columns, the effective length factor is $K = 1.0$ as expected.

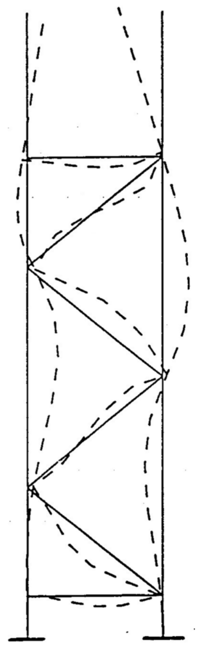

Fig. 3 Buckled shape for truss sign support structure with diagonals rigidly connected to the columns

3.2.3. Out-of-plane buckling due to the in-plane stresses

While the structure is primarily loaded by the in-plane wind forces, the truss may also buckle in an out-of-plane mode. In the current design approach, the effective length factor is assumed to be $K = 2.0$. This is based on assuming that the axial compressive load at the base of the column is the same throughout the full column length. This is in error because it does not treat the wind-induced axial loads properly. Due to wind, the axial force in the column on the compression side is maximum at the base and lowest at the top of the column. This then needs to be combined with the gravity load. Use of the correct axial forces gives K values below 2.0. The results for the truss

Table 2 Comparison of buckling modes: in-plane & out-of-plane behavior

<table>
<tr><th colspan="2" rowspan="2">Segment</th><th rowspan="2">In-Plane Buckling</th><th colspan="2">Out-of-plane Buckling Due to the In-plane Stresses</th></tr>
<tr><th>Top Not Restrained Against Rotation</th><th>Top Restrained Against Rotation</th></tr>
<tr><td>Columns</td><td>Column Length L</td><td>1) Left column: $K = 0.60$
2) Right column: $K = 0.62$</td><td>1) Left column: $K = 1.31$
2) Right column: $K = 1.37$</td><td>1) Left column: $K = 0.83$
2) Right column: $K = 0.82$</td></tr>
<tr><td colspan="2">Diagonals</td><td>$K = 0.50$</td><td colspan="2">----</td></tr>
</table>

Note: The comparison between in-plane and out-of-plane buckling is based on the column length L shown in Fig. 2; for out-of-plane buckling, the diagonals do not buckle.

considered in this investigation are shown in Table 2, assuming that the top is pinned or rigidly connected to the top sign box. The restrained case occurs when the connection between the top sign box and the support columns is sufficient to prevent rotation. As shown, if the top is not restrained, the effective length factors are $K = 1.31$ and $K = 1.37$ for the left and right support columns, respectively. If the top connection is fully restrained, the effective length factor are $K = 0.83$ and 0.82 for the left and right support columns, respectively.

3.3. Consideration of design parameters

The preceding results were based on the design loads described for truss sign support with the dimensions and properties shown in Fig. 2. In this section, the influences of the critical design parameters are discussed. This includes variation in the wind loads, holding gravity loads constant, and variations in the diagonal sizes. This shows how variations in design parameters will quantitatively influence the behavior. For this study, the diagonals were modeled as rigidly connected to the columns.

3.3.1. Wind load variations

For most sign support structures, the column axial loads are primarily due to the wind. For the truss in Fig. 2, the maximum axial compressive force from the wind is approximately twice that from the gravity load. Thus, the wind loading has a significant influence on the overall frame stability.

A study was carried out to see how variations in the wind load influences the resulting K values. The results are shown in Table 3. The first result is for the case with gravity load only. The buckling strength of the structure is high, and the resulting effective length factor for the columns is $K = 0.41$. The effective length factor is increased to $K = 0.62$ when the wind load is applied, as shown in the second set of results. To further study how wind loads influence the behavior, different relative wind loads are applied, shown as loading combinations (3) and (4) in Table 3. When the wind loading is doubled, the column effective length factor $K = 0.63$. When the wind load is decreased to half, the effective length factor for the support column is reduced from $K = 0.62$ to $K = 0.59$. The corresponding decrease in the effective length of the column is 4.8%. Thus, while wind loading significantly reduces the buckling strength, changes in the relative magnitudes do not change the overall buckling strength of the structure significantly.

Table 3 Influence of variations in the wind loading on the structure stability

Loading Combinations	Effective Length Factor K for Columns	Difference in Effective Length Factor (compared with case 1) %
(1) No Wind Loading	0.41	-----
(2) Full Wind Loading	0.62	+51.2%
(3) Double wind loading	0.63	+1.6%
(4) Half wind loading	0.59	-4.8%

Note: In this table, the results are those for the left column with compression due to wind loading; this is the critical column due to the location of diagonals.

Table 4 Influence of variation in the diagonal size on the stability

Diagonal Size	Effective Length Factor K for Columns	Difference in Effective Length Factor (compared with case 1)
(1) A = 13.7 cm^2 I = 209.4 cm^4	0.62	-----
(2) Double the size of diagonal: A = 2(13.7 cm^2) I = 2(209.4 cm^4)	0.43	-30.7% (comparing with case 1)
(3) Triple the size of diagonal: A = 3(13.7 cm^2) I = 3(209.4 cm^4)	0.35	-43.6% (comparing with case 1)

Note: The effective length factors are referred to the column length L shown in Fig. 2.

3.3.2. Diagonal size variations

The relationship between the relative size of the diagonals and the columns can affect the stability behavior of the frame. Analytical results with variations in the diagonal member's sizes are given in Table 4.

If the diagonal sizes are doubled, the effective length factor for the support column is reduced from $K = 0.62$ to $K = 0.43$. The resulting decrease in the effective length of column is approximately 31%. If the diagonal sizes are tripled, the effective length factor for the support column is reduced from $K = 0.62$ to $K = 0.35$. The decrease in the effective length of the column is approximately 44%. Thus, the increasing of the diagonal size can significantly increase the in-plane buckling strength of the support columns and therefore, the buckling strength of the structure. However, this is not an effective way to strengthen the structure when the strength is governed by the out-of-plane buckling mode, since the diagonals have no affect on out-of-plane buckling.

4. Design recommendations

In the design of rigid frames, it is common practice to isolate each member from the frame and design it as an individual beam-column, using beam-column interactive equations. As shown in this paper, the predicted strength of the compression members subjected to wind loading should be determined with an overall stability analysis that includes load variations along the columns, sidesway, and consideration of the actual end connections. This approach requires use of computer software for the stability analysis. The essential design implications from this study are:

(1) The presence of wind loading significantly decreases the overall buckling strength of the structure. However, major changes in the relative magnitude of the wind forces have only a small effect on the overall buckling strength.

(2) The diagonals are normally smaller than the columns. Changing the sizes of the diagonals has a significant influence on the overall column strength for in-plane buckling, but no influence on out-of-plane buckling.

(3) For the out-of-plane buckling mode, the buckling strength of the support column is much

higher when the top connections to the sign box structure rigid connections, thereby restricting the rotations of this joint.

5. Conclusions

In the current design practice for truss type highway sign support structures, the stability behavior of the structure is overly simplified by assuming a concentrated axial load applied to the top of the columns, no sidesway, and idealized end restraints. These assumptions do not provide a true representation of the actual behavior, and thus may lead to excessively conservative designs. This study was carried out because recently modified design provisions involve increased wind loads. Using the simple effective length factors for a revaluation of the capacity indicated that many signs would need to be replaced or modified.

A structure system stability analysis has been used to produce a more accurate evaluation of the truss highway sign support structures. This procedure accounts for sidesway, lateral stability provided by diagonal members, load variations along the columns, and consideration of the actual end restraints. It also can account for non-prismatic members. As shown in this study, a significant reduction in the effective length factors can be achieved for both columns and diagonals, compared with those used in the current design practice. The reduced effective length factors can then be used in the specification design code requirements to account for both linear and non-linear behavior.

The approach also provides guidance on ways to strengthen truss sign supports. As shown, increasing the diagonal sizes can significantly increase the in-plane buckling strength of the structure. However, in cases where the out-of-plane buckling mode governs, it is necessary to increase the rigidity of the connections between the top sign box and the support columns to increase the buckling strength.

Acknowledgements

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Notation

E = Elastic modulus of inertia
I = moment of inertia
K = Effective length factor
K_{system} = Equivalent effective length factor for the member
L = The length of member
P = Axial forces in the member
$P_{e,\,system}$ = Elastic buckling load of the structural system
P_u = Member axial forces resulted from static structural analysis
W = Horizontal wind load
Δ_f = Structure deflection vector at buckling
λ = Critical load parameters
λ_{system} = Lowest critical load parameter

$[K]$ = Structural global stiffness matrix
$[K_e]$ = Structural global elastic stiffness matrix
$[K_G]$ = Structural global geometric stiffness matrix
$[k]$ = Element stiffness matrix
$[k_e]$ = Element elastic stiffness matrix
$[k_g]$ = Element geometric stiffness matrix

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