Stability Analysis of Truss Type Highway Sign Support Structures

Final Report
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John T. DeWolf, Professor
Jun Yang, Graduate Student

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16. Abstract

The design of truss type sign support structures is based on the guidelines provided by American Association of State Highway and Transportation Officials Standard Specifications for Highway Signs, Luminaires 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 report provides insight into the problems with the simplified design approach for determining the effective lengths, discusses different approaches for overcoming these simplifications and presents an approach that can be used to better predict the design strength of truss type sign support structures.

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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

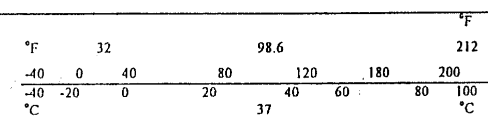
NOTE: Volumes greater than 1000 L shall be shown in m^3

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



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INTRODUCTION

The design of truss type sign support structures is governed by guidelines provided by American Association of State Highway and Transportation Officials *Standard Specifications for Highway Signs, Luminaires and Traffic Signals* (AASHTO, 1994) and American Institute of Steel Construction *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 approach does not directly address all issues associated with the determination of 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 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 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 and the resulting increase in the stability strength that occurs due to the variations in the member load along its length, which is common in truss configurations. The sidesway of the truss in the plane direction is not considered. The influence of the joint rigidity that significantly affects the bending stiffness of members is totally neglected, or accounted for by means of basic adjustments to effective length factors.

To determine properly 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 on 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 of truss systems, the variation in the load should be included in the evaluation.

There have been recent reports on sign support structures that have collapsed (Cook at 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 specifications (AASHTO 1998), it is even more important that the stability issue be reviewed. The result is that many existing signs are not able to meet the new requirements. This study provides an approach to obtain more accurate estimates of the stability behavior to better determine the capacities of the structure.

This report 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 (White and Hajjar, 1997a). Design recommendations for sign support structures are suggested based on the analytical results in this study.

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 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 rectangular shear-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 elements, and it is not possible to isolate stories. Thus the global buckling of the entire truss system is not equivalent to the sidesway buckling of a building frame.

A system buckling approach is the most general procedure, with only limited assumptions needed. It has been employed to develop a unified approach for design of regular steel frames (White and Hajjar, 1997a). It is also used successfully to study the accuracy and simplicity of different stability design approaches for regular steel frames with 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. This type of analysis seeks the lowest value of the load parameter λ_{system}, for which the determinant of the global structure stiffness matrix vanishes, i.e.,

$$\det[K] = 0 \quad \text{..} \quad (1)$$

The approach is based on the analysis developed by Hartz (Hartz 1965). In this procedure, the global stiffness matrix [K] is obtained by assembly of element stiffness matrices, which are developed analytically based on finite element interpolations for the displacements. The element is the typical six degree-of-freedom frame element used in two-dimensional frame analysis, shown in Figure 1, modified to include the first-order elastic stiffness matrix $[k_e]$, and the geometric stiffness matrix, $[k_g]$.

$$[k] = [k_e] + [k_g] \quad \text{..} \quad (2)$$

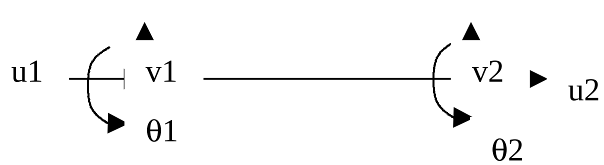


Figure 1. Stability Element with Degrees of Freedom

The element bending stiffness matrix [k_e] is developed from the slope-deflection equations [Chen et al 1987]. The general form of element elastic stiffness matrix [k_e] in local member coordinate system is:

$$[k_e] = \begin{bmatrix} \frac{EA}{L} & 0 & 0 & -\frac{EA}{L} & 0 & 0 \\ & 12\frac{EI}{L^3} & -6\frac{EI}{L^2} & 0 & -12\frac{EI}{L^3} & -6\frac{EI}{L^2} \\ & & 4\frac{EI}{L} & 0 & 6\frac{EI}{L^2} & 2\frac{EI}{L} \\ & & & \frac{EA}{L} & 0 & 0 \\ & symmetry & & & 12\frac{EI}{L^3} & 6\frac{EI}{L^2} \\ & & & & & 4\frac{EI}{L} \end{bmatrix} \quad \text{.................. (3)}$$

For structural stability analysis, both the bending stiffness matrix [k_e] and geometric stiffness [k_g] are needed. The geometric stiffness matrix [k_g] is

$$[k_g] = P\begin{bmatrix} 0 & 0 & 0 & 0 & 0 & 0 \\ & \frac{6}{5L} & \frac{1}{10} & 0 & -\frac{6}{5L} & \frac{1}{10} \\ & & \frac{2L}{15} & 0 & -\frac{1}{10} & -\frac{L}{30} \\ & & & 0 & 0 & 0 \\ & symmetry & & & \frac{6}{5L} & -\frac{1}{10} \\ & & & & & \frac{2L}{15} \end{bmatrix} \quad \text{.. (4)}$$

In this equation, P is the axial force in the element, using a negative value for compression.

The element stiffness matrices are assembled to obtain structural global stiffness matrix [K]. The governing equation obtained is:

$$[K]\Delta_f=0 \quad \text{.......................... (5)}$$

where Δ_f is the deflection vector, and [K]=[K_e]+λ[K_G]. [K_e] is the structure's global linear elastic stiffness based on its original undeformed geometry, and [K_G] is the structure's global geometric stiffness matrix based on the axial force P in each element. At bifurcation, the stiffness [K] of the structure vanishes, i.e.,

$$\det[K_e + \lambda K_G] = 0 \quad \text{.............................. (6)}$$

Thus, the solution of the eigenvalue problem governed by Eq.(6) gives the critical load parameter λ for the structure.

The analytical procedure to obtain the effective length factors through a system buckling analysis is as follows. The procedure begins with the determination of internal forces in each structural member by a conventional structural static analysis. These internal forces are used to construct element geometric stiffness matrices. The next step is to assemble the element elastic stiffness $[k_e]$ and geometric stiffness matrix $[k_g]$ together to obtain the structural global stiffness matrix [K]. The final step is to search for the lowest eigenvalue of [K], which is the critical load parameter λ_{system} of the structural system (ASCE, 1997). The approach need for the determination of the lowest eigenvalue of [K]is presented in Appendix A.

After the critical load parameter λ_{system} is known, the effective length factor K for each member is then calculated as follows. The axial force in the member at incipient buckling of the system model, λ_{system} P_u, is equal to the elastic buckling load $P_{e,system}$ of this member determined with an effective length factor K_{system}, i.e.,

$$\lambda_{system} P_u = P_{e,system} = \frac{\pi^2 EI}{(K_{system} L)^2} \quad \text{.............................} \quad (7)$$

Thus:

$$K_{system} = \sqrt{\frac{\pi^2 EI / L^2}{P_{e,system}}} \quad \text{...................................} \quad (8)$$

In this equation, the member axial force P_u is calculated from a linear elastic structural analysis using the design loading combination. It is commonly assumed that pre-buckling displacements have a negligible effect on the forces within the frame (ASCE, 1997). After the effective length factor K is determined, the design strength of the member is evaluated with the interactive equations specified in the appropriate design codes.

The formulation of the geometric stiffness matrix in Eq. (4) must be based on an assumed displacement function. The polynomial displacement function developed by Hartz is used (Hartz 1965). This approximation requires that the member must be divided into multiple elements to achieve sufficient accuracy. An initial study (Appendix B) shows that three elements are often sufficient. Further comparisons are carried out in the actual design example in this paper.

The procedure used is based on the assumption of elastic behavior. An elastic buckling analysis is generally accepted to give sufficient stability information for the most critical members within a structure (Chen et al, 1987). It should be noted that the design process for the individual members typically includes the consideration of inelastic behavior, based on the use of elastic effective length factors.

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.

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 results are then compared with that of current design practice. The effective length factors determined from Eq.(8) can then 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 state design.

A typical truss type sign support structure, widely used in Connecticut, is shown in Figure 2. It consists of three main parts, the 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 loads considered in the design are gravity loads (in the vertical direction) and wind loads (in the horizontal and perpendicular directions). The member sizes are also listed in Figure 2.

The load combinations, which usually control the 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, with 20% of that load in the perpendicular direction. (2) 60% wind load normal to the sign panel, with 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 loads, for the structure configuration as shown in Figure 2.

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. The frame's overall buckling behavior is not considered. This overly simplifies the actual behavior.

The vertical support columns are assumed as fixed at the base, and pinned to the top truss box. The column length L is shown in Figure 2. These end conditions give an effective length factor K=0.80 (AISC 1994), assuming that the truss fully braces the columns tops against sidesway. Nevertheless, the effective length factor is often conservatively assumed equal to 1.0 in design. Some engineers take a more conservative approach and use an effective length factor K=2.0, assuming the frame to be unbraced against sidesway in the perpendicular 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). For signs, a value of K =0.85 is assumed in design, allowing for some rotations. The resulting effective length factors are then used in the beam-column design equation to check its strength adequacy (AASHTO 1994).

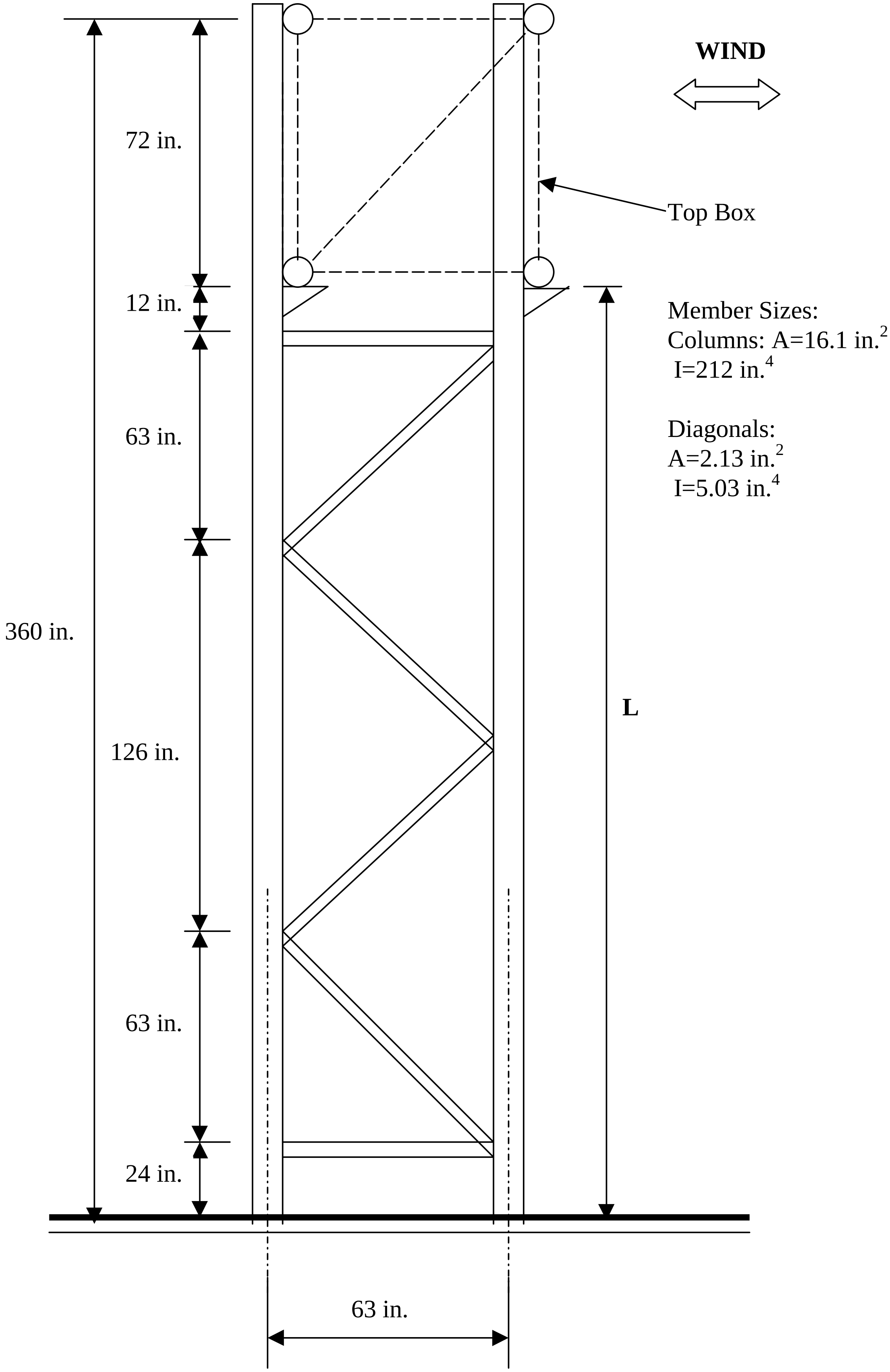


Figure 2. Truss Sign Support Structure

System Buckling Analysis Approach

The first consideration is the determination of the number of elements needed for each segment of the structure for in-plane behavior. Three, four and five elements were used for each of the segments. The results are given in Table 1.

Table 1. System Buckling Analysis Using Different Numbers of Elements for In-Plane Behavior

Number of Elements in Each Member	**System Buckling Analysis Result**		**Difference in the Effective Length Factor (%)**
	Buckling Load Parameter λ_{system}	**Effective Length Factor K for Columns**	
Three	17.83	0.61	----
Four	17.71	0.61	0.0% (comparing with three elements)
Five	17.44	0.62	+1.6% (comparing with four element)

Note: λ_{system} is the buckling load parameter, or the lowest eigenvalue, for the system stiffness matrix; the effective length factor K is based on the column length L shown in Figure 2.

It is concluded that the five-element model gives sufficient accuracy. Therefore, the following discussions are based on the five-element models. While it has been shown that three elements are sufficient for some cases, buckled shapes with changes in curvature require a great number of elements.

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 rigidly connected to the columns. In the second model, these ends were assumed pinned to the columns. In both, the columns were continuous over the full height. The results are given in Table 2.

The results for both loading combinations are also shown in Table 2. When the diagonals are rigidly connected to the columns, the effective length factor for loading

Table 2. 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
	(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
	(II) 1.0DL+0.6W +0.3W	0.85	0.50	1.0

Case (I) is K=0.62, based on the columns length L shown in Figure 2. For loading case (II), the effective length factor is K=0.60. Therefore, case (I) is critical, and the following discussions are based on this loading case. The conservative design assumptions for the columns now used are based on using K=1.0 for support columns, and thus, using 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 stronger than those assumed with current design practice.

There are small differences in the column effective lengths when the diagonals are rigidly connected or pinned to the columns. When the diagonals are assuming 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. The difference in the effective length factors for the left and right support columns is due to the locations of the connections of diagonals. The effective length factor for the diagonals are K=0.50 when the diagonals were rigidly attached to the columns. The value of 0.5 indicates that effectively the ends of diagonals are equivalent to fixed ends. This is not surprising consid-ering the large difference in the stiffness of the columns and diagonals. When the diagonals are assuming 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. For this case, the diagonal effective length factor is K=1.0. Thus, as the results show, 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 Figure 3.

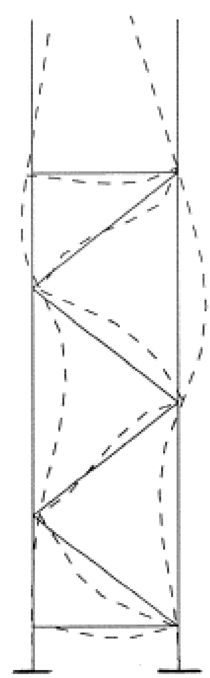


Figure 3. Buckled Shape for Truss Sign Support Structure with Diagonals Rigidly Connected to the Columns

It is noted that all previous discussions are based on the effective length factors for the most critical segment in the column. As described before, the calculation of the effective length factors by the system buckling approach is based on Eq. (8). For a structural system, the axial load varies along the major chord segments. The values of K given are based on the lowest segment in which the axial forces are largest, due to wind. The higher segments with lower axial forces will have larger effective length factors, as listed in Table 3.

Table 3. Comparison of Buckling Modes: In-Plane and Out-of-Plane Behavior

Segment		**In-Plane Buckling**	**Out-of-plane Buckling Due to The In-plane Stresses**	
			Top Not Restrained Against Rotation	**Top Restrained Against Rotation**
Columns	Column length L	1). Left column: K=0.60 2). Right column: K=0.62	1). Left column: K=1.31 2). Right column: K=1.37	1).Left column: K=0.83 2). Right column: K=0.82
Diagonals	Diagonals	K=0.50	----	
	Horizontal Diagonal	K=6.46	----	

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

For these members, the design is controlled by the critical segment.

Out-of-Plane Buckling Due to the In-Plane Stresses

While the structure is primarily loaded by in-plane 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=1.0. This is based on assuming that the axial compressive load at the base of the column is applied at the top of the column. 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, zero at the top of the column, varying along its height. The comparison of the in-plane and out-of-plane behavior is shown in Table 3. The results are based on two possibilities. In both, the column was analyzed assuming the bottom end was fixed. The top, at the joint with the top box was assumed either restrained against rotation, or totally free. 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 factors are K=0.83 and 0.82 for the left and right support columns, respectively.

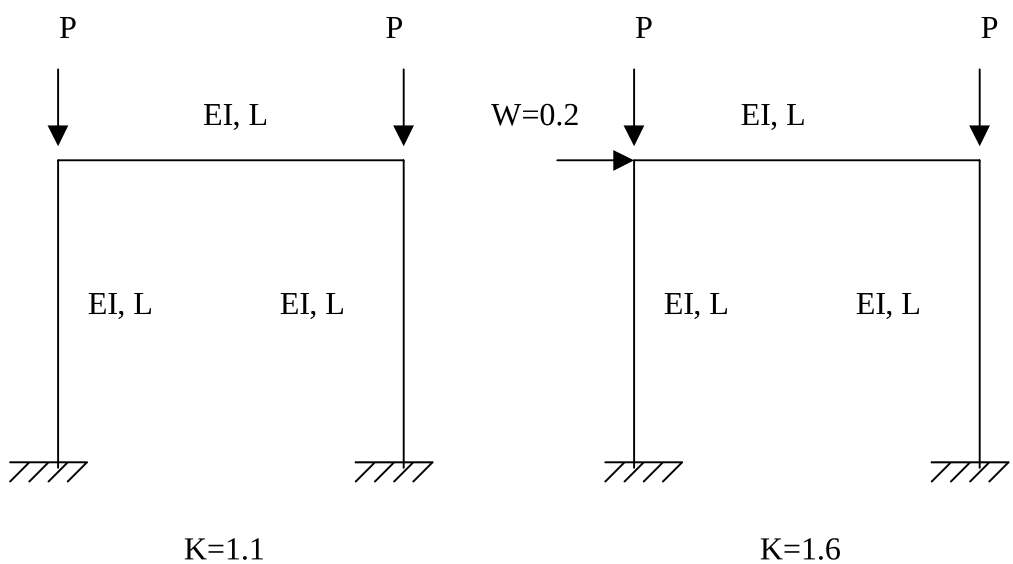
Consideration of Design Parameters

The preceding results were based on the design loads described for truss sign support with the dimensions and properties shown in Figure 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.

Wind Load Variations

For most sign support structures, the column axial load is primarily due to the wind. For the truss in Fig. 2, the design axial force from the wind is twice that from the gravity load. The overall frame stability is decreased significantly by the presence of the horizontal wind loading.

The first set of results in Table 4 is based on 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. This decrease is consistent with what has been found for frames. A simple frame is shown in Figure 4.



(a). Gravity Load Only
(b). Gravity Load Plus Horizontal Load

Figure 4. Example Showing How Horizontal Load Influences Frame

Stability

When horizontal loading is not included, the effective length factor K for the column is 1.16 (Chajes, 1974). When a small amount of horizontal loading W=0.2P is applied, the effective length factor K is increased to 1.65 (Horn, 1965). It is shown that the presence of a small horizontal load greatly decreases the buckling strength of the frame structure.

To further study how wind loads influence the behavior, different relative wind loads are applied, shown as loading combinations (3) and (4) in Table 4.

Table 4. Influence of Variations in the Wind Loading on the Structure Stability

Loading Combinations	**System Buckling Analysis Results**		**Difference in the Effective Length Factor (%)**
	Buckling Load Parameter λ	**Effective Length Factor K for Columns**	
(1) No Wind Loading	59.23	0.41	-----
(2) Full Wind Loading	26.02	0.62	+51.2% (comparing with case 1)
(3) Double wind loading	24.84	0.63	+1.6% (comparing with case 2)
(4) Half wind loading	28.36	0.59	-4.8% (comparing with case 2)

NOTE: *In this table, only the right wind is applied and the results are shown, as the right column is more critical than the left column due to the location of diagonals.*

As presented in the Table 4, under design factored wind loading, the effective length factor for the column is K=0.62. When the wind loading is doubled, the column effective length factor became 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.

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 5.

Table 5. Influence of Variation in the Diagonal Size on the Structure Stability

Diagonal Size	System Buckling Analysis Results		Difference in the Effective Length Factor (%)
	Buckling Load Parameter λ	Effective Length Factor K for Columns	
(1) A = 2.13 $in.^2$ I = 5.03 $in.^4$	26.02	0.62	-----
(2) Double the size of diagonal: A=2(2.13 $in.^2$) I = 2(5.03 $in.^4$)	53.08	0.43	-30.7% (comparing with case 1)
(3) Triple the size of diagonal: A=3(2.13 $in.^2$) I = 3(5.03 $in.^4$)	79.67	0.35	-43.6% (comparing with case 1)

NOTE: *The effective length factors are referred to the column length L shown in Figure 2.*

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, as the change of the diagonal sizes does not affect the out-of-plane buckling.

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 report, 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 limited 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 are restricted against rotation.

Strengthening of Sign Support Structures

The parametric study with variations in the connection rigidity, variations in the wind load, and variations in the diagonal sizes, provides information on the ways to strengthen existing sign support structures.

As shown, the in-plane buckling strength of the structure can be increased by increasing the diagonal size. However, for some structures, the design may be governed by the out-of-plane buckling mode. In this case, it is necessary to increase the out-of-plane buckling strength for a stronger structure. Solely increasing the diagonal size can increase the in-plane buckling strength. However, this will not change the out-of-plane buckling mode.

The out-of-plane buckling strength can be increased by increasing the restraint of the connection between the top of the columns and the sign box structure. If the connection between top box and column is pinned, the effective length factor is K=1.37. If it is fully restricted against rotation, the effective length factor is K=0.82. Therefore, increasing the rigidity of the connections at the top box increases the capacity.

SUMMARY AND 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 design. As a result, many sign structures have been replaced because conservative calculations indicated unsafe conditions. A more accurate analysis can give a different result, thereby saving costly replacements.

A structure system stability analysis is presented for truss highway sign support structures. This procedure accounts for sidesway, lateral stability provided by diagonal members, load variations along the columns, and considerations of actual end restraints. It also can account for non-prismatic member. As shown in this study, a significant reduction in the effective length factors can be achieved for both columns and diagonals, compared with those ones used in the current design practice.

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.

Unlike other types of civil engineering structures, wind loading is the dominant load for highway sign supports. Its presence significantly decreases the structural stability strength. Without proper consideration of the effect of wind loading on the structure stability, the design will be conservative, often significantly so. For this reason, the designer should conduct a full stability analysis for the given structure under a specific design loading to determine the proper buckling strength of the structure. This approach should also be used when evaluating existing structures, so structures are not replaced unnecessarily.

As shown in this report, the analysis procedure requires matrix manipulations that in turn require use of computer software. Software can be developed as outlined in this report. As an alternative, the stability approach could be incorporated into commercial software. The approach, developed for signs, can also be used to determine global stability for other applications involving trusses.

REFERENCES

Allen, H. G., and Bulson, P. S.(1980). “Background to Buckling.” McGraw-Hill, New York, NY.

Alampalli, S. (1997). “Wind Loads on Untethered-Span-Wire Traffic-Signal Poles.” Report FHWA/NY/SR-97/126, Transportation Research and Development bureau, New York State Department of Transportation.

American Association of State Highway and Transportation Officials. (1998). “Standard Specifications for Structural Supports for highway Signs, Luminaries, and Traffic Signals.” AASHTO, Washington, D. C.

American Association of State Highway and Transportation Officials. (1994). “Standard Specifications for Structural Supports for highway Signs, Luminaries, and Traffic Signals.” AASHTO, Washington, D. C.

American Institute of Steel Construction. (1994). “Manual of Steel Construction - Load & Resistance Factor Design”, second edition, American Institute of Steel Construction, Inc., Chicago, IL.

American Society of Civil Engineers . (1997). “Effective Length and Notional Load Approaches for Assessing Frame Stability: Implications for American Steel Design.” Task Committee on Effective Length, ASCE, New York, NY.

Bathe, K. (1982). “Finite Element Procedures in Engineering Analysis.” Prentice Hall, Englewood Cliffs, NJ.

Beskos, D.E. (1977). “Frame Work Stability By Finite Element Method.” Journal of the Structural Division, Proceedings of The American Society of Civil Engineers, Vol. 103, No. ST 11, Nov. 1977, pp. 2273-2276.

Chajes, A. (1974). “Principles of Structural Stability Theory.” Englewood Cliffs, N. J., Prentice-Hall.

Chen, W. F. and Lui, E. M. (1987). “Structural Stability: Theory and Implementation.” Elseiver Science Publishing Co., Inc., New York, NY, 1987.

Cook, R. A., Bloomquist, D., and Kalajian, M. A. (1999). “Mechanical Damping System for Mast Arm Traffic Signal Structure.” Proceedings of the 1999 Structures Congress: Structural Engineering in The 21st Century, April 18-21, 1999, New Orleans, Louisiana, pp. 1099-1102.

Cook, R. A., Bloomquist, D., and Agosta, A. M. (1997). “Truck-induced Dynamic Wind Loads on Variable-message Signs.” Transportation Research Record: No. 1594: Bridges, Other Structures, and Hydraulics and Hydrology, Transportation Research Board, Washington, D. C., pp. 187-193.

Fouad, F. H., Calvert, E. A., and Nunez, E. (1998). "Structural Supports for Highway Signs, Luminaires, and Traffic Signals." NCHRP Report 411, Transportation Research Board, National Research Council, Washington, D. C.

Golub, G. H., and Van Loan, C. F. (1989). "Matrix Computations." 2nd edition, Johns Hopkins University Press.

Gray, B., Wang, P., Hamilton, H. R., and Puckett, J. A. (1999). "Traffic Signal Structure Research at University of Wyoming." Proceedings of the 1999 Structures Congress: Structural Engineering in The 21st Century, April 18-21, 1999, New Orleans, Louisiana, pp. 1107-1110.

Hancock, G. J. (1984). "Structural Buckling and Vibration Analyses on Microcomputers." Civil Engineering Transactions, Institute of Engineers Australia, Vol. CE 26, No. 4, pp. 327-331

Hartnagel, B. A., and Barker, M. G. (1999). "Strain Measurements on Traffic Signal Mast Arms." Proceedings of the 1999 Structures Congress: Structural Engineering in The 21st Century, April 18-21, 1999, New Orleans, Louisiana, pp. 1111-1114.

Hartz, B. J. (1965). "Matrix Formulation of Structural Stability Problems." Journal of the Structural Division, Proceedings of The American Society of Civil Engineers, Vol. 91, No. ST 6, Dec. 1965, pp. 141-157.
Horne, M. R., and Merchant W. (1965) "The Stability of Frames" First Edition, Pergamon Press Ltd., Headington Hill Hall, Oxford, England.

Johns, K. W., and Dexter, R. J. (1999). "Truck-induced Loads on Highway Sign Support Structures." Proceedings of the 1999 Structures Congress: Structural Engineering in The 21st Century, April 18-21, 1999, New Orleans, Louisiana, pp. 1103-1106.

Kaczinski, M. R., Dexter, R. J., and Van Dien, J. P. (1998). "Fatigue-Resistant Design of Cantilevered Signal, and Light Supports." NCHRP Report 412, Transportation Research Board, National Research Council, Washington, D. C.

Kashar, L., Nester M.R., Jones, J.W., Hariri, M., and Friezner, S. (1999). "Analysis of the Catastrophic Failure of the Support Structure of A changeable Message Sign." Proceedings of the 1999 Structures Congress: Structural Engineering in The 21st Century, April 18-21, 1999, New Orleans, Louisiana, pp. 1115-1118.

White, D. W., and Hajjar, J. F. (1991). "Application of Second-Order Elastic

Analysis in LRFD Research to Practices." Engineering Journal, AISC, Vol. 28, No. 4, pp. 133-148.

White, D. W., and Hajjar, J. F. (1997a). "Buckling Models and Stability Design of Steel Frames: A Unified Approach." Journal of Constructional Steel Research, Vol. 42, No. 3, June, pp. 171-207.

White, D. W., and Hajjar, J. F. (1997b). " Accuracy and Simplicity of Alternative Procedures for Stability Design of Steel Frames." Journal of Constructional Steel Research, Vol. 42, No. 3, June, pp. 209-261.

APPENDIX A. SYSTEM BUCKLING ANALYSIS – COMPUTATIONAL METHOD

The determination of critical load is based on the solution of:

$$\det([K_e] - \lambda_{system}[K_G]) = 0 \quad \text{.....................} \quad (A1)$$

This is a linear eigenvalue problem. Solution techniques are given by Bathe (1982), Golub et al (1989), and Hancock (1984). Following is just a brief description of the solution technique used in this study to obtain the eigenvalues of a real, non-symmetric matrix.

For a real, non-symmetric matrix, a modified QR algorithm is used to obtain all eigenvalues. The QR designation is based on the matrix type, with Q applying to the orthogonal matrix and R applying to the upper triangular matrix. There are two intrinsic properties in non-symmetric matrices. First, the eigenvalues of a non-symmetric matrix can be very sensitive to small changes in the matrix elements. Second, the matrix itself can be defective, so that it is not possible to determine a complete set of eigenvalues. Thus numerical procedures must be selected carefully. Great effort is needed to obtain an accurate, stable solution.

The sensitivity of eigenvalues to rounding errors during the execution of QR algorithm is reduced through a procedure of balancing. The original matrix is replaced by a balanced matrix with identical eigenvalues. The matrix is further reduced to a simpler form based on a procedure developed by Hessenberg (Bathe 1982). The QR method is used to find all eigenvalues of the system. A FORTRAN code has been written for the stability analysis in this study based on above derivations for conducting system buckling analysis. The verifications are presented in Appendix B.

APPENDIX B. VERIFICATIONS

The validity of the computer program developed in this study is tested with various structures under various loading. This also provides information on the number of elements needed for each member in the stiffness matrix. Those test cases are shown in Figure Appendix B1 and described as following. The solutions are listed in Table Appendix B1.

Verification Case (a) - Single Column with Pinned Ends

As a simple check of the program, a single column with pinned ends was modeled using one element, two elements, three elements, and four elements. The computed solutions for effective length factors are 0.91, 1.00, 1.00, and 1.00, and the critical buckling load parameters are 12.00, 9.94, 9.89, and 9.87, correspondingly. The exact solution is λ_{cr}= 9.87, and K=1.00.

Thus, it is shown that using a single element model yields greatest error, and increasing the number of elements yields better results. A three element model results in an error of 0.16%. This error is reduced to 0.11% with a four element model.

Verification Case (b) - A Triangular Frame Structure with Pinned Bases

The second verification case is a triangular frame type structure with pinned bases and one concentrated load applied at top (as shown in Figure App.B1. (b)). The theoretical exact solution is λ_{cr}=27.31, and K=0.60 (Beskos, 1977). The predicted values are λ_{cr}=38.11, K=0.51, λ_{cr}=27.63, and K=0.60, λ_{cr}=27.38, K=0.60, and λ_{cr}=27.32, K=0.60 for models with 1-element, 2-element, 3-element, and 4-element, respectively. As shown in the Table App. B1, the three-element model yields a predicted buckling load with an error only 0.25%, and felt to be sufficiently accurate.

Verification Case (c) - A Rectangular Frame Type Structure with Pinned Bases

The third verified case is a rectangular frame type structure with pinned bases, two vertical loads and one diagonal bar (Figure App. B1.(c)). Again, the structure was modeled using 1-element, 2-element, 3-element, and 4-element for each member. Correspondingly, the predicted buckling loads are λ_{cr}=34.33, K=0.54, λ_{cr}=20.77, and K=0.69, λ_{cr}=20.64, K=0.69, and λ_{cr}=20.61, K=0.69. The theoretical exact solution is λ_{cr}=20.54, K=0.69 (Beskos, 1977). Again, using three-element for each member yields a solution with an error only 0.49%. This is felt to be sufficient.

Verification Case (d) - A triangular frame structure with pinned ends

A triangular frame structure with pinned ends and multiple loads is shown in Figure App. B1 (d). The exact solution is λ_{cr}=1.69, K=0.73 (Beskos, 1977). The structure was modeled using 1-element, 2-element, 3-element and 4-element for each member. Correspondingly, the predicted buckling loads are λ_{cr}=2.41, K=0.61, λ_{cr}=1.75, K=0.71, λ_{cr}=1.71, K=0.73, and λ_{cr}=1.70, K=0.73. The calculation results are listed in Table App.B1.

The validity of the program is substantiated by the test cases. The convergence of the program is properly achieved using more elements for each member. The approach provides critical load that is slighter larger than the exact values. Sufficient accuracy can be achieved by using three or more elements for each member. Additional elements are needed if more than one inflection point occurs in the member's buckled shape. Other researchers have also observed same phenomenon (Allen and Bulson, 1980; White and Hajjar, 1991). It is shown that the use of three elements per member is sufficient to achieve solution within one percent of the exact in normal cases.

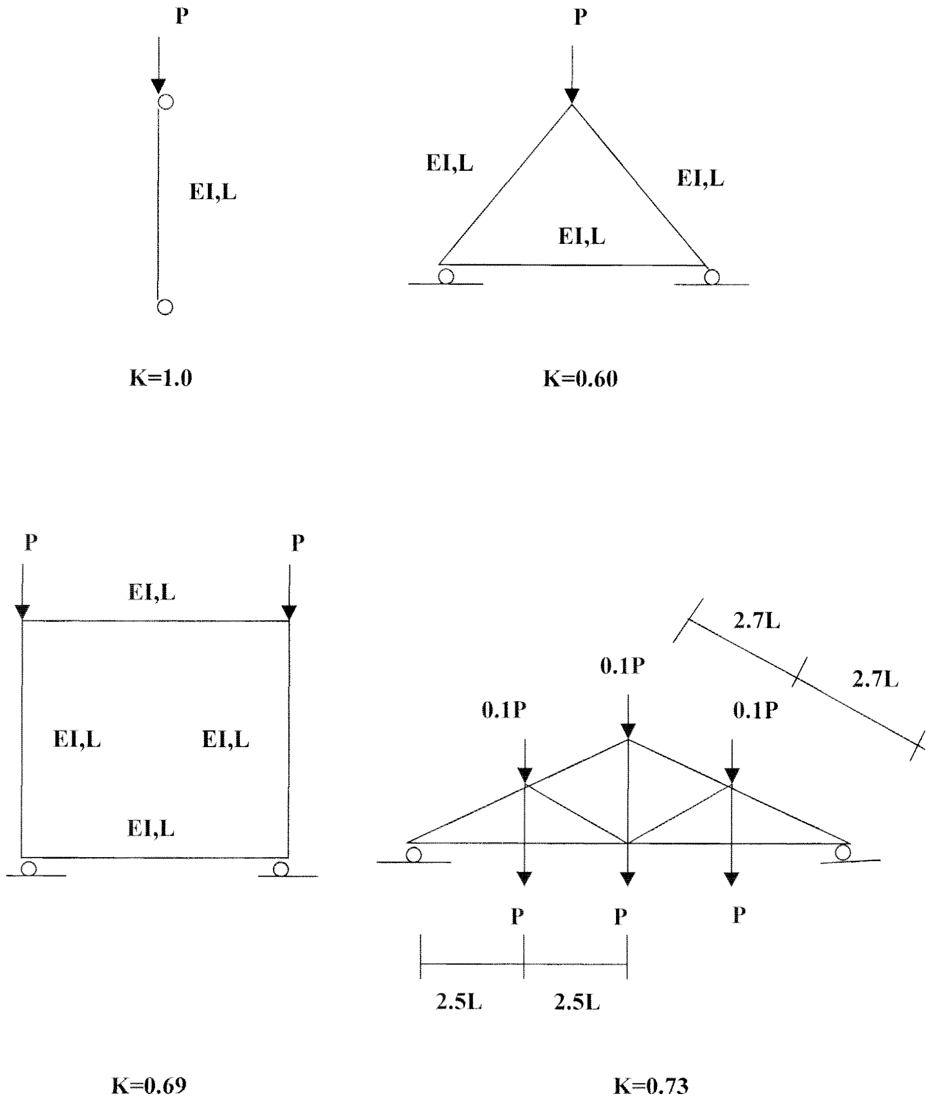


Figure Appendix B1. Verification Cases

Table Appendix B1.
Verifications: Critical Load by System Buckling Approach

Type of Structure		Finite Element Method: Buckling Load Parameter λ	Finite Element Method: Effective length Factor K	Error (%)	Exact Solution
a). Single column with pinned ends	1-element model	12.00	0.91	+21.59%	λ=9.87 K_e =1.00
	2-element model	9.94	1.00	+ 0.75%	
	3-element model	9.89	1.00	+ 0.16%	
	4-element model	9.87	1.00	+ 0.05%	
b). Triangular Framework	1-element model	38.11	0.51	+ 39.54%	λ=27.31 K_e =0.60
	2-element model	27.63	0.60	+ 1.18%	
	3-element model	27.38	0.60	+ 0.25%	
	4-element model	27.32	0.60	+ 0.04%	
c). Rectangular Framework	1-element model	34.33	0.54	+ 67.14%	λ=20.54 K_e = 0.69
	2-element model	20.77	0.69	+ 1.12%	
	3-element model	20.64	0.69	+ 0.49%	
	4-element model	20.61	0.69	+ 0.34%	
d). Triangular Framework	1-element model	2.41	0.61	+ 42.60%	λ = 1.69 K_e=0.73
	2-element model	1.75	0.71	+ 3.55%	
	3-element model	1.71	0.73	+ 0.18%	
	4-element model	1.70	0.73	+ 0.12%	