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Practical fatigue/cost assessment of steel overhead sign support structures subjected to wind load

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Abstract. Overhead sign support structures number in the tens of thousands throughout the trunk-line roadways in the United States. A recent two-phase study sponsored by the National Cooperative Highway Research Program resulted in the most significant changes to the AASHTO design specifications for sign support structures to date. The driving factor for these substantial changes was fatigue related cracks and some recent failures. This paper presents the method and results of a subsequent study sponsored by the Michigan Department of Transportation (MDOT) to develop a relative performance-based procedure to rank overhead sign support structures around the United States based on a linear combination of their expected fatigue life and an approximate measure of cost. This was accomplished by coupling a random vibrations approach with six degree-of-freedom linear dynamic models for fatigue life estimation. Approximate cost was modeled as the product of the steel weight and a constructability factor. An objective function was developed and used to rank selected steel sign support structures from around the country with the goal of maximizing the objective function. Although a purely relative approach, the ranking procedure was found to be efficient and provided the decision support necessary to MDOT.

Keywords: overhead sign support; cantilever; fatigue; wind load; steel design.

1. Introduction

Recent fatigue-related problems in overhead sign support structures (Kaczinski, *et al.* 1998, Dexter and Ricker 2002), particularly cantilever structures, prompted the American Association of State Highway Transportation Officials (AASHTO) to update the Standard Specification for Structural Supports for Highway Signs, Luminaires, and Traffic Signals in 2001 (AASHTO 2001). An earlier version of the specification was published in 1994 and according to recent studies, did not adequately address the issue of fatigue. Because U.S. state Departments of Transportation are "deemed to comply" to the new specification, this change in standard specifications will result in

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the need for states to check and/or modify existing structural designs. This paper presents the method that was developed for the Michigan Department of Transportation to help them identify sign support structures that were likely to meet these new requirements. Once the technique presented herein is applied the structural supports must still be checked for direct compliance with the specification since the approach is a relative measure of cost and performance. The approach presented in this paper has two distinct advantages over performing a design check. The first is the ability to weight both fatigue life and cost, rather than simply provide a yes or no answer to compliance. The second is that this method is extensible to any type of structure and if problem-specific details are worked out it may be able to provide a combined qualitative/quantitative decision assistance tool.

Overhead sign support structures are subjected to loads caused by natural wind gusts and truck gusts. The latter of these is typically a problem only for luminaires, often resulting in vortex-induced vibrations. Wind gusts acting on the sign portion of the structure cause significant load effects in the bolted and welded connections as well as the members themselves. This paper presents the method and results of a study that developed and applied an approximate technique to identify overhead sign support structures that are most likely to meet the new 2001 AASHTO specifications. The method uses utility functions, a well-known technique in decision theory and analysis, to rank cantilever and bridge-type overhead sign support structures. The performance metric is the approximate fatigue life as computed using Crandall and Mark's (1963) approach whose basis is the Palmgren-Miner rule. Specifically, the fatigue life is estimated by assuming a free vibration duration and the statistics of the stress time history computed for use in the Crandall and Mark approach. Although galloping is a common problem for overhead signs it is not included in the analysis. It was felt that structural supports identified and ranked based on wind loads only would provide a sound basis for helping justify full design checks of certain structural supports. The cost metric is modeled as the product of the steel weight and a constructability factor ranging from 0.9 to 1.2, which will be described in detail later.

The combination of the performance measure with cost to form a composite metric is presented for two different combinations of performance and cost which includes (1.) Neglecting cost entirely, and (2.) Cost providing 25% weighting in the decision. In order to determine a starting point for selection of the suite of overhead sign support structures to analyze fully, a brief survey was sent to

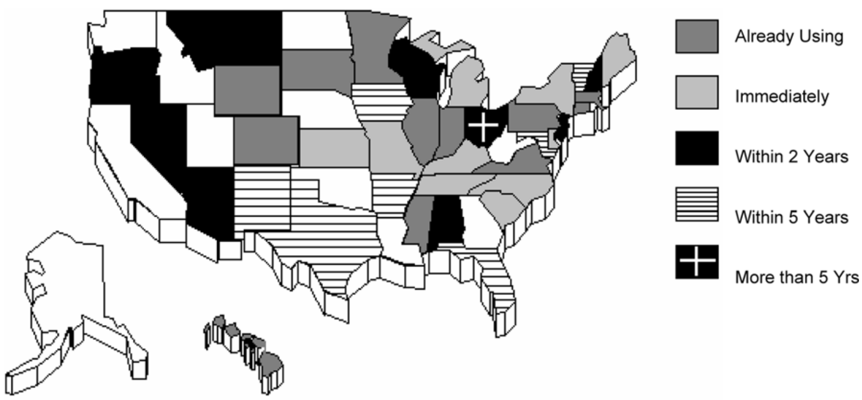

Fig. 1 Flow chart for fatigue life analysis

appropriate personnel in each U.S. state's department of transportation (DOT). One of the questions in the survey asked each state transportation official when they plan to adopt the 2001 design specification. Choices included: 1) already using, 2) immediately, 3) within 2 years, 4) within 5 years, or 5) more than 5 years. Fig. 1 presents the results for the thirty-eight states that responded to the survey. A complete summary of responses to this question as well as the results to all of the questions in the survey can be found in Ahlborn, *et al*. (2003). Responses to these questions were used to determine which overhead sign support structures to include in the full dynamic fatigue analysis portion of the study.

2. Calculation of fatigue performance

The response of overhead sign support structures to wind loading is assumed to be an elastic response, which results in a simplification of the method used to calculate (or estimate) fatigue life. A flowchart is presented in Fig. 2 to help clarify the approach used to estimate fatigue life in this study. Details of Fig. 2 are discussed later in this paper. However, it is important to note at this point that the wind history statistics and 25 wind forces are generalized by dividing a wind velocity probability density function into 25 bins. Consider an individual stress cycle for each structure, which can vary randomly in amplitude, but the period remains approximately constant provided it

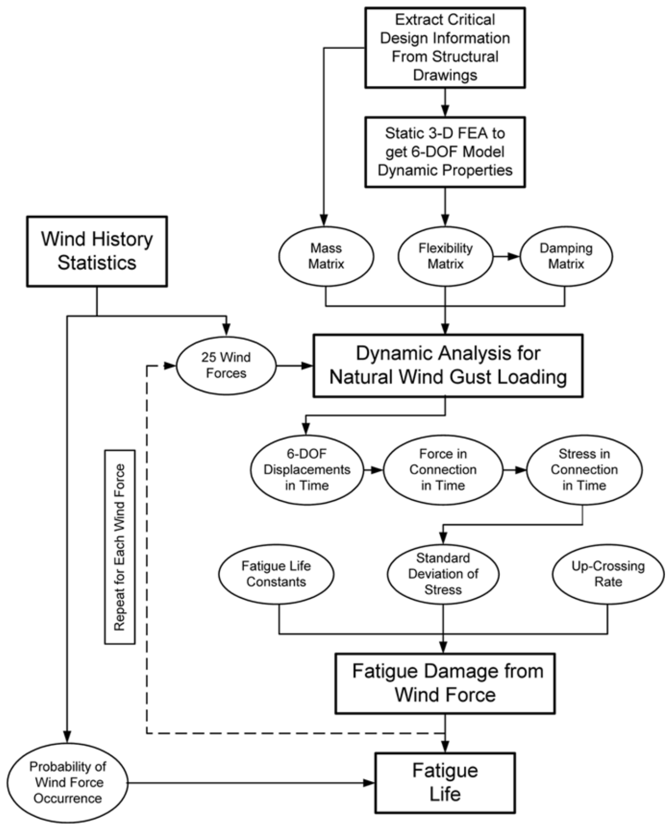

Fig. 2 Simplified structural model and degrees of freedom

remains in its linear range. According to Palmgren (1924) and Miner (1945) each cycle causes some amount of damage over time duration T. At some point in time, i.e., time to failure T_f, the accumulated damage reaches unity, indicating failure. Crandall and Mark (1963) showed that the expected value of the damage at one standard deviation of the stress level, σ_y, can be shown to be

$$E[D(T)] = \frac{\nu_o^+ T}{c}(\sqrt{2}\sigma_{yi})^b \Gamma\left(1 + \frac{b}{2}\right) \tag{1}$$

where Γ is the gamma function, ν_o^+ is the zero up-crossing rate, b and c are constants from the S-N curve for fatigue, and T is the time duration at the stress level in the same units as the zero up-crossing rate. Then, the fatigue life can be calculated as

$$F_{life} = \frac{1}{\sum_{i=1}^{n} F_i P_{oi}} \tag{2}$$

where $F_i = E[D(T)]$ and P_{oi} is the probability of the wind force associated with (causing) that σ_{yi}^2. The wind probability distribution is critical to the accuracy of the fatigue life estimates.

As mentioned, in order to estimate the fatigue life using Crandall and Mark's approach it was necessary to procure the wind statistical distribution information. Records of the peak wind gust every few minutes or even every hour for one or more years is considered very high resolution data and very difficult to find for specific locations. It was reasoned that since the overhead sign support structures under investigation were for virtually any location in the 144 k/hr region (ASCE 1998), it would be advantageous to use a high resolution composite wind distribution. In fact, the design wind velocity in the majority of the U.S. is currently 144 k/hr. (ASCE 1998). The wind distribution used in this project was that developed by Groisman and Barker (2002) at NOAA.

In order to compute the variance, σ_{yi}^2, of the stress at critical structural locations, i.e., connections, for an overhead sign support structure a simplified method was employed.

Consider an overhead sign structure, that is defined by a mass, damping, and stiffness matrix. The equilibrium of the forces acting on the mass at any instant in time for a MDOT system were analyzed numerically. Now, consider a wind gust having a velocity, V_w, acting on the sign portion of the structure. For purposes of simplification, the wind force acting on the sign itself will be considered, and the wind forces acting on the tubular or truss support structure will not be considered. This will be consistent for all signs, thus it should provide consistent results for a relative procedure. The pressure exerted on the sign by a wind gust having velocity V_w is given in AASHTO (2001) as

$$P_z = 0.613 K_Z G V_W^2 I_r C_d \tag{3}$$

where K_Z is a height and exposure factor, G is a gust effect factor, I_r is an importance factor, C_d is a drag coefficient, and P_Z is the resulting pressure in N/m^2. The total force on the sign due to the wind gust can be treated as a static force, F_W, given as

$$F_W = P_Z L W \tag{4}$$

where L and W is the length and width of the sign, respectively. The entire force vector, $\boldsymbol{F}$, can be determined by inserting F_W into the proper degree(s) of freedom and inserting zeros in the remainder of the vector. Then, the initial deflection of the overhead sign structure, $\mathbf{x_o}$, can be thought of as displacing the structure by an amount equal to $\mathbf{x_o} = \mathbf{K}^{-1}\mathbf{F}$. The equation of motion is then solved for a free vibration problem. The free vibration approach, while most likely underestimating the fatigue life, was used because it was felt to provide a good relative measure and it was felt that any relative change in estimated fatigue life between two structural supports due to the same external force was approximately linear. The simplicity of the approach also lent itself well to the investigation. There are also some closed form solutions available in the literature for certain types of tubular structures (see e.g. Robertson, *et al.* 2001, Homes 2002, Robertson, *et al.* 2003). If an actual fatigue life estimate was desired, it would be advisable to generate the appropriate forcing functions from the PDF of the wind velocity.

3. Structural models for fatigue analysis

The simplified structural model for cantilever structures consists of three-nodes and six degrees-of-freedom (DOF) as presented in Fig. 3. Node 1 is at the base of the vertical member, node 2 is at the intersection of the vertical pole and the horizontal arm or arms, and node 3 is at the midpoint of the span of the horizontal arm or arms. In the case of multiple arms, nodes 2 and 3 were oriented along the center-line of the vertical distance between the upper and lower arms. Each node inherently has six degrees-of-freedom (DOF), i.e., it can translate and rotate about any of the three axes that define three-dimensional space. However, for the purposes of this analysis, not all of the DOF's were needed at each node. The X at nodes 2 and 3 indicate a translational DOF in/out of the plane of the schematic. The double arrows, DOF θ_{y2} and θ_{y3} indicate rotations into the plane of the schematic. The other two DOF's θ_{z2} and θ_{z3} are moments within the plane of the schematic. The flexibility matrix, defined as the inverse of the stiffness matrix, was determined by using a finite element model, placing a unit load, or moment at each DOF, and measuring the model's displacement response at each of the other DOF's.

For tubular cantilever structures with two arms, such as the Michigan cantilever modeled in Fig. 3, half of the unit load was applied simultaneously to each arm at each arms translational degree of

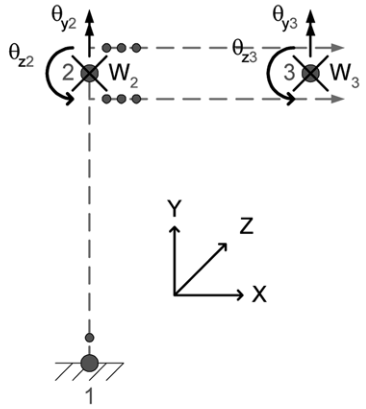

Fig. 3 Location of nodes for common connections on cantilever structures

freedom when determining the structural properties. The displacement due to this loading for each arm at each degree of freedom was then averaged to determine the final flexibility matrix entry terms. For example, to determine the f_{W3W3} flexibility matrix term for a two-arm cantilever structure, half of the unit load was applied along the Z-axis at the midpoint of the span of each arm simultaneously. The displacement of the upper and lower arms was then averaged to estimate the displacement along the center line of the vertical distance between them. The stiffness matrix can then be readily completed based on its definition as $\mathbf{k} = \mathbf{f}^{-1}$ and the mass matrix, $\mathbf{m}$, was modeled as a diagonal mass matrix.

The rotational inertia may be neglected in finite element analyses of this type because they often have little effect on the results. In order to validate this assumption, a sensitivity analysis was performed to determine the importance of the rotational mass matrix terms and Cook, *et al*. (2002) observation was confirmed, hence the rotational mass terms were neglected. The damping matrix was set constant at 1% of critical damping (Dexter and Ricker 2002) and was modeled as Rayleigh damping.

Critical information was gathered from the construction/shop drawings provided by the appropriate department(s) of transportation or fabricators and critical information extracted. Once this information was organized, it was used as input into an object-oriented Finite Element Analysis program and details were added to the models with some approximations made to the elements, if necessary. However, consistency in the complexity from structural model to structural model was of the utmost importance to ensure the integrity of the performance measure in the subsequent ranking procedure. The typical level of detail provided in the FEA models is presented in Fig. 4. Each member shown in Fig. 4 is modeled according to its individual structural characteristics. For example, in the MDOT cantilever base connection, the short member extending from node 1 to the first circular point represents the circular base plate, which, although it is very short relative to the length of the upright pole, is extremely stiff. The next member in that same figure represents a combination of the upright pole and the gusset plates welded to the pole to increase the stiffness at its base. The section above the stiffeners, indicated by the upward arrow, models the pole itself up to where the pole-arm connections are reached. Cantilever structures that were not able to be analyzed as planar structures were modeled and analyzed using 3-D FEA to determine properties for the 6-DOF dynamic model. Fig. 5 presents a 3-D FEA model for a Florida Department of

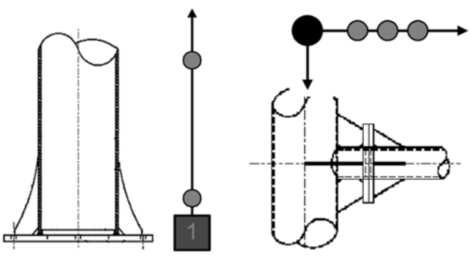

Fig. 4 FEA complexity of typical numerical models used in this study

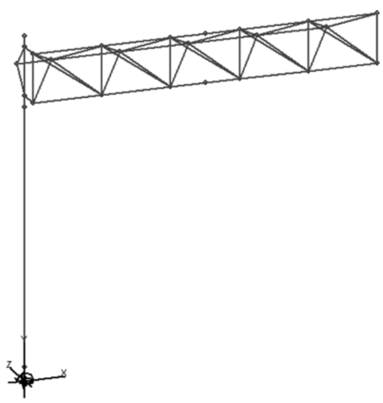

Fig. 5 Florida DOT FEA model from visual analysis

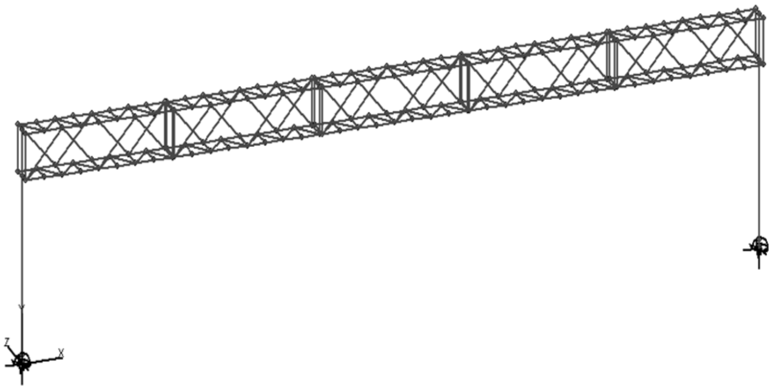

Fig. 6 Minnesota DOT FEA 3-D model of a 4-arm space truss

Transportation cantilever overhead sign support structure with a triangular space truss arm.

Although they have historically performed better than cantilever overhead sign structures, bridge type structures were also analyzed within the scope of this study. The same dynamic six-DOF model was developed using three-dimensional finite element analysis. In order to reduce the model to the same six-DOF used to model the cantilevers, symmetry was exploited. An example of a Minnesota space truss is presented in Fig. 6.

4. Wind load on overhead sign supports

Loading for the dynamic analyses was based on the probability distribution for wind velocity determined from Groisman and Barker (2002). The dynamic (free vibration) response of each overhead sign support structure was determined for twenty-five different wind velocities. Therefore, a lognormal PDF fit to the data was divided into twenty-five 'bins'. The bins probability of occurrences, p_{oi}, and associated wind velocities, V_w are used in Eq. (2) to determine the fatigue life. The magnitude of the applied point load is proportional to the area of the supported sign.

Once the mass, flexibility, and damping matrices were determined, dynamic analyses were performed to calculate the structural response to each impulsive wind load as outlined previously. These translations (and rotations) were then converted to stress time histories using basic mechanics. The details of each structure were determined from construction drawings provided by their corresponding department of transportation. A vibration duration of 30 seconds was arbitrarily selected. Sensitivity analysis was performed and it was found that any change in vibration analysis duration did not affect the fatigue life of one structure relative to another, i.e., the ratio remained constant. The standard deviation of the stress time history itself can be shown to be sensitive to this duration, but this is not discussed in detail here. Dexter and Ricker (2002) identified the mast-arm-to-column connections and column-to-base plate connections as the critical fatigue sensitive areas in overhead sign support structures. Fatigue failure in these connections will most likely occur in either the connection welds or the connection bolts. Therefore, the welds and bolts in the base and arm-pole connections were the focus of the fatigue life analysis, i.e., performance assessment.

5. Ranking procedure: Performance and cost

In order to rank the structures it was necessary to develop a common unit that could be used to combine performance (unit of years) and cost (a combination of weight and a constructability factor). This was accomplished using utility functions (Benjamin and Cornell 1970). The utility function for cost provides a maximum utility, $U[c]$, equal to 1.0 for low cost and a utility equal to 0.0 for very high cost which helps to reward the objective function for economy, clearly a desirable quantity.

The utility function for fatigue performance, $U[p]$, provides a low utility for small values of fatigue life and larger utility values as fatigue life increases. This helps to reward the optimization function for longer fatigue life. However, it was reasoned that fatigue life calculations for an overhead sign support structure are only advantageous up to the approximate design life, or perhaps more appropriately to some factor, e.g., 3 to 5, beyond that design life. Because of the inherent uncertainties and inaccuracies, fatigue life estimation methods may provide for larger value beyond this range. In order to provide significant weight for the performance assessment, but also provide a reasonable upper limit, a utility function that only significantly penalizes designs having fatigue lives less than some design life, e.g., 50 years was needed. Therefore, an exponential utility function was selected for performance. Mathematically, the performance utility function can be expressed as

$$U(p) = 1 - \exp(-\lambda F_{life}) \tag{5}$$

where λ is a scale fitting parameter and F_{life} is the fatigue life computed in Eq. (2). The performance utility function is presented in Fig. 7. For the purposes of this study, λ was set equal to 0.025 to attain the desired performance utilities, which were defined as having a value of approximately 0.9 for a 100-year fatigue life.

The utility function for cost can be expressed as

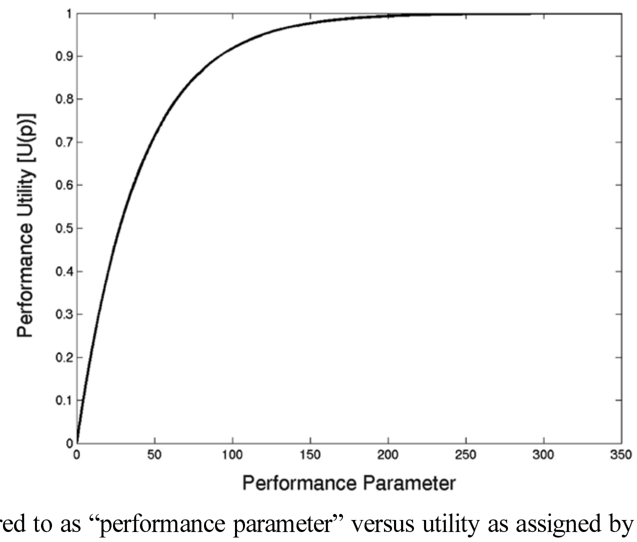

Fig. 7 Fatigue life, referred to as "performance parameter" versus utility as assigned by the expression in Eq. (12)

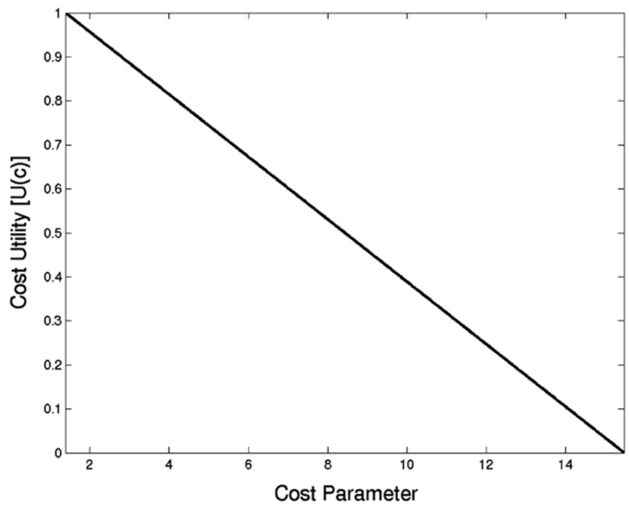

Fig. 8 Linear cost utility function used in this study

$$U(c) = 1 - \left(\frac{c - c_l}{c_h - c_l}\right) \tag{6}$$

where c is some cost measure of steel and c_h and c_l are the highest and lowest cost in the sign group being ranked, e.g., overhead cantilevers. The cost utility function is presented in Fig. 8. In order to combine these into a single objective function, R, weighting factors may be applied such that

$$R(p, c) = a_p\{1 - \exp(-\lambda F_{life})\} + a_c\left\{1 - \left(\frac{c - c_l}{c_h - c_l}\right)\right\} \tag{7}$$

where a_p and a_c are the performance and cost-weighting factors subject to the mathematical constraint

$$a_p + a_c = 1 \tag{8}$$

For example, if a_c is assigned a value of 0.25, then the cost utility, $U(c)$, accounts for 25% of the ranking parameter. Accordingly, the performance utility, $U(p)$, then accounts for 75% of the ranking parameter and a_p would equal 0.75. The maximum theoretical magnitude for R is 1, however this is impossible as $U(p)$ can never reach a value of 1 because it is an exponential function. The magnitude of the objective function can then be used as a basis for comparison of the overhead sign support structures analyzed in this study.

6. Illustrative example

This illustrative example demonstrates application of the procedure presented herein to help determine which overhead sign support structures perform best given a particular sign area requirement. Recall that the sign area significantly changes the magnitude of the wind load. The maximum allowable sign area for a specific structural design is determined within each state. In this

Table 1 Estimated fatigue life for illustrative example

Index Number	State	Description	Estimated Fatigue Life (yrs)			
			Pole-Base		Arm-Pole	
			Welds	Bolts	Welds	Bolts
	Cantilever Sign Structures					
1	AK	1-Arm	3	25	51	39
2	AK	2-Arm	6	12	93	56
3	AK	Monotube	9	> 500	> 500	> 500
4	CO	Monotube	> 500	> 500	> 500	> 500
5	IN	1-Arm	325	> 500	> 500	> 500
6	IN	2-Arm	110	344	> 500	> 500
7	MI	2-Arm C	> 500	485	> 500	102
8	MI	2-Arm D	434	107	> 500	465
9	MI	2-Arm E	350	157	350	237
10	MO	1-Arm	< 3	< 3	9	110
11	MO	2-Arm Truss	78	39	> 500	> 500
12	WV	2-Arm Truss	7	84	49	340
13	CA	4-Chord Truss	48	> 500	> 500	> 500
14	FL	3-Chord Truss	144	> 500	> 500	50
15	MN	4-Chord Truss	264	> 500	> 500	> 500
16	WI	4-Chord Truss	37	36	> 500	> 500
	Bridge / Span Sign Structures					
17	CO	Monotube	27	87	20	< 3
18	CA	Monotube	7	9	< 3	> 500
19	FL	3-Chord Truss	215	> 500	> 500	49
20	MI	4-Chord Truss	> 500	14	745	180
21	MN	4-Chord Truss	84	> 500	345	7
22	MO	2-Arm Truss	6	150	215	461
23	OR	4-Chord Truss	22	190	> 500	99
24	SD	2-Arm Truss	29	15	> 500	3

study, the sign size for each structure was set equal to the sign size currently used in a target state; in the present example this was the state of Michigan (MDOT 2001). This provided a relative performance measure, since any sign area smaller than the necessary size would not be able to be used within the state due to signing area requirements. There are three sign sizes currently in use in Michigan, so the sign size for other states were rounded up to the nearest Michigan sign size.

Table 1 presents the results of the fatigue life calculations for the pole-to-base connection and the arm-to-pole connection for the 16 cantilevers and 8 bridge-type overhead sign structures selected for analysis. These two connections were suggested as the location most likely to develop fatigue problems (Dexter and Ricker 2002). The table is sub-divided into bolted and welded connections because *S-N* curves for each of these differ significantly. The *S-N* curves were extracted based on connection type, from AASHTO (2001). Table 2 presents the details for the estimation of the cost utility for the signs. Table 3 presents the results of the ranking analysis when cost is excluded and

Table 2 Cost utility for illustrative example

Index Number	State	Description	Steel Weight	Constructability* Factor (CF)	Cost Utility
	Cantilever Sign Structures				
1	AK	1-Arm	2.52	1	2.52
2	AK	2-Arm	4.48	1	4.48
3	AK	Monotube	7.26	1.15	8.35
4	CO	Monotube	5.94	1.15	6.83
5	IN	1-Arm	4.87	1	4.87
6	IN	2-Arm	9.20	1	9.20
7	MI	2-Arm C	3.09	1	3.09
8	MI	2-Arm D	3.72	1	3.72
9	MI	2-Arm E	4.68	1	4.68
10	MO	1-Arm	1.38	1	1.38
11	MO	2-Arm Truss	2.89	1	2.89
12	WV	2-Arm Truss	3.19	1	3.19
13	CA	4-Chord Truss	12.90	1.2	15.48
14	FL	3-Chord Truss	6.15	1.2	7.38
15	MN	4-Chord Truss	11.95	1.2	14.34
16	WI	4-Chord Truss	5.03	1.2	6.03
	Bridge / Span Sign Structures				
17	CO	Monotube	23.09	1.15	26.55
18	CA	Monotube	23.35	1.15	26.85
19	FL	3-Chord Truss	40.00	1.2	48.00
20	MI	4-Chord Truss	27.11	1.2	32.53
21	MN	4-Chord Truss	31.26	1.2	37.51
22	MO	2-Arm Truss	13.11	1	13.11
23	OR	4-Chord Truss	39.81	1.2	47.77
24	SD	2-Arm Truss	19.82	1	19.82

*Tubular structures were assigned a CF of 1.0 as a benchmark. Truss structures were assigned a CF of 1.2 to reflect the increase in fabrication effort. Monotubes were assigned a CF of 1.15 to reflect the hot bending procedure used to form the 90 degree turn.

only fatigue life, or performance is considered. This example is limited to the sixteen cantilever structures because they have been shown to have more fatigue related problems. The left side of the Table 3 shows the rankings at the base connections while the right side shows the rankings based on the pole-to-arm connections. Both Indiana and Colorado are consistently in the top three. When cost is weighted as 25% of the ranking metric, the Indiana sign support structure remains near the top, but Colorado's drops to rank number 5 and 7 as shown in Table 4, for the base and pole-to-arm connections, respectively. Some measure of subjectivity is required to select several final candidates for design checks, but it can be argued that Colorado's monotube and Indiana's 1-arm tubular cantilever sign support structures performed very well based on the approximate ranking procedure presented herein. Although only two utility function weighting combinations were presented here, i.e., 100% performance & 0% cost, and 75% performance & 25% cost, numerous combinations can

Table 3 Sign support structure rankings based on 100% fatigue performance (Excluding Cost Estimate)

Ranked by Cantilever Base Connection					Ranked by Cantilever Arm-to-Pole Connection				
Index	State	Description	Performance	Rank	Index	State	Description	Performance	Rank
4	CO	Monotube	0.999	1	3	AK	Monotube	0.999	1
7	MI	2-Arm C	0.999	1	4	CO	Monotube	0.999	1
5	IN	1-Arm	0.999	1	5	IN	1-Arm	0.999	1
15	MN	4-Chord Truss	0.998	4	6	IN	2-Arm	0.999	1
9	MI	2-Arm E	0.980	5	11	MO	2-Arm Truss	0.999	1
14	FL	3-Chord Truss	0.972	6	13	CA	4-Chord Truss	0.999	1
6	IN	2-Arm	0.936	7	15	MN	4-Chord Truss	0.999	1
8	MI	2-Arm D	0.931	8	16	WI	4-Chord Truss	0.999	1
13	CA	4-Chord Truss	0.698	9	8	MI	2-Arm D	0.999	1
11	MO	2-Arm Truss	0.622	10	9	MI	2-Arm E	0.997	10
16	WI	4-Chord Truss	0.593	11	7	MI	2-Arm C	0.921	11
3	AK	Monotube	0.201	12	2	AK	2-Arm	0.753	12
12	WV	2-Arm Truss	0.160	13	14	FL	3-Chord Truss	0.713	13
2	AK	2-Arm	0.139	14	12	WV	2-Arm Truss	0.706	14
1	AK	1-Arm	0.072	15	1	AK	1-Arm	0.622	15
10	MO	1-Arm	0.072	15	10	MO	1-Arm	0.201	16

Table 4 Sign support structure rankings based on 75% fatigue performance & 25% cost estimate

Ranked by Cantilever Base Connection					Ranked by Cantilever Arm-to-Pole Connection				
Index	State	Description	Performance	Rank	Index	State	Description	Performance	Rank
7	MI	2-Arm C	0.969	1	11	MO	2-Arm Truss	0.973	1
5	IN	1-Arm	0.937	2	8	MI	2-Arm D	0.958	2
9	MI	2-Arm E	0.926	3	9	MI	2-Arm E	0.939	3
8	MI	2-Arm D	0.906	4	5	IN	1-Arm	0.938	4
4	CO	Monotube	0.903	5	16	WI	4-Chord Truss	0.917	5
14	FL	3-Chord Truss	0.873	6	7	MI	2-Arm C	0.910	6
6	IN	2-Arm	0.813	7	4	CO	Monotube	0.903	7
15	MN	4-Chord Truss	0.769	8	3	AK	Monotube	0.876	8
11	MO	2-Arm Truss	0.690	9	6	IN	2-Arm	0.861	9
16	WI	4-Chord Truss	0.612	10	15	MN	4-Chord Truss	0.770	10
13	CA	4-Chord Truss	0.524	11	2	AK	2-Arm	0.760	11
12	WV	2-Arm Truss	0.338	12	13	CA	4-Chord Truss	0.749	12
10	MO	1-Arm	0.304	13	12	WV	2-Arm Truss	0.747	13
2	AK	2-Arm	0.299	14	1	AK	1-Arm	0.696	14
1	AK	1-Arm	0.283	15	14	FL	3-Chord Truss	0.678	15
3	AK	Monotube	0.277	16	10	MO	1-Arm	0.401	16

be investigated to examine trends and/or sensitivities to help transportation officials select a design.

7. Conclusions

The release of the AASHTO 2001 *Sign Specification* resulted in widespread evaluation and the potential for re-design of overhead sign support structures throughout the US. A method was presented and applied that provided for a comparison of overhead sign support structures which accounted for both performance and cost. The method itself was numerical and thus, quantitative, whereas the weighting of performance and cost was left up to the user and was thus qualitative. The measure of performance considered in the analysis was the estimated fatigue life of critical structural connections subject to natural wind gust loading. The measure of cost was modeled as the product of the steel weight and a constructability factor.

Initially, a nationwide survey aimed at the State Departments of Transportation was conducted in order to determine the current status of their overhead sign support structures, i.e., whether they believe that they are or plan to meet the new design specifications. In general, while some problems were reported, State Departments of Transportation were aware of the changes in the AASHTO Sign Specification and appear to be working toward compliance within the next few years. Design plans and specifications were requested from the appropriate state transportation officials and gathered from State Departments of Transportation, and of those collected twenty-four were selected for analysis. Fatigue life estimates were determined for the performance measure of the ranking procedure using 6-DOF finite element models of the overhead sign support structures. The dynamic response of these models to natural wind gust loading was accounted for using linear dynamic analyses rather than simplified methods, such as equivalent static stress ranges. The response time histories were then converted to stress time histories in the pole-to-base and arm-to-pole critical connections. Evaluation of stress time histories were coupled with fatigue constants extracted from the appropriate *S-N* curves, and fatigue lives were estimated. Because the dynamic analysis was a free, rather than a forced, vibration approach the estimated fatigue lives are simply relative to one another. A weighted objective function combined cost and performance into a single quantitative optimization parameter for this performance-based assessment.

The objective function approach presented herein uses a single quantitative measure with which to compare overhead sign support structures based on two of the most critical decision making parameters: cost and performance. The method described herein is applicable to all types of overhead sign support structures and could be used by any U.S. State Department of Transportation (or other) to compare overhead sign support structural designs to one another and determine which may be most appropriate for their states needs. However, the decision methodology presented herein is generally applicable to any family of structures for which decision support is needed, as well.

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