STABILITY OF THE TRUSS BRIDGE

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1. Introduction

In developing countries, especiaL; in Asia, steel truss bridge is most widely used because of its several advantages. Steel truss bridges can be designed, constructed and placed in service very rapidly because the individual members of trusses have relatively low weight and consequently can be handled and erected with relative ease. However, because of the high strength/weight ratio of steel, steel compression members are, in general, more slender and more susceptible to buckling than reinforced concrete compression members.

Truss structures are usually constructed of short members pin-connected at the joints. Rarely are the chords continuous over several joints or unsupported by stiff members in a perpendicular plane at the joints. The only exception is the pony truss, in which the top chord is not restrained laterally.

In this study, the pony steel truss bridge is selected as a case study, which has 10 panels with total span length 30.48 m for one lane traffic is shown in Fig. 1. The stability limit load of the pony steel truss bridge has been obtained by the second order inelastic analysis using MARC finite element program. The idealized elastic-perfectly plastic stress-strain relationship with the load control method is employed in the analysis. Nonlinear geometric behavior is included by use of element geometric stiffness matrices and an updated Lagrangian formulation. Three dimensional analysis has been carried out.

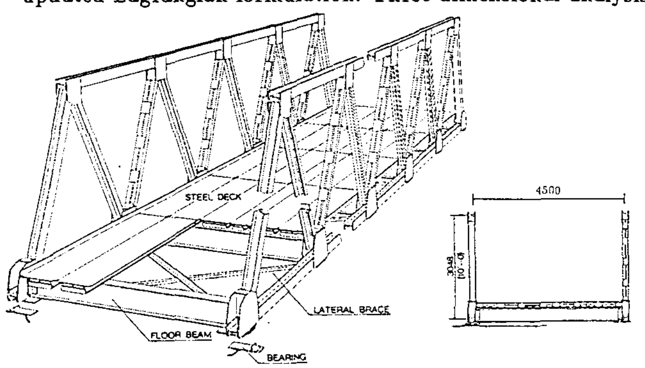

Fig. (1) Pony Steel Truss Bridge

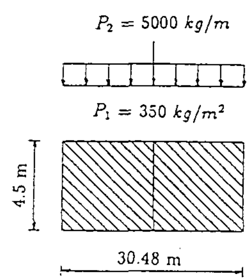

Fig. (2) Load distribution of JRA

2. Assumptions

T-20 lane loading of JRA as shown in Fig. 2 as live load and structure member own weight as dead load are applied to the truss. Since the purpose of this study is to find the maximum limit load of the structure, local buckling of members must be prevented from occurring before the attainment of ultimate load. To assure this point, this study checks the member thickness ratio according to the AISC as well as JRA specification. In this analysis, H 200x200x8/12 steel member of SM 50 steel material is considered and found to satisfy the limit of both specifications.

Structural members always include a number of imperfections. Residual stress, one of the imperfections of steel structure elements, is assumed a linear variation from a maximum compressive stress of 0.3 σ_y at the flange tip to a maximum tensile stress of $0.3\sigma_y$ at the flange-web juncture. Interactive effects of geometric imperfections, both local and system imperfections are considered in the study. Local imperfections are the initial curvature of the truss elements in a truss structure. System imperfections are deviations of the geometry of the real structure from that of the perfect one.

In the case of truss structure, system imperfections are represented by shifting the location of nodal points relative to the position in which they would be for a perfect structure. The imperfection is of a half-sine wave form applied to the upper chords of the truss in lateral direction. In the case of local imperfection, the mid span amplitude of the truss member's initial imperfection is a_o while in global imperfection the mid span amplitude of the total span length of upper chord's is l_o. The maximum initial imperfections are assumed to be equal to 0.1 and 0.33 percent of the member or total span length for local and global imperfection respectively. The initial imperfection is represented by a_i , where

$$a_i = a_o \sin \frac{\pi x}{L_i}; \quad a_o = 3 \text{ and } 10\ mm; \quad l_o = 30 \text{ and } 100\ mm$$

Table 1: summary of load and deflection at center of upper chord

Case	δ (mm)	P (ton)	$P_r = \frac{P}{P_{ASD}}$	σ_c/σ_y
Case I	85	263	2.4	0.98
Case II	81	252	2.3	0.95
Case III	81	250	2.27	0.94
Case IV	81	248	2.2	0.91
Case V	80	230	2.1	0.86

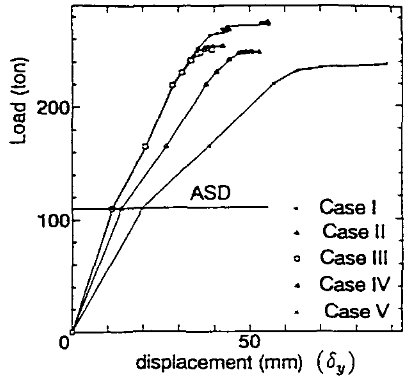

Fig. (3) Load vs. displacement at upper chord

3. Numerical result

The truss structure in this study has 39 elements, each on both sides of the floor beam, and is subjected to a uniformly distributed load and a concentrated load at the floor beam according to JRA allowable stress design load, 110 ton. Five different cases investigated in the study are as follows: Case I is the perfect structure without any geometric imperfections. Case II and Case III are the structure with 0.1% and 0.33% of member imperfections in upper chords and diagonal members. Case IV and V are the structure with 0.1% and 0.33% of global geometric imperfections.

Calculations for each case were carried out until the ultimate load was observed. Fig. 3 shows the deflection of the mid span upper chord in the vertical direction as a function of the applied load for all cases. Table 1 summarizes the results of all five cases. The load ratio (P_r) is the ratio of applied load to the ASD load (110 ton).

Case I exhibits an entirely stable equilibrium path in the load-displacement space. At a load of about 263 ton, the global buckling occurs. However, the equilibrium path is still stable. In Case II and Case III the member imperfection of L/1000 and L/300 are considered in upper chord and diagonal members. Even though the ultimate load is reduced to 252 and 250 ton, member imperfection of this allowable tolerance has not much influence on the behavior of the structure. The behavior is the same as perfect structure. However, after the ultimate load is reached, the stability is no longer maintained. The effects of imperfections at global level, case IV and V, have the similar effect. But the limit load is reduced to 248 ton in Case III while 230 ton in Case IV. As compared to the perfect structure, the reduction is 6% and 13%. The variation of load-deflection curve in the lateral direction is higher when the the imperfection increased.

In the first case, the compression member reached the member strength 195 ton which is close to the result from the single column analysis. Because of the imperfections the compressive strength is reduced to 189 and 187 for case II and case III and 181 and 171 ton for case IV and V, respectively.

4. Conclusion

1. Using the second order inelastic analysis, the behavior (displacement, stress and system buckling loads) of geometrically imperfect pony truss structures has been investigated.
2. The numerical results presented show that system imperfections have greater influence than the local imperfections on the behavior of truss structures.
3. It was shown that nonlinear effects are very important in analyzing the structure. Elastic analysis which does not take into account such nonlinear effects, can result in inaccurate results.

5. References

1. W. F. Chen and E. M. Lui: "Structural Stability; Theory and Implementation", McGraw-Hill, 1987.
2. Japan Road Association: "Specifications for Highway Bridges", March, 1984.
3. MARC Research Analysis Corporation: "MARC Program Manual", Vol. A to Vol. E, 1991.