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STRUCTURAL STABILITY RESEARCH COUNCIL
Fritz Engineering Laboratory 13
Lehigh University
Bethlehem, Pennsylvania 18015 U.S.A.

Phone: 215/758-3519
FAX: 215/758-4522

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FOREWORD

The Structural Stability Research Council (SSRC) held their 1990 Annual Technical Session in St. Louis on April 10-11. This city is the site of many major innovative bridge structures including the 115 year-old Eads Bridge with its magnificent arches spanning the beautiful Mississippi River. It was appropriate, therefore, that the subject selected for the Theme Session was "Bridge Stability Problems", a major concern of the Council.

It was 10 years ago in New York City that the SSRC last addressed the subject of Bridges. At that time, bridge engineers were concerned with stability problems since there has been a recent spate of buckling failures leading to major box girder collapses. Today, bridge engineers are having new stability concerns arising from:

- Bridges becoming lighter and more flexible because of the refinements in design and analysis and the increasing use of higher strength steels

- The proposed use of limit states design codes which require a new look at stability provisions

- The need to a evaluate compression members of existing bridges that exhibit damage and/or large deflections

Participants from 10 different countries attended the Technical Session which was very successful and worthwhile. There were 32 presentations on a wide variety of stability topics. Recent trends included the increasing use of computers to solve stability problems and the use of second order inelastic analysis in the design of actual structures. We thank the speakers for their presentations and for the papers they submitted for inclusion in these Proceedings.

We were fortunate to have as our luncheon speaker, Robert L. Nickerson of FHWA, who gave an excellent talk on "A New Era in Bridge Research". He outlined the future research program which would be implemented and sponsored by the FHWA. He also challenged SSRC to forward bridge stability research topics for FHWA consideration. The Executive Committee will address this challenge and welcomes suggestions from the membership as well as the task groups. We thank Mr. Nickerson for his very fine presentation.

The Panel Discussion on "Stability of Bridges" was led by Jackson L. Durkee with John M. Kulicki, Man-Chung Tang and C. Walter Brown as panelists. We thank these practicing engineers for their presentations. Their success in meeting their assignment was evidenced by the long and spirited discussion that took place following their prepared remarks.

SSRC operates on a modest budget and funding of the Technical Session must depend on organizations that recognize the value of such meetings in stimulating basic structural stability research. For this meeting we are grateful to the following sponsors:

- National Science Foundation
- American Iron and Steel Institute
- Yokogawa Construction Company
- St. Louis Section, ASCE
- Council for the Advancement of Steel Bridge Technology

We also owe our thanks to the many persons who worked so hard to ensure a successful Technical Session including Don Sherman and his Session Program Committee who were responsible for the selection of papers; Roger A. LaBoube who put together the Theme Session with the help of the Theme Organizing Committee and also headed the local Organizing Committee; Jerry Iffland and his Finance Committee; Yixian Gu our Technical Secretary, who made sure that everything ran smoothly in the meeting; Lesleigh Federinic, our Administrative Secretary, who planned, made, and coordinated the arrangements for all the meetings; Diana Walsh who made sure we were all registered in properly; and Kevin Truman who did the initial hotel site inspections and provided and coordinated the group of students from Washington University who provided valuable assistance in the meeting room and at the book table. Last but not least, we thank Lynn S. Beedle, our Executive Director, who seemed to be everywhere at the meeting, offering suggestions, evaluating the program, and making sure that everything was up to the standards of SSRC. Dr. Beedle suffered a heart attack last year shortly after the New York meeting of SSRC. We are grateful that he has made a good recovery and is again providing the leadership needed in the SSRC.

I became chairman of the SSRC last October and I thank the membership for their confidence in electing me to this position. I am indeed honored. On behalf of the membership of SSRC, I thank Sam Errera, immediate past chairman, who has earned our gratitude for successfully guiding the SSRC for the past 3 years. He was a conscientious leader who handled the problems that arose in a prompt and effective manner. All of us in SSRC are pleased that he will continue to serve on the Executive Committee.

At the meeting, I had the sad task of reporting the death of three SSRC members; Gene Wilhoite who was chairman of the ASCE Structural Division Executive Committee, Ravi Kinra who was past member of the SSRC Executive Committee, and Bruce Johnston was was a founding member of SSRC. The World View Document (2nd Edition) developed by SSRC and soon to be published, will be dedicated to Bruce Johnston and will contain a tribute to his memory.

The 1991 Technical Session will be held on April 15-17 in Chicago, Illinois, with the theme, "Inelastic Behavior and Design of Frames".

Gerard F. Fox
Chairman

New York, NY
July 1990

TABLE OF CONTENTS

TASK GROUP 4 - FRAME STABILITY AND COLUMNS AS FRAME MEMBERS

TASK GROUP 6 - TEST METHODS

TASK GROUP 13 - THIN-WALLED METAL CONSTRUCTION

TASK GROUP 17 - DOUBLY CURVED SHELLS & SHELL-LIKE STRUCTURES

TASK GROUP 29 - 2ND ORDER INELASTIC ANALYSIS FOR FRAME DESIGN

OTHER RESEARCH TOPICS

PANEL DISCUSSION

Interactive Instability in Deck-Type Steel Arch Bridges

Tetsuya Yabuki[1] and Sriramulu Vinnakota[2]

1. Introduction

Specifications generally require the arch axis of long span arch bridges to closely follow the shape of the equilibrium polygon for full dead load, which is generally the main load for designing the cross sectional properties of long-spanning arch ribs. In this case, all cross sections of the arch rib are subjected to compression with negligible shear and bending moment and the rib may develop instability problems. Among various arch bridge types feasible, the deck-type arch bridge structure is the most vulnerable to the instability phenomenon.

Generally, the fundamental instability mode for the inplane buckling of arch ribs is in the form of a reverse curve with part of the arch rib going down the other part going up, and the crown moving horizontally (i.e., unsymmetrical buckling mode). However, in the system with long panel-intervals between the posts, rib failure in a panel may occur. That is, local member instability may precede the overall instability of the bridge system, due to the so-called beam-column behavior of the arch rib in a panel. The local member instability and the evaluation of the associated loss of strength are of primary concern in the design of deck-type arch bridges.

Very little research has been reported so far on the fundamental characteristics of the instability in deck-type arch bridges (1,2). The data available at present is not sufficient to develop a direct statistical, empirical relationship that includes the interactive effect of local and overall instabilities of the arch rib in the deck-type bridge system. In this paper, the appearance of the local member instability mode and its progress until the inplane ultimate limit state, as the deck-type arch bridge system is loaded into inelastic and finite deformation range, are studied by an accurate nonlinear finite element approach. This approach takes into consideration material nonlinearities, spread of yielding zones in the cross section and along the longitudinal axis, and unloading caused by strain reversal. Next, interactive effects of the local member buckling on the ultimate stability strength of the bridge system are examined in detail. Based on the results obtained from the study, a practical formula for evaluating the interactive strength between the local member buckling and the overall instability is proposed. It is shown that the proposed formula is sufficiently accurate for practical applications to deck-type arch bridge systems.

2. Description of Bridge Model Studied

The arch bridge structural model considered in this study has an arch rib, a deck girder, and connecting elements between them (i.e., posts). The model is illustrated in Fig. 1 and its properties are also listed in the figure. The arch rib has a symmetric parabolic axial configuration and a constant box-shaped cross section with a welding residual stress distribution idealized as in Fig. 1. The rib and the

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[1]Professor, Department of Civil Engineering, University of Ryukyus, Okinawa, Japan

[2]Professor, Department of Civil Engineering, Marquette University, Milwaukee, WI 53233

deck girder over it are connected intermittently, at the panel points, by posts with hinged-hinged ends. The deck girder rests on rollers at the ends and is rigidly connected with the rib at the crown. Thus the deck does not carry any longitudinal axial forces. The deck girder is of a constant, I-shaped cross section. The idealized distribution of welding residual stresses in the I-shaped cross section is shown in Fig. 1. The girder has the same material as the arch rib. The cross sectional areas of the posts are chosen so as to avoid their premature failure. The arch structure is loaded by a series of concentrated loads at each panel point. The load acting at each panel point on the left half of the structure is q and that on the right half is r.q. In the stability calculations that follow, a value of r = 0.99 (a quasi-symmetrical loading) is adopted instead of the perfectly symmetrical loading (r = 1) to avoid convergence problems in the numerical iteration and to consider effects of geometrical imperfections.

The ultimate stability analysis has been carried out by a nonlinear finite element procedure using a modified incremental load method and the tangent stiffness approach (2,3). In all the numerical calculations, the arch and the deck girder are divided into 60 equal segments each. The elements of tangent stiffness matrix are evaluated numerically, by dividing the cross sections of the arch rib into 36 segments and the deck girder into 27 segments. It is assumed that no out-of-plane deformation occurs anywhere in the structure, so that the buckling strength would be governed by instability failure in the vertical plane. The component plates of the cross sections are assumed not to fail prematurely by local plate buckling. Two types of instability analysis have been performed, one is the elastic analysis and the other the inelastic analysis and both include the effects of finite deformations. The inelastic analysis takes into account spread of yielding, unloading and reloading of the yielded parts in the cross section and along the length, and residual stress due to welding. The load vs. deformation relationship of the arch bridge is obtained by successively incrementing the load until the maximum value is reached. For each load increment the tangent stiffness method and the Newton-Raphson iterative procedure are used.

The structural parameters examined in the study are: the rise-to-span ratio R/L, the slenderness ratio of the entire structural system of the arch bridge λ_T, which is defined by the ratio of the curvilinear length of the arch axis to the square root of $(I_d + I_a)/A_a$, the stiffness ratio of deck girder to arch rib I_d/I_a. The ranges of these parameters selected are given below; they are generally within those found in existing arch bridges.

$$R/L = 0.1 \sim 0.3,\ \lambda_T = 100 \sim 300,\ I_d/I_a = 0.1 \sim 10$$

The yield stress of the material F_y and the Young's modulus E are kept constant at 320 N/mm^2 and 2.1×10^5 N/mm^2, respectively.

3. Behavior and Instability of Arch Bridges

Typical results of the instability deformation mode with R/L = 0.15, λ_T = 200, 6-panels and 2-hinged supports are shown in Fig. 2. Fig. 2.a shows the overall instability for a Lohse type (I_d/I_a = 0.1) and Fig. 2.b the interactive instability of overall structural buckling and local member buckling for a Langer type (I_d/I_a = 10). It can be seen from the figure that the local member failure initiates originally in the end panel.

The difference in the instability deformation modes will be clear from the bending moment diagrams shown in Fig. 3. In the overall instability case, the highest bending moment of the arch rib is produced in the vicinity of the quarter point as shown in Fig. 3.a, while in the case of the interactive instability, the bending moment has its maximum value in the end panels. Since the arch rib has an axis that is curved continuously corresponding to parabolic configuration, local bending moments are generated by the offset given by the continuously curved configuration in a panel. It can be visualized that the arch rib in a panel of a Langer-Deck-Type bridge system, locally shows a beam-column member behavior with initial out-of-straightness from a line connecting with adjacent post locations (panel points). This initial out-of-straightness generates an additional bending moment in the arch rib (hereafter, this behavior will be termed beam-column model effect), and the local member buckling in a panel (especially in the end panel) is apt to produce.

The spread of yielding zones corresponding to the instability deformation mode is shown in Fig. 4. The springing of the arch rib has a wide spread of yielding corresponding to the local member buckling deformation as shown in Fig. 4.b.

The resultant bending moment vs. axial thrust relationships at the quarter and springing points of arch rib are given in Fig. 5 for various values of I_d/I_a (i.e., Langer and Lohse types). It can be seen from Fig. 5 that the resultant bending moment at the quarter point increases normally as the axial thrust increases until the ultimate stability state is reached. On the other hand, the bending moment at the springing becomes smaller, showing nonlinear behavior as the axial thrust increases. This behavior corresponds to the beam-column model effect in the end panel.

Some selected results of the study are shown in Fig. 6, where the load-deflection relationships at a quarter point of an arch rib with R/L = 0.15, λ_T = 200, 6-panels and 2-hinged supports for the Lohse and Langer types. In what follows, the load q is nondimensionalized with respect to a reference load q_p (3). The results of the elastic instability analysis are shown as the dashed curves and those of inelastic analysis are given by the solid curves. The elastic analysis shows that for the Lohse type its maximum load is nearly equal to the linear bifurcation buckling load of the arch bridge structural system, because both deformation modes are the fundamental instability mode described before (i.e., unsymmetric buckling mode). However, for the Langer type its maximum load can be lower than the bifurcation load. This discrepancy is caused by the interactive effect of local member failure. The true ultimate stability strength of the arch is the maximum load q_{max} of the inelastic analysis, which is generally much lower than the elastic value.

Typical results of analysis for 2-hinged and fixed arch bridges for the several values of I_d/I_a, λ_T and panel number are given in Table 1, where the hinged and fixed arches have identical geometrical and material properties. The ultimate stability strength decreases with increasing the slenderness ratio λ_T. The increase in strength of a Lohse type arch bridge (I_d/I_a = 0.1) over a Langer type bridge (I_d/I_a = 10) is confirmed. It can be seen that the interactive instability phenomenon occurs in the Langer type arches with 6 and 8 panels.

4. Ultimate Interactive Stability Strength Design Criteria

The extensive numerical results obtained from the study permit the development of an ultimate strength design procedure for the interactive instability of local member buckling and overall buckling in the deck-type arch bridge structures. The analytical study of the Langer-Type arch bridge structures with various values of panel numbers shows that the decrease in strength is due primarily to the local member buckling at the end panel, because of the interactive effect by the beam-column model behavior described previously.

A simple way of incorporating this interactive effect into the overall structural system design is to use a strength reduction factor ϕ. By multiplying the standard ultimate stability load by this reduction factor, the associated loss of strength in the design of deck type arch bridges can be evaluated. The factor ϕ could be evaluated by the following equation:

$$\phi = F_{cug,local} / F_{cug,overall} \qquad (1)$$

where $F_{cug,local}$ = strength of the aforementioned beam-column model in the end panel of arch rib prone to local member buckling and $F_{cug,overall}$ = strength of the beam-column model at the end panel of a standard arch rib in which the local member buckling does not occur until the ultimate state is reached -- that is, the ultimate strength is characterized by the so-called overall instability. The Langer-Type arch bridge system with 10 panels is herein adopted as the standard because the overall instability characterizes the ultimate stability strength of this bridge system.

The interactive ultimate strength $\bar{q}_{max,interact} = q_{max,\ interact} / q_p$ can therefore be determined as follows:

$$\bar{q}_{max,interact} = \phi\ \bar{q}_{max,overall} \qquad (2)$$

where $\bar{q}_{max,overall}$ = the nondimensional ultimate stability load intensity of the standard arch bridge system. The arch bridge structural system for which $q_{max,interact}$ is being calculated has all structural properties identical with the abovementioned standard one, except for the slenderness ratio of the beam-column model.

The numerical results obtained in this study allow a practical formulation of ϕ. The first step is to establish the formulation of $F_{cug,overall}$. The $F_{cug,overall}$ for the standard arch rib was computed using the same computer program as used herein. In analyzing the aforementioned beam-column model, its boundary conditons are taken as hinged-hinged for a 2-hinged arch and fixed-hinged for a fixed arch and given an initial crookedness corresponding to the parabolic configuration of the standard arch rib. The axial load was next applied to the initially bent column. By applying regression analysis to statistics of the computed results, a prediction formula for $F_{cug,overall}$ can be obtained as follows:

$$F_{cug,overall} = (1.239 - 0.224\ \bar{\lambda}_T)\ K_S\ F_Y \ : \text{for hinged arch}$$
$$= (1.194 - 0.218\ \bar{\lambda}_T)\ K_S\ F_Y \ : \text{for fixed arch} \qquad (3.a)$$

$$K_S = 1.104 - 0.005\ (I_d/I_a) - 0.001\ (I_d/I_a)^2 \qquad (3.b)$$

where $\bar{\lambda}_T = (\lambda_T/\ \pi)\ \sqrt{F_Y/E}$. It is proposed to use these results for $F_{cug,overall}$ to evaluate ϕ defined by Eq. (1).

The second step is to establish the formulation of $F_{cug,local}$. The $F_{cug,local}$ is seen from Eqs. (1) and (2) to be:

$$F_{cug,local} = \phi\, F_{cug,overall} = \frac{\bar{q}_{max,interact}}{\bar{q}_{max,overall}}\, F_{cug,overall} \qquad (4)$$

By substituting the numerical results of $\bar{q}_{max,interact}$ for 6- and 8-panel arch bridge systems and of $\bar{q}_{max,overall}$ for 10-panel obtained from the study, and value of $F_{cug,overall}$ from Eq. (3) into Eq. (4), the $F_{cug,local}$ is obtained. On the other hand, since $F_{cug,local}$ is the strength of the beam-column model at the end panel of the arch rib with local member failure, the $F_{cug,local}$ may also be evaluated by a column-strength-formula. For instance, the formula is taken as follows (4):

$$\begin{aligned} F_{cug,local} &= F_Y & &;\ \text{for } \bar{\lambda}_\ell < 0.2 \\ &= F_Y\,(1.109 - 0.545\,\bar{\lambda}_\ell) & &;\ \text{for } 0.2 \le \bar{\lambda}_\ell < 1.0 \\ &= F_Y/(0.733 + \bar{\lambda}_\ell^{2}) & &;\ \text{for } 1.0 \le \bar{\lambda}_\ell \end{aligned} \qquad (5)$$

and

$$\bar{\lambda}_\ell = (K_e\, \ell\, /r_a)\,(1/\pi)\,\sqrt{F_Y/E} \qquad (6)$$

where ℓ = axial length of the beam-column model at the end, r_a = radius of gyration of the arch rib cross section. Equating the values of $F_{cug,local}$ from the relations (4) and (5), $\bar{\lambda}_\ell$ and hence the coefficient K_e are obtained. Eventually the effects of the boundry conditions, the initial crookedness, and the interaction of overall and local instabilities are included in the K_e. By applying the regression analysis to the values of K_e obtained thus, a prediction formula for K_e was derived as follows:

$$\begin{aligned} K_e &= 1.0 : \text{for hinged arch} \\ &= 0.9 : \text{for fixed arch} \end{aligned} \qquad (7)$$

It is proposed to use the $F_{cug,local}$ given by Eqs. (5), (6) and (7) to evaluate ϕ defined by Eq. (1).

The accuracy of using the strength reduction factor defined by Eq. (1) in evaluating the ultimate interactive stability strength of a deck type arch bridge system ---- in which the reduction factor ϕ can be obtained by substituting Eqs. (3), (5), (6) and (7) into Eq. (1) ----, is illustrated in Fig. 7 for 2-hinged arch bridge structures and Fig. 8 for fixed ones. The solid curves show the interactive strength evaluated by Eq. (2), in which the 10-panel system is adopted as the standard arch bridge and it is analyzed by the ultimate strength approach. The circular marks indicate the q_{max}/q_p for 8- and 6-panel systems that are also calculated by the ultimate strength analysis. Additional results of analysis for two hinged arch bridges for a range of values of I_d/I_a and panel number are given in Table 2, where they are compared with the interactive strength evaluated by the design criteria proposed herein. The correlation is considered satisfactory for design purpose. It may be concluded that the interactive strength concept proposed herein gives results that are sufficiently accurate for practical applications.

5. Summary

An ultimate stability strength design criteria for deck-type steel arch bridges has been presented. The key step in establishing the criteria is to relate the standard ultimate stability strength for overall instability to the ultimate stability strength for interactive instability of the local member buckling, using a reduction factor defined by Eq. (1). Then, multiplying this factor by the standard ultimate stability load, the associated loss of strength by the local member buckling can be evaluated for deck-type arch bridges. No interactive instability analysis needs to be performed on the deck-type arch bridge structure. It has been demonstrated that the ultimate interactive stability strength can be determined fairly accurate by the criteria.

Current and future research on this topic should include studies of practical formulation on the ultimate overall stability load for the standard arch bridge structural system, rotation capacity for fixed arch rib, and spatial behavior and strength. In addition, careful evaluations of other bending conditions and dynamic effects are needed.

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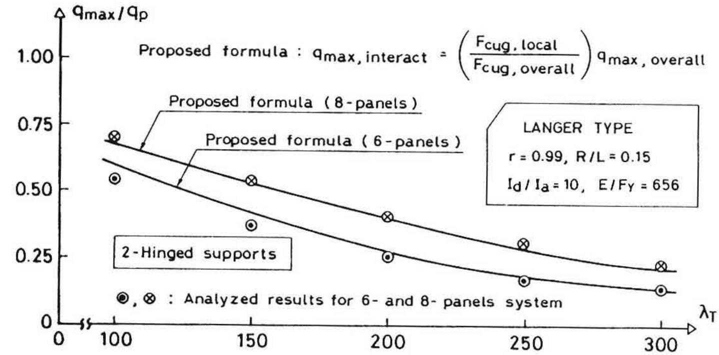

Fig.8 Comparison of Calculated Strength with Predicted Strength Based on Proposed Criteria(for 2-hinged type).

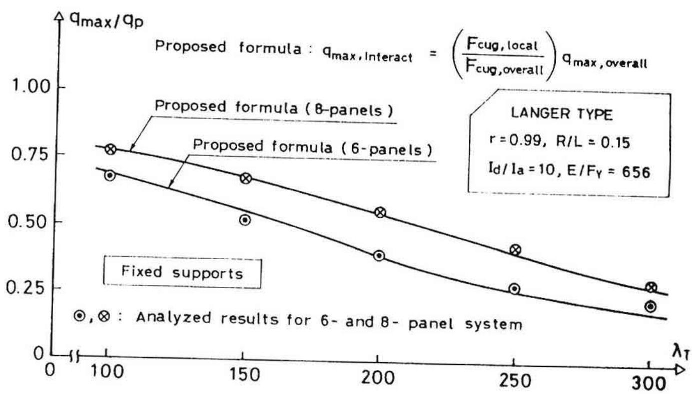

Fig.9 Comparison of Calculated Strength with Predicted Strength Based on Proposed Criteria(for fixed type).

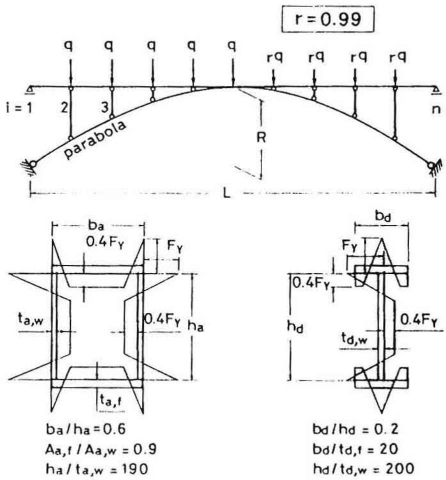

Fig.1 Reference Bridge Model Studied.

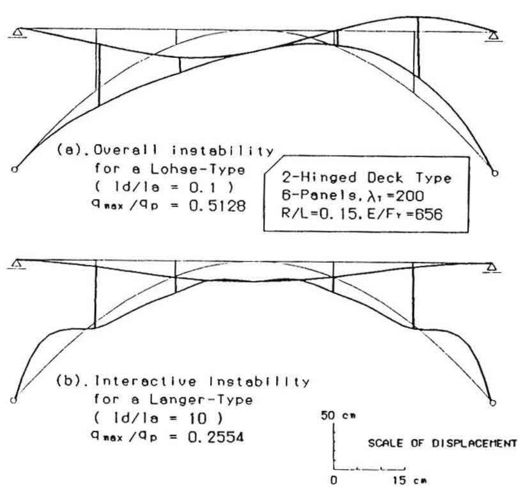

Fig.2 Variation of Instability Deformation Mode.