

Analysis of Stress Ribbon Bridge

Siddhartha Ray, D. M. Joshi, Rahul Chandrashekar

Abstract— A stressed ribbon bridge (also known as stress-ribbon bridge or catenary bridge) is a tension structure which is very similar to suspension bridges. The suspension cables form the part of the deck which follows an inverted catenary between supports. The ribbon is stressed such that it is in compression, which increases the stiffness of the structure where as a suspension spans tend to sway and bounce. The supports in turn support upward thrusting arcs that allow the grade to be changed between spans where multiple spans are used. Such bridges are typically made RCC structures with tension cables to support them. Such bridges are generally not designed for vehicular traffic but where it is essential, a certain degree of additional stiffness is required to prevent excessive flexure of the structure. A stress ribbon bridge of 45 meter span is modelled and analyzed using ANSYS version 12. For simplicity in importing civil materials and civil cross sections, CivilFEM version 12 add-on of ANSYS was used. A 3D model of the whole structure was developed and analyzed and according to the analysis results, the design was performed manually.

Index Terms— Stress Ribbon, Precast segments, Prestressing, Dynamic Analysis, Pedestrian Excitation.

I. INTRODUCTION

Stress-ribbon bridge is the term that has been created to describe on which pedestrians can directly walk and is in the shape of an inverted arch. The structure consists of tensioned cables which have slight sag in them, embedded in a concrete deck which is very slender when compared with the span. This deck serves the purpose of distributing the loads and to maintain the continuous nature of the deck. It is a type of a suspended structure where the cables are tensioned so tightly that the pedestrian load can be placed directly on the concrete deck in which the cables are embedded. It's a very simple structure when compared to other complex structures like suspension bridges. The only drawback of these bridges is the very fact that the tensioning force is so high in the cables that one has to anchor its abutments which makes the bridge expensive.

A stressed ribbon bridge is a structure primarily in tension which is very similar in many aspects to a suspension bridge. The stress ribbon design is rare in the world. Very few people including bridge engineers are familiar with this form and fewer than 50 have been built worldwide. Unlike a simple suspension bridge, the ribbon is stressed in compression which increases the stiffness of the structure. Such bridges are typically made RCC structures with tension cables to support them. They are used mainly for pedestrian and cycling traffic. Stress ribbon bridges are maintenance free, aesthetic and economical when compared with their spans.

Revised Version Manuscript Received on August 11, 2016.

Mr. Siddhartha Ray, Civil Engineering/ SCOE, Kharghar, Navi Mumbai, India.

Prof. D.M. Joshi, Civil Engineering/ SCOE, Kharghar, Navi Mumbai, India.

Mr. Rahul Chandrashekar, Bridge department L&T construction, Mumbai, India.

The slender deck allows them to make the minimum use of construction material. At present, studies are undergoing on combining stress ribbon bridges with cables or arches, to build most economical stress ribbon bridges.



Fig. 1 Redding Pedestrian stress ribbon bridge.

There are 3 types of stress ribbon bridges, namely traditional, stress ribbon bridge supported on an arch and stress ribbon bridge suspended on an arch. Fig. 1 is an example of a traditional stress ribbon bridge.

For stress ribbon bridges supported on arches, the arch serves as a support (saddle) from which the slender deck can rise during post-tensioning and during temperature drop, and can rest during a temperature rise. The Fig. 2a shows the general structural arrangement of a stress ribbon bridge supported on arches. In its initial stages, the stress-ribbon behaves as a two-span cable supported by the central support (saddle) that is fixed to the end supports or its abutments (ref. Fig.2b). The supporting arch is loaded by the self weight of the stress-ribbon, the weight of the saddle segments and the radial forces exerted by the tensioned cables (ref. Fig.2c) [4]. After post-tensioning the stress-ribbon and arch behave as one structure.

The horizontal forces in the stress ribbon H_{SR} and the arch H_A play a critical role since they have to be chosen in such a manner that they are equal. If the aforementioned criteria are met, the stress ribbon and the arch can be connected by an inclined strut which shall primarily be in compression. An overturning moment shall be created by horizontal cable forces $H_{SR} \cdot h$ which shall be then resisted by the arch reaction $\Delta V \cdot L_p$. Hence a self-balancing system is created with only vertical reactions (ref Fig.2d) [4]. Another alternative is to suspend the stress-ribbon from the arch. Self-balancing systems can be made for the same as well.

Fig. 3a, shows a fixed arch at the end anchorages of a slender prestressed concrete deck.

Analysis of Stress Ribbon Bridge

The loadings on the arch is its own weight in addition to the self weight of the stress-ribbon combined with the radial forces of the tensioning cables. Fig. 3b shows a structure that has a similar static behaviour as the structure presented in Fig. 2d. Fig. 3c shows a similar structure in which the slender prestressed concrete deck has increased bending stiffness in the portion of the structure not suspended from the arch [4].

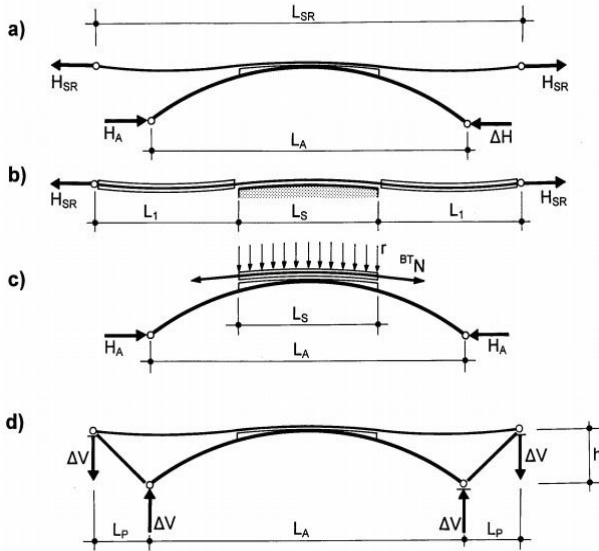


Fig. 2 Stress ribbon supported by arch.

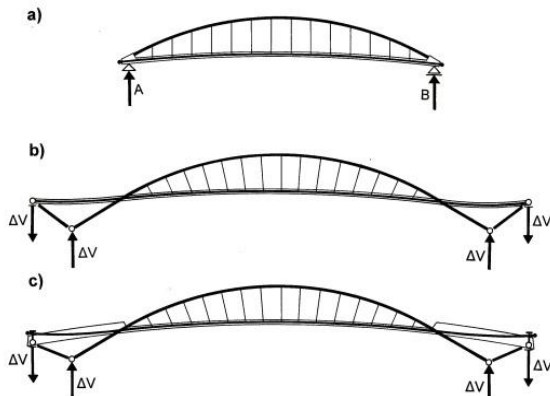


Fig. 3 Stress ribbon suspended on arch.

The aim of the present study is to analyse a traditional stress ribbon for a span of 45 meters for IRC loading conditions and a safety check for pedestrian excitation.

II. MODELLING AND RESULT ANALYSIS

The deck segment selected was 4 meters wide, 250 mm thick and 1 meter long. The bridge would consist of 45 such segments supported on bearing cables. The meshing was performed according to the length of segments. The deck had to be given a sag by the formula $f = 0.02 * L$ where L is the length of the bridge [2]. The final geometric model of the bridge is shown in Fig. 4.

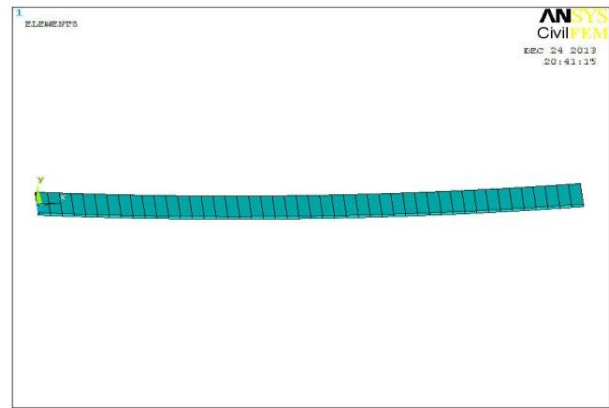


Fig. 4 Final geometric model.

The structure was loaded with live, wind and temperature loads as per IRC 6-2010. Wind and temperature loads were calculated for Mumbai region. The force in the bearing cable was calculated as per $P = \frac{w * L^2}{8 * f}$, where w is the self weight intensity of the bridge deck [2]. The cable forces come to 7320 kN which will require 4 12K15 prestressing cables to transfer the load to the abutment. The structure was analyzed in ANSYS for the individual load cases and load combinations as per IRC 6-2010. The results are shown in table 1.

Table 1 Loading Results

| Sr. No. | Load Case | Bending Moment (kNm) Maximum | | Deflection (mm) |
|---------|--------------------------------|------------------------------|-----|-----------------|
| | | Sag | Hog | |
| 1 | Live Load | 104 | 212 | 65.66 |
| 2 | Transverse Wind Load | - | - | 0.3 |
| 3 | Vertical Wind Load | 36 | 72 | 22.36 |
| 4 | Temperature Load (+) | 80 | 158 | 49.26 |
| 5 | Temperature Load (-) | 158 | 80 | 49.26 (upward) |
| 6 | Normal Combination | 104 | 212 | 65.66 |
| 7 | Temperature Combination | 184 | 370 | 115 |
| 8 | Wind & Temperature Combination | 219 | 443 | 137.3 |

The critical load combination in terms of prestressing steel comes out to the temperature combination which also proves that stress ribbons are critical to temperature [2]. It was for this combination the design was carried out as per IRC 18-2000 manually and 6, 4K15 cables stressed at a force of 800 kN each, including losses was found out as the post tensioned steel requirement.

The structure was then analyzed for the different critical prestressing conditions in ANSYS and the deflection for the structure in the final state under temperature combination and prestress is shown in Fig. 5. Also since stress is an important parameter under prestressing conditions the maximum stress results under temperature combination and prestress are also shown in Fig. 6. The table 2 shows the results due to prestress under transfer and temperature combination. Non critical cases and bending moments are not mentioned in table 2. Minimum reinforcement in the form of untensioned steel has to be provided in the segments for safety against temperature.



Fig. 5 Deflection due to temperature combination and prestress in meters.

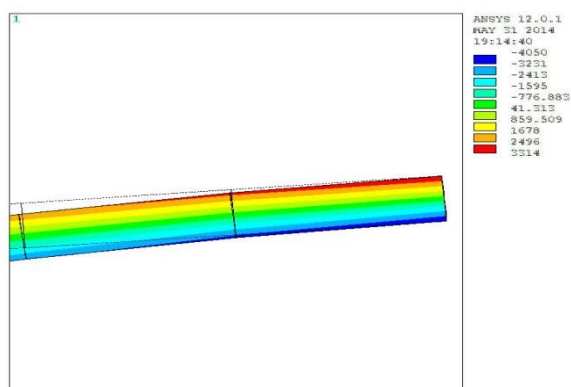


Fig. 6 Stress due to temperature combination and prestress in kN/m².

Table 2 Prestressing Results

| Sr. No. | Load Case | Bending Moment (kNm) Maximum | | Deflection (mm) | Stress (N/mm ²) |
|---------|-------------------------------------|------------------------------|-----|-----------------|--------------------------------|
| | | Sag | Hog | | |
| 1 | Dead Load & Prestress | 217 | 106 | 67.08 (upward) | 10.16 (Comp) & 0.233 (Tensile) |
| 2 | Prestress & Temperature Combination | - | - | 47.84 | 4.06 (Comp) & 3.31 (Tensile) |

The initial approach was to design a standard abutment that is generally used for bridges with the proportioning of its

dimensions as given in standard books used for bridge engineering. The drawback however was that the horizontal deflection of the abutment due to the cable forces was very high, almost 400 mm deflection for 5 meter height of the abutment. Hence a stepped abutment was taken into consideration whose deflections were much lesser than the standard abutment. The cross sectional dimensions of the stepped abutment are shown below in Fig. 7. The top width of the abutment is 4 m which is the width of the deck of the bridge. A gradually decreasing drop of around 300 mm is provided so as to accommodate the first segment of the deck having a depth of 250 mm and an extra 50 mm for the provisions of elastomeric bearings while maintaining the slope of the deck. The bottom width of the abutment provided is 9 m so as to satisfy the S.B.C. conditions of the strata below. The stratum below is assumed as rock having a S.B.C. value of 900 kN/m².

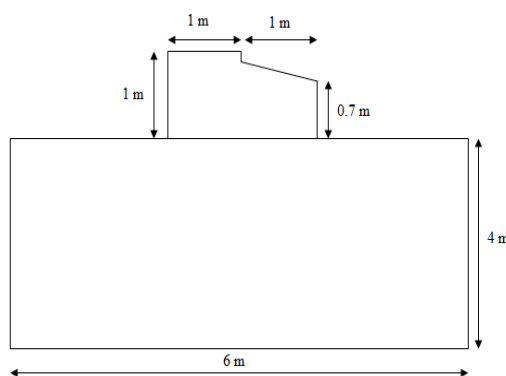


Fig. 7 Abutment cross section.

The most critical loading condition for the abutment is the cable forces acting on it [2] and structural engineers around the world design the abutment for cable forces only. The abutment was analyzed for cable forces in ANSYS and the bending moment results for the same are shown in Fig. 8. The design was performed manually as per IRC 21-2000 and the untensioned reinforcement for the top part of the abutment comes out to 20 tor bars @ 125 mm c/c and for the bottom part comes out to 32 tor bars @ 100 mm c/c.

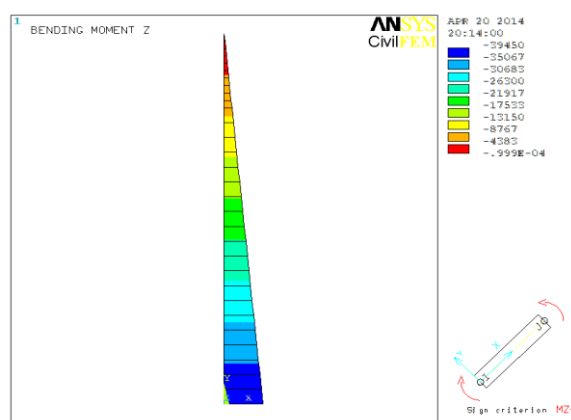


Fig. 8 Bending moment in abutment due to cable force in kNm.

While most bridge abutments have only



compressive forces on the soil, a stress ribbon bridge abutment will also have tension on its abutment base. The soil pressure results are shown in Fig. 9 under normal working conditions without any soil improvement methods. It is due these high tension forces on the abutment, rock anchoring is generally suggested to keep the abutment in place.

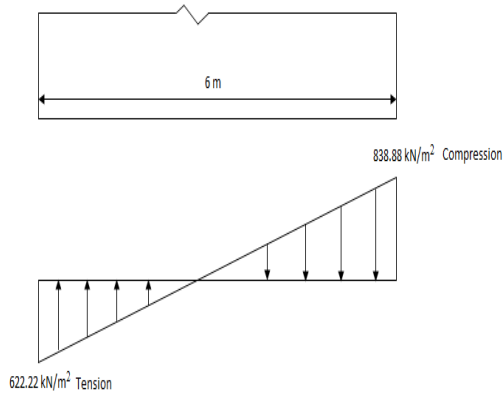


Fig. 9 Soil pressure results.

III. DYNAMIC ANALYSIS

There are three primary mode shapes for the deck, namely, vertical, lateral and torsion modes. The first three modes are vertical. The first lateral mode is the fourth mode and the first tensional mode is the seventh mode.

The first two vertical modes have a frequencies $f_1=1.379$ Hz and $f_2=1.788$ Hz respectively and the mode shapes are shown in Fig. 10 and Fig. 11 respectively. Normal practice suggests that that the second mode f_2 shape should actually be the first mode shape but, this mode requires an elongation of the cable, hence the corresponding frequency in some cases is higher and mode shape of f_1 becomes the first mode [2].

The first lateral mode is the fourth mode of the structure with a frequency $f_4=3.959$ Hz. Due to the vertical curvature of the prestressed band, a horizontal movement is always combined with torsion and it is therefore difficult to find a pure torsional mode [2]. The first torsional mode is the seventh mode for the structure with a frequency of $f_7=7.129$ Hz. Both the mode shapes are shown in Fig. 12 and Fig. 13 respectively.

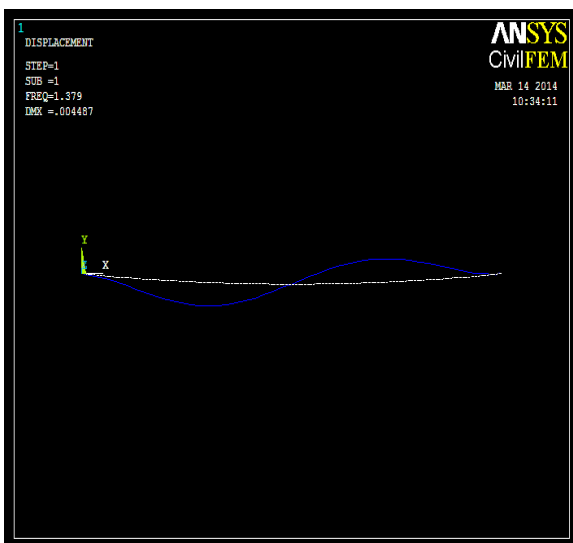


Fig. 10. Mode shape for frequency $f_1=1.379$ Hz.

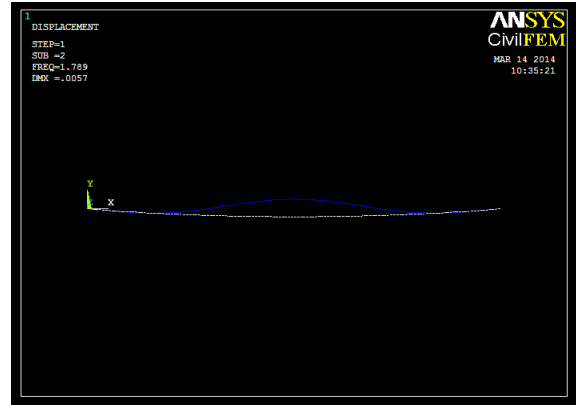


Fig. 11 Mode shape for frequency $f_2=1.788$ Hz.

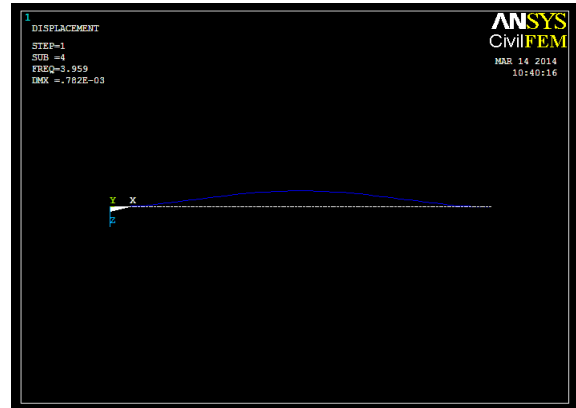


Fig. 12 Mode shape for frequency $f_4=3.959$ Hz.

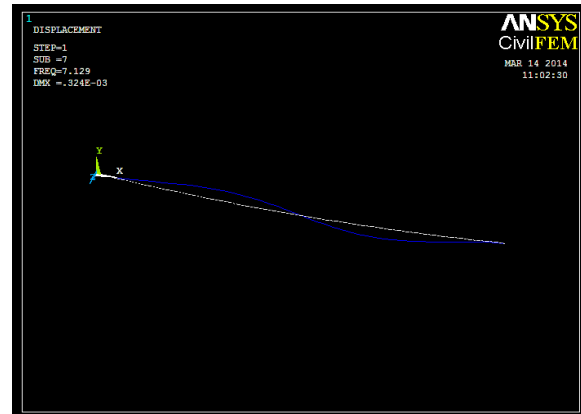


Fig. 13 Mode shape for frequency $f_7=7.129$ Hz.

Stress ribbon bridges are very slender in nature and have a natural frequency very close to the pedestrian walking frequency of 2 Hz [Error! Reference source not found.], [4], [5]. The codes of choice for structural engineers around the world for pedestrian excitation analysis are The British Standard BS 5400 Part 2 and Ontario Highway Bridge Design Code OHBDC ONT 83 [2], [6]. For this article, I have used *fib* 32 guidelines and BS 5400 limiting criteria. The *fib* 32 has a calculation method which determines the maximum vertical acceleration resulting from the passage of one pedestrian walking / running with a pace rate equal to the fundamental natural frequency of the bridge. Therefore, it is modelled as an equivalent single degree-of-freedom oscillator.

The formula is given below,

$$a = 4 * \pi^2 * f^2 * y * \alpha * \phi$$

Where, a = Acceleration in m/s^2 ,

f = Vertical natural frequency of the bridge in Hz,

y = Static deflection at mid span in m for a force of 700N,

α = Fourier coefficient of the relevant harmonic of the walking or running rate,

ϕ = Dynamic amplification factor for one pedestrian moving across simple span (See Fig. 14).

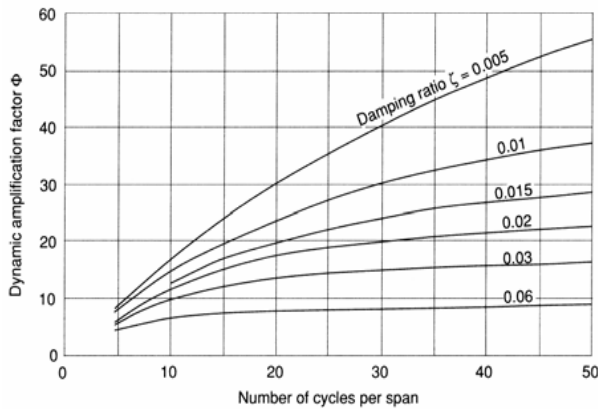


Fig. 14 Dynamic amplification factor.

For given conditions, the value of α would be 0.4 for a step frequency of 2 Hz for the first natural frequency and number of cycles per span for a 45 m span bridge is 60 [1]. BS 5400 Part 2 also considers the higher order harmonics of the structure and hence it has a limiting value of frequencies to be considered till 5 Hz. However research has shown that higher order harmonics (frequency above 2.4 Hz) will not produce important oscillations in a structure due to the lower force component of higher harmonics [1]. The permissible vertical acceleration is, $a_{v,max} = 0.5 * \sqrt{f}$. The results of pedestrian excitation is given in table 3.

Table 3 Pedestrian Excitation Results

| Frequency (Hz) | y (m) | α | η | a (m/s^2) | $a_{v,max}(m/s^2)$ |
|----------------|----------|----------|--------|-----------------|--------------------|
| $f_1=1.379$ | 0.000508 | 0.4 | 25 | 0.381682 | 0.587154 |
| $f_2=1.788$ | 0.000508 | 0.1 | 25 | 0.160416 | 0.668581 |

The above table shows that the vertical accelerations of the bridge deck for the first two frequencies are well within the permissible limits and hence the pedestrians will not feel discomfort while walking on the bridge. Hence the bridge will be safe from pedestrian excitation at its critical frequencies.

IV. CONCLUSION

The basic objective of this study is to elaborate on what a stress ribbon bridge is, its types and its advantages over conventional pedestrian bridges. This study elaborates on the various parameters essential before the actual design or analysis and the sources from which they are obtained. It also briefly explains the process of modelling, loading and analysis of a stress ribbon bridge in ANSYS. The critical conditions were identified and designed for manually. In this

study a model of a stress ribbon bridge of 45 meter span is modelled and analyzed using ANSYS version 12. the following conclusions can be made:

- Design of stress ribbon structures have confirmed that a slender concrete deck supported by an internal and/or external cables can be very efficient.
- Their designs have to be based on understanding of the cable tension, bending stiffness and on the understanding of the function of the prestressed concrete deck.
- The bending moment in a stress ribbon bridge is much lesser when compared to a beam because of its inverted arch shape.
- Temperature is a critical condition in a stress ribbon bridge due to its slender deck and flexible nature.

The abutment has to transfer high cable forces due to which it requires a very good stratum below it (preferably rock).

REFERENCES

1. D. E. Newland, 2004. "Pedestrian excitation of bridges", Institution of Mechanical Engineers (IMechE), Volume 218, Part C, 477-492.
2. Jiri Strasky, et al., 2005. "Guidelines for the design of footbridges", federation internationale du béton (fib), Bulletin 32.
3. Jiri Strasky, 2006. "Stress Ribbon and Cable Supported Pedestrian Bridges", Academy of Sciences of the Czech Republic, Brno, Czech Republic.
4. Jiri Strasky, 2008. "Stress Ribbon Pedestrian Bridges Supported or Suspended on Arches", Chinese-Croatian joint colloquium, Long Arch Bridges, Brijuni Islands, Croatia, 135-147.
5. Jiri Strasky, 2010. "Stress Ribbon and Arch Pedestrian Bridges", 6th International Conference on Arch Bridges, Fuzhou, China, 38-45.
6. Dr. Chung C. Fu, "Dynamic Response of Pedestrian Bridges", Research Professor, University of Maryland.