

Dominant failure mode identification and structural reliability analysis for a CFST arch bridge system

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ABSTRACT

Identification the subset of dominant failure modes and reliability calculation are the most important part of system reliability estimation. An efficient dominant failure mode identification method for bridge systems is proposed based on stage critical strength branch and bound algorithm. The method could identify the dominant failure mode in the decreasing order of the system final critical strength and get the expressions of them. The reliability index of each dominant failure mode is calculated by FORM method and the reliability index of the bridge system is calculated by PNET method. The method is applied to analyze the system reliability of a real CFST arch bridge. The results reveal various combinations of the failure modes in significantly reduced time and efforts, compared with the traditional permutation method.

1. INTRODUCTION

Bridge safety has become a public concern after several collapses of bridges in recent years. Nowadays, the increasing axle loads and traffic density are the main causes of bridge accidents in China. Actual truck loads are noticeably higher than the design loads, which leads to a higher risk. The demands for systematic and efficient risk-safety assessment of bridges are increasing to prevent possible disasters subsequently. As a method for bridge safety assessment, structural system reliability analysis is widely accepted (Nowak 2004, Wang et al. 2011a; Wang et al. 2011b).

Failure mode approach (FMA) (Quek 1987) is a popular and widely accepted method for the reliability analysis of multi-member systems such as bridges. In the FMA method, reliability analysis of bridge systems can be divided into two steps: (a) identification of failure modes; (b) estimation of failure probabilities of individual modes

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and the overall system.

Relatively, identifying the failure modes is the key point of the FMA method. Unfortunately, bridge may collapse in different failure modes, depending upon the combination of applied loads and the strengths of various elements. Identification, enumeration and description of all these failure modes poses a difficult combinatorial problem, and interest in this task has varied with time (Avinash et al. 1987). However, in most cases, only a small fraction of the modes contributes significantly to the overall failure probability of the system, which is called dominant failure mode. If the subset of dominant failure modes has been identified, the true failure probability of the system can be approximated as Eq. 1:

$$p_f = P\left[\bigcup_{\text{all possible paths}} S_i\right] \approx P\left[\bigcup_{j=1 \dots m} S_j^*\right] \quad (1)$$

Where S_j^* are the dominant failure mode. Hence, an important part of system reliability estimation by the FMA method is to setting up the dominant failure mode identification strategy.

Several different approaches have been developed to identify the dominant failure mode of a structure system. Moses et al. developed a failure mode identification method called incremental loading method based on the mean value of the input random variables (Moses & Rashedi 1983). Other methods include beta-unzipping method (Thoft-Christensen & Murotsu 1986), branch and bound method, truncated enumeration, etc. Even though some of these methods present elegant approaches for identifying the dominant failure modes, seldom are applied to a real bridge structure.

The objective of this article is to present an innovative strategy on how to search the dominant failure modes for real bridge structure under traffic load. A traditional method called Stage Critical Strength Branch and Bound Method (SCSBB Method) (Dong 2001) is referred and modified to suit for bridge dominant failure modes identification. The advantage of the proposed strategy is discussed through applying it to a CFST arch bridge.

2. STAGE CRITICAL STRENGTH BRANCH AND BOUND METHOD

2.1 Introduction

SCSBB Method is a kind of incremental load method (Moses 1982). It provides a systematic and rational procedure to identify the various failure paths. The method involves four main operations:

- (1) Calculation of load factor $a_{r_k}^{(i)}$

The state of the structure system in which the i th components have failed already is called damage state i . $a_{r_k}^{(i)}$ is a load factor (or influence coefficient) and it is equal to the load effect in component r_k at damage stage i due to the standard external load.

Traditional SCSBB Method usually uses the unit concentrated load as the standard external load. In order to make the SCSBB Method suit for bridge structure, here the standard external load is equal to the bridge traffic load design value from

Design Code. The $a_{r_k}^{(i)}$ is calculated by FE modeling and analysis using influence lines.

(2) Calculation of the component residual resistance $R_{r_k}^{(k)}$

$R_{r_k}^{(k)}$ is the residual resistance of component r_k . The basic formula to compute $R_{r_k}^{(k)}$ is as Eq.2:

$$\begin{cases} R_{r_k}^{(k)} = R_{r_k}^{I_{r_k}} - G_{r_k}^{(k)} - I_{r_k} \times \sum_{i=1}^{k-1} a_{r_k}^{(i)} \square F_{r_i}^{(i)} m_{r_i} \\ \square F_{r_i}^{(i)} = \frac{R_{r_i}^{(i)}}{a_{r_i}^{(i)}} \\ I_{r_k} = \text{sign}[a_{r_k}^{(k)}] \end{cases} \quad (2)$$

where $R_{r_k}^{I_{r_k}}$ is the original resistance of component r_k ; m_{r_i} is a failure type indicator variable, so the proposed method is general enough to include two types of component failure: brittle or ductile. For brittle failure, each failed member is removed from the structure FE model before reanalysis and $m_{r_i} = 0$. For ductile failure, each failed member is removed from the model before reanalysis but a force equal to the load carrying capacity of the component acting along the components is applied and $m_{r_i} = 1$; $G_{r_k}^{(k)}$ is the load effect by dead loads.

(3) Calculation of stage critical strength of bridge $R_{S,r_k}^{(k)}$

$R_{S,r_k}^{(k)}$ is the stage critical strength of bridge system at stage k supposing the failure component number is r_k . The basic formula to compute the $R_{S,r_k}^{(k)}$ is as Eq.3

$$\begin{cases} R_{S,r_k}^{(k)} = \square F_{r_k}^{(k)} + \sum_{i=1}^{k-1} \square F_{r_i}^{(i)} m_{r_i} = \mathbf{m}^{(k)} \Delta \mathbf{F}^{(k)} = \sum_{i=1}^k \beta_{R_i}^{(k)} R_{r_i}^{I_{r_i}} - \beta_G^{(k)} g \\ \mathbf{m}^{(k)} = [m_{r_1}, m_{r_2}, \dots, m_{r_{k-1}}, 1] \\ \Delta \mathbf{F}^{(k)} = [\square F_{r_1}^{(1)}, \square F_{r_2}^{(2)}, \dots, \square F_{r_{k-1}}^{(k-1)}, \square F_{r_k}^{(k)}]^T \end{cases} \quad (3)$$

where g is the acceleration of gravity.

(4) The criteria of branch and bound operation

As the name suggested, this part involves two main operations, namely, the branching operation and the bounding operation. In the branching operation, starting from an intact structure, failure is imposed at the most likely location indicated by the stage critical strength analysis of the bridge system and many potential failure elements are chosen out. This process is continued progressively till the bridge structure fails. The branching operation is carried out until all possible failure paths are exhausted.

The main purpose of the bounding operation is to discard indominant failure sequences by comparing the stage critical strength ratio and bounding parameter. This operation can help save computation time. A significant mode may be defined as one that affects the overall probability of failure (Reza & Fred 1988). In the context of second-moment formulation, a mode is significant provided it has a low safety index

compared to other modes. Alternatively, modal mean alone can be used as an indication of the significance of a mode, i.e., failure modes with low mean capacity (or mean safety margin) are considered significant. The branching and bounding criterions are shown as Eq. 4:

$$\begin{cases} R_{S(\min)}^{(k)} = \min[R_{S,r_k}^{(k)}] \\ R_{S,r_k}^{(k)} \leq c_k R_{S(\min)}^{(k)} \\ 0 \leq c_k < \infty \\ r_k [r_k \in (1, 2, \dots, n), r_k \notin (r_1, r_2, \dots, r_{k-1})] \end{cases} \quad (4)$$

where c_k is the bounding parameter, with a chosen value based on the required degree of accuracy. The components fit for the formula are the proposed failure components at stage k which are going to be saved in the failure tree as branches. If at stage k , the structure system failed, the system final critical strength for failure mode $r_1 \rightarrow r_2 \rightarrow \dots \rightarrow r_k$ could be expressed as Eq. 5:

$$Z_{r_1 \rightarrow r_2 \rightarrow \dots \rightarrow r_k} = \left(\sum_{i=1}^k \beta_{R_i}^{(k)} R_{r_i}^{I_{r_i}} - \alpha_G \beta_G^{(k)} g \right) \times F_{traffic} - F_{traffic} \quad (5)$$

2.2 Modification.

In the traditional SCSBB Method, the unit concentrated load is usually used as the standard external load which couldn't reflect the uncertainty of traffic load. So a new load model which can reflect the character of traffic load should be developed.

In order to make the SCSBB Method suit for bridge structure, the standard external load is suggested to use the model from bridge design code, which contains tow parts: a uniformly distributed load and a concentrated load. The uniformly distributed load represents the normal traffic load while the concentrated load represents the heavy truck which sometimes may overload. The uncertainty of traffic load concentrates on its random location. In order to reflect the uncertainty of load location, the modified SCSBB Method uses the influence line method to calculates the $a_{r_k}^{(1)}$ by FE program.

3. PROCEDURE OF BRIDGE DOMINANT FAILURE MODE IDENTIFICATION METHOD

A bridge may collapse in different failure modes, depending upon the combination of applied loads and the strengths of various elements. Identification, enumeration, and description of all these failure modes pose a difficult combinatorial problem, and interest in this task has varied with time. Although focus on the dominant failure modes has already simplified the problem greatly, identification of dominant failure modes is still a very complex and time-consuming process.

SCSBB Method is the core of bridge dominant failure mode identification method, but it can only choose out the potential and dominant failure elements at each damage state. In order to quickly and automatically identifying or enumerating the dominant failure modes, the proposed method is actualized in the combination with the finite element package ANSYS and the MATLAB procedure. The bridge dominant failure

mode identification method may then be summarized as Fig. 1.

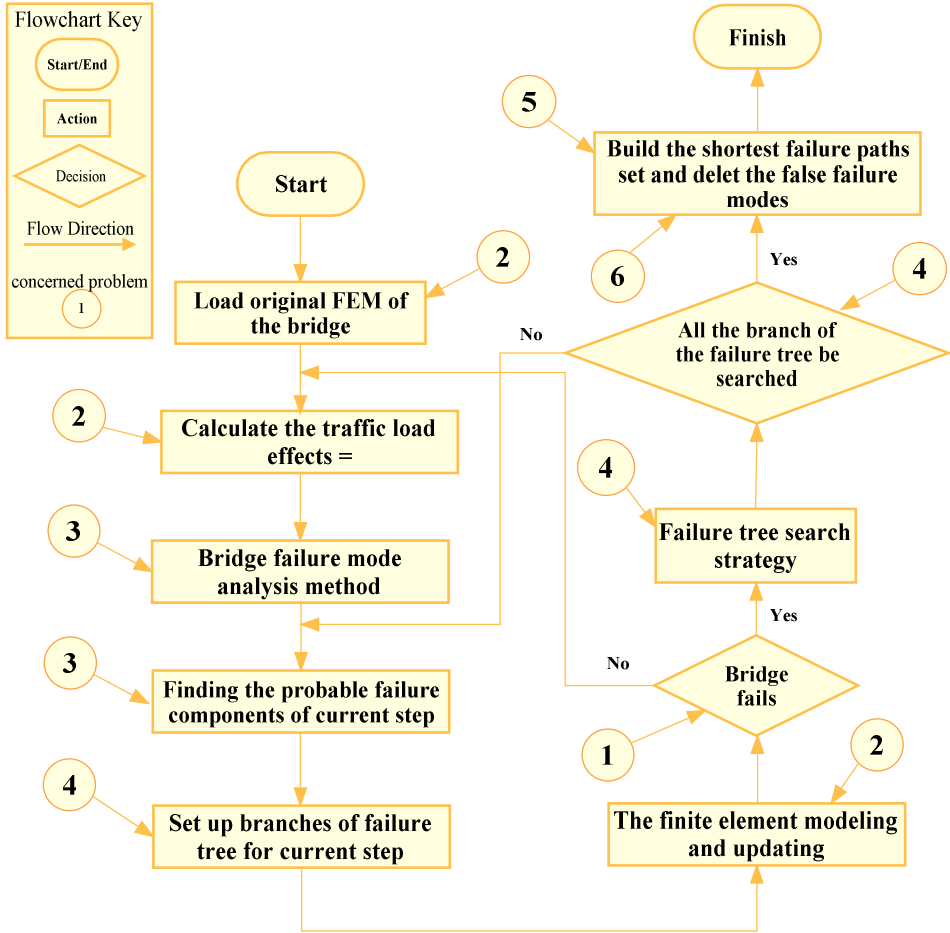


Fig. 1 Flow chart of bridge dominant failure mode identification method

At each damage state, the bridge FE model will be updated by removing the critical component. The failure tree is a graphical layout of all possible orders of component failures. At each step of bridge damage, there may be several proposed failure elements which build up the branches of a failure tree. The failure tree search strategy is used to determine which proposed failure component should be searched next. Here, the preorder traversal algorithms in data structures theory is used as the search strategy (Yan 2000). If all the components of a path are contained in any other failure paths, this path is called shortest failure path. The other failure paths can be replaced by the shortest failure path which can drastically reduce the number of traced failure paths. which proposed failure component should be searched next. Here, the preorder traversal algorithms in data structures theory is used as the search strategy (Yan 2000). If all the components of a path are contained in any other failure paths, this path is called shortest failure path. The other failure paths can be replaced by the shortest failure path which can drastically reduce the number of traced failure paths.

4. APPLICATION TO A CFST ARCH BRIDGE

4.1 Failure mode analysis of the arch rib

This section illustrates the application of the proposed bridge dominant failure mode identification method to an arch bridge in China. The bridge has 2 lanes and is 13 m in width and 138 m in span length. The finite element model by ANSYS program for the bridge system is shown in Fig.2, which shows 58 link elements for suspenders and tie bar and 6482 beam elements for other components. Suppose the other components is strong enough, here we only analysis the failure of arch rib. The section and material of the arch rib is shown in Figure. 2. The arch rib of this bridge is treated as an ideal truss structure. For practical structures, it is important to include the failure of the joints in progressive failure analysis. In this study, only member failures in tension and compression are considered. The resistance of chord member and web member are calculated according to Chen (2007) and their failure behaviors are assumed to be ductile.

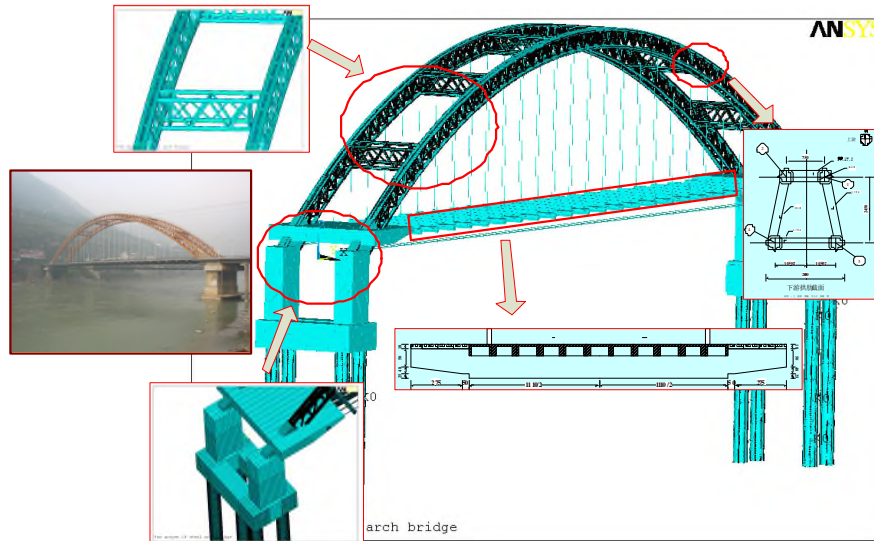


Fig.2 FEM of CFST arch bridge

The design traffic load of this bridge is Road Class II. The load pattern is offset load at the upper lane and the location of concentrated load is at the position of each hanger crossbeam (Fig. 3). Because this bridge is a symmetric structure, only half span is analyzed. By the influence line method, $a_{r_k}^{(1)}$ is calculated. The searching results of dominant failure components at first damage stage with $c_1 = 1.2$ are shown in Fig. 3.

The failure of bridge systems is defined as maximum deflection attained ($\Delta = \Delta_{\text{limit}}$), where $\Delta_{\text{limit}} = 3L/800$ and L is the span length. The bounding parameter is $c_2 = \dots = c_k = 1$. Finally, flowed the flow chart, 70 dominant failure modes are gotten. Table 1 shows the expressions and system final critical strength at different load location. Because only the dominant failure modes are needed to be calculated, time and efforts are significantly reduced in comparison to the previous permutation method.

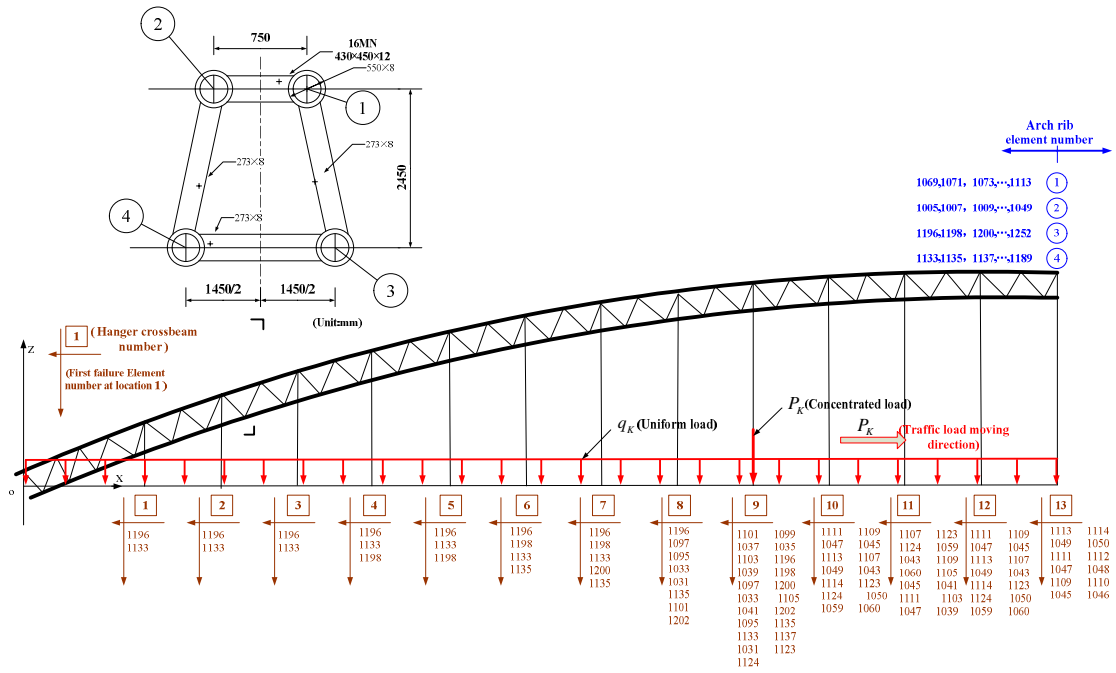


Fig. 3 The searching results of dominant failure components at first damage stage

Table 1. Expressions and system final critical strength at different load location

Location	Expression of system final critical strength $\times F_{traffic}$
1	$R_{sys} = \begin{pmatrix} -3.45 \times R_{1196}^- - 3.30 \times R_{1133}^- - 0.15 \times R_{1201}^- - 0.09 \times R_{1138}^- \\ -4.08 \times R_{1075}^- - 4.64 \times R_{1011}^- \end{pmatrix} \times 10^{-4} - 0.23g = 9.93$
2	$R_{sys} = (-2.33 \times R_{1196}^- - 2.14 \times R_{1133}^- - 3.27 \times R_{1115}^- - 3.80 \times R_{1051}^-) \times 10^{-4} - 0.17g = 7.32$
3	$R_{sys} = (-1.65 \times R_{1196}^- - 1.53 \times R_{1133}^- - 2.92 \times R_{1081}^- - 3.40 \times R_{1017}^-) \times 10^{-4} - 0.14g = 6.03$
4	$R_{sys} = (-1.24 \times R_{1196}^- - 1.14 \times R_{1133}^- - 2.74 \times R_{1085}^- - 3.24 \times R_{1201}^-) \times 10^{-4} - 0.12g = 5.31$
5	$R_{sys} = (-1.05 \times R_{1196}^- - 0.95 \times R_{1133}^- - 2.62 \times R_{1087}^- - 3.10 \times R_{1203}^-) \times 10^{-4} - 0.11g = 4.90$
6	$R_{sys} = (-0.80 \times R_{1196}^- - 0.71 \times R_{1133}^- - 2.66R_{1091}^- - 3.20 \times R_{1027}^-) \times 10^{-4} - 0.11g = 4.66$
7	$R_{sys} = (-0.65 \times R_{1200}^- - 0.49 \times R_{1135}^- - 2.81 \times R_{1093}^- - 3.41 \times R_{1029}^-) \times 10^{-4} - 0.11g = 4.62$
8	$R_{sys} = (-0.42 \times R_{1196}^- - 0.36 \times R_{1133}^- - 2.85 \times R_{1097}^- - 3.51 \times R_{1033}^-) \times 10^{-4} - 0.11g = 4.45$

Table 1. Continued

Location	Expression of system final critical strength $\times F_{traffic}$
9	$R_{sys} = \begin{pmatrix} -0.20 \times R_{1101}^- - 0.20 \times R_{1037}^- - 0.74 \times R_{1099}^- - 0.72 \times R_{1035}^- \\ +0.06 \times R_{1096}^+ - 2.30 \times R_{1198}^- - 4.03 \times R_{1135}^- \end{pmatrix} \times 10^{-4} - 0.16g = 4.84$
10	$R_{sys} = \begin{pmatrix} -1.03 \times R_{1123}^- + 0.02 \times R_{1050}^+ - 0.99 \times R_{1059}^- - 2.90 \times R_{1206}^- \\ +0.02 \times R_{1112}^+ - 4.48 \times R_{1143}^- \end{pmatrix} \times 10^{-4} - 0.16g = 5.75$
11	$R_{sys} = \begin{pmatrix} -0.38 \times R_{1107}^- - 0.62 \times R_{1123}^- - 0.39 \times R_{1043}^- - 0.57 \times R_{1059}^- \\ -0.15 \times R_{1109}^- - 0.16 \times R_{1045}^- + 0.04 \times R_{1204}^+ - 2.67 \times R_{1206}^- \\ -4.05 \times R_{1143}^- \end{pmatrix} \times 10^{-4} - 0.15g = 5.49$
12	$R_{sys} = \begin{pmatrix} -1.03 \times R_{1123}^- + 0.02 \times R_{1050}^+ - 0.99 \times R_{1059}^- - 2.90 \times R_{1206}^- \\ +0.02 \times R_{1112}^- - 4.48 \times R_{1143}^- \end{pmatrix} \times 10^{-4} - 0.16g = 5.75$
13	$R_{sys} = \begin{pmatrix} -0.90 \times R_{1111}^- - 0.20 \times R_{1112}^- - 0.89 \times R_{1047}^- - 0.18 \times R_{1048}^- \\ -2.66 \times R_{1210}^- - 0.23 \times R_{1211}^- - 4.00 \times R_{1147}^- \end{pmatrix} \times 10^{-4} - 0.14g = 5.61$

The failure sequence and critical strength are different at different load location. The failure mode with minimum system final critical strength is found at load location 8 where near 1/3 span. The minimum system final critical strength is increase from location 8 to the mid-span.

4.2 System reliability analysis of the arch rib

The axial limit state function is determined from arch rib's axial strength and external axial force. The P_f of the ultimate limit state is calculated by the FORM method. The limit state function is:

$$Z_{r_1 \rightarrow r_2 \rightarrow \dots \rightarrow r_k} = \left(\sum_{i=1}^k \alpha_{R_i} \beta_{R_i}^{(k)} R_{r_i}^{I_{r_i}} - \alpha_G \beta_G^{(k)} g \right) \times F_{g_k + p_k} - \alpha_{g_k + p_k} F_{g_k + p_k} \quad (6)$$

The probabilistic distribution and statistical parameters obtained from the literature survey and assumptions are shown in Table 2. All these random variables are assumed to be statistical independent.

Table 2. Probabilistic properties of random variables.

variable	Type	Parameter	Reference
α_{R_i}		$\mu_{\alpha_{R_i}} = 1.05, \sigma_{\alpha_{R_i}} = 0.10$	(Nowak & Cho 2007)
α_G	Normal	$\mu_{\alpha_{R_i}} = 1.0212, \sigma_{\alpha_{R_i}} = 0.0462$	(Ministry of Construction 1999)
$\alpha_{g_k + p_k}$		$\mu_{\alpha_{R_i}} = 0.6684, \sigma_{\alpha_{R_i}} = 0.1994$	

Based on the expressions of all the dominant failure modes, the reliability index of each failure mode is calculated by FORM method. The results are shown in Fig.4.

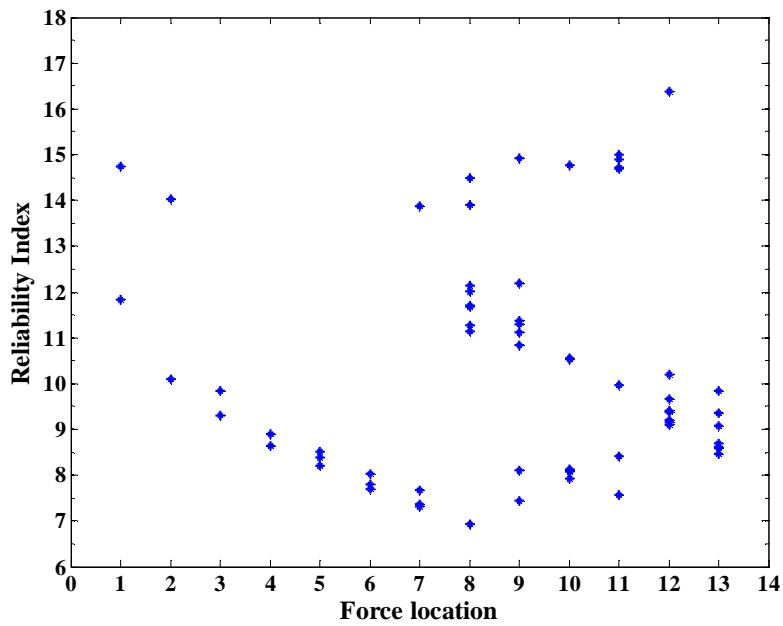


Fig.4 Reliability indices of 70 dominant failure modes

The failure mode with minimum system final critical strength is found at load location 8 and the failure sequence is shown in Fig5.

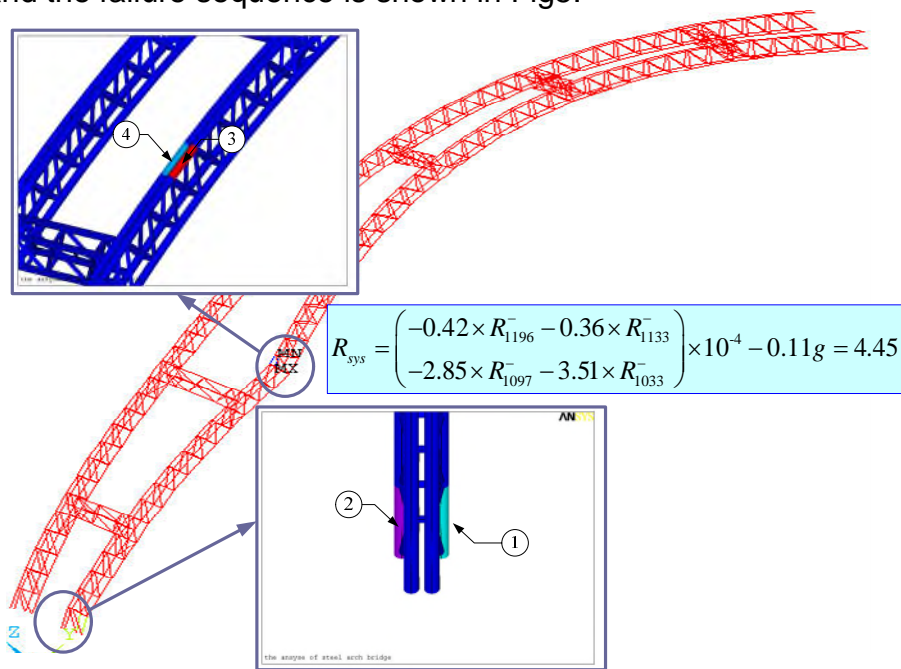


Fig.5 Failure sequences of failure modes at location 8.

The correlation coefficients between each failure mode are shown in Figure. 6. Finally, the reliability index of the bridge system is gotten by PNET method, which equals 6.72.

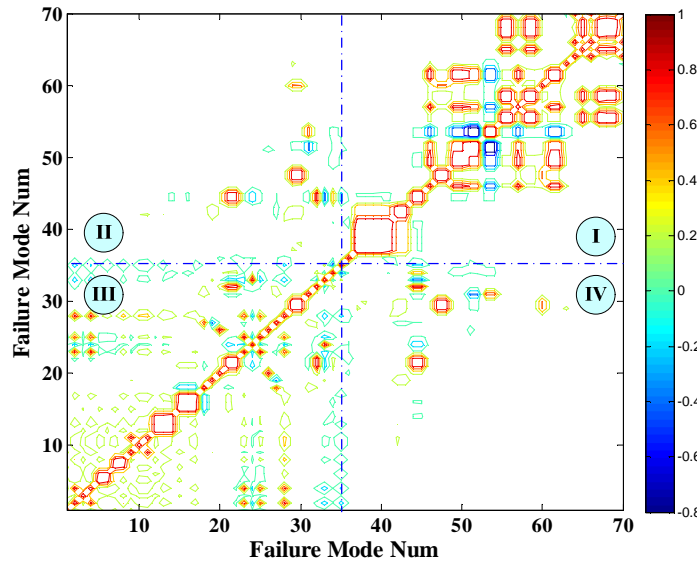


Fig.6 Contour plot of correlation coefficient of failure mode

5. CONCLUSIONS

An innovative method is proposed for the dominant failure mode identification of bridge structural systems. The suggest method is applied to a concrete filled steel tubular (CFST) arch bridges. The proposed algorithm is found to be efficient and reasonably accurate. The method overcomes the limitations of the analytical techniques. Computational effort is not wasted in enumerating a large number of failure modes, most of which may not contribute to the failure probability of the system. Meanwhile, the expressions of limit state function of dominate failure mode could be gotten, which makes the reliability index and correlation coefficient of failure mode could be easily calculated. This algorithm can be applied to any kind of bridge structural systems without having to do much additional programming for which the component failure modes can be defined through limit state equations.

6. ACKNOWLEDGEMENTS

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