Implementation of Damage Detection Algorithms for the Alfred Zampa Memorial Suspension Bridge

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ABSTRACT

This study investigated a number of different damage detection algorithms for structural health monitoring of a typical suspension bridge. The Alfred Zampa Memorial Bridge, a part of the Interstate 80 in California, was selected for this study. The focus was to implement and validate simple damage detection algorithms for structural health monitoring of complex bridges. Accordingly, the numerical analysis involved development of a high fidelity finite element model of the bridge in order to simulate various structural damage scenarios. The finite element model of the bridge was validated based on the experimental modal properties. A number of damage scenarios were simulated by changing the stiffness of different bridge components including suspenders, main cable, bulkheads and deck. Several vibration-based damage detection methods namely the change in the stiffness, change in the flexibility, change in the uniform load surface and change in the uniform load surface curvature were employed to locate the simulated damages. The investigation here provides the relative merits and shortcomings of these methods when applied to long span suspension bridges. It also shows the applicability of these methods to locate the decay in the structure.

1. INTRODUCTION

Structural health monitoring of bridges ensures the safety of the structure by providing valuable information about any existing anomalies, damage or deterioration in the bridge [1]. Visual inspections and nondestructive test methods can be effective in detecting anomalies [2]. However, a more efficient approach is global monitoring provided that it yields sufficient information. Among these methods are vibration based techniques. The basic idea behind these methods is that the modal parameters (namely frequencies, mode shapes, and modal damping) are functions of the physical properties of the structure (mass, damping, and stiffness). Therefore, changes in these properties will cause detectable changes in the modal properties. There are some damage detection methods which are based on the mode shapes or their derivatives such as: Enhanced Coordinate Modal Assurance Criteria (ECOMAC) [3]; Damage Index (DI) method [4], Mode Shape Curvature (MSC) method [5]; Change in modal flexibility matrix [6]; changes in stiffness matrix [7]; changes in the uniform load surface and changes in the curvature of uniform load surface [8].

This study aims at the numerical investigation of some mode shape based techniques, namely change in the modal flexibility matrix, change in the stiffness matrix, change in the uniform load surface and change in the curvature of uniform load surface for the detection of structural damage. This study concentrates on the finite element modeling of a suspension bridge with the goal of finding the relative advantages and drawbacks of the mode shape based techniques.

Evaluation of the damage assessment methods in this study involved development of the finite element model for the Alfred Zampa Memorial Suspension Bridge in California. The finite element model of the bridge is validated based on the experimental modal properties obtained from the ambient vibration measurements. A brief description of the damage assessment techniques for the detection of simulated damages in this bridge follows the finite element model. It is concluded that these methods are to some extend capable of detecting the decay in the structure. These methods implemented in a MATLAB code along with ambient vibration data from health monitoring system can be used to continuously assess the health of the structure.

2. FINITE ELEMENT MODELING OF THE BRIDGE

The Alfred Zampa Memorial Bridge also known as the New Carquinez bridge is the first new suspension bridge built in the United States since 1973. The bridge connects Vallejo to Crockett in California. The length of the main span of the bridge and total length of the bridge are 728 m and 1056 m, respectively. The bridge includes two main concrete towers, two suspension cables and an orthotropic steel box girder deck. Cables are clamped to the deck at the middle of the span and anchored to the rock at both ends of the bridge. Concrete towers include two columns with two struts connecting them at the top of the columns and bottom of the deck. The deck is connected to the bottom struts with rocker links. Each tower is seated on top of two pile foundations.

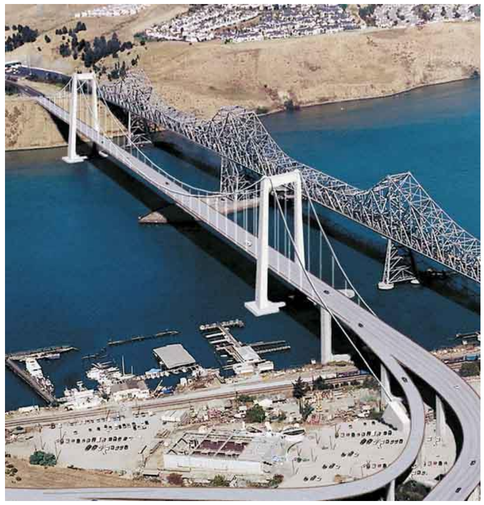


Figure 2-1. A view of the Alfred Zampa Memorial Bridge (courtesy of Metropolitan Transportation Commission)

A detailed FE model of the bridge was developed in ADINA software [9]. The towers, piers, foundation piles and main suspension cables were modeled with elastic beam elements. The suspension ropes were modeled with truss elements. Nonlinear force-displacement behavior was assigned to the 3D truss elements in order to accurately model the soil/rock properties around the piles. The steel box girder deck of the suspension bridge was modeled explicitly using multi-layer shell elements. The steel box girder deck of the suspension bridge was modeled using isotropic elastic shell elements and the ribs were modeled using orthotropic shell elements. A cross-section of the steel box girders and the bulkhead details are shown in Figure 2-3. The construction sequence was included in the FE model of the bridge using an in-house developed program SC-Cable [10]

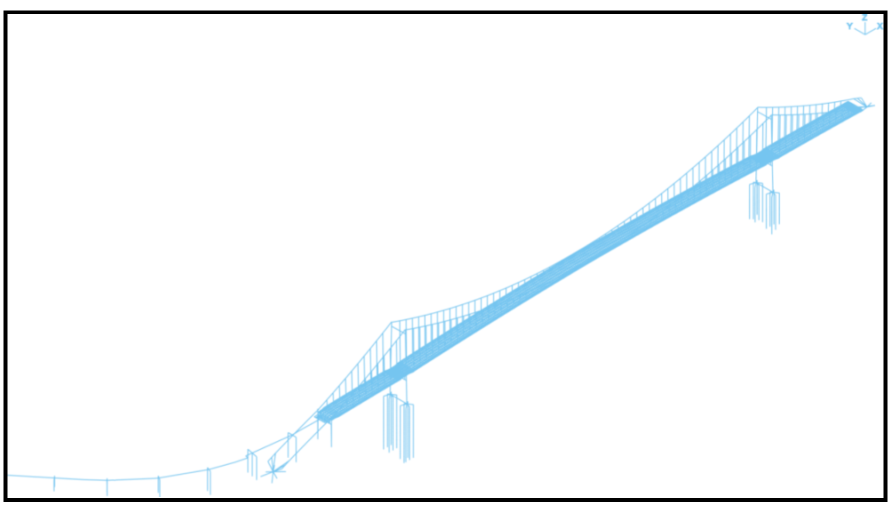


Figure 2-2. Finite Element Model of the Alfred Zampa Memorial Bridge

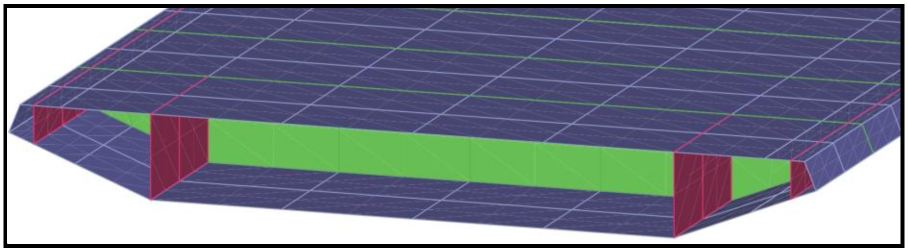


Figure 2-3. Finite Element Model of the Deck

3. FE VALIDATION BASED ON MEASURED MODAL PROPERTIES

A health monitoring system based on wireless sensors was designed and installed on the bridge to continuously measure its vibration response (Figure 3-1). Each sensing unit consists of a tri-axial accelerometer, an on-board analog-to-digital convertor for data collection, a Texas Instrument transceiver, and a rechargeable battery pack (Figure 3-2). Sensor data was collected by the closest server and transferred to a remote database via a wireless cellular modem. An internet-enabled cyber infrastructure framework handles transfer, maintenance and access to the data from the database server. The recorded accelerometer data was used to extract modal properties of the bridge. Table 3-1 compares the frequencies of the bridge extracted from sensors data with those from the FE model of the bridge. In general, there is good agreement between the measured and calcualted frequencies which shows the accuracy of the finite element model.

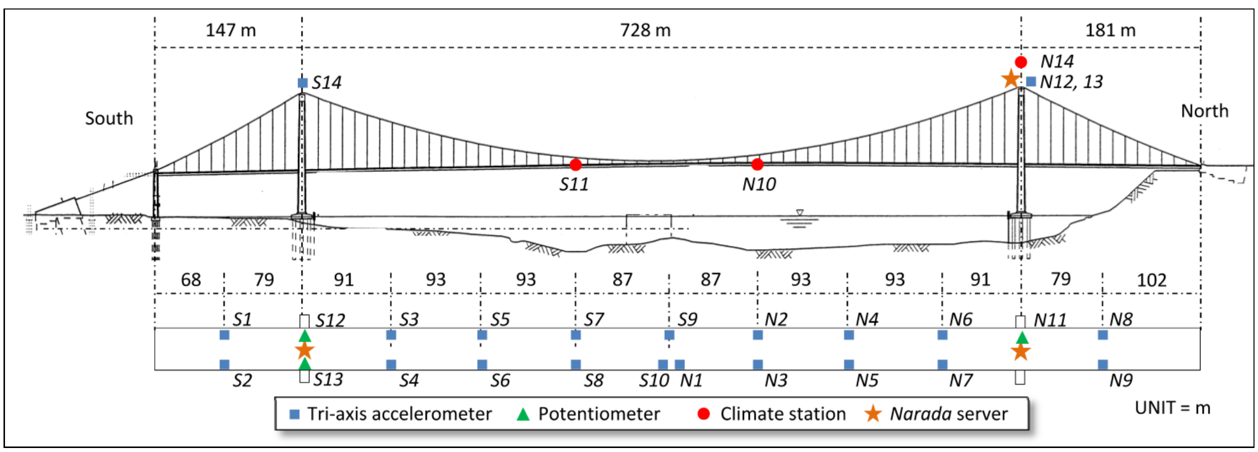


Figure 3-1: Bridge instrumentation

Figure 3-2: components of structural health monitoring system

Table 3-1: Measured and calculated frequencies of the bridge

Mode number	1	2	3	4	5
Measured Frequencies	0.194	0.260	0.351	0.413	0.487
FE Frequencies	0.195	0.256	0.347	0.394	0.459

4. DAMAGE DETECTION PROCEDURE

The possibility of evaluating the structural behavior of bridges from their dynamic response has motivated the development of damage detection methods. Among these methods are those based on the change of modal properties of the bridge due to the damage. Figure 4-1 shows the damage detection procedure based on measured model properties. Two sets of modal properties are used to estimate the location and intensity of the damage in the structure. The baseline modal properties represent the current/healthy state of the bridge which can be extracted from calibrated numerical model of the bridge. Continuous measurements of the bridge vibration provides the second set of modal properties which may indicate some damages in the structure once compared with baseline properties. These two sets of modal properties are used as inputs to the damage detection algorithms to locate the damage in the bridge structure. In this section, four different damage detection algorithms which will be used to locate the simulated damage scenarios in the Alfred Zampa Memorial suspension bridge are presented. Since no damages have been observed in the structure, the numerical model of the bridge is used to provide both sets of modal properties (baseline and damaged modal properties). The objective is to show the applicability of these methods, their relative merits and shortcomings when applied to the suspension bridges. It will be shown that these methods are capable of detection of decay in the structure. Having installed a health monitoring system on the bridge, the modal properties can be continuously extracted and used for real-time damage detection.

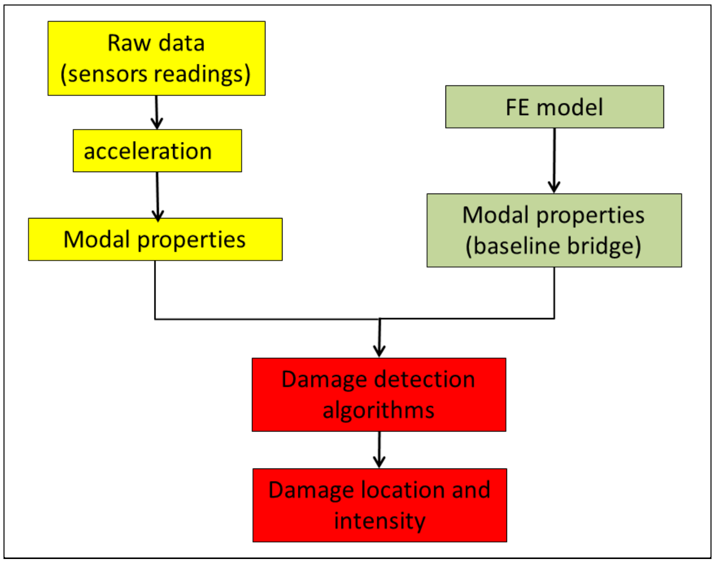


Figure 4-1: Damage detection procedure

The following damage identification methods including the change in modal flexibility, change in modal stiffness, change in uniform load surface and change in uniform load surface curvature are used for this study.

- Change in Modal Flexibility

Change in the flexibility matrix of the structures due to damage has been introduced as a method to identify damage presence as well as its location [6]. With the mode shape normalized to the mass matrix, the flexibility matrix of the damaged and baseline structure can be estimated as:

$$F = \Psi * \Omega^{-1} * \Psi^{T} = \sum_{i=1}^{N} \frac{1}{\omega^{2}} \psi_i \psi_i^{T} \tag{1}$$

$$F^{*} = \Psi^{*} * \Omega^{*-1} * \Psi^{*T} = \sum_{i=1}^{N} \frac{1}{\omega^{*2}} \psi^{*}{}_i \psi^{*T}{}_i \tag{2}$$

Here ω^* and ω are the i[th] modal frequencies of the damaged and baseline structure, $\Psi = [\psi_1,...,\psi_N]$ is the mode shapes matrix and $\Omega = [\omega_1,...,\omega_N]$ is the modal frequencies matrix. From these flexibility matrices, changes in the flexibility matrix can be obtained as:

$$\Delta F = F - F^* \tag{3}$$

- Change in Modal Stiffness

Change in the stiffness matrix of the structures due to damage can be used to identify damage presence. With the mode shape normalized to the mass matrix, the stiffness matrix of the damaged and baseline structure can be estimated as:

$$K = \sum_{i=1}^{N} \omega^2 \psi_i \psi_i^T \tag{4}$$

$$F^* = \sum_{i=1}^{N} \omega^{*2} \psi^*{}_i \psi^*{}_i^T \tag{5}$$

Here ω^* and ω are the i[th] modal frequencies of the damaged and baseline structure, $\Psi = [\psi_1,...,\psi_N]$ is the mode shapes matrix and $\Omega = [\omega_1,...,\omega_N]$ is the modal frequencies matrix. From these stiffness matrices, changes in the stiffness matrix can be obtained as:

$$\Delta K = K - K^* \tag{6}$$

- Change in uniform load surface

This method proposed by Zhang and Aktan [8] is based on flexibility matrix computation. It is defined as:

$$F_j = \sum_{i=1}^{n} f_{ji} \tag{7}$$

$$\Delta F = \left| F_j^* - F_j \right| \tag{8}$$

where $F^*{}_j$ and F_j denote the summation of the j[th] row of the flexibility matrix for damaged and baseline structures.

- Change in uniform load surface curvature

This method proposed by Zhang and Aktan [8] is based on the flexibility matrix computation. It is defined as:

$$F_j^{"} = \sum_{i=1}^{n} f_{ji}^{"} \tag{9}$$

$$\Delta F^{"} = \left| F_j^{*"} - F_j^{"} \right| \tag{10}$$

where $F^{*"}{}_j$ and $F_j^{"}$ denote the summation of the j[th] row of the curvature of the flexibility matrix for damaged and baseline structures.

5. DAMAGE SCENARIOS

The damage scenarios considered in this study are shown in Table 5-1. Here damage is defined as a reduction in the modulus of elasticity of the bridge components. Scenario D1 and D4 simulates failure in the suspenders of the main span. D9 simulates the damage in the main cable of the bridge at the mid span. D10 simulates the damage in the tower leg. D11 and D12 simulate the damage in the bulkhead and deck of the bridge respectively. Different damage intensities were also considered for two cases as shown in Table 5-1. To investigate the capability of damage detection methods to estimate the decay in the structure, in one case, the intensity of the damage was increased from 50% to 100% for one of the suspenders and in another case, the intensity of the damage in the deck was increased from 25% to 90%.

Table 5-1: Damage scenarios

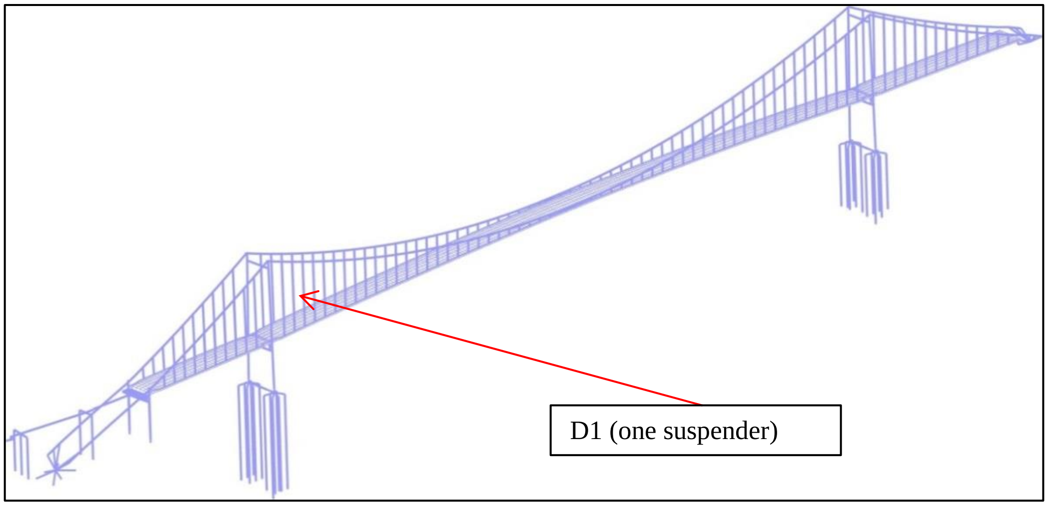
Damage Scenario	location	Damage Type	Damage Intensity
D1	main span-south	one suspender	100%
D4	main span-south & north	four suspenders	100%
D9	main span- mid-span	main cable	90%
D10	south	tower leg	90%
D11	main span- mid-span	bulkhead	90%
D12	main span- mid-span	deck	90%
D1	main span-south	one suspender	100%-90%-75%-50%
D12	main span- mid-span	deck	90%-75%-50%-255

6. RESULTS

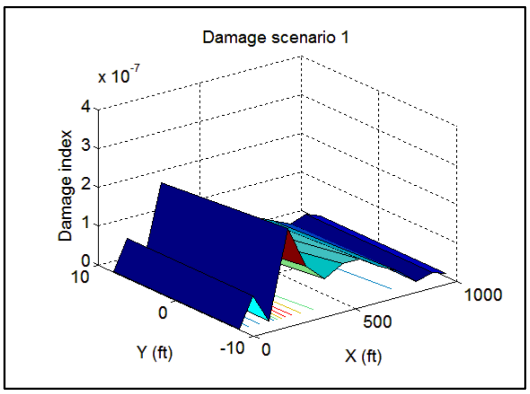
Damage scenarios D1 and D4 simulate damage in the suspenders. All four methods were able to locate the damage in the structure as shown in Figure 6-1 and Figure 6-2. The only method which could locate the damage in the main cable (D9) was the change in the uniform load surface method (Figure 6-3). None of the methods were able to locate the damage in the tower leg (D10). The damage in the tower leg significantly changes the global behavior of the structure and introduces changes in the mode shapes at all the locations which makes it difficult to specify the damage location (Figure 6-4). D11 simulates the damage in the bulkhead. The change in the stiffness and flexibility methods were able to detect the simulated damage in the bulkhead (Figure 6-5). All the methods were able to locate the damage in the deck as seen in Figure 6-6. The uniform load surface was able to locate the decay in two cases as shown in Figure 6-7 and Figure 6-8.

Table 6-1: Damage detection analysis results

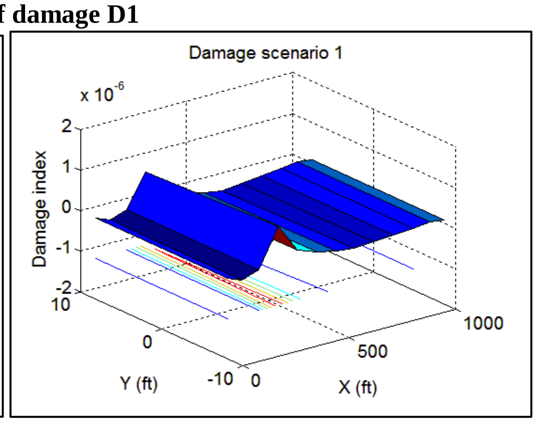
Damage Scenario	Flexibility method	Stiffness method	Uniform load surface method	uniform load surface curvature method
D1	O	O	O	O
D4	O	O	O	O
D9	X	X	O	X
D10	X	X	X	X
D11	O	O	X	X
D12	O	O	O	O



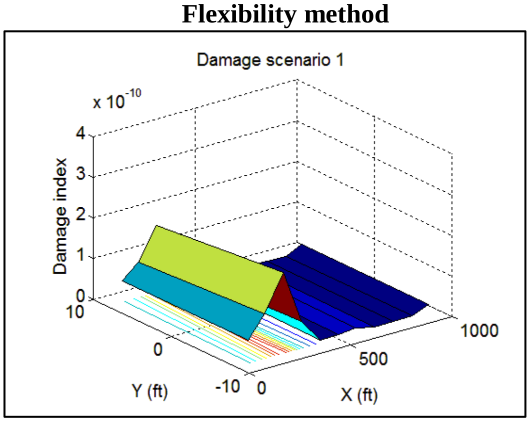
Location of damage D1



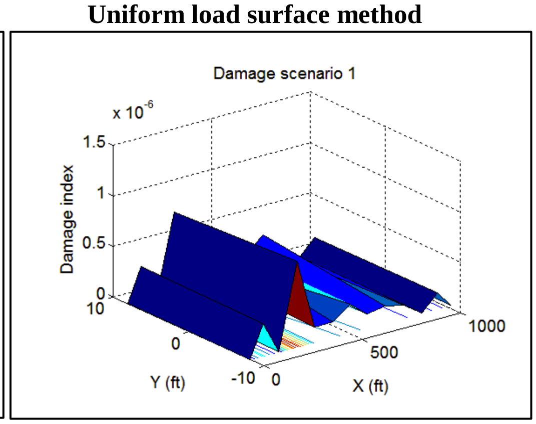
Flexibility method



Uniform load surface method

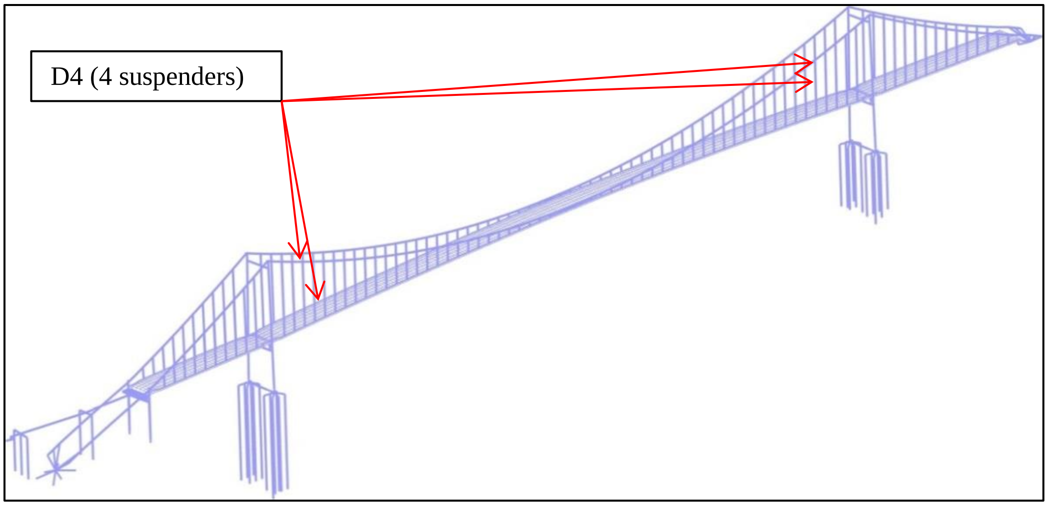


Uniform load surface curvature method

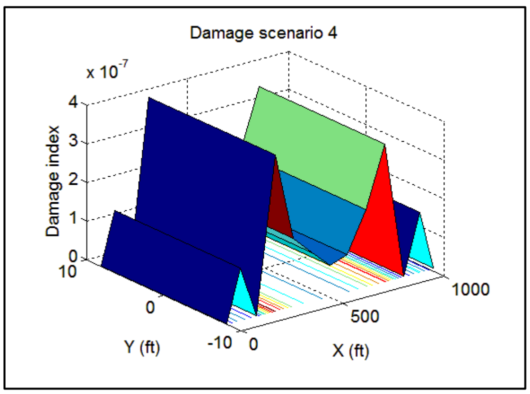


Stiffness method

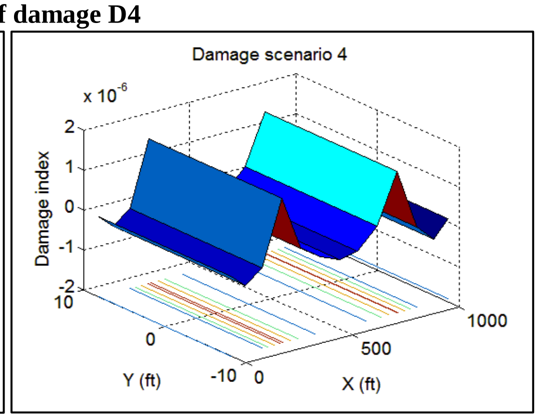
Figure 6-1: Results for damage scenario D1



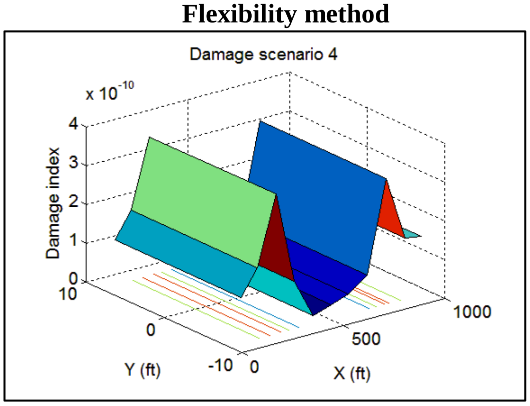
Location of damage D4



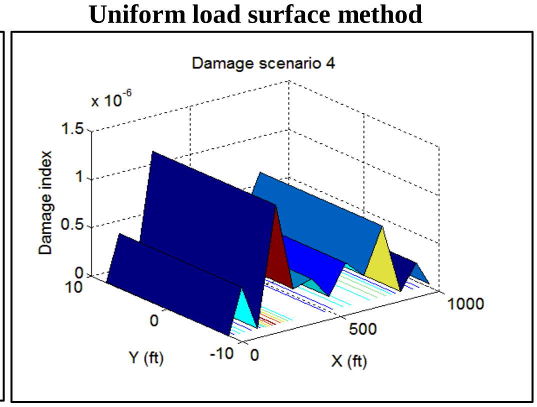
Flexibility method



Uniform load surface method

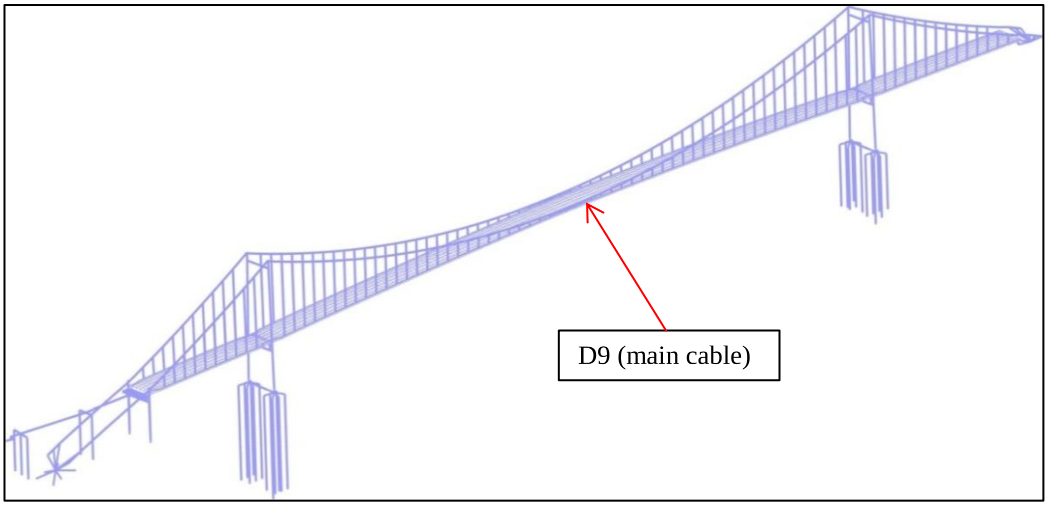


Uniform load surface curvature method

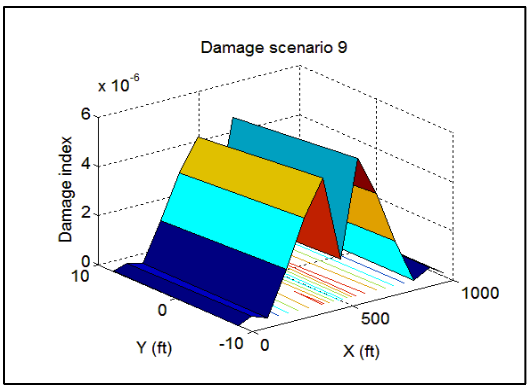


Stiffness method

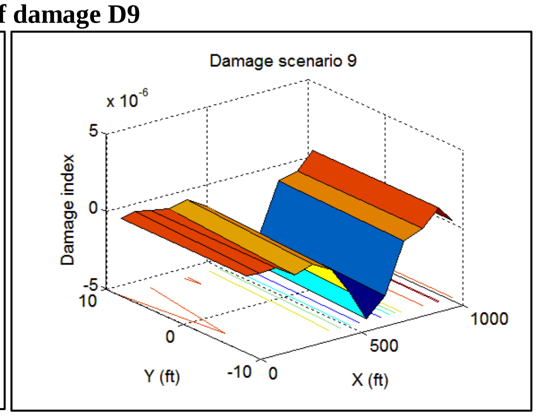
Figure 6-2: Results for damage scenario D4



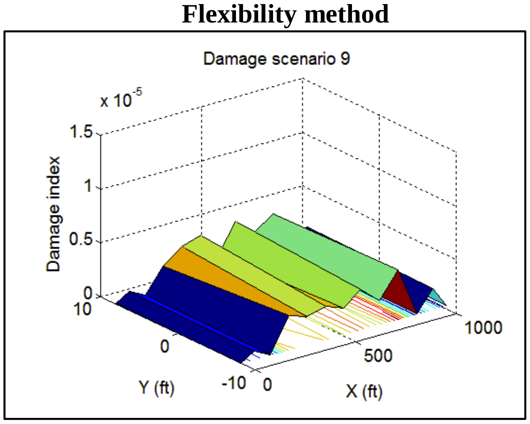
Location of damage D9



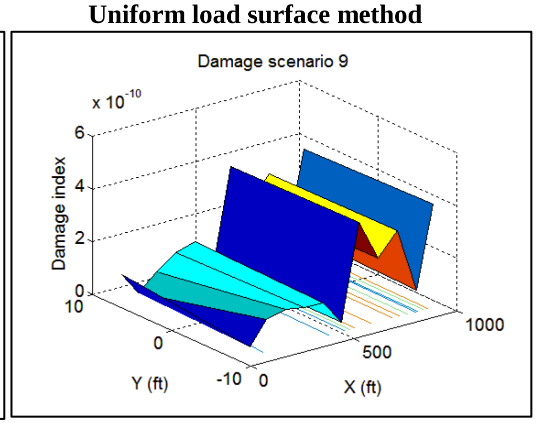
Flexibility method



Uniform load surface method

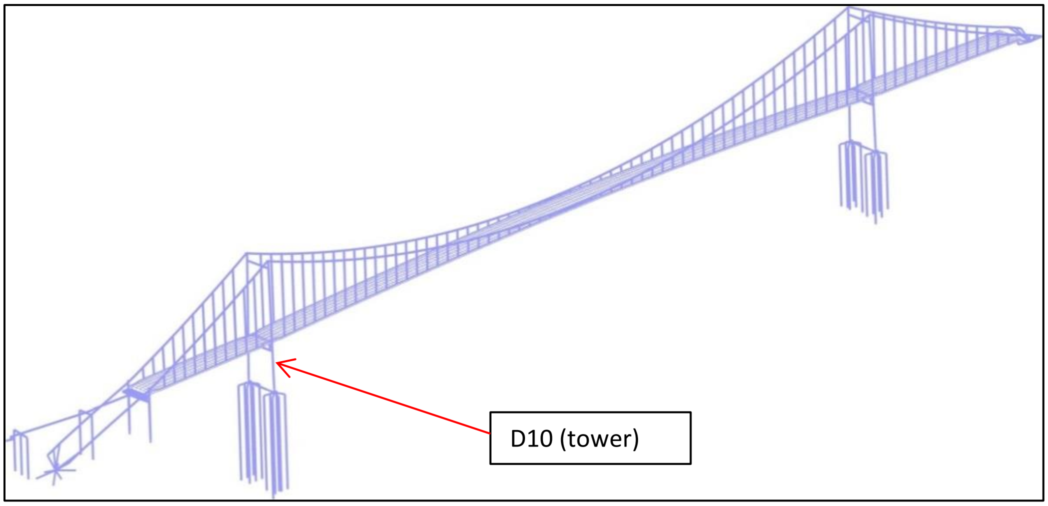


Uniform load surface curvature method

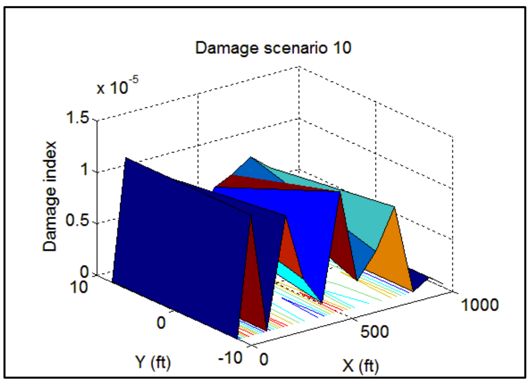


Stiffness method

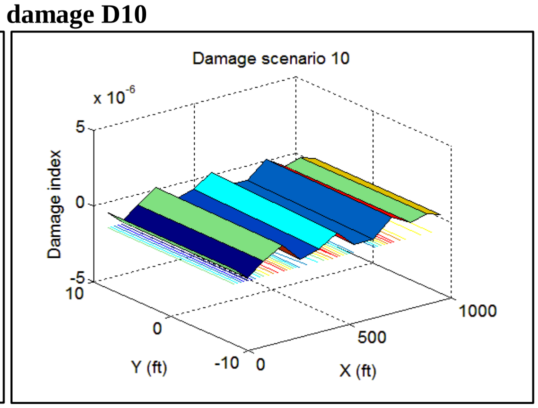
Figure 6-3: Results for damage scenario D9



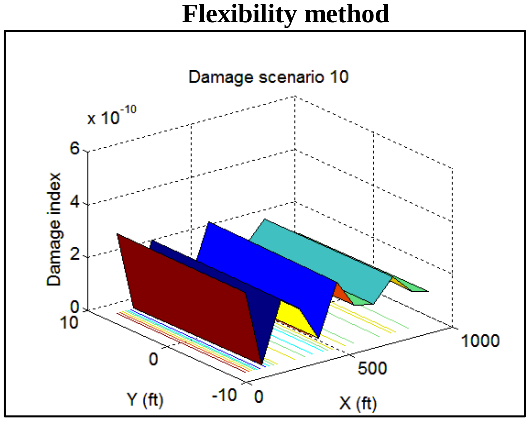
Location of damage D10



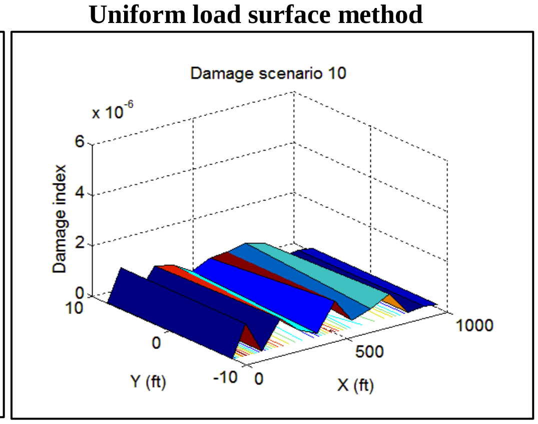
Flexibility method



Uniform load surface method



Uniform load surface curvature method



Stiffness method

Figure 6-4: Results for damage scenario D9

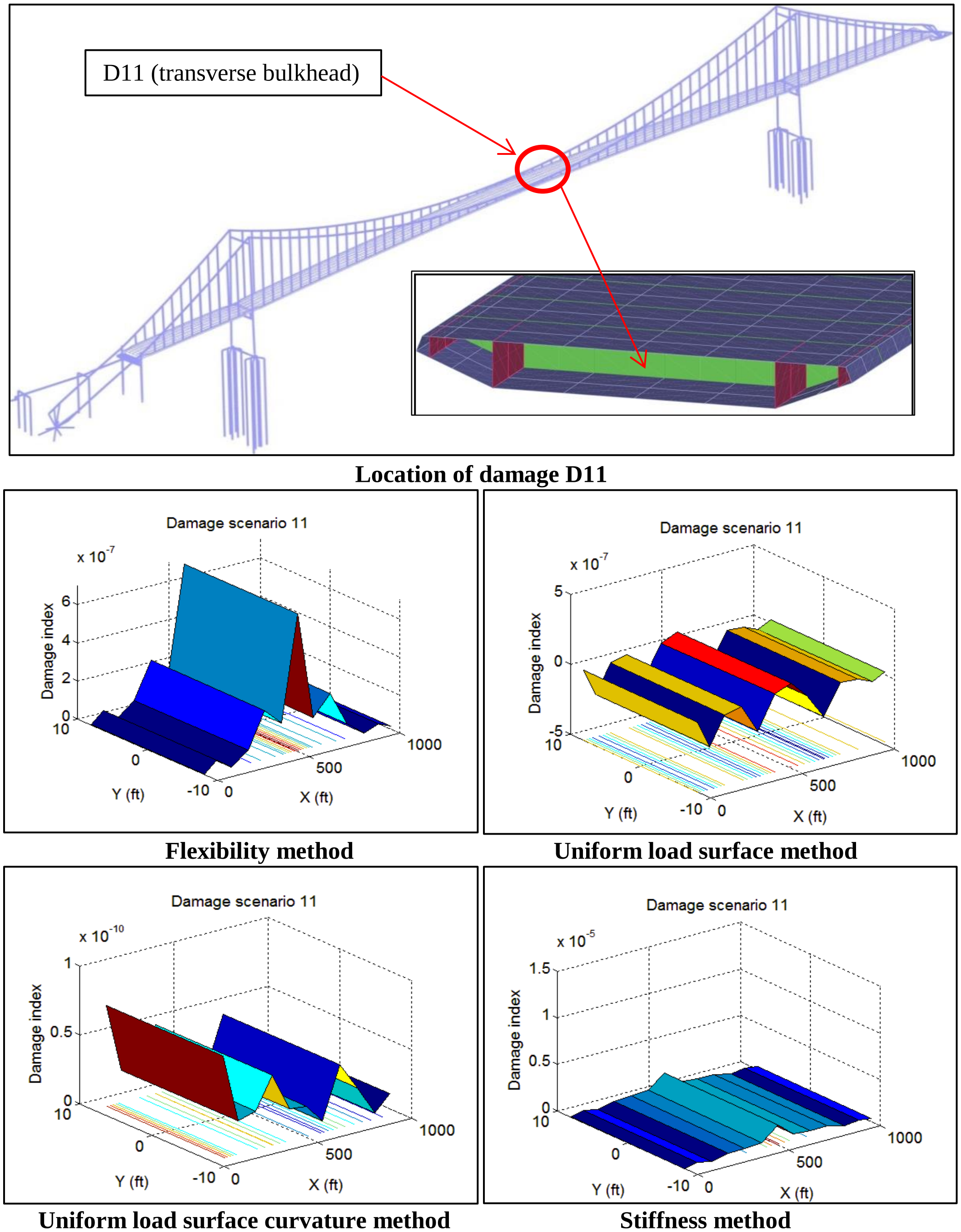
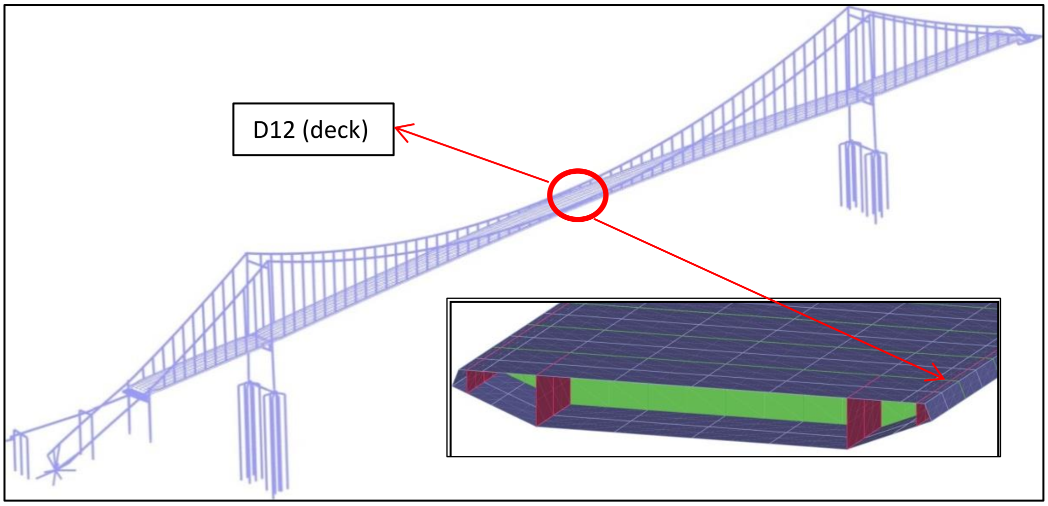
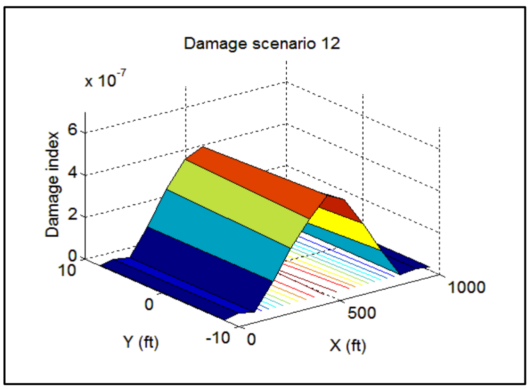


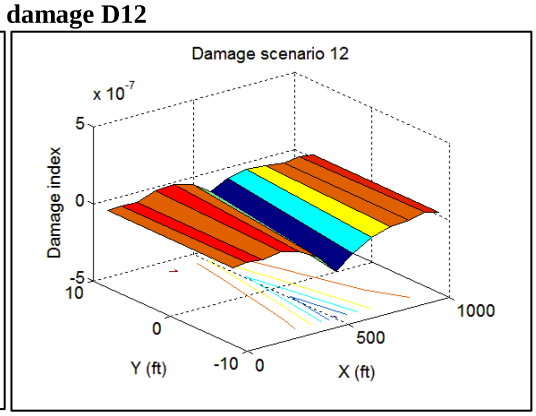
Figure 6-5: Results for damage scenario D11



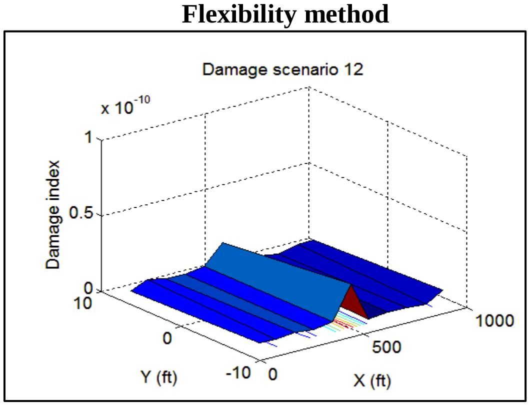
Location of damage D12



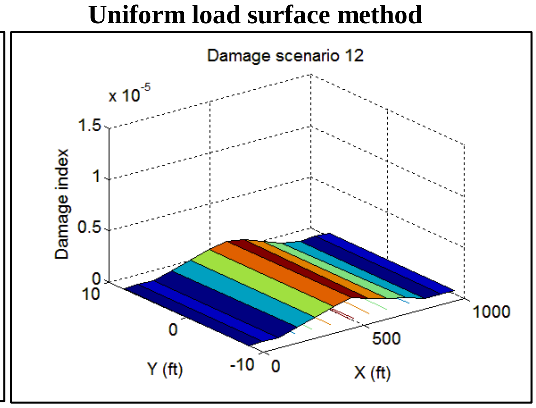
Flexibility method



Uniform load surface method

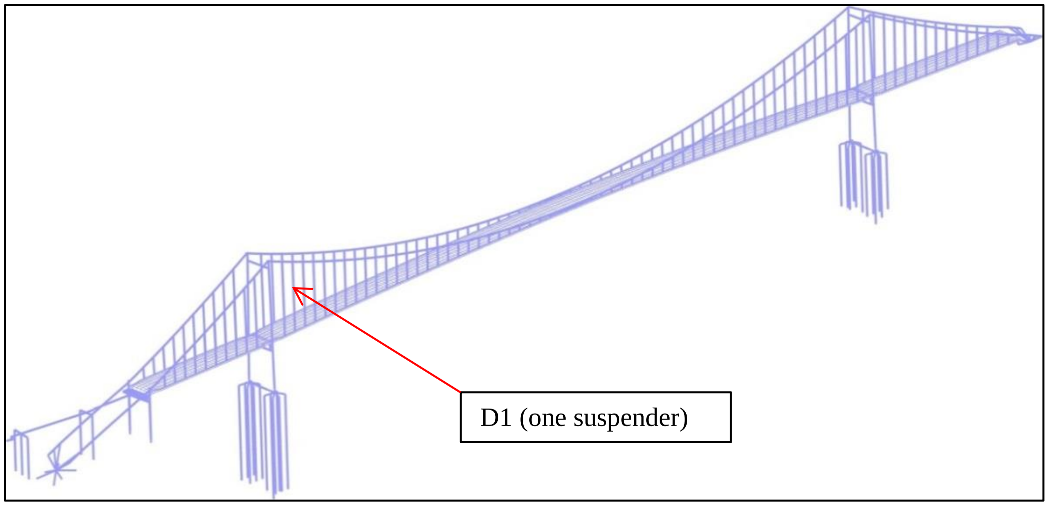


Uniform load surface curvature method

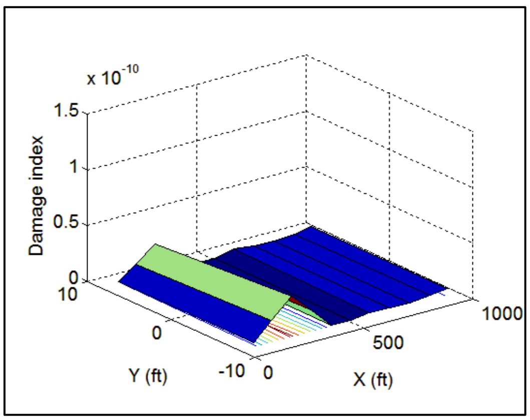


Stiffness method

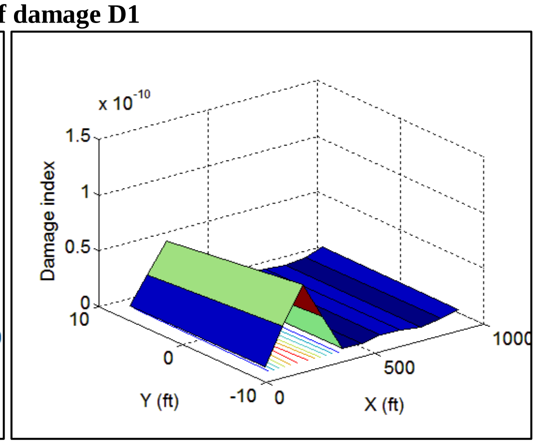
Figure 6-6: Results for damage scenario D12



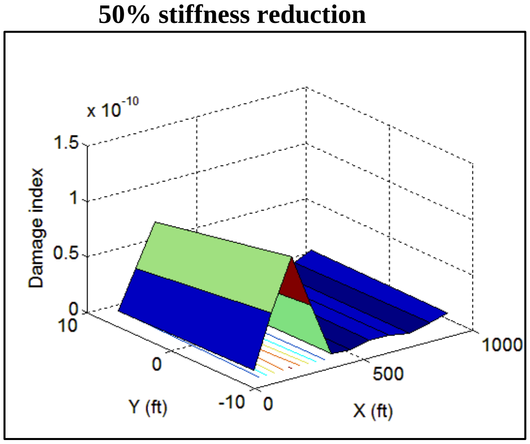
Location of damage D1



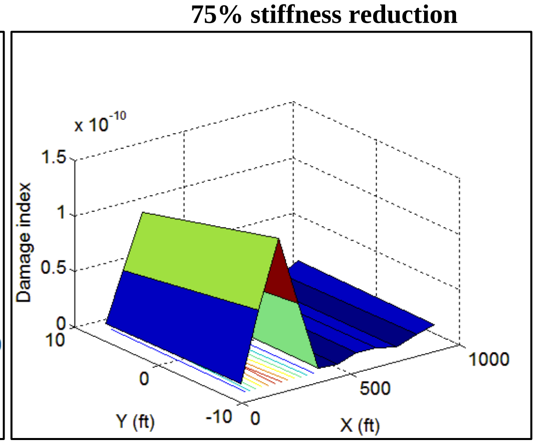
50% stiffness reduction



75% stiffness reduction

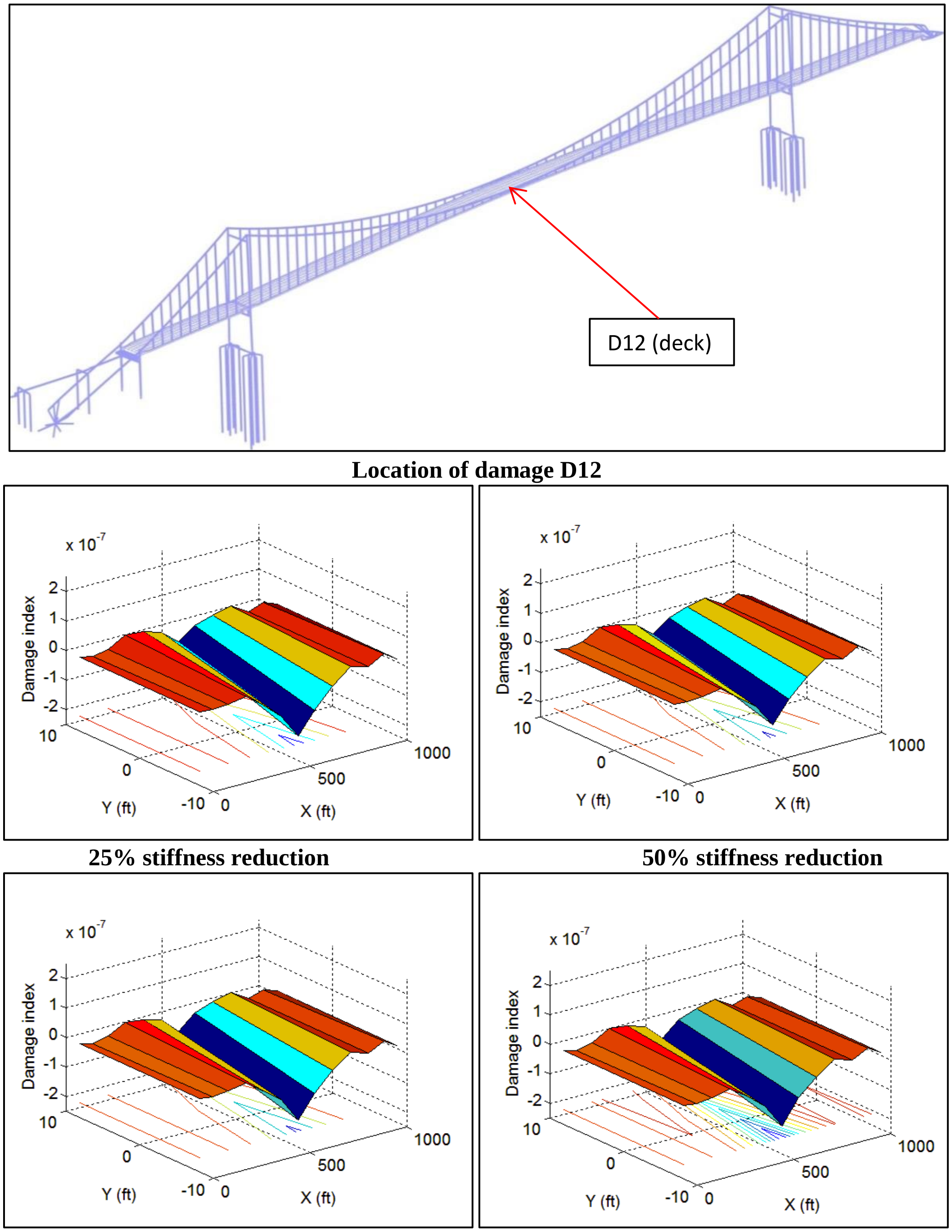


90% stiffness reduction



100% stiffness reduction

Figure 6-7: Results for damage scenario D1 (different damage intensities)



Location of damage D12

25% stiffness reduction

50% stiffness reduction

75% stiffness reduction

90% stiffness reduction

Figure 6-8: Results for damage scenario D12 (different damage intensities)

7. CONCLUSIONS

A numerical study of applying four different vibration based damage detection algorithms on the Alfered Zampa suspension bridge was done in this paper. Changes in the flexibility matrix, changes in the stiffness matrix, and changes in the uniform load surface and uniform load surface curvature were used to locate damage in six scenarios simulated in different locations of the bridge. Damages were simulated by changing the stiffness of different bridge components including suspenders, main cable, bulkheads and deck. The finite element model of the bridge was validated based on the experimental modal properties obtained from ambient vibration test. The maximum error between measured and numerical modal frequencies was less than 6%. This model was used to generate mode shapes and natural frequencies for both damaged and baseline structure. These data were used as an input to the damage detection algorithms to locate the damage in the structure. It was concluded that all methods performs relatively well for most of the cases. The change in the uniform load surface method was also able to detect the decay in the suspenders and bridge deck even with low damage intensity of 25%. Damages in the deck and suspenders seemed to be easily detected using all methods. However, damage in the tower leg could not be detected by any of damage detection methods. These methods can be used to continuously monitor the health of the structure provided that the real time modal properties can be extracted from the health monitoring system installed on the bridge.

8. ACKNOWLEDGMENTS

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