An Iterative Method for Structural Health Monitoring in a Jacket Type Offshore Platform Based on Mode Reduction

Alireza Mojtahedi and Farhad Hosseinlou

Abstract—**Structural safety assessment is one of the most important items in extraction of energy resources by using offshore structures. Despite uncertainty in determining the most important parameters for the structure final design, it is usually complicated. Thereupon, damage detection techniques have received significant attention in order to assess the safety and reliability of offshore structures during their service life. This research represents the cross-model cross mode (CMCM) method in combination with the two-stage proprietary reduction (TPR) technique that is capable of detecting the damage to individual members by using results of the experiment on physical model of the offshore jacket platforms, when limited, spatially incomplete modal data is available. We evaluated selection procedure inactive degrees of freedom in process of the model reduction with a reasonable criterion by using the sensitivity analysis of system response under base excitation. Meanwhile, the finite element model updating based on the empirical model utilized to overcome the uncertainty in modeling. This performance indicates that the convergence rate and the compu-ting time of the proposed method are significantly superior to those of the prior iterative method with or without noise.**

Index Terms—**Offshore jacket platforms, damage detection model updating, model reduction, sensitivity analysis.**

I. Introduction

Jacket-type offshore platforms are by far the most common kind of offshore structures and they play an important role in oil and gas industries in shallow and intermediate water depth. As jacket structures require more critical and complex designs, the need for accurate considerations to determine uncertainty and variability in analytical models, loads, geometry, and material properties has increased significantly. Also, damage assessment and detection of a Jacket structure in a timely fashion are required in order to identify probabilistic damages and to ensure safety.

As always, marine structures during their service life continually accumulate damage as a result of the action of various environmental forces. These defects include the effects of fatigue, corrosion members, vessels collision, falling objects from the deck of the platform, the forces result of severe storms and installation and maintenance activities that could lead to a reduction in the modulus of elasticity of structural members [1]-[4]. Clearly the development of robust techniques for primary damage detection is very effective in the prevention of the catastrophic structural decay. Since the cumulative damage may cause the change on the stiffness distribution of the structural system, consequently the modal properties of the structural system, such as natural frequencies and mode shapes, may alter as well. Finite-element-model updating has been used to detect damage in structures. Damage detection methods using updated techniques based on modal parameters can be divided into two parts: direct and iterative methods. The direct methods solve for the updated matrices by forming a constrained optimization problem. The differences of various direct methods are due to either the variation of the selected objective function to be minimized, the constraints placed on the problem via the Lagrange multiplier technique, or numerical scheme used to implement the optimization. The excellence of direct methods is that they are computationally straightway and efficient; they are not required addressing the problem of whether the solution converges because the result of the computation is unique. However, because of changes in the connectivity of dynamic matrices in the mathematical model during the updating process, the physical meanings of the original practical structures cannot be preserved. In the iterative methods, the basic procedure consists in solving an optimization problem, in which the discrepancies between the analytical and measured dynamic characteristics are minimized by adjusting the unknown model properties. In contrast to direct methods, iterative updating methods preserve physical meanings. The connectivity of the original dynamic matrices that belong to the mathematical models is maintained even after iterative updating. Therefore, the major advantage of iterative over direct methods is their ability to maintain the initial correspondence between the degrees of freedom (DOFs) within the dynamic matrices of the practical structures.

Typically, the dynamic characteristics employed are modal parameters, including both modal frequencies and mode shapes, and the unknown model properties are the updating parameters which commonly are the dimensionless correction factors for each element [5]-[7]. The focus of the present paper is on updating the stiffness matrix to extend the CMCM method to detect damage and quantify the severity for fixed offshore platforms (3D space frame) under the assumption that only the first few lower-order modal parameters have been identified. Damage detection methods are based on the fact that any change in stiffness caused by defect in a structure leads to an alteration in modal parameters of the structure such as natural frequencies and mode shapes. The change of modal parameters can be used as the basis for these methods. For instance, the ratio of the modal frequency change between any two modes was used as a damage index [8], [9]. The techniques for damage detection are generally classified into four levels and commonly

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accepted as follows [10]: Level 1–Determination that damage is existing in the structure, Level 2 – Level 1 beside detection of the geometric location of the damage, Level3–Level2 beside quantification of the severity of the damage, Level 4–Level 3 beside prediction of the remaining service life of the structure. It is well known that the natural frequencies can be obtained from any specific response which is recorded anywhere on a jacket. Also, the natural frequencies can be extracted exactly. Consequently, many vibration-based damage detection methods for offshore structures have been proposed, such as sensitivity analysis methods, perturbation methods, modal strain energy decomposition method and artificial neural networks methods [11]-[15]. Those methods were represented to be more effective on localizing damaged members, but determining the damage severity for offshore structures remained a challenge in practice. This paper develops a direct, physical property adjustment model updating method, named as cross-model cross-mode (CMCM) method. This method is capable of updating stiffness matrices based on only a few modes of the damaged structure [6], [7], [16]. Another very important capability is that it does not require to pair the measured and analytical modes. We are facing two major challenges in the damage detection for situ platforms by using the CMCM method: the lack of coordination of measurement sensors and degrees of freedoms (DoFs) of the analytical model, namely the spatial incompleteness and the noisy data measurement. In dealing with spatially incomplete situations, we can be used model reduction scheme. The assessment methods are included the frequency domain methods and the time domain methods. The prior is based on an alteration in the modal parameters with respect to an alteration in the system parameters, whereas the second is usually based on the relationship between the responses of the structural system and the excitations. The response sensitivity based method was introduced by Jahn in 1940s, and the sensitivity method for damage detection has been investigated widely since [17], [18]. Lu and Law proposed the sensitivities of the response under various dynamic excitations, and a sensitivity-based method was presented to identify the local damages [19]. This research represents the CMCM method in combination with the two-stage proprietary reduction (TPR) technique, to detect damage in an offshore jacket platform model using spatially incomplete and with noise in modal data. In imposing the proposed method, the reduced stiffness matrix based on the Guyan reduction Scheme along with decreasing the analytical model relies on the sensitivity analysis is applied. Another crucial feature is, implementing experimental modal testing on a laboratory physical model to evaluate the utilized finite element model in verifying of the proposed method. Vibration phenomena have always been a cause of concern to engineers, even more so today as structures are becoming lighter and more flexible due to increased requests for efficiency, safety and tranquility. The effects of vibration present significant hazards and operating limitations ranging from discomfort, malfunction, reduced performance, early damage and structural failure which, in the worst case, can be catastrophic. It is clear that a thorough understanding of the existing vibration levels in service is essential. Accordingly, accurate analytical models of marine structures are required to explain the vibration characteristics. For more marine structures the most widely used analytical tool is the Finite Element (FE) method, modal testing and analysis being the experimental counterpart. The FE method is widely used in industry as it can produce a good representation of a true structure. However, one must bear in mind that, due to limitations in the FE method, an FE model is always an approximation of the structure under study. Inaccuracies and errors in an FE model can occur due to: Inaccurate estimation of the physical properties of the structure; in individual element shape functions and/or a poor quality mesh; poor approximation of boundary conditions; inadequate modeling of joints; introduction of additional inaccuracies during the solution phase such as; and computational errors. The experimental approach relies on extracting the vibration characteristics of a structure from measurements. It consists of two steps, (i) taking the measurements and (ii) analyzing the measured data. In the last two decades substantial progress has been made in the experimental approach thanks to continued development of modal analysis techniques, the benefits of better data-acquisition and measurement equipment as well as advances in computing hardware and software. Vibration measurements are taken directly from a physical structure, without any assumptions about the structure, and as such they are considered to be more reliable than their FE counterparts. However, limitations and errors in the experimental approach can occur due to: experimental errors due to noise, the application of windows and filters; the assumption of linear response while there can also be non-linear structural response and/or non-linearities in the measurement system and so on. It is generally believed that more confidence can be placed on experimental data as measurements are taken on the true structure. Therefore, the analytical model of a structure is usually updated on the strength of the experimental model [20]. We evaluated selection procedure of the degrees of freedom passive in stage of the model reduction with a reasonable criterion by using the sensitivity analysis of system response under a base excitation. This performance leads to faster convergence of iterative algorithm. Also, in this paper to overcome problem of uncertainty in modeling has been utilized the FEM updating relies on the experimental modal data. Since the major problem inherent to dynamic structural analysis is the time-consuming and costly amount of computation required, so using this method savings will be both in time and cost.

II. CMCM Method and Its Application to Identification and Classification Damage Section

The main framework of the algorithms being used in this study is based on the structure of the approach similar to CMCM [6]. In this section, this method introduce briefly. First, the equation of motion for a multi-degree of freedom an undamped dynamic system is given as follows:

$$M\ddot{V} + KV = 0 \tag{1}$$

In which **0,** M and K are a zero vector, mass matrix and stiffness matrix, respectively. In addition, V and $\ddot{V}$ denote the vectors of the displacement and acceleration. The eigensolution of the target system consists of the eigenvalues and eigenvectors. The ith eigenvalue and eigenvector associated with *K* and *M* is expressed as

$$K\Phi_i = \lambda_i M\Phi_i \tag{2}$$

where *M* and *K* is the mass matrix and stiffness matrix for the undamaged model and λ_i and Φ_i is the ith eigenvalue and eigenvector associated with *K* and *M*. In the development of the CMCM method, it is assumed that the stiffness and mass matrices of the structure denoted by *K* and *M* are obtained from a finite-element model. Assume that the stiffness matrix K^* of the actual (experimental) model is a modification of *K* to be formulated as

$$K^* = K + \sum_{n=1}^{N_e} \alpha_n K \tag{3}$$

where K_n is the stiffness matrix corresponding to the nth element, N_e is the number of elements, and α_n are unknown correction factors to be determined. Herein, for simplicity in presentation, it is assumed that each element involves a parameter to be updated, such as the Young's modulus of each element. In most studies for the damage detection, particularly in relation to offshore structures usually changes in the mass matrix are negligible.so, it is assumed that:

$$M = M^* \tag{4}$$

Express the *jth* eigenvalue and eigenvector associated with K^* and M^* as

$$K^*\Phi_j^* = \lambda_j^* M^*\Phi_j^* \tag{5}$$

It is assumed that a few of λ_j^* and Φ_j^* are known measurements available from modal testing. Premultiplying Eq.(5) by $(\Phi_i)^t$ yields

$$(\Phi_i)^t K^*\Phi_j^* = \lambda_j^* (\Phi_i)^t M^*\Phi_j^* \tag{6}$$

Where the superscript ''*t*'' is the transpose operator. Substituting Eqs. (3) and (4) into Eq. (6) yields

$$C_{ij}^{\int} + \sum_{n=1}^{N_e} \alpha_n C_{n,ij}^{\int} = \lambda_j^* \left(D_{ij}^{\int}\right) \tag{7}$$

After using a new index v to replace *ij*, Eq.(7) becomes

$$C_v^{\int} + \sum_{n=1}^{N_e} \alpha_n C_{n,v}^{\int} = \lambda_j^* \left(D_v^{\int}\right) \tag{8}$$

where $C_{n,v}^{\int} = (\Phi_i)^t K_n \Phi_j^*$, $C_v^{\int} = (\Phi_i)^t K \Phi_j^*$ and $D_v^{\int} = (\Phi_i)^t M \Phi_j^*$.

Rearranging Eq. (8), one obtains

$$\sum_{n=1}^{N_e} \alpha_n C_{n,v}^{\int} = f_v^{\int} \tag{9}$$

where, $f_v^{\int} = -C_v^{\int} + \lambda_j^* D_v^{\int}$.

When N_i modes are taken from the analytical (baseline) finite element model, and N_j modes are measured from the damaged structure, totally $N_v = N_i \times N_j$ equations can be formed from Eq. (9). Equations formed based on Eq. (9) are named CMCM equations because they involve two modes of two models. Rewriting Eq. (9) in a matrix form, one shows

$$C^{\int}_{N_v \times N_e} \alpha_{N_e \times 1} = f_{N_v \times 1} \tag{10}$$

When N_v is greater than N_e, a least-squares technique can be taken to expect for α. The obtain of α is written as

$$\alpha = \left(C^t C\right)^{-1} C^t f^{\int} \tag{11}$$

III. Model Reduction Techniques and Spatial Incompleteness

A. Analysis of Vibration Frequencies

It can be shown that for the real, symmetric, positive definite mass and stiffness matrices which pertain to stable structural systems, all roots of the frequency equation will be real and positive. By MATLAB software, [V,D]=eig(A) produces matrices of eigen-values (D) and eigenvectors (V) of matrix A, so that A×V = V×D. Matrix D is the canonical form of A-a diagonal matrix with A's eigenvalues on the main diagonal. Matrix V is the modal matrix - its columns are the eigenvectors of A.

B. Guyan Reduction Method

The major problem inherent to dynamic structural analysis is the time-consuming and costly amount of computation required. In modal analysis, the burden is in computing natural frequencies and mode shapes. As practical finite element models can contain tens of thousands of degrees of freedom, the time and expense of computing all of the frequencies and mode shapes are prohibitive. Fortunately, to obtain reasonable approximations of dynamic response, it is seldom necessary to solve the full eigenvalue problem. Two practical arguments underlie the preceding statement. First, the lower-valued frequencies and corresponding mode shapes are more important in describing structural behavior. This is because the higher-valued frequencies most often represent vibration of individual elements and do not contribute significantly to overall structural response. Second, when structures are subjected to time-dependent forcing functions, the range of forcing frequencies to be experienced is reasonably predictable. Therefore, only system natural frequencies around that range are of concern in examining resonance possibilities [21].

The first model reduction method is a static reduction method introduced by Guyan (1965) [22]. This technique partitions the mass and stiffness matrices, and the displacement vector into a set of master and slave DoFs. The Guyan transformation matrix and the reduced Guyan mass and stiffness matrices are presented as follows:

$$\begin{bmatrix} [M_{mm}] & [M_{ms}] \\ [M_{sm}] & [M_{ss}] \end{bmatrix} \begin{Bmatrix} \ddot{V}_m \\ \ddot{V}_s \end{Bmatrix} + \begin{bmatrix} [K_{mm}] & [K_{ms}] \\ [K_{sm}] & [K_{ss}] \end{bmatrix} \begin{Bmatrix} V_m \\ V_s \end{Bmatrix} = \begin{Bmatrix} 0 \\ 0 \end{Bmatrix} \tag{12}$$

Here, the subscripts *m* and *s* correspond to master and slave coordinates, respectively. The inertia terms are neglected to obtain the equation:

$$[K_{sm}]\{V_m\}+[K_{ss}]\{V_s\}=[T_s]\{V_m\} \quad (13)$$

This equation may be used to eliminate the slave coordinate to leave the following:

$$\begin{Bmatrix} V_m \\ V_s \end{Bmatrix}=\begin{bmatrix} [I] \\ -[K_{ss}]^{-1}[K_{sm}] \end{bmatrix}\{V_m\}=[T_s]\{V_m\} \quad (14)$$

$$[T_s]=\begin{bmatrix} [I] \\ -[K_{ss}]^{-1}[K_{sm}] \end{bmatrix} \quad (15)$$

T_s is Guyan transformation matrix and *I* is identify matrix. The reduced Guyan mass and stiffness matrices are then given by

$$[M_R]=[T_s^t]\ [M]\ [T_s] \quad (16)$$

$$[K_R]=[T_s^t]\ [K]\ [T_s] \quad (17)$$

In dealing with spatial incompleteness, usually applies model reduction schemes. The transformation matrix the master coordinates of the full order coordinates for the baseline model is denoted as *T*. The final relations are produced by applying $\Phi_i=T(\Phi_i)_m$ and $\Phi_j^*=T^*(\Phi_j^*)_m$ the previous equations. Where $(\Phi_i)_m$, $(\Phi_j^*)_m$ and T^* are the i^{th} mode shape of the baseline structure calculated only at the master coordinates, the j^{th} mode shape of the damaged structure measured only at the master coordinates and the counterpart of *T* for the damaged structure respectively.

$$C_v^{\int}=[(\Phi_i)_m]^t T^t K T^*(\Phi_j^*)_m \quad (18)$$

$$C_{n,v}^{\int}=[(\Phi_i)_m]^t T^t K_n T^*(\Phi_j^*)_m \quad (19)$$

$$D_v^{\int}=[(\Phi_i)_m]^t T^t M T^*(\Phi_j^*)_m \quad (20)$$

That here, equal is with the reduced stiffness matrix ($[K_R]=[T^t][K][T^*]$). For the implementation of the proposed technique, initially the mass and stiffness matrices were extracted by *ANSYS* software under *SUBSTRUTUR* analysis. All calculations including: the frequencies, displacement and eigenvalues vectors, select the master degrees of freedom, the transformation matrix, formation and solving the Eq.(10) were performed with MATLAB software. In this method, the only source of errors stems from T^*, assuming that $(\Phi_i)_m$ (measured only at the master coordinates) has been a noise-free measurement. Because the reduction matrix T^* which is obtained from the experimental model is unknown during model updating process, the iteration should be utilized and T^* should be replaced by T_r which is the reduction matrix of the FEM at the r^{th} iteration. After the first iteration, T^* then can be computed based on the damaged model obtained from the previous iteration. The iteration could carry on until a converged damaged model is reached.

C. Selections of (DOFs) Based on the Dynamic Sensitivity Analysis

Sensitivity analysis allows an analyzer to evaluate the effects that changes in a certain parameter will have on the model's responses. Also it can help the analyzer to identify the parameters which are the main factors of a model's results. By reporting extensive outputs from sensitivity analysis, designers are able to consider a wide range of scenarios and, as such, can increase the level of confidence that an analyzer will have in the model. In this study, spectrum analysis (Single-Point Response Spectrum) was used for the sensitivity analysis. In general, two types of spectrum analyses are supported by ANSYS software: the deterministic response spectrum method and the nondeterministic random vibration method. Also, both excitation at the support and excitation away from the support are allowed. Three response spectrum methods are known as the single-point, multiple-point and dynamic design analysis method. For single-point response spectrum analysis and dynamic design analysis method, the structure is excited by a spectrum with known direction and frequency components. It applies uniformly on all support points or on specified unsupported master DOFs. Both base excitation and excitation away from the supports are allowed for the single-point response spectrum analysis. The general process for performing a single-point response spectrum analysis consists of six primary steps: Step 1: Build the model, Step 2: Obtain the modal solution, Step 3: Obtain the spectrum solution, Step 4: Expand the modes, Step 5: Combine the modes, Step 6: Review the results. The modal solution is required because the structure's mode shapes and frequencies must be available to calculate the spectrum solution. Only linear behavior is valid in a spectrum analysis.

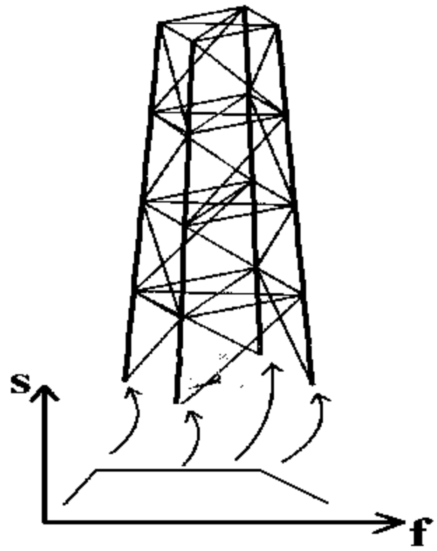

Fig. 1. spectrum analysis (single-point response spectrum)

In this paper, the platform model was excited in the range of the first modes (between 1-70 Hz) in the vertical direction, as shown in Fig. 1. As a result, seismic displacement in the form of equivalent nodal stress was checked as response of the platform. *ANSYS* offers five different mode combination methods for the single-point response spectrum analysis.

Here, the Square Root of Sum of Squares (SRSS) method was used.

IV. Numerical and Experimental Modal Analysis and Finite Element Model Updating

Modal analysis is the procedure of identifying the intrinsic dynamic properties of a system in forms of natural frequencies, damping factors and mode shapes, and using them to formulate an analytical model for its dynamic behavior. In this paper, the *Block Lanczos* method has been applied for solving the modal analysis. Modal testing is an experimental method utilized to derive the modal model of a linear time-invariant vibration system. The theoretical basis of the method is secured upon establishing the relationship between the vibration response at one location and excitation at the same or another location as a function of excitation frequency. In summary, experimental modal analysis involves three constituent phases: test preparation, frequency response measurements and modal parameter detection. The preparation involve selection of a structures support, type of excitation force(s), location(s), hardware to measure force(s) and responses; designation of a structural geometry model which consist of points of response to be measured; and detection of mechanisms which could lead to inaccurate measurement. However, owing to the complexity and uncertainty of the structure, it is unrealistic to expect such an FE model to be faithfully representative. A fundamental approach is to take a measurement of the structure, derive its modal model and use it to correlate with the existing FE model in order to update it. The objective of model updating is to adjust the analytical model of the structure so that the model predictions are in compromise with the test results. In the present article, only the first four lower natural frequencies are used in the updating process. Hereof, *modal assurance criterion (MAC)* method is applied for updating of the model using *FEMtools3.3.0* software. The modal assurance criterion *(MAC)*, which is also known as mode shape correlation coefficient, between analytical mode Φ_i and experimental mode Φ_j is defined as:

$$MAC(\Phi_i, \Phi_j) = \frac{\left|\Phi_i^T \Phi_j\right|^2}{(\Phi_i^T \Phi_i)(\Phi_j^T \Phi_j)} \tag{21}$$

A *MAC* value close to 1 suggests that the two modes are well correlated and a value close to 0 indicates uncorrelated modes [23].

V. Description of The Physical Model and Test Set Up

Experimental modal tests were performed on a fixed jacket-type offshore platform modal. The measured responses were obtained from the shaker tests. Also, during the implementation of the test, the structural responses were acquired as the time series signals. Three dimensional views of the physical model and the finite element model of the platform are shown in Fig. 2. The structure, consisting of 46 steel tubular members with outer diameter 18 mm, wall thickness 2.5 mm for leg members and outer diameter 12mm, wall thickness 1.5 mm for other members, is fixed at the ground. The physical model was constructed of stainless steel pipes that were welded together using argon arc welding to ensure proper load transfer. The mass density of the members is $\rho = 7850$ kg/m^3 and the Young's modulus of steel is $E = 2.07 \times 10^{11}\, pa$. There are 16 nodal points in the finite element model, three translational DoFs (U_x, U_y, θ_z) at each node, thus total 48 translational DoFs. The test set up and instruments are illustrated in Fig. 3. The excitation (based on white noise signals) was enforced by means of an electrodynamic exciter driven by a power amplifier (model 2706). The frequency sampling of the test setup was chosen to be 10 KHz, and the frequency rang was 0 to 200 Hz.

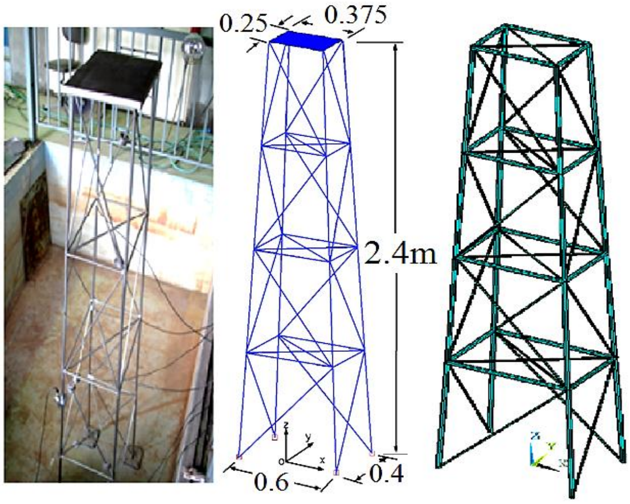

Fig. 2. The geometrical properties of the physical model and finite element model.

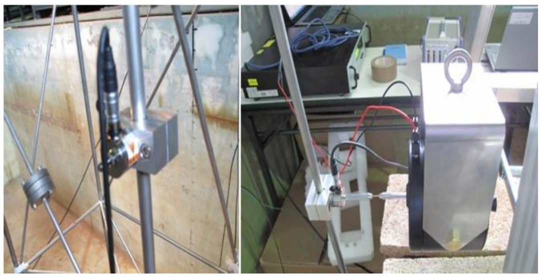
Fig. 3. Connections between amplifier, exciter, load cell and structure.

A. Results of the Finite Element Model Updating

The platform was modeled using 3-D finite element software, *ANSYS*, modal analysis was performed and mode shapes of the numerical and experimental modal analysis are shown in Fig. 4, also frequencies of numerical and experimental model and *MAC* value are listed in Table I. Finally, it can be concluded that there is perfect correlation between the numerical modal and experimental modal vectors as shown in Fig. 5. This means that, *MAC* value is close to 1 and the numerical and experimental models have appropriate correspondence.

B. Results of the Sensitivity Analysis

The response and the stress changes will be similar to Fig. 6 due to applying the spectrum analysis (SPRS) on the updated model of the platform with and without horizontal braces in floors. According to Fig. 6, it can be seen that removals of the horizontal braces do not have significant effect on the structural response or the stress changes. We reached to the reduced model via regardless of the horizontal brace elements of the platforms, as shown in Fig. 7. In this study, the damage severity is defined as the percentage stiffness loss of an element. The damaged structure, simulated by a finite element model as well, has three damage elements, including a damaged beam (element 10) with 50% stiffness loss, a 60% damaged column (element 18) and a 40% damaged brace (element 42). The locations of those damaged elements are highlighted in Fig. 7.

TABLE I: MODEL UPDATING AND THE FIRST FOUR NATURAL FREQUENCIES

Mode no.	Natural frequencies (Hz)			Differences (%)	MAC
	Initiall FEM	Experimental model	Updated FEM		
1	67.29	58.34	58.5	0.27	0.994
2	91.46	94.13	93.93	0.21	0.991
3	100.8	106.21	105.88	0.3	0.995
4	125.1	130.28	130.67	0.29	0.992

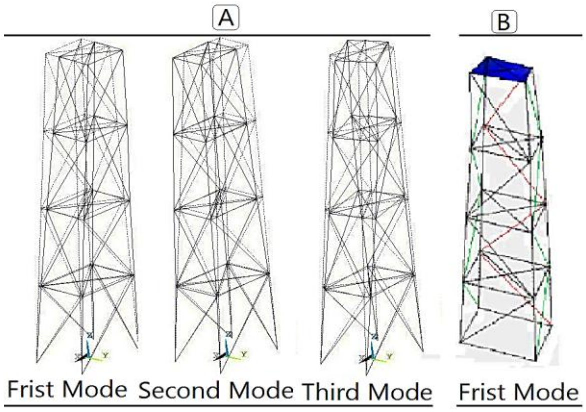

Fig. 4. The mode shape using (A) numerical modal analysis, (B) experimental modal analysis.

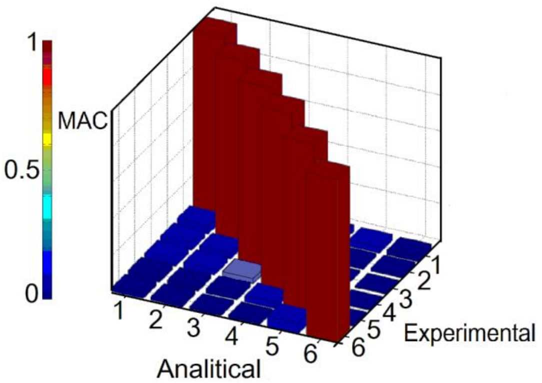

Fig. 5. MAC values between the numerical and experimental mode shapes.

VI. RESULTS AND DISCUSSION

A. Identify the Severity and Location of the Damage

Damage detection results based on *Guyan* reduction approach with 24 DoFs (without and with iteration) and also the effects of removal on a number of the degrees of freedom of the structure in two different areas have been presented in the form of graphs (8) and (9). These results were obtained with adopting four modes of the damaged structure and the twelve modes of the intact structure. Elements 19 and 31, (column members between the second and the third floors), are poorly estimated. The damage detection result via iterations is shown in Fig. 8(b). When the iterative procedure was applied, T calculated based on the damaged model obtained from the previous iteration. Obviously, applying the iterative procedure improves the performance of the damage detection. Sometimes in practice, it is often desirable to reduce the number of sensors installed at the test structure to save costs. To examine the effects of ommiting of some degrees of freedom, was removed 8 degrees of freedom (U_y at 8 node) located on the top floor (deck) and the second floor in the two separate steps of 48 degrees of freedom in the structure. As shown in Fig. 9 errors are great when the degrees of freedom on the top floor have been removed. The Large error is related to the dynamic behavior of structures in the first mode. As a result, the better results can be obtained when the available sensors are installed in the upper part of the platform.

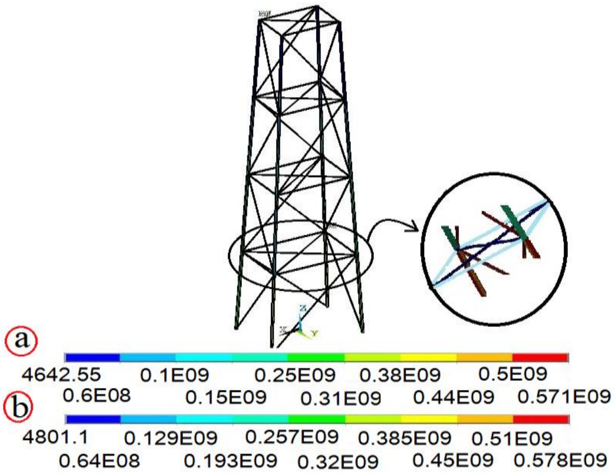

Fig. 6. The stress changes for the updated model of the platform (a): with and (b): without horizontal braces.

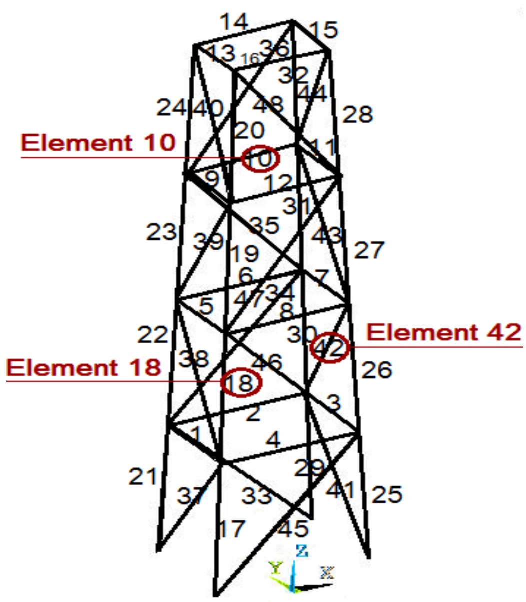

Fig. 7. Sketch of the offshore platform structure and location of the damaged members.

B. Damage Detection Along with Adding Noise in the Measured Data

Actually, modal measurements always contain errors. Mild noise environment and fluctuations caused by the impairments in the measuring tools are origin of these errors. Here, the measurement of the polluted *x*th mode shape of the damaged structure at the *z*th DoF, denoted by $\overline{\Phi}^*_{xz}$, has been simulated by adding a Gaussian random error to the corresponding true value Φ^*_{xz},

$$\overline{\varphi}^*_{xz} = \varphi^*_{xz}\left(1 + \varepsilon\gamma_{xz}\right) \tag{22}$$

where γ_{xz} denotes a noise level, and ε is a Gaussian random number with zero mean and unit standard deviation. In the numerical study, the results are always obtained from taking a 500 repeated Monte Carlo simulations. In other words, to obtain statistical knowledge about the identification result, 500 simulations were performed.

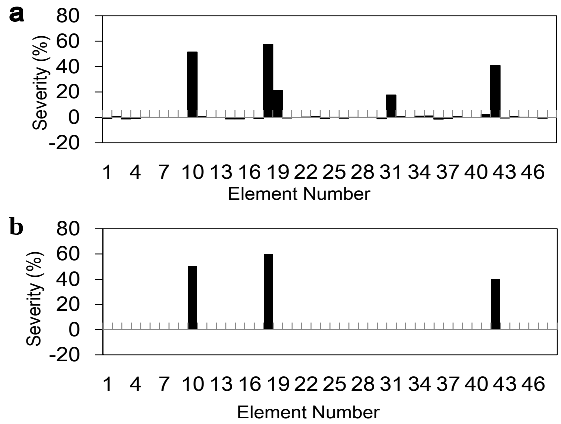

Fig. 8. Damage identification results based on the reduction of Guyana with 48 degrees of freedom: (a) without repetition (b) with repeated.

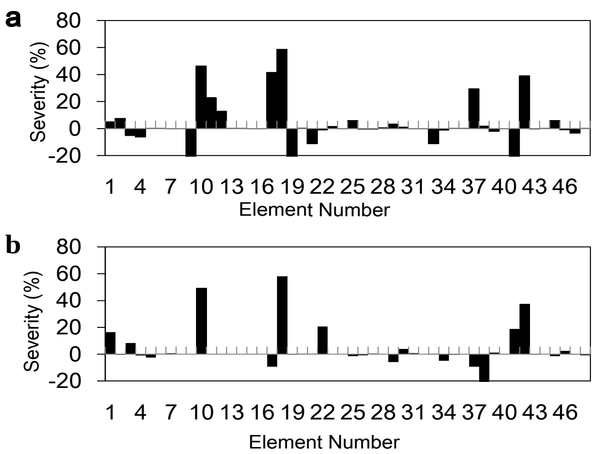

Fig. 9. Damage identification results via eliminating some degrees of freedom with 40 degrees of freedom: (a) top floor (b) second floor.

A factor called *correct detection probability* is defined in order to evaluate the noise effect on the accuracy of the proposed method. If N_n is used to represent the number of Monte Carlo simulations for a given level of noise and N_c the number of realizations that an actual damage is detected, the percentage of correct detection probability S_R will be given by:

$$S_R(\%) = \frac{N_c}{N_n} \times 100 \tag{23}$$

For the level of noise from 1% to 2%, the applied noise distribution is illustrated in Fig. 10. Also the plot in Fig. 11 showes the mean and standard deviation (σ) of the damage severity estimates obtaining by 500 noise simulations. Where each simulation is based on 1% and 2% error level. We employed four measured modes (e.g. $N_j = 4$) for using the CMCM method. For example, the detection probability is S_R=92.7% for a 1% noise level and S_R=75.9% for a 2% noise level.

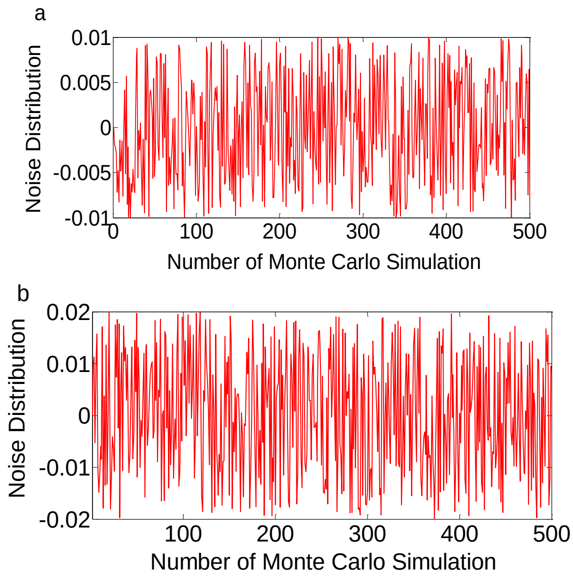

Fig. 10. The applied noise distribution for the level of noise (a): 1% and (b): 2%.

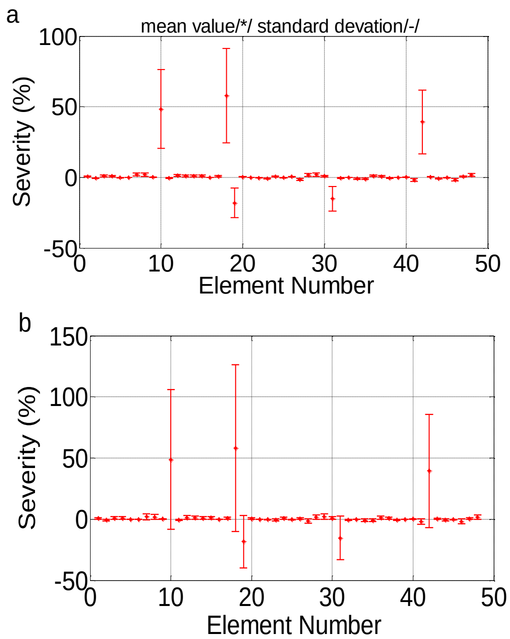

Fig. 11. Damage identification results using CMCM with Guyan model reduction, 500 Monte Carlo simulations under (a): 1% and (b): 2% error.

Members 31 and 19 (columns which are located on the third floor) possessed the largest σ values which indicates that these two members are most sensitive to measurement errors. Elements 12–15 (at the top floor), element 17 (at the first floor), elements 46–48 (at the second and top floor) contain low sensitivity and elements 7–8 (at the third floor), elements 29–28 (at the first and top floor) contain more sensitivity**.** Increasing the noise level caused the addition in the standard deviation and reduction in the detection probability of the damage. Therefore, the measurement noise has a much larger effect on the vertical columns and members closer to the damaged area than that of slanted braces or horizontal members.

VII. Conclusions

An efficient FEM updating method for detecting damages of an offshore jacket structures using the measured of the first few lower-order modal parameters is investigated. An initial FE-model is modified through updating the analytical model with consideration of the experimental modal analysis results. The first four natural frequencies which are obtained from the experimental modal analysis are 58.34, 94.13, 106.21 and 130.28 Hz, respectively, and after recognition of the equivalent frequencies by the FE-model, the initial numerical model has been updated based on the MAC parameter. In this process, the parameters of the elastic modulus and the stiffness of the supports at the base of the structure are considered as the more efficient factors. Removal of the uncertainty effects on the numerical results has been presented as an objective for this research for providing an advanced damage detection method which is less sensitive to uncertainties arising from analytical modeling. Moreover, the reflection of the sensitivity analysis on the updated model has been considered as a perspective to reduce the model for assessment the improved CMCM method via the application of an appropriate criterion to select the degrees of freedom. This performance leads to faster convergence of iterative algorithm. Since the major problem inherent to dynamic structural analysis is the time-consuming and costly amount of computation required, so using this method savings will be both in time and cost. The accuracy and efficiency of the proposed method were evaluated by a physical model of jacket structure via modal parameters realized from vibrational behavior of the structure. Results represent that the proposed approach is executable and efficient to locate damages and estimate severities under noise-free measurement and with a reasonable percentage for noisy data. The measurement noise has a much larger effect on the vertical columns. Also, the sensitivity of the damage detection algorithm resulting from the removal of some available sensors is examined. The sensors of the located on the top floors of the platform possess the most roles in terms of performance and influence at the process of damage detection.

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