Updating For Structural Parameter Identification of the Model Steel Bridge Using OMA

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Abstract- **There are many varieties of the structural and architectural structures in the world. Common features of these structures, it can be managed to survive under static and dynamic loads. In this study, the dynamic characteristics of a model steel bridge with a bolt connection constructed in a 6.10m span and 1.88m height laboratory were determined by finite element method and operational modal analysis methods. In the study, firstly, finite element model was created in SAP2000 software of model steel system and dynamic characteristic were obtained numerically. Then, accelerometer was placed at certain points of the system and dynamic characteristics were determined by operational modal analysis method. As known OMA methods, SSI is used to estimation dynamic parameters of model steel bridge. For this purpose, analytical analysis of the model steel bridge with finite element method and the dynamic parameters obtained as a result of the operational modal analysis of the model steel bridge were compared. The Stochastic Subspace Identification (SSI-PC)is used through output-only modal identification. The modal parameters obtained experimentally were used to calibrate a finite element model of the structure. Based on the eigen ensitivity-based FE model updating procedure a summary of the changes the FEM results to the OMA results is presented graphically and numerically in percent to the initial state of the structure. As seen from the modal updating result MAC values were generated between analytical and experimental mode shapes. Main difference between mode shapes of the FEM and EMA was explained.Modal updating from the MAC that the 90% approach in the mode shapes nearly reached 100% after the ±5% increase in mass density which is made from the material properties (ρ)**

Keywords: **System Identification, FEM, Model Updating, OMA, Steel Bridge, MAC, SSI**

I. INTRODUCTION

There are many varieties of the structural and architectural structures in the world. Common features of these structures, it can be managed to survive under static and dynamic loads. Structures under dynamic loads and vibrations occurred impact consists of vibrations that do not require or require intervention on the structure brings many damages occurs. In this case, the vibration should be known and may occur in nature and will be focused on the effects generated by these vibrations. In recent years, several earthquakes have occurred in the world and are given as a result of heavy losses. If the resulting perceived to pose several problems for the countries of heavy losses, the structure of the receipt of the knowledge of the current situation and how important it is understood that the necessary measures. In this case, experimental determination of the behavior they showed against vibrations from the structures and obtained the theoretical and the creation of finite element model to represent the actual structure by comparing the experimental value are emerging requirements. As known forced (shaker, impact, pull back or quick release tests) and ambient vibration techniques are available for vibration testing of large structures[7]

Most of structures located in regions prone to earthquake hazards suffer from various types of destruction caused by seismic loads. Under such earthquake occurring, the parts (especially the columns) of building structures suffer damage. Looking on the other side, especially considering the performance of such buildings under seismic occurrence, there is a great need to strengthen the columns even without changing their building masses; this clearly shows that there is a need to investigate the connection between technical repairing or strengthening procedures and the column capacity. In this understanding, more researches are being conducted to get required performance of structures under seismic loading, by means of looking at different point of view and directions. [19]In recent years, one of the reasons for increasing the importance of observing the health of the building civil engineering, scientific research circles; which damage can be identified first? How long is the usable life of the building to develop methods to answer questions such as. These studies are increasing. This issue is given importance due to factors such as the health of the structure against natural and artificial influences and its economic longevity. In all construction systems, damage starts at the material level. As the damage in the system increases, it reaches a value defined as deterioration. Civil engineering structures are exposed to a variety of natural and artificial effects throughout their lifetime. These effects are the forces that can affect the dynamic characteristics of the structure and thus the service life. Model identification, system-related, based on physical laws based on the preliminary information and the size of the system (introduction magnitude or input signal) from the system's response to these magnitudes (output magnitude or output signal)[26]

Ambient vibration testing (also known as Operational Modal Analysis) is the most economical non-destructive testing method used to obtain vibration data from large civil engineering structures for Output Model Definition only.

General characteristics of structural response (appropriate frequency, displacement, velocity, acceleration rungs), suggested measuring quantity (such as velocity or acceleration) depends on the type of vibrations. This structures Response characteristics gives a general idea of the preferred quantity and its rungs to be measured. A few studies the analysis of ambient vibration measurements of buildings from 1982 until 1996 are discussed in Ventura and Schuster [23].Last ten years Output-Only Model Identification studies of buildings are given in appropriate references structural vibration solutions. For the modal updating of the structure it is necessary to estimate sensitivity of reaction of examined system to change of parameters of a building[10].The modal parameter identification using output-only measurements presents a challenge that needs the use of special identification techniques, which can deal with very small magnitudes of ambient vibration contaminated by noise without the knowledge of input forces. A newstructuralidentificationtool is proposedtoidentifythemodalproperties of structures. At l ast, aftercollectingmodalresponsesfromtheavailablesensors, t hemodeshapevectorforeach of thedecomposedmodes in thes ystem is identifiedfromallobtainedmodalresponsedata[20]. Over the past decades, the technique of experimental modal parameter identification of civil engineering structures has developed very fast. The benchmark study has been carried out to compare modal parameter identification techniques for evaluating the dynamic characteristics of a real building on operation conditions from ambient vibration data. Infact, the mathematical background of output-only modal parameter identification methods is often very similar. The difference is often due to implementation aspects such as data reduction, type of equation solvers, sequence of matrix operations, etc. Consequently, the question arises to compare those analysis techniques with regard to full-scale structures. The paper is intended to compare the modal parameter identification techniques for evaluating the dynamic characteristics of a real building and a real bridge on operation conditions from ambient vibration data. The modal properties of the benchmark steel bridge were computed using analytical approach for a comparison with the experimental modal frequencies. system identification method is efficient and accurate in identifying modal data of the structures.[9]Two system identification techniques use dare the frequency domain-based peak picking (PP) method and the time domain-based stochastic subspace identification (SSI) method. It has not been the intention to elect a winner among the system identification techniques from ambient vibration data. The intention is to convey the fact that several of the methods can complement one another in practical application. The preferable method depends solely on the nature of actual application.According to these is micdamage types of stee lbridges, the dynamicstability and bearing capacity, the po st-earthquake service ability and reparability, low-cyclefati gueare some important contents for seismic safety checki ng [25].It is generallyacceptedthatnon-uniformdistribution of massandstiffness, both in plan and in elevation, is the main cause of therotationalresponse of building structures during strong groundmotions, and in many cases this res ponse has ledtopartialor total collapse[4]The determination of the real state of the building as non-destructive by measuring the responses of the structures against natural and artificial effects[12-13-14].In this study, non-destructive determination of structure parameters, system identification, model update and damage detection methods have been examined in detail and a new calculation algorithm and computer software have been developed. With the development of new methods for structural health monitoring in the studied sources, there has been an acceleration in the development of the devices used [3]. Operational modal analysis is being widely used in aerospace, mechanical and civil engineering. Common research fields include optimal design and rehabilitation under dynamic loads, structural health monitoring, modification and control of dynamic response and analytical model updating. In many practical cases, influence of noise contamination in the recorded data makes it difficult to identify the modal parameters accurately[21]

II. METHOD

The Stochastic Subspace Identification Technique (SSI) is a time-domain method that works directly with raw time data without the need to convert them to correlations or spectra. The stochastic subspace identification algorithm defines state space matrices based on measurements using robust digital techniques. When the mathematical definition of construction (state-space model) is found, modal parameters are simple to determine. The theoreticaly distances is given in [22].The model of the vibrational structures can be described by a series of linear, constant-coefficient and second-order differential equations, Peeters (2000):

$$m\ddot{u}(t) + c\dot{u}(t) + ku(t) = F(t) = df(t) \qquad (1)$$

Where m, c, k are the mass, damping and stiffness matrices, $F(t)$ is the stimulation force, and $u(t)$ is the displacement vector at continuous time t.d is an input influence matrix, characterizing the locations and type of known inputs $f(t)$.The state-space model is derived from the control theory, but it also appears in mechanical-civil engineering to calculate the modal parameters of a dynamic structure with a general viscous damping model. The motion equation (1) is transformed into space-space, which is the first of the first-order equations, that is, the system is regarded as a continuous-time state-space model.

$$\dot{z}(t) = A_c z(t) + B_c f(t) \quad (2)$$

$$A_c = \begin{bmatrix} 0 & I \\ -m^{-1}k & -m^{-1}c \end{bmatrix}$$

$$B_c = \begin{bmatrix} 0 \\ m^{-1}d \end{bmatrix}$$

$$z(t) = \begin{bmatrix} u(t) \\ \dot{u}(t) \end{bmatrix} \quad (3)$$

Where $A_c$is the state matrix, $B_c$is the input matrix and $z(t)$is the state vector. The number of elements of the state space vector is the number of arguments needed to describe the state of the system. Assuming that the measurements are evaluated only at one sensor position and that these sensors are speedometers, speed or displacement transducers (accelerometers), and the observation equation is

$$y(t) = C_a\ddot{u}(t) + C_v\dot{u}(t) + C_d u(t) \quad (4)$$

Where $y(t)$ are the outputs, and C_a, C_v, C_d are the output matrices for acceleration, velocity, displacement. With this definitions

$$C = [C_d - C_a m^{-1} k C_v - C_a m^{-1} c] \quad (5)$$

$$D = C_a m^{-1} d$$

Equation (4) can be transformed into:

$$y(t) = Cz(t) + Du(t) \quad (6)$$

Where $C$is the output matrix and $D$is the direct transmission matrix. Equations (2) and (6) form a continuous-time deterministic state-space model. Continuous time means that the expressions can be evaluated at each time instant $t \in \mathbb{R}$ and deterministic means that the input-output quantities $u(t), y(t)$ can be measured exactly. Of course, this is not realistic: measurements are available at discrete time instants $k\Delta t$, $k \in \mathbb{N}$withΔt, sample time and noise always influence the data. After the example, the state-space model looks like this:

$$z_{k+1} = Az_k + Bu_k \quad (7)$$

$$y_k = Cz_k + Du_k$$

Where $z_k = z(k\Delta t)$is the discrete-time state vector, is the pr ocess noise due to disturbance and modeling imperfection s; v_k is the measurement noise due to sensors' inaccuraci es; It includes stochastic noise and we obtain the following discrete-time combined deterministic-stochastic state-space model:

$$z_{k+1} = Az_k + Bu_k + w_k (8) \quad y_k = Cz_k + Du_k + v_k$$

w_k, v_kVectors are non-measurable, but they assume that there is zero average and white noise. If this white noise hypothesis is violated, in other words if the input contains also some dominant frequency components in addition to white noise, These frequency components are indistinguishable from the system's own frequencies and appear as eigenvalues of the system matrix A.

$$E\left[\begin{pmatrix} w_p \\ v_p \end{pmatrix} \begin{pmatrix} w_q^T & v_q^T \end{pmatrix}\right] = \begin{pmatrix} Q & S \\ S^T & R \end{pmatrix} \delta_{pq} \quad (9)$$

Where E is the expected value operator and δ_{pq} is the Kronecker delta. Vibration information available in structural health monitoring (SHM) is often the reaction of a structure induced by operational inputs, some of which are unmeasured inputs. Due to the lack of input information it is not possible to distinguish deterministic input u_k from the noise terms w_k, v_kin [2]. If the deterministic input term $u_k$is modeled by the noise terms w_k, v_k the discrete-time purely stochastic state-space model of a vibration structure is obtained:

$$z_{k+1} = Az_k + w_k \quad (10) \qquad y_k = Cz_k + v_k$$

Equation (10)Operational vibration measurements provide the basis for defining the time-consuming system. The stochastic subspace method defines state space matrices based solely on output measurements and robust digital techniques.

Output only modal analysis is a type of operational modal analysis. This method aims to determine the reactions of the structure under ambient vibrations. Although it is a widely used method recently, the reliability of its results is also an important positive aspect.

In this study, output only modal analysis method was used. As is known in this method, under ambient vibrations, responses are taken from various joints of the model steel bridge.

III. DESCRIPTION OF MODEL STEEL BRIDGE

In this study, 6.10 m span, 1.88 m height model steel Bridge used. The model steel bridge is shown in Figure 1. The bridge model has a deformed arch geometry. The legs tilted inward in the direction of the long axis of the deck provided the console operation of the end sections of the deck. The legs have a 45 degree bending. The profiles along the axis of the deck are made of box profile with a thickness of 2.5cm. Circular profiles with a diameter of 2 cm are used in trusses. In the Diagonal and Cross Connection elements, 10mm diameter steel material is used.

Figure 1.Formation of model steel bridge

The model steel bridge of the experiment is located in the laboratory environment and away from the external effects. Thus, the desired results are obtained more clearly and healthier. The clarity and accuracy of the results and the true representation of the model play an important role in scientific studies. In this study, measurements and experiments were carried out with careful attention to all scientific procedures.

IV. ANALYTICAL MODAL ANALYSIS OF MODEL STEELBRIDGE

A finite element model was generated in SAP2000 software in Fig.2. Bridge modeled as an absolutely rigidity (rigid diaphragm). The selected bridge is modeled as a space frame bridge with 3D elements. Model steel bridge is modeled using an equivalent thickness and frame elements with isotropic property. All supports are modeled as fully fixed. The members of model steel bridge are modeled as rigidly connected together at the intersection points. The model steel bridge is modeled with finite elements (Figure 2). When modeling the model steel bridge, the modulus of elasticity was taken as $E = 2.0x10^5$MPa, material density ρ = 7850kg / m3 and poisson ratio $\upsilon = 0.3$

A total of 40 joints and 71 bar elements (straight diagonal-beam) were used in the model. A total of five natural frequencies of the model steel bridges are attained which range between 5 and 25 Hz. The first five vibration mode of the model steel bridge is shown in Fig 3. Analytical modal analysis results at the finite element model are shown in Table1.

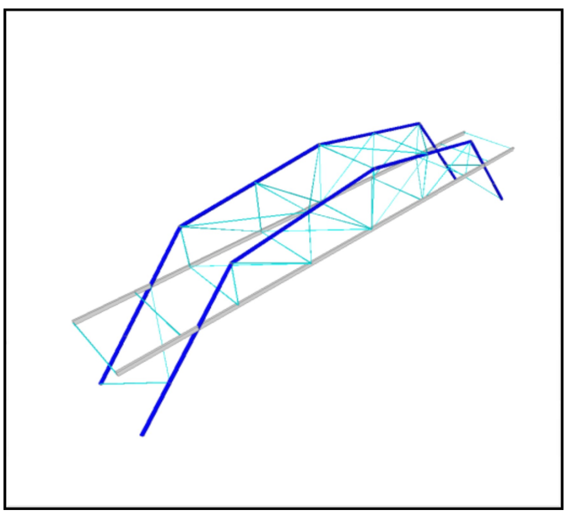
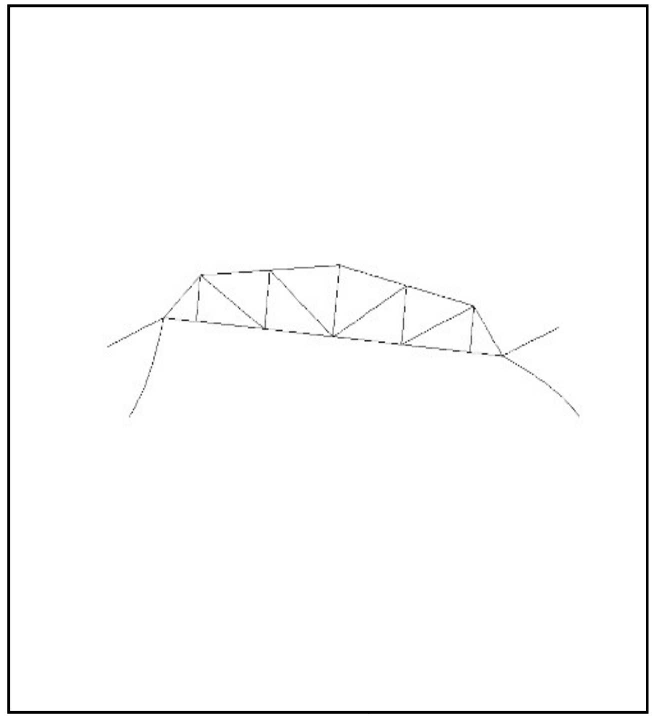


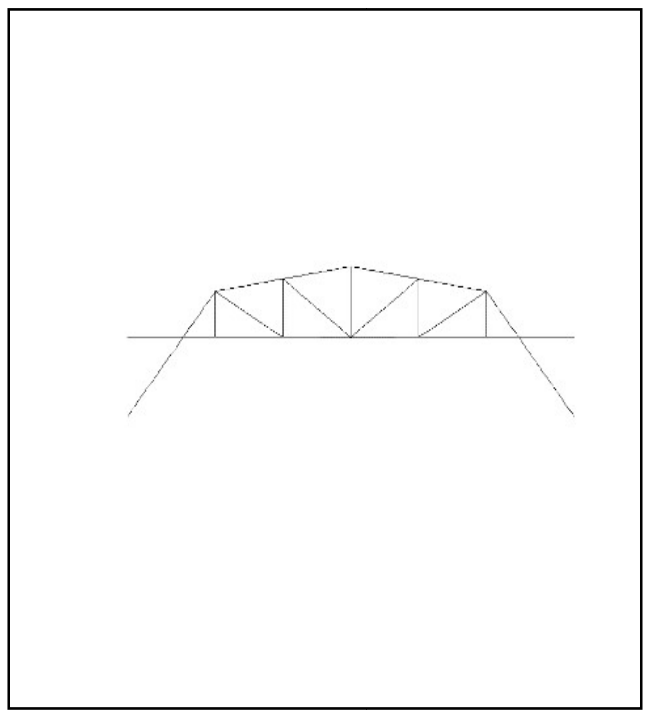
Fig2.Finite Element model of model steel bridge

TABLE 1. VALUES OBTAINED BY FEM (SAP 2000)

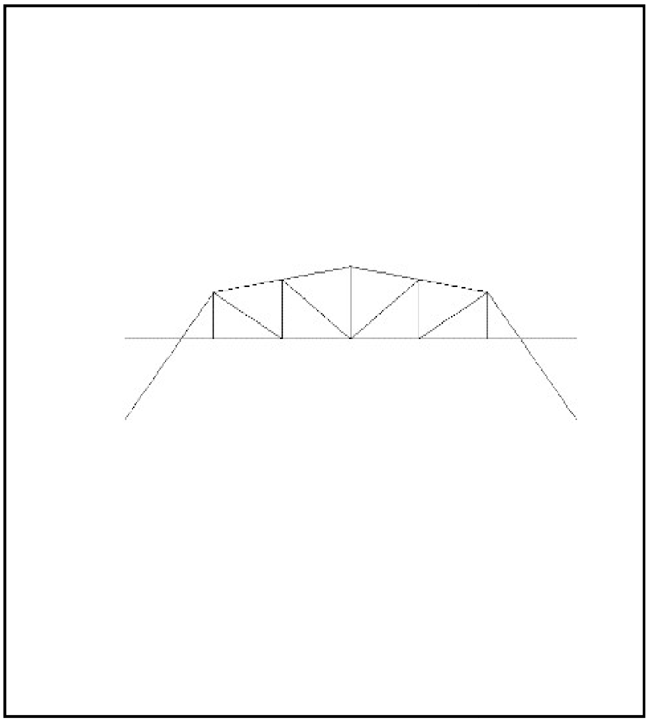
Mode	1	2	3	4	5
Frequency (Hz)	5.743	11.539	17.180	21.097	25.799



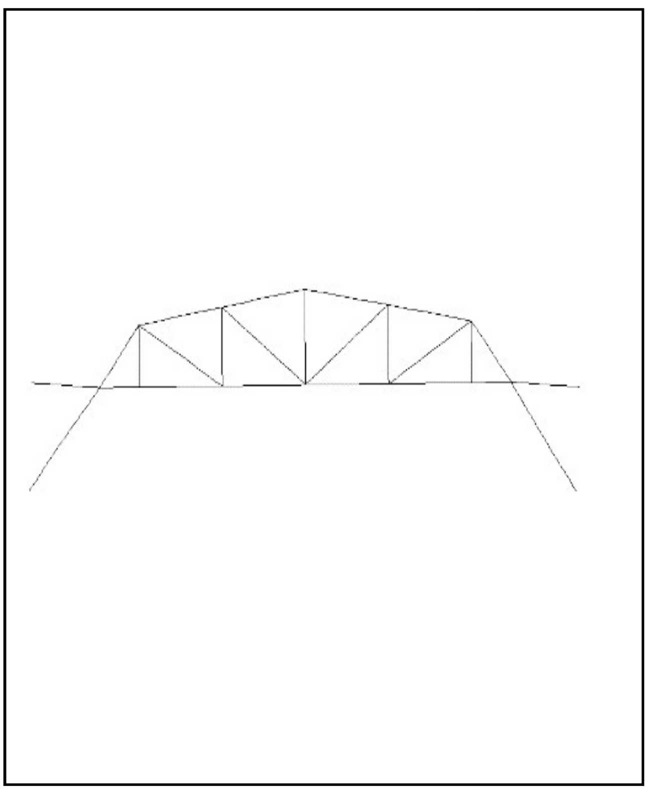
1st Mode Shape (f=5.743 Hz)



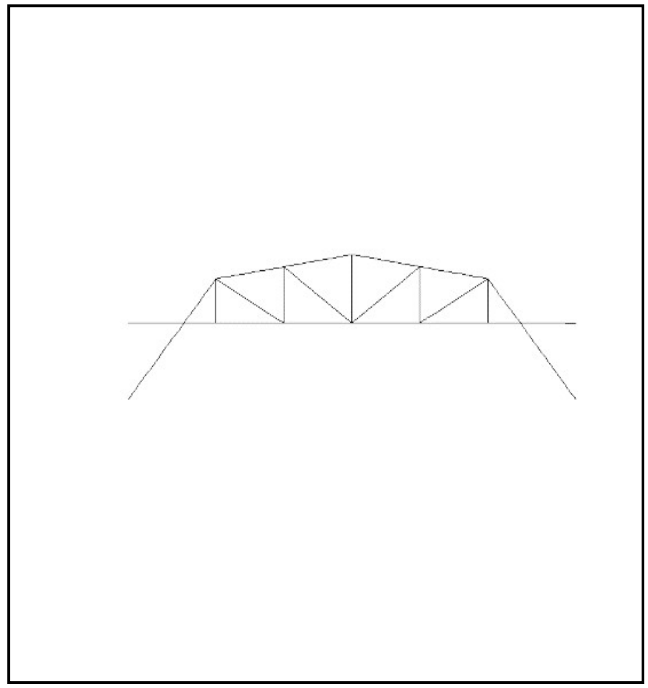
2nd Mode Shape (f=11.539Hz)



3rd Mode Shape (f=17.180 Hz)



4th Mode Shape (f=21.097 Hz)



5th Mode Shape (f=25.799 Hz)

Fig. 3. Analytically identified mode shapes of model steel bridge

V. OPERATIONAL MODAL ANALYSIS OF MODEL STEEL BRIDGE

Two accelerators, one of which is three-axis and the other one, are rigidly fixed to the required nodes. While the accelerometers are being placed, the spirit level is provided to place the millimetric directions to the joints. Thus, it is ensured that the errors that may occur are minimized. Placed accelerometers are fixed with strong adhesive material prevent any possible shift, displacement. The layout of the accelerometers and the reference accelerometer can be clearly seen in the Fig 4.

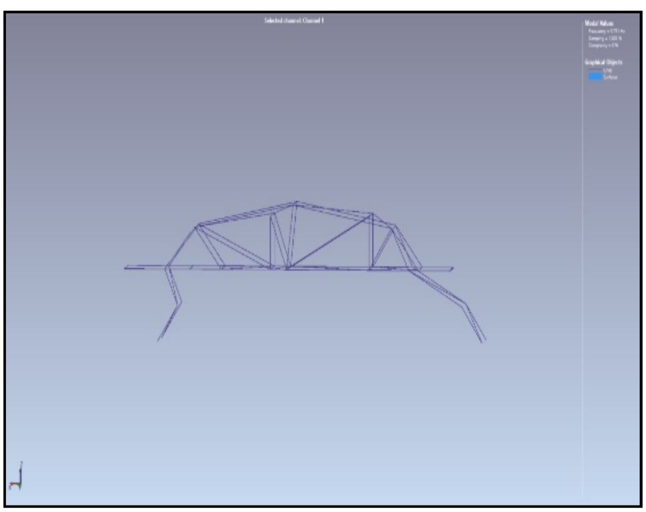
TABLE 2. OPERATIONAL MODAL ANALYSIS RESULT AT THE MODEL STEEL BRIDGE

MODE	Frequency [Hz]	Damping [ξ]
1	5.751	0.018
2	11.605	0.024
3	17.219	0.020
4	21.151	0.026
5	23.949	0.020

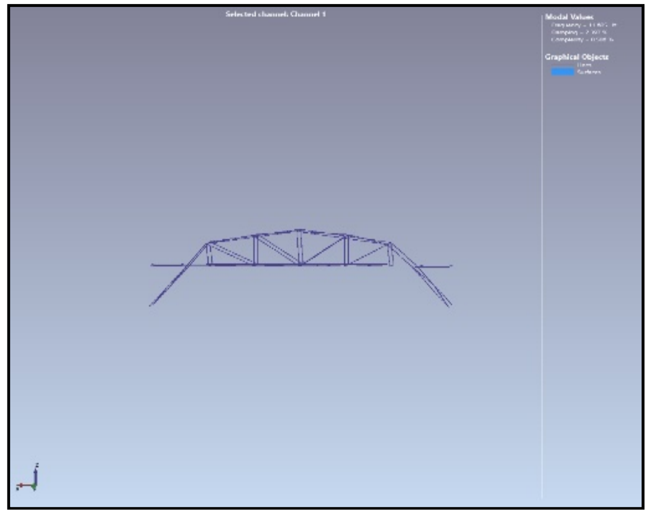
Artemis package program was used for output only modal analysis. Figure 4 shows the accelerometer placed on the model steel bridge and the measuring device. One data logger was used on each floor for measurements. In addition, one seismometer was used to observe the ambient vibration. Thus, a total of three data loggers were used.

Figure 4.Accelerometer placed on the model steel bridge

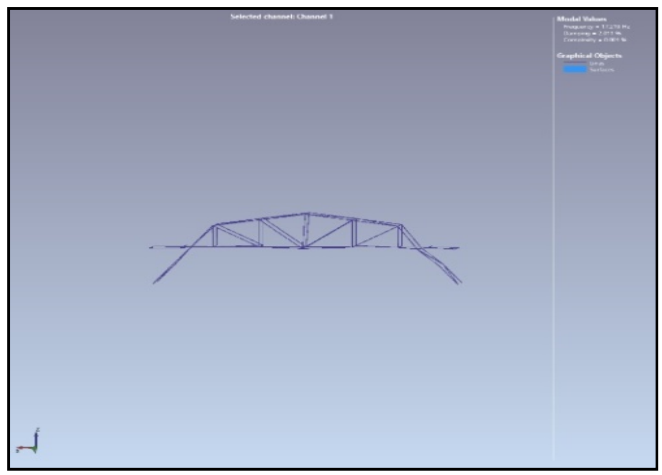
The measurement time was 600 seconds. The direction of the accelerometer is given in the accelerometer layout plan. The data obtained in these directions and measurement times are presented separately in each floor and in each channel. During the measurements, it was taken care not to create white noise and during the measurement, there was no negative effect on the measurement area. The experimental setup is in the laboratory as shown in the figures. In this way, it has been tried to provide a more ideal measurement. Not only during the measurement, but also after the measurement, the data were reviewed and reliable data were obtained by repeating the measurements in any negative situation. Natural frequencies acquired from the all measurement setup are given in Table 2. The first five mode shapes extracted from operational modal analyses are given in Fig.5When all measurements are examined, it can be seen that there is best accordance is found between experimental mode shapes.



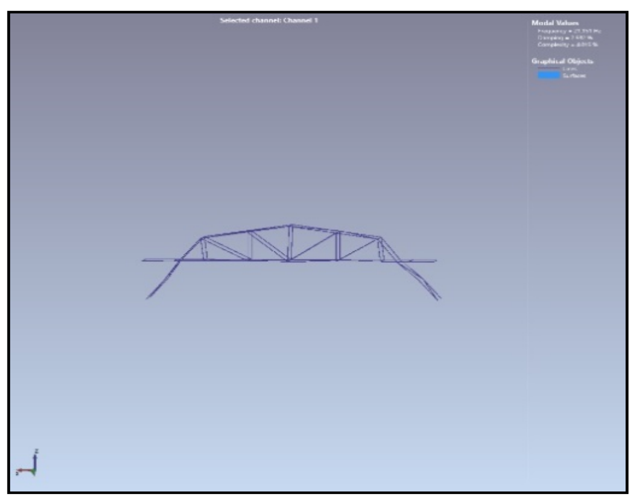
1st Mode Shape SSI-PC (f=5.751Hz, ξ=0.018)



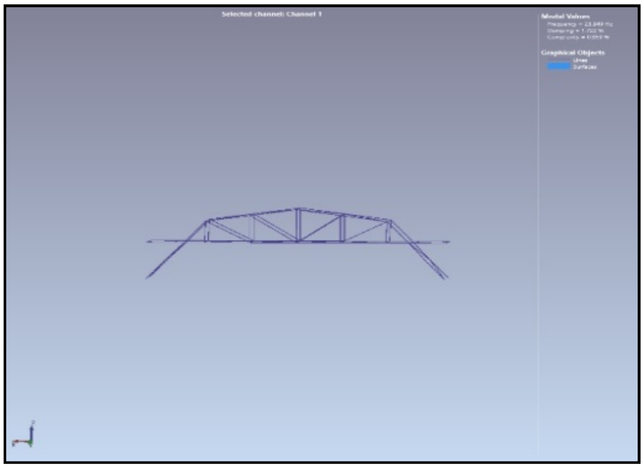
2ndModeShapeSSI-PC (f=11.605 Hz, ξ=0.024)



3rd Mode Shape SSI-PC (f=17.219 Hz, ξ=0.020)



4th Mode Shape SSI-PC (f=21.151 Hz, ξ=0.026)



5th Mode Shape SSI-PC (f=21.151 Hz, ξ=0.026)

Fig. 5Experimentally identified mode shapes of model steel bridge

VI. FEM UPDATING STUDY

This study involved the comparison of the natural frequencies and mode shapes of the experimental model analysis and FE models until an acceptable correlation was achieved. Details of the FE model used for this study and the parameters selected for the model updating is given in the following sections.

6.1 Finite Element Model Calibration of the Structure

A finite element model was generated in SAP2000. Beams and columns were modeled as 3D beam column elements.When modeling the model steel bridge, the modulus of elasticity was taken as E = 2.0x10^5MPa, material density ρ = 7850kg / m^3 and poisson ratio υ = 0.3

6.2 Selection of Parameters for Model Updating

When the table of comparison of the theoretical and experimental frequencies of the model steel bridge is examined, it is seen that there are some differences between the natural frequencies obtained analytically and experimentally results.A sensitivity analysis of the dynamic response of the finite element model of the structure to a change in element properties was first conducted on a large number of parameters. A parameter refers to a selected property of a given element. Mass per unit volume (ρ) was chosen as parameter for sensitivity analysis. Table.3

TABLE 3.MATERIAL UPDATED PARAMETERS

	Before Fem Updating	After Fem Updating
Material	Mass per unit volume ρ(kg/m3)	Mass per unit volume ρ(kg/m3)
Bridge	7850	8240

6.3 The Eigen sensitivity-Based Finite Element Model Updating

In mention method, the relationship between the perturbation in the updating parameters $\delta\{P\}=\{P\}-\{P_{\text{cur}}\}$ and the difference $\delta\{D\}=\{D_{\text{mea}}\}-\{D_{\text{cal}}\}$ between the measured $\{D_{\text{mea}}\}$ and calculation results $\{D_{\text{cal}}\}$ from the finite element model can be represented by a sensitivity matrix $[S]$ as [5]:

$$\delta\{D\}=[S]\delta\{P\} \tag{1}$$

in which $\{P\}$ and $\{P_{\text{cur}}\}$ are updated and current vectors of the updating parameters, respectively; Elements of the sensitivity matrix are determined as:

$$S_{ij}=\frac{\partial\{D_i\}}{\partial\{P_j\}} \tag{2}$$

Where $\{D_i\}$ the *i*-th component of the modal is vector, and $\{P_j\}$ is the *j*-th component of the updating parameter vector. Through differentiating the eigen equation $[k]\{\phi\}=\lambda[m]\{\phi\}$ of a structural system with respect to updating parameters $\{P_j\}$, the derived formula for natural frequencies can be obtained as follows [8]:

$$\frac{\partial\lambda_k}{\partial P_i}=\{\phi_k\}^T\frac{\partial[k]}{\partial P_i}\{\phi_k\}-\lambda_k\{\phi_k\}^T\frac{\partial[m]}{\partial P_i}\{\phi_k\} \tag{3}$$

Where λ_k is the current *k*-theigen value; $\frac{\partial\lambda_k}{\partial P_i}$ is the notation for the sensitivity of the *k*-theigen values λ_k with respect to updating parameter P_i; $\{\phi_k\}$ is the current *k*-th mode shape which is normalized to the mass matrix $[m]$; $[k]$ is the current stiffness matrix. In ambient tests, higher natural frequencies are often obtained with less accuracy than the lower order ones. Therefore, a weighting matrix $[W_P]$, whose entries are often obtained from the reciprocals of the variance of the corresponding modal data, is introduced in the FE model updating algorithm. If only the weighting matrix of the updating parameters $[W_P]$ is considered, the best estimation for the updating parameters can be obtained through the weighted least squares method. In this way, the solution for simultaneous equation (1) can be obtained by considering a constrained optimization problem as follows:

Minimize $\delta\{P\}^T[W_P]\delta\{P\}$ subject to (4)

$$\delta\{D\}=[S]\delta\{P\}$$

Its corresponding solution is

$$\delta\{P\}=[W_P]^{-1}[S]^T\left([S][W_P][S]^T\right)^{-1}\delta\{D\} \quad (5)$$

If both the weighting matrices $[W_P],[W_D]$ are included, the best estimation of the updating parameters can be obtained by the Bayesian estimation technique. The associated FE model updating procedure can be regarded as seeking the solution of the following constrained optimization problem:

Minimize

$$\left(\delta\{D\}-[S]\delta\{P\}\right)^T[W_D]\left(\delta\{D\}-[S]\delta\{P\}\right)+\delta\{P\}^T[W_P]\delta\{P\}$$

Subject to

$$\delta\{D\}=[S]\delta\{P\} \quad (6)$$

The corresponding solution can be obtained as [3]:

$$\delta\{P\}=[W_P]^{-1}[S]^T\left([W_D]^{-1}+[S][W_P]^{-1}[S]^T\right)^{-1}\delta\{D\} \quad (7)$$

In order to avoid the updated results being physically meaningless, the lower and upper limits for the updating parameters are necessarily set in the FE model updating procedure, these are listed in Table 2.

The convergence criteria were also set in each iteration loop as follows:

$$|f_k-f_{\bullet k}|\le \text{Specified limit of natural frequency difference} \quad (8)$$

$$\mathrm{MAC}\left(d_k,d_{\bullet k}\right)_{k=1,n}\ge\alpha \quad (9)$$

$$\{P_{lower}\}\le\{P_k\}\le\{P_{upper}\} \quad (10)$$

Where f_k, $f_{\bullet k}$ are the current analytical and corresponding experimental values of the natural frequency, respectively; $\{P_{lower}\}$, $\{P_{upper}\}$ are the lower and upper limits of the updating parameters, respectively; α is the lower limits of the MAC matrix; n is the compeered appropriate mode's number, another word it is the considered number of compeered degree of freedom of the structural system; $\mathrm{MAC}\left(d_k,d_{\bullet k}\right)_{k=1,n}$ is the modal assurance criterion indices for between the FE computational d_k and experimental $d_{\bullet k}$ mode shapes, which indicate how well the FE mode shapes fit to the corresponding measured ones and calculated as:

$$\mathrm{MAC}\left(d_k,d_{\bullet k}\right)_{k=1,n}=\frac{\left(\sum_{j=1}^{n}\phi_{jk}\phi_{\bullet jk}\right)^2}{\sum_{j=1}^{n}\left(\phi_{jk}\right)^2\sum_{j=1}^{n}\left(\phi_{\bullet jk}\right)^2} \quad (11)$$

In which ϕ_{jk}, $\phi_{\bullet jk}$ are the *j*-th coordinates of the *k*-th analytical and measured mode shapes, respectively. Once all the conditions listed in equations (8-11) are satisfied, the iteration process ends, and the final FE model updated results are obtained.

VII. MODAL UPDATING RESULTS

In order to overcome these differences between natural frequencies and minimize them, the bridge's finite element model should be improved according to the experimental measurement results. It can be seen from the MAC graph (Fig.6-7) that the 90% approach in the mode shapes nearly reached 100% after the ±5% increase in mass density, which is made from the material properties. Table 3.

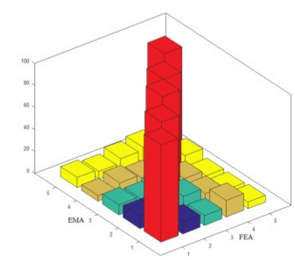


Figure 6. 3D view of the parameters-shape modes response.3D plots of MAC matrices to five mode shapes of structure before updating parameters.

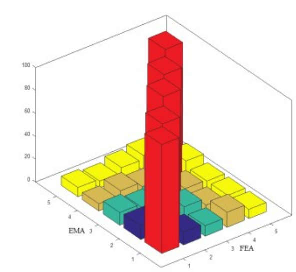


Figure 7. Comparison of 3D plots of MAC matrices to five mode shapes of structure after updating parameters

VIII. CONCLUSION

In this paper, analytical and experimental modal analysis of model steel bridge was presented. Comparing the result of study, the following observation can be made:

From the finite element model of model steelbridge a total of

5 natural frequencies were attained analytically, which range between 5 and 25 Hz. 3D finite element model of model steel bridge is constructed with SAP2000 software and dynamic characteristics are determined analytically. The ambient vibration tests are conducted under ambient vibration data on ground level. Modal parameter identification was implemented by the Stochastic Subspace Identification Technique-PC(SSI-PC). Comparing the result of analytically and experimentally modal analysis, the following observations can be made:

The modal parameters obtained experimentally were used to calibrate a finite element model of the building. MAC values were generated between analytical and experimental mode shapes. Main difference between mode shapes of the FEM and EMA was explained.

Based on the eigensensitivity-based FE model updating procedure a summery of the changes the FEM results to the EMA results is presented graphically and numerically in percent to the initial state of the structure. As seen from the modal updating from the MAC graph (Fig.6-7) that the 90% approach in the mode shapes nearly reached 100% after the ±5% increase in mass density, which is made from the material properties (ρ).As seen from the mac graphics, it was a complete overlap.

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