Damage Characterization in Building Structures Due to Blast Actions

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ABSTRACT

Structural identification is a technique that can be used to assess/characterize the damage state through the variation in eigenfrequencies, damping ratios and modal shapes in a structure or element. It has recently received more attention for the practical implementation in several fields, including damage assessment for structures following blast or explosion events. At present, large infrastructure components, like civil engineering structures, are the most convenient issue of consideration for structural identification. Structures can be moderately or severely deteriorated due to accidental or intentional blasts or explosions. Structural engineers and other stakeholders, like rescue and emergency agencies, are more concerned about the design of structures, design life span, proper maintenance, repair and residual capacity of structural systems in many countries. This research work focuses on the experimental and analytical modal analysis of a full-scale steel frame structure building aiming to develop coherent scenarios that combine the probability of the hazard event with the structural vulnerability in case of a close-in detonation. Field tests were carried out by forced vibration testing under hammer excitations. First, a series of tests were conducted for the undamaged structure using classical experimental modal analysis. Then, in order to model a structural damage, a secondary beam was dismantled (thus a damage was created artificially) and the measurements were repeated. The change in structural behaviour was observed by identifying the changes in the stiffness and natural frequencies of the structure. The modal parameters measured from field tests were then used to validate finite element models using SAP2000 program. They were corrected, so that the numerical natural frequencies and mode shapes match the experimental data. Good agreement was found in identifying the frequencies for the three-dimensional finite element models for both damaged and undamaged structures. Then, using the calibrated numerical model, several blast-induced damages were used in a numerical study. For the internal damage or non-visible crack, four different damage scenarios were made by the FE model for internal and external blast actions. The modal parameters changed significantly for higher modes for higher reduction of stiffness at the column-beam and base connections. The results (experimental data, calibrated numerical model) can be used as reference values of the undamaged structure for further investigations after blast tests.

KEYWORDS: SHM, Modal parameters, FEM modelling, Damage characterization, EMA.

INTRODUCTION

Structural identification is getting more importance through finite element model updating and experimental modal analysis technique to assess dynamic properties, as well as structural health and performance monitoring (Timothy Kernicky et al., 2017). Structural identification basically uses performance-based civil engineering modeling that is performed by field/experimental measurements and the validation is done by using numerical models. In the study, SAP2000...
program was used for calibration of the experimental data obtained using Bruel and Kjaer vibration measurement technology and equipment. Structural design validation, practical quality control of construction, performance for retrofitting and rehabilitation effectiveness, damage detection and lifecycle analysis for long-term performance and structural health and performance monitoring (Çatbaş et al., 2013) are promising issues for structural identification and characterization (Timothy Kernicky et al., 2017). Structural identification studies for large civil engineering structures, like long-span bridges, high-rise buildings, wind turbines, offshore structures and towers, are subjected to undesired and/or unexpected hazards. In case of blast action, structural deterioration of the frame building, structural responses for pre-blast and post-blast conditions are conducted by blast overpressure transducers and shock accelerometers. Performance-based (Aktan, A. et al., 2013) analysis for civil engineering structure and health monitoring has some key challenges for structural identification because of uncertainties of parameter estimation and the necessity of finding alternative better solutions for physical properties of the structures. The main goal of model updating is to find the solution to get the best possible match of stiffness and mass matrices of an analytical model of the structure to the experimentally measured values (Timothy Kernicky et al., 2017). Two important methods: deterministic and probabilistic, are used for finite element model updating. The uncertainty parameters of the model can be assessed by deterministic methods for structural identification of several full-scale structures (Bakir et al., 2008; Deng, L. et al., 2009; Marwala, T., 2010). Modal parameter estimation is performed through system identification using both deterministic and stochastic Subspace Identification (SSI) system algorithms. Structural identification for the existing structures is an auspicious solution for decision-making to minimize unnecessary cost for repairing, retrofitting and replacement (Romain Pasquier et al., 2016). Unexpected errors of modeling can be minimized for the existing civil engineering structures by a structural identification process based on residual minimization approaches (Yarnold et al., 2015; Fontan et al., 2014; Baroth et al., 2010; Schlune et al., 2009). Physical properties and structural conditions from the identification results are observed by the diagnostic process. Higher number of sensors is required for computing structural identification for diagnosis and prognosis of existing structures.

The response of the structures under reacting forces defines the structural behavior that is urging to analyze. External forces can be introduced in different ways and characterize the dynamic properties (modes and natural resonant frequencies). Modal parameters are important, because they describe the inherent dynamic properties of a structure. The set of modal parameters constitutes a unique set of numbers that can be used for model correlation and updating, design verification, benchmarking, troubleshooting, quality control or structural health monitoring. The structure is excited with a hammer or shaker and its response is measured with accelerometers. Techniques, like operational modal analysis (OMA) and operating deflection shape analysis (ODS), work while the structure is in operation, allowing to get a realistic picture without having to artificially excite the structure (Bruel and Kjaer module: Type 7765, 7765-A, 7765-B and 8761).

The three important modal parameters of natural frequencies, damping ratios and mode shapes have become a major concern for representing structural dynamics. Many researchers did a lot of work related to modal parameter estimation for describing the dynamic behavior of structures.

R. Cantieni (2004) conducted a very valuable research on experimental methods used in system identification of civil engineering structures, like buildings, bridges, dams, wind turbines, towers and road networks that are vibrated due to dynamic forces. Brownjohn et al. (2010) examined the dynamic behavior of Humber bridge based on operational modal analysis in ambient conditions. Ahmet Can Altunışık et al. (2017) experimented the structural identification of a cantilever beam with multiple cracks by three different
operational methods (EFDD, CFDD and SSI) and the modal parameters were verified by finite element tool ANSYS. J. B. Hansen et al. (2017) proposed “a new scenario-based approach to damage detection using operational modal parameter estimates”. He did vibration-based damage introduction and identification by the modal parameters. Structural health monitoring (SHM) of structures was measured by the damage detection techniques of four important factors: detection, localization, quantification and prognosis (A. Rytter, 1993). Mustapha Dahak et al. (2017) measured the physical properties of cantilever beam and made it flexible to reduce natural frequencies. Damages were introduced in the different zones on the cantilever beam, while experimental investigation was done for measuring the modal parameters and ANSYS software was used for the verification of the experimental results. MAC value can correlate the modal shapes of the damaged and undamaged structures (W. M. West, 1984). J. Zhang et al. (2013) presented structural identification for long-span bridge by three separate post-processing experimental methods: Peak Picking, PolyMAX and Complex mode indicator function. Aktan, A. E. et al. (1997) identified structural parameters by experimental analysis and calibrated them by numerical models. Kijewski-Correa et al. (2007) measured the dynamic parameters experimentally and performed validation by finite element tools. Conte J.P. et al. (2008) identified normalized vibration mode shapes using MNExT-ERA based on ambient vibration data (S = Symmetric; AS = Anti-Symmetric; H, V and T = Horizontal, Vertical and Torsional mode, respectively). Wei-Xin Ren et al. (2004) conducted laboratory experimental testing to measure the modal parameters of a steel arch bridge in operating conditions. Álvaro Cunha et al. (2004) applied output only modal identification methods to perform modal parameter extraction for different types of civil engineering structures. Brian J. Schwarz et al. (1999) conducted “experimental modal analysis” for measuring the FRF (modal parameters) by using the FFT analyzer and a set of FFT curve fitting. Ibsen et al. (2006) represented on “experimental modal analysis” for wind turbine and estimation of modal parameters by using ambient response testing and modal identification (ARTeMIS). Dongming Feng et al. (2017) introduced an advanced technique to monitor the structural health in a cost-effective way. Advanced noncontact vision-based systems offer a promising alternative of conventional experimental methods. M. Molinari et al. (2009) represented on “damage identification of a 3D full-scale steel–concrete composite structure with partial-strength joints at different pseudo-dynamic load levels” and highlighted system identification methods for damage detection, localization and quantification by measuring actual modal parameters of a structure. Ruqiang Yan et al. (2015) discussed the structural health monitoring of a spindle by stochastic subspace identification (SSI), using Peeters B. et al. (1999) experimental methods in operating conditions. M. Hassan Haeri et al. (2017) introduced an innovative method; namely, inverse vibration technique, for offshore jacket platforms. Jia He et al. (2017) applied two sets of investigation: MR dampers were employed for vibration control and the EKF-based approach was used for damage detection for Kobe earthquake and Northridge earthquake. G. Acunzo et al. (2017) proposed a new method, Multi Rigid Polygons (MRP) model, for building modal estimations. E. Peter Carden et al. (2008) examined structural health monitoring (SHM) for a civil engineering structure, which was done by Autoregressive Moving Average (ARMA) model to detect and locate damage. T. H. Ooijevaar et al. (2010) developed damage detection methods applied experimentally for a carbon fiber PEKK-reinforced composite T-beam. S. Nagarajaiah et al. (2009) introduced a new technique for modal parameter estimation. He applied output-only modal identification for structural damage detection.

Research Objectives and Scope

Experimental validation is the most reliable way to demonstrate the performance of a building structural system. However, there are still some uncertainties about the expected performance of the building system...
when subjected to blast load because of the many variables included in the process. The application of structural identification for full-scale laboratory testing of a steel frame building subjected to blast allows a better understanding of blast effects, including the post-blast condition and residual capacity of the building structure. The research mainly focused on the assessment of dynamic properties of a full-scale steel frame building model and some preliminary evaluations regarding the structural vulnerability in case of a close-in detonation.

Application of St-Id in Structural Engineering

The development of the society and the need for more advanced, more economical and longer lifetime of buildings and infrastructures, like tall buildings, dams, large cable stayed or suspension bridges, towers, wind turbines, aircraft structures or other special structures, require adequate methods and tools to allow for accurate structural identification of the most relevant static and dynamic properties. Numerical modelling, although it is a powerful tool that witnessed important developments in the past decades, requires the validation of the results through analytical and/or experimental means.

A lot of research work had been done in the past to identify the dynamic characteristics of civil engineering structures. Some applications of Bruel and Kjaer experimental modal analysis (EMA) for estimation of modal parameters for different structures are presented in Figures 1-5. Many researchers have conducted experimental modal analysis and analytical modal analysis for long-span bridges, tall buildings, traffic roads, towers, wind turbines,… and so on. A lot of research related to structural identification and structural health monitoring has been done by different researchers based on experimental modal analysis for civil engineering structures.

Figure (1): Modal parameter estimation for wind turbine (Bruel and Kjaer manual: type 8760)
Figure (2): Modal parameter estimation for bridge (Abdurrahman Sahin et al., 2016)

Figure (3): Modal parameter estimation for a beam structure (Bruel and Kjaer manual: access code: 636 832 431, 2017)
Figure (4): Modal parameter estimation for aircraft (Brue and Kjaer manual: Type 8761)

Figure (5): Different physical mode shapes by experimental modal analysis (Peter, 2017)
Damage Detection by Experimental Modal Analysis

Structural health monitoring after long-term operation is the key source to detect damage. Time-to-time health monitoring indicates physical changes of structures, which helps identify damage occurrence, location of damage and severity of damage. Mainly, damage indicators are the key parameters as input and they are unified to a single control value with a corresponding statistical threshold. This threshold acts as a control chart for identifying damage automatically when the threshold is being passed after an analysis. As example, a bridge is taken (Figure 6) for detecting damage after long-term service of that bridge, where eight reference measurements representing the undamaged state were performed. Eight measurements were recorded for the undamaged bridge and other 14 measurements were taken after introducing damage. The first six measurements out of the eight measurements were used as a baseline (reference) model determined by the module. The threshold is automatically estimated based on statistical evaluation of the damage indices of the six reference measurements. The last two reference measurements remain below the reference threshold (green bars). This means that the bridge is still serviceable or undamaged. The last 14 measurements that were recorded after damage introduced all pass the threshold significantly and indicate a permanent damage (red bars). Mode tracking was done as well. The lower left display indicates that the first two modes are basically unaffected by the damage, whereas the natural frequency for the highest mode changes, disappears completely during the first set of damage measurements and reappears again later. Tracking of the third mode is impossible after damage is introduced (Bruel and Kjaer product document: OMA Pro BZ-8553).

Figure (6): Damage detection of bridge by operational modal analysis
(Bruel and Kjaer manual: Type 8760-8762; OMA Pro BZ: 8553)
Building Frames under Blast Action

Blast loading is an explosion with a rapid release of stored energy characterized by bright flash and an audible blast. A part of the energy is released as thermal radiation (flash), while another part goes into the air as an air blast and into the soil as a ground shock, both as rapidly expanding shock waves. Blast loads on structures can be classified into the two following main groups on the basis of confinement of the explosive charge. Unconfined explosion includes free air burst, air burst and surface burst explosion having un-reflected and reflected loads, respectively. Confined explosion includes fully vented explosions, partially confined explosions and fully confined explosions.

Blast loading effects on structural members may produce both local and global responses associated with different failure modes (Figure 7-11). The type of structural response depends mainly on the loading rates, the orientation of the target with respect to the direction of the blast wave propagation and boundary conditions. Failure modes accompanying global response are: flexure, direct shear or punching shear. Failure modes associated with local response (close-in effects) are: localized breaching and spalling.

Figure (7): Column responses subject to near-contact blast charges (T. Brewer et al., 2016)

![Image of column responses](image_url)

Figure (8): Time history function of blast wave pressure on building (Islam, 2016)

![Graph of blast wave pressure history](image_url)
Figure (9): Typical pressure-impulse diagrams associated with increasing levels of damage (Fulvio Parisi et al., 2016)

Figure (10): Overpressure-distance diagram to buildings (Török et al., 2015 and FEMA-IS156)
Research Methodology

Description of Frame Building Model

The full-scale building model is a two-span, two-bay and two-story steel structure (Figure 12). The bays and spans measure 4.50 m and 3.0 m, respectively, while each story is 2.5 m high. The structural system is made of moment-resisting frames in the transversal direction, while in the longitudinal direction it is made of concentrically braced frames placed on perimeter frames. The extended end-plate bolted beam-to-column connections in the moment-resisting frames are designed as fully rigid and fully restrained connections. Secondary beam-to-column connections and secondary beam-to primary beam connections are pinned connections. Columns are rigid at the base. The design of the structure for permanent and seismic (low seismicity, 0.10 g horizontal acceleration) design conditions resulted in an IPE 270 section for main beams; IPE 200 section for main secondary beams; and secondary beams between frames were of IPE 160 section, while columns were of HEB 260 section. Note that structural steel S275 (yield strength of 275N/mm²) was used for beams and columns (Dinu et al., 2017-2018).

Instrumentation and Vibration Measurements on Initial (Undamaged) Structure

The experimental test for full-scale building in Petrosani was done in two distinct phases. First step is to fix each component of the instrument. Bruel and Kjaer experiment consists of many important tools for measuring vibrations. Accelerometer, impact hammer, force transducer, laptop and connecting cables are the most important parts of the total set of instruments. Transducer and laptop are connected by the connecting cables. The model of the building frame is drawn by the numerical tool SAP2000. For experimental modal analysis, geometry is needed. It can be drawn directly there or can be exported from AUTO CAD file (str file; uff), file formats and SAP2000 in dxf file format. The model was exported from SAP2000 and imported to the MTC hammer measurement software. The external longitudinal frame and internal transverse frame were selected for measuring the vibration. Corner column and central column of the longitudinal frame were considered for experimental testing. The location of the best points to excite the structure was determined so as to create almost equal levels of response in the several modes of interest. A 3.20 lb black and hard hammer is
attached at a distance of 0.625m, 1.25m and 1.875m height from the bottom of the column carefully in plane and out of plane. The positions where the average response level is low will be better locations to attach the accelerometers. Accelerometers are very sensitive. If they are not fixed properly to the position of the structure vibration, measurement data will not be accurate. Accelerometers are fixed perfectly to the position of measurement of the structure to make it a proper connection with the member. For corner column, longitudinal and transverse frame 1st accelerometer was fixed at the base of the column. The next accelerometers were positioned by keeping equal clear distance of 1.25m. The last accelerometer is positioned at the top (5.0m from bottom) of the column. All accelerometers are positioned in strong and weak axes for measuring vibration in-plane and out of plane of the structural frame. Two accelerometers are placed at the mid length (2.25m from the end of beam) of the longitudinal beam for out of plane measurement, but not for in-plane measurement (Figure 13-15). No accelerometers were fixed in the secondary beam. After positioning all accelerometers, very sensitive cables were fixed in one end of the accelerometers and the other end of the signal processing device LAN-XI. The point number on the frame and the channel number in the LAN-XI device must be the same for best estimation of response signal. Accelerometers, hammers and channel number are reconfigured by MTC to make sure that all connections are fixed properly.

Figure (12): Views and details of full-scale building frame: 3D view
Figure (13): Positioning of accelerometers for the longitudinal frame (in plane)

Figure (14): Positioning of accelerometers for the longitudinal frame (out of plane)
Once the set-up is completed, the second step was to perform experiments and measure the vibration of the frame for excitation by the hammer. Measuring time, frequency span, average number of readings, number of hits for excitation, proper hits detection and property for checking the frequency range are very essential to be set before starting measurement. The total length of measurement time is (6400/1000) 6.4 seconds and the span of frequency for FFT was fixed as 1000Hz. Five measurements were recorded for average. The more number of measurements for average, the more accuracy of the measurement.

Frequency domain method for measuring frequency response $H_1$ is used for exporting spectrum for experimental modal analysis. Hammer weighing, response weighing and hammer tip test were properly performed during the measurement by uniform response variation. After measurement, the initial mode shape can be observed from MTC hammer test FRF validation operation. The exact modal parameters are conducted by pulse reflex experimental modal analysis for different numbers of iterations and interpretations.

**Experimental Results and Discussion**

**Modal Parameter Estimation for Longitudinal Frame Column (in-Plane)**

The results obtained from the Bruel and Kjaer experimental modal analysis and finite element analysis are matched with good agreement if there is a smaller number of factors that affect the results. The factors of uncertainties are mainly responsible for error in the obtained results. In that case, additional calibration/updating is needed for EMA and the model
is updated by the numerical software. The finite element numerical tools represent the best solution to correlate, modify and update the modal parameters (mode shapes, natural frequency and damping). Finite element software SAP2000 was used for correlations and updating of the experimental results. The natural frequencies are determined from an eigenvalue analysis of the 3D FEM and these results are compared with the forced vibration data using the EMA post-processing method. The vibrations of the structure were recorded by means of 10 sensitive accelerometers and the sensitivity of accelerometers was 1 V/g. A picture of the test structure including position and orientation of the accelerometers for longitudinal frame (in-plane) is shown in Figure 13. The comparative presentation of modal parameters for longitudinal frame (in-plane) is illustrated in Table 1.

<table>
<thead>
<tr>
<th>Mode</th>
<th>Frequency (Hz)</th>
<th>Damping (%)</th>
<th>Complexity (%)</th>
<th>SAP2000 Frequency (Hz)</th>
<th>Error (%)</th>
<th>Mode type</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>12.54</td>
<td>1.22</td>
<td>0.00</td>
<td>14.06</td>
<td>10.81</td>
<td>Bending</td>
</tr>
<tr>
<td>2</td>
<td>51.42</td>
<td>1.36</td>
<td>0.00</td>
<td>51.55</td>
<td>0.25</td>
<td>Bending</td>
</tr>
<tr>
<td>3</td>
<td>126.37</td>
<td>0.32</td>
<td>0.00</td>
<td>120.81</td>
<td>-4.40</td>
<td>Bending</td>
</tr>
<tr>
<td>4</td>
<td>150.38</td>
<td>2.23</td>
<td>0.04</td>
<td>151.38</td>
<td>0.66</td>
<td>Bending</td>
</tr>
<tr>
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<td>0.85</td>
<td>0.09</td>
<td>164.21</td>
<td>-2.62</td>
<td>Bending</td>
</tr>
<tr>
<td>6</td>
<td>192.17</td>
<td>2.78</td>
<td>0.03</td>
<td>194.79</td>
<td>-1.72</td>
<td>Bending</td>
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<table>
<thead>
<tr>
<th>Mode</th>
<th>EMA (Hz)</th>
<th>SAP2000 initial (Hz)</th>
<th>SAP2000 corrected (Hz)</th>
<th>Damping (%)</th>
<th>Error (%)</th>
<th>Mode type</th>
</tr>
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<tr>
<td>1</td>
<td>12.54</td>
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<td>12.53</td>
<td>1.22</td>
<td>0.08</td>
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<td>51.55</td>
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<td>Bending</td>
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<td>125.59</td>
<td>0.32</td>
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<td>Bending</td>
</tr>
<tr>
<td>4</td>
<td>150.38</td>
<td>151.38</td>
<td>150.04</td>
<td>2.23</td>
<td>0.23</td>
<td>Bending</td>
</tr>
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<td>164.21</td>
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<td>190.79</td>
<td>188.39</td>
<td>2.78</td>
<td>1.97</td>
<td>Bending</td>
</tr>
</tbody>
</table>

**Finite Element Model Calibration and Updating**

The main aim of the finite element model updating procedure is to correct the initial FEM values of the selected parameters to minimize errors between experimental result and numerical results. The main steps to calibrate the model values are: i) development of an initial vibration-based FEM model and identifying the dynamic characteristics by experimental measurement of the actual structure and ii) calibrating and validating the vibration-based FEM model to suit the objectives of the St-Id application. Hence, the calibration step is one of the most important tasks of
St-Id of a structure. In this study, the model is corrected by modifying the stiffness at the beam-to-column connections and column-base connections.

Comparison of frequencies before and after model calibration is summarized in Table 2 and shows good improvement in frequencies. Parametric study was then introduced to find the optimum level of allowable parameter change to improve the results of the updated FE model. Here, the stiffness of connections was chosen as the parameter for the parametric study. In the parametric study, the stiffness of beam-to-column and column-base connections was reduced by 53% and 32%, respectively. Figure 16 illustrates the relationship between error in frequency (against EMA frequency) and the corrected FEM value. The table shows the EMA frequencies and the FEM frequencies before and after model calibration for the first six modes. From Table 2, it can be seen that five out of six modes of the calibrated FEM are in excellent match with the corresponding EMA modes with only 0.08% or less error. The largest error of 4.77% is with the fifth mode which still shows a very good numerical-experimental correlation for practical modelling purposes, especially when considering the low frequency characteristics of this particular mode as well as the scale of this building structure. Mode shapes are found after experimental data interpretation and numerical model updating by calibration for longitudinal frame in plane, as shown in Figure 17.

**Figure (16): Experimental measurement and numerical FEM corrected model value for longitudinal frame (in plane)**
Figure (17): Comparison of experimental and numerically estimated mode shapes for longitudinal frame (in plane)
Modal Parameter Estimation for Longitudinal Frame (out of Plane)

Modal parameter estimation was done by the same procedure for longitudinal frame (out of plane) that was used for the longitudinal frame (in-plane). 12 accelerometers and 6 reference hammers (DOF) are fixed out of plane of the frame. The positioning and orientation of the accelerometers and hammers for longitudinal frame are shown Figure 14. Modal parameters were measured by data interpretation for longitudinal frame out of plane as shown in Figure 18. Initial and calibrated estimated experimental and numerical modal parameters for longitudinal frame (out of plane) are presented in Table 3. Mode shapes are found after experimental data interpretation and numerical model updating by calibration for longitudinal frame out of plane as illustrated in Figure 19.

Table 3. Corrected modal parameters with respect to EMA and initial and calibrated SAP2000 model value for longitudinal frame (out of plane)

<table>
<thead>
<tr>
<th>Mode</th>
<th>Modal parameter estimation</th>
<th>Error (%)</th>
<th>Mode type</th>
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<tbody>
<tr>
<td></td>
<td>EMA (Hz)</td>
<td>SAP2000 initial (Hz)</td>
<td>SAP2000 corrected (Hz)</td>
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<td>106.49</td>
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<td>110.63</td>
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</tr>
<tr>
<td>6</td>
<td>334.18</td>
<td>333.34</td>
<td>334.09</td>
</tr>
</tbody>
</table>
Figure (19): Comparison of experimental and numerically estimated mode shapes for longitudinal frame (out of plane)
Modal Parameter Estimation for (Damaged) Longitudinal Frame (out of Plane)

The physical properties (stability, stiffness, modal mass, etc.) of the frame changed for dismantled one or more secondary beams. The secondary beam was removed for observing the changes of modal parameters as shown in Figure 20. Experimental modal parameters were computed by the same procedure described for longitudinal frame (out of plane). 11 accelerometers and 6 reference hammers (DOF) were fixed out of plane of the frame. The positioning and orientation of the accelerometers and hammers for longitudinal frame (out of plane) are shown in Figure 15. Initial modal parameters were measured by data interpretation for longitudinal frame (out of plane) after secondary beam dismantling. The modal parameters were corrected by reducing the stiffness of base connections along both axes after calibration as presented in Table 4. The table shows the EMA frequencies and the FEM frequencies before and after model calibration for the first six modes. From Table 4, it can be seen that five out of six modes of the calibrated FEM are in excellent match with the corresponding EMA modes with only 0.04% or less error. The largest error of 8.97% is with the 4th mode as shown in Figure 21. Mode shapes are found after experimental data interpretation and numerical model updating by calibration for longitudinal frame out of plane as illustrated in Figure 22.

Experimental Results and Discussions

The frequencies measured for undamaged and damaged frames have significant changes for three specific modes for longitudinal frame (out of plane). For the 2nd mode, the frequency is 75.48 Hz for undamaged condition and 75.75 Hz for damaged condition from EMA. On the other hand, numerically, this value is 74.77Hz and 75.78 Hz for both conditions, respectively. The mode calibration is good enough for EMA and SAP2000 in mode 4 for damaged condition. The frequency is 226.66Hz and 222.96 Hz for EMA and SAP2000, respectively for longitudinal frame (out of plane). But, for removing secondary beam, there is significant difference in the frequency for undamaged and damaged conditions. In undamaged condition, the frequency is 226.66Hz, but it is 177.56Hz (Table 5) for damaged condition in EMA. Numerically, there are also big changes of frequency in the same mode. Frequency is 222.96 Hz and 193.49 Hz for undamaged and damaged conditions, respectively [Figure 23 and Figure 24]. Experimentally and numerically, the structural properties changed significantly for the 6th mode. The frame is more flexible because of low frequency (334.18Hz and 277.30Hzs, respectively for both conditions). Except for the 4th and 6th modes, all other modes have no big differences of frequency for both conditions in EMA and SAP2000. The 4th and 6th modes are affected more due to physical property changes after dismantling of secondary beam.

<table>
<thead>
<tr>
<th>Mode</th>
<th>EMA (Hz)</th>
<th>SAP2000 initial (Hz)</th>
<th>SAP2000 corrected (Hz)</th>
<th>Damping (%)</th>
<th>Error (%)</th>
<th>Mode type</th>
</tr>
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<tr>
<td>1</td>
<td>19.49</td>
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<td>96.14</td>
<td>110.25</td>
<td>0.97</td>
<td>-4.50</td>
<td>Bending</td>
</tr>
<tr>
<td>4</td>
<td>177.56</td>
<td>186.54</td>
<td>193.49</td>
<td>1.35</td>
<td>-8.97</td>
<td>Bending</td>
</tr>
<tr>
<td>5</td>
<td>268.05</td>
<td>275.07</td>
<td>275.04</td>
<td>0.49</td>
<td>-2.61</td>
<td>Bending</td>
</tr>
<tr>
<td>6</td>
<td>277.30</td>
<td>285.77</td>
<td>284.88</td>
<td>0.31</td>
<td>-2.73</td>
<td>Bending</td>
</tr>
</tbody>
</table>

Table 4. Corrected modal parameters with respect to EMA and initial and calibrated SAP2000 model value for longitudinal damaged frame (out of plane)
Table 5. Correlation of frequency for longitudinal frame (out of plane) for undamaged and damaged frame

<table>
<thead>
<tr>
<th>Mode</th>
<th>Measured undamaged $f_n$ (Hz)</th>
<th>Calibrated model undamaged $f_n$ (Hz)</th>
<th>Measured damaged $f_n$ (Hz)</th>
<th>Calibrated model damaged $f_n$ (Hz)</th>
<th>Mode type</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>16.91</td>
<td>15.37</td>
<td>19.49</td>
<td>18.79</td>
<td>Bending</td>
</tr>
<tr>
<td>2</td>
<td>75.48</td>
<td>74.77</td>
<td>75.75</td>
<td>75.78</td>
<td>Bending</td>
</tr>
<tr>
<td>3</td>
<td>106.49</td>
<td>110.63</td>
<td>105.50</td>
<td>110.25</td>
<td>Bending</td>
</tr>
<tr>
<td>4</td>
<td>226.66</td>
<td>222.96</td>
<td>177.56</td>
<td>193.49</td>
<td>Bending</td>
</tr>
<tr>
<td>5</td>
<td>277.29</td>
<td>275.12</td>
<td>268.05</td>
<td>275.04</td>
<td>Bending</td>
</tr>
<tr>
<td>6</td>
<td>334.18</td>
<td>334.09</td>
<td>277.30</td>
<td>284.88</td>
<td>Bending</td>
</tr>
</tbody>
</table>

Figure (20): Dismantled secondary beam (red marked beam)
Figure (21): Frequency measured by EMA and SAP2000 for longitudinal damaged frame (out of plane)
Figure (22): Comparison of experimental and numerically estimated mode shapes for longitudinal damaged frame (out of plane)
Figure (23): Frequency measured by EMA for longitudinal damaged frame (out of plane)

Figure (24): Frequency measured by SAP2000 for longitudinal damaged frame (out of plane)
Figure (25): View of the structure with the position of the blast charge for external and internal blast tests

Figure (26): Relative frequency deviation between intact frame and damaged frame

**INTACT FRAME VS DAMAGE SCENERIO DC3-4**

90% REDUCTION OF STIFFNESS AT BASE CONNECTION

Figure (26): Relative frequency deviation between intact frame and damaged frame
Blast Testing and Damage Prediction

The stiffness values at beam-to-column connection and column-base connection of the intact frame are 32000 kNm/mrad and 1100000 kNm/mrad, respectively. The building will be subjected to blasts (TNT or equivalent) with different charges in sizes and locations, resulting in different scaled distances. As the scaled distance reduces, the peak overpressure increases, thus causing the shear failure of the elements located in the proximity. The potential for progressive collapse following local damage will be also investigated in the column and beam.

The column web will be affected by the external charge (Figure 26 a) and the column flange and the bottom flange of the primary beam are affected by the internal blast (Figure 26 b). The effects of gravity load on the columns and beams are not considered for blast action during the test. Four different scenarios depending on stiffness reduction at the column-base connection for damage prediction are considered subjected to external blast action.

Numerical Result for Damage Prediction and Discussion

In this research, constructive implementation of practical structural identification for full-scale building steel structure is shown. The modal parameters have to be updated in a convenient way by considering the finite element model, geometry and material properties of structural elements. The actual situations of the practical structure can be analyzed by combining numerical analysis and experimental investigation. The real strength of FE model updating of a structure is confirmed. FE model is the best tool to explain the essential physics of the system of interest and it is capable of capturing and simulating its critical physical behaviour.

Different damage predictions were considered for calibrated numerical models and applied to preliminary investigations on a full-scale building structure, using different blast loading conditions. This prediction is important, because it helps get a good idea about the non-visible crack/damage due to loss of stiffness at connections and building elements. Different damage scenarios were considered for external and internal blast action.
Comparisons were made in terms of structural modal parameters for each damage scenario. Due to blast effects at the connections and other elements of the structure, the dynamic properties depend on the percentage reduction of stiffness at connections. From the presented numerical result above, it can be concluded that very few numbers of lower modes are affected. But, for higher modes, significant changes in the frequency exist for all damage scenarios. Damage prediction at base connection when stiffness is reduced simultaneously along the strong and weak axes is more operative for some higher modes, see Figure 26. Most of the higher modes are highly affected by internal blast when reduced stiffness is applied simultaneously at the external and internal column-beam connections and base connections, see Figure 27.

**Pulse Reflex Correlation Analysis**

The correlation between experimental and numerical eigenvalues and mode shapes is examined by pulse reflex correlation analysis. There is an extensive application of pulse reflex modal analysis to evaluate different test and FEM strategies and to find out the shortcomings in modal test and quality of FEM. The mode shape, modal assurance criteria, auto-orthogonality and paired modal data can be extracted after correlation. It can clearly be indicated which mode of FEM is correlated with the measured mode. Experimental modal analysis is conducted for full-scale steel building in the field to find the modal parameters and to demonstrate the flexibility of the frame after damage initiation. In this section, pulse reflex correlation analysis is carried out to see the similarity of the modal results for damaged and undamaged frames. Damage location can be clearly extracted from the mode shapes and eigenvalues.

**Summary**

Structural identification is used to illustrate the damage state through the variation in eigenfrequencies, damping ratios and modal shapes in a structure or element. This research work focuses on the experimental and analytical modal analysis of a full-scale steel frame structure building aiming to develop coherent scenarios that combine the probability of the hazard event with the structural vulnerability in case of a close-in detonation. The field tests were carried out by forced vibration testing under hammer excitation. First series of tests were conducted for the undamaged structure using classical experimental modal analysis. Then, in order to model a structural damage, a secondary beam was dismantled and the measurements were repeated. The change in structural behaviour was observed by identifying the changes in the stiffness and natural frequencies of the structure. The modal parameters measured from field tests were then used to validate finite element models using SAP2000 program. Careful modelling of structural components was used in mitigating uncertainty from the analytical point of view and produced good correlation between the results obtained from St-Id and the experiment. Significant change was found in the frequency for damaged initiations in longitudinal frame (out of plane) for both EMA and numerical results. Experimentally and numerically, the structural properties changed significantly for the 6th mode. The frame is more flexible because of low frequency (334.18Hz and 277.30Hz, respectively for both conditions). Except for the 1st, 4th and 6th modes, all other modes have no big differences of frequency for both conditions in EMA and SAP2000. The 1st, 4th and 6th modes are affected more due to stability, stiffness and modal mass changes.

**CONCLUSIONS**

Within the research framework of FRAMEBLAST project (experimental and numerical validation of blast load models and structural response of a typical frame building system under blast loading), the objective of this study was to investigate the structural identification and damage characterization of a full-scale building structure with structural deterioration and develop coherent scenarios.

The experimental program gives an active area of
research related to the direct applications of modal parameter estimation to identify health monitoring of structures in damaged conditions. Extensive vibration analyses were performed on the undamaged structure and on the damaged structure. The experimental program presents the modal parameter identification and vibration-based damage detection of a full-scale building structure. The results obtained from the experimental method in the experimental modal analysis study are focused on: (a) the effects of degree of uncertainty (noise, vibration, temperature, humidity, etc.) during estimation of the identified modal parameters. A more effective means for mitigating these sources of uncertainty may be to better integrate analytical model simulation results and heuristics with the modal parameter identification process; (b) the process of selecting the DOFs used to derive the model of the testing; (c) the use of the method for the prediction of the response caused by excitation forces applied at different DOFs. The main conclusions from the study can be summarized as follows:

- To reduce data errors, various data pre-processing techniques, like inspection of the signals, time window selection, digital filtering, cross-correlation construction, exponential windowing and data averaging in time or frequency domain, had been developed.

- The measured values from experimental modal analysis have good correlation with the calculated values according to MAC factors derived from mode shapes and measurements. This means that the modal characteristics of the frame considered can also be obtained experimentally from forced vibration tests.

- According to the frequency response synthesis diagram, the modal parameters from experimental tests and calculated values have big influences on the structural identification/dynamic properties' characterization.

- Number of degrees of freedom and structural modeling are very important issues for modal parameter estimation. For the full-scale building, three different models were selected during the experiment. The corner column of the building, as well as the longitudinal frame in-plane and out of plane were examined by experimental modal analysis. The results obtained by modal analysis are correlated, validated and updated by the three-dimensional finite element software SAP2000. The main outcomes from the parametric study are as follows:

  - Experimental modal parameters for the longitudinal frame (in-plane) have a good relation with the finite element analysis results. The first bending mode has the maximum deviation of 4.27% and it is still below 5%.

  - Three-dimensional FEM analysis and experimental modal analysis were also performed for the longitudinal frame out of plane. Careful modeling of structural components in mitigating uncertainty from the analytical point of view produced good correlation between the results obtained from St-Id and the experiment. FEM analysis results are helpful in judging the reliability of a field experiment and integrating analysis and experiment together for understanding how the actual behavior of structural systems is. For the longitudinal frame (in-plane), the errors between EMA and SAP is 4.77% for the fifth bending mode. This indicates that the calibration for the frame was done properly. The other modes for the external frame are also properly calibrated and the gap is below 5%.

  - There are significant changes in the frequency for damaged initiations in longitudinal frame (out of plane). From the experimental results, frequency has the maximum difference for the 1st mode, where the frequency is 16.91Hz and 19.49Hz in undamaged condition and damaged condition, respectively. On the other hand, numerically, this value is 15.37 Hz and 18.79 Hz for both conditions, respectively. But, after secondary beam removal, there is a significant difference in frequency between undamaged and damaged conditions. In undamaged condition, frequency is 226.66Hz, but it is 177.56Hz in
damaged condition in EMA for the 4th mode. Numerically, there are also big changes of frequency in the same mode. Frequency is 222.96 Hz and 193.49 Hz for undamaged and damaged condition, respectively. Experimentally and numerically, the structural properties changed significantly for the 6th mode. The frame is more flexible because of low frequency (334.18 Hz and 277.30 Hz, respectively for both conditions). Except for the 1st, 4th and 6th modes, all other modes have no big differences of frequency for both conditions in EMA and SAP2000. The 1st, 4th and 6th modes are affected more due to stability, stiffness and modal mass changes.

Acknowledgments
This work was partially supported by a grant of the Romanian National Authority for Scientific Research and Innovation, CNCS/CCCDI - UEFISCDI, project number PN-III-P2-2.1-PED-2016-0962, within PNCDI III: “Experimental validation of the response of a full-scale frame building subjected to blast load” - FRAMEBLAST (2017-2018), as well as by the European master program on SUSCOS. This course is funded by the European commission for European Erasmus Mundus Master, 520121-2011-1-CZ-ERA MUNDUS-EMMC project. The focus of master course SUSCOS_M is to provide attendees the engineering ability and know-how to design and construct structures in a balanced approach between economic, environmental and social aspects, enhancing the sustainability and competitiveness of steel industry.

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