EXPERIMENTAL STUDY ON EFFECT OF SEISMIC DAMPER TO REDUCE THE DYNAMIC RESPONSE OF BENCH-SCALE STEEL STRUCTURE MODEL

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ABSTRACT

There are various strengthening methods to protect all structures against dynamic loads. One of them is known to be the use of dampers in moment transmitting frames. For this reason, it is necessary to do more research about such structures and the necessity of better recognition of these structures. In addition to this, the state of the structure should be compared before and after reinforcement and the effect should be seen. This study was also tested with an earthquake simulator at the shaking table using recorded micro-vibration data to obtain the dynamic behavior of the steel structure model. Then, a new building model was introduced by applying seismic shock absorbers to the existing structure. This building model has also been tested with the earthquake simulator at the same shake table. The aim here is to reveal the dynamic behaviors of the last reinforced model and reveal the differences. EFDD, SSI, etc. Various OMA methods are used to measure responses in the environment. In addition, this comparison is based on the OMA method of the reinforced model, which is used to obtain information about the effects of seismic dampers. Micro-tremor ambient vibration data is generated on the ground using the vibration table. EFDD was also used with output modal identification only. At the end of all these studies, modalities, damping rates and a moderate correlation between periods were obtained. The purpose of this study is to reveal the structural and dynamic responses of the steel model structure in terms of effecting the dynamic behavior of the seismic shock absorber application. The average frequency difference between the steel model structure and the reinforced steel model structure is 99.44%. In addition, it can be seen that OMA method can be used in periodic and stiffness determination studies in reinforced structures.

Keyword: - *OMA, EFDD, Seismic damper, Steel structures, Shake table*

1. INTRODUCTION

Generally forced and ambient vibration methods are used in the purpose of vibration testing of structures. Ambient vibration or operational modal analysis is the most appropriate and economic nondestructive technique. Overall properties of the structural response (acceleration, velocity, displacement and frequency) and measuring quantity depends on the nature of vibration (earthquake, traffic, wind, machinery etc.). Ambient vibration measurement of number building has been presented by Ventura and Schuster **Error! Reference source not found.**. There are plenty of studies in the literature regarding modal identification, some of them are mentioned in this paper. The development and improvement of a mathematical model of a physical system and its system identification is studied by Kasimzade **Error! Reference source not found.**. Similar experimental investigations regarding model identification are presented in many studies. Various implementations of system identification in civil engineering structural systems are presented in the following studies [42]. The process of the extraction of physical system parameters from previously identified state-space transformation was studied in the following references [2],[43].

Generally, the modal identification methods are categorized into three types: modal-structural parameter identification, modal-parameter identification and model-control identification.
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 2015 are discussed in Brincker and Ventura (2015) [13]. 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. Kasimzade (2006) System identification is the process of developing or improving a mathematical representation of a physical system using experimental data investigated in HO and Kalman (1966), Kalman (1960), Ibrahim and Miculcik (1977), Ibrahim (1977), Bendat (1998), Ljung (1999), Juang (1994), Van Overschee and De Moor (1996), and system identification applications in civil engineering structures are presented in works Trifunac (1972), Huang and Chen (2017), (Li *et al.* 2016), (Park *et al.* 2016), (Ni *et al.* 2015) (Brincker *et al.* 2000), Roeck (2003), Peeters (2000), (Cunha *et al.* 2005), Wenzel and Pichler (2005), Kasimzade and Tuhta (2007a, b), (2009). Extracting system physical parameters from identified state space representation was investigated in references. Alvin and Park (1994), Balmes (1997), (Juang *et al.*1988), Juang and Pappa (1985), (Lus *et al.* 2003), (Phan *et al.* 2003), Sestieri and Ibrahim (1994), (Tseng *et al.* 1994). The solution of a matrix algebraic Riccati equation and orthogonality projection more intensively and inevitably used in system identification was deeply investigated in works of Aliev (1998). In engineering structures there are three types of identification: modal parameter identification; structural-modal parameter identification; control-model identification methods are used. In the frequency domain the identification is based on the singular value decomposition of the spectral density matrix and it is denoted Frequency Domain Decomposition (FDD) and its further development Enhanced Frequency Domain Decomposition (EFDD). In the time domain there are three different implementations of the Stochastic Subspace Identification (SSI) technique: Unweighted Principal Component (UPC); Principal component (PC); Canonical Variety Analysis (CVA) is used for the modal updating of the structure Friswell and Mottershead (1995), Marwala (2010). It is necessary to estimate sensitivity of reaction of examined system to change of random or fuzzy parameters of a structure. Investigated measurement noise perturbation influences to the identified system modal and physical parameters. Estimated measurement noise border, for which identified system parameters are acceptable for validation of finite element model of examine system. System identification is realized by observer Kalman filter (Juang *et al.* 1993) and Subspace Overschee and De Moor (1996), algorithms. In special case observer gain may be coincide with the Kalman gain. Stochastic state-space model of the structure are simulated by Monte-Carlo method [12], [35], [36], [37].

In this research firstly experimental modal analysis of a steel structure model (Fig -1) was conducted, then the steel structure model was strengthened with gas spring dampers and the same analysis process repeated. Lastly the result of both analyses has compared to each other. In order to conduct the dynamic operational modal analysis (OMA); Quanser Shake Table II seismic simulator were used. The input excitation was applied as ambient vibration based on recorded microtremor data on ground level. The Quanser Shak-Table II is an earthquake simulation device which is an effective tool for structural dynamic, seismic simulation etc. experiments and its widely implemented in various similar experiments.

2. METHODOLOGY FOR ACQUISITION OF MODAL PARAMETER

The Enhanced Frequency Domain Decomposition technique is an extension to Frequency Domain Decomposition (FDD) technique. This t technique is a simple technique that is extremely basic to use. In this technique, modes are easily picked locating the peaks in Singular Value Decomposition (SVD) plots calculated from the spectral density spectra of the responses. FDD technique is based on using a single frequency line from the Fast Fourier Transform analysis (FFT), the accuracy of the estimated natural frequency based on the FFT resolution and no modal damping is calculated. On the other hand, EFDD technique gives an advanced estimation of both the natural frequencies, the mode shapes and includes the damping ratios (Jacobsen *et al.* 2006). In EFDD technique, the single degree of freedom (SDOF) Power Spectral Density (PSD) function, identified about a peak of resonance, is taken back to the time domain using the Inverse Discrete Fourier Transform (IDFT). The natural frequency is acquired by defining the number of zero crossing as a function of time, and the damping by the logarithmic decrement of the correspondent single degree of freedom (SDOF) normalized auto correlation function Peeters (2000).

In this study modal parameter identification was implemented by the Enhanced Frequency Domain Decomposition. The relationship between the input and responses in the EFDD technique can be written as, In this method, unknown input is represented with $x(t)$ and measured output is represented with $y(t)$

$$\left[G_{yy}(j\omega)\right]=[H(j\omega)]^{*}[G_{xx}(j\omega)][H(j\omega)]^{T}$$
(1)

Where $G_{xx}(j\omega)$ is the $r\ x\ r$ Power Spectral Density (PSD) matrix of the input. $G_{yy}(j\omega)$ is the $m\ x\ m$ Power Spectral Density (PSD) matrix of the output, $H(j\omega)$ is the $m\ x\ r$ Frequency Response Function (FRF) matrix, and * and superscript T denote complex conjugate and transpose, respectively. The FRF can be reduced to a pole/residue form as follows:

$$[H(\omega)]=\frac{[Y(\omega)]}{[X(\omega)]}=\sum_{k=1}^{m}\frac{[R_k]}{j\omega\text{-}\lambda_k}+\frac{[R_k]^{*}}{j\omega\text{-}\lambda_k^{*}}$$
(2)

Where n is the number of modes λ_k is the pole and, R_k is the residue. Then Eq. (1) becomes as:

$$G_{yy}(j\omega)=\sum_{k=1}^{n}\sum_{s=1}^{n}\left[\frac{[R_k]}{j\omega\text{-}\lambda_k}+\frac{[R_k]^{*}}{j\omega\text{-}\lambda_k^{*}}\right]$$
$$G_{xx}(j\omega)\left[\frac{[R_s]}{j\omega\text{-}\lambda_s}+\frac{[R_s]^{*}}{j\omega\text{-}\lambda_s^{*}}\right]^{\bar{H}}$$
(3)

Where s the singular values, superscript is H denotes complex conjugate and transpose. Multiplying the two partial fraction factors and making use of the Heaviside partial fraction theorem, after some mathematical manipulations, the output PSD can be reduced to a pole/residue form as fallows;

$$\left[G_{yy}(j\omega)\right]=\sum_{k=1}^{n}\frac{[A_k]}{j\omega\text{-}\lambda_k}+\frac{[A_k]^{*}}{j\omega\text{-}\lambda_k^{*}}+\frac{[B_k]}{\text{-}j\omega\text{-}\lambda_k}+\frac{[B_k]^{*}}{\text{-}j\omega\text{-}\lambda_k^{*}}$$
(4)

Where A_k is the k th residue matrix of the output PSD. In the EFDD identification, the first step is to estimate the PSD matrix. The estimation of the output PSD known at discrete frequencies is then decomposed by taking the SVD (singular value decomposition) of the matrix;

$$G_{yy}(j\omega_i)=U_iS_iU_i^{\bar{H}}$$
(5)

Where the matrix $U_i=[u_{i1},u_{i2},\dots,u_{im}]$ is a unitary matrix holding the singular vectors u_{ij} and s_{ij} is a diagonal matrix holding the scalar singular values. The first singular vector u_{ij} is an estimation of the mode shape. PSD function is identified around the peak by comparing the mode shape estimation u_{ij} with the singular vectors for the frequency lines around the peak. From the piece of the SDOF density function obtained around the peak of the PSD, the natural frequency and the damping can be obtained.

3. CHARACTERISTICS OF STEEL STRUCTURE MODEL

In this study, a two storey steel structure model has been used to conduct the experiment. This structure model is installed on a bench scale shake table (Quanser II) to perform dynamic test, shake table device is controlled using specific software synchronized with installed accelerometers in the stories of the structure. Overall view of shake table and its basic properties are given in Fig -1 and Table -1.

Table -1: The properties of shake table and steel structure model

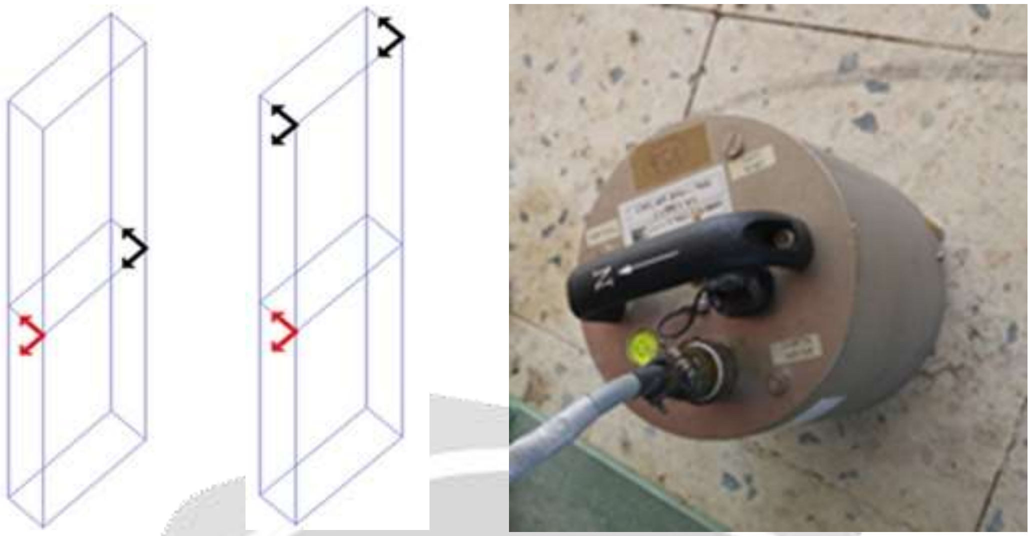
Shake Table Characteristics		Steel Structure Model Specifications	
Dimension (W x L x H)	(0.13 x 0.46 x 0.61) m	Elasticity modulus	2.000E11 N/m^2
Mass	27.2 kg	Poisson ratio (μ)	0.3
Pay-load area (W x L)	(0.46x0.46) m	Mass per unit (ρ)	78500 N/m^3
Maximum pay-load at 2.5 g	7.5kg	Thickness of elements	0.001588 m
Maximum travel	± 0.076m	Story height	0.53m
Operational bandwidth	10 Hz	Length	0.32m
Maximum velocity	0.665 m/s	Width	0.11m
Maximum acceleration	2.5 g	Total height	1.06m
Lead screw pitch	0.127 cm/rev	Number of accelerometers	2
Servomotor power	400W		
Amplifier maximum continuous current	12.5A		
Motor maximum torque	7.82N.m		
Lead screw encoder resolution	8192counts/rev		
Effective stage position resolution	1.55μm/count		
Accelerometer range	± 49 m/s^2		
Accelerometer sensitivity	1.0g/V		

Fig -1: Overview of the shake table and steel structure model

4. ANALYSIS OF NON-RETROFITTED MODEL

In this experiment three accelerometers (able to measure bidirectional vibrations) has been used. Among them, the red one is reference sensor attached in the first floor of the model, remaining accelerometers acts as roving sensors shown in black in the as shown in the Fig -2 (a) and (b). The behavior of the model has been measured as two data sets as illustrated in Fig -2 (a) and (b), these data sets measured within 100 seconds and contains 3 and 5 DOF records respectively.

During analysis, first the synchronized computer to shake table will apply the ambient vibration (Fig -3) as excitation. Then the data files from last setup are saved in the computer. In this experiment MATLAB Wincon toolbox has been used for acquisition of data and the ambient vibration was recorded using Guralp Systems seismometer shown in Fig -2 (c). For estimating the modal parameter based on ambient vibration data the OMA software ARTEMIS software was used [9],[11]. In case if there is unexpected signal drifts or unwanted noise in the display, the stored data must be discarded and the analysis process should be repeated. Before the staring the analysis the cables and all accessories of the derive should be checked carefully. After every analysis, the roving accelerometers are automatically positioned from storey to storey until the analysis is finished.



a) b) c)

Fig -2: Location of accelerometers in steel structure model, (a) first setup and (b) second setup, seismometer device to record the ambient vibration (c).

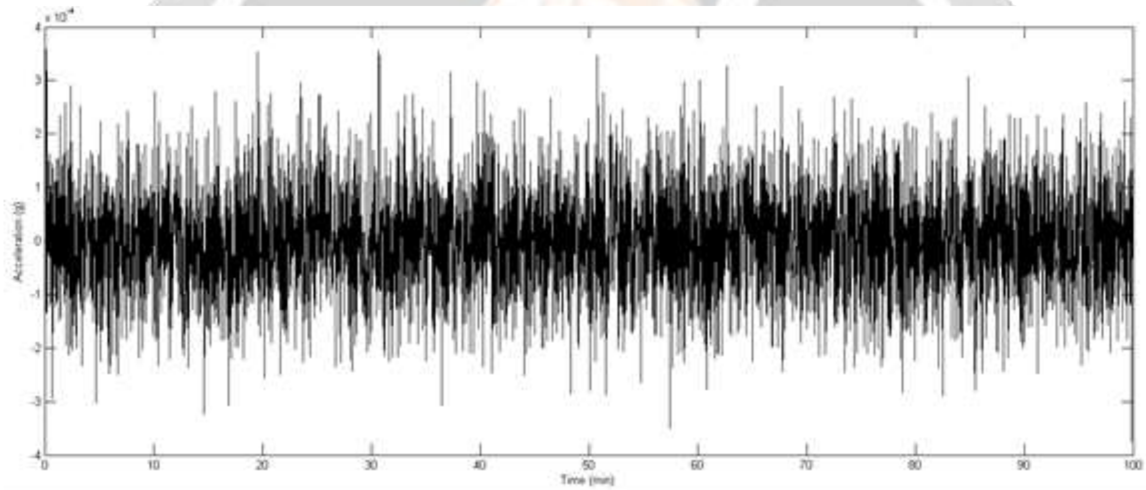


Fig -3: Ambient vibration time-history, applied in ground level of shake table

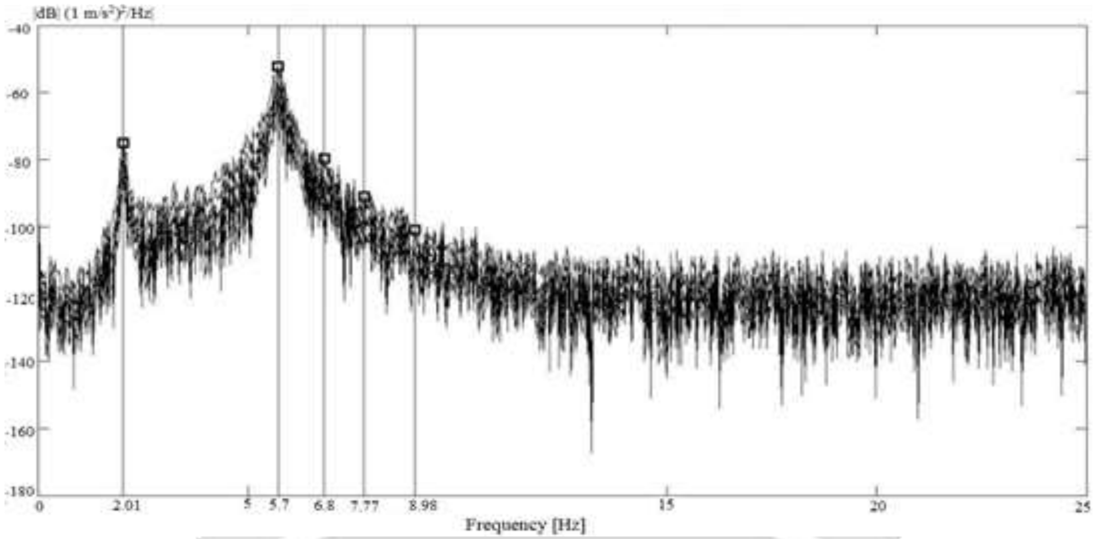
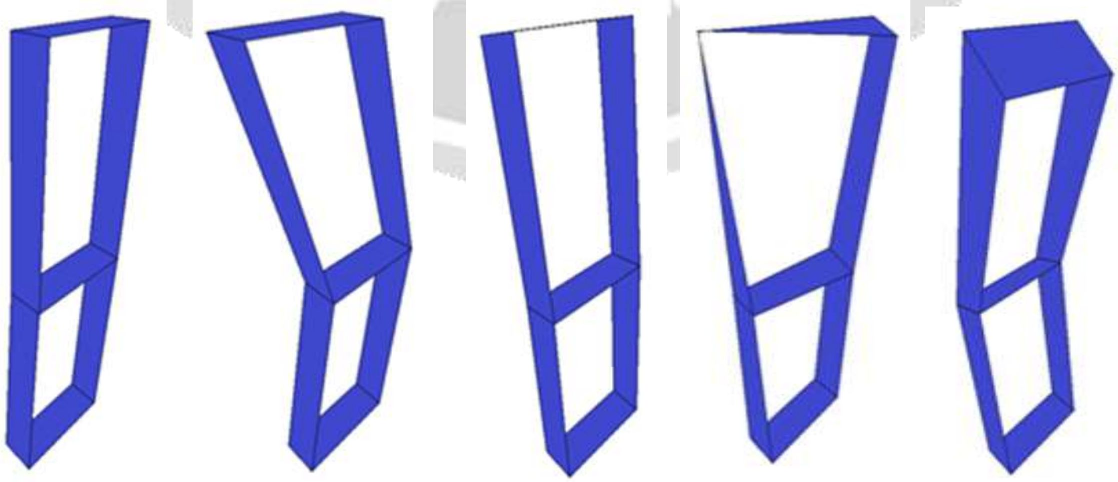


Fig -4: Singular values of the spectral density matrices non-retrofitted model

The Eigen frequencies can be obtained using simple peak picking method (PPM), these frequencies are found as peaks of nonlinear parametric spectrum estimates. In case, there is relatively similar Eigen frequencies, noisy analysis data and low excited modes. Vibration data is the main source to obtain the singular spectral density matrix values, the peak picking technique could be used to obtain these data as shown in the Fig -4. The natural frequencies of all measurements are presented in Table -2. The primary 5 modes acquired as result of experimental analysis are shown in the Fig -5. According results of conducted analyses, when both experimentally measured modal parameters sets are checked it is clearly seen that there is very good agreement between mode shapes of the analyses.

Table -2: Mode shape properties of the non-retrofitted steel structure model according to experimental modal analysis

Mode Shape	1st	2nd	3rd	4th	5th
Frequency [Hz]	2.017	5.725	6.828	7.770	8.987
Damping ratio (%)	0.67	1.82	1.04	0.55	0.67



1st Mode shape 2nd Mode shape 3rd Mode shape 4th Mode shape 5th Mode shape

Fig -5: Five primary mode shapes of the non-retrofitted model identified through experimental analysis

5. ANALYSIS OF THE RETROFITTED MODEL

In the case of retrofitted beams, the following are studies made on it to check and examine the efficiency of using damper (gas spring): the model steel structure is retrofitted with two dampers (diagonal placement). The damper (gas spring) and its components Sam-Gas is product of Sam-Gas Corporation (Fig -6b). The properties of the damper (gas spring) are: Gas Spring K-Factor is the ratio of the compressed force (P2) and extended force (P1) expressed as P2/P1=1.2, rod diameter=10mm and tube diameter=27 mm, extended length min=200 mm and max=900 mm.

a)

b)

Fig -6: a) Damper on steel structure model, b) The Gas Spring damper

The steps to pass through during retrofitting are shown below in details: two dampers (gas spring) is applied (Fig -6a) to the diagonally, after these setups, ambient vibration tests are followed to get the experimental dynamic responses of the system. Similarly, in order to acquire the comparative measurements, the same properties will be used. The SVSDM is presented in Fig -7. and the natural frequencies and the modal damping ratios which identified during analysis are presented in Table -3. Similarly, the 5 primary modes acquired as result of experimental analysis for retrofitted model are shown in

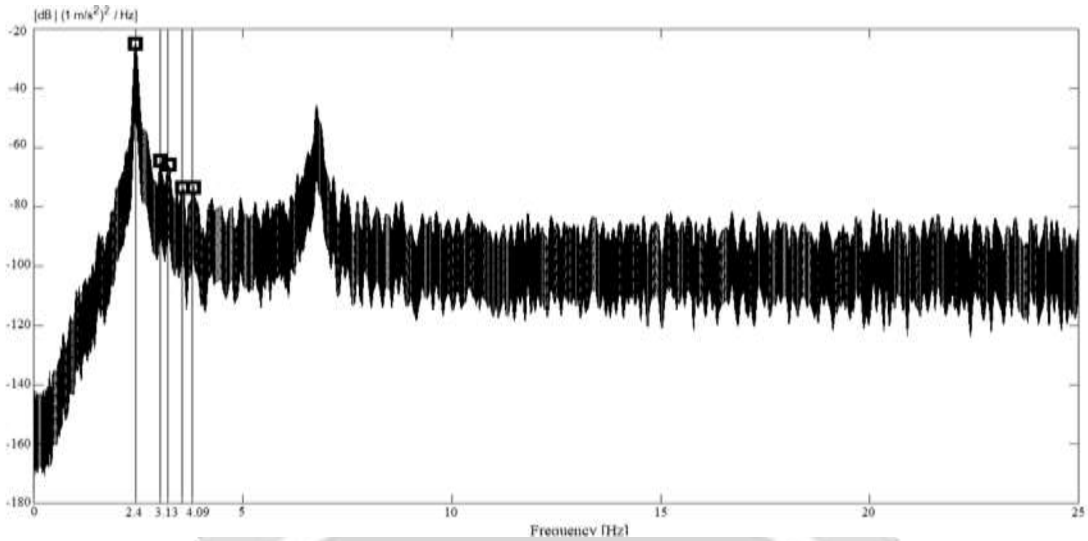
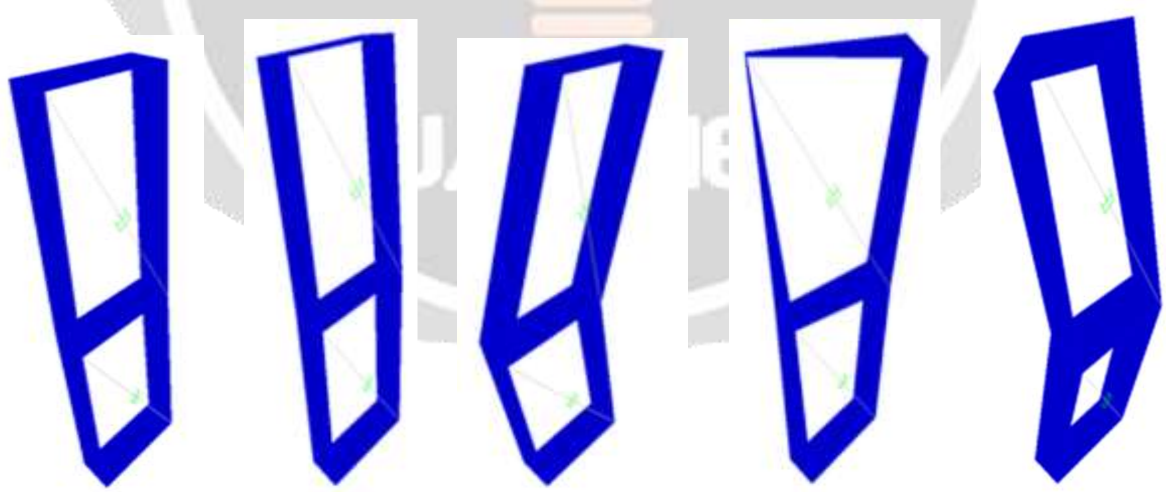


Fig -7: Singular values of the spectral density matrices non-retrofitted model

It is clear that using damper (gas spring) seems to be significantly effective for the purpose of strengthening of the steel frame by increasing its stiffness. this research aims to find out how damper (gas spring) usage affects structural behavior of model steel structure by changing of dynamic characteristics.

Table -3: Mode shape properties of the retrofitted steel structure model according to experimental modal analysis

Mode Shape	1st	2nd	3rd	4th	5th
Frequency [Hz]	2.441	3.027	3.223	3.564	3.81
Damping ratio (%)	15.86	7.12	12.22	5.70	5.20



1st Mode shape 2nd Mode shape 3rd Mode shape 4th Mode shape 5th Mode shape

Fig -8: Five primary mode shapes of the retrofitted model identified through experimental analysis

Table -4: Comparison of existing and retrofitted modal analysis results

Mode Shape	1st	2nd	3rd	4th	5th
Non-Retrofitting Model Frequency [Hz]	2.075	5.890	7.025	7.994	9.246

Retrofitting Model Frequency [Hz]	2.441	3.027	3.223	3.564	3.809
Difference (%)	17.63	94.58	117.96	124.29	142.74
Non-Retrofitting Model Damping ratio (%)	0.67	1.82	1.04	0.55	0.67
Retrofitting Model Damping ratio (%)	15.86	7.12	12.22	5.70	5.20
Difference (%)	15.19	5.30	11.18	5.15	4.53

6. CONCLUSIONS

In this research, the conducted were both modal experimental analysis of non-retrofitted steel structure model and damper (gas spring) retrofitted model steel structure. Based on comparison of the results followings aspects are noticed:

• According to ambient vibration experiment, the natural frequencies ranges between 2 to 10 Hz.
• The modal frequency difference lies in the interval of 17.63%-142.74% for non-retrofitted and retrofitted case and it provides the increase of frame structure stiffness about 99.44%; for the retrofitted model, using damper (gas spring) applied to frame.

• Modal damping ratios increased more than frequencies.

• Obtained results ensure and confirm the efficient usage of microtremor data as ambient vibration input excitation for conducting dynamic analysis using bench scale earthquake simulator (Quanser Shake Table II).

• The conclusion of the experiment strongly suggests that the damper (gas spring) retrofitting should be very efficient to increase stiffness and natural frequencies.

• In this study, it is shown that OMA may be used to evaluate the period and rigidity of the retrofitted structures.

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