Nonlinear Seismic Response Evaluation of Gradually Damaged Steel Shear Frames

Mohammad S. Miah*
Faculty of Civil Engineering, Dresden University of Technology - TU Dresden, 01187 Dresden, Germany
Email: Mohammad_Shamim.Miah@tu-dresden.de

Md J. Miah, Md. M. Hossain, and Md. F. A. Faisal
Department of Civil Engineering, University of Asia Pacific, Dhaka-1215, Bangladesh
Email: jihad.miah@uap-bd.edu; Omega.mamun@gmail.com; fn.faisal.alam@gmail.com

***Abstract*—The objective of this paper is to investigate the dynamical responses of damaged and undamaged steel frames under earthquake loads. To do this end, experimental examination on steel frames have been conducted to investigate seismic response. Most of the high rise buildings are made of steel and prone to extreme dynamic loads such as earthquake, gale, blast and so on. For the experimental investigation, herein, two two-storey steel shear frame type structures have considered executing experimental tests by employing scaled El Centro earthquake data. In order to compare the damaged frame response a reference undamaged frame is considered. The damaged frame was tested by damaging its column step-by-step in terms of reducing columns sizes of 5, 10, 15 percent, respectively. The experimental results have shown that the displacement of damaged structures (i.e., reduced column section) is higher than the undamaged structure. As the percentage of damage increased, the displacement of the structure increased. It is also observed that the frame was vibrating more for 25 sec than 20 sec probably due to the resonance. Additionally, numerical simulations are also conducted by using SAP2000 and the results are compared with experimental data and quite good agreement is observed.**

***Index Terms*—dynamic response, damaged steel structure, experimental, seismic loads, collapse.**

I. INTRODUCTION

A great number of modern mid and high rise buildings are made of steel frame can be found to resist the lateral load. Typically, the steel frames provide large plastic deformation in bending and shear. Due to such advantage, the aforementioned frame is treated as the safest solution withstand earthquake loads. In 1994 Northridge and the 1995 Kobe earthquake, all the confidence about steel frame was significantly shaken in the form of brittle fracture of this frame [1]-[3]. After those incidents, almost all of the seismic design guidelines have been changed except few developing countries like Bangladesh. Furthermore, colossal earthquake of Haiti in 2010 and Nepal in 2015 has reintroduced developing country into a great challenge and laid its brutal sign causing large number of causalities, injuries, social, infrastructural and economical destruction [4]-[8].

In earthquake, numerous aftershocks may trigger by a large main-shock within a short period. Besides this, the structure may damage partially due to main-shock of earthquake and following main-shock, aftershocks have the potential to cause additional cumulative damage to structures leading to total collapse [9]. For example, total number aftershock happened 52 in Haiti till 2012 [6] and 304 in Nepal till 2016 [8], respectively.

Last few decades, the dynamic loads have become a great thread for the Structural Engineers. In reciprocation, many research has done and proceed to measure the structural responses, location of damage, and quantification of damage. In addition, it is most important to identify structural damage or failure modes due to earthquakes. The intensity of ground shaking and the quality of the engineering of structures in the seismic region are vital variables which are affecting earthquake damage. For instant, low rise buildings are vulnerable to high frequency and low frequency worst for high rise building [1], [10]-[13].

The scheme which is a process of implementing in order to detect damage in the structure is known as Structural Health Monitoring (SHM). The SHM is the system which monitors the structural gradual and sudden damage or progressive collapse due to any extreme loads such as earthquake [11], [14]. Seismic load develops extreme stresses and undesirable displacement which may cause serious damages to the structures [15]. Typically, the damage of Civil Structures causes due to mismanagement in construction, lack of quality control, temperature variation, initiating of cracks due to cyclic loadings, etc. [16]. The damages may change the geometric properties, boundary conditions and the characteristics of the systems which may lead to the collapse of the structures [17]. In order to identify or prevent progressive collapsed, damaged structures were studied experimentally and numerically as well as the analytical model has been developed [18]-[20]. As the poor reliability achieved in prediction and quantification of damage, indeed, it is necessary to predict structural response more accurately.

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In this study, the response of two-DOFs of steel frame under constant floor mass by varying the stiffness of column and employing seismic load is investigated. Changing of cross-section of the column is taken as gradually change of stiffness which is as damaged of the frame. The sequence of damage of columns is also described. After damage by analyzing seismic load by shake table, it is observed that how much the response happened for that damage with refer to the intake structure. Through the height of column has great impact concerning the response evaluation as the approximate fundamental natural period of vibration is a function of the height of a structure [21]. In this study, a structure is scaled down to one of fourth in terms of the original dimension of the structure. Indeed, the response of the structure due to wind load increases with the height of the structure, the effect of wind load is not considered in this study due to low storey frame (i.e. two-storey) structure. The main scope of this study is to investigate the dynamic response of a structure in its damaged condition relating to its intake condition.

II. Problem Statement

A scaled 3D 2-storey, which is dynamically known as 2-DOFs, the steel frame is employed. In order to analyze the dynamic response of a structure, it is necessary to take into account a dynamical form [22]-[23]. In this context, the aforementioned 2-DOFs can be defined by a dominant equation of motion as follows:

$$\underbrace{\begin{bmatrix} m_1 & 0 \\ 0 & m_2 \end{bmatrix}}_{M}\underbrace{\begin{bmatrix} \ddot{x}_1 \\ \ddot{x}_2 \end{bmatrix}}_{\ddot{X}} + \underbrace{\begin{bmatrix} c_1 + c_2 & -c_2 \\ -c_2 & c_2 \end{bmatrix}}_{C}\underbrace{\begin{bmatrix} \dot{x}_1 \\ \dot{x}_2 \end{bmatrix}}_{\dot{X}} + \underbrace{\begin{bmatrix} k_1 + k_2 & -k_2 \\ -k_2 & k_2 \end{bmatrix}}_{K}\underbrace{\begin{bmatrix} x_1 \\ x_2 \end{bmatrix}}_{X} = -M\ddot{Z}_g \quad (1)$$

In which, $\boldsymbol{M}$, $\boldsymbol{C}$, **and** $\boldsymbol{K}$ denote 2×2 Mass, damping, and stiffness matrices of the structure, respectively. The input dynamic excitation signal has been indicated by $\ddot{\boldsymbol{Z}}_{\boldsymbol{g}}$. In addition, $\ddot{\boldsymbol{X}}$, $\dot{\boldsymbol{X}}$, and $\boldsymbol{X}$ are defined as the acceleration, velocity, and displacement.

III. Frame Information

The frame used in this study was to evaluate the dynamic response of damage and the intact steel frame which has two major characteristics features. It behaves as shear frame and the slab (see Fig. 1) could move at any height of the column. The aforementioned frame has two stories named as the bottom story and top story. Here in the tabular form of dimension and weight of rigid slabs. Column name denoted as TC1 for Top column number 1, TC2 for Top column number 2 as TC3 and TC4 denoted. BC for Bottom column number 1, BC2 for Bottom column number 2 as similar BC3 and BC4, see Fig. 1.

TABLE I. Frame Dimensions

Floor	Frame Size (cm^2)	Column Height (cm)	Column Size (mm^2)				Slab Size (mm)	Slab Thickness (mm)	Slab Weight (kg)
			TC1	TC2	TC3	TC4			
			BC1	BC2	BC3	BC4			
Bottom	45.72 × 45.72	76.20	5.37 × 20.05	5.40 × 20.28	5.60 × 19.50	5.42 × 19.72	50.80 × 47.63	5.99	11.56
Top									11.53

Note: TC = Top Column, BC = Bottom Column

The Fig. 1 represents the image of the steel frame structure which has been used for the dynamic test.

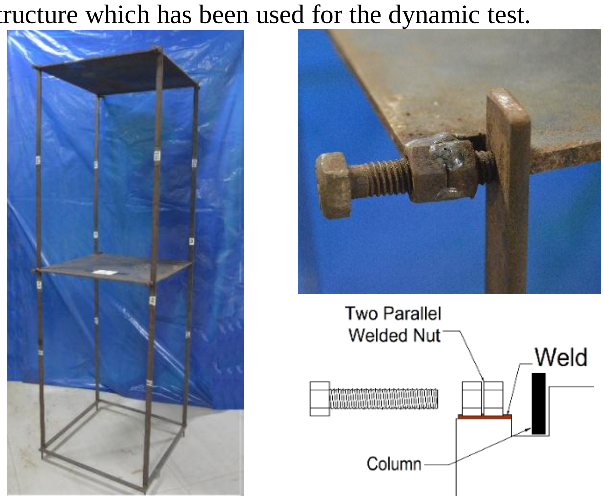

Figure 1. Image of the experimental steel frame.

IV. Experimental Work

The experimental work was done in the Structural Mechanics and Strength of Materials Laboratory of University of Asia Pacific (UAP). The damage process of columns could be done in many sequences. Like damage in all columns in the same percentage at a time. But we have done at a broad sequence that is almost cover all sequences. As per damage percentages we have examined in fifty different cases to observe the response of frame for scaled El Centro ground motion.

A. Cases of Work

Firstly, two experiments for 25 sec and 20 sec executed in undamaged condition. Later, 5% damage on BC1 implemented and two experiments for 25 sec and 20 sec has performed, respectively. As a consequence, five percent damage on BC2, BC3, and BC4 have conducted as well as the experiment are performed for scaled El Centro of 20 sec and 25 sec, accordingly. In a similar way, damaged has accomplished for 15% along with input excitations.

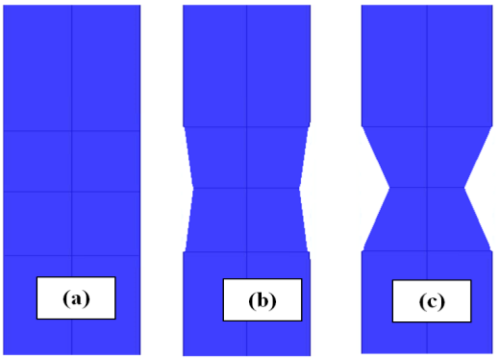

Figure 2. Sequences of damage: (a) reference structure (0%), (b) 5%, and (c) 15% damaged, respectively.

After 15% damage at the four bottom columns, damaging of the structure is continued for top column (TC1, TC2, TC3, and TC4) and experiment has performed with aforementioned seismic excitations. Additionally, numerical simulations are also conducted by using SAP2000 v19 and the results are compared with experimental data. The summary of all cases is described not only in tabular form but also in the diagram (see Fig. 2, Table II and III).

TABLE II. SEQUENCES OF DAMAGES

Experiment Serial no.	Damage Percentage (%)							
	Bottom Floor Column (BC)				Top Floor Column (TC)			
	BC1	BC2	BC3	BC4	TC1	TC2	TC3	TC4
1	0	0	0	0	0	0	0	0
2	5	0	0	0	0	0	0	0
3	5	5	0	0	0	0	0	0
4	5	5	5	0	0	0	0	0
5	5	5	5	5	0	0	0	0
6	15	5	5	5	0	0	0	0
7	15	15	5	5	0	0	0	0
8	15	15	15	5	0	0	0	0
9	15	15	15	15	0	0	0	0
10	15	15	15	15	5	0	0	0
11	15	15	15	15	5	5	0	0
12	15	15	15	15	5	5	5	0
13	15	15	15	15	5	5	5	5
14	15	15	15	15	15	5	5	5
15	15	15	15	15	15	15	5	5
16	15	15	15	15	15	15	15	5
17	15	15	15	15	15	15	15	15

TABLE III. SUMMARY OF CASES

Sequence of damage	Locality of damage	Percentage of damage	Duration
1	REF	Nil	20 and 25 seconds
2	BC1	5	
3	BC2	5	
4	BC3	5	
5	BC4	5	
6	BC1	15	
7	BC2	15	
8	BC3	15	
9	BC4	15	
10	TC1	5	
11	TC2	5	
12	TC3	5	
13	TC4	5	
14	TC1	15	
15	TC2	15	
16	TC3	15	
17	TC4	15	

V. RESULTS

The post-processing of the test results is done by the help of AutoCAD 2012, Midas Gen 2015, MS Excel 2016, and MATLAB R2017a. Including two cases of intake frame, a total of 17 cases tests have been performed by employing scale El Centro of 20 sec and 25 sec, respectively. In order to show the identical response of numerical and experimental responses, bottom three columns and top two column with 5% damage for 20 sec and 25 sec are described in Figs. 3 and 4, respectively. In Figs. 3 and 4, green and red solid lines represent the experimental (Exp.) observations and numerical (Num.) responses individually. It can be seen that the predicted numerical results are in good agreement with the experimental results. Almost similar response was found for 15% damage, which is not shown here in this paper.

Compression of experimental responses of 5% and 15% damage of all bottom columns for 20 sec are depicted in Figs. 5 and 6, respectively. Almost similar response has been observed for 25 tests. In the aforementioned figures, there are 4 solid lines (e.g. amber, red, blue, and ash) indicate the result for 5% damage of the first bottom column (BC1), first and second bottom columns (BC2), first, second and third bottom column (BC3) and all four bottom columns (BC4), respectively. While black line represents the reference column (intake column, Ref).

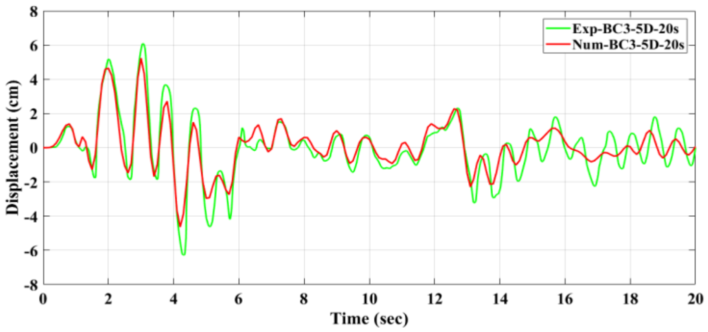

Figure 3. Comparison experimental observation versus numerical simulations for 5% damage (5D) of bottom three columns for 20 sec.

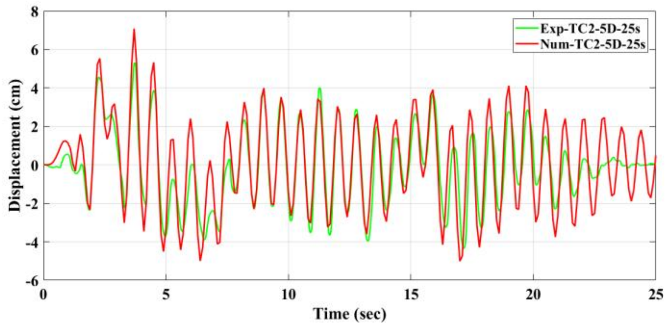

Figure 4. Comparison experimental observation versus numerical simulations for 5% damage of top two columns for 25 sec.

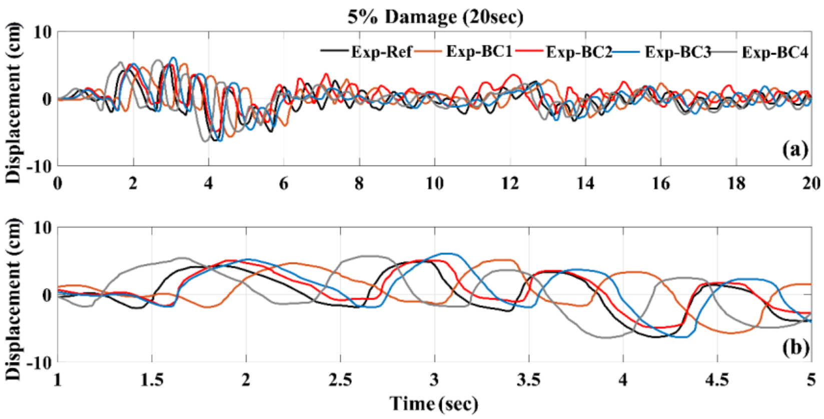

Figure 5. Comparison experimental observation between intake and 5% damage for all the cases of bottom columns: (a) full time history (20 sec) and (b) a zoom view of 1-5 sec.

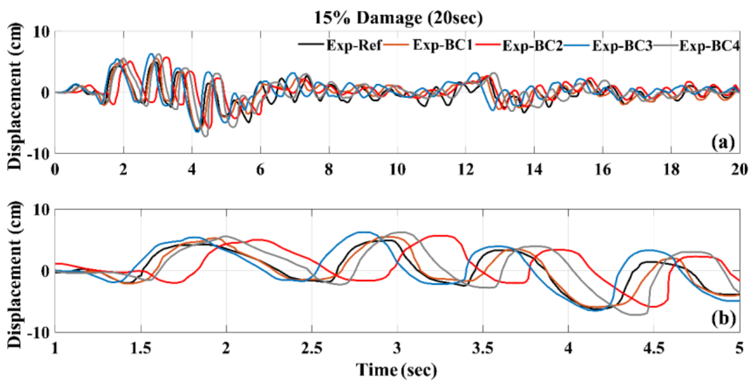

Figure 6. Comparison experimental observation between intake and 15% damage for all the cases of bottom columns: (a) full time history (25 sec) and (b) a zoom view of 1-5 sec.

Compression of experimental responses for the 5% and 15% damage of all top columns for 20 sec are depicted in Figs. 7 and 8, respectively. The color code remains as same as Figs. 5 and 6. From Figs. 5-6, it is visible that responses are increasing with gradual increase in damage. For the simplicity, the color coding kept same as before. In order to better analysis of the results, the maximum displacements of all the cases at the bottom and top columns for 20 sec and 25 sec are plotted in Fig. 9. In Fig. 9, for the 20 sec, it can be seen that the displacement is increased with the increasing damage number of the column for 5%, a similar trend was found for the 15% damage, which is in good agreement from one test to another. As expected, higher displacement is observed for the 15% than 5% damage. However, it can be seen that even at 5% column damage seems frame is more excited than some of 15% cases it is due to the resonance effect. As concerned the 25 sec tests, the displacement is a bit higher than the 20 sec, especially for the reference tests (intake columns), which is an inconsistency with the theory. This behavior could be due to the slenderness of the steel columns. Since the columns are slender, hence the share frame is more vibrated even if the intensity is low (i.e. 25 sec), which could explain the higher displacement of this behavior.

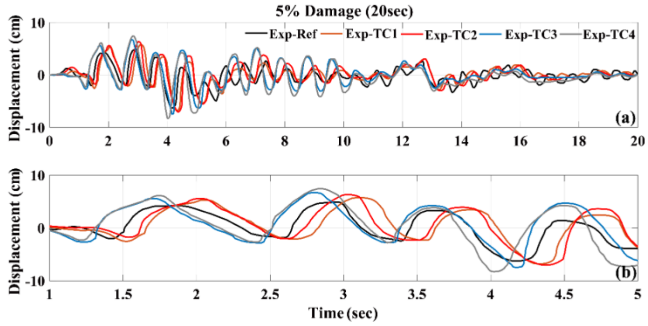

Figure 7. Comparison experimental observation between intake and 5% damage for all the cases of top columns: (a) full time history (20 sec) and (b) a zoom view of 1-5 sec.

Note: Ref = Intake column, TC1 = 5% damage of the first top column, TC2 = 5% damage of the first and second top column, TC3 = 5% damage of the first, second, and third top column, TC4 = 5% damage of all four top column.

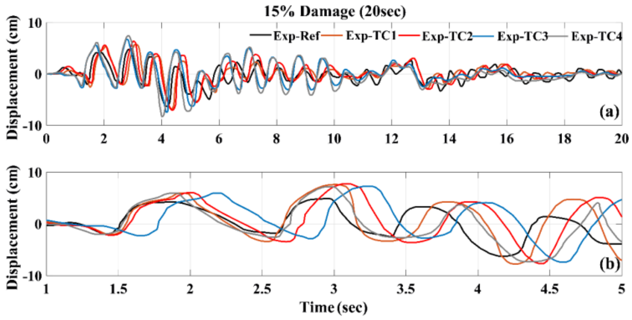

Figure 8. Comparison experimental observation between intake and 15% damage for all the cases of top columns: (a) full time history (20 sec), and (b) a zoom view of 1-5 sec.

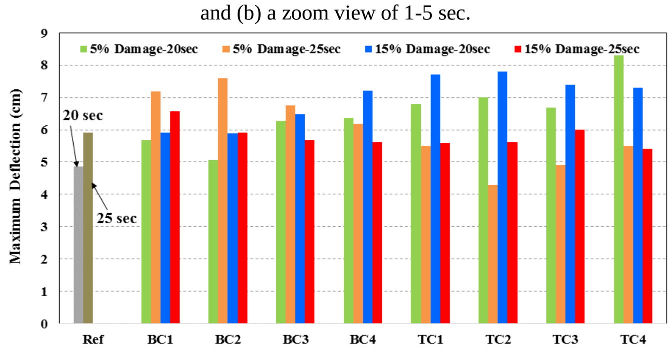

Figure 9. Comparison of maximum displacement of intake/reference frame versus all damage cases for 20 and 25 sec.

VI. Conclusion

Dynamic responses of damaged and undamaged steel frames are studied both experimentally and numerically. Same earthquake data for different periods are employed such as 20 and 25 secs. Despite some discrepancies, all of the cases of the experimental results have shown quite good agreement with the numerically obtained response. Herein it is found that if the structures have lower stiffness (meaning somewhat the size of the column is reduced) the structure will have more displacement compared to the undamaged structures. As the percentage of damage (i.e., reduced column section) increased, the displacement of the structure increased. In this study, the stiffness variation or process of damaged are done by reducing the column cross section. And it is observed that in several cases the frames are facing resonant effect meaning having quite large deformation. Additionally, it is noticeable that frame was struggling more for the longer duration e.g. 25 sec than smaller duration e.g. 20 sec.

Finally, it can be stated that the less the column stiffness higher the displacement. During the variation of excitations time, displacement increased for longer excitation. Therefore, this study revealed that damaged structure showed quite irregular responses. As a result, the response of damaged structures needs to be considered and extensive research should be carried out. Last but not least, the gradually damaging scenario will assist to select the appropriate retrofit scheme of the damaged structures.

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M. Shamim Miah received his Doctor of Science degree (Dr. sc. ETH Zürich) from the Swiss Federal Institute of Technology Zurich (ETH Zürich), Switzerland, 2015, the Masters in Engineering, from the Kunsan National University, South Korea, 2011, and B.Sc. in Civil & Environmental Engineering, from the University of Asia Pacific, Bangladesh, 2007. He is currently at the Faculty of Civil Engineering, Institute for Structural Analysis, at Dresden University of Technology - TU Dresden, Dresden, Germany. Prior to joining to TU Dresden he worked as an Assistant Professor in the Department of Civil Engineering at University of Asia Pacific, Dhaka, Bangladesh. His research focuses are structural dynamics, real-time vibration mitigation and control, smart materials and structures and structural health monitoring.

Md J. Miah is an assistant professor in the Department of Civil Engineering at the University of Asia Pacific, Dhaka, Bangladesh. He received his Ph.D. from the University of Pau and Pays de l'Adour, France, 2017 and his Masters in Civil Engineering for Risk Mitigation from Politecnico di Milano, Italy, 2013. Dr. Miah is an active member of RILEM TC 256-SPF: “Spalling of concrete due to fire: testing and modeling”. His research interests include the concrete structural response in fire, incorporating of mineral admixture in cement, fair-faced concrete, and high performance concrete. Recently, he started working on the vibration mitigation of RC/steel structures via various controlling techniques such as Translational Tuned Mass Damper, Tuned Liquid Damper, and Pendulum Type Dampers.

Md. M. Hossain was born in Dhaka, Bangladesh, in 1994. He received the B.Sc. degree in Civil Engineering from the University of Asia Pacific, Bangladesh, in 2017.

In August 2017, he joined the Department of Civil engineering, University of University of Asia Pacific, as a Research Assistant. His current research interests include Dynamic response of steel structure, structural vibration mitigation using passive control approaches, mechanical evaluation of low strength and high performance concrete.

Md. F A Faisal was born in Comilla, Bangladesh, in 1993. He completed his B.Sc degree in Civil Engineering from University of Asia Pacific, Dhaka, Bangladesh, in Spring 2017.

Now, he working for Buildtech Construction Solutions from December 2016 in Comilla, Bangladesh as a Structural draftsman and designer. Besides of this contractual job, he involved several research projects. Dynamic behavior of steel frame, Performance of high and low strength concrete and Tuned Mass Liquid Damper (TLD) are most interesting research projects to him.