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Seismic strengthening of rigid steel frame with CFRP

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ABSTRACT

Steel frames are a very popular choice in building construction and are used extensively in high seismic risk regions across the world. These existing and future constructed steel frames may need to undergo seismic strengthening to mitigate the high collapse risk during possible earthquakes in the future. In this study a finite element (FE) model was developed, analysed and the results compared with the present self-performed experimental study using shake table tests of steel frames strengthened with externally bonded carbon fibre reinforced polymers (CFRP) composites to validate the modelling techniques. The validated modelling technique are then used for a comprehensive parametric study on the effects of frequency of excitation, maximum acceleration, modulus of CFRP, thickness of CFRP, number of CFRP layers and adhesive type on the seismic response of the frame structure. The results indicate that externally bonded CFRP strengthening is very effective for seismic strengthening of steel frames. The

CFRP strengthening technique reduced the lateral deflection by improving the stiffness and energy absorption capacity of the steel frames.

KEYWORDS

CFRP, Steel Frame, Seismic strengthening, Shake table test, Finite element analysis.

1. Introduction

An earthquake is one of the worst natural disaster which can cause extensive destruction and loss of human lives and property. Each year more than 3000 detectable earthquakes have occurred and these could result in an average of 10,000 deaths [1]. In the 20th century there has been 1.87 million deaths worldwide due to earthquakes thus far. In addition, due to earthquakes an average of 2,052 fatalities occurred per event between 1990 and 2010 in the world [2].

Steel rigid and semi-rigid frames are very popular in regions of seismic activities [3][4]. However, recently there has been concerns on the fracture and brittle failure of welded beam-column joints in major earthquakes [5][6][7]. After the Northridge earthquake in 1994, it was evident that the most common type of damage occurred at welded beam–column connections. From the surveyed buildings, around 70% of the floors had at least one welded joint that was seriously damaged, whereas only 25% of the connections were found with no damage. More than 40% connections of 20% of the building frames had been damaged and in some instances all connections of one or more floors were damaged [8]. Tsai and Wu [9] investigated the steel welded beam-column joint failure modes for seismic upgrading. They found three main failure modes namely, flange-Heat Affected Zone, fracture of flange-weld and flange buckling with percentages of 43%, 27% and 16% respectively [9].

Steel moment connections require high strength and ductility to resist shaking during strong earthquakes. Brittle fracture is the main failure mode of steel moment connections and has occurred due to heavy stress concentration in welded joints [5][10]. It is common practice to weld existing members by steel cover plates for rehabilitation and seismic strengthening of steel structures [11][12]. But welding by steel cover plates is difficult to apply and can be susceptible to corrosion and fatigue damages [13][14]. Carbon fibre reinforced polymer (CFRP) composites are very effective to overcome these disadvantages and have many other advantages (e.g. high tensile strength and strength-weight ratio [15], resistance to corrosion [16] etc.). The CFRP strengthening technique is very effective in delaying local buckling [17][18] and improving the energy absorption capability of the steel members [19]. In addition CFRP strengthening technique help to improving the strength and stiffness of steel members [20] as well as steel structures [21]. Fatigue strengthening of steel connections is one of the most important aspects of CFRPs [22][23][24]. With this in mind, the aim of the present study is the seismic upgrading of existing and future constructed rigid joint as well as steel frame by CFRP wrapping to minimize the high seismic risk.

Seismic retrofitting through CFRP wrapping or by other means (such as increased reinforcement at beam –column connections) is normally carried out based on two generic theoretical considerations which are briefly discussed below:

i) Resonance: The fundamental natural frequency of the structure is moved to be outside the range of the dominant frequencies of common earthquakes, usually in the range: 0.5 Hz – 2.5 Hz (a few may be outside this range). The fundamental natural frequency of framed building structures depends greatly on its height and framed buildings in the range of 8 storeys to 25 storeys (in Australia) are typically vulnerable. The first theoretical consideration of CFRP wrapping will therefore be to shift the fundamental natural frequency to be outside the range of the dominant frequencies of earthquakes. The fundamental natural frequency as well as the other natural frequencies of the bare and wrapped structure are determined through the equation :

$$\{[K] - \omega^2 [M]\} \{\phi\} = 0 \quad (1)$$

In the above equation, [K] and [M] are the structure stiffness and mass matrices respectively, while ϕ and ω are the mode shape vector and the circular natural frequency (natural frequency $f = \omega/2\pi$) respectively.

ii) Plastic hinge within strong column-weak beam concept: The second theoretical basis for CFRP wrapping is to ensure that any potential failure through the formation of plastic hinge does not occur at beam-column junctions, (especially not on the column). The CFRP wrapping therefore must enable the potential plastic hinge formation, if any, to occur on the beam, but at a location away from beam column junctions (strong column-weak beam concept). Seismic response of structures (bare and CFRP wrapped) are govern by the equation:

$$[M]\{\ddot{U}\} + [K]\{U\} + [C]\{\dot{U}\} = -[M]\{\ddot{U}_g\} \quad (2)$$

In the above equation, [M], [K] and [C] are the mass, stiffness and damping matrices respectively, while $\{U\}$, $\{\dot{U}\}$ and $\{\ddot{U}\}$ are the displacement, velocity and acceleration respectively and $\{\ddot{U}_g\}$ is the ground (seismic) excitation.

Due to CFRP wrapping the stiffness matrix [K] is enhanced and will in general result in reduced displacements from equation (2) and higher natural frequencies from equation (1).

This study focuses on numerical study of the CFRP strengthened steel frames subjected to seismic actions and the modelling techniques have been validated using self-performed experimental results as well those in the literature [25]. In this paper, the simpler form of harmonic excitation is used to represent seismic action [25]. Numerical analyses have been carried out in Strand7. A comprehensive parametric study has been performed to evaluate the effect of peak ground acceleration (PGA) and frequency of excitation, thickness and properties CFRP as well as CFRP reinforcing layer and the properties of adhesive on the seismic response of CFRP strengthened rigid steel frame under seismic action.

2. Experimental Program for the validation of FE model

2.1 Materials

Hot-rolled structural steel of grade 300PLUS used for specimens was supplied by OneSteel Limited, Brisbane, Australia. The flat bar used as column and the steel plate used as slab were manufactured as per AS/NZS 3679.1:2010 [26] and AS/NZS 3678:2011 [27] respectively. The mechanical properties of the steel were obtained from the manufacturer as listed in Table 1.

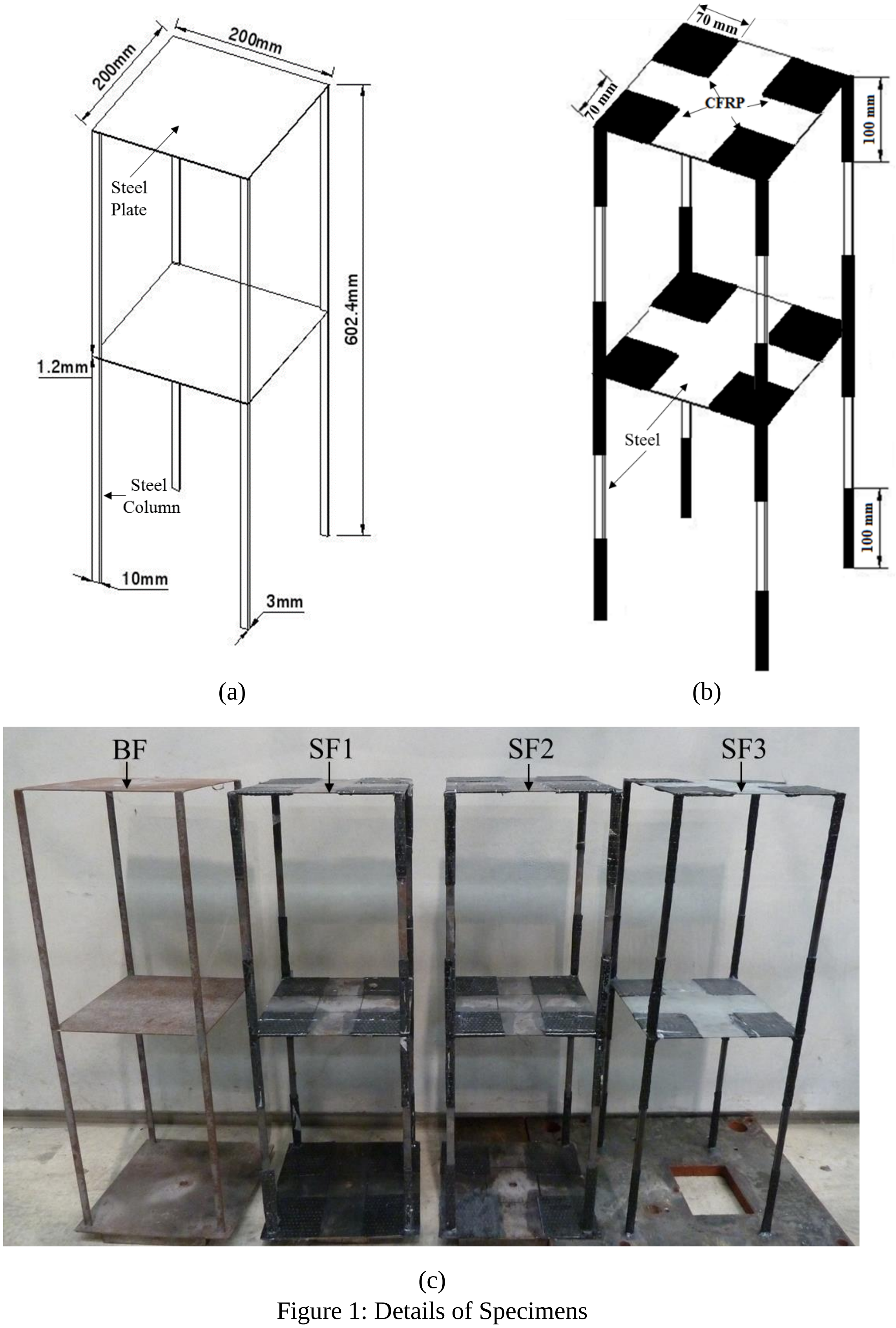
Normal modulus CFRP composites sheets were used in this study for seismic strengthening using epoxy adhesive. Unidirectional CF130 was chosen as carbon fibre sheet. The nominal thickness and fibre weight of CF130 are 0.176 mm 300 g/m^2 respectively. MBrace saturant was used as adhesive which has two part epoxy resin. Both CFRP and adhesive were supplied by BASF construction chemicals, Brisbane, Australia. The CFRP and adhesive mechanical properties are obtained from the previous study done by one of the Author through coupon test [28] and shown in Table 1.

Table 1: Material properties

Property	Steel	CFRP	Adhesive
Density (kg/m^3)	7850	1700	-
Elastic modulus (GPa)	200	125	2.028
Tensile strength (MPa)	-	3800	25
Poisson’s ratio	0.25	0.28	0.32

2.2 Test Specimens and Retrofitting Schemes

Four steel frames, constructed in Design and Manufacturing Centre of Queensland University of Technology (QUT), were tested during this experiment. The models were of a small-scale by considering the limitations of the shake table and the Laser Displacement Sensor (LDS) for measuring deflection (must be less than 30 mm) and the models should fit onto the shake table. The first frame (denoted as BF) remained bare (unstrengthened) as the control specimen. The second frame (denoted as SF1) had one layer of CFRP applied to the critical regions of steel frame, the third frame (denoted as SF2) had two layers of CFRP (the first layer applied to the steel members and the second layer applied directly over the top of the first layer), and the fourth frame (denoted as SF3) had column and plate of specimen were wrapped by one layer and two layers of CFRP respectively. All specimens were two story frames with one bay and have same spans of 200 mm in both direction. Both bottom and top storeys were designed to have the same height of 300 mm. Hot-rolled structural steel column of cross section 10 mm×3 mm and steel plate of 1.2 mm thickness were used for specimen preparation. The specimens were manufactured as per AS/NZS 5131:2016 [29]. CFRPs were applied to the critical joint regions of steel frame across 100 mm from the joint, both above and below the column and across an area of 70x70mm from the joint on the top and bottom of the plate. Figure 1(a), shows the dimensions of the bare specimen, Figure 1(b), shows the retrofitting scheme and Figure 1(c) is a photo of the four specimens that were tested.



(c)

Figure 1: Details of Specimens

2.3 Specimen Preparation and Strengthening Process

Surface preparation is the first stage of the adhesively bonded FRP strengthening process. Appropriate surface-preparation techniques should be applied to ensure excellent bonding between the steel substrate and FRP sheet. Early studies have shown that grit blasting or sandblasting methods are highly effective for removing impurities from the steel surface and to obtain a uniform surface [30]. Sandblasting method was hence used in this study to prepare the outer surface of the steel frame specimens. Sandblasting was done also in the Design and Manufacturing Centre of Queensland University of Technology (QUT) using a granite abrasive system and the average grit size was 0.425 mm [31]. The dust particles and week layers of sandblasted specimens were removed by cleaning with acetone. A similar type of surface-preparation method was used for steel members in recent studies to achieve good bonding between the CFRP and steel surface [32] [33]. The cleaned sandblasted specimens were treated with MBrace 3500 primer before applying epoxy adhesive. After mixing properly the two-part primer of MBrace 3500 was applied on the surface of the specimens by using a brush. Then they were allowed to cure for 1 h before applying the two-part epoxy adhesive. The epoxy adhesive was mixed around 5 min to get homogeneous mixer of Part A and Part B. The mix was then applied on top of primer-coated steel surface. The CFRP sheet was cut into required sizes based on the fiber orientation and wrapped on the adhesive applied surfaces. The wrapped CFRP sheets were rib rolled using an appropriate rib roller in the direction of CFRP fibers to remove entrapped air bubbles and obtain a uniform epoxy/CFRP laminate thickness. Rib rolling was performed until the CFRP fabrics were completely saturated to ensure bleeding of adhesive through the laminates. This process helped to form a composite epoxy/CFRP plate after curing the specimens. The wrapping process was carried out within the pot life of the adhesive, so that workability of the epoxy resin could be used effectively before becoming hardened. In case of multilayer strengthening, the second layer was wrapped consecutively following the same method as the first layer. The multilayer wrapping process was completed on the wet surface of first wrapped layer; thus, after curing, they act as a single composite plate of epoxy/CFRP laminate. Immediately after finishing the strengthening work, the specimens were wrapped with masking tape to prevent premature debonding and achieve uniform thickness of epoxy/CFRP laminate through the length of wrapping. This technique of application of masking tape was found highly effective for strengthening steel members in recent studies [34]. After 24 hours of curing the masking tape was removed from the specimens and they were cured again for at least 2 weeks before the testing [35].

2.4 Test Setup and Instrumentations

The experiments were performed by using a uniaxial shaker table in the Banyo Pilot Plant Precinct of QUT, Australia. The shaker table is of size 1.5 m × 1.5 m with a limit of 1000 Kg test specimen weight. The maximum acceleration and displacements capacities are 1.0g and ±75 mm respectively in the horizontal direction.

The time histories of the structural responses were observed during the shaker table tests. Three LDSs were positioned to measure the displacements of the base plate, the first level and the second level of the structure. LDSs are able to measure up to 0.001 mm displacements accurately for a maximum frequency of 200 Hz. A rigid frame fixed to the ground was used to attach these LDSs so that displacements relative to the ground could be measured. An accelerometer sensor was installed on the base plate of the shaker table in the vibrating direction to verify the accuracy of the input acceleration and a good agreement was observed. The layout of the LDS, accelerometer sensor and experimental setup are shown in Figure 2. All test data was recorded simultaneously by using a data acquisition system.

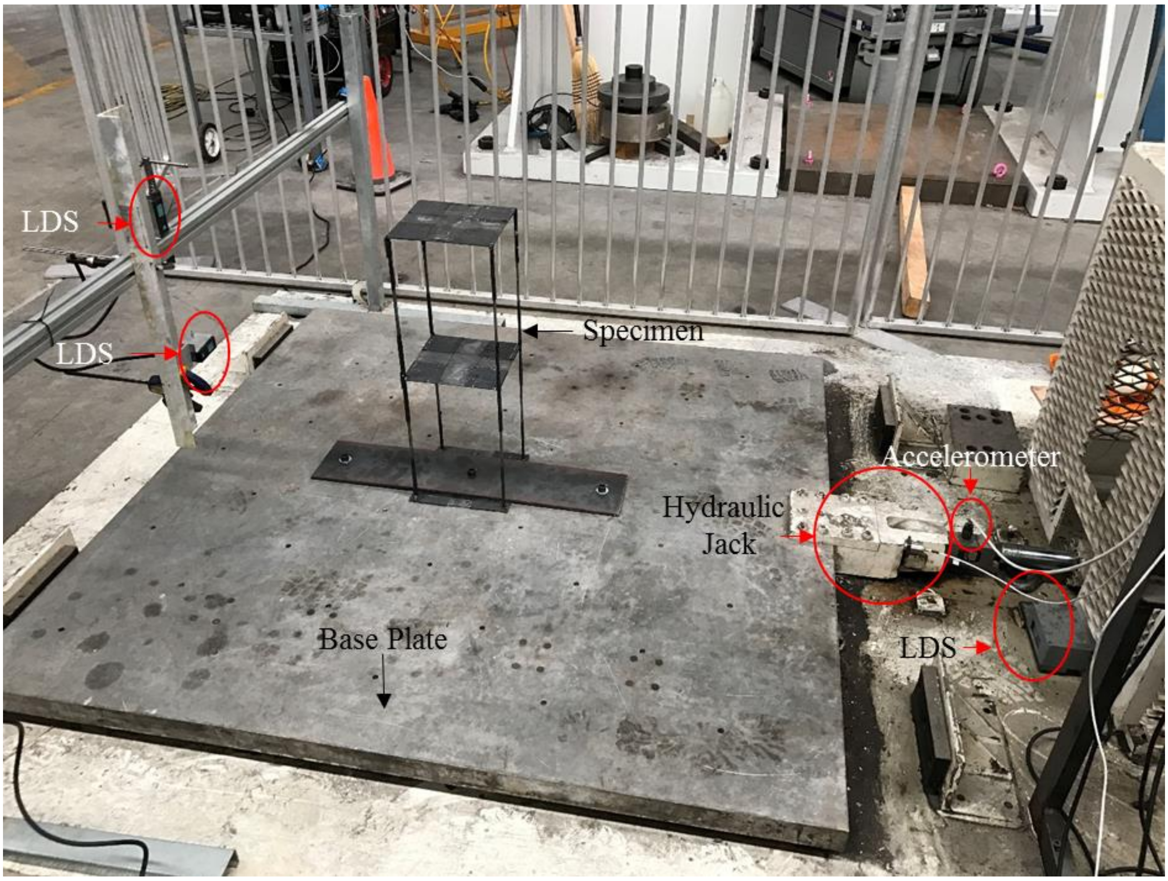


Figure 2: Experimental set-up

Considering the natural frequency of steel frame model and the displacement limits of the shake table, the seismic action was initially established as an ideal sinusoidal wave with a 5 Hz frequency and 10 mm amplitude, as derived below [36]:

$$\text{Acceleration} = \text{A} \times \left(\frac{2\pi}{T}\right)^2 \times \sin\left(\frac{2\pi t}{T}\right)$$

Where; Amplitude, A = 10 mm, Time step = 0.01s, Period, T = $\frac{1}{\text{Frequency,f}}$ = 0.2

A high strength steel plate, shown in figure, was placed over the base plate of the frame, and bolted to the shake table, simulating the structure being fixed to the ground. Once the frame was attached to the shake table and the lasers were calibrated, the hydraulic jack proceeded to apply horizontal acceleration to the table for 10 seconds along the weaker axis of the frame. This process was then repeated for the remaining frames. The frames were not tested for failure, but to investigate the ability of CFRP strengthening to minimise deflections/lateral sway.

3. FE modelling

Strand7 of version R2.4.4 finite element (FE) computing package [37] was used for seismic simulation of the bare and CFRP strengthened steel frames. The design of the frame in Strand7 would eventually become the physical specimen constructed for shake table testing. In Strand7 FE modelling, the mechanical properties of the materials were taken to be the same as the experimental properties (Table 1). Choosing the proper mesh size is important for the FE model. A convergence study was conducted to determine when convergence occurred i.e. at what point the deflection and natural frequency of the structure did not significantly change upon the change in the number of elements. Different mesh sizes were used in different regions of the model to minimise the analysis time. A relatively smaller size mesh was selected for the beam-column joint region where as a larger size mesh was selected for regions away from the beam-column joints. The creation of this model in Strand7 was split into two components; steel structure and CFRP/Adhesive composite design, using the properties listed in Table 1.

3.1 Steel modelling

Modelling the steel structure is the first phase in FE modelling. The steel structure was built using basic nodes and jointed using Hexa8 brick elements. Creating the 8 columns and 2 plate levels results in a 10 total of modelled elements. The modelled elements were discretised into small meshes using the subdividing tool. This was established through convergence testing i.e. at what point the deflection and natural frequency of the structure did not significantly change upon change of the number of brick elements. The rigidity between steel column and plates was ensured by node to node connection in between them. Upon subdivision, the bottom nodes of the structure were fixed in each direction, to replicate the effect of a 4-sided weld during shake table testing.

3.2 Composite of CFRP and Adhesive modelling

After completing the steel structure, the same nodes and brick elements can be copied over into a new file to create a CFRP wrapped model for comparative testing. The method for wrapping a Strand7 FEM involves using a ply and laminate function. The mechanical properties of CFRP and adhesive were exactly replicated by utilizing the ply material function. In that option, to replicate the CFRP weave direction, unidirectional was chosen for CFRP and both directional mechanical properties were given as input. In Strand7, a laminate may consist of up to 300 plies. Each ply can have its fibre directions at different angles and can be made of a different material of a given thickness. This means that the weave direction of CFRP can define exactly as per experiment in Strand7 FE model.

For the FE model, the layers of CFRP wrapping were simplified into a layer of adhesive and the remaining layers were modelled as an equivalent layer. This was to simplify the model and avoid results that indicate failure in the individual CFRP layers. The equivalent layer thickness was equal to the sum of the thicknesses of the layers. The equivalent modulus and Poisson's ratio were averaged values based on the individual layer thicknesses [38]. The equivalent modulus of elasticity and Poisson's ratio of two layers of CFRP wrapping were calculated as 77.4 GPa and 0.296 respectively.

3.3 Modal Analysis

Before attempting an experiments, natural frequency analysis has been conducted in Strand7 for finding modal properties of the all experimental frames. The Modal Properties of the all experimental frames are showing in Table 2.

Table 2: Modal Properties of the all experimental frames

Frame	Mode	Frequency (Hz)	Dominant Eigenform	Frame	Mode	Frequency (Hz)	Dominant Eigenform
BF	1st	6.90	Bending	SF1	1st	7.88	Bending
	2nd	13.64	Bending		2nd	14.30	Bending
	3rd	20.59	Torsional		3rd	21.62	Torsional
	4th	25.45	Bending		4th	28.55	Bending
SF2	1st	8.79	Bending	SF3	1st	8.16	Bending
	2nd	14.93	Bending		2nd	14.22	Bending
	3rd	22.60	Torsional		3rd	21.63	Torsional
	4th	31.41	Bending		4th	27.80	Bending

4. Validation of FE Model

The validation of seismic simulation results of the FE models was carried out using the results of the shake table tests conducted in this study. Non-linear transient dynamic analyses of bare and strengthened frame were conducted considering the non-linearity of materials and geometry. The dynamic response of a structure to an external excitation depends on its stiffness, mass and damping. In this paper the Rayleigh damping model [39] is adopted in the numerical analyses. This model is often used to define the damping effect in the seismic response of structures. In the Rayleigh damping model the damping matrix [C] is assumed to be a linear combination of the stiffness and mass matrices, [K] and [M] respectively, in the form:

$$[C] = a_o\,[M] + a_1\,[K] \qquad (3)$$

In the above equation, a_o and a_1 are the constants selected to achieve the desired damping ratio at two preselected periods/frequencies.

Strand7 uses the relationship of damping ratio to frequencies to define Rayleigh damping. Two frequencies have to be chosen for this purpose and they should be as close as possible to the lower and upper limits of the frequency range of interest, to minimise error. The frequency of the first mode was chosen for the lower range and the frequency of the mode, where the mass and stiffness participation factors are higher than 90%, is chosen as the upper range. For steel framed structures the damping ratio of the first mode is usually under 2% [40]. In this study, 1% has been taken as the damping ratio of first mode. The damping ratio at higher mode is calculating by using the following equation [39]:

$$\zeta n = \zeta_1.[1+0.11(R_f-1) \quad (4)$$

Where, ζ_1 and ζn are the damping ratios at the first mode and n^{th} mode respectively and R_f is the ratio of frequency at the n^{th} mode and first mode. The top lateral displacement-time responses and the maximum lateral displacements at each floor level are compared. The validations of FE results for the top lateral displacements in both the bare and retrofitted frames are carried out by comparing the numerical and experimental displacement-time responses. The top deflection is the single most important parameter is evaluating the seismic response of a frame structure [41].

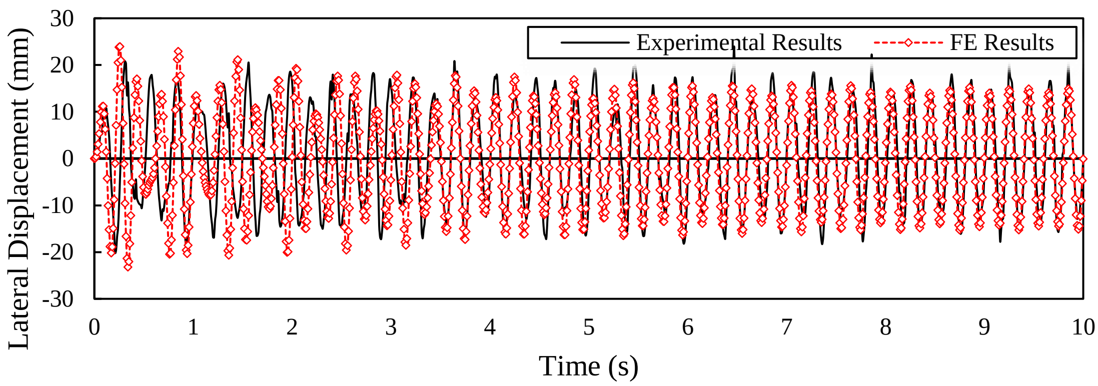


Figure 3: Comparison of top lateral displacements of bare specimens

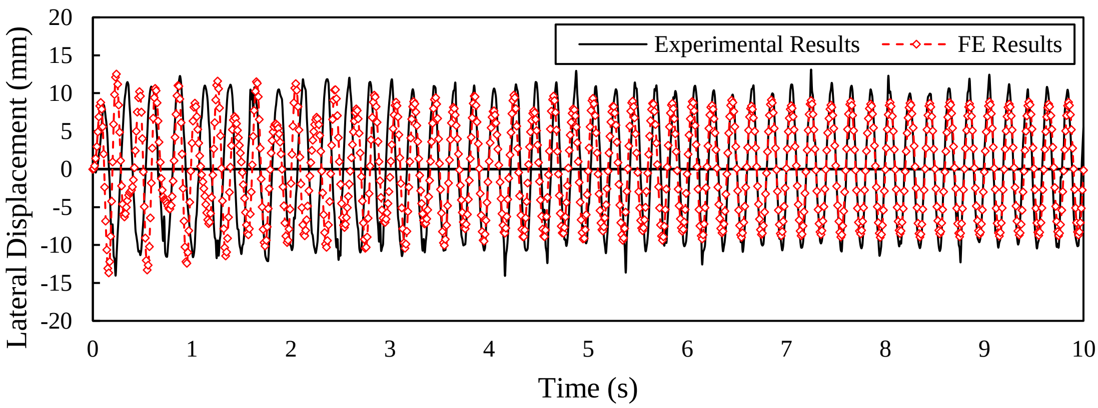


Figure 4: Comparison of top lateral displacements of strengthened frame with 1 layer of CFRP

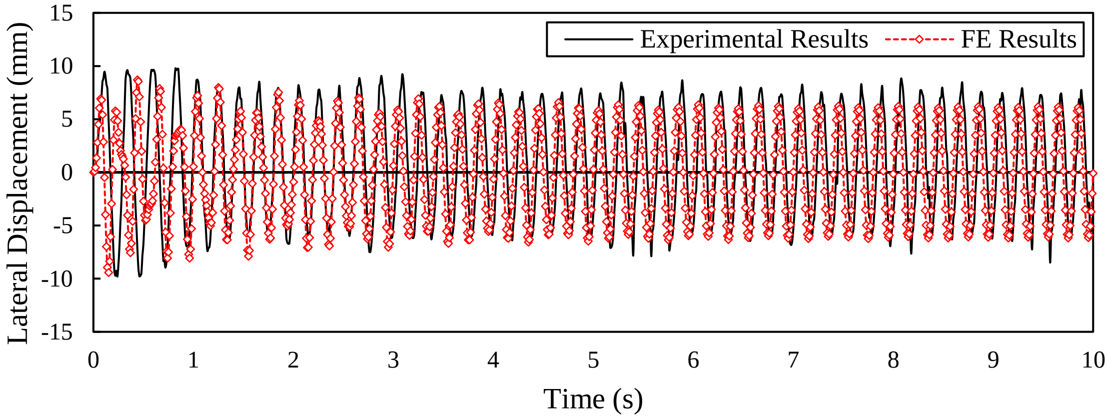


Figure 5: Comparison of top lateral displacements of strengthened frame with 2 layers of CFRP

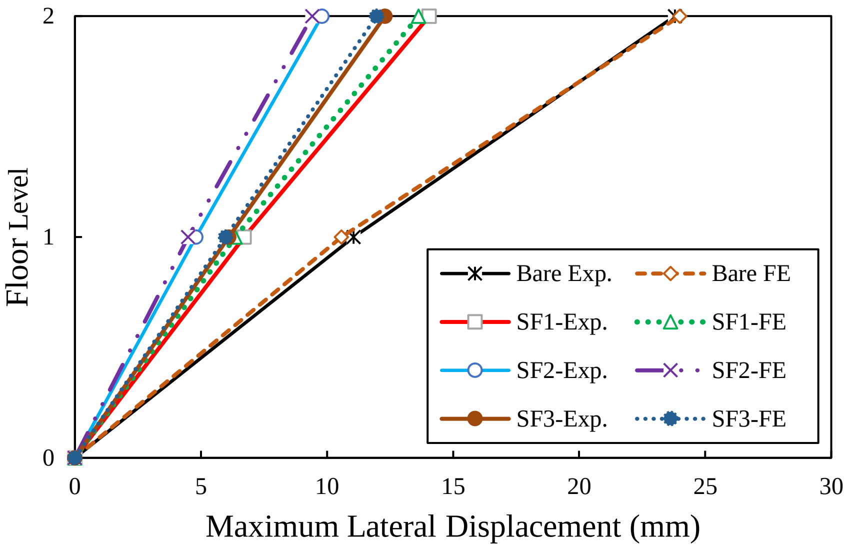
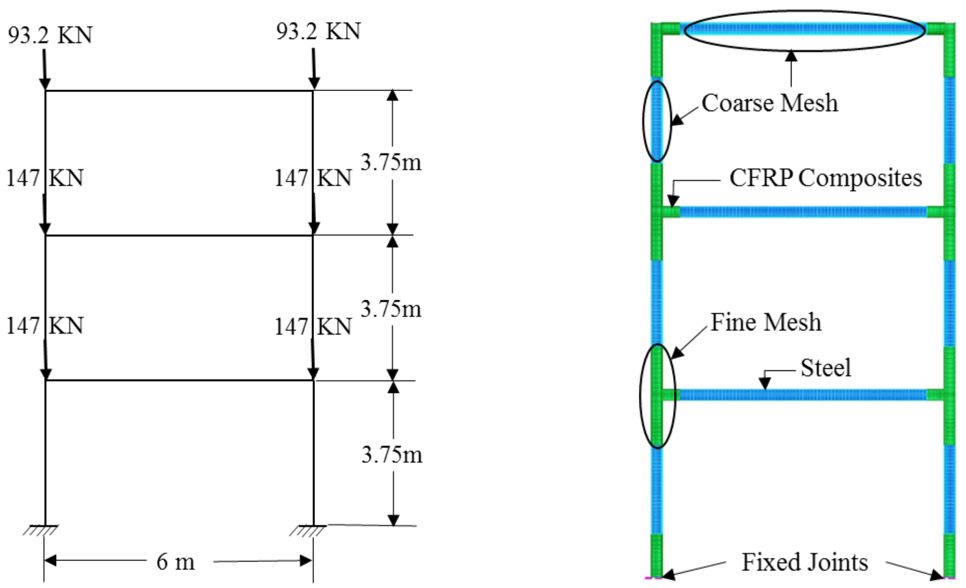


Figure 6: Comparison of maximum lateral displacements at each floor level

A good agreement between the two sets of results can be observed in the top lateral displacement–time responses curves of the bare and strengthened steel frames as presented in Figures 3-5. Figure 6 depicts the comparisons of the experimental and FE simulation results for the maximum lateral displacements of the bare and strengthened frames at each floor level. Here too, a good matching of the two sets of results can be seen. The mean ratio and COV of maximum lateral displacements at each floor are 0.991 and 0.024 respectively and confirm the validation between experimental and FE simulation models. From the Table 2, it is evident that the natural frequencies of the wrapped frames has been increased, which confirms that the wrappings have strengthened the steel frame, increasing its rigidity as well as its stiffness and hence it's natural frequency based on theoretical considerations (i). From the Figures 3-6, it can be seen that the strengthened frames displace far less than the unwrapped frame due to the addition of CFRP wrapping against seismic action. As after the wrapping the frames become stiffer, the tip deflections of the steel frames with one layer and two layers of CFRP have been reduced by 41% and 59% respectively based on theoretical considerations (ii), which proves the effectiveness of using CFRP wrapping in steel structures.

5. Parametric Study

In this study, numerical models of bare and CFRP strengthened full scale single-bay three-storey steel frames are developed by using the validated modelling techniques described in Section 3. Real size HEB220 section was used for the beams and columns in the frame. The lumped vertical gravity forces were been applied at the nodal point loads as shown in Figure 7(a) and calculated as per Vogel calibration frame [42].



(a) (b)

Figure 7: Details of FE modelling (a) Frame Parameters (b) FE Strategies

Non-linear transient dynamic analysis of the frames subjected to sinusoidal ground motion was carried out for investigating the seismic mitigation capacity of CFRP strengthened steel frame structures. Moment carrying capacity of steel beam-column joints are reduced due to local buckling. The extension of buckling zone for forming a plastic hinge in a beam is very important in seismic action [43]. For this reason, the potential plastic hinge regions of the beams are wrapped by CFRP following the previous literature [44]. The plastic hinge length is taken as span length, L/16 [45]. The columns wrapped length was ¼ the column height i.e. up to the point of contra flexure. Brittle fractures and cracking of the column-bottom beam flange weld are two major failure patterns in welded joints [5]. Thus, wrapping the whole joints will be very useful to prevent those failures. Figure 7 shows the details of FE models of bare and strengthened single-bay three-storey steel frames. Furthermore, the present FE modelling technique has been showed a good agreement with seismic response of steel frame found in literature [25]. The effects of CFRP modulus, CFRP thickness, no of layers, adhesive type, vibrating frequency and PGA on the seismic response of the steel frames are evaluated.

5.1 Effect of CFRP modulus

To assess the effect of CFRP modulus, the bare single-bay three-storey steel frame was strengthened with CFRP composites of normal modulus (NM) and high modulus (HM). MBrace CF130 unidirectional CFRP and MBrace CF530 unidirectional CFRP, manufactured by BASF construction chemicals, are chosen as the NM and HM CFRP composites respectively.

Table 3: Material properties of NM and HM CFRP

Material properties	Normal modulus CFRP (CF130)	High modulus CFRP (CF 530)
Density (kg/m3)	1700	2100
Elastic modulus (GPa)	240	640
Tensile strength (MPa)	3800	2650
Thickness (mm)	0.176	0.19

The listed properties of both CFRP composites in Table 3 were provided by manufacturer and obtained from previous studies [46]. The mechanical properties of adhesive layers are taken from Table 1. The same sinusoidal ground motions of 1g×Sin ($2\pi f t$) i.e. PGA value of 1g and vibrating frequency of 2.5 Hz is using for all analysis.

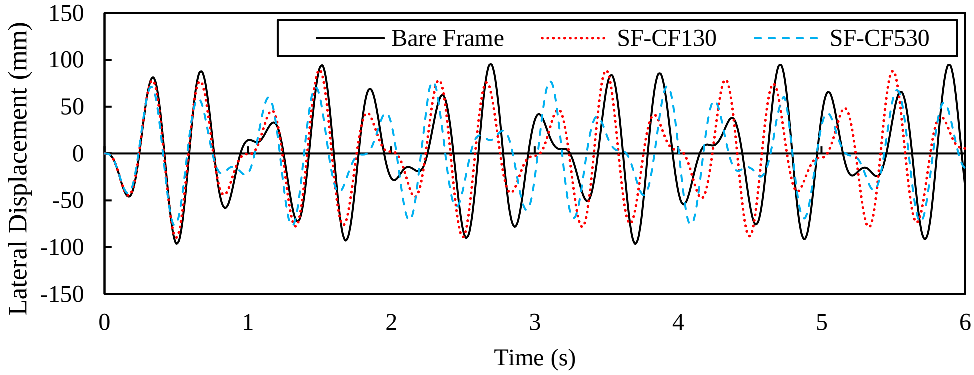


Figure 8: Comparison of top lateral displacements with different types of CFRP

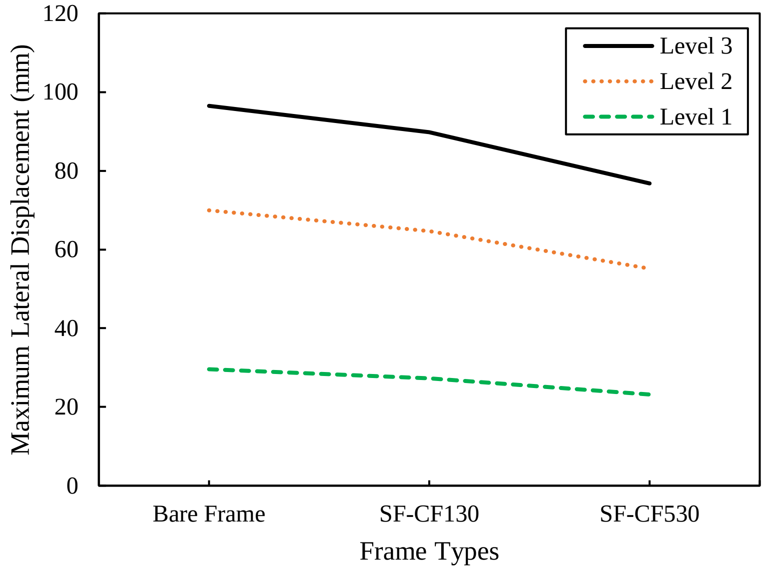


Figure 9: Comparison of maximum lateral displacements at each floor level for different types of CFRP.

The seismic responses obtained from the non-linear transient dynamic analysis of bare and strengthened steel frame with three layers of NM and HM CFRP composites are compared in Figures 8 and 9. The effects of CFRP modulus are noticeable as a significant variation of maximum lateral displacements are observed between strengthened steel frames with NM and HM CFRP composites. Figure 8 depicts the comparison of the time histories of the top lateral deflection plots of bare and strengthened steel frames with NM and HM CFRP composites. The maximum lateral displacements at each floor level of bare and strengthened frames with NM and HM CFRP composites are shown in Figure 9. The reduction of top maximum lateral displacement for the strengthened steel frames with HM CFRP wrapping is 20.42%, which is much higher than the 6.95% reduction obtained with NM CFRP wrapping. Previous studies had concluded that the HM CFRP is superior to NM CFRP in strengthening of steel joints [46] and HM CFRP has approximately 2-3 times the stiffness of NM CFRP [47]. Thus, here the stiffness of the CFRP is the controlling factor and by using HM CFRP wrapping the structure is stiffer and resulted in a much lower deflection.

5.2 Effect of CFRP Thickness

The effect of CFRP thickness was examined in the current study by varying the thickness of CFRP layer. The bare single-bay three-storey steel frame was been strengthened with three layers of normal thickness (NT) and high thickness (HT) CFRP composites. MBrace CF130 unidirectional CFRP of thickness 0.176 mm is chosen as NT CFRP and the mechanical properties are kept the same as in Table 3. On the other hand, QuakeWrap TU27C CFRP of thickness 0.524 mm, manufactured by QuakeWrap Australia, is chosen as HT CFRP. The mechanical properties HT CFRP were provided by manufacture. The density, elastic modulus and tensile strength of QuakeWrap TU27C CFRP are 1800 Kg/m^3, 231 GPa and 3800 MPa respectively i.e. except thickness all of the properties are almost the same. This will enable to evaluate the effect of varying the thickness of CFRP. The mechanical properties of adhesive layers are taken similar to those shown in Table 1. The bare and strengthened steel frames were analysed under the same sinusoidal ground motions of PGA 1g and vibrating frequency 2.5 Hz.

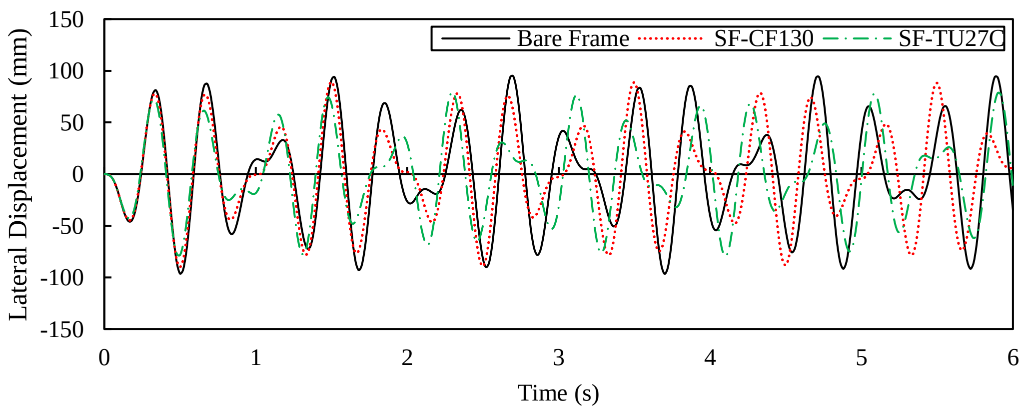


Figure 10: Comparison of top lateral displacements for CFRPs with different thickness

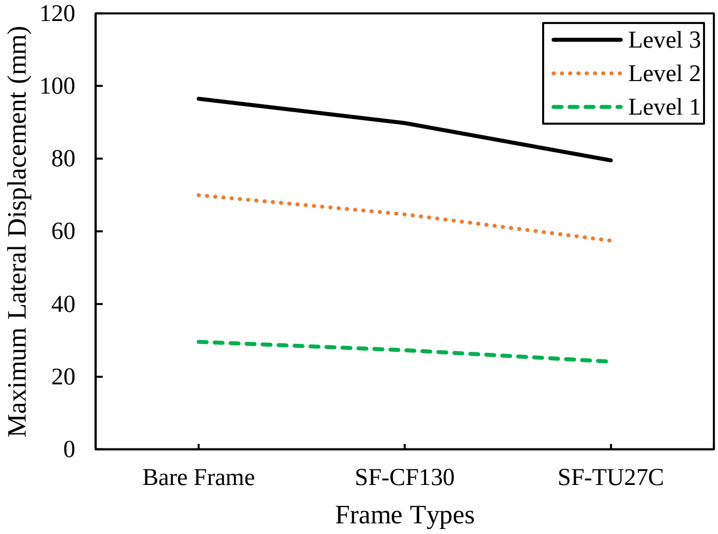


Figure 11: Comparison of maximum lateral displacements at each floor level for different CFRPs with different thickness

The time history of the top lateral displacement is shown in Figure 10. The maximum lateral displacements at each floor level are shown in Figure 11. It is clear from these figures that the thickness of CFRP has a great influence in the reduction of deflection. After strengthening by HT CFRP the top maximum lateral deflection has been reduced by 17.58%, while for NT CFRP it was only 6.95%. Thus for seismic mitigation of steel frames through strengthening the joints, the use of HT CFRP is more effective compared to NT CFRP.

5.3 Effect of Layers of CFRP

The effects CFRP layer number has been evaluated by increasing the CFRP layers from 1 to 5. For the rest of study the steel frames are strengthened by three layers of TU27C CFRP and the properties are kept as described in Section 5.2. The mechanical properties of adhesive layers are as shown in Table 1. The input sinusoidal ground motion is keep similar in the as previous analyses.

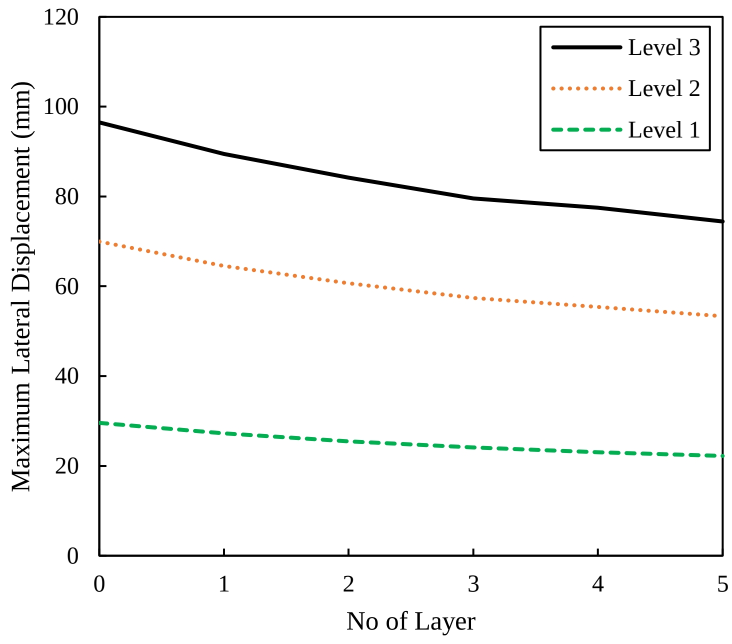


Figure 12: Maximum lateral displacement comparison for each floor level at different no of layer of CFRP

Figure 12 depicts the maximum lateral displacements at each floor level of strengthened steel frames with 1 to 5 layers of CFRP. The influence of number of CFRP layers on the lateral displacement is clear from this figure. Due to increasing the number of CFRP layers from 1 to 3 and to 5, the reductions in the top lateral deflection changed from 7.3% to 17.6% and 22.9% respectively. Thus increased CFRP layers provides energy absorption capability of CFRP strengthened frames and reduce the lateral deflection. Previous studies also concluded that the energy absorption capacity of CFRP strengthened members increased with the increase in the number of CFRP layers [19]. However, after the application of 3 layers of CFRP, there is a decrease in the rate of decrease of the top lateral deflection.

5.4 Effect of Adhesive Properties

MBrace and Sika 30 are two commonly available adhesives and have been chosen to evaluate the effect of adhesive properties on strengthened steel frame under seismic action. The mechanical properties of Sika 30 epoxy adhesives are obtain from previous literature [48].The elastic modulus and tensile strength of Sika 30 epoxy are 11.25 GPa and 22.34 respectively. The mechanical properties of the other adhesive MBrace are kept similar as listed in Table 1. HT CFRP described in Section 5.2 is used with both adhesives to investigate the effect of adhesives properties. The frames are analysed under the sinusoidal ground motion with the PGA and frequency specified as 1g and 2.5 Hz respectively.

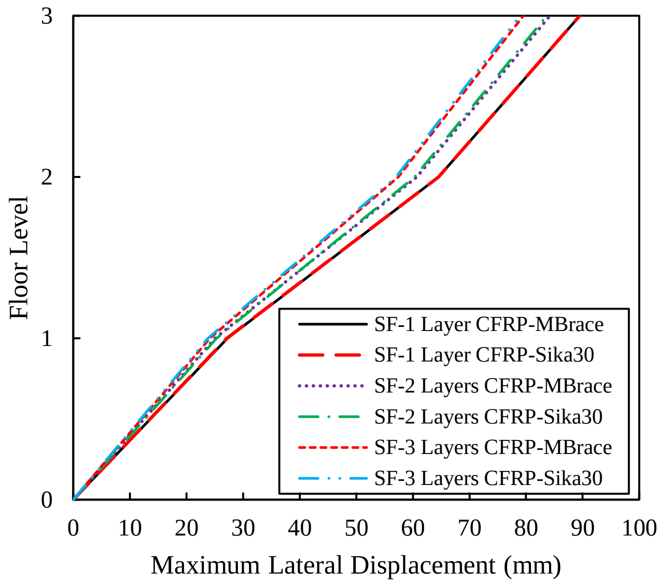


Figure 13: Comparison of maximum lateral displacements at each floor level for different type of Adhesives.

Figure 13 shows the maximum lateral displacements at each floor level obtained with the MBrace and Sika 30 adhesives. The results show that for all steel frame strengthened with one, two and three layers of CFRP with Sika 30 show slightly better performance compared to those with MBrace. Although the stiffness of Sika 30 is much higher than the MBrace, its tensile strength is slightly lower compared to MBrace. Thus, the adhesive properties stiffness has no significant effect on the stiffness of steel frames.

5.5 Effect of Ground Acceleration

The influence of the ground acceleration (maximum value or PGA) on the seismic response of the steel frame strengthened with CFRP composites is shown in Figures 14 and 15. To observe the effect of the peak ground acceleration, the frequency of the ground motion is maintained at 2.5 Hz and the peak acceleration is varied from 0.6g to 0.8g and 1g. HT CFRP described in Section 5.2 is used as CFRP and MBrace saturant listed in Table 1 is used as adhesive.

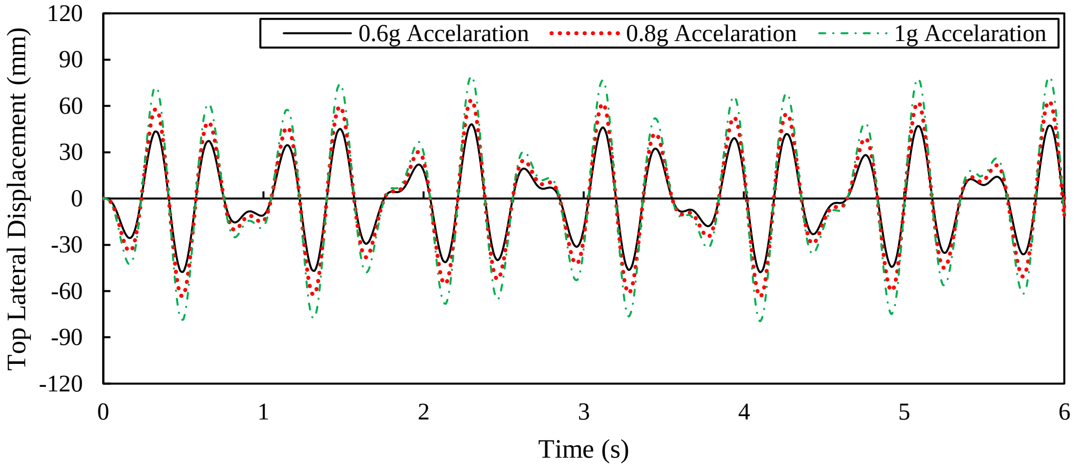


Figure 14: Comparison of top lateral displacements for different peak accelerations

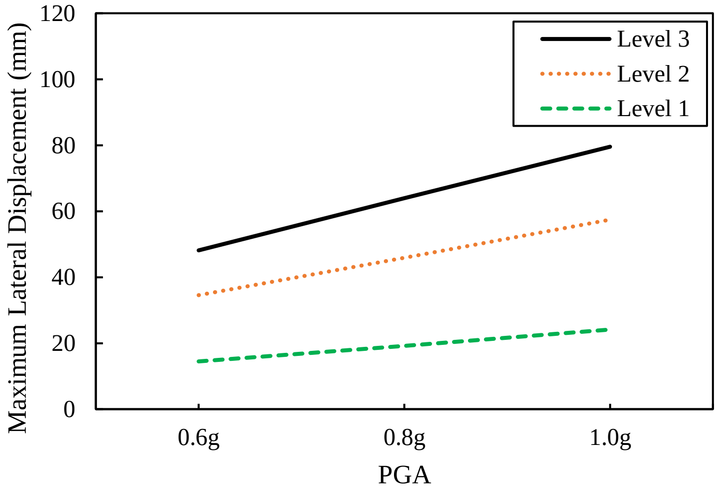


Figure 15: Comparison of maximum lateral displacements at each floor level for different peak accelerations

There is a noticeable effect of the peak acceleration, as expected, on the seismic response of steel frame strengthened with CFRP composites. As the applied seismic force on the structure is increased with the increase of the peak acceleration of the seismic action [49], the lateral displacement responses increase. The maximum top lateral displacement has been increased from 48.1mm to 63.9mm and 79.5mm when the peak acceleration increased from 0.6g to 0.8g and 1g respectively. Previous studies have also shown that the seismic response of a structure evaluated with respect to the lateral displacement increased with the increase in the peak ground acceleration [44].

5.6 Effect of Frequency of ground acceleration

The first natural frequency of the full scale structural model was 0.9 Hz. To evaluate the effect of the frequency of ground acceleration, the vibrating frequency of the sinusoidal ground motion is selected to range from a value close to the first natural frequency of the model and then varied from 1 Hz to 1.75 Hz and 2.5 Hz. This range will include the natural frequencies of some of the higher modes of the structure. These frequencies are also within the range of most seismic records. The input PGA value remained constant as 1g to observe the effects of the frequency change on the seismic response of the strengthened steel frames with CFRP. HT CFRP described in Section 5.2 is used as CFRP and MBrace saturant listed in Table 1 is used as adhesive. Figure 16 illustrates the top lateral displacements and Figure 17 illustrates the maximum lateral displacements at each floor level. From the figures it is clear that the frequency of the ground motion has a great influence, as expected, on the seismic responses of strengthened steel frames with CFRP. The lateral displacements increase with the increase in the input frequency. The maximum top lateral displacements for the input frequency 1 Hz, 1.75 Hz and 2.5 Hz are 33.8mm, 47.5mm, 79.5mm respectively. The lateral deflection of strengthened steel frame was magnified under increasing value of the input harmonic frequency.

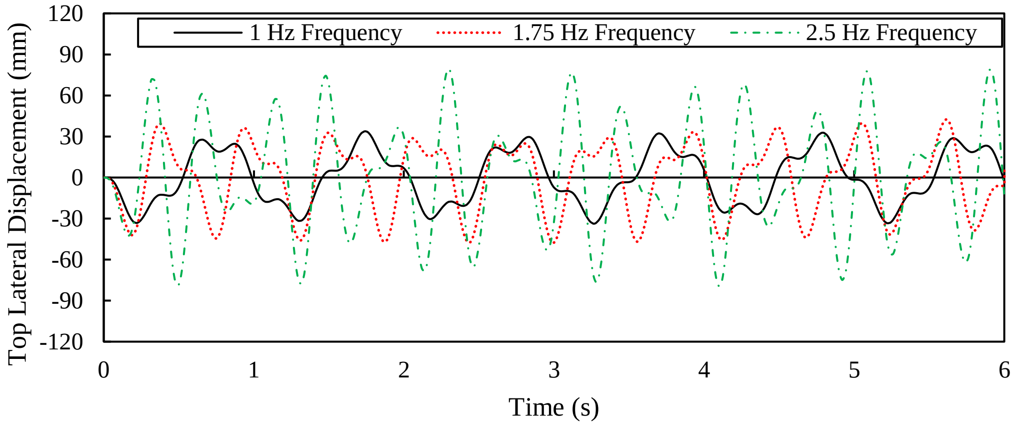


Figure 16: Comparison of top lateral displacements for different input frequencies

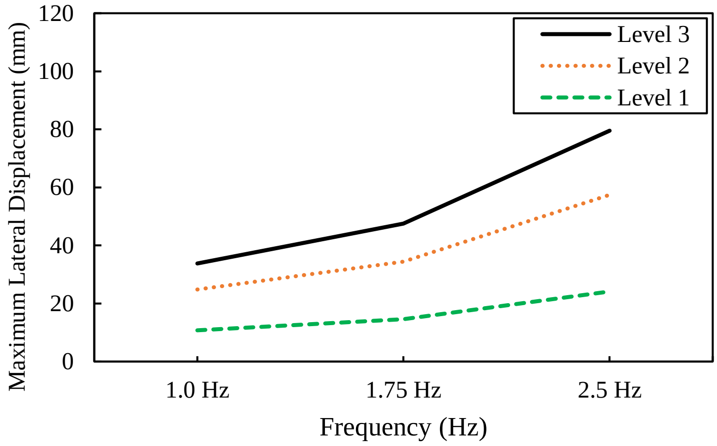


Figure 17: Comparison of maximum lateral displacements at each floor level for different input frequencies

6. Conclusions

The behaviour of CFRP strengthened steel frames subjected to seismic action has been investigated in this paper through experimental testing, FE analysis and a detailed parametric study. Initially validation of the FE model was carried out by comparing results with self-performed experiments. Than detailed numerical simulations were carried out to investigate the effects of important parameters on the seismic response of a full scale bare and strengthened steel frames. Results from the FE simulation conclude that the CFRP is very effective for strengthening the steel frame and improving its seismic mitigation capacity. Based on this study, the following major observations can be drawn:

- Good agreement between experimental and FE simulation results proved the effectiveness of FE modelling for investigating the seismic responses of CFRP strengthened steel structures.
- Stiffness of CFRP has a great influence on seismic responses of steel structures. This shows why HM CFRP is superior to NM CFRP for seismic mitigation of steel structures.
- HT CFRP is more effective than the NM CFRP for seismic strengthening of steel structures.
- The lateral displacements reduced gradually with the increase of the thickness of CFRP composites. Due to this, the energy absorption capacity of CFRP strengthened steel structures increased under seismic action. But after a certain thickness the rate of increase in the reduction gradually decreased.
- The stiffness of adhesive does not have a significant effect and the stiffness of CFRP is the dominating factor for seismic mitigation of steel structure.
- There is a prominent effect of ground acceleration on the lateral displacement responses of strengthened steel structures under seismic action. The increased lateral displacement increased with the peak ground acceleration, as expected. The effective seismic load on the structure increased with the increase in peak acceleration. Hence, CFPR strengthening is not interrupted the natural seismic response of steel frames.
- The input frequency is a key parameter for seismic response of strengthened steel structures under seismic action as the lateral displacements are magnified due to the increase in the input frequency.

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