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Seismic vulnerability assessment of CFRP strengthened RC structures

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Subject review

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Seismic vulnerability assessment of CFRP strengthened RC structures

Seismic vulnerability of a Reinforced Concrete (RC) frame retrofitted with the Carbon Fibre Reinforced Polymer (CFRP) is analysed in this paper. An experimental programme was conducted to determine the way in which concrete strength is affected by CFRP. The improved properties were modelled for different levels of concrete strength using a finite element based software. Using the results obtained during analysis, seismic vulnerability curves were derived for the unstrengthened and strengthened frames. The curves show significant improvement in the performance of unstrengthened RC structure after retrofitting.

Key words:
CFRP, seismic vulnerability assessment, retrofitting, PERFORM 3D, nonlinear analysis

Pregledni rad

Hammad Salahuddin, Shaukat Ali Khan, Muhammad Usman Ali, Arslan Mushtaq

Analiza seizmičke osjetljivosti ab konstrukcija ojačanih CFRP-om

U radu se istražuje seizmička osjetljivost ab okvira ojačanog s CFRP-om (polimer armiran ugljičnim vlaknima). Proveden je program eksperimentalnih istraživanja kako bi se utvrdilo na koji način ojačanje CFRP-om utječe na čvrstoću betona. Poboljšana svojstva modelirana su za razne čvrstoće betona i u tu je svrhu korišten program temeljen na metodi konačnih elemenata. Na temelju rezultata analize izvedene su krivulje seizmičke osjetljivosti neojačanih i ojačanih okvira. Dobivene krivulje pokazuju da su nakon ojačanja ostvarena znatna poboljšanja neojačanih armiranobetonskih konstrukcija.

Ključne riječi:
polimer armiran ugljičnim vlaknima, analiza seizmičke osjetljivosti, poboljšanje, PERFORM 3D, nelinearna analiza

Übersichtsarbeit

Hammad Salahuddin, Shaukat Ali Khan, Muhammad Usman Ali, Arslan Mushtaq

Analyse der seismischen Vulnerabilität durch CFRP nachgerüsteter Stahlbetonkonstruktionen

In dieser Arbeit wird die seismische Vulnerabilität durch kohlenfaserverstärkten Kunststoff (CFRP) nachgerüsteter Stahlbetonrahmen untersucht. Es wurde ein Programm experimenteller Untersuchungen durchgeführt, um den Einfluss kohlenfaserverstärkten Kunstoffs auf die Betonfestigkeit zu erforschen. Die verbesserten Eigenschaften wurden durch verschiedene Werte der Betonfestigkeit mittels eines Finite-Elemente-Programms modelliert. Aufgrund der Resultate wurden Kurven der seismischen Vulnerabilität für Stahlbetonrahmen in anfänglichem und nachgerüstetem Zustand hergeleitet. Diese zeigen, dass durch die Nachrüstung ein deutlich verbesserter Zustand erzielt wird.

Schlüsselwörter:
CFRP, Beurteilung der seismischen Vulnerabilität, Nachrüstung, PERFORM 3D, nichtlineare Analyse

1. Introduction

The seismically deficient and non-engineered (not designed by a qualified Engineer) building stock in many countries including Pakistan has become an issue of great concern, as it was partially or completely damaged by the jolts of earthquake in recent past. In Pakistan, Kashmir earthquake (2005) and Balochistan earthquake (2008, 2013) caused devastating damage to both property and lives and severely affected the socio-economic situation in these areas and in the country as a whole [1].

Almost 10-15 % of the total building stock of Pakistan consists of reinforced concrete buildings [2] and most of them are non-engineered and designed to withstand gravity loads only [3]. Prior to Kashmir earthquake (2005), local designers were unaware of the need for seismic detailing and, hence, many of the buildings were designed without considering seismic loads. Many of such buildings were severely damaged in Kashmir earthquake [3]. Post-earthquake damage analyses exposed many deficiencies in the construction and design that caused the damage. These deficiencies are mostly due to the unawareness of the need to conduct seismic design, and to the lack of skilled manpower. Some of deficiencies of this type are: soft storey mechanism, irregular plans and elevations, poor quality and low strength construction materials, provision of insufficient reinforcement in joints, weak column–strong beam, exposed rebars in structural members, anchorage and development length, insufficient lap splices, deficient or no seismic hooks, inadequate transverse reinforcement, etc. [1, 4]. Some of these deficiencies are shown in Figure 1.

It is therefore essential to strengthen the existing deficient building stock by means of suitable retrofitting techniques, so that earthquake forces can properly be harnessed, and in order to prevent further loss of human lives and infrastructure.

The retrofitting of structural members such as beams, columns, etc. has been conducted as a means to strengthen RC structures. Among all structural members, reinforced concrete (RC) columns are of critical importance for the performance and the safety of structures as they are mostly compression controlled members, and they carry the load of other members. The confinement of concrete is an efficient technique for increasing the load carrying capacity and/or ductility of a column. It is precisely the lateral pressure that induces in concrete a tri-axial state of stress and, consequently, an increment of compressive strength and ultimate axial strain, [5].

Strengthening of RC columns was first conducted by means of steel jackets grouted to the concrete core, but the use of Fibre Reinforced Polymer (FRP) jackets gained more importance from the beginning of the 1990s. The FRP confinement is accomplished by placing the fibres mainly transverse to the longitudinal axis of the column providing passive confinement, which is activated once the concrete core starts dilating as a result of Poisson's effect and internal cracking. The confinement of non-circular columns is widely accepted to be less efficient than the confinement of circular columns, since in the latter case, the wrapping provides circumferentially uniform confining pressure to the radial expansion of the concrete. In non-circular columns, the confinement is concentrated at the corners rather than across the entire perimeter [6].

Extensive work in both the experimental and analytical areas has been conducted on both full scale RC frames and small-scale plain concrete specimens of circular and non-circular cross-sections confined with FRP and subjected to pure axial compressive loading [6-10]. The effect of CFRP strengthening on hollow steel members has also been studied by Kabir et al. [11]. Studies focusing on RC columns of both circular and non-circular cross-sections of considerable size (minimum dimension cross-section of about 300 mm [12]) have also been conducted [12-17]; however, these researches were focused on the effect of confinement on structural behaviour, while insignificant research has been done on Earthquake Risk Assessment (ERA).

ERA is the first step required for earthquake risk mitigation. To carry out ERA, seismic hazard and vulnerability assessment are required. In Pakistan, insignificant work has been done on seismic vulnerability assessment of reinforced concrete (RC) buildings [3]. In recent work on seismic vulnerability assessment in Pakistan as conducted by Ahmed

Figure 1. a) Improper beam column joint; b) Poor quality construction material; c) Exposed rebars near beam column joint

[3] and Muhammad [18], analytical fragility functions and socioeconomic loss functions were derived for a particular segment of building stock in Pakistan.

The purpose of the seismic hazard and vulnerability assessment is to develop damage indicators of buildings for different levels of hazard, and to represent them with vulnerability curves. In different parts of the world the vulnerability of structures may differ considerably, which is due to the difference in available construction materials, different construction methods and practices.

The seismic vulnerability assessment was conducted in this study on CFRP wrapped deficient RC structures. For that purpose, an experimental program was conducted, which involved testing of cylindrical concrete specimens. The specimens were wrapped with CFRP in single and double layers and then tested in compression. The test results were used for modelling and analysis in a finite element based software. Finally, vulnerability curves were generated for the un-strengthened and strengthened frames.

2. Experimental program

An experimental program was conducted to investigate behaviour of cylindrical concrete specimens confined by wrapping CFRP composites in different arrangements, under monotonic (axial) load. The materials, concrete and instrumentation details are presented in the following sections.

2.1. Materials

2.1.1. Fibre reinforced polymer composite

The Carbon Fibre Reinforced Polymer (CFRP) was used in this study as confining material, while epoxy resin was used to adhere it to the specimens. The CFRP was chosen on the basis of availability in the local market. The properties of CFRP, as provided by the supplier, are shown in Table 1.

2.1.2. Epoxy adhesive

The epoxy adhesive used in this research was also obtained from local market. The properties are shown in Table 2. The epoxy contained two mixtures named resin (A) and hardener (B). As per manufacturer's guidelines, the mix ratio was set to 4(A) to 1(B) by weight and cured for seven days at room temperature.

Table 2. Properties of epoxy adhesive

Properties	Epoxy adhesive (Chemdur 300)
Density	1,31 kg/litre
Pot life	30 min. at 35°C
Viscosity	Pasty, does not flow
Tensile Strength	30 N/mm^2

2.1.3. Concrete

Keeping in view the compressive strength typically used in Pakistan for low and medium rise structures, a normal strength concrete with compressive strength of 20 MPa was used throughout the program. No additive was added. The mix ratio of concrete was 1:2:4 (cement: sand: gravel) by weight to achieve the target strength.

2.2. Test specimen details

The total of 9 cylindrical specimens 150 mm in diameter and 300 mm in height were cast and tested in axial compression during the experimental work. The following designations were used for these specimens: controlled specimens were designated as C1, C2 and C3. For CFRP wrapped specimens, the letter W indicates wrapped specimen; it is followed by the number designating the number of layers, and by the second number denoting the sample number. For example, W-1-1 denotes the first wrapped specimen with one CFRP layer. Other specimens were: W-1-2, W-1-3, W-2-1, W-2-2, and W-2-3.

2.3. Instrumentation

All tests were performed using a 2000 kN capacity testing machine at the loading rate of 0.15 MPa/s. A data logger was used for data acquisition while an extensometer was fixed onto the specimens to measure axial strain. Figure 2 shows the test setup and specimens ready for testing with load cell and extensometer.

Table 1. Properties of confinement material

Confinement material	E [kN/mm^2]	F_{ult} [MPa]	ε_{ult} [%]	t [mm]	Density [gr/cm^3]	Type of fiber roven
CFRP	335	4100	1.7	0.15	1.76	Unidirectional

Figure 2. Specimen ready for testing with full set up

2.4. Experimental results

2.4.1. Test results

All confined specimens failed abruptly by rupture of FRP jackets. Figure 3.a and 3.b shows typical failure of FRP wrapped specimens. There was a good bond between the FRP wrap and concrete, as a thin layer of concrete was still attached to FRP sheet after failure. Due to sufficient overlap of FRP wraps, no failure was observed at this location. For all confined specimens, the failure was sudden and there were no warning signs. Moreover, specimens confined with the double layer of FRP failed explosively, and the specimens were fully disintegrated.

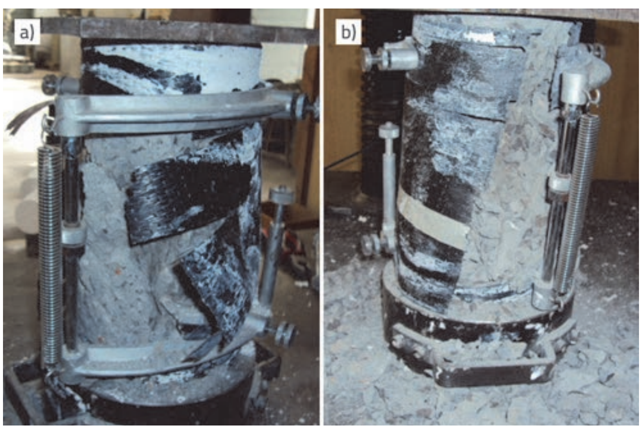

Figure 3. Typical failure of specimens: a) W-1-1; b) W-2-1

2.4.2. Axial stress-strain response

The average stress-strain curves for controlled, CFRP single (W-1) and double layered (W-2) confined specimens, obtained from the test results, are shown in Figure 4.

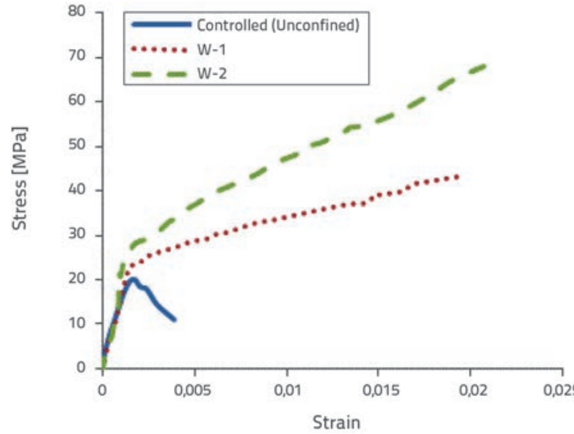

Figure 4. Axial Stress-strain Behaviour of unconfined and confined test specimens

The stress-strain behaviour shows a visible improvement of ductility and peak strength when CFRP wrapped specimens are used. The controlled specimens gave an average strength of 20.3 MPa with the peak strain of 0.0018, while the specimens with a single layer of CFRP gave an average strength of 43.1 MPa and the ultimate strain of 0.019. An increase in ultimate strain was also observed. When two layers of CFRP were used, there was a remarkable increase in both ultimate stress and strain to an average of 68 MPa and 0.021, respectively.

2.5. Comparison with existing stress-strain models

The existing stress-strain models, suggested by different researchers, had to be compared with the experimental stress-strain curves for the purpose of analytical modelling. Many researchers proposed stress-strain models for different confinement mechanisms. Some of them predicted the values that are quite close to experimental values, and are therefore widely used to predict the stress-strain values. Out of those models, the ones proposed by Saman [19], Mander [20] and Lam and Teng [6] were compared with experimental results. Figure 5.a and 5.b show the comparison of these three models with experimental values of CFRP single layer and CFRP double layer results, respectively.

The comparison clearly shows that the model proposed by Lam and Teng [6] closely predicts the stress-strain behaviour for both single layered and double layered CFRP wrapping.

As will be discussed later in Section 3, the RC frame, which was used for modelling, had square columns, and the specimens used in the testing were circular in shape. The effective confinement reduces drastically due to shape effect [21], so the shape factor k_{s1} for strength modification and k_{s2} for strain modification was incorporated.

As the stress-strain model by Lam and Teng [6] provides a more accurate prediction, stress-strain curves for 3 and 4

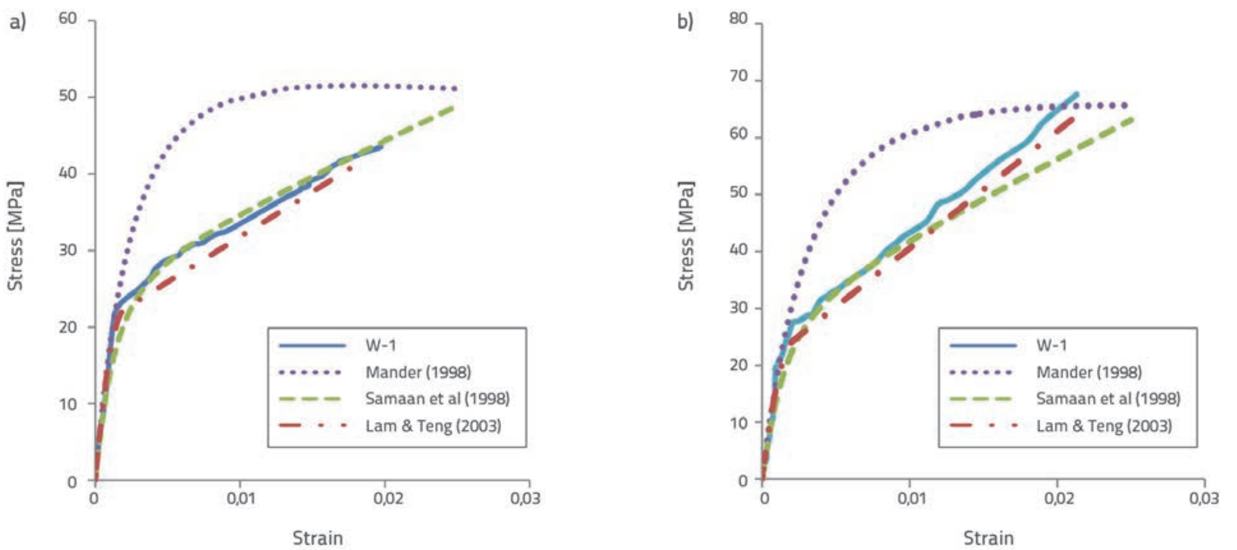

Figure 5. Comparison of experimental stress-strain curve with other models: a) Single wrap CFRP; b) Double wrap CFRP

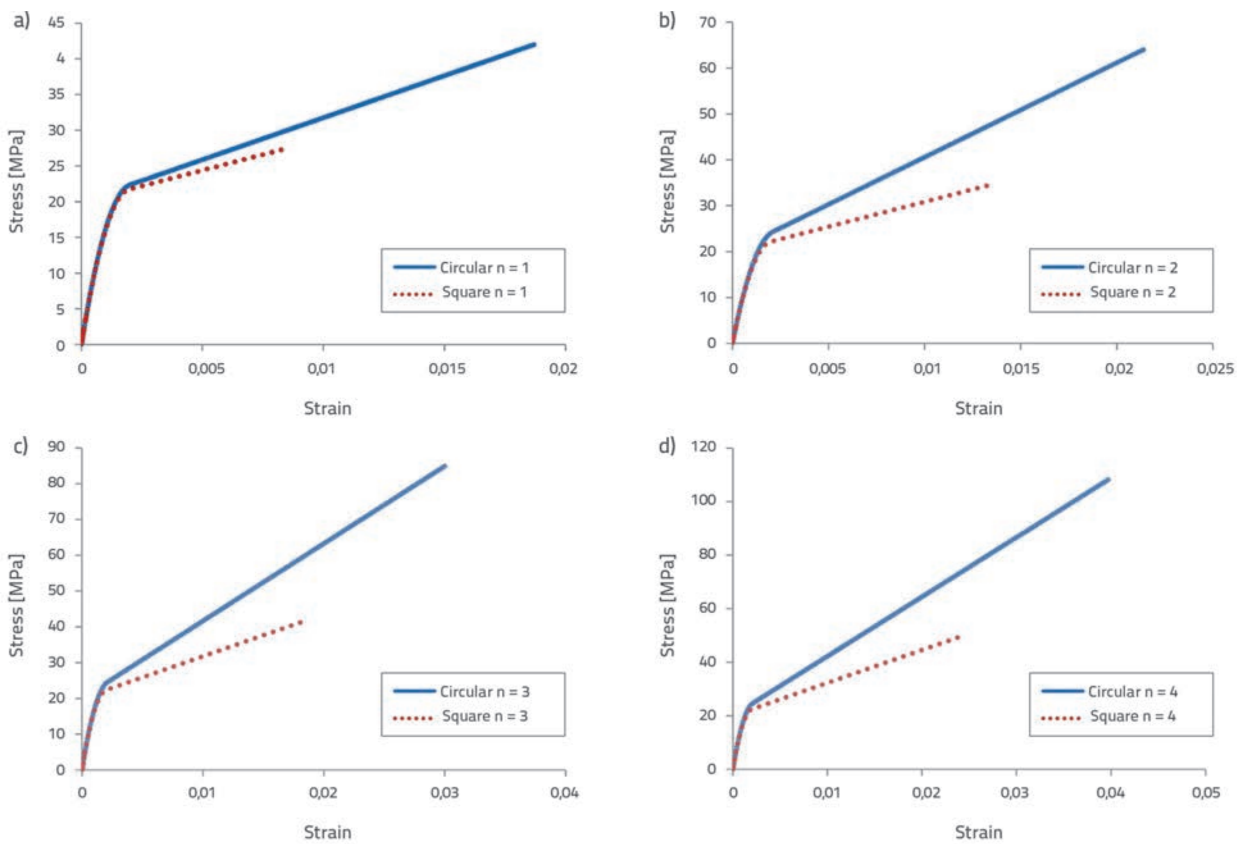

Figure 6. Conversion of stress-strain curves to equivalent square specimens: a) n=1; b) n=2; c) n=3; d) n=4

layers can also be predicted by changing the number of layers in the equations and, after converting them to an equivalent square, the predicted behaviour is obtained as shown in Figure 6.a, 6.b, 6.c and 6.d.

3. Modelling and analysis

A generic RC frame was selected for the analysis based on the existing building stock in Pakistan where about 10-15 %

of buildings are made of reinforced concrete, and have 2-8 storeys [2]. These privately owned low to mid rise buildings are usually non-engineered and contain previously mentioned deficiencies. The frame was originally designed for gravity loads only with non-seismic detailing. As discussed in Section 2.5, Lam and Teng model, being closer to the experimental results, was used to model the properties of concrete with varying level of CFRP confinement. Concrete cover was taken to be 30 mm away from the axis of steel bars. The yield and ultimate strength of reinforcement amounted to f_y = 551 MPa and f_u = 656 MPa, respectively. An one-bay two-storey frame was selected for the study. The bay width was taken as 3.70 m, while the storey height was 3.60 m. The beam cross-section was 0.26 m x 0.46 m, and the column cross-section was 0.26 m x 0.26 m. The length of FRP confinement was chosen from largest of plastic hinge lengths, 0.5D and 12.5 % of member length as suggested by [22]. The plastic hinge length was taken as l_p = 0.5h using guidelines as recommended by ATC-40 [23], where l_p is plastic hinge length, and h is the overall section depth. Figure 7 shows the geometry of the selected frame with regions confined with CFRP, and Figure 8 shows reinforcement details of beam and column cross-sections.

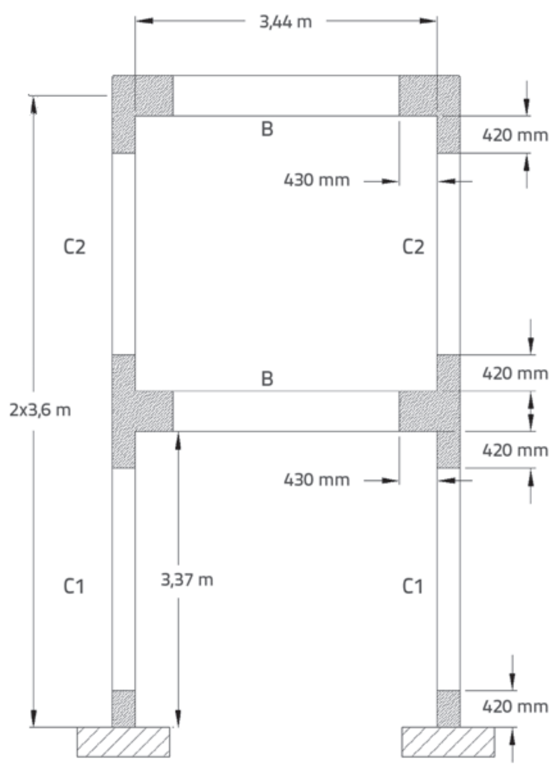

Figure 7. Frame geometry showing CFRP confined regions

The modelling was conducted using the PERFORM 3D software [24]. It is a finite element based analytical tool with graphical user interface, capable of assigning different material properties to the same structural members, while also allowing the use of the bar pullout effect during analysis.

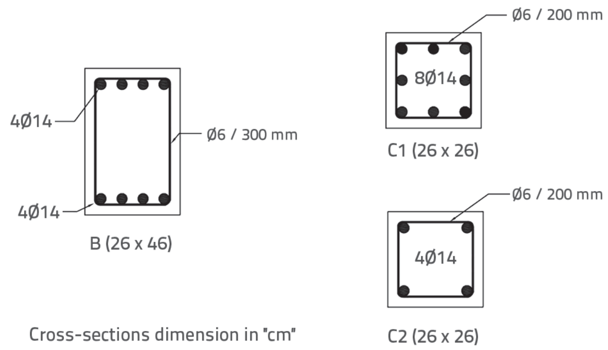

Figure 8. Reinforcement details of beam and column cross-sections

3.1. Bar pullout effect

Different levels of confinement and other parameters influence the bond strength and hence the type of bond failure [25, 26]. Harajli [27] proposed a relationship that is widely used for predicting bond strength improvement due to confinement, and also for the bar pullout failure. The model is shown in Figure 9.

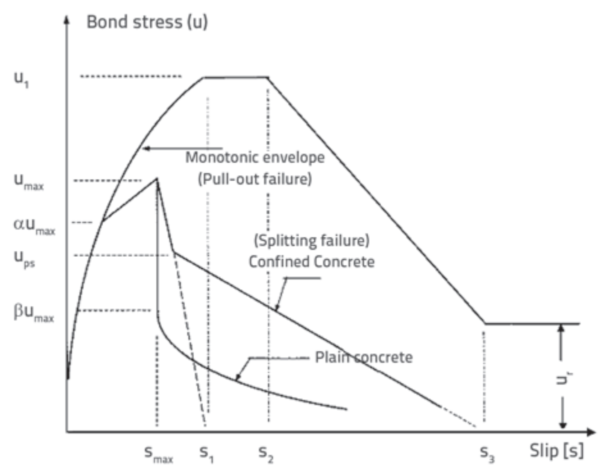

Figure 9. Bond stress-slip model by Harajli [27]

The monotonic envelope (pull-out failure) describes the stress-slip behaviour of concrete confined with FRP. For well confined concrete, the bond stress slip relationship is given in Equation (1).

$$u = u_1\left(\frac{s}{s_1}\right)^{0,3} \tag{1}$$

where:
u - bond stress
s - slip

u_1 - the maximum stress that the bar can develop and is given by $2{,}5\sqrt{f_c'}$ (MPa),

s_1 = 0,15 c_o, where c_o - clear distance between the ribs of reinforcing bars and can be taken as 10 in the absence of bar data.

$s_2 = 0{,}35c_o$, $s_3 = c_o$ and $u_f = 0{,}35u_1$.

The pull-out failure curve was used to model the bond stress-slip relation of steel and concrete in regions where CFRP is used for confinement.

3.2. Modelling of beam and column Cross-sections

Beams and columns were modelled in Perform 3D [24] using the in-elastic fibre section. The cross-section of the beam was split into different number of fibres for both concrete and steel, with 6 fibres for concrete and 2 fibres for 4 bars at the top and bottom of the beam, as illustrated in Figure 10. The area and local axis were used to define the fibre sections. Modelling details of column cross-sections are shown in Figures 11 and 12.

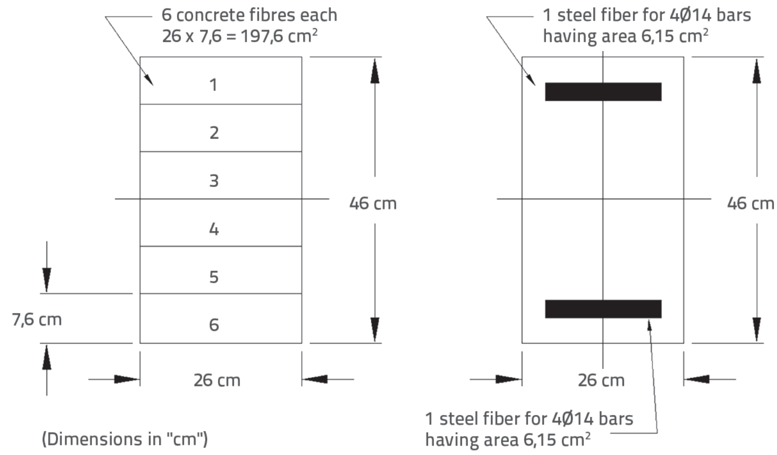

Figure 10. Beam cross-section composed of 6 concrete fibres (left) and 2 steel fibres (right)

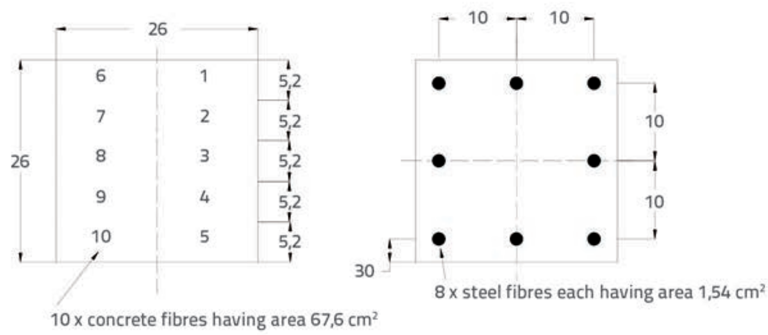

Figure 11. Column cross-section for first floor composed of 10 concrete fibres and 8 steel fibres (left) for concrete and (right) for steel

3.3. Analysis

The non-linear static cyclic analysis was run for the reference and all retrofitted frames and their hysteresis loops were generated. From the hysteresis loops, backbone curves (plot of displacement and base shear) were produced for all frames, as shown in Figure 13, where "n" represents the number of layers of CFRP confinement. Hysteresis loops for all four cases clearly point to improvement in base shear when CFRP wrap is used in the areas near joints in RC columns.

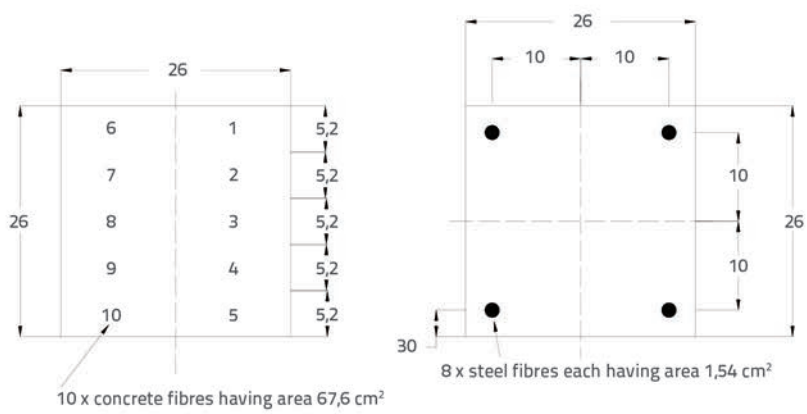

Figure 12. Column cross-section for second floor composed of 10 concrete fibres and 4 steel fibres (left) for concrete and (right) for steel

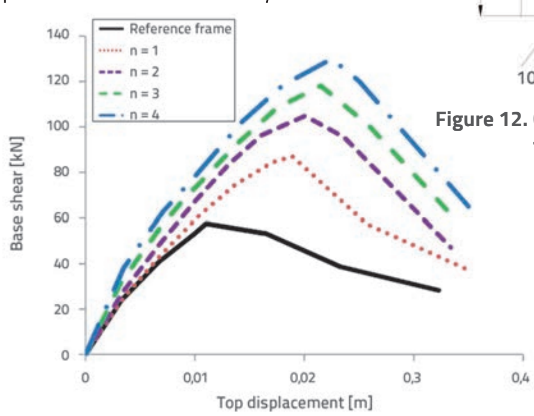

Figure 13. Backbone curve comparison for all frames

3.4. Analytical seismic vulnerability assessment

The Results of analytical model in terms of backbone curves were used for development of the seismic vulnerability curve

(PGA vs Damage Index) for different PGA levels for reference and strengthened frames. For this reason, a methodology for seismic vulnerability assessment as suggested by Kyriakides [28] was used. Kyriakides proposed a technique based on the Capacity Spectrum Method (CSM) by FEMA 440 [29]. The CSM modification was done by Kyriakides who reversed the order to reach peak ground accelerations for a specific damage level.

The design spectrum of UBC-97 [30] is used in this research as it is also implemented by the local building code i.e. Building Code of Pakistan [31]. As this design spectrum depends on different factors such as the type of soil, earthquake zone, and location of active fault line nearest to the site, the design spectrum preparation was done in accordance with UBC 97, and the soil type for the case structure was assumed to be SD, and the near source factor as being equal to one.

To apply CSM, the design spectrum was transformed into the SA-SD space known as Acceleration-displacement response spectrum ADRS (β_0). The back bone curves obtained after the analysis represented capacity of a Multi Degree of Freedom System. CSM requires this curve to be converted to the representative curve of an equivalent Single Degree of Freedom System. The capacity curve was idealized to an elastic perfectly plastic form, so as to establish ductility levels for every displacement. The non-linearity of the structure was incorporated by multiplying the acceleration ordinate with the reduction factor M to get MADRS (β_{eff}, M). Each point on the capacity curve was taken to be a performance point and the corresponding hazard level, described in terms of PGA, was the required output for developing a vulnerability curve.

The next step in developing the vulnerability curve was to quantify damage potential at an approximated structural response. Kyriakides [32] obtained the damage index (DI) for each performance point and suggested no damage state at DI=0 and total collapse at DI = 100. Kyriakides [32] further correlated the Damage Index (DI) with the Mean Damage Ratio (MDR) i.e. ratio of repair to replacement cost, and linearly correlated the DI with the MDR by assuming correlation coefficient to be equal to 1 (Equation 2).

$$\mathbf{MDR} = f\,(\mathbf{DI}) \qquad (2)$$

3.4.6. Seismic vulnerability curves

By using the aforementioned procedure, seismic vulnerability curves were developed for the reference frame and four strengthened frames. The vulnerability curves for the frames are shown in Figure 14. Four different damage levels as suggested by HAZUS [33] are also marked on the curves showing Slight Damage (SD) ranging from 0-40 % MDR, Moderate Damage (MD) ranging from 40-70 % MDR, Extensive Damage (ED) ranging from 70-100 % and Collapse at 100 % MDR.

It can be seen in Figure 14 that 100 % damage is predicted for the reference frame at 0.42 PGA, and that the slope of vulnerability curve becomes very steep at 0.38 PGA, which shows that the structure exhibits brittle failure. This is due to brittle failure of joints due to slip in the main reinforcement in the reference frame. The initial slope of all frames is almost the same as the CFRP starts assuming stress after failure of concrete, and improvement due to confinement is effective thereafter.

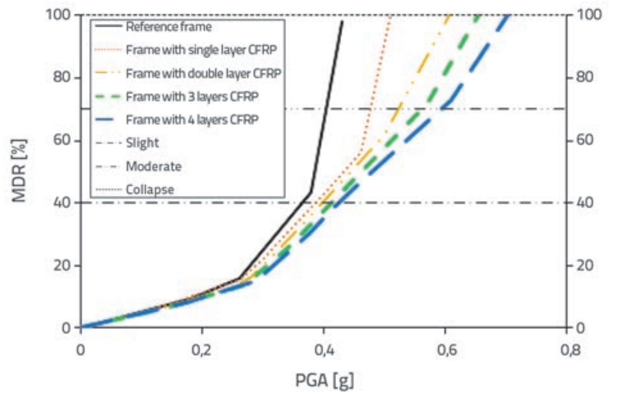

Figure 14. Comparison of vulnerability curves for reference and strengthened frames at different damage levels

The structure with a single layer CFRP wrap showed improved behaviour and complete damage occurred at 0.52 PGA, though the slope was as steep as the reference frame with some improvement in the bond slip behaviour. The two layers CFRP strengthened frame showed further improvement both in complete damage PGA and in the slope of damage. The slope for this structure was gradual which means the damage was gradual due to further improvement in bond between the bar and concrete and more effective confinement provided by CFRP. The frames with three and four layers of CFRP showed further improvement at all damage levels with an overall increase in confinement effect, bar slip behaviour and frame ductility that led to ductile failure. The PGA at damage levels for all frames is shown in Table 3, and improvement percentage at 100 % damage for all frames is shown in Figure 15.

Table 3. PGA of frames at various damage levels by HAZUS [30]

PGA at various damage levels	Reference frame	n=1	n=2	n=3	n=4
PGA at slight damage	0.364	0.384	0.397	0.41	0.424
PGA at moderate damage	0.404	0.475	0.523	0.559	0.592
PGA at collapse	0.42	0.52	0.605	0.654	0.702

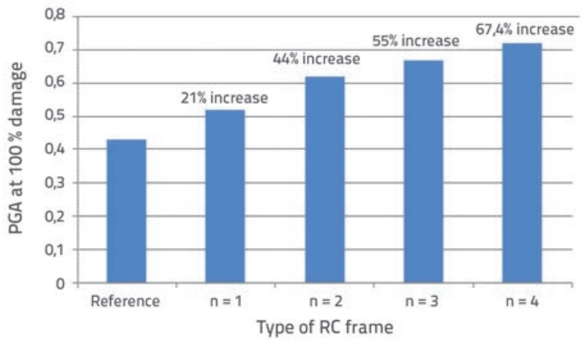

Figure 15. Comparison of performance of RC structures at collapse

The structure with one layer of CFRP showed 21 % increase in performance at collapse level, and the structure with two layers of CFRP wrap showed 44 % increase in performance. The increase in performance for three layers wrap and four layers wrap is 55.8 % and 67.4 %, respectively.

4. Conclusions

The confinement of concrete with a single layer of CFRP increased the ductility by 322 % and, for a double layer of CFRP, the increase was 366 %. The strength of the specimen increased by 115 % for a single layer CFRP confinement and 240 % for a double layer CFRP confinement.

The stress-strain model for Lam and Teng [5] was found appropriate for representing experimental results.

Seismic vulnerability curves were developed for the reference frame and for four strengthened frames. Comparison of vulnerability curves showed improvement in PGA at various damage levels suggested by HAZUS. The collapse hazard level improved by 21 % for a single layer CFRP confinement, 44 % for a double layer CFRP confinement, 55.8 % for three layers and 67.4 % for four layers. CFRP confinement was found to be effective in improving vulnerability of the deficient RC building stock in Pakistan.

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