SEISMIC PERFORMANCE OF RC FRAMES RETROFITTED BY FRP AT JOINTS

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SUMMARY
Different schemes may be used to retrofit RC joints by FRP sheets including web bonding, flange bonding and a combination of the two. In this paper, a flange-bonded scheme, with the aim of relocating plastic hinges away from the joint is presented and its performance is compared with that of the web-bonded scheme. Nonlinear pushover analyses of retrofitted joints of a benchmark RC frame are first carried out and the optimal thicknesses of the FRP sheets for relocating the plastic hinges are determined. The results of pushover analyses on the joints are then used to create a representing model for the RC frame. Further nonlinear pushover analyses are carried out on the retrofitted and the original frame to evaluate such seismic performance parameters as ductility, behaviour factor and performance points in relation to a specified demand earthquake for each frame. Results point to the marked superiority of the flange-bonded scheme compared to the web-bonded scheme in different aspects.

Keywords: RC frames, FRP, joint, seismic retrofitting, flange bonded

1. INTRODUCTION

Beam-column joints are critical components of a frame both in terms of structural stability and its seismic performance. FRP is widely used for seismic upgrading of existing RC structures, repairing and strengthening of damaged structures and strengthening deficient members. The seismic retrofitting of an RC frame may include strengthening members such as beams, columns and beam-column joints. Beam-column joints are critical components of a frame both in terms of structural stability and its seismic performance. FRP retrofitting of an RC beam-column joint can be performed to achieve one or more of the following objectives: (i) to enhance the shear capacity of a shear-deficient joint so that an undesirable shear (brittle) failure is changed into a more favourable flexural (ductile) failure, (ii) to enhance moment capacity and to relocate the plastic hinge away from the face of the column and further into the beam to avoid undesirable formation of plastic hinge inside the joint, and (iii) to change a weak column-strong beam behaviour into a more desirable strong column-weak beam response by enhancing column strength at the joint area. To achieve the first objective, web-bonded schemes are generally used (Fig. 1). To achieve the second objective, both web-bonded and flange-bonded schemes can be employed and the web-bonded and flange-bonded schemes may be used together with wrapping of column sections at joint to achieve the third objective. It should be pointed out that to achieve the second objective, i.e. relocation of the plastic hinge; the web-bonded retrofitting schemes suffer from being impractical in the actual 3D frames due to the presence of cross beams and the integrated slab at the joint. They can only be effectively used to retrofit 2D frames.

Different retrofitting schemes for RC joints using FRP have been the subject of a large number of studies [1-10]. The aim of majority of these methods of retrofitting has been to strengthen a deficient connection. Appropriate FRP retrofitting schemes can also improve the performance of beam-column joints through relocating the plastic hinge away from the face of the column and further into the beam. Mahini and Ronagh [11] tested the effectiveness of FRP web-bonding of scaled beam-column RC joints in relocating the plastic hinge away from the column face. Their experimental studies showed that the FRP web-bonding scheme can restore/upgrade the integrity of the joint, keeping/upgrading its strength, stiffness and ductility, as well as, shifting the plastic hinge from the column face further into the beam. The practicality and effectiveness of using web-bonded FRPs on plastic hinge relocation has also been reported by Smith and Shrestha [12].

In another recent study, Niroomandi et al. [13] carried out a detailed numerical investigation into the effectiveness of FRP web-bonding of joints in relocating the plastic hinge and in enhancing the seismic performance level of an RC frame thus retrofitted. They compared the results of retrofitting the frame at joints web by FRP sheets with those obtained from retrofitting the same frame using a steel X-bracing scheme [14] and found that both retrofitting schemes have comparable abilities to increase the behaviour factor, R, of the frame; the former comparing better on the ductility component and the latter on the overstrength. They also highlighted the limitations of web-bonded scheme in relocating the plastic hinge in large beam-column joints.

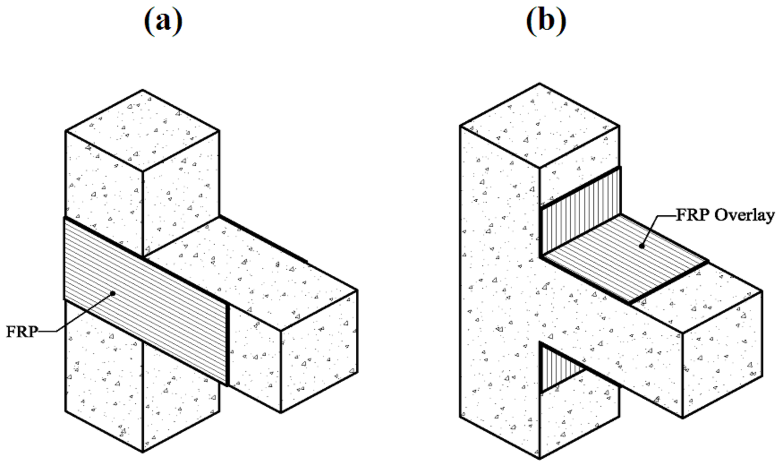

Figure 1. Schematic representation of (a) web-bonded and (b) flange-bonded FRP retrofitting schemes

The practical limitations of the current web-bonded and web-flange-bonded schemes for retrofitting RC joints has lead the authors to introduce a practical flange-bonded scheme which can be applied to both 2D and 3D frames and also allows for the presence of slab in the joint area. The objective of the proposed flange-bonded scheme is to relocate the plastic hinge away from the face of the column and ensure a strong column-weak beam performance. It is assumed that the beams have sufficient shear capacities so that failure is ductile. The ability of the proposed configuration in achieving its objectives is compared with that of the joint web-bonded retrofitting scheme. A performance-based investigation is then carried out on an RC frame retrofitted at joints using the flange-bonded scheme and the results are compared with those reported previously for the same frame, retrofitted using the FRP web-bonded scheme [13]. The numerical investigations are carried out in two parts; first, nonlinear pushover analyses are conducted on detailed numerical models of the individual joints of the frame before retrofitting and after retrofitting. The moment-rotation capacity curves of the joints are then extracted from the results of pushover analyses and used in the second part of the investigations to form an accurate numerical model of the full RC frame. Nonlinear pushover analyses are then carried out on the numerical models of the frame before and after retrofitting and their respective capacity curves are obtained. The capacity curves are, in turn, utilised to extract the seismic behaviour parameters of the frames; including ductility ratio and the behaviour factor and to carry out a performance-based evaluation of the frame before and after retrofitting.

2. ANALYSES OF THE JOINTS

2.1 Numerical Models of the Retrofitted Joints

A 2D, three-bayed, eight-storey RC frame first designed and investigated when retrofitted with steel bracing by Maheri and Akbari [14] is selected for the present investigations. The same frame was later used by Niroomandi et al. [13] to study the effects of retrofitting joints by FRP web-bonded scheme. This moment resisting frame is used in the present study; with the flange-bonded FRP-retrofitted joints replacing the steel bracing system. Details of the design and material and section properties of the frame are given in the above references.

For the numerical analysis of the joints, ANSYS analysis software was used. Concrete was modelled using an eight-node solid element, specially designed for concrete material (ANSYS element solid65). This element is capable of handling plasticity, creep, cracking in tension and crushing in compression. The 5-parameter William-Varnk model was used as the failure criterion. The longitudinal and transverse reinforcements were modelled individually using two-node link elements (ANSYS element link8). The FRP overlays and wrapping strips were modelled using the eight-node, three-dimensional, multi-layered solid element; denoted as solid46 in ANSYS software. The material uses a linear stress-strain curve in both compression and tension and in any of the Cartesian directions. The material properties assigned to the FRP laminates are the same as those used in reference [13].

The finite element mesh of a typical internal joint retrofitted using the flange-bonded scheme is shown in Fig. 2. Representing boundary conditions were applied to the columns ends, so that the columns constant axial compression force at the joint could be applied throughout the analysis. To perform the nonlinear pushover analyses on the external joints, the stepwise load was applied downwards at the tip of the beam, whereas, for the internal joints, equal and opposite loads were applied to the beams ends as shown in Fig. 3. The solution to the nonlinear pushover problem was carried out using the modified Newton-Raphson method.

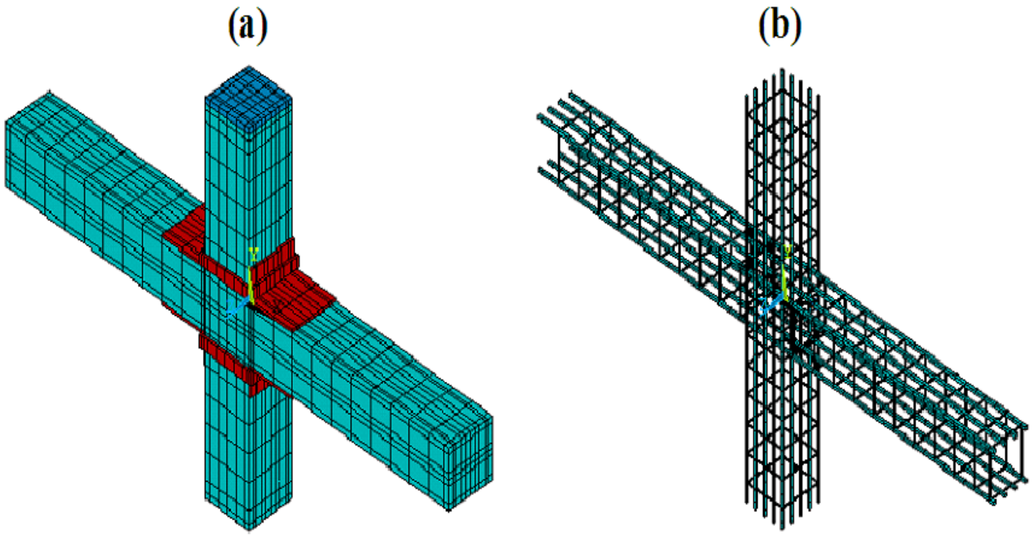

Figure 2. The FE model of a typical internal joint (a) concrete and FRP elements and (b) steel reinforcement.

2.2 Pushover Analyses of the Retrofitted Joints

In order that the required thickness of the overlay for successful relocation of the plastic hinge could be determined, for each joint the nonlinear pushover analysis of the joint was performed using different FRP thicknesses and for each thickness the state of strain in the tensile reinforcements of the beam was used to determine the position of plastic hinge. The length of FRP sheets was kept constant as 500mm for all joints based on the Pauley and Priestly [15] design approach for obtaining the desirable plastic hinge relocation. As a typical example, the maximum strain variation in the longitudinal tensile reinforcements of the beam joining the external joint No. 1 (Fig. 3), retrofitted with 4, 5 and 8 layers of FRP laminates, corresponding respectively to thicknesses of 0.66mm, 0.825mm and 1.32mm, are compared with the reinforcement strain variation in the original (non-retrofitted) beam in Fig. 4. As it is evident in this figure, for the original model the maximum reinforcement strain, corresponding to the location of plastic hinge, occurs expectedly close to the column face. When the joint is retrofitted with 4 layers of laminates, it can be seen that there are two peaks, one at the column face and the other at the end of FRP laminates. Because the two peaks are close in values, it is assumed that a safe relocation has not occurred. However, when the joint is retrofitted with 5 layers of laminate, the amplitude of the peak at the end of FRP overlay clearly dominates the peak formed at the column face; indicating a successful relocation of the plastic hinge to the end of overlay. Larger thicknesses of FRP, such as the case of 8 layers shown in Fig. 4, although showing similar abilities for relocation, would naturally be uneconomical. Therefore for this joint a 5-layer overlay is considered as the optimal thickness for successful plastic hinge relocation. The results of the nonlinear pushover analyses of all joints are summarised in Table 1.

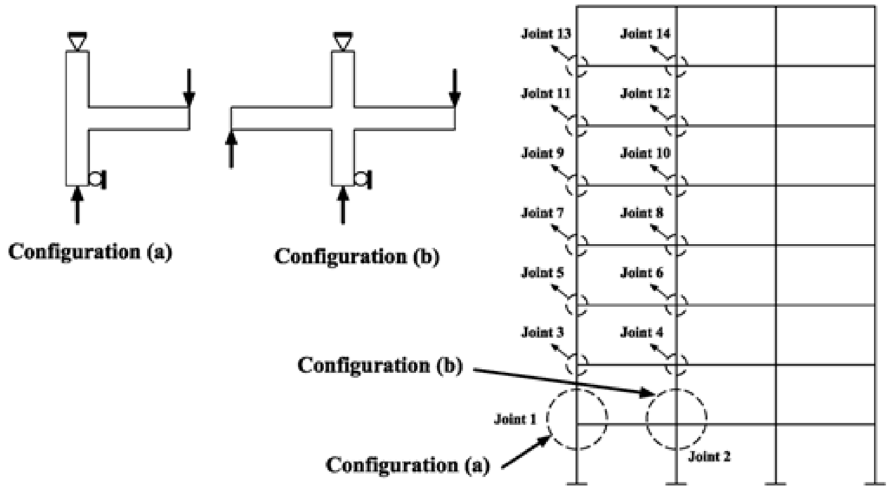

Fig. 3. Selected joints of the frame and the loading and boundary condition configurations for pushover analysis.

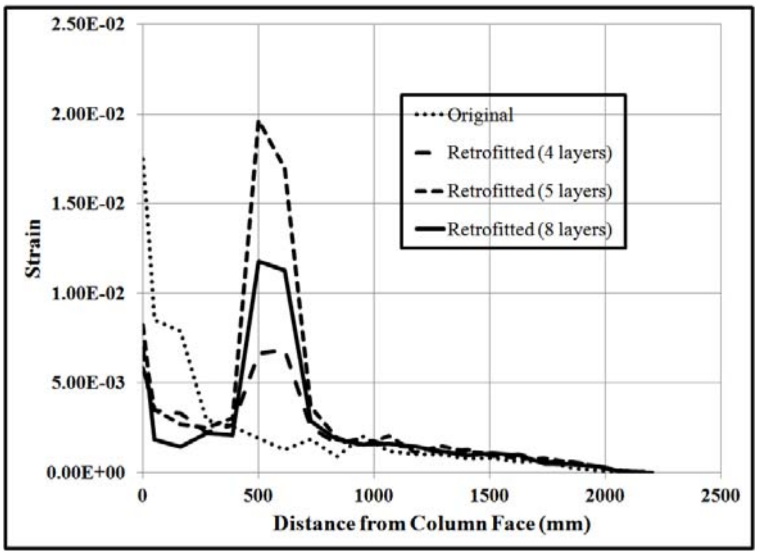

Figure 4. The maximum strain variation in the tensile reinforcements of the beam joining external joint 1

Regarding the term "successful relocation", three further points should be taken into consideration, as follow;

1) Plastic hinging inside the beam: To ascertain a weak beam-strong column performance in the joints, the state of strain in the longitudinal bars of the columns were monitored and compared with that of the beams. It was found that for all the joints, the plastic hinge occurred in the beams and not the columns.

2) Location of the first reinforcement yield: To ensure that the location of the maximum strain in the beam tensile reinforcement corresponds to the location of the first reinforcement yield, the location of the latter was extracted from the pushover results and it was found that in all the retrofitted joints the location of the maximum strain indeed corresponds to the location of the first yield.

3) The ability of a plastic hinge to perform well by ductile rotation also depends on the state of concrete compressive strains. Since FRP overlays do not work in compression, the critical location to monitor the state of concrete compressive strain would be at the column face. The state of concrete compressive strain would be automatically monitored by the ANSYS software and the analysis would be stopped when the maximum strain reaches the limiting strain if in the concrete material section the option "crash on" is activated. A look at the state of concrete compressive strains shows that due to inherent ductility in the original RC frame under investigation, crashing of concrete did not occur in any joints and all failures were as a result of excessive straining of the tensile reinforcements.

The force-beam tip vertical displacement curves obtained from the nonlinear pushover analyses for the retrofitted and the original joints were converted to moment-rotation curves. Moment-rotation curves of the original and the retrofitted joint No.11, as a typical example, are compared in Fig. 5. The ability of the flange-bonded retrofitting scheme in enhancing both the capacity and ductility may be deduced from these curves. The effects of retrofitting scheme on the moment capacity and the rotational ductility of all the joints are highlighted in Table 1. Increases ranging from 18% to 138% for the moment capacity and from 8% to 241% for ductility are noted due to the retrofitting scheme.

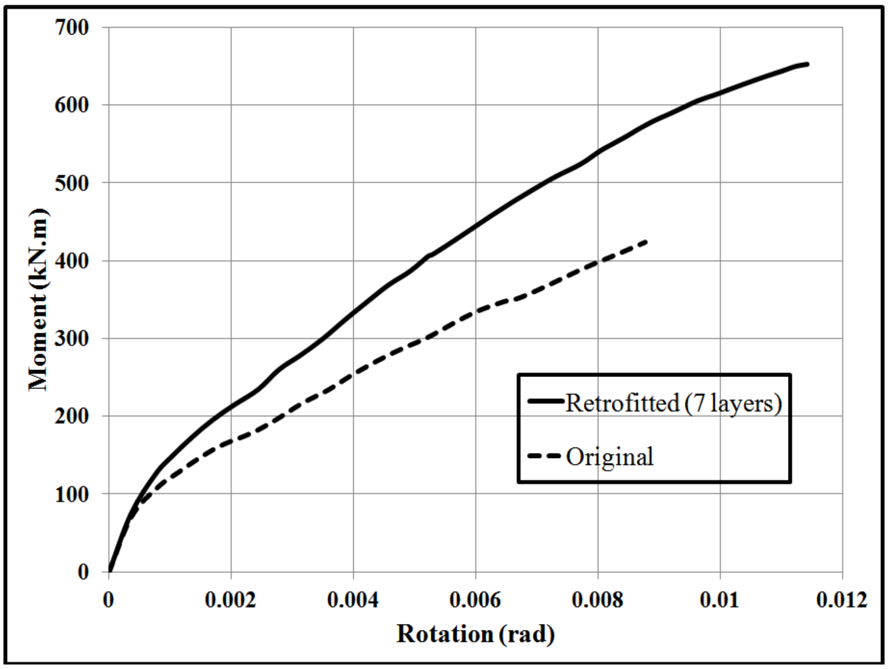

Figure 5. Moment-rotation curves for the external joint 11.

Table 1. Retrofitting details and strength and ductility properties of the joints

Joint	Side	No. of FRP layers	Moment Capacity (M_{max}) (kN.m)			Ductility (μ)		
			Original	Retrofitted	% Difference	Original	Retrofitted	% Difference
13	R	3	407	481	18	21.5	13.4	-37
11	R	7	424	653	54	11.6	22.0	89
9	R	5	452	664	49	10.2	15.5	52
7	R	6	474	708	49	9.5	14.4	51
5	R	7	510	686	34	16.8	22.2	32
3	R	5	518	653	26	22.5	17.1	-24
1	R	5	518	653	26	22.5	17.1	-24
14	L	4	190	253	33	23.2	25.0	8
	R	4	306	445	45	22.6	26.4	17
12	L	6	262	364	39	19.5	25.5	30
	R	7	343	456	33	18.5	24.5	32
10	L	7	159	379	138	6.0	17.0	183
	R	10	-	-	-	-	-	-
8	L	11	209.8	462.2	121	3.75	12.8	241
	R	12	-	-	-	-	-	-
6	L	7	303	467	54	9.8	13.1	33
	R	9	378	600	58	15.2	24.5	61
4	L	5	-	-	-	-	-	-
	R	5	495	640	29	20	22.0	10
2	L	5	-	-	-	-	-	-
	R	5	495	640	29	20	22.0	10

R = Right beam, L = Left beam

3. SEISMIC PERFORMANCE OF THE FULL FRAME

3.1 Pushover Analysis of the Full Frame

Following the analysis of the frame joints, nonlinear pushover analyses are conducted on the original and the retrofitted full RC frame under investigation using the SAP2000 numerical analysis software. The FE model of the frame is shown in Fig. 6. In this model, the contributions of the flange-bonded overlays to the rotational stiffness (Ki) of the retrofitted joints, evaluated from the moment-rotation capacity curves in the previous section, are modelled as nonlinear rotational link elements (NLLink). These elements are assumed to be located at a distance of 500mm away from the column face, corresponding to the length of the FRP overlay. Nonlinear elements capable of modelling plastic hinging are used for the ends of the beams and columns. Flexural moment hinges are assigned to the ends of the beams, while axial-moment hinges are assigned to the ends of columns. Force-deformation criteria for plastic hinging is defined based on the ATC-40 [16] and FEMA356 [17] patterns.

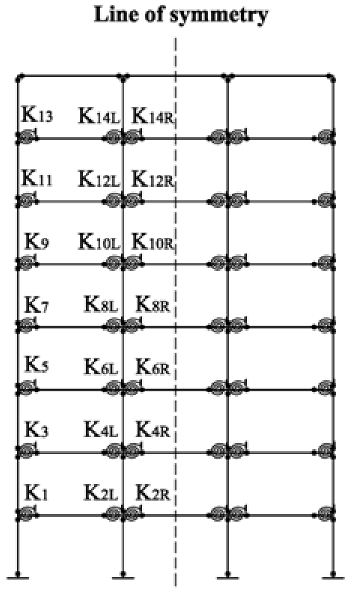

Figure 6. The FE model of the full frame for the nonlinear pushover analyses in SAP2000

An inverted triangular distribution over the height is used as the lateral load pattern. P-Δ effect is also considered in the analysis. The base shear versus roof displacement curves for the original and the retrofitted frame, obtained from the pushover analyses, are compared in Fig. 7. As it is shown, the flange-bonded FRP-strengthening of the joints resulted in an increase of around 160% in the ultimate lateral load carrying capacity of the RC frame. This substantial increase is due to a number of factors including; change in the response of the frame from weak column-strong beam to weak beam-strong column, increased rotational stiffness of the joints due to flange-bonded FRP and particularly, the increased stiffness of the beams due to plastic hinge relocation, in effect reducing the effective length of the beams.

3.2 Seismic Performance of the Frame

In a performance-based design or evaluation, the seismic performance level of a structure describes its state of damage on a capacity spectrum curve. To evaluate the seismic performance of the structure, the nonlinear base shear-roof displacement capacity curve of the structure is first determined using any of the static (pushover), cyclic or dynamic methods. The capacity curve is then converted to the 'capacity' Acceleration-Displacement Response Spectrum (ADRS) curve (S_a-S_d) and its performance level in relation to a specific, code-recommended, ADRS 'demand' curve is obtained using the instructions provided by ATC-40 [16], where, S_a and S_d are the spectral acceleration and displacement, respectively. Fig. 8, shows how the performance point of the FRP-retrofitted frame is obtained using these instructions. In this figure, the capacity ADRS curve is compared with the demand ADRS curve provided by the Iranian seismic code [18] for a design base acceleration (A) of 0.25g and 5% elastic response damping. Considering that the actual damping of the structure undergoing inelastic response is far in excess of the assumed elastic damping of the demand ADRS, appropriate modification to the demand ADRS curve is necessary. Such modification may be carried out using an iterative procedure based on the guidelines given in [16] and shown in Fig. 8. In this figure, β_{eff} is the effective inelastic damping of the frame and the 'banana-shaped' curve relates to the iterative procedure used to evaluate the demand ADRS curve corresponding to the effective inelastic damping. The performance point of the structure in relation to the demand earthquake is then considered as the point of intersection of the capacity and the demand ADRS curves. To evaluate the performance level of a structure, its capacity curve is first idealised by a multi-linear curve similar to that shown in Fig. 9. Each line segment of the curve represents a performance region and any point on the curve may represent a performance level. Key performance levels for a seismic performance-based design or retrofitting include; Immediate Occupancy (IO), Life Safety (LS) and Collapse Prevention (CP) (Fig. 9).

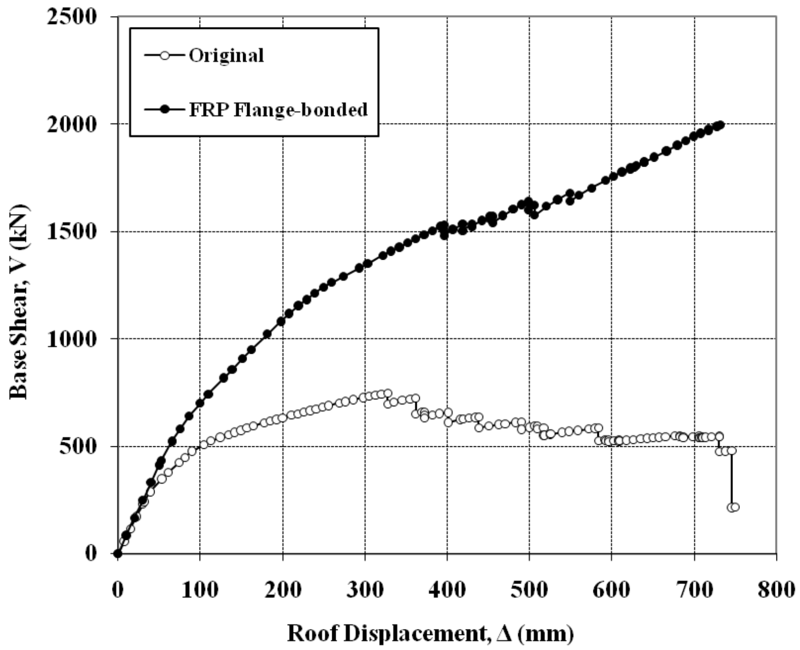

Figure 7. Pushover capacity curves for the retrofitted and the original frames

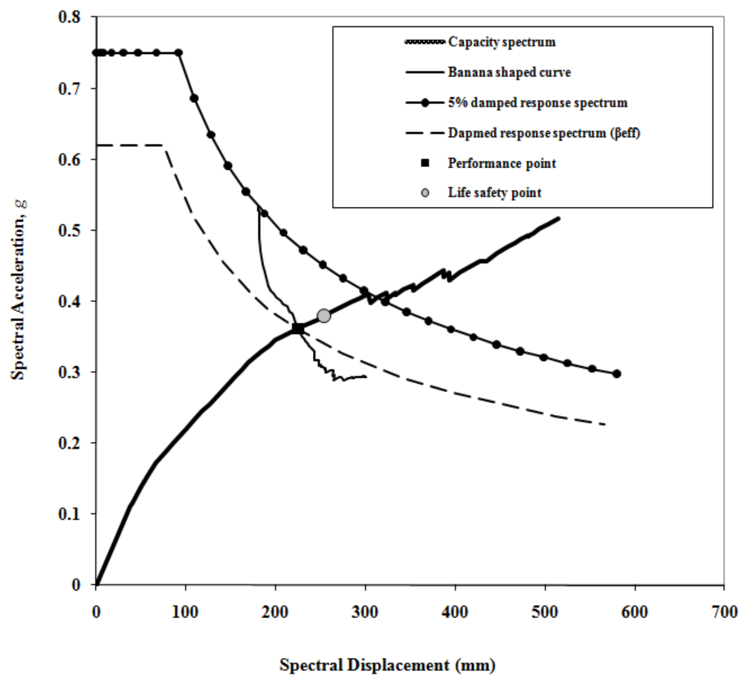

Figure 8. Procedure to evaluate the performance point on an ADRS capacity-demand diagram

The capacity and demand ADRS curves and the performance points for the original frame and the frame retrofitted with the FRP flange-bonded scheme were evaluated in the manner described above and are shown in Fig. 10. The performance point coordinates of the frames are also listed in Table 2.

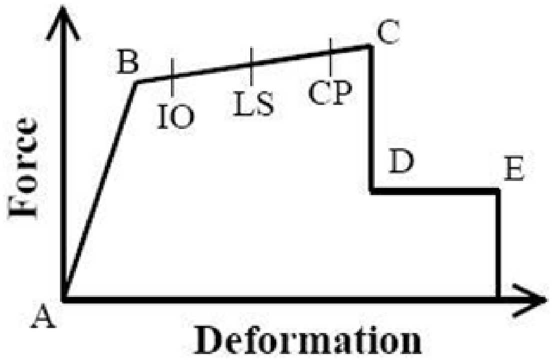

Figure 9. Seismic performance levels on an idealised response curve

With reference to Fig. 10, it is clear that the original frame does not meet the Life Safety requirement of the demand earthquake (A=0.25g), as its performance point falls short of the LS point; whereas, the retrofitted frame easily satisfies the LS requirement. The retrofitted frame not only satisfies the LS requirement for this demand earthquake, but also satisfies the LS requirement for a stronger demand earthquake with a design base acceleration of, A=0.30g.

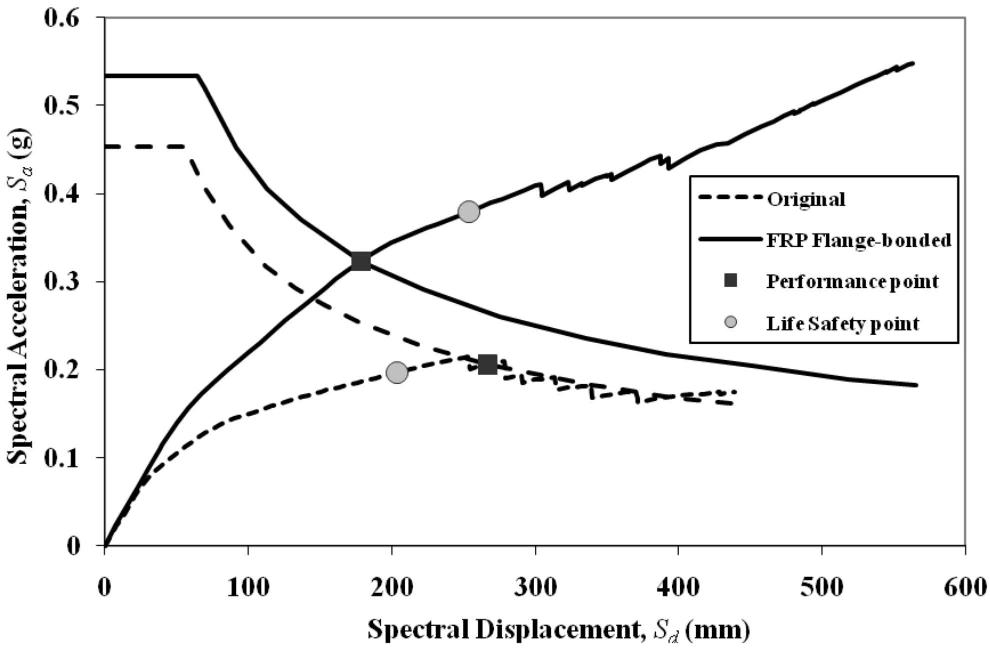

Figure 10. Performance points of the original frame and the flange-bonded retrofitted frame in a demand earthquake with A=0.25g

Table 2. Performance point coordinates of the frames

Frame	Frame Performance Point		Performance Point for Life Safety (LS)	
	S_a (g)	S_d (mm)	S_a (g)	S_d (mm)
Original (A=0.25g)	0.206	267	0.197	204
FRP Flange-bonded (A=0.25g)	0.323	179	0.379	254
FRP Flange-bonded (A=0.30g)	0.361	225	0.379	254
FRP Web-bonded (A=0.25g) [13]	0.263	212	0.269	223

In order that the seismic performance of the flange-bonded scheme could be compared with the performance of the web-bonded scheme, the capacity and demand ADRS curves and performance points (demand earthquake A=0.25g) of the original frame and the frame retrofitted with different schemes are plotted in Fig. 11 and their performance points are listed in Table 2. Comparing the capacity curves of the flange-bonded and the web-bonded FRP retrofitting schemes, it is clear that the flange-bonded scheme shows superiority, not only in terms of increased strength and ductility, but also the seismic performance level. For the flange-bonded scheme, the performance point relates to a spectral displacement of 179 mm; whereas, that of the web-bonded scheme has a spectral displacement of 212 mm. The lower spectral displacement for the flange-bonded scheme indicates that the inelastic lateral load resistance has been enhanced compared to the web-bonded scheme. Also, the marked difference in the spectral acceleration values for the two schemes (23%) shows that the seismic load capacity of the flange-bonded scheme is much more than the web-bonded scheme. Furthermore, with reference to Fig. 11, it is noted that while the frame retrofitted with the flange-bonded scheme satisfies the LS requirement for the demand earthquake having A=0.3g, the same is not true for the web-bonded scheme.

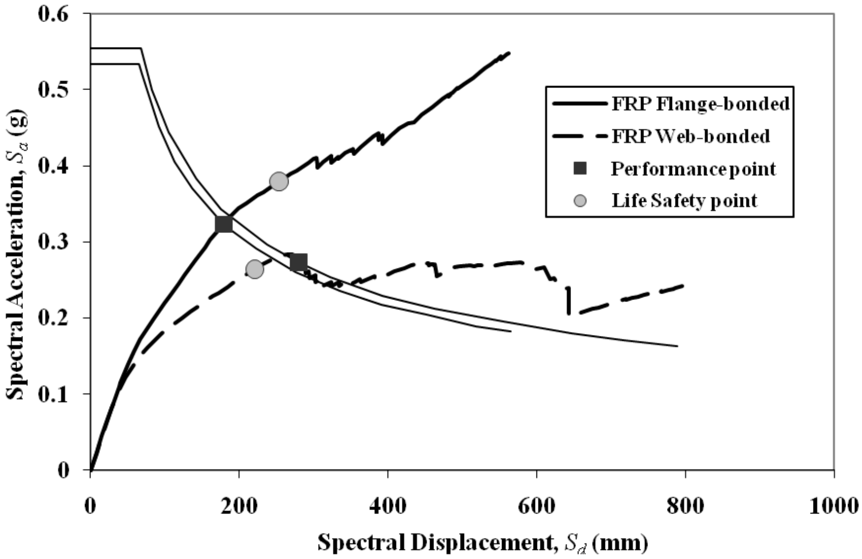

Figure 11. Performance points of the web-bonded and the flange-bonded retrofitted frames in a demand earthquake of A=0.30g

4. CONCLUSIONS

The main conclusions drawn from comparing the performance of the frame retrofitted with this method and that of the original frame and the frame retrofitted by other methods are summarised below.

1. Flange-bonded FRP laminates substantially increase moment capacity and ductility of an RC beam-column joint. For the joints of frame under investigation, increases range, respectively, from 18% to 138% and from 8% to 241%.
2. Retrofitting the joints of an RC frame at flanges by FRP laminates can effectively change a weak column-strong beam frame into a weak beam-strong column frame by relocating the plastic hinges. It can also greatly enhance the lateral load resisting capacity.
3. Retrofitting joints at flanges by FRP greatly enhances the seismic performance point of the frame. A frame failing the Life Safety requirement for a demand earthquake with A=0.25g, was retrofitted to pass, not only the LS requirement for this earthquake, but also the LS requirement for a stronger demand earthquake with A=0.30g.
4. Seismic performance of the flange-bonded scheme far outweighs that of the web-bonded scheme. Substantial differences in the lateral load resisting capacity and ductility, as well as, the performance level of the two schemes are noted. While the frame retrofitted with the flange-bonded scheme could satisfy the LS requirement of a demand earthquake having A=0.30g, the same is not true for the frame retrofitted with the web-bonded scheme.

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