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Performance-based and damage assessment of SFRP retrofitted multi-storey timber buildings

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Abstract. Civil structures should be designed with the lowest cost and longest lifetime possible and without service failure. The efficient and sustainable use of materials in building design and construction has always been at the forefront for civil engineers and environmentalists. Timber is one of the best contenders for these purposes particularly in terms of aesthetics; fire protection; strength-to-weight ratio; acoustic properties and seismic resistance. In recent years, timber has been used in commercial and taller buildings due to these significant advantages. It should be noted that, since the launch of the modern building standards and codes, a number of different structural systems have been developed to stabilise steel or concrete multistorey buildings, however, structural analysis of high-rise and multi-storey timber frame buildings subjected to lateral loads has not yet been fully understood. Additionally, timber degradation can occur as a result of biological decay of the elements and overloading that can result in structural damage. In such structures, the deficient members and joints require strengthening in order to satisfy new code requirements; determine acceptable level of safety; and avoid brittle failure following earthquake actions. This paper investigates performance assessment and damage assessment of older multi-storey timber buildings. One approach is to retrofit the beams in order to increase the ductility of the frame. Experimental studies indicate that Sprayed Fibre Reinforced Polymer (SFRP) repairing/retrofitting not only updates the integrity of the joint, but also increases its strength; stiffness; and ductility in such a way that the joint remains elastic. Non-linear finite element analysis ('pushover') is carried out to study the behaviour of the structure subjected to simulated gravity and lateral loads. A new global index is re-assessed for damage assessment of the plain and SFRP-retrofitted frames using capacity curves obtained from pushover analysis. This study shows that the proposed method is suitable for structural damage assessment of aged timber buildings. Also SFRP retrofitting can potentially improve the performance and load carrying capacity of the structure.

Keywords: timber buildings; performance-based assessment; damage detection; pushover analysis and Sprayed Fibre Reinforced Polymer (SFRP)

1. Introduction

The requirement for lightweight, resistant, sustainable and cost-effective structures has been increasingly in demand worldwide due to reduced supply of raw materials and energy sources. The efficient and sustainable use of materials in building design and construction has received

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significant attention by civil engineers and environmentalists. Therefore, possessing all the foregoing characteristics, timber is being extensively used as one of the main materials in civil infrastructure particularly in terms of aesthetics; fire protection; strength-to-weight ratio; acoustic properties and seismic resistance. It is the oldest structural material and still continues to be a popular choice in modern infrastructure (Sweeney 2012). Prior to the 20th century timber has been the only structural material that has been extensively used in the construction of the buildings and footbridges in Australia, Europe and the rest of the world (Lyons and Ahmed 2005, Rijal 2013, Smith 2011). Recently, there is an increasing interest on using timber product due to its low embodied energy and low environmental impacts. Furthermore, timber structures also perform better under fire than steel structures. Conversely steel is very weak under fire and fails catastrophically, whereas in high temperature larger timber sections will not fail and stay stable over longer time. In fact, it forms a layer of insulating char when exposed to flame (Buchanan 2007, Purkiss and Li 2013).Therefore, engineers and researches are encouraged to take the advantages offered by timber in design and construction of multi-storey structures (Vessby 2011). For instance, Lam (2009) stated that more than 90% of residential buildings in North America and Japan are built with timber frame and the demand for timber as a structural material keeps increasing for single and multi-story buildings, as well as low-rise commercial buildings.

Prior to the introduction of new design codes and standards, most of the structures were designed based on vertical/gravity loads only. Therefore, those structures might not satisfy the specific requirements of new codes and need to be replaced or retrofitted to upgrade their structural integrity in order to withstand standard loads, such as earthquake actions (Banthia *et al*. 2002, Soleimani, 2006). It should be mentioned that, although demolishing and replacing degraded structures with new structures is a straightforward solution, it is costly and time-consuming. Therefore, to avoid the replacement of degraded structures, it is vital that the existing older structures be routinely inspected and any loss of capacity and integrity be promptly addressed. In this regard, Talukdar (2008) reported that repairing and/or retrofitting degraded structures are feasible and cost-effective solutions compared with replacement. However, disadvantages associated with traditional rehabilitation or retrofit methods have resulted and that researchers have developed new techniques using new materials, such as advanced fibre reinforced polymers (FRPs), to tackle these issues (Talukdar and Banthia 2010). Recent applications have demonstrated that fibre composites can be effectively and economically used for new structures, as well as in the strengthening and retrofitting of existing civil infrastructure (Hollaway and Teng 2008, Mahini and Ronagh 2010, 2011). FRP is a material with high stiffness and strength to weight ratio; high Young's Modulus; high fatigue performance; and very capably reinforces timber (Juvandes and Barbosa 2012, Valipour and Crews 2011). Moreover, its other advantages such as being lightweight; having superior resistance to corrosion; and some flexibility have led this material to be outstanding and exceptional alternative to steel, especially in the aggressive and maritime environments (Akbar *et al*. 2010).

One of the main concerns of engineers is to evaluate the integrity of existing structures which were designed based on older codes, particularly those structures that were not designed for the earthquake actions. In such structures, deficient members and joints require strengthening in order to satisfy new provisions, acceptable level of safety and to avoid brittle failure subjected to earthquake actions (Lim *et al.* 2013, Yadav and Nim 2014). Despite numerous structural systems have been developed to design steel or concrete multi-storey buildings since the launch of the new standards, structural performance of multi-storey timber buildings subjected to lateral loads has not yet been fully understood (Vessby 2011). One approach is to retrofit the beam-column joints in order to relocate potential plastic hinges away from the joint and increase the ductility of the frame (Mahini and Ronagh 2010). For the joints to remain elastic, plastic hinges should be induced to develop sufficiently away from the joint core (Lim *et al.* 2013). Past experimental studies indicated that fibre reinforced polymer (FRP) repairing/retrofitting can force the plastic hinge away from the column face and into the beam. This system upgrades the integrity of the joint and increases the strength, stiffness and ductility of the joint in such a way that the joint remains elastic (Hadigheh *et al*. 2014, Niroomandi *et al*. 2010).

FRP retrofitted timber members may be subject to premature failure due to de-bonding. To mitigate the effect of de-bonding, anchor devices can be used to prevent the peel-off, however, this method will increase the cost and time of the project and also its implementation is complicated (Lee *et al.* 2005). Recently, a novel method of applying fibre-reinforced polymers was developed whereby FRPs are sprayed onto the surface of member. In this method, the surface of the element needs to be covered with a bonding agent/adhesive prior to application. However, it needs careful work to ensure that the substrate surface is not contaminated. By using a bonding agent the bond can be enhanced. In this technique a roving format of glass fibre or carbon fibre are cut into desired lengths using a fibre chopping device that is attached to the nozzle and injected into the spray stream. Simultaneously, a mixed adhesive, as a single compound, is sprayed onto the surface. These two streams - FRP and adhesive - combine and continue onto the spraying surface together and coat the member surface (Talukdar and Banthia 2010). The main advantages of this technique over traditional methods are that it is highly cost-effective, is less labour intensive, can be applied to a number of strengthening projects, including those involving seismic retrofits (Boyd *et al*. 2008). Apart from its use in strengthening and retrofitting structures, this method can provide protective coatings to structures in aggressive environments such as offshore platforms and as protective linings in/on structural systems in sub-soil conditions with adverse groundwater (e.g. acid sulphate soils) (Banthia 2002).

In order to investigate the seismic performance of multi-storey timber buildings, recently a collaborative research has been conducted by the University of Basilicata (UNIBAS), in Potenza, Italy and the University of Canterbury, in Christchurch, New Zealand. In this research, the feasibility of applying 'jointed ductile post-tensioning technology', originally conceived for use in concrete structures, to Glue Laminated Timber (glulam) has been examined (see Fig. 1). Ponzo *et al*. (2012) reported a significant decrease in drift and up to 40% decrease in total drift of the frame. In this study, pushover analyses of a multi-storey timber building with and without sprayed FRP reinforcement have been modelled in SAP2000® . All the materials and the frame configuration are exactly the same with the timber frame considered by Ponzo *et al.* (2012) in the structural laboratory of the University of Basilicata in Potenza. It should be noted that for the purpose of comparison, just one frame of the subject timber building is used in this study. It is three storeys high with single bays in the direction of main beams. In the real frame, each level is 3 m high and the frame footprint is 6 m. However, Ponzo *et al.* (2012) applied a scale factor of 2/3 to the prototype structure, resulting in a storey height of 2 m and a building footprint of 4 m. This study also follows the recommendations of Ponzo *et al*. (2012) and the SAP2000® model is scaled to two-thirds the size of the prototype building (scale factor $\lambda = 2/3$). Therefore, considering all the design criteria of length, force, moment, mass and weight of the simulated structure is required to be scaled by λ, λ^2, λ^3, λ^2 and λ^2, respectively (Mahini 2005, Smith, 2014). This building was designed as an office structure with live load of 3 MPa. The type of columns and beams assumed glulam grade GL32h 200 mm wide and 320 mm deep. Table 1 shows characteristic strength and stiffness values of glulam grade GL32h. As GL32h is among materials with a lower failure stress in tension than the proportional limit stress in compression, Buchanan (1990) stated that bending failures occur in the tension zone, without any compression yielding. Therefore, the moment-curvature relationship is linear to failure.

This study aims to investigate the influence of sprayed FRP-retrofitted beams, as shown in Fig. 2, on the performance of multi-storey timber frames based on response spectrum of the ATC-40 (Comartin *et al*. 1996) and the Australian Standard (AS1170.4, 2007) using pushover analysis and capacity curves (base shear-roof displacement relations) (Hadigheh *et al.* 2014, Niroomandi *et al*. 2010). In this study, FRP composites are sprayed onto the tensile zone of timber beams to increase the flexural loading capacity and stiffness. In Table 2, the mechanical properties of the glass fibre used to retrofit timber beams are tabulated.

Fig. 1 Experimental post-tension frame constructed in UNIBAS laboratory, Italy, in collaboration with the University of Canterbury in Christchurch, New Zealand (Ponzo *et al.* 2012)

Table 1 Characteristic strength and stiffness of GL32h

Modulus of elasticity (N/mm^2)		Bending strength (N/mm^2)	32
parallel to grain (mean)	13700	Shear strength (N/mm^2)	3.8
parallel to grain (5 % fractile)	11100	Shear modulus (N/mm^2)	850
perpendicular to grain (mean)	460	Density (kg/m^3)	430
Tension strength (N/mm^2)		**Compression strength (N/mm^2)**	
parallel to grain	22.5	parallel to grain	29
perpendicular to grain	0.5	perpendicular to grain	3.3

Table 2 mechanical properties of glass fibre

Density	2.55 - 2.6	g/cm^3
Modulus of Rupture	3300 - 3450	MPa
Shear Modulus	30 - 36	GPa
Tensile Strength	1950 - 2050	MPa
Modulus of Elasticity	72 - 85	GPa

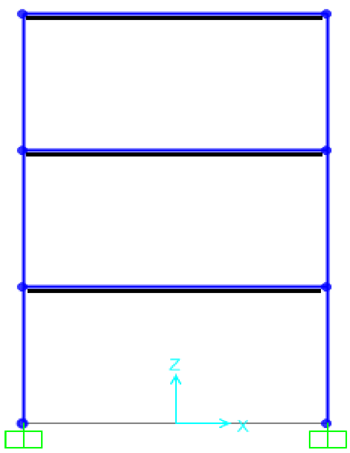


Fig. 2 SFRP retrofitting scheme (retrofitted area is highlighted with thicker black lines)

2. Pushover analysis of original and SFRP-retrofitted timber frames

Simplified linear-elastic techniques are not adequate to assess the performance of a multi-storey timber building. Therefore, a new generation of design and seismic concerns requires considering the inelastic behaviour of a building subjected to seismic loading (Lim *et al.* 2013). The non-linear pushover analysis can be employed to sufficiently evaluate the seismic performance of a structure without requiring complex modelling (Hadigheh *et al.* 2014, Lim *et al*. 2013, Niroomandi *et al*. 2010). Non-linear static pushover analysis is a specialised procedure used in performance-based design for seismic loading. In this method, monotonically increasing forces are applied to a non-linear mathematical model of the structure until the displacement of the control node exceeds the target displacement (Fajfar 2000, Naeim 2001, Niroomandi *et al*. 2010). Based on FEMA356 (2000) guidelines for a specific earthquake, the building should have enough capacity to withstand a specified roof displacement. This defines the performance point or the target displacement, Δ_t, and is intended to represent the maximum displacement likely of the roof of a building to be experienced during the design earthquake (Naeim 2001, Niroomandi *et al*. 2010). FEMA356 (2000) also states that the target displacement, Δ_t, at each floor level can be calculated using the following expression

$$\Delta_t = C_0 C_1 C_2 C_3 S_a \frac{T_e^{\,2}}{4\pi^2} g \tag{1}$$

where C_0 is a modification factor to relate the spectral displacement and likely building roof displacement; C_1 is a modification factor to relate expected maximum inelastic displacements to displacements calculated for linear elastic response; C_2 is a modification factor to represent the effect of hysteresis shape on the maximum displacement response; and C_3 is a modification factor to represent increased displacements due to dynamic $P\text{-}\Delta$ effects. S_a is the response spectrum acceleration at the effective fundamental period, T_e, denotes the effective fundamental period of the building in the direction under consideration, and g is acceleration of gravity. Further explanation of these values is provided by FEMA356 (2000).

3. Capacity curve

The generation of a capacity curve (base shear vs roof displacement Fig. 3) defines the capacity of the building uniquely for an assumed force distribution and displacement pattern. If the building displaces laterally, its displacement response must lie on this capacity curve. The performance point can be found on the capacity curve by correlating this capacity curve to the seismic demand generated by a specific earthquake or ground shaking intensity. The location of this performance point relative to the performance levels defined by the capacity curve indicates whether or not the performance objective is met (Naeim 2001). The Capacity Spectrum Method which is usually known as the Acceleration-Displacement Response Spectrum (ADRS) is one of the methods used to determine the performance point. This method requires that both capacity curve and the demand curve be represented in response spectral ordinates. It characterises the seismic demand initially using a 5% damped linear-elastic response spectrum and reduces the spectrum to reflect the effects of energy dissipation to estimate the inelastic displacement demand. The intersection of the capacity curve and the reduced demand curve denotes the performance point at which capacity and demands are equal (Fajfar 2000, Naeim 2001, Niroomandi *et al*. 2010).

To convert a spectrum from the standard format (Spectra Acceleration, S_a vs Period, T) to the ADRS format (see Fig. 4), the value of Spectral Displacement, Sd_i, for each point on the standard curve (Sa_i, T_i) is required to be determined. This can be done using the following equation

$$\mathrm{S}d_\mathrm{i} = \frac{T_i^2}{4\pi^2} Sa_i g \tag{2}$$

The capacity spectrum can also be developed using the pushover curve by a point by point conversion to the first mode spectral coordinates. Comartin *et al*. (1996) and Naeim (2001) stated that any point V_i (Base Shear) and Δ_i (Roof Displacement) on the capacity (pushover) curve is converted to the corresponding point Sa_i, Sd_i on the capacity spectrum using the equations

$$\mathrm{S}a_\mathrm{i} = \frac{V_i / W}{\alpha_1} \tag{3}$$

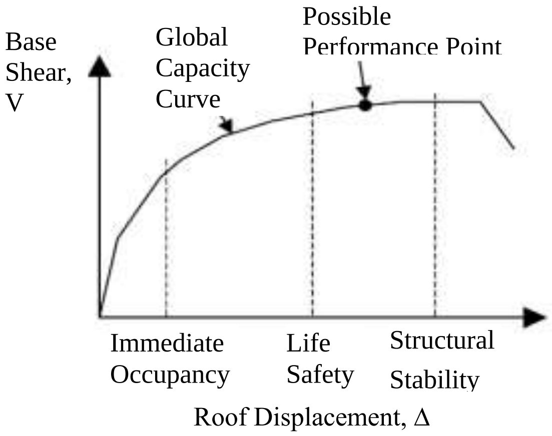


Fig. 3 Building Capacity Curve (Comartin *et al.* 1996).

$$Sd_i = \frac{\Delta_i}{(PF_1 x \phi_{1,roof})} \tag{4}$$

Where α_1 is modal mass coefficient, W is the weight of structure, PF_1 is participation factor for the first natural mode of the structure and $\varphi_{1,roof}$ is the roof level amplitude of the first mode. The modal participation factors and modal coefficient are completely described in Naeim (2001).

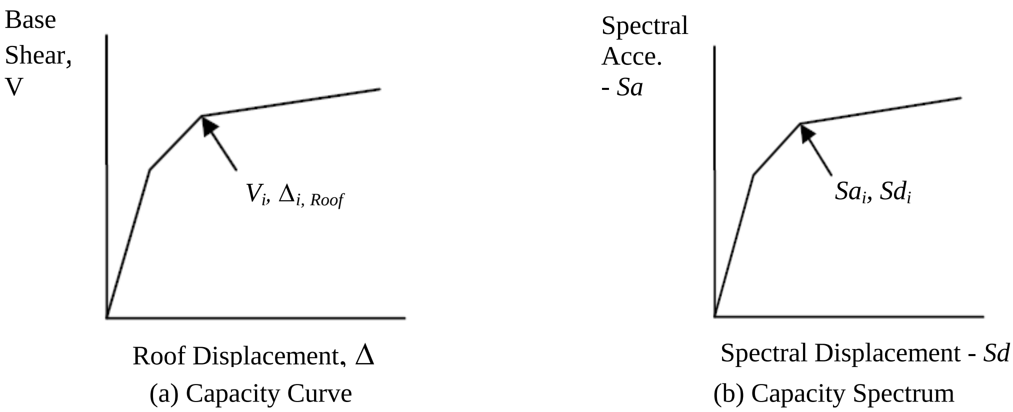


Fig. 4 Capacity Spectrum Conversion (Comartin *et al*. 1996).

Comartin *et al.* (1996) and Naeim (2001) stated that to account for the damping, the response spectrum is reduced by reduction factors SR_A and SR_V which are given by

$$\mathrm{S}R_{\mathrm{A}} = \frac{3.21 - 0.68\ln(\beta_{eff})}{2.12} \tag{5}$$

$$\mathrm{S}R_{\mathrm{V}} = \frac{2.31 - 0.41\ln(\beta_{eff})}{1.65} \tag{6}$$

where β_{eff} is the total effective damping.

4. A preliminary seismic assessment of the retrofitted and original frames

The SAP2000® software was employed to model non-linear static (pushover) analyses of the timber frame. For this purpose, a constant gravity load, equal to the total permanent load (dead load) plus 40% of the live load, based on the AS1170.4 (2007), was applied to each frame. P-Δ effect was also considered in the analysis. In the studies of Smith *et al.* (2013) and Ponzo *et al.* (2012), type of the soil was assumed as a medium soil (S_B). ATC-40 (Comartin *et al.* 1996) states for each earthquake hazard level, the structure is assigned a seismic coefficient C_a and a seismic coefficient C_v. The seismic coefficient C_a represents the effective peak acceleration (EPA) of the ground. The seismic coefficient C_v represents 5 percent-damped response of a l-second system. Based on ATC-40 for sites situated on soil type S_B, the value of C_a should be taken to be equal to 0.4 times the spectral response acceleration (units of g) at a period of 0.3 seconds and the value of C_v should be taken to be equal to the spectral response acceleration (units of g) at a period of 1.0 second. C_a and C_v are calculated using this guideline recommendation being 0.4 for both. To obtain the bending moments and forces at the beam and column, the beam-column joints are modelled by giving end-offsets to the frame elements. It is also notable that all the columns at foundation level are considered as fixed. To accurately set up the pushover analysis, the non-linear behaviour of the structural elements is to be taken into account. In order to model nonlinearity requirements that are essential for timber frames, a point-plasticity method is considered based on FEMA356 (2000) guidelines. In the present study, the plastic hinges are assumed to be concentrated at a specific point in the frame members under consideration. Normally the hinge proper ties for each of the six degrees of freedom are uncoupled from each other. However, SAP2000® provides the opportunity for users to specify coupled axial-force/bi-axial-moment behaviour. This is called the *P-M2-M3* or *PMM* hinge. Beams and columns, in this study were modelled with flexure hinges at possible plastic regions under lateral load. The flexural hinges in beams were modelled with uncoupled moment (*M3*) hinges, whereas the flexural hinges in columns were modelled with coupled *P-M2-M3* properties that include the interaction of axial force and bi-axial bending moments at the hinge location.

It should be mentioned that in the original frame (Fig. 1), several devices including a post-tensioning system, a connecting angle, as well as an energy dissipater, were installed to induce the required ductility and dissipation into the frame. In this paper, however, these fasteners/elements have not been considered. The analytical models of the retrofitted timber frames using sprayed FRP composites were considered in this study. The sprayed FRP material was considered as linear elastic isotropic until failure. In the present study, E_{FRP} = 82 GPa and ν_{FRP}

= 0.3 are the elastic modulus and Poisson's ratio of FRP material, respectively. Adhesive is also assumed isotropic with modulus of elasticity E_{Epoxy} = 2.78 GPa and a Poisson's ratio ν_{Epoxy} = 0.27.

To increase the flexural loading capacity and stiffness of timber beams, 6 mm FRP composites will be sprayed on the tensile soffit of timber beams. Fig. 5 shows the base shear-roof displacement curves for both original and the retrofitted timber frame. As shown in Fig. 5, the sprayed FRP strengthening of the beams resulted in a 18% increase in the lateral load carrying capacity of the original timber frame. It can also be seen that the sprayed FRP retrofitted timber frame had a large displacement capacity without exhibiting any loss of strength compared to the original timber frame.

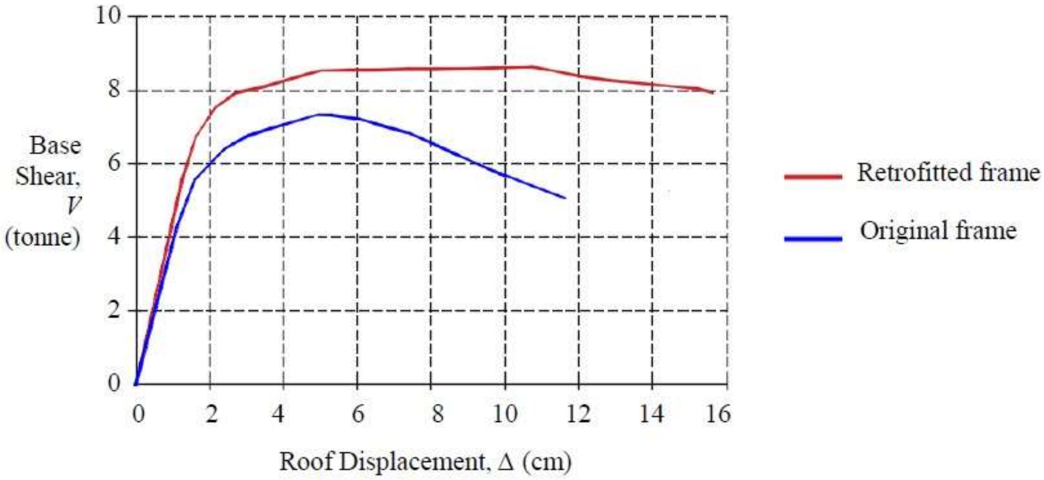


Fig. 5 Base shear – roof displacement curves of original timber and retrofitted timber frame

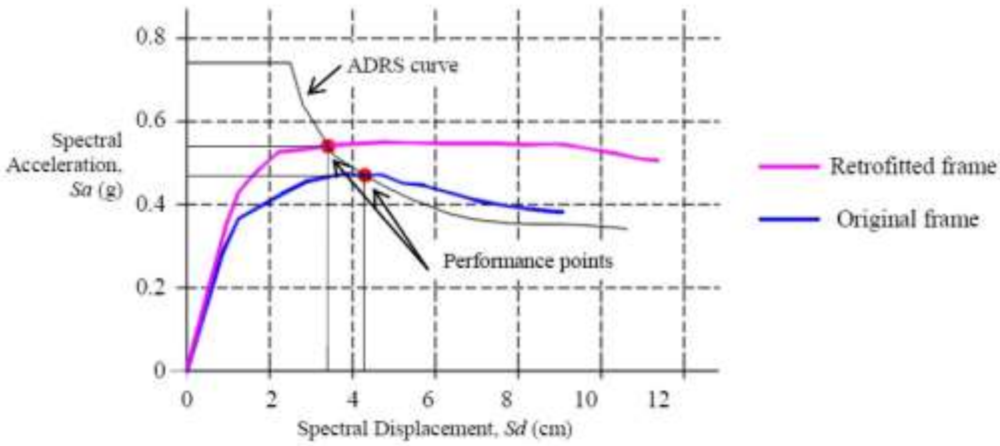
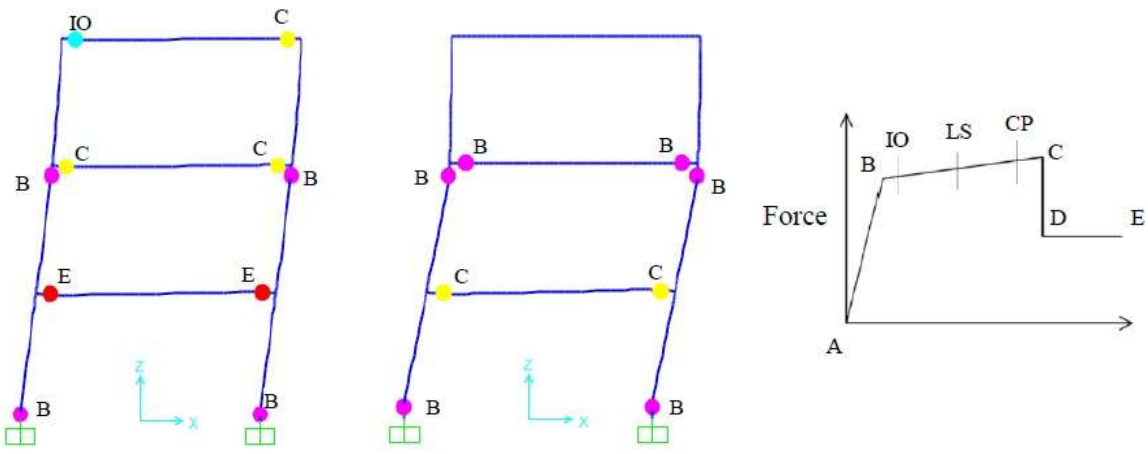


Fig. 6 ADRS curve and the performance level of original and SFRP retrofitted frame based on ATC40

The capacity curve, which is obtained from the pushover analysis, has to be converted into an equivalent bi-linear curve. Fig. 6 shows the capacity ADRS curves and the performance points of the retrofitted and original timber frame using the instructions provided by ATC-40 (Comartin *et al.* 1996). The results of the retrofitted frame compared with original frame shows 20.8% decrease in spectral displacement. According to Fig. 6, the performance point of the original frame has a displacement of 4.30 cm, while in the retrofitted frame the performance point has a spectral displacement of 3.40 cm. The spectral displacement reduction of the sprayed FRP retrofitted frame indicates that the inelastic lateral load resistance has been enhanced through sprayed FRP retrofitting. In addition, the spectral acceleration value has been increased from 0.468 g to 0.54 g, indicating that there has been an increase in the seismic load capacity for the SFRP retrofitted frame.

Figs. 7(a) and 7(b) show the plastic hinge distributions and their performance levels for the original and FRP retrofitted timber frames at the target displacement point, respectively. Pushover analysis indicated that in the plain timber frame (Fig. 7(a)), the performance level of the beam at the first floor reached collapse during the design earthquake load. However, after retrofitting the beams by sprayed FRP, the plastic hinges remained in ultimate capacity (*C*) (Fig. 7(b)). In addition, for the original timber frame (Fig. 7(a)) the value of the plastic hinge rotations in the beam of the second floor indicate that the these hinges laid in ultimate capacity on the performance curve; while the plastic hinge rotations for the retrofitted frame (Fig. 7(b)) is in the linear behaviour (*B*) range; where point *B* is the yield point. These consequences illustrate that the values of the plastic rotations in the beams of retrofitted frame have decreased with the result that the inelastic lateral load carrying capacity has been enhanced when the frame is retrofitted with SFRP.



(a) original timber (b) timber frame reinforced with sprayed-FRP (c) the performance level of the Building

Fig. 7 Pushover analysis of the timber frame

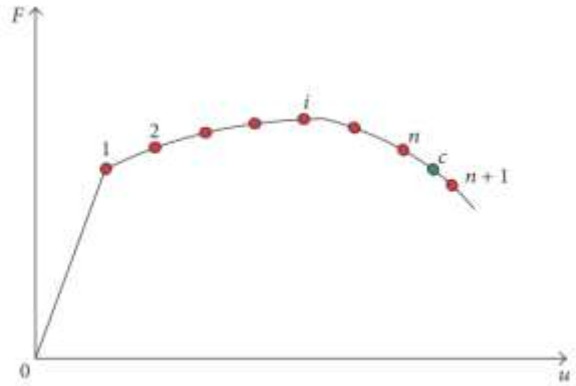


Fig. 8 Multiple linear force-deformation curve (He *et al*. 2013)

As can be seen in Fig. 7(c) (force-displacement capacity curve), the lateral force is applied at the deformed state of the general loading from point *A*. The structure will remain in linear behaviour until point *B* is reached, and therefore, no hinges will be formed before this point. However, after point *B* one or more hinges will start to form. The ultimate capacity and residual strength of the structure will be reached at points *C* and *D*, respectively. Finally, complete failure of structure will occur at point *E*. The notations of *IO*, *LS* and *CP* stand for Immediate Occupancy, Life Safety and Collapse Prevention, respectively.

5. Damage assessment of the retrofitted and original frames

The damage index is an indicator describing the state of the lateral load-carrying capacity and the reserve capacity of existing structures. Therefore, the study on damage index is necessary. Pushover analysis, non-linear time history analysis, and vulnerability analysis are some of the most common techniques and approaches for damage analysis of structures. The most acceptable damage index, D_{PA}, is the Park *et al.* (1984) damage index combining both ductility and cumulative hysteretic energy demand (He *et al*. 2013, Van Cao and Ronagh 2014)

$$\mathrm{D_{PA}} = \frac{\delta_m}{\delta_u} + \frac{\beta}{\delta_u F_y} \int dE_h \tag{7}$$

where δ_m and δ_u are the maximum experienced deformation and the ultimate deformation of the element, respectively. F_y is the yield strength of the element, $\int dE_h$ is the hysteretic energy absorbed by the element during the response history, and β is the model constant, which was suggested to be 0.1 for nominal strength deterioration. It is worth noting that the maximum damage index obtained using Park *et al*.'s (1984) is greater than 1 and nearly close to 2 in some cases. He *et al*. (2013) believed that Park *et al*.'s (1984) damage index is not an appropriate theory for nonlinear static pushover analysis or normal capacity spectrum method since the cumulative damage does not occur in this case. Therefore, a new and comprehensive damage index needs to be proposed taking into account cumulative effect. To solve this issue, a general global damage index, D_c, for capacity curve represented by He *et al*. (2013), see Eq. (8). The value of damage index, using the following equation, is typically between 0 and 1, where zero illustrates undamaged state while 1 represents the collapse state of the building

$$\mathrm{D_c} = 1 - \frac{\sum_{i=0}^{n-1} k_i (u_{i+1} - u_i) + k_n (u_c - u_n)}{k_0 u_c} \tag{8}$$

in which, k, k_0, u, u_n and u_c are the slope of the base shear in a specific point, the initial stiffness of elastic stage, displacement in a specific point, displacement at *n*th point and current displacement, respectively (see Fig. 8). In Eq. (8), the damage index varies from 0 to 1 and can be calculated based on only one capacity curve (only one pushover analysis). The value obtained using Eq. (8) is a suitable damage index for the capacity curve.

Van Cao and Ronagh (2014) have categorised the damage index in the following four groups: light, moderate, severe and collapse; where the damage indices are in the range from 0–0.25, 0.26–0.50, 0.51–0.75 and 0.76 to 1.00, respectively. In this study, the damage analyses are conducted for the original and retrofitted frames using Eq. (8). The results of damage analyses showed that the damage index of the retrofitted frame is reduced. The damage analysis of the original frame has a damage index of 0.79 indicating that the plain frame remains in the collapse state, whilst in the retrofitted frame the damage index is 0.69. This analysis illustrated that the SFRP technique reduces damage index and as such makes a positive change on the damage states. Therefore, SFRP retrofitting method can be recommended for upgrading deficient structures.

6. Conclusions

The key objective of the present study was to investigate the performance point of a three-storey timber frames subjected to lateral loads with and without SFRP reinforcement. The model timber frame was examined previously using ‘jointed ductile post-tensioning technology’. In this paper, the flexural stiffness of the sprayed FRP retrofitted beams was implemented in the analytical model of the retrofitted frame to carry out a preliminarily nonlinear static and pushover analyses of the frame. Using the results of these analyses, the following conclusions can be made:

- SFRP retrofitted timber frames resulted in an 18% increase in the lateral load carrying capacity of the original timber frame.
- The retrofitted frame showed a 20.8% decrease in spectral displacement compared with the original frame. This reduction demonstrates that the inelastic lateral load resistance has been improved through sprayed FRP retrofitting.
- A notable improvement in spectral acceleration was also achieved when the frame is retrofitted with sprayed FRP.
- Pushover analysis also indicated that in the original frame the performance level of the beam at the first floor reached to collapse during the design earthquake load. However, after retrofitting the beams with SFRP, the plastic hinge remained in ultimate capacity.
- The results of damage analysis showed that the damage index after retrofitting reduced by 0.10, which has a positive change on the damage states.

More research is ongoing in order to assess the exact performance and damage levels of the original and the SFRP-retrofitted building frames, including the ductile fasteners, as well as the post-tensioning effects.

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