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PERFORMANCE-BASED SEISMIC RETROFIT DESIGN FOR RC FRAMES USING FRP COMPOSITE MATERIALS

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Keywords	Abstract
FRP Retrofitting, Pushover Analysis, Seismic Performance, RC Frames, Confinement.	Seismic retrofitting of reinforced concrete (RC) members using fiber reinforced polymer (FRP) composite materials has become a well-established technique for repairing and strengthening seismically deficient systems. Although there has been significant progress on behavioral modeling of FRP retrofitted RC members as well as on implementation of performance-based analysis and design concepts, the studies investigating the overall seismic behavior of the retrofitted frames are still limited. This paper presents a methodology for performance-based FRP retrofit design for RC frames and provides an analytical investigation of this through numerical studies performed on 2-D frame models with different retrofit configurations. More specifically, frame systems with square or rectangular columns were assumed to be retrofitted through FRP strengthening of beams for improved flexural capacity and/or wrapping of columns for additional confinement and resulting ductility. Nonlinear pushover analyses of frames before and after retrofitting were performed using nonlinear hinge parameters determined from moment-curvature analyses based on comparative use of various steel- and FRP-confined concrete models proposed in the literature. Analysis results reveal the expected contribution of each retrofit configuration to the seismic behavior and performance of the frames, highlighting various design considerations for proper FRP retrofit decisions.

FRP KOMPOZİT MALZEMELER İLE GÜÇLENDİRİLMİŞ BETONARME ÇERÇEVELERİN PERFORMANSA DAYALI SİSMİK RETROFİT TASARIMI

Anahtar Kelimeler	Öz
FRP Güçlendirme, İtme Analizi, Deprem Performansı, Betonarme Çerçeveler, Sargılama.	Fiber takviyeli polimer (FRP) kompozit malzemelerin, betonarme (RC) elemanların sismik olarak güçlendirilmesi, sismik açıdan yetersiz sistemlerin onarılması ve güçlendirilmesi amacıyla kullanımı köklü bir teknik haline gelmiştir. FRP ile güçlendirilen betonarme elemanların davranışının modellenmesi ve performansa dayalı analiz ve tasarım kavramlarının uygulanması konusunda son yıllarda önemli ilerlemeler kaydedilmesine rağmen, güçlendirilmiş çerçevelerin genel sismik davranışını araştıran çalışmalar hala sınırlıdır. Bu makale, betonarme çerçeveler için performansa dayalı FRP güçlendirme tasarımına yönelik bir metodoloji sunmakta ve farklı güçlendirme konfigürasyonlarına sahip iki boyutlu çerçeve modelleri üzerinde gerçekleştirilen sayısal çalışmalar aracılığıyla bunun analitik bir incelemesini sunmaktadır. Bu kapsamda, kare veya dikdörtgen kolonlu çerçeve sistemlerin eğilme kapasitesini arttırmak amacıyla kirişlerin, ilave sargılama ve süneklik için ise kolonların sarılması şeklinde FRP ile güçlendirilmesi ele alınmıştır. Çerçevelerin güçlendirme öncesi ve sonrası doğrusal olmayan itme analizleri, literatürde önerilen çeşitli çelik sargılı ve FRP sargılı beton modellerinin kullanımına dayalı moment-eğrilik analizlerinden belirlenen doğrusal olmayan mafsal parametreleri kullanılarak karşılaştırmalı olarak gerçekleştirilmiştir. Analiz sonuçları ışığında her bir güçlendirme konfigürasyonunun çerçevelerin sismik davranışına ve performansına katkısı ortaya konularak FRP ile güçlendirme kararlarına kılavuzluk yapacak çeşitli tasarım hususları öne çıkarılmıştır.

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Highlights

- A practical methodology for FRP retrofit design for RC frame structures is proposed.
- Performance-based evaluation and design concepts are incorporated into the proposed methodology.
- Different material models are implemented in the numerical analyses for comparison.
- Influence of various geometric and retrofit parameters on the structural performance is investigated.

Purpose and Scope

The primary objective of the research presented in this paper was to investigate the lateral load behavior and performance of RC frames with FRP retrofitted beams and/or columns and a comparative evaluation of the influence of various geometric, material, and retrofit parameters on the behavior and performance. The scope was limited to RC frames with no shear walls and the FRP retrofitting was limited to column and beam elements.

Design/methodology/approach

Following an evolutionary research approach, nonlinear static pushover analyses are carried out for bare frames to establish a reference baseline. The moment-rotation relations for the plastic hinges at the column ends are obtained using four different unconfined concrete models and three different steel models for comparison. Resulting custom hinge properties are fed into a commercially available structural analysis program for pushover analyses and the resulting frame capacity curves were compared with those obtained using the recommended hinge properties by FEMA-356. In the following stage, frames having square or rectangular columns with widely spaced transverse reinforcement were assumed to be retrofitted through GFRP wrapping of the columns for additional confinement. Using three different FRP-confined concrete models proposed in the literature the moment-rotation relations and the corresponding capacity curves for the frames are obtained. In the last stage, both columns and beams of frames are assumed to be FRP retrofitted and the corresponding capacity curves as well as the resulting performance are compared with those obtained for the case of retrofitting only the columns. Based on this comparison, an assessment of the benefits and costs associated with retrofitting beams in addition to columns was made from seismic performance viewpoint.

Findings

FRP retrofitting of RC frames by wrapping of columns for additional transverse reinforcement can significantly enhance the lateral deformation capacity and hence the ductility of the structure without significant increase in its lateral load capacity. Retrofitting of beams in addition to columns for improved flexural and shear capacity can enhance both the lateral load and deformation capacity of the retrofitted frame, although the increase in deformation capacity is limited compared to the configuration in which only columns are retrofitted. Both retrofit configurations can be used to obtain a performance point. Use of different reinforcing steel and unconfined concrete models in the moment-curvature analyses is found to result in negligible differences in the resulting curves. The three steel-confined concrete models implemented in the analyses resulted in moderately close capacity curves for bare frames. The three FRP-confined concrete models implemented in the analyses produced similar capacity curves in most cases despite their significantly different characteristics. Comparative investigations have revealed that the recommended hinge properties by FEMA-356 should be used with caution in the analyses of bare frames.

Research limitations/implications

In the numerical model, the joints were assumed to have sufficient shear capacity due to internal reinforcement or external strengthening so that plastic hinges would form outside the joint regions. Also, potential debonding problems in retrofitted beams were ignored, assuming proper anchorage of the flexural FRP reinforcement.

Originality

It provides a comparative evaluation of the influence of various geometric, material, and retrofit parameters on the behavior and seismic performance of the frame systems integrating the performance-based design and evaluation concepts that can be utilized for assessing the FRP retrofit design options for existing structures.

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1. Introduction

Fiber reinforced polymer (FRP), made of high strength-to-weight ratio fibers and binding resins, is an effective method for strengthening reinforced concrete (RC) structures. Compared to traditional techniques, using FRP reduces the additional dead load on a retrofitted member. It is non-corrosive, flexible, and can be molded into any shape or configuration. FRP achieves strength quickly and requires minimal setting time. Its flexibility makes it ideal for reinforcing structural elements like beams, columns, and slabs without requiring sophisticated equipment. They can be rapidly applied in the form of FRP wraps without disrupting a building's occupancy, which is a significant advantage as it minimizes downtime for businesses and the need to relocate residents (Bousselham 2010).

Use of FRP composite materials to enhance stiffness, load capacity, and ductility characteristics of structural members has been a popular area of research in the past few decades beginning with the pioneering studies led by Meier (Meier and Kaiser 1991; Meier 1992). Since then, various techniques have been developed to integrate them effectively into structural elements to improve performance of the structural systems. Significant progress has been achieved in this area allowing their utilization in various strengthening applications to increase the ductility of columns, flexural and shear capacity of beams and slabs, and in- and out-of-plane resistance of walls (Buyukozturk et al., 1999; Triantafillou 2001; Teng et al., 2003; Motavalli and Czaderski 2007; Pendhari et al., 2008; Danraka et al., 2017; Durgadevi et al., 2021; Ghosh et al., 2024).

Extensive research on investigation of retrofitting technique on RC frame elements including beams (Pham et al., 2004; Li et al., 2006; Ferreira and Barros 2006; Rougier and Luccioni 2007; Bournas et al., 2007; Bousias et al., 2007; Sheikh and Li 2007; Fu et al., 2018; Kadhim et al., 2019), columns (Binici and Mosalam 2007; Colomb et al., 2008; Promis et al., 2009; De Luca et al., 2011; del Ray Castillo et al., 2018; Fosetti et al., 2018), and joints (Alhaddad et al., 2012; Mahini and Ronagh 2010; Bousselham 2010; Polies et al., 2010; Alsayed et al., 2010; Del Vecchio et al., 2016; Pohoryles et al., 2019).

Specifically, shear strengthening beams with FRP U-wraps perpendicular to the beam axis (Akguzel and Pampanin 2012a; Alsayed et al., 2010; Antonopoulos and Triantafillou 2003; Engindeniz et al., 2008) has been shown to significantly enhance the shear capacity and structural integrity of the beams, effectively prevent shear failure, and improve overall performance under loading conditions. Using FRP jackets where the fibers are oriented only or predominantly in the hoop direction to confine the concrete has been reported to significantly enhance both its compressive strength and ultimate compressive strain (Al-Salloum and Almusallam 2007; Del Vecchio et al. 2014). Wrapping FRP sheets fully around the column for shear strengthening has proven to be quite effective (Lee et al., 2010). Column flexural strengthening using straight FRP sheets aligned with the column axis (Antonopoulos and Triantafillou 2003), L-shaped FRP (Akguzel and Pampanin 2012a; Garcia et al., 2014; Yu et al., 2016), near-surface-mounted FRP (Hasan et al., 2016; Prota et al., 2004), and FRP anchors (Shiohara et al., 2009) have been explored for their performance in preventing undesired column hinging failure. Placing FRP sheets strategically across unobstructed joint panels in various orientations, including horizontal, vertical, or diagonal, has been shown to enhance the shear strength of the beam-column joint without the need for extensive construction work (El-Amoury and Ghobarah 2002, Le-Trung et al., 2010, D'Ayala et al., 2003, Alhaddad et al., 2012; Mahini and Ronagh 2010; Bousselham 2010; Polies et al., 2010; Alsayed et al., 2010; Del Vecchio et al., 2016; Pohoryles et al., 2019). However, most of these studies focus on the local element/component level with applications primarily limited to the behavior and mechanics of FRP-retrofitted subassemblies and beam-column joints rather than the global behavior of the retrofitted structural system.

Numerous experimental studies have been conducted to evaluate the effects of FRP retrofitting in upgrading deficient RC structures. Balsamo et al. (2005) tested a full-scale RC dual frame system strengthened using carbon FRP (CFRP) laminates through pseudo-dynamic loading. Despite the failure problems encountered at the column/wall-footing interfaces due to termination of laminates at the bottom of columns/walls, the study confirmed the potential of the method and resulted in increased load, deformation and energy dissipation capacity after retrofitting. Pampanin et al. (2007) tested a 2/3 scale three story three-bay 2-D frame system as well as six 2-D beam-column joint subassemblies under quasi-static reversed loading. Di Ludovico et al. (2008a, b) tested a full-scale three-story frame under bi-directional pseudo-dynamic loading corresponding to different peak ground accelerations (PGA). The columns and the joints of the frame were retrofitted using glass FRP (GFRP) composites and the test results corresponding to as built and FRP retrofitted configurations were provided for both directions. Garcia et al. (2010) tested a full-scale two-story RC frame with poor reinforcement detailing in the beam-column joints on a shake table leading to damage at the column ends and joint regions. The damaged frame strengthened using CFRPs was retested with varying levels of PGA. These experimental studies demonstrated the effectiveness of FRP confinement in changing the plastic hinge mechanism from column-sway to beam-sway, in line with the principles of modern seismic design. The experimental investigation carried out by Shin et al. (2016) on a full-scale two-story non-ductile RC frame that was retrofitted with FRP jacketing system on the first story columns demonstrated the effectiveness of the retrofitting method to prevent soft-story mechanism. Saqan et al. (2018) conducted an experimental program to evaluate the performance of externally bonded (EB) CFRP fabric and near-surface mounted (NSM) CFRP bars for seismic strengthening of RC frame members. The three different test specimens; the control specimen with no strengthening, the strengthened specimen with EB CFRP sheets and the strengthened specimen with NSM CFRP bars were subjected to cyclic load tests in a displacement-controlled manner to compare the performance of these assemblies. The results showed that both strengthening strategies improved the performance of the system in terms of strength, total energy dissipation capacity as well as stiffness degradation.

On the analytical aspect of the investigations, initial studies involved nonlinear modeling of the bare and retrofitted frame using Drain-3DX (Prakash et al., 1994) that are calibrated using experimental data from shake table tests. Zou et al. (2007) studied optimal design of FRP column jackets for seismic retrofitting of frames. Limiting the retrofit action to column jacketing and setting the thicknesses of FRP jackets as the design variables, they applied an optimization technique to minimize the use of FRP materials while satisfying the constraint on the inter-story drift ratio as the performance objective. The merit of the developed procedure is offset by its impracticality for the practicing engineer, its limitation to column jacketing, and perhaps above all, the question of whether FRP jacketing of columns in a frame structure is a real optimization problem considering the quick convergence of design iterations. Galal and El-Sokkary (2008) performed nonlinear dynamic analyses of a 5-storey low-rise, a 10-storey mid-rise, and a 15-storey high-rise RC frame assumed to be retrofitted in various configurations using FRP composites and subjected to a set of scaled ground motion records. Analyses and evaluations concluded that for low-rise frames retrofitting columns was sufficient to increase performance while for mid-rise and high-rise frames retrofitting columns as well as beams was found to be more effective. Retrofit application along half the building height was found to be inefficient, just as retrofitting only damaged elements or only a selected group of elements rather than all elements in the frame structure. Choi et al. (2014) proposed a seismic retrofitting method that uses FRP wraps for shear-critical RC frames. Cao and Pham (2020) presented a retrofitting approach based on expected seismic damage distribution and that selectively applies the FRPs to the plastic hinge locations. This approach minimizes the cost of the intervention and the related service interruption.

Accumulated literature on the performance-based evaluation and design of FRP retrofitted RC frames reveals that the methodology, approach, and the modeling tools show much variation. Although extensive research efforts have been dedicated on investigating the feasibility and effectiveness of the method for frame structures by combining various FRP retrofit applications and configurations tested under various loading conditions, their verification through modeling pose challenging issues especially for practicing engineers who are charged with the task of evaluating the FRP retrofit design option for existing frame buildings.

The primary objective of the research presented in this paper was to investigate the lateral load behavior and performance of RC frames with FRP retrofitted beams and/or columns and a comparative evaluation of the influence of various geometric, material, and retrofit parameters on the behavior and performance. The work benefited from the existing experimental and modeling research on strengthening beam and column elements using FRPs, and combined this knowledge with performance-based evaluation and design concepts. The scope was limited to RC frames with no shear walls and the FRP retrofitting was limited to column and beam elements. The joints were assumed to have sufficient shear capacity, due to internal reinforcement or external strengthening so that the plastic hinges would form outside the joint regions. Also, potential debonding problems in retrofitted beams were ignored, assuming proper anchorage of the flexural FRP reinforcement.

Following an evolutionary research approach (Tumer, 2006), initial studies involved nonlinear static pushover analyses of bare frames having square or rectangular columns with closely or widely spaced transverse reinforcement to establish a baseline for the following comparisons. The moment-rotation relations for the plastic hinges at the column ends were obtained from moment-curvature analyses using four different unconfined concrete models and three different steel models for comparison. Resulting custom hinge properties were fed into a commercially available structural analysis program for pushover analyses and the resulting frame capacity curves were compared with those obtained using the recommended hinge properties by FEMA-356 (FEMA 2000), hereafter referred to as the default hinge properties. In the following stage, frames having square or rectangular columns with widely spaced transverse reinforcement were assumed to be retrofitted through GFRP wrapping of the columns for additional confinement. The moment-rotation relations were obtained from moment-curvature analyses using three different FRP-confined concrete models proposed in the literature for comparison and the corresponding capacity curves for the frames were obtained from pushover analyses. In the last stage, both columns and beams of frames with square columns were assumed to be FRP retrofitted and the corresponding capacity curves as well as the resulting performance were compared with those obtained for retrofitting only the columns. Based on this comparison, an assessment of the benefits and costs associated with retrofitting beams in addition to columns was made from seismic performance viewpoint.

2. Performance-Based FRP Retrofit Design Methodology

The methodology implemented in this research for performance-based design of FRP retrofitted RC frames is illustrated in Figure 1. As can be seen from the figure, the methodology essentially involves altering the nonlinear hinge properties at the ends of the retrofitted members by determining the force-deformation relations in the form of moment-rotation curves, followed by pushover analysis of the retrofitted frame using the determined hinge properties, and subsequent performance evaluation using the capacity spectrum method (ATC 1996; FEMA 2005). The moment-rotation curves for the bare and retrofitted members were obtained from moment-curvature analyses of the respective sections using various unconfined, steel-confined, and FRP-confined concrete models. The iterative procedure for moment-curvature analysis can be found in most textbooks on mechanics of reinforced concrete (Wight and MacGregor 2009). Figure 2 illustrates the strain compatibility and force equilibrium for bare and FRP retrofitted column sections, highlighting the regions that require proper material models for moment-curvature analysis. The whole section is confined in the FRP retrofitted section, the effect of which is added to the effective lateral confining pressure from the transverse reinforcement on the core concrete (Mander et al., 1988; Maalej et al., 2003).

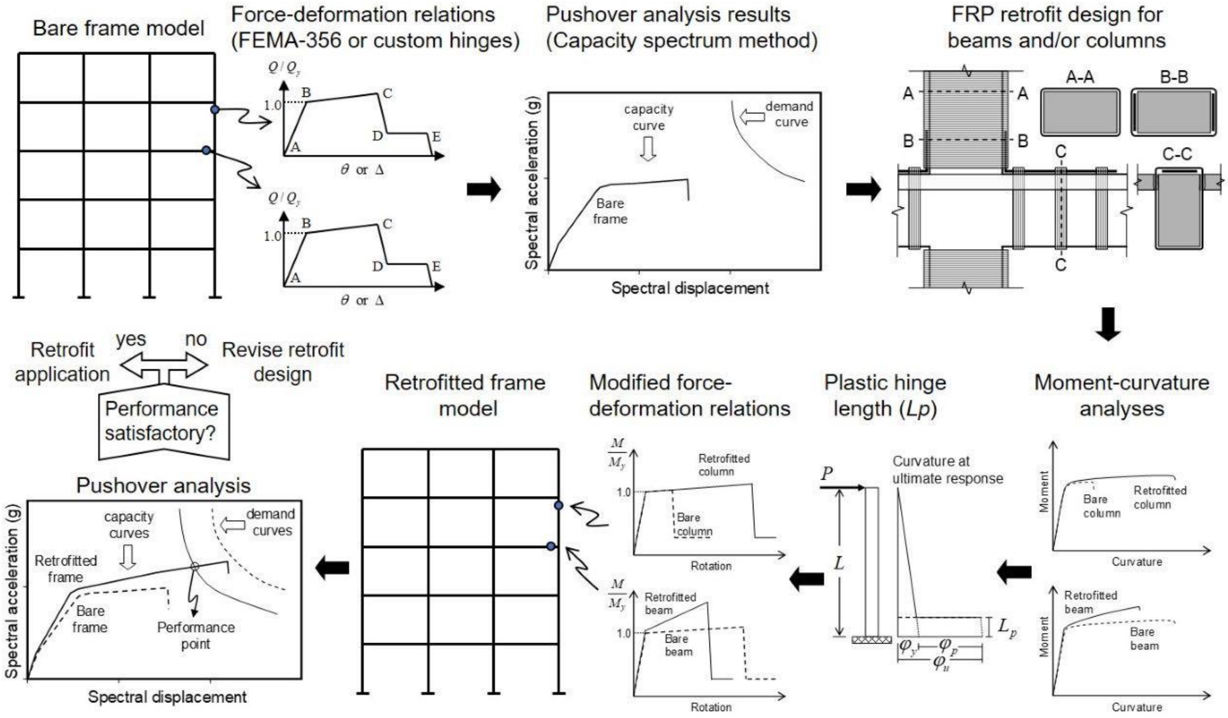
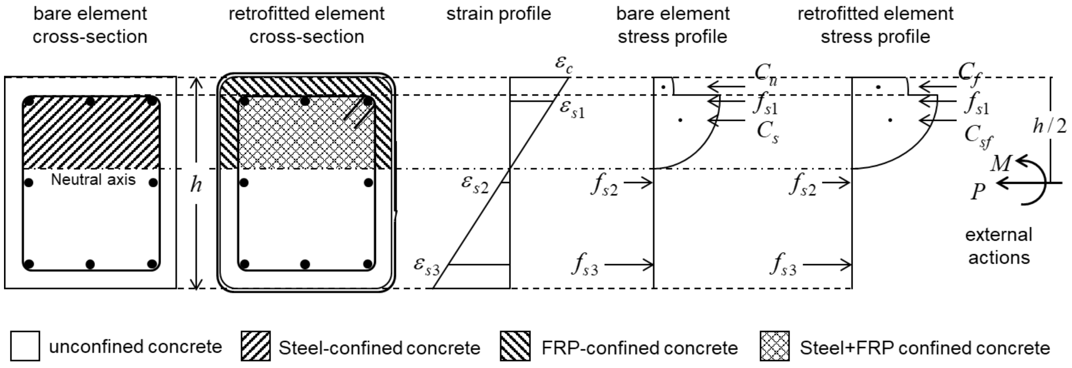


Figure 1. The methodology implemented for performance-based FRP retrofit design for RC frames



(ε_c: Strain at the top fiber of concrete, ε_{s1}, ε_{s2}, ε_{s3}: Strains in the top, middle and bottom reinforcement)
(f_{s1}, f_{s2}, f_{s3}: Stresses in the top, middle and bottom reinforcement)
(C_u, C_s: Compressive force on unconfined and steel confined concrete sections)
(C_s, C_{sf}: Compressive force on FRP confined and FRP+steel confined concrete sections)

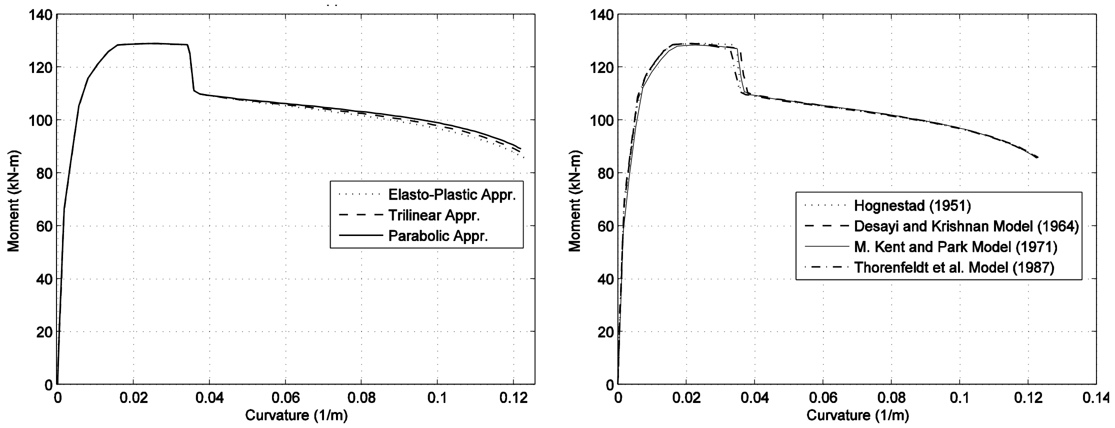
Figure 2. Moment-curvature analysis of bare and FRP retrofitted element sections

2.1. Material Models

Table 1 provides a list of the material models implemented for comparative evaluation of the results produced by their use in the analysis of frame elements and systems. Initial analysis studies compared use of different unconfined concrete and steel models for moment-curvature analysis of columns. Figure 3 shows the resulting curves for a 300x300 mm^2 column with a compressive strength of 16 MPa for concrete and a yield strength of 220 MPa for steel, under an axial load of 500 kN. The curves clearly depict the nonlinear behavior of the section, including the sudden drop in moment capacity upon spalling of the unconfined concrete cover. As can be seen from the figure, the choice of material model for steel or unconfined concrete makes no significant difference in the moment-curvature response of the section. For this reason, the simple elastoplastic material model was used for steel and the Hognestad (1951) model was used for unconfined concrete in the remaining analyses.

Table 1. Material models implemented in moment-curvature

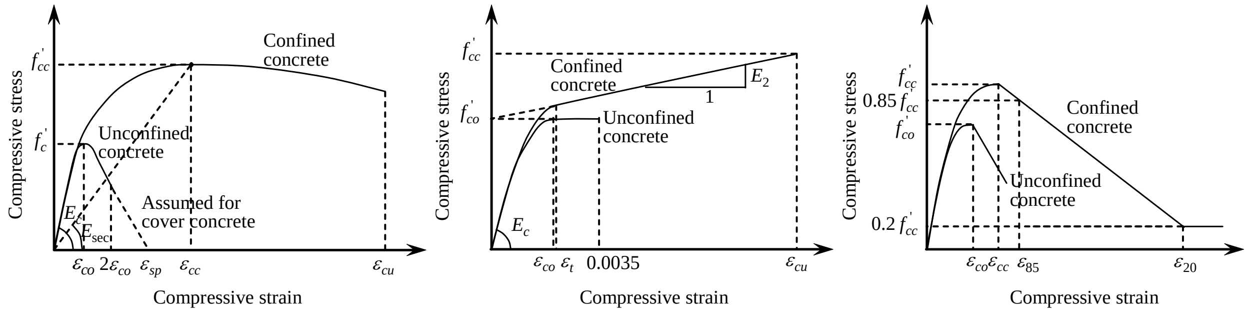
Unconfined Concrete Models	**Steel-Confined Concrete Models**	**FRP-Confined Concrete Models**	**Steel Models** (Priestley et al., 1996)
Hognestad (1951) Model	Mander et al. (1988) Model	Mander et al. (1988) Model	Elasto-Plastic approximation
Thorenfeldt et al. (1987) Model	Modified Kent and Park (Park et al, 1982) Model	Lam and Teng (2003) Model	Trilinear approximation
Desayi and Krishnan (1964) Model	Saatcioglu and Razvi (1992) Model	Maalej et al. (2003) Model	Parabolic approximation
Kent and Park (1971) Model			



(a) (b)

Figure 3. Influence of (a) different steel material models and (b) unconfined concrete models on the moment-curvature response of a 30x30 cm^2 column under 500 kN axial load (f'_c=16 MPa, f_y=220 MPa)

The FRP-confined concrete models listed in Table 1 are illustrated in Figure 4. These characteristically different models are either based on previously developed steel-confined concrete models – with modifications that account for the behavior and material properties of FRPs – or based on experimental results compiled from the literature. The model by Mander et al. (1988) was originally developed for steel-confined concrete but the definitions of the ultimate compressive strain of concrete (ε_{cu}) and the volumetric ratio of confinement (ρ_s) were later modified to adapt the model to FRP-confined concrete (Priestley et al. 1996). The model describes the stress-strain behavior of confined concrete by a single expression and resorts to an energy balance approach instead of strain compatibility to estimate the ultimate compressive strain. The model by Lam and Teng (2003) was based on the experimental results reported in the literature. The model is formed by a parabolic first portion which is smoothly connected to the linear second portion, and it reduces to the unconfined concrete model when the FRP reinforcement parameters are set to zero. Although easier to use, this model does not allow for the consideration of the combined confinement effects of steel and FRP reinforcement. The model proposed by Maalej et al. (2003) was originally intended for rectangular and wall-like (high aspect ratio) RC sections wrapped with FRPs. The model is based on the steel-confined concrete model by Saatcioglu and Razvi (1992) and is formed by a parabolic ascending and a linear descending branch. The model allows for the superposition of the confinement provided by the FRP reinforcement to that by the existing transverse steel reinforcement in the core concrete. All three models shown in Figure 4 relate the confinement in columns with rectangular cross-sections to equivalent circular section columns through transformations described in the respective references. Considering the significant difference in the development and shape of these confinement models, it is of practical and research interest to compare the results produced by their use in the analysis of bare and FRP retrofitted frames.



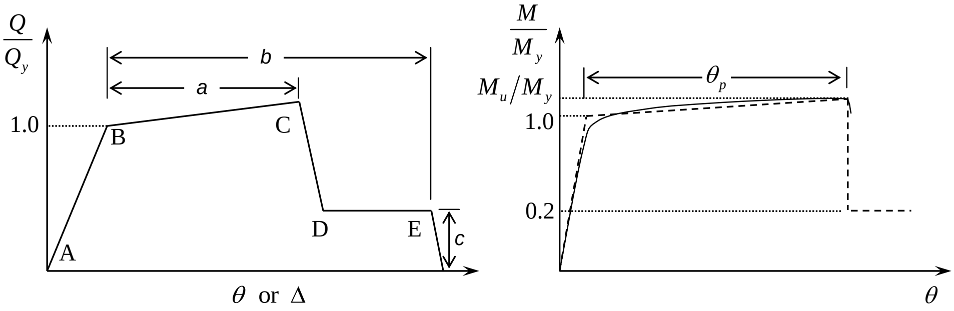
(a) (b) (c)

Figure 4. FRP-confined concrete models implemented in this research: (a) Mander et al. (1988) Model, (b) Lam and Teng (2003) Model, (c) Maalej et al. (2003) Model

2.2. Moment-Rotation Behavior

Estimating the nonlinear moment-rotation behavior at critical sections of the bare and retrofitted elements is a crucial component of the methodology illustrated in Figure 1. Following the moment-curvature analysis, the plastic curvature capacity, φ_p, is determined as the difference between the ultimate curvature, φ_u, corresponding to limiting compressive strain in concrete and the yield curvature, φ_y, corresponding to yield strain in the steel reinforcement. Plastic curvature is assumed to take place in the idealized plastic zone identified as the plastic hinge length, L_p. Several empirical expressions were proposed to estimate the plastic hinge length (Priestley et al. 1996; Nilson et al. 2005). The commonly used expression by Priestley et al. (1996) was used in this research. Once the plastic hinge length is known, the plastic rotation capacity can then be calculated by simply multiplying the plastic curvature capacity with the plastic hinge length.

Idealization of the moment-rotation behavior at plastic hinge regions of RC frame members is illustrated in Figure 5. The generalized force-deformation relation stipulated in FEMA-356 (FEMA 2000) is shown in the figure. The values of the deformations (or rotations) at the points B, C and D should ideally be derived from experiments or rational analysis, the latter of which was implemented in this research. For practical applications, FEMA-356 (FEMA 2000) provides sets of recommended values for a, b, and c based on the geometric, reinforcement, and loading characteristics of frame members. These recommended values, also called the default hinge parameters, are used by most structural analysis programs capable of performing nonlinear pushover analysis. In order to define custom hinge properties to model the behavior of bare or retrofitted frame members, it is essentially sufficient to determine the ratio of the moment at ultimate rotation to the yield moment, M_u/M_y, which may as well be less than one, and the plastic rotation capacity, θ_p, both of which are indicated in Figure 5. The yield moment and the corresponding elastic rotation can easily be computed by the program used, and the residual capacity is usually set to $0.2M_y$ as recommended by FEMA-356 (FEMA 2000) mainly for computational stability.



(a) (b)

Figure 5. Idealized moment-rotation relations for column/beam elements: (a) Generic force-deformation relation (FEMA 2000), (b) Idealized moment-rotation relation

3. Nonlinear Analyses of Bare and Retrofitted Frame Elements and Systems

The moment-rotation relations obtained from moment-curvature analyses of bare and retrofitted frame elements can be used to obtain the nonlinear behavior of respective frame structures through pushover analyses. Pushover analysis is a nonlinear static analysis procedure used to determine the nonlinear behavior of a frame structure based on those of the individual frame elements represented by nonlinear hinges defined on each element. These hinges represent the full force-deformation characteristics of the frame elements under individual or combined effects of flexure, axial load, and shear. The so-called capacity curve and the interstory drift ratios obtained from pushover analysis are compared with the seismic demand and the drift limits, respectively, to estimate the level of seismic safety. This comparison provides the designer with the estimated 'performance level' of the structure under seismic action, which is critical to any performance-based evaluation and design. The method was introduced in the Applied Technology Council publication ATC-40 (ATC 1996), discussed and used in FEMA-273 report (FEMA 1997) and FEMA-356 pre-standard (FEMA 2000), recently improved in FEMA-440 report (FEMA 2005), and included in the ASCE/SEI 41-06 standard (ASCE 2007).

A number of pushover analysis investigations were performed on two 2-D frame models with square and rectangular columns before and after FRP retrofitting of beams and/or columns in order to investigate their expected lateral load behavior and seismic performance as influenced by the use of different confined concrete models. Using elastoplastic model for steel, Hognestad (1951) model for unconfined concrete, and the steel and FRP confined concrete models listed in Table 1, moment-curvature analyses were performed using a program developed in MATLAB® environment (MathWorks, 2022). The plastic rotation capacities were calculated from the moment-curvature relations by means of the plastic hinge length and the M_u/M_y ratio (see figs. 1 and 5) to define custom hinge properties for pushover analyses to obtain the nonlinear behavior of the modeled frame. Separate shear hinges that include the contribution of the FRP reinforcement were also defined at member ends to consider potential brittle shear failures. SAP2000® structural analysis and design software was used for all pushover analyses (CSI 2020) and displacement controlled loading was applied in the form of the first mode shape of the frame.

3.1. Description of the Frame Models

A five story RC frame with no shear walls was used in the analyses with two different column configurations as shown in Figure 6. The frame represents a simplified model of a real-life building frame in Istanbul. The frame is 16.00 m tall with a typical floor to floor height of 3.20 m and column to column distance of 5.00 m. The dead and participating live loads were represented by a 50 kN/m uniform load on the beams. All beams have dimensions of 25x60 cm^2 with 10.3 cm^2 (4ϕ16+2ϕ12) top reinforcement and 8.04 cm^2 (4ϕ16) bottom reinforcement at the support regions. The reinforcement configurations for columns are shown in Figure 6, for which ϕ10 ties were assumed to be used as transverse reinforcement with 10 cm or 25 cm spacing. The frames with square and rectangular columns are hereafter referred to as Model I and Model II, respectively. These two model frames were carefully designed to have very similar stiffness and lateral load capacity so that direct comparison of the pushover analysis results could be made for the retrofitted frames.

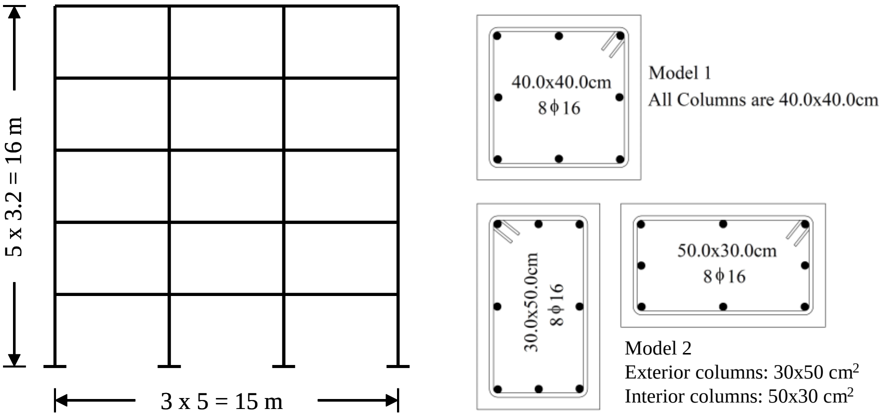


Figure 6. The frame models used in pushover analyses

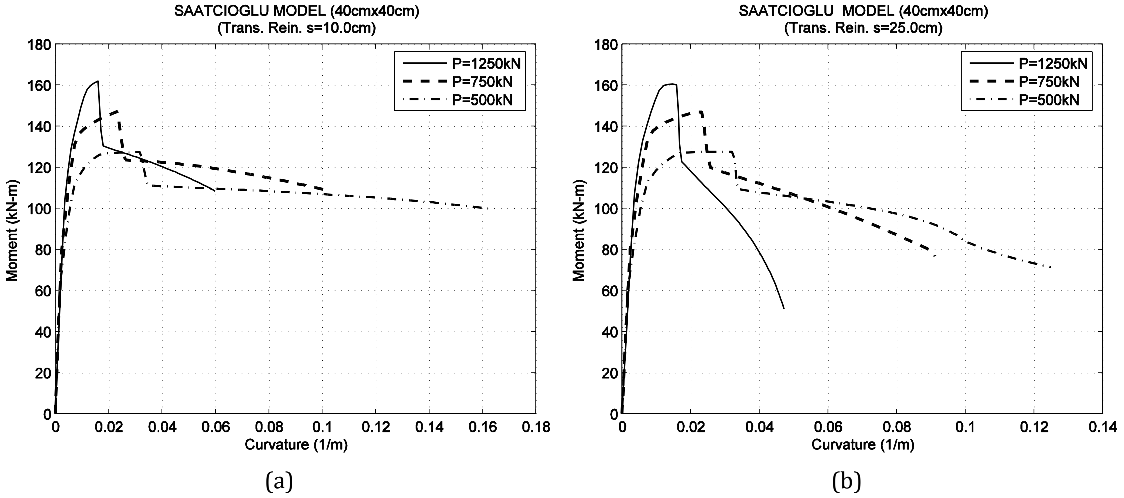
Material properties were taken as 16 MPa for the compressive strength of concrete and 220 MPa for the yield strength for both longitudinal and transverse reinforcement. These values are representative of the material properties for existing frame buildings built before the introduction of modern seismic codes in developing countries such as Turkey. The FRP material used for retrofitting was assumed to be Tyfo SEH-51A (Fyfe, 2008) glass FRP fabric. Columns were assumed to be retrofitted using three layers of FRP wrap while the beams were assumed to be retrofitted by a single layer of FRP sheet bonded to their top faces and anchored at column faces as illustrated in Figure 1.

3.2. Influence of Column Tie Spacing on the Frame Capacity Curve

Initial analysis studies on the frame models investigated the influence of column tie spacing on the behavior of frames under lateral loading in order to establish a baseline for the following comparisons of the analysis results for bare and FRP retrofitted frames and assessment of the effectiveness of FRP retrofitting. As the lateral reinforcement configuration in columns is implicitly considered in the nonlinear hinge properties, another important feature of this investigation is that it allows a comparison of the results obtained using custom hinge properties determined from moment-curvature analyses with those obtained using the default hinge properties based on FEMA-356 (FEMA 2000) recommendations.

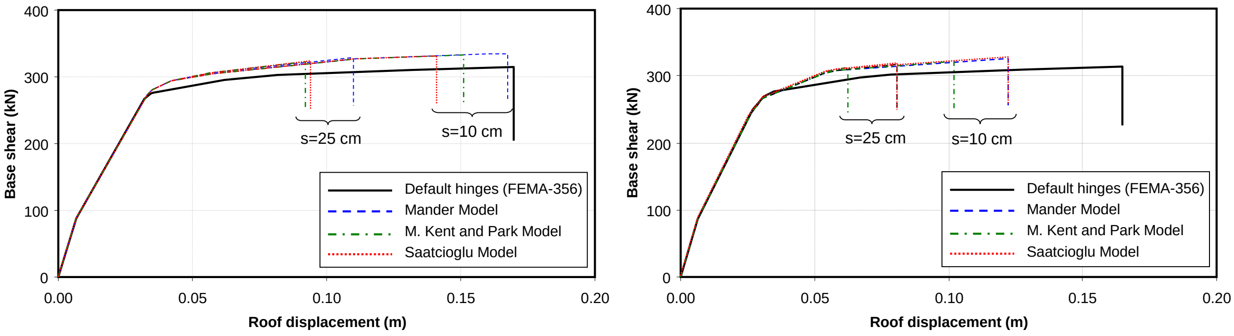
The moment-curvature relations obtained using Saatcioglu Model (Saatcioglu and Ravzi, 1992) for the square columns in Model I are shown for three hypothetical levels of axial loading in Figure 7; the column tie spacing was assumed to be 10 cm and 25 cm in (a) and (b), respectively. As expected, larger tie spacing results in reduced effective confinement and ductility. Quantitative representation of the member behavior in Figure 7 in the form of moment-rotation relation as illustrated in Figure 5 allows for quantitative assessment of the influence of column tie spacing on the frame behavior under lateral loading.

The capacity curves obtained from pushover analyses of both frames using three different steel-confined concrete models are shown in Figure 8. The base shear and roof displacement capacity values are also listed in Table 2. Real column axial forces calculated from SAP2000 analysis were used to obtain the moment-curvature relationship for each column. A comparison of the capacity curves reveals that all three confinement models result in approximately the same lateral load capacity for both frame models. This is expected since the variation in transverse reinforcement spacing affects the ductility of the columns rather than their lateral load carrying capacity. For Model I, closely spaced transverse reinforcement (s=10 cm) results in deformation capacities which are close to that obtained using default hinge properties, while large tie spacing (s=25 cm) results in a significantly less ductile behavior. This result shows that the nonlinear hinge properties recommended by FEMA-356 (FEMA 2000) assume proper lateral confinement by transverse reinforcement. Use of these hinge properties for frame elements with inadequate or questionable confinement conditions is likely to produce unconservative results in terms of ductility. For Model II, default hinge properties produce more ductile behavior compared to the custom defined hinge properties. The reasons for this behavior are: (1) The default hinge properties do not consider column orientation when specifying the rotation capacities; (2) The high level of load on the interior columns resulted in a less ductile rotation capacity compared to the default hinge properties. Based on these results, it can be concluded that default hinge properties may lead to unconservative estimation of the structural deformation capacity for rectangular columns with largely spaced ties or those under moderate to high axial loading. As opposed to an increase in the base shear capacity within 5%, reducing the tie spacing from s=25 cm to s=10 cm has remarkably increased the deformation capacity more than 50% for both frame models, which is significant in performance-based evaluation.



(a) (b)

Figure 7. Influence of tie spacing on the moment-curvature behavior of columns in Model I: (a) s=10cm, (b) s=25cm.



(a) (b)

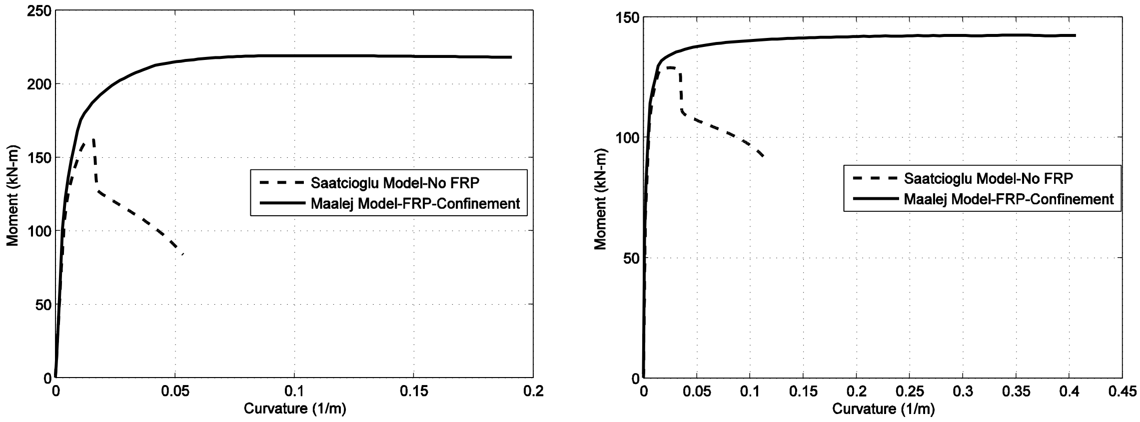
Figure 8. Influence of tie spacing on the frame capacity curve:(a) Model I (square columns), (b) Model II (rectangular columns)

Table 2. Comparison of the base shear and roof displacement capacities of the default hinge properties (FEMA-356) with those calculated using different steel-confined concrete models.

MODEL I (Square columns)				
	Base Shear		Roof Displacement	
Default Hinges (FEMA-356)	317.8		0.170	
	s=25 cm		s=10 cm	
	Base Shear	Roof Displacement	Base Shear	Roof Displacement
Mander Model	328.8	0.110	335.5	0.167
Kent & Park Model	320.1	0.094	333.3	0.151
Saatcioglu Model	324.4	0.092	331.1	0.141
MODEL II (Rectangular Columns)				
	Base Shear Capacity		Roof Displacement	
Default Hinges (FEMA-356)	313.3		0.165	
	s=25 cm		s=10 cm	
	Base Shear	Roof Displacement	Base Shear	Roof Displacement
Mander Model	317.8	0.075	326.7	0.122
Kent & Park Model	308.9	0.062	322.2	0.102
Saatcioglu Model	317.8	0.075	326.7	0.122

3.3. Influence of Column Retrofitting on the Capacity Curve

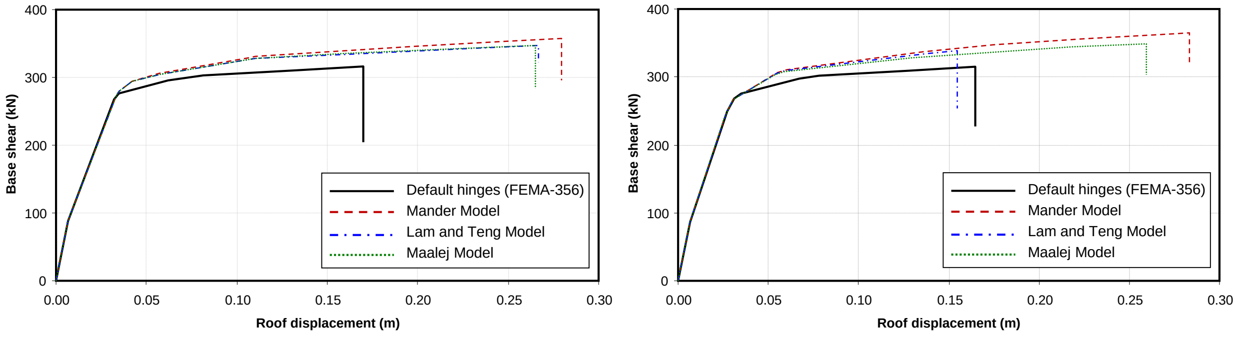
In this next stage, columns of both frame models were assumed to be retrofitted by three layers of GFRP composite wrap. A tie spacing of 25 cm was assumed for all columns. Figure 9 shows the moment-curvature relations for columns of Model I before and after FRP-retrofitting for two levels of column axial load. Saatcioglu (Saatcioglu and Ravzi, 1992) and Maalej et al. (2003) models were used for steel- and FRP-confined concrete, respectively. The figure shows that FRP wrapping of columns result in a significant improvement in the member ductility, which may be accompanied by a considerable increase in the moment capacity depending on the column axial load level.



(a) (b)

Figure 9. Moment-curvature relations for bare vs. FRP retrofitted columns for two axial load levels: (a)1250kN, (b) 500kN

The capacity curves obtained from pushover analyses of both frame models considering FRP retrofitted columns are shown in Figure 10 together with those obtained using the default hinge properties for comparison. As can be seen from the figure, improvements in the performance of FRP-retrofitted columns have increased the base shear capacity of both models around 10% while increasing their deformation capacity significantly around 60% except for one case. Figure 10(b) shows that for Lam and Teng (2003) model, the deformation capacity of the retrofitted frame falls below that obtained using default hinge properties only for Model II which has rectangular columns. This is because the model introduces a shape factor, k_s, which reduces the effective confinement by multiplying with the aspect-ratio of the column cross-section, (b/h). While this is a conservative approach, it reduces the effective confinement pressure by 40% for the rectangular columns used in Model II, resulting in a large difference in comparison with other models. The model produces results similar to others when k_s is ignored in the calculations.



(a) (b)

Figure 10. Influence of FRP-retrofitting of the columns on the capacity curve: (a) Model I (square columns), (b)Model II (rectangular columns)

3.4. Influence of Column+Beam Retrofitting on the Capacity Curve

In this stage the beams of both frame models were assumed to be retrofitted in addition to the columns using a single layer of FRP reinforcement bonded to the top faces of the beams to serve as longitudinal reinforcement. A separate program developed by Gunes (2004) was used to obtain the moment-curvature relation shown in Figure 11. As can be seen in the figure, the behavior of beams retrofitted in flexure using longitudinal FRP reinforcement is fundamentally different than that of columns retrofitted using transverse FRP reinforcement for additional confinement. As opposed to an increase in the moment capacity, the ductility of the beam is typically reduced. While additional transverse FRP reinforcement can be used to increase the shear capacity of the beam to prevent brittle shear failures, the additional confinement contribution of transverse reinforcement in beams is generally ignored in the analyses.

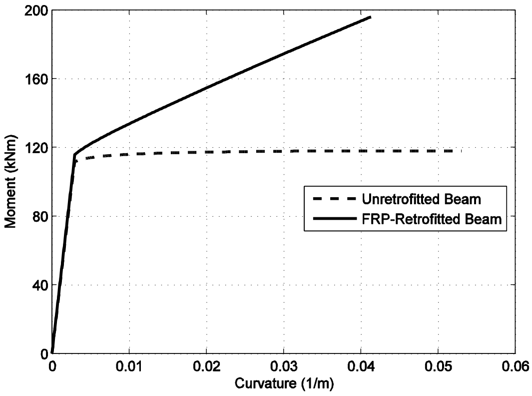


Figure 11. Moment-curvature relations for the beams before and after retrofitting

The capacity curves obtained from pushover analyses of both frame models before and after retrofitting the beam and/or column elements are shown in Figure 12. Only Maalej (2003) Model was used for the retrofitted columns in this case due to its modeling capabilities and conservative results. The figure successfully depicts the fundamental behavioral difference caused by the retrofitting of beams, which increases their moment capacity while reducing their ductility as shown in Figure 11. This behavior combined with the improvement in column performance results in an increase in the base shear capacity of the frames, but this happens at the expense of reduced deformation capacity. Hence, the retrofit designer has to consider this trade-off when making a retrofit evaluation and decision.

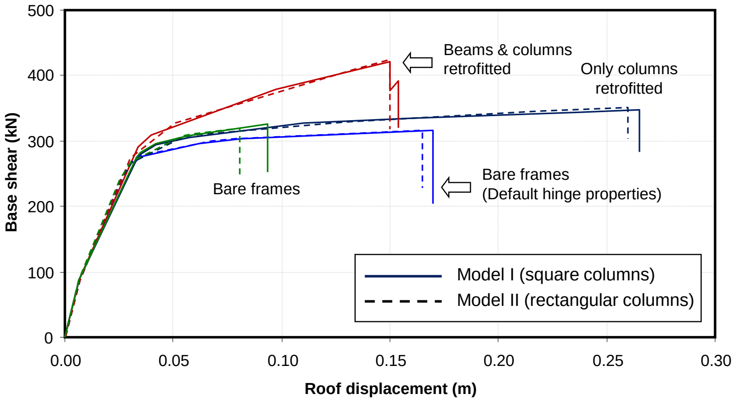


Figure 12. Moment-curvature relations for the beams before and after retrofitting

4. Seismic Performance of FRP Retrofitted Frames

Seismic performance levels are used to set performance objectives before constructing a new building or retrofitting an existing one. Once the capacity curve is obtained from pushover analysis, performance evaluation can be performed using the Capacity Spectrum Method in the acceleration-displacement response spectrum (ADSR) format as initially described in the ATC-40 report (ATC 1996) and later improved in ATC-55 (ATC 2001; FEMA 2005). This simplified nonlinear procedure provides a clear graphical representation of a building's performance level and the impact of various retrofit strategies by comparing the capacity spectrum with spectral acceleration response spectra representations of earthquake demands. A performance level is associated with the intersection of the capacity and demand curves, the so-called performance point. The deformation (drift) limits under lateral loading for various seismic performance levels are defined or referenced in related standards and guidelines (ATC 1996; FEMA 2000; ASCE 2007) and integrated in many nonlinear structural analysis software such as the one used in this research.

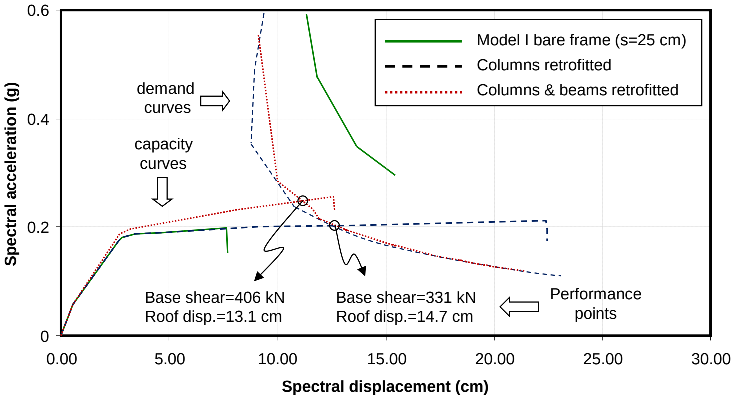


Figure 13. Performance levels of bare and FRP retrofitted frame (Model I)

The capacity curves obtained in the previous sections were used in this section to investigate the influence of FRP retrofitting on the seismic performance of the frame Model I. The frame was assumed to be in a high seismicity region and on poor soil condition and was modeled as such to define the seismic demand (ATC 1996). Figure 13 shows three pairs of capacity and demand curves obtained for Model I as (1) bare frame with column tie spacing, s=25 cm; (2) columns retrofitted; (3) both columns and beams retrofitted. Modified Kent and Park (Park et al. 1982) steel-confined concrete model was used for the bare frame and Maalej et al. (2003) FRP-confined concrete model was used for the columns of the retrofitted frames. As can be seen from the figure, no performance point exists for the bare frame, meaning that the bare frame is expected to collapse under seismic design loads. The benefits of retrofitting the columns using FRP composite wraps are twofold: (1) While the lateral load capacity essentially stays the same, the deformation capacity is drastically increased; (2) due to the increase in the deformation capacity, the seismic demand on the frame is reduced owing to increased energy dissipation and damping associated with damage. Hence, there exists a performance point for the frame with retrofitted columns, for which the performance level can be determined using code-specified drift limits. Retrofitting beams in addition to columns increases both the lateral load and the deformation capacity, although the increase in the deformation capacity is relatively limited. The fundamental difference between retrofitting only the columns and retrofitting both beams and columns is that the former only serves to obtain a performance point, while the latter can additionally result in an improvement in the seismic performance level.

A proper retrofit design decision for a frame structure should consider the influence of the two retrofit strategies illustrated in Figure 13, among other alternatives. FRP wrapping of columns can be considered first to increase the lateral deformation capacity to obtain a performance point for the frame. If a performance point cannot be obtained (low lateral load capacity) or that obtained does not satisfy the drift limits for the target performance level (too much lateral deformation), then retrofitting of the beams in addition to columns can be considered – at additional retrofit cost – to improve the seismic performance aware of the tradeoff between the increase in the lateral load capacity and the reduction in deformation capacity.

5. Concluding Remarks

This paper has presented and implemented a practical methodology for performance-based FRP retrofit design for RC frame structures and performed comparative investigations of different retrofit configurations, column geometries and material models. These investigations have led to a number of conclusions that contribute to the knowledge and practice of using FRP composites for seismic protection of RC frame structures:

- Nonlinear pushover analyses and seismic performance evaluation of RC frame structures using custom hinge properties obtained from moment curvature analyses of bare and FRP retrofitted members can quantitatively capture the lateral load behavior and seismic performance characteristics of the frame system for use in retrofit design and evaluation.
- FRP retrofitting of RC frames by wrapping of columns to serve as additional transverse reinforcement can significantly enhance the lateral deformation capacity and hence the ductility of the structure without significant increase in its lateral load capacity. This retrofit configuration can be used to obtain a performance point to prevent collapse under seismic design loads but once a performance point is reached, it does not provide any further improvements in the seismic performance level, i.e. the performance point stays nearly stationary.
- Retrofitting of beams in addition to columns for improved flexural (and shear) capacity can enhance both the lateral load and deformation capacity of the retrofitted frame, although the increase in deformation capacity is limited compared to the configuration in which only columns are retrofitted. This retrofit configuration can be used to obtain a performance point as well as to improve the seismic performance level of a frame structure.
- Use of different reinforcing steel and unconfined concrete models in the moment-curvature analyses was found to result in negligible differences in the resulting curves. The three steel-confined concrete models implemented in the analyses resulted in moderately close capacity curves for bare frames. The Modified Kent and Park (Park et al., 1982) model produced conservative results in most cases whereas the Mander et al. (1988) model consistently resulted in higher member and frame performance.
- The three FRP-confined concrete models implemented in the analyses produced similar capacity curves in most cases despite their significantly different characteristics. The Lam and Teng (2003) model was found to have a shortcoming when used for rectangular columns due to the aspect ratio multiplier used in calculating the shape factor in this model. The Maalej et al. (2003) model was preferred for its better modeling capabilities and conservative results.
- The capacity curves for Model I and II with square and rectangular columns, respectively, had no discernible difference when the FEMA-356 recommended hinge properties were used in the analyses of bare frames. When custom hinge properties were used for bare frames, the performance of Model II was found to be significantly lower than that of Model I due to the reduction in effective confinement for rectangular columns. This difference was reduced to a large extent upon retrofitting except for the case of Lam and Teng (2003) model used for Model II. It is recommended that the aspect ratio multiplier be ignored when this model is used for rectangular columns.
- Comparative investigations have revealed that the recommended hinge properties by FEMA-356 should be used with caution in the analyses of bare frames as these properties consider properly confined members and do not consider the loss of confinement in rectangular columns as well as the influence of column orientation.

Conflict of Interest

No conflict of interest was declared by the authors.

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