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RECENT ADVANCES IN SEISMIC RETROFIT OF RC FRAMES

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ABSTRACT

This paper reviews the results of some recent works conducted by the author on new methods of retrofitting the RC frames. On the local retrofit of RC members, it includes the work on the application of a new high performance fibre-reinforced cementitious composite material. The composite can be applied either as a wet mix to the desired thickness or attached as precast sheets or strips to the face of the member using a suitable epoxy adhesive. The suitability of this technique of member retrofit to enhance the strength and ductility of the retrofitted member compared with other methods of local retrofit, such as steel plates and FRP laminates, is discussed. Other works reviewed in this paper include those carried out recently on global retrofit of RC frames using direct internal steel bracing. Results of inelastic pushover tests on scaled models of ductile RC frames, directly braced by steel X and knee braces are presented which indicate that such bracings can increase the yield and strength capacities and reduce the global displacements of the frames to the desired levels. Also, the results of direct tensile tests on three full scale models of three different types of brace/RC frame connections are presented. Finally, the values of seismic behaviour factor, R, for this type of brace/frame system, evaluated from inelastic pushover analysis of dual systems of different heights and configurations are presented.

Keywords: RC frame, bracing, seismic retrofitting, HPFRCC, behaviour factor

1. INTRODUCTION

A number of reasons may necessitate the need to retrofit existing structures. It may be the rehabilitation of a structure damaged by an earthquake or other causes, or the strengthening of an undamaged structure made necessary by revisions in structural design or loading codes of practice. Here, the collective term, retrofit, which implies the addition of structural components after initial construction, is applied to both rehabilitation and strengthening processes. Earthquakes are by far the most common cause of damage to structures in earthquake-prone areas. Also, as the seismic loading and design codes are subject to more frequent revisions than the rather established gravity-based codes, earthquake consideration becomes a prime reason for the need to strengthen existing structures. A number of

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techniques may be used to retrofit concrete structures. Retrofitting may be carried out on a global basis by adding extra load-resisting elements such as steel frames or steel braces to the structure or it can be performed on a local basis by retrofitting the existing structural elements.

1.1 Local Retrofit Techniques

On the local basis of retrofitting the RC elements, externally bonded steel plates and FRP laminates have been extensively investigated [1-4]. The technique of retrofitting using externally bonded steel plates has gained widespread popularity, being fast, causing minimal site disruption and producing only small changes in section size. However, it suffers from a number of problems, including undesirable shear failures, difficulty in handling heavy steel plates, corrosion of the steel and the need for butt joint systems as a result of limited workable length [5-7]. FRP materials, on the other hand, have high strength to weight and stiffness to weight ratios and are chemically quite inert, offering significant potential for lightweight and durable retrofit [8-9]. A major shortcoming of the FRP is its vulnerability to undesirable brittle failures due to large mismatch in the tensile strength and stiffness with that of concrete.

Several studies undertaken in recent years at Cardiff University, directed at investigating the feasibility of using a high performance fibre-reinforced cementitious composite (HPFRCC) as a retrofitting material, have resulted in the development of CARDIFRC[®] [10-13]. This HPFRCC material can be applied to the face of a concrete member to the desired thickness as a wet mix or as an adhesively-bonded prefabricated slab or strip. The key advantage of CARDIFRC[®] mixes for retrofitting is that unlike steel and FRP, their tensile strength, stiffness and coefficient of linear thermal expansion are compatible with those of the retrofitted members. In a recent study, Alaee and Karihaloo [14] carried out experimental and theoretical investigations on some model RC beams retrofitted with CARDIFRC[®] strips of different thickness and configurations. They showed the ability of the strips to enhance the shear and flexural strength and to reduce the deflection of the retrofitted beams. Maheri et al. [15] expanded on the above study by looking at the potential of CARDIFRC[®] as a seismic retrofitting material. They obtained high values of toughness, ductility and overstrength for retrofitted beams.

1.2 Global Retrofit Techniques

On a global basis of resisting earthquake loads, shear walls are commonly used in RC framed buildings, whereas, steel bracing is most often used in steel structures. In the last two decades, a number of reports have also indicated the effective use of steel bracing in RC frames. The bracing methods adopted fall into two main categories, namely (i) external bracing and (ii) internal bracing. In the external bracing system, existing buildings are retrofitted by attaching a local or global steel bracing system to the exterior frames. Sekiguchi, et al. [16], Del Vall Calderon, et al. [17] and Badoux and Jirsa [18] have reported on practical examples of retrofitting carried out using this method. Scaled model testing was also reported by Bush et al. [19]. Architectural concerns and difficulties in providing

appropriate connections between the steel bracing and RC frames are two of the shortcomings of this method.

In the internal bracing method, the buildings are retrofitted by incorporating a bracing system inside the individual units or panels of the RC frames. The bracing may be attached to the RC frame either indirectly or directly. In the indirect internal bracing, a braced steel frame is positioned inside the RC frame. As a result, the transfer of load between the steel bracing and the concrete frame is achieved indirectly through the steel frame. Successful retrofits of existing buildings by indirect internal bracing using different forms of X, V and K concentric and eccentric braces within steel frames have been reported [20-25]. Instead of using a steel frame, Hjelmstad et al. [26] jacketed the RC columns with steel plates and connected the brace to the jackets. Tagawa et al. [27] also carried out model testing on K-braced frames. They concluded that the combined capacity of the RC frame and the braced steel frame may be assumed as the direct sum of the capacities of the individual frames. The indirect internal bracing method can be costly and technical difficulties in fixing the steel frame to the RC frame can be inhibiting. The retrofitted frame is also susceptible to diverse effects of dynamic interaction between the steel frame and the concrete frame during earthquake loading.

To overcome the shortcomings of the indirect internal bracing, Maheri and Sahebi [28, 29] proposed a direct connection between the steel bracing and RC frame without the need for an intermediary steel frame. The direct internal bracing method was proposed not only as a retrofit measure but also as a shear-resisting element in the seismic design of new buildings. They conducted experiments on scaled model frames strengthened by X-bracing, directly connected to the RC frames. Test results showed a 3-fold increase in the ultimate shear capacity of the frame. Nateghi-Alahi [30] used this technique to retrofit an existing building. The later experimental work on directly braced model frames by Tasnimi and Masoomi [31] also showed the applicability of this method of retrofit. Numerical work carried out by Abou-Elfath and Ghobarah [32-33] on both concentric and eccentric direct internal bracing in non-ductile RC buildings also showed an improvement in the seismic performance, particularly when using eccentric bracing. Recently, Maheri et al. [34-36] furthered earlier work by experimentally investigating the seismic performance parameters of different frame-brace systems as well as providing the design basis for the connections between the brace and the frame. In this paper the results of these studies on the direct internal bracing of frames as well as the results of recent studies by the author and colleagues on the HPFRCC local retrofitting of the RC members are discussed.

2. HPFRCC LOCAL RETROFIT OF RC MEMBERS BY CARDIFRC COMPOSITE

CARDIFRC is a HPFRCC which is characterised by high tensile/flexural strength and highenergy absorption capacity. The main ingredients of the CARDIFRC mix include; cement, water, microsilica, quartz sand, superplasticiser and fine steel fibres. Brass-coated fibres, 6mm and 13mm long and 0.16mm diameter are used to prevent corrosion. A detailed account of the mix proportions and properties are given by Alaee et al. [11-12]. The HPFRCC composite possesses an indirect tensile strength of 24 MPa, a compressive strength of over 200 MPa and a Youngs modulus of around 50 GPa. The composite can be either applied as a wet mix to the desired thickness on the surface of the RC member or it can be made into precast sheets or strips with specific thicknesses. The sheets or strips can subsequently be cut into desired sizes and attached to the face of the RC element using a suitable epoxy adhesive.

2.1 Seismic Performance Parameters of CARDIFRC

The seismic performance parameters under consideration are those statically determined parameters or properties frequently encountered when dealing with the level of seismic loading on a structure and the structural non-linear response to the seismic loading. These parameters include; the energy absorption capacity (toughness), ductility and overstrength. Toughness is defined as the area under the force-deflection response curve, whereas ductility, μ , is defined as the ratio of the maximum inelastic structural displacement Δ_{max} to the displacement corresponding to the idealised yield strength, Δ_y . The overstrength factor, R_s , signifying structural overstrength, depends to a large extent on the level of internal force redistribution. Other important factors including strain hardening, deflection constraints, higher material strength than that specified in the design and member oversize, also contribute to the structural overstrength.

To verify the ability of CARDIFRC strips to enhance the ductility of non-ductile (or lowductility) beams as well as verifying their ability to maintain the ductility levels of retrofitted ductile beams, two types of beam, namely non-ductile and ductile, were made. In total 46 concrete beams were made from a standard concrete mix having an average compressive strength of 46 MPa. Four of the non-ductile beams and three of the ductile beams were tested to failure as control beams. CARDIFRC strips were made in two thicknesses of 16mm and 20mm. Beams were retrofitted using three different types of strip configuration as is shown in Figure 1. For each type of beam, the control beams were first tested to failure. The remaining beams were then pre-loaded to about 75% of the average failure load of the control beams. The damaged beams were subsequently retrofitted with CARDIFRC strips having different strip thicknesses and strip configurations. The retrofitted beams were again tested and their force-deflection responses to failure were recorded. A detailed account of the test procedure is given in [15]. Typical force-deflection curves for non-ductile control beams and retrofitted beams are shown in Figure 2. Only one of the retrofitted beams failed in pure shear and as expected the remaining beams failed in a pure flexural mode. None of the retrofitted beams showed signs of the undesirable debonding of the strips or delamination of concrete cover.



Figure 1. Retrofit strip configuration: (a) one strip bonded on the tensile face, (b) one strip bonded on the tension face and four short strips bonded on the vertical sides and (c) one strip bonded on the tension face and two strips covering the length of vertical faces [15].



Figure 2. Selected force-displacement response curves of the non-ductile beams

For comparison, the average toughness values for beams retrofitted with different strip thicknesses and configurations are plotted in Figure 3.a. As expected, non-ductile control beams show the least amount of toughness. As the retrofitting strips are applied, the general failure mode changes from shear to flexure and a sharp increase in toughness is observed. Toughness is seen in Figure 3.a to be highly dependent on the number of retrofitting strips and the type of strip configuration. A substantial increase of 245% is noted for beams retrofitted with three, 16mm thick strips (configuration (c)). The three control beams that failed in shear evidently possess no ductility. Application of one, 16mm thick strip to the tension face (configuration (a)) however changed the failure mode and increased the ductility by an average of 57%. When strip configurations (b) and (c) were used, substantial increases in ductility (to 200%) were achieved. This is because the possibility of brittle shear failure was totally removed through vertical strips bonded to the areas of high shear stresses. The average values of ductility calculated for different retrofitted beams of non-ductile type are plotted in Figure 3.b. The overstrength factors evaluated for the retrofitted non-ductile beams are also plotted in Figure 3.c. Similar to toughness and ductility, an increasing trend, ranging from 10% to 78%, is seen in the overstrength factor as stronger retrofitting configurations are utilised. The 20mm strips appear to fair less favourably on all three parameters of toughness, ductility and overstrength when compared with their 16mm thick counterparts.

As expected, all the ductile beams, including the control beams, failed in a ductile flexural manner. Toughness was evaluated for each of the control and retrofitted beams and the average values are plotted in Figure 3.d. Retrofitting the ductile beams with one, 16mm thick strip on the tension face increases the toughness by 22%, whereas a larger increase of 60% is obtained when using three, 16mm thick strips (configuration (c)). An interesting point to note is that when comparing the toughness of the ductile beams with that of the nonductile beams, the control beams of ductile type and the three-strip configuration retrofitted non-ductile beams appear to have a similar toughness. This indicates that for our test beams, the three retrofitting strips provide a shear capacity for the non-ductile beams which is no less than the shear capacity provided by the shear reinforcement in the ductile beams. To evaluate the ductility factor, μ , of the ductile beams, the inelastic maximum displacements (Δ_{allow}) were considered to be 1% of the effective span of the beam. Average values of ductility factor for different retrofitted beams calculated using Δ_{allow} are plotted in Figure 3.e. This figure shows that the addition of the retrofitting strips results in a sharp increase in the level of ductility of these already ductile beams. For the 16mm thick strips, the increase (about 60%) appears to be irrespective of the type of retrofitting configuration, however, for the 20mm thick strips, an increasing trend in ductility can be seen as the number of strips increases. Figure 3.f shows the overstrength factor evaluated for the control and the retrofitted ductile beams. It can be seen that one, 16mm thick strip has increased the overstrength by 20% whereas three, 16mm strips have resulted in an increase of over 80%. For the case of the 20mm thick strips, the overstrength factor actually decreases noticeably when the short vertical strips are added. An explanation to the cause of this performance may be provided by the notion that the length of the fibre reinforcements rather than the thickness of the strips govern the flexural behaviour and strength of the CARDIFRC strips. It follows that there is an optimum strip thickness for fibres of specific length and strength.



Figure 3. Average (a) toughness, (b) ductility factor and (c) overstrength factor for non-ductile beams and (d), (e) and (f) for ductile beams, respectively

3. DIRECT INTERNAL BRACING METHOD OF RETROFIT

This global method of RC frame retrofit was mentioned in the introduction to consist of directly bracing a unit RC frame or panel without the help of an intermediary steel frame. This fast, economical and simple method of increasing the in-plane strength of the frame can be used both as a retrofitting measure and as a load resisting element at design level. Recently, in three different studies conducted by the author and his colleagues, different aspects of this method of retrofitting were investigated. Brief accounts of these investigations follow.

3.1 Seismic Performance Parameters of Braced Frames

In an experimental study, pushover tests were conducted on scaled models of ductile unit frames, directly braced by X and knee steel braces. The objective of the study was to compare some response and design parameters, including; load capacity, stiffness, toughness, ductility and performance factor of different unbraced and X and knee-braced RC frames in order that suitable forms may be proposed for different seismic design and retrofit needs. Model unit frames constructed for experimental investigations were 1:3 scaled models of a typical 3mx3m unit ductile frame. For the purpose of comparison, RC beams and columns in all the model frames (braced or unbraced) had identical dimensions and reinforcement detail. The corresponding horizontal loads estimated for the ultimate capacity of the model frame and the bracing systems were, 33 kN for the unbraced frame, 79 kN for the X-bracing system alone and 33 kN for the knee bracing system alone. In total six model frames were constructed so that the repeatability of the tests could be verified. Two model frames (F1 and F2) were identical unbraced frames, two models (FB1 and FB2) were identical X-braced and two models (FK1 and FK2) were identical knee-braced frames. The set-up for the pushover testing of the model frames is illustrated in Figure 4. A detailed account of test set-up and observations are given in [34].

3.1.1 Load Capacity

Over 2.5 fold and 3.5 fold increases in the lateral load capacities were achieved for the kneebraced and X-braced model frames tested in this study, respectively. Test results show that the load capacity of an existing ductile frame can be increased to the desired level by directly adding a bracing system to the frame, without the need for prior strengthening of the existing frame. This point is further substantiated when it was considered that the load corresponding to the appearance of the first plastic hinge in the ductile RC beam (10 kN for the unbraced frame) increased by 90 kN when X-bracing was used and by 30 kN when knee-bracing was employed.

3.1.2 Stiffness and Toughness

For comparison, the horizontal force-displacement curves for frames F2, FB1 and FK1 are plotted together in Figure 5. Three distinct regions may be marked on all three curves. The first region, extending to the formation of the first plastic hinge in the frame-brace system, covers the linear range of response. The second region corresponds to the formation of other plastic hinges in the bracing system and RC beams and the third region, extending to the

ultimate capacity, marks the formation of plastic hinges in the RC columns. Figure 5 also shows the best-fit lines, fitting the specified regions. Toughness of three test frames, determined as the area under the pushover force-displacement curve show a five-fold increase for the X-braced frame and around three-fold increase for the knee-braced frame, both compared with the unbraced frame. This indicates the ability of the braced frames to absorb large energies.



Figure 4. The pushover test set up for braced frames

3.1.3 Ductility, Overstrength and Performance Factor

Ductility, overstrength and performance factor parameters were determined for the six test model frames and are given in Table 1. It should be noted that the performance factor parameters listed in this Table are specific to the model frames tested and do not represent those of full size frames. The Table indicates that when a ductile frame is braced, in return for the increase in strength and stiffness, ductility, overstrength and the performance factor are reduced. This is particularly true for the X-braced frames. The knee-braced frames exhibit much larger ductility compared with the X-braced frames. The overall seismic performance of knee-braced frames, regarding load capacity, stiffness and ductility appears to be more favourable than either the unbraced frame or the X-braced frame.

The test results lead us to conclude that X-bracing is more suitable for a strength-based design. However, the relatively small post-yield capacity and the somewhat brittle failure mode of the X-braced frame make this system unfavourable for a ductile design. Knee bracing, on the_other hand, is suitable for both the strength-based and ductility-based

designs. A knee bracing system can therefore be successfully utilised to design for both the damage-level and collapse-level earthquakes for which the damage level may be considered as the yield capacity of the knee elements.



Figure 5. Relative stiffness of the three, unbraced, knee-braced and X-braced frames [34]

Table 1.	Ductility and performance factor parameters of the test frames calculated using the
	displacement capacity (Δ_{max}) of 10mm

Frame	Δ _y (mm)	Δ _{max} (mm)	Μ	V _s (kN)	Vy (kN)	V _e (kN)	R_{μ}	Rs	R
F1	2.78	10.0	3.60	9.5	22.0	79.1	3.60	2.31	8.3
F2	2.65	10.0	3.77	10.0	21.0	79.5	3.77	2.10	7.9
FB1	6.41	10.0	1.56	60.0	74.0	115.4	1.56	1.23	1.9
FB2	6.45	10.0	1.55	59.5	75.0	116.2	1.55	1.26	1.9
FK1	5.21	10.0	1.92	28.0	57.0	109.4	1.92	2.04	3.9
FK2	4.10	10.0	2.44	27.5	46.0	112.2	2.44	1.68	4.1

3.2 Brace-Frame Connection

Similar to other types of composite systems, the successful transfer of loads between the RC frame and bracing system can be achieved through a robust connecting mechanism. Tests carried out on scaled brace/frame systems have indicated the need for investigating the appropriate connection types and their design [34]. The type of direct connection to be used depends on whether the bracing system is to be connected to an existing concrete frame or to a frame under construction. It also depends on the type of bracing system to be connected. A number of different direct connection techniques have been suggested for both retrofitting existing frames [28, 31] and strengthening frames at design level [28, 34]. The main difference in these connection methods is in the manner in which the gusset plate is connected to the concrete members. Two main techniques were investigated by Tasnimi and Massomi [31] using scaled model frames. These included; (i) a technique whereby the concrete members forming the connection are jacketed with steel plates and the gusset is subsequently welded to the steel jacket and (ii) the technique in which a steel plate is bolted to the connection face of the concrete member and the gusset is welded to this plate. They concluded that the jacketing technique lacks the robustness of the bolted plate technique.



Figure 6. Details of (a) connection type (b) and (b) connection type (c) constructed for direct tensile tests

In an experimental study undertaken by Maheri and Hadjipour [36], a number of fullscale connection systems using the bolted plate technique were investigated. The connections were designed by combining the existing provisions of codes of practice for steel and concrete structures. Static tests were then conducted on full-scale brace/RC frame connections to verify the applicability of the existing design code provisions for these types of connections. Three different connection methods were investigated. These connections are termed types (a), (b) and (c). Connection type (a) is a typical corner connection for joining an X-bracing system to the intersection of a beam and a column in a concrete frame. The brace is welded to the gusset, which in turn is welded to the concrete beam/column

connecting plates. The connecting plates are, in turn, bolted to the frame members using hooked anchor bolts embedded in concrete. Connection type (b) is identical to type (a) except that the plates are connected to the concrete members using straight bolts, anchored at the opposite face with a back plate and nuts. In connection type (c), the corner of the frame is built-up with concrete so that only one connecting plate is used to transfer the brace load directly through the joint. This connection type would be suitable for drag-through brace connections in frames under construction so that special provisions for the built corner can be made at design level. Three full scale models of the said connection types were constructed for direct tensile tests. Details of the models for connection types (b) and (c) are shown in Figure 6. A full account of the design, construction and testing procedure is given elsewhere [36].

3.2.1 Connection Type (a)

The force/displacement response of the brace shown in Figure 7 indicates that yielding of the brace occurred at the load of 134 kN. This experimental yield strength is close to the design yield strength of the brace (131 kN). Other connection elements, including the gusset and connecting plates, remained elastic up to and beyond the brace yield strength. Some erratic response observed on the gusset corresponded to the response of the brace/gusset weld and the redistribution of load in the gusset at, and beyond, yielding of the brace. The ultimate (rupture) strength of the brace was measured as 213 kN. This value corresponds well with the design rupture strength (210 kN). The rupture of the brace started at the edge of one of the brace angle sections and gradually developed through the entire section to the opposite edge. Displacements measured for the connecting plates/concrete arm were comparatively small and the response was linear up to the rupture of the brace.



Figure 7. Overall force/displacement response curves for connection types (a), (b) and (c)

3.2.2 Connection Type (b)

The response of this connection type was expected to be similar to that of the connection type (a). Figure 7 indicates that the brace yielded at the load of 138 kN, which is close to the expected yield capacity of 145 kN. The rupture of the brace occurred at the ultimate load of 232 kN. Proximity of the experimental rupture load to the expected brace rupture strength (233 kN)

points to the absence of a credible load eccentricity in this test. Response of the gusset and the connecting plates/concrete arm were linear up to the yield strength of the brace. Also, similar to the behaviour observed in the connection type (a), some load redistribution occurred in the gusset at, and beyond, the yielding of the brace. No failures could be visually detected in any of the connection elements up to the rupture of the brace. Similar to connection type (a), displacements recorded for this connection on the connecting plates/concrete arm were small and the response remained linear up to the final load.

3.2.3 Connection Type (c)

Results of the tensile force test carried out on this connection type, also plotted in Figure 7, show that the brace element yielded at 135 kN which is close to its expected yield strength. Unlike the connection type (a) and (b), however, the post-yield response of this connection was dominated by the response of the connecting plate. In this connection, the tensile force is transferred through the connecting plate mainly in the out-of-plane bending. The stiffened connecting plate responded elastically to the applied load up to 165 kN. At this load, a sharp change in the slope of the force/displacement response curve of the connecting plate/concrete arm indicates the beginning of the buckling of the connecting plate. The inelastic response of both the brace member and the connecting plate continued until the rupture of the brace at the ultimate load of 216 kN. An interesting point to observe is that the yielding of the connecting plate prior to the rupture of the brace did not result in any loss of strength in the connection, nor did it reduce the pre-yield stiffness of the connection. However, it resulted in an increase in ductility of the connection. The increase in ductility may be deduced from the increase in the overall inelastic displacement of this connection type (about 19mm) when compared to that of the connection types (a) and (b) (about 14mm) as seen in Figure 7. The behaviour of the connecting plate in connection type (c) tested here, is therefore not dissimilar to the behaviour of a knee element in a knee bracing system.

3.3 Determination of Behaviour Factors

In forced-based seismic design procedures, behaviour factor, R [37] (or R_w) also referred to by other terms including, response modification factor (UBC code [38] and NEHRP Provisions [39]), is a force reduction factor used to reduce the linear elastic response spectra to the inelastic response spectra. In other words, behaviour factor is the ratio of the strength required to maintain the structure elastic to the inelastic design strength of the structure. The behaviour factor, R, therefore accounts for the inherent ductility and overstrength of a structure and the difference in the level of stresses considered in its design. It is generally expressed in terms of its three components,

3.3.1 Inelastic Pushover Analysis

In a study by Maheri and Akbari [35], the behaviour factor for steel-braced RC frames was evaluated. In that study, 4-storey, 8-storey and 12-storey frames were considered. These are typical numbers of storeys used by some other investigators to cover low-rise to medium-rise framed buildings. All frames were three-bay wide with the central bay braced in the braced dual systems. An earlier study on braced steel frames by Assaf [40] showed that the number of bays in a frame has little effect on the R values. Two bracing systems were considered, including; (i)

X bracing and (ii) knee bracing. In designing the dual systems, the share of bracing system from the load is a parameter to be decided at design stage. In other types of dual systems, very little work is reported on the effect of assigned share of brace from base shear on the R factor. Considering the dual purpose of bracing and RC frame, it was found necessary to investigate this effect. Three different shares from the base shear of 0%, 50% and 100% for bracing are investigated. The steel bracing systems are thus designed to resist the above load shares and the RC frames are designed to resist the remaining code-specified base shears. Details of the design, modelling and analysis of the selected frame-brace systems are reported elsewhere [35]. A finite element representation of X-braced and knee-braced unit frames are shown in Figure 8.



Figure 8. Finite element representation of the unit RC frame with (a) X-brace and (b) knee-braces



Figure 9. Normalised pushover response curves for X-braced RC frames [35]

Inelastic pushover analysis of the multi-storey systems under investigation was carried out at horizontal load steps equal to 2% of their design capacities and the corresponding inelastic displacements were calculated. To gain a comparative view of the pushover curves, selective curves were normalised to their respective design capacities as shown in Figure 9 for the X-braced frames.

3.3.2 Tentative R Factors for Steel-Braced RC Frames

Before proceeding to discuss the *R* factor for steel-braced RC frames, it is useful to draw some comparisons between evaluated *R* values of the unbraced RC frames considered in this paper and those of the similar RC frames given by some codes of practice. For an intermediate ductility, moment-resisting RC frame, behaviour factors of R = 5.0, R = 5.5 and R(q) = 3.75 are given by NEHRP [39], UBC [38] and Eurocode-8 [37], respectively. The *R* values evaluated for similar moment resisting RC frames by Maheri and Akbari [35], range from 4.0 for the 12-storey frame to 5.3 for the 4-storey frame. These values fall within the range of values given by different codes of practice.

Number	Bracing	Share of brace (%)	Ductility Demand			
of Storeys	system		$\mu = 2$	<i>M</i> = 3	$\mu = 4$	μ = 5
		0	4.8	7.1	9.4	11.7
	Х	50	5.7	9.1	12.4	15.8
4		100	5.4	8.5	11.6	14.7
		50	4.8	7.3	9.8	12.3
	Knee	100	5.0	7.8	10.5	13.3
		0	4.1	6.0	7.9	9.8
	Х	50	6.6	9.9	13.3	16.6
8		100	5.9	8.9	11.8	14.8
		50	5.0	7.5	10.0	12.5
	Knee	100	6.3	9.5	12.6	15.8
		0	4.0	5.8	7.7	9.6
	V	50	6.7	10.0	13.3	16.6
10	А	100	4.8	7.1	9.5	11.9
12		50	5.7	8.4	11.2	14.0
	Knee	100	6.3	9.3	12.4	15.4

Table 2. R factors for different ductility demands

Drawing our attention to the steel-braced RC frame dual systems, it is evident that little basis for similar comparison exists between the R values evaluated for this newly proposed load-resisting dual system and the R values of the well-established dual systems presented in codes of practice. Of the three variable parameters discussed in this paper as affecting the Rvalue, the number of storeys appears to be the predominant variable. The other variables, including the type of bracing system and the share of bracing from the applied load, have more localised influences and therefore do not warrant a similar generalisation. The significant effect of the number of storeys on R factor of steel-braced RC frames stems from the fact that shorter braced frames exhibit larger ductility than taller frames and therefore possess higher ductility 'capacity'. It is therefore prudent to recalculate the R factors for the frames under consideration using specific ductility 'demands'. R factors were re-evaluated for all the systems under consideration using ductility demand values of $\mu = 2$, $\mu = 3$, $\mu = 4$ and $\mu = 5$. The results are presented in Table 2. With a view at the results presented in Table 2, tentative R values for steel-braced intermediate ductility, moment resisting RC frame dual systems are presented in Table 3. The proposed R factors are given for different ductility demands that constitute the generally accepted range of 'intermediate ductility' response. Further work is needed to arrive at R factors for low ductility and high ductility steel-braced RC frame dual systems.

Table 3. Tentative values of R factor for steel-braced, RC frame dual systems

Ductility Demand	μ = 2	$\mu = 3$	μ = 4	$\mu = 5$
R	5.0	7.0	9.0	12.0

4. CONCLUSIONS

The HPFRCC material, designated CARDIFRC, has a great potential for seismic retrofitting of both damaged and undamaged RC members. It is capable of increasing the strength, stiffness, energy absorption capacity and ductility of the members. As for the global retrofit of the frame by steel bracing, addition of the bracing system through direct connections to the frame can enhance the stiffness and strength of the system. Both X braces and knee braces may be utilised to design or retrofit for a damage level earthquake. However, when designing or retrofitting for a collapse level earthquake for which ductile behaviour is necessary, the knee bracing system is most suitable. On the connections between the brace system and the frame, the full scale tests indicated that different types of connections can be designed successfully by combining the current guidelines and provisions set out by codes of practice for designing brace/steel frame connections and base plate/RC member connections. Strength test results corresponded

well with the design predictions for individual elements of the connection types tested. **REFERENCES**

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