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# **INTERNAL STEEL BRACING OF RC FRAMES**

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## ABSTRACT

Steel bracing systems can be used effectively for seismic retrofitting of existing RC buildings as well as for seismic design of new buildings. Although adaptation of bracing to upgrade the lateral load capacity of existing RC frames has been the subject of a number of successful studies, guidelines for its use in newly constructed RC frames need to be further developed. This paper reports on some recent experimental and numerical work conducted by the author and his colleagues on internal bracing of RC frames using direct connections between the bracing system and the frame. The effects of X-bracing and knee bracing on enhancing the seismic capacity of the frames are investigated experimentally through pushover tests as well as cyclic tests. A compression release device has also been introduced and tested to enhance the seismic performance of the bracing system by avoiding the buckling of the compression member. An important consideration in the design of steel-braced RC frames is the level of interaction between the strength capacities of the RC frame and the bracing system. In this paper, results of experimental and numerical investigations aimed at evaluating the level of capacity interaction between the two systems are also discussed. It is found that the capacity interaction is due primarily to connections overstrength. Based on the numerical results the connection overstrength has been quantified and guidelines for the seismic design of the internally braced RC frames with direct connections are provided.

### **1. INTRODUCTION**

Steel bracing is generally used to increase the lateral load resistance of steel structures. In recent years, the concept of steel bracing has also been applied to the retrofitting of reinforced concrete frames. Increased architectural flexibility, reduced weight of the structure, ease and speed of construction and the ability to choose more ductile systems can be considered as the main advantages of steel bracing in comparison with RC shear walls. Two bracing systems are generally used, external bracing and internal bracing. In external bracing, steel trusses or frames are attached either as a global external support to the building exterior or, more locally, to the face of individual building frames. A number of investigators have reported on the efficiency of external bracing in seismic retrofitting of existing RC buildings [1-4]. Architectural concerns and difficulties in providing appropriate connections between the bracing system and RC frames are two of the shortcomings of this method.

In internal bracing, steel bracing members are inserted in the empty space enclosed

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by columns and beams of RC frames. As a result, each unit frame is individually braced from within. 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 carried out 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 have been reported in the literature [5-8]. In some repair and retrofitting cases, provision of the steel frame may be necessary to reduce the strength demand on an already damaged and weakened RC frame; however, in other instances the steel frame acts only as a costly connecting mechanism with inhibiting technical difficulties in fixing the steel frame to the RC frame.

To overcome the shortcomings of the indirect internal bracing, Maheri and Sahebi [9] first recommended using direct connections between the brace elements and RC frame without the need for an intermediary steel frame. In an experimental work, they showed the ability of this bracing system to enhance the strength capacity of RC frames.Later experimental work on directly braced model frames by Tasnimi and Masoomi [10] also showed the applicability of this method. Recent analytical work carried out by Abou-Elfath and Ghobarah [11, 12] 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. In continuation of their previous work, Maheri et-al [13] conducted experimental investigations on pushover response of scaled RC frames; braced with both diagonal bracing and knee bracing systems. In this study the effectiveness of the two bracing systems in increasing some seismic performance parameters was shown. Also, in a theoretical study, Maheri and Akbari presented the behaviour factor, R, for this class of dual systems [14].

Appropriate design of direct connections between the bracing members and the RC frame is important to achieve the required lateral load capacity. Maheri and Hadjipour [15] proposed a connection that minimizes the eccentricity of the brace member force. This allows transferring the brace force to the corner of the RC frame without producing local damage in concrete members. Using the results of an experimental program conducted on a number of full-scale connections, they also presented design guidelines for the brace-frame connections in new construction. Recent experimental works by Youssef et-al [16] and Ghaffarzadeh and Maheri [17-19] have shown further that different directly-connected internal bracing systems can be used effectively in retrofitting of existing concrete frames as well as shear resisting elements for construction of new RC structures.

# 2. SEISMIC RESPONSE PARAMETERS OF X-BRACED FRAMES 2.1. Pushover Tests

In an experimental study, details of which are presented in reference [13], pushover tests were conducted on scaled models of ductile unit frames, directly braced by X steel braces. The objective of the study was to compare some seismic response and design parameters, including; load capacity, stiffness, toughness, ductility and





performance factor of different unbraced and X-braced RC frames in order that suitability of the X bracing of RC frames could be investigated. 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 and 79 kN for the X-bracing system alone. In total four model frames were constructed so that the repeatability of the tests could be verified. Two model frames (F1-P and F2-P) were identical unbraced frames and two models (FB1 and FB2) were identical X-braced frames. A detailed account of test set-up and observations are given in [13].



Figure 1. The pushover test of frame F1-P

# Load Capacity, Stiffness and Toughness

Over 3.5 fold increase in the lateral load capacity was achieved for the X-braced model frames tested in that study. 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 is noted that the load corresponding to the appearance of the first plastic hinge in the ductile RC beam (10kN for the unbraced frame) increased by 90 kN when X-bracing was employed. Cross-bracing also appears to increase the initial stiffness of the RC frame. The increased stiffness, together with the increased capacity, substantially increase the toughness of the braced frame compared to the unbraced frame. Toughness of the test frames, determined as the area under the pushover force-displacement curves shows a five-fold increase for the X-braced frame. This indicates the ability of the X-braced frame to absorb large energies.

# **Ductility, Overstrength and Performance Factor**

Ductility, overstrength and performance factor parameters were determined for the four test model frames and are given in Table 1. The parameters given in this Table are presented in Figure 2, in which  $V_s$ ,  $V_v$  and  $V_e$  are forces corresponding to the





first yield, structural yield and the elastic response, respectively and  $R_{\mu} = V_e/V_y$ ,  $R_s = V_y/V_s$  and  $R = R_{\mu}R_s$ . 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 the full size frames. The Table indicates that when a ductile frame is braced, in return for the increase in strength, stiffness and toughness, ductility, overstrength and the performance factors are reduced.



Figure 2. Pushover curve parameters

Table 1: Seismic response parameters of the test frames

Frame	Δ <sub>y</sub> (mm)	$\Delta_{max}$ (mm)	μ	V <sub>s</sub> (kN)	Vy (kN)	V <sub>e</sub> (kN)	$\mathbf{R}_{\mu}$	R <sub>s</sub>	R
F1-P	2.78	10.0	3.0	9.5	22.0	79.1	3.60	2.31	8.3
F2-P	2.65	10.0	3.8	10.0	21.0	79.5	3.77	2.10	7.9
FB1	6.41	10.0	1.6	60.0	74.0	115.4	1.56	1.23	1.9
FB2	6.45	10.0	1.5	59.5	75.0	116.2	1.55	1.26	1.9

The test results lead us to conclude that X-bracing is more suitable for a strengthbased 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.

### 2.2. Cyclic Tests

In another experimental study, cyclic tests were carried out on half-scale RC unit frames braced with X-bracing. Details of the test set-up and results are reported elsewhere [18]. Unit frames where selected from the third floor of a three-bayed, four-storey frame of a residential building. Two lateral load resisting systems, namely; an RC moment frame and an X-braced RC frame, were considered for the study. The gravity and earthquake forces acting on these unit frames were determined in accordance with the Iranian seismic code [20] using the seismic force reduction factor for moment frames with moderate ductility. The size of the

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test specimens was determined based on the available laboratory space and the equipment limits. A 2/5 scaled model, measuring 1.76 m by 1.36 m, was found to be satisfactory. The forces acting on the panels were also scaled down resulting in a lateral load of 22kN and two vertical loads of 35kN for the moment frame and the same lateral load of 22kN and two vertical laods of 38.5kN for the braced frames. One moment resisting RC frame model, namely F1 and two braced RC frame models, namely FX1 and FX2, were designed using the above gravity and lateral loads. The moment frame was designed according to ACI 318-02 [21] and its detailing was done in accordance with the ACI special provisions for seismic design. Reinforcement details for this frame are shown, on the left hand side, in Figure 3. AISC-LRFD [22] was used to design the brace members and their welded connections to the guest plates. Reinforcement details for the braced frames are also shown on the right hand side of Figure 2. A double-angle brace cross-section, consisting of two  $25 \times 25 \times 3.2$ mm angles, giving a cross-sectional area of 300 mm<sup>2</sup>, was chosen for the frame FX1 and a C 30×3.5 mm channel with a cross-sectional area of around 500mm<sup>2</sup> was selected for the frame FX2 (Figure 3). The difference in the brace member cross-section, therefore, made the FX2 frame somewhat stronger than the FX1 frame.



Figure 3. Detailing of the moment RC frame (F1) and the braced RC frames (FX1 & FX2)

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The model frames were subjected to gravity loads using two hydraulic jacks. For the cyclic test, the actuator was first pulled to a displacement,  $d_1$ , of 5 mm then pushed to the same displacement. The value of  $d_1$  was increased in the following cycles by an increment of 5 mm. The behaviour of the test models was monitored by using electrical and mechanical instrumentations including: Load cells attached to the hydraulic jacks and the actuator to measure applied loads, Linear Voltage Differential Transformers (LVDTs) to measure the lateral deformations and electronic strain gauges to monitor local strains in the reinforcement bars as well as steel bracing elements.



Figure 4. Test set-up and pattern of cracking in the moment frame (F1)

### Hysteretic Response and Load Capacity

Figure 4 shows details of crack patterns in frame F1. The hysteretic lateral loaddrift curves for the three frames F1, FX1 and FX2 are also shown in Figure 5. For the moment frame F1, at a load of 37.5kN, yielding of the lower bars of the lower beam initiated the plastic response. Failure occurred by plastic hinging at the ends of the upper and lower beams at a load of 55kN. At a drift of 1.9%, corresponding to a lateral load of 105kN, yielding of the double-angle bracing member of the braced frame FX1 initiated the plastic response. A significant drop in the lateral load capacity was observed at a load of 140kN (drift of 4.0%). This was noted to be due to the buckling of brace members. Following this, the lateral load capacity was mainly provided by the RC frame, which failed when plastic hinges were formed at the ends of the lower and upper beams. In the frame FX2, the yielding occurred at a load of about 140kN. The lateral capacity of this frame was not however affected because the bracing members were still acting in the elastic range. Testing was continued to a load of 200kN, which was the loading capacity of the actuator and subsequently the test was terminated. A summary of the yield loads and the maximum sustainable loads and their corresponding displacement ratios for the three tested frames are presented in Table 2.



Figure 5. Lateral load-drift hysteresis of frames (a) F1, (b) FX1 and (c) FX2

 Table 2: The yield and ultimate strength capacities and their corresponding displacements

Frame	Yield strength (kN)	yield displacement (%)	Ultimate strength (kN)	displacement at ultimate strength (%)
F1	37.5	1.5	55	4.6
FX1	105	1.9	140	4.0
FX2	140	2.8	200	3.9

## **Stiffness Degradation**

The lateral stiffness was calculated as the slope of the line joining the peak of positive and negative loads at a given cycle. The lateral stiffness is an index of the response of the frame from one cycle to the following cycle. Figure 6 illustrates a plot of the lateral stiffness for the three tested frames. Before buckling of the compressive brace, the diagram shows that the lateral stiffness of the frame FX1 was more than double that of the frame F1 and that the rate of stiffness degradation for both systems was almost equal. However, after buckling of the compressive brace, the lateral stiffness of the frame FX1 dropped and became comparable to that of the moment frame (Figure 6). Also, the FX2 frame, having more robust bracing members compared to the frame FX1, shows higher hysteretic stiffness compared to the later. However, both frames show a similar rate of stiffness degradation.





#### **Energy Dissipation Capacity (Toughness)**

The energy dissipated by the three tested frames during the cyclic load testing was calculated as the area enclosed by each hysteretic loop. Figure 7 shows a plot of the energy dissipated during a load cycle versus the lateral drift. Also the energy dissipated by each test frame after a number of selected cycles is presented in Table 3. It is observed that at low drift levels, the energy dissipated by the frames FX1 and FX2 was comparable with that of the frame F1. At higher levels of drift, it is clear that the energy dissipated by the braced frames is much higher than that by the moment frame. This proves that the overall seismic performance of the braced frames regarding capacity, stiffness and toughness is expected to be superior to that of the moment frame. This was also deduced from the results of the pushover tests presented earlier.



Figure 6. Degradation of the lateral stiffness of test frames

Figure 7. Variation of energy dissipation with the applied displacement

Table 3: Energy dissipation capacity of the test frames									
	Cumulative energy dissipated (kN.mm)								
Frame	Cycle 5	Cycle 10	Cycle 15	Cycle 20	Cycle 25				
F1	600	2229	5619	13256	25474				
FX1	451	4367	13163	27276	32875				
FX2	570	3807	11540	26714	-				

#### Ductility

In these tests, ductility is measured both as the ratio of the displacement pertaining to the maximum force  $\Delta_{max}$ , to the displacement at yield  $\Delta_y$  and as the ratio of the maximum displacement  $\Delta_{available}$  to the displacement at yield point  $\Delta_y$  of the model frames. These are calculated and shown in Table 4. As it was expected, the addition of X-bracing system somewhat reduces the ductility of a ductile frame, but the reduction in ductility does not affect the energy dissipation capacity of the frames.

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Frame	Yield displacement (∆ <sub>y</sub> ) (mm)	displacement at ultimate strength $(\Delta_{max})$ (mm)	Maximum available displacement (∆ <sub>available</sub> ) (mm)	Ductility corresponding to Δ <sub>max</sub>	Ductility corresponding to ∆ <sub>available</sub>				
F1	18.0	55.4	68.0	3.1	3.8				
FX1	22.5	47.5	62.5	2.1	2.8				
FX2	33.0	45.6	-	1.4	-				

#### 2.3. Seismic Behaviour Factor

In forced-based seismic design procedures, behaviour factor, R is a force reduction factor used to reduce the linear elastic response spectra to the inelastic response spectra. 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. In another study carried out by Maheri and Akbari [14], the seismic behaviour factor (R)was evaluated for steel X-braced RC buildings. The R factor components including ductility reduction factor and overstrength factor were extracted from inelastic pushover analyses of brace-frame systems of different heights and conFigureurations. 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. DRAIN-2DX program was utilised to carry out nonlinear pushover analysis of each system. Inelastic pushover analysis of the multi-storey systems under investigation was carried out at horizontal load steps equal to 2% of the design capacity. A constant gravity load equal to total dead load plus 20% live load was also applied to each frame.

The effects of some parameters influencing the value of *R* factor, including the height of the frame, share of bracing system from the applied load and the type of bracing system were investigated. Of the three variable parameters investigated, 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 steelbraced RC frames, stems from the fact that shorter braced frames exhibit larger ductility than taller frames, therefore they possess higher ductility 'capacity'. It was therefore found to be prudent to calculate the *R* factors for the frames under consideration using specific ductility 'demands' of  $\mu = 2$ ,  $\mu = 3$ ,  $\mu = 4$  and  $\mu = 5$ . Based on the results obtained, tentative *R* values for steel-braced intermediate ductility, moment resisting RC frame dual systems were presented as shown in Table 5. The proposed *R* factors are given for different ductility 'response.

Table 5: Tentative values of R factor for steel-braced, RC frame dual systems

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



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## **3. CONNECTION OVERSTRENGTH**

An important consideration in the design of internally-braced RC frames with direct brace-frame connections is the level of interaction between the strength capacities of the RC frame and the bracing system. In this paper, results of experimental and numerical investigations aimed at investigating the causes and evaluating the level of this interaction are also discussed. Results of the three halfscale RC frames representing a moment frame (F1) with moderate ductility, and two braced frames (FX1 and FX2) already tested under cyclic loading are used for this purpose. These results are also used as basis for developing and calibrating numerical models of full-scale frames. Using the numerical models, a parametric investigation is carried out to determine the role of the main variable parameters affecting the level of capacity interaction between the RC frame and the bracing system.

## **3.1. Experimental Brace-Frame Capacity Interaction**

To investigate the level of interaction in the tested model frames, the corresponding forces in the bracing systems alone were evaluated by considering the relevant test displacements on the diagonals. A simple bilinear model for steel, which accounts for cyclic effects, was assumed and used to represent the force-deflection envelop curve of bracing system alone. The envelop curve of the calculated force-drift relationship for the FX1 bracing system alone (marked as No. 2 in the Figure) is plotted in Figure 8. Also plotted in this Figure, for comparison, are the experimental envelop of the force-drift relationship of the moment frame alone, F1, (marked as No. 1 in the Figure) and the experimental envelop of the force-drift curves of the FX1 braced frame. To be able to gain an insight into the level of capacity interaction between different elements, the envelop curves of the bracing system alone (2) and the moment RC frame (1) are added together to obtain the sum strength capacity of the two elements as also are presented in Figure 4 ((1) +(2)). By comparing the sum strength capacity of the two constituent elements with the actual strength capacity of the braced frame, it is evident that the actual braced frame exhibits a larger capacity than the sum of the capacities of the two elements. This means that by adding a bracing system to an RC frame, the capacity of the RC frame is increased beyond the capacity of the bracing system. The capacity interaction for the frame FX1 is measured, as the minimum of all the evaluated values, as 8.5 percent. It should be noted that the dimensions and reinforcement details and therefore the flexural capacities of the RC frames in F1 and FX1 models are the same. This enables us to make a viable capacity interaction comparison as discussed above. Considering the experimental results, it is evident that the capacity interaction is an overstrength which can be attributed mainly to the effects of brace-frame connections in reducing the effective lengths of the RC beams and columns, hence increasing the stiffness and strength of the frame.

# 3.2. Numerical Evaluation Of Overstrength

To investigate the level of connection overstrength in full-scale X-braced RC frames, nonlinear pushover numerical analyses of the moment frame, braced





frames and the bracing systems were carried out. The OpenSEES (Open System for Earthquake Engineering Simulation) program was utilised to numerically model the frames. Details of the numerical models and the numerical analyses are given elsewhere [19]. The numerical models were calibrated and their accuracy ascertained by comparing the results of the nonlinear cyclic analysis of the moment frame F1 and the braced frame FX1 with the results obtained from their respective cyclic tests.



Figure 8. Capacity interaction caused by connection overstrength

After calibrating the numerical models, a series of nonlinear pushover analyses were conducted on full scale 2-D frames of different heights and widths with different bracing conFigureurations. These included frames, 4, 8 and 12 storeys high and 3, 6 and 9 bays wide. The number of braced bays in each frame was also made a function of the number of bays such that the three, six and nine-bay frames had, respectively, one, two and three bays braced. All frames consisted of 3m high and 5m wide unit frames. Another variable parameter in this investigation is the apportioned share of bracing system from the applied loading. Load shares of 30%, 50%, 80% and 100% for bracing system are considered. As it was mentioned earlier, the main factor contributing to the interaction is the effect of connections on reducing the effective lengths of beams and columns. Therefore, considering the nature of this interaction, a representing parameter can be introduced as the ratio of the effective stiffness of the RC frame with brace-frame connections  $(K_r)$  to the stiffness of the RC frame without the brace-frame connections  $(K_i)$  and designated as  $\rho$ . Considering that the connections reduce the effective lengths of RC beams and columns, the effective stiffness of the frame with brace-frame connections corresponds to the stiffness of a reduced frame as shown in Figure 9. For simplicity and conservatively, the reduced frame is assumed to have beams and columns of lengths equal to the distances between the centroids of the four gusset plates as seen in Figure 9. Also, for practical purposes, the parameter  $\rho$  is calculated as the ratio of the linear stiffness of the reduced RC frame of a central floor  $(K_r)$  and the linear stiffness of the initial RC frame of the same floor  $(K_i)$ , also shown in Figure 9.



Figure 9. Reduced frame for calculation of connection overstrength

The stiffness ratio,  $\rho$ , as described above was calculated for all the frames analysed. The overstrength factors, R, previously determined for these frames with different problem variables were plotted against the stiffness ratio for different frame geometries considered. To condense the results of the 9 relations thus obtained, the linear relation for the 4-storey, 3-bay frame is considered as the base overstrength,  $R_{b}$ , and the effects of the two main variable parameters including the number of braced bays (number of bays in the frame) and the number of storeys are considered respectively as correction factors  $\alpha$  and  $\beta$ . Therefore;

$$R = \alpha \beta R_{b} (\%)$$
(1)  
$$R_{b} = 32\rho - 27$$

where,

In order that quantitative relations can be drawn between the factors  $\alpha$  and  $\beta$  and the stiffness ratio  $\rho$ , the former parameters are plotted against the latter in Figure 10.a and Figure 10.b, respectively. Noting the near linear variation of  $\alpha$  against  $\rho$ the following relations can be presented for this correction factor;

$$\alpha = 0.16m + 0.84, \text{ for } 0.0 < \rho \le 1.0$$
  

$$\alpha = 0.09m + 0.91, \text{ for } 1.0 < \rho \le 1.25$$
(2)  

$$\alpha = 0.06m + 0.94, \text{ for } 1.25 < \rho \le 1.40$$

Also, as the variation of  $\beta$  with  $\rho$  is small, this correction factor can be presented independent of the stiffness ratio in the following form;

$$\beta = 0.0425n + 0.84 \tag{3}$$

In equations (2) and (3), m and n are the number of braced bays and the number of storeys, respectively.



Figure 10. Variation of parameters  $\alpha$  and  $\beta$  with respect to stiffness ratio  $\rho$ 

## 4. FORCE-RELEASE DEVICES

#### 4.1. Knee Bracing

Knee bracing is used in steel construction to increase the ductility and to increase the seismic performance of the frames. Parallel to the work carried out on the pushover tests of model frames F1-P, F2-P, FB1 and FB2 described in section 2.1, two identical RC frames braced with knee-bracing system were also constructed. The RC frames of these models were identical to the unbraced and X-braced frames and the brace dimensions were also identical to the bracing system of the X-braced frames; the only difference being the four knee elements used at the ends of the diagonal bracing. Details of the bracing system and test set-up and the test results are given elsewhere [13]. The object of the tests was to investigate the role of knee bracing in increasing the ductility of the dual system while maintaining the strength and stiffness requirements. Tests similar to that described in 2.1 were conducted on these frames (Figure 11). The ultimate capacities of the knee-braced frames were found to be 2.5 times that of the unbraced frame. In Figure 12 a comparison is made between the three unbraced, X-braced and knee-braced frames regarding their stiffness and toughness. It is evident that the knee bracing has enabled the frame to possess considerable capacity and stiffness with good capacity to absorb energy. By extracting the ductility ratio from the pushover curves of knee-braced frames as around  $\mu = 2.2$ , it becomes evident that knee-bracing has also substantially increased the frames ductility compared to the ductility of the X-braced frames ( $\mu = 1.5$ ).



Figure 11. Test set-up for the knee Braced RC frame



Figure 12. Comparative pushover curves for the knee-braced frame





### 4.2. Compression Release Tool

In this section another force release tool is presented and its performance is evaluated. This novel tool, named a 'compression release tool' (CRT) when installed in a brace member, releases its compressive force. The proposed CRT is shown in Figure 13. It is composed of two steel plates separated by a gap. The two plates are to be attached together with a maximum of four bars. A cylindrical steel pipe (cylinder) is attached to one of the plates. A steel rod (piston) is attached to the second plate. The cylinder is padded with rubber material. A typical brace member can be divided into two pieces; each is to be welded to one of the CRT steel plates. When this member is subjected to a compressive displacement, the piston will slide inside the cylinder and thus the member will not have any compressive stresses. When it is subjected to a tensile displacement, the bars will transfer the tensile force between the two brace pieces. The bars should be chosen such that the sum of their yield resistances is less than the yield resistance of the brace member. Following a strong earthquake, the brace member is expected to be easily retrofitted by replacing the bars.



Figure 13. Schematic detail of the CRT

Parallel to the experimental work carried out on the X-braced model frames, constructed for cyclic loading as described above, an experimental study was also conducted to evaluate the effectiveness of the CRT. Two, similar half scale RC frames were constructed and the CRT installed. The CRT can be installed anywhere along the brace member. For the tested specimens, it was decided to install the CRT at the location shown in Figure 13. The size of the steel plates in the CRT was chosen to be  $120 \times 120 \times 10$ mm. The expected axial deformation in the brace members were calculated and based on that it was decided that a 135 mm gap between the steel plates of the CRT is required. To create this gap, the length of the cylinder and the piston were chosen to be 135mm. The inner diameter and wall thickness of the cylinder were chosen to be 40 mm and 5 mm, respectively. The piston was chosen to be 35mm steel rod. The bars connecting the steel plates were different in specimen FXS1 than those in specimen FXS2. They were two-12.7mm

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and two-16mm steel bars in specimens FXS1 and FXS2, respectively. Tensile load tests on the steel rods revealed that their yield stress is 350 MPa. A photo of an installed CRT is shown in Figure 13. Details of the test specimens and test set-up and results are given by Ghaffarzadeh and Maheri [17].

The frames with CRT (FXS1 and FXS2) were tested under cyclic loading the same way as the moment frames (F1 and F2) and the X-braced frames (FX1 and FX2). The seismic parameters evaluated from the test results include; stiffness degradation, energy dissipation capacity (toughness) and ductility. A discussion of the test results is given as follows:

The lateral load-deformation response for specimen FXS1 indicates the formation of first plastic hinge at a drift level of 1.2%. This was due to the yielding of the two-12.7mm steel bars joining the steel plates of the CRT. This happened at the lateral load of 75kN. The frame failed at the drift of 4.8% corresponding to lateral load of 182kN due to tensile failure of the two-12.7 mm bars. The behaviour of specimen FXS2 was similar to that of specimen FXS1. Yielding of two-16 mm steel bars in the CRT occurred at a drift of 2.5% (lateral load of 140kN). By increasing drift, cracks became visible. Strains in the top reinforcement of the top beam indicate that steel yielded at a drift of 3.4%. The test was terminated because of localized concrete failure in the vicinity of the supports.

#### **Stiffness Degradation**

The initial stiffness of specimens FX1 and FX2 was higher than that of the specimens FXS1 and FXS2. This is a direct result from the lower elastic stiffness of bracing members equipped with CRT. The steeper degradation in the lateral stiffness observed in specimens FX1 and FX2 however indicates that using the CRT minimized the cracking in the RC frame and kept the lateral stiffness of the frame almost constant.

## **Energy Dissipation Capacity (Toughness)**

The cumulative energy dissipated by the frames after 5, 10, 15, 20 and 25 cycles were also calculated. It was noted that, at lower displacements, the energy dissipated by the braced frames with the CRT (specimens FXS1 and FXS2) is somewhat less than that of braced frames without the CRT (specimens FX1 and FX2). With increasing displacements and as the bars in CRT yield, the energy dissipated by the frames with CRT is increased to levels higher than those of the frames without the CRT. This indicates that the installation of the CRT did not, by and large, affect the energy dissipation capacity of the braced frames.

#### Ductility

The available ductility of the four specimens is given in Table 7. It can be observed in this Table that the overall behaviour of the specimen with CRT (specimen FXS1) is more ductile in comparison with specimen FX1 without CRT. The sudden drop in load-drift response curve of specimen FX1 after buckling of compression brace indicates a brittle behaviour. However, in specimen FXS1, in which buckling is inhibited and failure happens by yielding of steel bars of CRT,



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the behaviour is evidently more ductile (almost two folds). This shows the effectiveness of the CRT in increasing the ductility of the braced frame. By comparing the results of the stronger braced frames without CRT (specimen FX2) and with CRT (specimen FXS2), the favourable effect of the CRT on the ductility of the frame can also be noted.

Test Specimen	Δ <sub>y</sub> (mm)	Δ <sub>max</sub> (mm)	μ	V <sub>s</sub> (kN)	<i>V<sub>y</sub></i> ( <b>k</b> N)	V <sub>e</sub> (kN)	$R_{\mu}$	$R_s$	R
FX1	22.5	47.5	2.11	105	112	284	2.53	1.06	2.68
FX2	33.0			150	168	352	2.09	1.12	2.34
FXS1	17.5	71.5	4.08	75	118	296	2.51	1.57	3.94
FXS2	35.0			134	160	324	2.03	1.19	2.42

Table 7: Ductility and performance factor parameters of the CRT test specimens

# 5. CONCLUSIONS

The results of the experimental and numerical investigations presented in this paper lead us to the following conclusions;

- 1. Internal bracing of RC frames with direct brace-frame connections is not only suitable for seismic retrofitting of existing building but it can also be used as a viable alternative to shear walls as shear resisting elements for the newly constructed buildings.
- 2. 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 less favourable for a ductile design.
- 3. The proposed CRT can be effectively used in steel bracing systems to eliminate buckling failure. Its use will also result in an adequate energy dissipation capacity for the brace-frame system.
- 4. The inclusion of CRT can also greatly enhance the ductility of the braced frame. The desired level of ductility can be achieved by appropriate design of the CRT bars.
- 5. To increase the ductility and maintain the strength and stiffness capacities of the braced frames, Knee bracing of the frame or using CRT on the brace members is recommended. Such systems can 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.
- 6. The overstrength in a braced RC frame is due to the stiffening effects of connections. This overstrength is termed the capacity interaction or connection overstrength. It is significant and needs to be considered in design.
- 7. Presentation of the connection overstrength in the form of a frame stiffness ratio,  $\rho$ , enable us to use the results and formulations presented here for other types of concentric and eccentric bracing systems.

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