SEISMIC BEHAVIOR OF REINFORCED CONCRETE FRAMES RETROFITTED BY PRESTRESSED STEEL STRIPS

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ABSTRACT

Failure of RC columns under a combination of axial compressional load and seismic moments could result in the collapse of RC frames during severe earthquakes. This paper presents a new technique for retrofitting RC columns by wrapping them with pre-stressed steel strips. Previous experimental studies have proved this technique to be effective and promising. A previously introduced stress-strain model is used to obtain moment-curvature diagrams for retrofitted columns using a nonlinear analysis. Subsequently, these diagrams were employed in nonlinear pushover analyses to study the seismic performance of retrofitted RC frames as compared with the normal frames. The results indicate that this retrofitting approach enhances the seismic performance markedly by increasing the ductility of the columns, and hence, reducing the level of damage. In particular, this retrofitting technique replaced the undesirable failure of the columns by the rather more desirable failure of beams.

INTRODUCTION

In buildings that are designed according to specifications of former codes, or do not comply with the current codes, it is possible that deficiencies like inadequate transverse reinforcement and poor reinforcement detailing may lead to undesirable phenomena such as shear failure and axial failure of columns by occurrence of earthquakes. Hence, upgrading of these building, especially their columns is important. Various approaches to retrofitting of RC buildings has been introduced thus far and used like shear walls, wrapping of columns by FRP sheets, jacketing, etc. Each of these methods has its own advantages and disadvantages. One of the most important criteria in the implementation of these methods is their cost-effectiveness.

One of common methods of retrofitting in seismic regions is actively or passively wrapping of columns. This method increases ductility and rotation capacity of columns and as a result, increases overall ductility of buildings.

Wrapping of columns with pre-stressed steel strips is a new technique that is economic and can be easily implemented. This technique was invented at the Sheffield University by Frangou and Pilakoutas (1995). Commercially available strapping tensioners and sealers make it easy to post-tension the strip and fix the strip ends in the seals. The strips can be tensioned about 30% of their yield stress. Hence, an effective lateral stress is applied to the column prior to loading. This has many benefits such as full utilization of the strip capacity and prevention from premature crushing of confined concrete. (Frangou and Pilakoutas., 1995).

Based on experimental evidence, actively confinement of small scale concrete columns by pre-stressed steel strips enhances the ductility and ultimate compressive strength of the retrofitted ones. (Moghaddam et al., 2010) It can also be shown that in large scale experimental results, retrofitting with pre-tensioned steel metals enhances the rotational capacity of RC columns, decreases the level of damages and delays bar buckling.(Moghaddam et al., 2011)

In the Figs. 1.a and 1.b final state of damages in both unretrofitted and retrofitted specimens CP2 are shown.

a.  b. 

Fig. 1. Final state of damages of a. control column CP2-0 and retrofitted column CP2-2

STRESS-STRAIN BEHAVIOR OF RETROFITTED COLUMNS BY STEEL STRIPS

For analysis of columns retrofitted by pre-stressed steel strips, determining axial behavior of concrete confined by steel strips is essential and for this purpose the proposed model by Moghaddam et al (2010) may be used. Based on experimental results of small scale compression columns, Moghaddam et al (2010) proposed the following second order polynominal equation to achieve pre-yield branch of stress strain curve.

$$Y = \frac{f_{cc} - f_{cr}\dfrac{\varepsilon_{cc}}{\varepsilon_{cr}}}{\varepsilon_{cc}^2 - \varepsilon_{cc}.\varepsilon_{cr}} X^2 + \left(\frac{1}{\varepsilon_{cr}}\right)\left[f_{cr} - \left(\frac{\varepsilon_{cr}^2}{\varepsilon_{cc}^2}\right)\frac{f_{cc} - f_{cr}\dfrac{\varepsilon_{cc}}{\varepsilon_{cr}}}{1 - \dfrac{\varepsilon_{cr}}{\varepsilon_{cc}}}\right]X \tag{1}$$

In the equation above, X, Y, f_{cc}, ε_{cu}, f_{cr}, ε_{cc}, and ε_{cr} are strain, stress, ultimate stress, ultimate strain, critical stress and critical strain, respectively. The values of f_{cr}, ε_{cr}, f_{cc} and ε_{cc} are obtained from the equations 2 to 5.

$$f_{cr} = 0.85 f_{cc} \tag{2}$$

$$\varepsilon_{cr} = 3.1\times10^{-4}\frac{f_{cr}}{\sqrt{f_{co}}} \tag{3}$$

$$\frac{f_{cc}}{f_{co}} = 1 + 8\frac{f_{le}}{f_{co}} - 4\left(\frac{f_{le}}{f_{co}}\right)^{1.2} \tag{4}$$

$$\varepsilon_{cc} = \varepsilon_{co}\left(\frac{f_{cc}}{f_{co}}\right)^{1.1} \tag{5}$$

It should be noted that in the equations above, f_{co}, and ε_{co} are ultimate stress and strain of unconfined concrete respectively, and f_{le} is the effective confining pressure based on the method proposed by Mander et al. (1988).

For the post-yield branch, Moghaddam et al (2010) used a straight line with slope of α which is obtained from the following equation:

$$\alpha = -62300\left(\frac{f_{le}}{f_{co}}\right)^2 + 31150\left(\frac{f_{le}}{f_{co}}\right) - 3900 \tag{6}$$

Finally, for determining the ultimate point of stress-strain curve that coincides the rupture of steel strips, ε^*_{ult}, the ultimate strain of compressive concrete, ε_{ult}, can be calculated from Eq. (7).

$$\varepsilon_{ult} = 0.003e^{\left(160\frac{f_{le}}{f_{co}}\varepsilon^*_{ult}\right)} + 1.3\frac{f_{le}}{f_{co}}\varepsilon^*_{ult} \tag{7}$$

STRESS-STRAIN BEHAVIOR OF CORE CONCRETE

In order to analyze the axial behavior of the core of columns, a superposition-based method proposed by Moghaddam et al. (2008) was applied. The stress-strain state of columns was obtained through the integration of the nonlinear uniaxial stress-strain response of the individual fibers into which the section has been subdivided. The column section was divided into two regions cover and core. At the first step, the whole column section was assumed to be confined by only steel strips and its analytical stress-strain behavior was obtained by using the model presented by Moghaddam et al (2008). Then, confinement effect of transverse reinforcement on the core was calculated by assuming the behavior obtained from the first step for stress-strain behavior of cover and the unconfined behavior for core of the retrofitted column by using Mander et al. (1988)یs theorical stress-strain model.

MOMENT CURVATURE AMALYSIS AND PERFORMACE POINTS OF RETROFITTED COLUMNS

For analysis of reinforced concrete columns, fiber analysis is considered. In this method, any section of column is subdivided to some fibers and by assuming uniaxial behavior for each fiber, the moment-curvature of column is calculated. It should be noted that in a common column usually three types of fiber considers: cover concrete, core concrete and longitudinal reinforcement. For this purpose the software of XTRACT is employed.

In order to determine the performance parameters of retrofitted columns, two low and high confinement ratios were considered. For highly confined columns the ultimate point was determined based on crushing of the concrete cover which means that the strain of core concrete has reached to the ultimate strain demonstrated in Eq. (8). In case of retrofitted columns lowly confined by steel strips with regards to having a high effect of transverse reinforcement, the ultimate point was determined based on the reaching of core strain to the ultimate strain of core concrete confined by steel strips (Eq.7) plus the ultimate strain of core concrete confined by transverse reinforcement that is calculated from the equation proposed by Priestly et al. (1996) shown in Eq. (8).

$$\varepsilon_{cu1} = 0.004 + \frac{1.4\rho_s f_{yh}\varepsilon_{su}}{f'_{cc}} \tag{8}$$

where ε_{cu1} is the ultimate compressive strain, ρ_s is the volumetric ratio of confining steel, f_{yh} is the yield strength of transverse reinforcement, ε_{su} is the maximum strain of transverse reinforcement and f'_{cc} is the peak compressive strength of confined core concrete.

By using fiber approach and considering the axial behavior of each fiber, the moment-curvature behavior of columns was calculated. It should be noted that for the axial behavior of longitudinal reinforcement the experimental results of Moghaddam et al. (2011) was used. Finally, the moment-curvature curves was converted to the bilinear curves. The method of converting was based on equivalent areas of two main and converted curves and also intersection of bilinear and original curves in the yielding of longitudinal reinforcement.

Finally, according to the Eq. (9), the retrofit parameter a is calculated. In the mentioned equation, L_p (Eq. 10) is the plastic hinge length proposed by Priestley et al. (1996) and Φ_u and Φ_y are yield and ultimate curvatures respectively.

$$a = (\phi_u - \phi_y) L_p \tag{9}$$

$$L_p = 0.08L + 0.022 f_y d_b \tag{10}$$

In the Eq.(10), L is the critical distance from the critical section of the plastic hinge to the point of contraflexture, f_y and d_b are respectively the yield strength and the diameter of longitudinal reinforcement.

In Fig. 2, the moment-rotation curves of analytical and experimental results of actively retrofitted columns experimented by Moghaddam et al. (2011) are shown. Based on the shown results it can be said that the analytical method is practical in determining behavior of actively lowly confined columns (column CP1-1 that strapped with 1 layer, 150 millimeter spaced steel strips) and highly confined columns (column CCP2-2 that strapped with 2 layer, 33 millimeter spaced steel strips) retrofitted by pre-stressed steel strips.

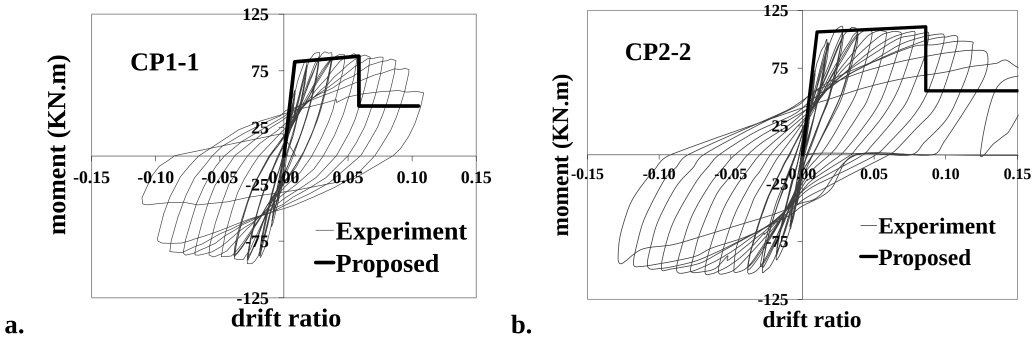

Fig. 2. Analytical moment-rotation results of actively a. highly and b. lowly confined of retrofitted columns

DETERMINATION OF ACCEPTANCE CRETARIA

By calculation of the ultimate rotation capacity of an element, acceptance criteria are defined. In this study, 3 points of acceptance criteria are defined: Immediate occupancy (IO), life safety (LS) and collapse prevention (CP) corresponding to 10%, 60%, and 90% of ultimate rotation (point C). In Fig. 3 the definition of acceptance criteria is shown.

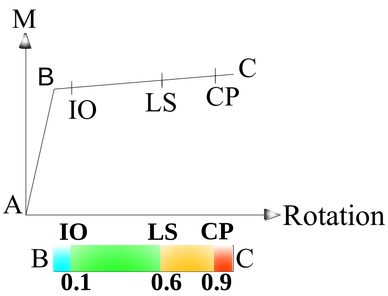

Fig. 3. Moment-Rotation of atypical plastic hinge

DESIGN ASSUMPTIONS

For investigation of effects of column retrofitting, three and six story frames are considered. The details of the considered frames are shown in Fig. 4. Properties of longitudinal and transverse concrete reinforcements are considered similar to those of experiments that were carried out by Moghaddam et al. (2011) and are shown in table 1. Ultimate stress of non-confined concrete was assumed to be equal to 254 kg/cm^2. The distributed gravity and live loads on the beams were chosen 3900 and 1200 kg/cm^2 respectively.

To determine the amount of longitudinal bars of beams and columns, the Iranian standard code 2800 (2008) by assuming a high seismic risk region (base acceleration equal to 0.3g) was used. The size and amount of longitudinal and transverse steel beams and columns are specified in Fig. 4 and table 2. It is worth noting that the effect of shear deformation and shear failure are not included in the analysis.

The columns of the first story at both three- and six-story frames was assumed to be strengthened by pre-stressed steel strips. Two retrofitting methods was considered, lowly and highly confined by steel strips in order to account the amount of confinement at retrofitting. The number and space of strip layers were chosen based on the experimental work performed by Moghaddam et al. (2011). Retrofitting details of steel strips are shown in table 3. It is also assumed that the length of retrofitting straps is sufficient and retrofitted columns have a flexural behavior. Strap yield strength and ultimate strain was considered equal to 8584 kg/cm^2 and 6.5 percent, respectively. Pre-stressing amount of straps was considered equal to 30 percent of the yield strength.

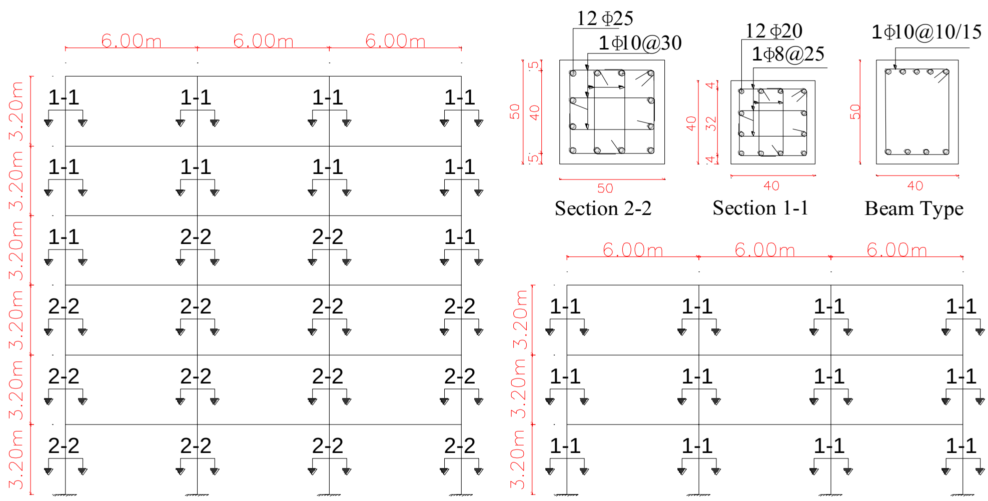

Fig. 4. Three and six story frame details

Table 1. reinforcement details

Reinforcement type	Yield Strength	Ult Strength	Yield strain	Ult Strain
Longitudinal	5377	6981	0.0029	0.163
Transverse	5821	6128	0.0028	0.063

Table 2. Three and six story beam longitudinal reinforcement detail

	3 Story Beam Reinforcement		6 story beam Reinforcement	
story	**Top Reinf**	**Bot Reinf**	**Top Reinf**	**Bot Reinf**
1	4D25	3D18	5D25	4D20
2	4D22	3D18	6D25	5D20
3	4D20	3D18	6D25	5D20
4	-	-	5D25	4D20
5	-	-	4D25	3D18
6	-	-	4D20	3D18

Table 3. Retrofitting method details

Retrofit Method	Number of layers	Space of Steel Strips(cm)
Low Confinement-C40	2	20
Low Confinement-C50	3	3.2
High Confinement-C40	2	18
High Confinement-C50	4	3.3

ANALYSIS

Moment-curvature diagrams of the columns and beams was obtained using the assumptions mentioned previously, and then, they were converted to the bilinear curves, and finally, the modeling parameters were derived. It should be noted that due to differing amount of top and bottom longitudinal reinforcement in beams, the plastic hinge length vas assumed 35 cm. For the seismic weight of frames the sum of dead load plus 20% live loads was considered. For analyzing the frames the program PERFORM3D-5 was used.

The nonlinear push-over analysis was carried out in the frames. The plastic behavior of each element was considered to be concentrated in plastic hinges located at the two ends of the beams and columns. The remaining length of the element was assumed to be elastic (i.e., a lumped plasticity model was used). The lateral force was exerted by a first mode load pattern. To determine target displacement, the ASCE41-06 (2007) codeِs displacement modification was used. Two confinement levels in beams were also assumed to consider the effect of beam ductility in retrofitting: moderate and high ductility beams with 15cm and 10cm space of transverse reinforcement, respectively.

RESULTS AND DISCUSSION

Figs. 5 and 6 display the typical base shear-roof drift curves for different types of column retrofitting and beam ductility. It is observed that by retrofitting the columns of first floors of both three- and six-story frames, the ductility increases. High confinement of columns and increasing the ductility of beams both increase the enhancement of ductility. It is also evident that low retrofitting has a slight effect on increasing base shear whereas highly retrofitting method enhances the base shear noticeably.

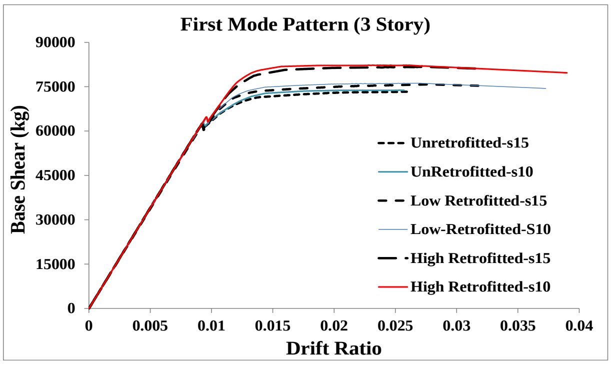

Fig. 5. Pushover curves for a three story frame using different levels of retrofitting

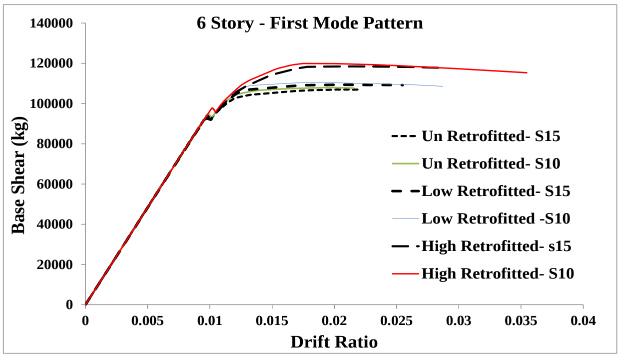

Fig. 6. Pushover curves for a six-story frame using different levels of retrofitting

DISTRIBUTION OF PLASTIC HINGES AND ACCEPTANCE CRETARIA

In order to investigate the distribution of plastic hinges and acceptance criteria, plastic rotations and hence capacity ratios of beams and columns that was previously defined are shown in Figs. 7 to 10.

The investigation is performed in two points, target displacement and ultimate that is defined as a reaching of one element to the ultimate point C shown in Fig. 3.

Figs. 7 and 8 show the capacity ratios of three and six story frames at target displacement. It is observed that capacity ratios of the first floor of the frames are decreased by retrofitting and this shows the reduction in damages of retrofitted columns. In other elements of the frames the capacity ratios remain almost instant and it seems that the retrofitting method has a slight effect on performance of other elements of frames. Furthermore, it has been seen that frames with different confinement ratios of retrofitted columns and also confinement ratios of beams have a similar patterns of distribution in their plastic hinges and capacity ratios. Based on this similarity, in this paper only the results of frames with moderate ductile beams and frames retrofitted by highly confined steel strips are shown.

By investigating the Figs. 9 and 10, which show the capacity ratios and distribution of plastic hinges at ultimate point (i.e. the point that first element reaches to its lateral failure), it can be said that retrofitting of the columns of the first floor prevents the undesired failure of columns and this retrofitting method transmits the failure of columns to beams. It is also obvious that the capacity ratios have decreased in retrofitted columns of first floor and hence, damages of retrofitted columns have reduced noticeably.

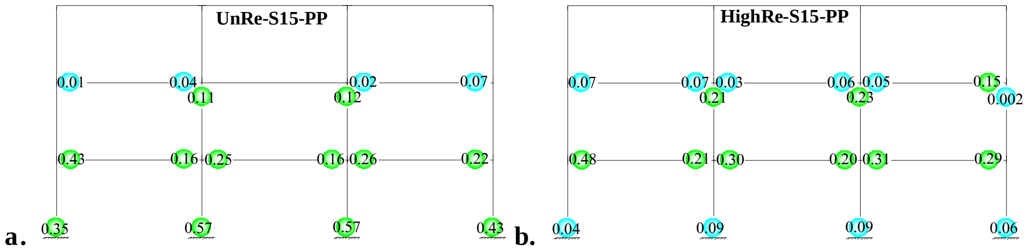

Fig. 7. Distribution of plastic hinges in the a. un-retrofitted and b. highly retrofitted 3-story frame at the target displacement

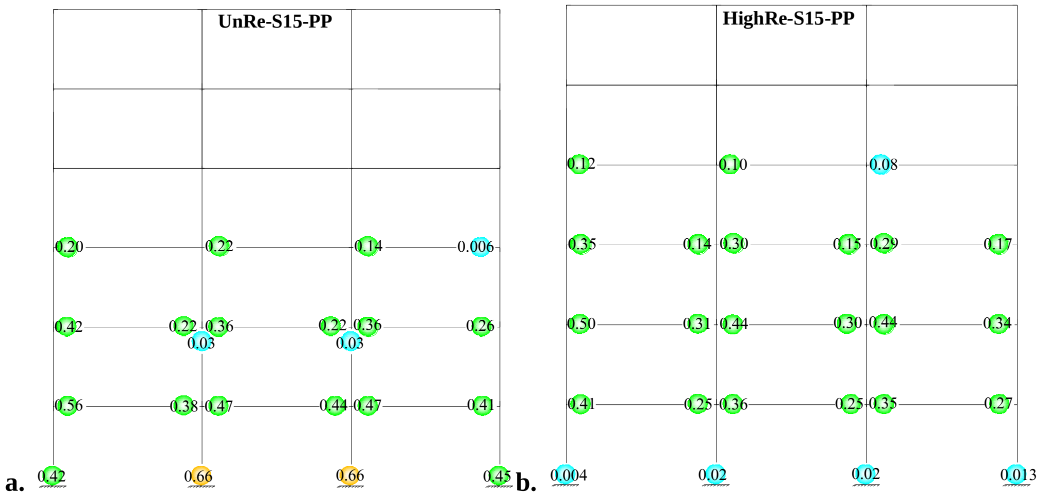

Fig. 8. Distribution of plastic hinges in the a. un-retrofitted and b. highly retrofitted 6-story frame at the target displacement

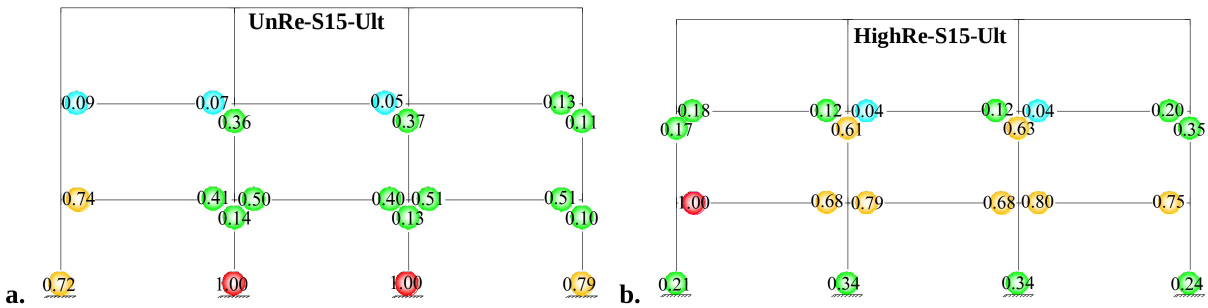

Fig. 9. Distribution of plastic hinges in the a. un-retrofitted and b. highly retrofitted 3-story frame at the ultimate point

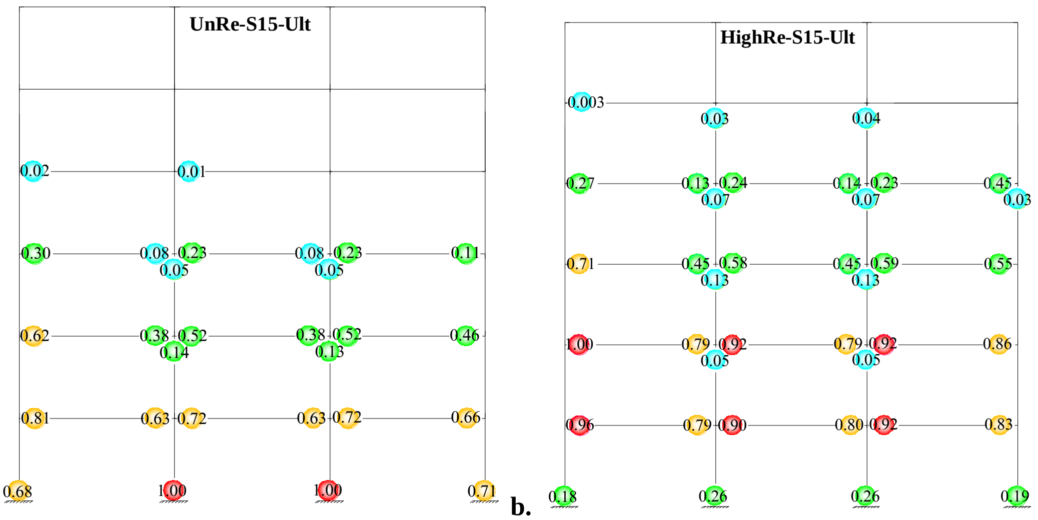

Fig. 10. Distribution of plastic hinges in the a. un-retrofitted and b. highly retrofitted 6-story frame at the ultimate point

SUMMARY AND CONCLUSIONS

In the present study, three- and six-story frames were considered and the columns of their first story were retrofitted by actively confined steel strips. Nonlinear pushover analysis was used to examine the seismic behavior of unretrofitted and retrofitted frames. Based on the results of nonlinear analysis, an increase in the ductility of retrofitted frames in comparison with un-retrofitted ones was observed. By using retrofit method, retrofitting capacity ratios of plastic hinges in the columns of the first story at the performance point reduced and the results reflected lesser amounts of damages. By defining the lateral failure of the first member as an ultimate point, this retrofitting technique replaced the undesirable failure of the columns by a rather more desirable failure of beams.

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