Seismic upgrade
of a reinforced concrete building

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Abstract

This paper discusses results from an analytical investigation conducted to study the ductility demands imposed on reinforced concrete beams and columns of three-bay six-story reinforced concrete frames subjected to earthquake type loads. The ultimate goal of this study is to investigate and compare the behavior of Frame-NS, the ordinary moment resisting frame, and Frame-S, the special moment resisting frame designed to comply with the seismic design requirements of ACI 318-99\textsuperscript{[1]}, UBC-97\textsuperscript{[2]}, CAN A23.3-94\textsuperscript{[3]} and NBC-95\textsuperscript{[4]}. Determination of the critical locations in these frames and devising upgrade schemes that are consistent with the ductility demands on these locations constituted significant milestones in the seismic retrofit design. Although Frame-S displayed a higher level of ductility in comparison to Frame-NS, the pushover analyses results indicated that the most critical column hinges of Frame-S exploited their deformation capacities just prior to collapse. The most critical beam hinge, on the other hand, exploited 37\% of its deformation capacity. More significantly, only 21\% of the beam deformation capacities were exploited in the Frame-NS just prior to its collapse. The potential plastic hinge regions of the columns of Frame-S and Frame-NS were then wrapped using carbon fiber reinforced polymer fabrics impregnated with epoxy. Due to better confinement of the plastic hinge regions of the columns, their sectional deformation capacities increased considerably. Hence, the available deformation and energy absorption capacities of the beam hinges were mobilized to considerably higher levels (up to 48\% in Frame-NS and up to 75\% in Frame-S).
1 Introduction

Current design provisions of the North American Codes [1,2,3,4] provide guidelines for the analysis, design and detailing of buildings in earthquake prone regions. One of the objectives of the building codes is to avoid catastrophic failure and to minimize the life safety hazard consistent with the performance level implied in the design. In this context, seismic design is frequently performed for reduced lateral forces, and the members are detailed such that the ductility associated with the reduced base shear can be supplied. Damage such as crushing of concrete and yielding of steel at certain locations is allowed to meet the design objective criteria. Reinforced concrete moment resisting frames are one of the most common types of lateral force resisting structural systems in the western U.S. and Canada. In the seismic design, formation of plastic hinges is allowed, and in fact relied upon to dissipate the energy. In addition, the “strong column-weak beam” concept is recommended by the aforementioned codes to ensure the integrity of the structure such that it can undergo large inelastic deformations without losing its vertical load carrying capacity.

In this paper, results from case studies are presented for two frames designed and detailed using the 1963 and 1999 editions of the ACI code. In the 1963 edition of the code there was no special detailing recommendations for seismic resistant design. The frame designed according to the 1963 edition of the ACI Code is referred to as Frame-NS. The frame designed to satisfy the seismic resistant design provisions of ACI 318-99 is referred to as Frame-S. In the pushover analyses, both frames were subjected to incremental lateral loads. Redistribution of moments upon plastification and hinging sequences were studied under increasing lateral deformations. Case studies focused on determining the effectiveness of CFRP wrap systems to improve the seismic response of both the frames. Based on the analyses results, the performance level of upgraded Frame-NS is compared to the performance level of Frame-S.

2 Frame-NS and Frame-S prior to Upgrade

Figure 1 shows the plan and elevation of the 6-story reinforced concrete office building used for this study. The building utilized moment resisting frames in both directions and had 7-6m bays in the N-S direction and 3 bays in the E-W direction, consisting of 2-9m office bays and a central 6m corridor bay. The interior columns were 500x500mm while exterior columns were 450x450mm. The slab floor system consisted of a 110mm thick slab and the beams of both the N-S and E-W frames were 400 x 600mm. The design strength of concrete for all of the members was 30MPa, and the yield strength of steel is 400MPa both for flexural and transverse reinforcement. Frame-S [5] was designed according to CSA A.23.3-94 [3] and NBC-1995 [4] requirements. The design of the frame also met the requirements of a special moment resisting frame for 1997 UBC Zone 4 [2] and ACI 318-99[1]. The geometry and member dimensions of Frame-NS and Frame S were the same. The stirrup spacing of the
columns and the beams were considerably higher as they had to satisfy the requirements of ACI 318-63. This frame would be classified as a non-ductile moment resisting frame according to the current code provisions [1,2,3,4].

The analyses of the building were performed for a typical interior frame in the E-W direction using DRAIN-2DX [6] and TEMPEST [7] programs. In the DRAIN-2DX analytical models, columns and beams were represented with frame elements that had rotational springs at the ends. The beam column joints were modeled by rigid offsets at the ends of these elements. Moment curvature relations for rotational springs at each beam and column end were obtained using the sectional analysis program SECRES [8]. Sheikh and Uzumeri confinement model [9] was employed in SECRES to predict the behavior of column sections. Plastic rotation capacities of the hinge regions were computed using SECRES, which incorporates the reinforcing cage-concrete core interaction as outlined by Bayrak and Sheikh [10]. The gravity loads due to the dead and sustained portion of the live loads were included in the analyses and hence the deformation capacity of each column hinge was different due to the varying level of axial load. For a given axial load, the behavior of a column or a beam hinge was obtained by evaluating the sectional response of each column and beam hinge concurrently with pushover analyses and integrating the curvatures in the plastic hinge regions. The frames were subjected to earthquake induced lateral forces in an incremental manner. The plastic rotations at the member ends, i.e. at the potential plastic hinge regions, were monitored and compared with the available rotation capacities corresponding to the axial load.
at each analysis step. The analyses were continued until the ultimate member deformation capacities were reached and the models were adjusted to incorporate the failed elements. The results obtained from DRAIN-2DX were compared with those obtained from TEMPEST. For Frame-S, DRAIN-2DX underpredicted the maximum deformation capacity of the frame—obtained by using TEMPEST—by about 5% and overpredicted the maximum base shear force by about 3%. In this comparative analysis the confinement effects of the ties were neglected. It was then concluded that the use of DRAIN-2DX concurrently with a sectional analysis program (SECRES), which can accurately model reinforcing cage concrete core interaction, would result in reliable predictions. It is important to note that TEMPEST was specifically developed and experimentally verified [7] for the nonlinear analysis of reinforced concrete plane frames. As a result, it incorporates very complex material and geometric nonlinearities into the analysis. In fact, that is the reason why the results from TEMPEST were used as benchmark values. However, although the computer program TEMPEST employs rational constitutive models for the behavior of reinforced concrete, it was not developed to model the confining effects of ties and CFRP wraps. The computer Program SECRES [8], on the other hand, is specifically developed for accurate modeling of reinforcing cage concrete-core interaction and modeling of confinement due to ties and FRP wrap systems. Hence, by using SECRES it was possible to obtain reasonably good behavior predictions of reinforced concrete elements wrapped with fiber reinforced polymers and to use that relationship as a backbone curve and an input parameter for DRAIN-2DX.

3 Seismic Upgrade of Frame-NS and Frame-S

Tyfo® SCH-35 reinforcing fabric and Tyfo® S Epoxy were used in the upgrade of the reinforced concrete columns. Tyfo® SCH-35 is a custom stitched unidirectional carbon fabric. The fibers were oriented in the circumferential direction, and two layers of the fabric were used to wrap the potential plastic hinge regions of the columns. The tensile strength as obtained from tensile tests conducted using the ASTM D-3039 method was 991 MPa. This commercially available product was used in the current analytical investigations since Sheikh, Iacobucci and Bayrak [11] used the same material to experimentally investigate the effectiveness of the Fibwrap® system in the seismic retrofit of square columns. Figure 2 illustrates the comparative effectiveness of stirrups and CFRP wraps.

As can be observed in Figure 2, due to the existence of the cross ties, the effectively confined area of the concrete core in the tied columns of Frame-S is reasonably high. As the CFRP wrap is applied to the exterior of the column it can only provide confining stresses along the four corners of the columns and hence the effectiveness of the confinement provided by these forces is lower than that provided by the tie arrangement shown in Figure 2. However, as the CFRP wrap is continuous over the length of the potential plastic hinge region it
Figure 2. Confinement of Concrete Due to Stirrups and FRP Wrap

will be very effective in preventing the premature buckling of the longitudinal reinforcing bars and hence improve the deformability. The computer program SECRES [8] takes into account all of these behavioral issues and provides a rational estimation to the experimentally observed response of square columns reported in the literature [11]. Analyses results of SECRES indicated that wrapping column sections by CFRP resulted in increases of the plastic rotation capacities of the sections by about 50% for the columns of Frame-S. Even more significantly, CFRP wraps increased the plastic hinge rotation capacities of the columns in the Frame-NS up to 200%. Behavior of these frames prior to and after the seismic retrofit will be discussed in the following section.

4 Results and Discussion

Pushover analysis was performed by applying the code recommended lateral loads on the frame in an incremental manner. First floor beam hinges were observed initially. Then, hinging of the upper story beams was noted along with some plastic deformation in columns. Following that, first story column hinges above the foundation level formed. Figure 3a gives a snapshot of the hinges when the overall drift was 1%.

The maximum attainable rotation capacities of the beam or column’s plastic hinges were not reached at this stage of the analysis. The state of plastic hinging at 2% drift ratio and at failure are given in Figure 3b. Failure of the frame was defined by the exploitation of available rotation capacities of several column and beam hinges and the formation of a collapse mechanism. Failure occurred at 755 mm roof displacement, which corresponds to an overall drift ratio of 3.26%. At the failure stage the most critical column consumed all of its rotation capacity, whereas the most critical beam hinge used about 37% of its plastic rotation capacity. It can be seen in Figure 3b that prior to failure, all the column hinges are concentrated at the first story level and nearly all possible beam hinges on the structure are formed.

Having observed the formation of column hinges in the first story and the exploitation of all of the available rotation capacities at the most critical column hinges, it was decided to wrap the first story columns with two layers of CFRP. The pushover analysis was then performed after incorporating the new moment
curvature relationships for the first story columns as obtained from the computer program SECRES. The sequence plastic hinge formation in the upgraded frame followed a very similar pattern at 1%, 2% and ultimate drift ratios. Additional plastic rotation capacities of the critical column sections increased the overall ductility of the frame significantly. Base shear versus drift ratio behavior of the two frames (Figure 4) shows that the roof displacement at failure increased by about 70%. Increase in lateral load carrying capacity is about 2%. It is important to note that the CFRP wrap system changed the initial stiffness of the frame insignificantly (Figure 4). In the upgraded frame 75% of the most critical beam deformation capacity was exploited -twice the amount observed in the beams of the first frame.

Frame-NS was analyzed in a similar manner. Figure 5a illustrates the plastic hinge formations at 1% drift ratio. The sequence of plastic hinging was similar to the ductile moment resisting frame. First, the beam hinges were observed in the first floor and then hinging of the beams of upper stories was noted. The most critical plastic hinge, which is just above the foundation of the first story exterior column, utilized all of its plastic rotation capacity at about 336mm roof displacement (1.45% overall drift). The plastic hinges corresponding to this stage are shown in Figure 5b. The ductility of this frame was considerably lower than the seismic resistant special moment resisting frame “Frame-S”. The failure of this structure at a smaller drift ratio was mainly because of the lack of confinement reinforcement in plastic hinge regions of the first story columns. To increase the plastic rotation capacity of the columns in the first story, two layers of CFRP wrap were applied to these members. The behaviors of the wrapped plastic hinges were obtained using SECRES [8].
Wrapping of the columns added very little to the initial response and hence the hinging sequence remained similar to the unwrapped case at 1% drift. More importantly, the failure of the columns at 1.45% overall drift was no longer observed after the Frame-NS was upgraded with the CFRP wrap system. Columns became more ductile and could sustain significantly large plastic rotations. The additional hinges shown in Figure 5b formed for the upgraded structure beyond 1.5% drift. The most critical member at the failure stage was

![Figure 4. Base Shear vs. Overall Drift Behavior of Frame-S](image)

![Figure 5. Plastic Hinge Formation in Frame-NS](image)
still the first story exterior column, and the roof displacement at failure was 865mm and the base shear was 894.7kN. The retrofit of the first story columns increased the lateral load carrying capacity of the frame about 9%. A substantial increase in the ductility of the frame after retrofit was noted (Figure 6).

Table 1 provides a summary of the analyses results. As can be seen in Table 1 and Figures 4 and 6, the application of CFRP wraps to the first story columns resulted in insignificant increases in lateral load carrying capacity of the frames and the initial stiffness of the overall load-deformation responses. The displacement ductilities calculated by dividing the ultimate roof displacement by the roof displacement at first yield show that CFRP wrapping of the first story columns is a promising technique for increasing the useful deformation capacities of deficient frames (Table 1). The seismic retrofit scheme employed herein not only increased the overall ductility, but also mobilized the unused portion of plastic rotation capacities of the beams. Although the strong column-weak beam concept was used in the design, column hinging was not completely avoidable. The available deformation capacities of the columns dictated the overall ductility of the frame, as the beam hinges had reserve deformation capacities at failure. It is important to note that the overall deformation and energy dissipation capacities of the upgraded Frame-NS are slightly better than those of the Frame-S. It is possible to upgrade a frame designed to carry gravity loads only using CFRP wraps such that the performance of the upgraded frame is equivalent to or better than a similar frame designed using the seismic provisions of the current codes [1, 2, 3, 4].

![Figure 6. Base Shear vs. Overall Drift Behavior of Frame-S](image-url)
Table 1. Summary of Analyses Results

<table>
<thead>
<tr>
<th>Type of Frame</th>
<th>Critical Member</th>
<th>(\frac{(\theta_p)<em>{fail}}{(\theta_p)</em>{avail}})</th>
<th>Base Shear at Frame Failure (kN)</th>
<th>(\mu = \Delta_u/\Delta_y)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Frame-S</td>
<td>Critical Beam</td>
<td>0.37</td>
<td>889</td>
<td>7.2</td>
</tr>
<tr>
<td>Prior to Upgrading</td>
<td>Critical Column</td>
<td>0.99</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Frame-S</td>
<td>Critical Beam</td>
<td>0.75</td>
<td>906</td>
<td>12.4</td>
</tr>
<tr>
<td>After Upgrading</td>
<td>Critical Column</td>
<td>0.98</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Frame-NS</td>
<td>Critical Beam</td>
<td>0.21</td>
<td>822</td>
<td>3.2</td>
</tr>
<tr>
<td>Prior to Upgrading</td>
<td>Critical Column</td>
<td>0.99</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Frame-NS</td>
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<td>895</td>
<td>8.2</td>
</tr>
<tr>
<td>After Upgrading</td>
<td>Critical Column</td>
<td>0.99</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

\(\Delta_u\) = Roof Displacement at Frame Failure  
\(\Delta_y\) = Roof Displacement at First Member Yield  
\((\theta_p)_{fail}\) = Plastic Rotation of the Member at Frame Failure  
\((\theta_p)_{avail}\) = Available Plastic Rotation of the Member

The number of column and beam hinges at different overall drift ratios is shown in Figure 7. At small drift ratios, the behavior was mainly dependent on the initial stiffness of the structure and its members. Hence, similar numbers of column and beam hinges were obtained at 1% drift for all three cases. However, at higher drift ratios the columns controlled the ductility of a moment resisting frame. More specifically, the behavior of the column hinges at large inelastic rotations was the most influential factor on the overall drift capacity and stability of the reinforced concrete frames.

![Number of Plastic Hinges at Various Drift Ratios](image)

Figure 7. Number of Plastic Hinges at Various Drift Ratios

5 Concluding Remarks

In this paper, results from case studies are presented for two frames. Frame-S complies with the current seismic design provisions of North American Design Codes. Frame-NS can be classified as a non-ductile moment resisting
frame and does not comply with the current seismic design requirements. Frame-NS was upgraded using a commercially available CFRP wrap system. Having completed the experimental verification of the effectiveness of CFRP wrap systems [11] in seismic column retrofit applications, extensive analyses were performed to investigate the impact of the retrofitted column behavior on the overall response of reinforced concrete frames. The following conclusions can be reached based on the analytical investigations reported herein.

i. The unidirectional carbon fiber polymer wrap system employed to upgrade the columns of Frame-NS proved to be effective.

ii. Wrapping the columns at the potential plastic hinge regions of the frames did not only improve their seismic performance but also increased the percentage of exploitation of their beam plastic hinge rotation capacities.

iii. The behavior of Frame-NS after the upgrade was as good as, if not better than, Frame-S, which was designed to comply with the current seismic provisions of North American Design Codes.

References