SHAKE TABLE TESTS ON HAUNCH RETROFITTED REINFORCED CONCRETE FRAMES

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Shake table tests were conducted on five 1:3 reduced scale two story reinforced concrete (RC) moment resisting frames having construction and design deficiencies (built in low strength concrete, lacking confining ties in beam-column joint panels, practicing larger tie spacing and reduced longitudinal & transverse reinforcements). The deficient frames were observed with severe joint damageability, resulting in joint panel cover spalling and core concrete crushing. Haunch retrofitting technique was adopted to upgrade the seismic behavior of the deficient RC frames. Additional four deficient RC frames were built and retrofitted with steel haunch; both axially stiffer and deformable having energy dissipation capacity, installed at the beam-column connection to reduce shear demand on joint panels. The as-built and retrofitted frames’ seismic response modification factor R is calculated and compared to evaluate the viability of haunch retrofitting technique. The haunch retrofitting technique increased the lateral stiffness and strength of the structure, resulting in the increase of structure overstrength. The retrofitting increased R factor by 60% to 100%. The technique can significantly enhance the seismic performance of deficient RC frames, particularly against the frequent and rare earthquake events, hence, it is promising for seismic risk mitigation.

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Introduction

RC constructions in Pakistan are on rise due to the rapid increase in urbanization, particularly in the commercial sectors (multistory Plazas and Flats) and critical public facilities (Hospitals, Banks and Schools). Field reports based on surveys conducted in more than 40 sites in Pakistan have shown that proper execution of designs in the field is still a challenge and many disparities can be found in actual constructions. Despite the modern nature of concrete constructions, RC frame structures in Pakistan have shown very poor performance in past earthquakes [1]. Among the total RC structures exposed to 2005 Kashmir Mw 7.5 earthquake, 50 percent of the structures were severely damaged: either partially or completely collapsed. This poor performance of RC structures is attributed to non-seismic design of structures and non-compliant nature of constructions. This is not only common in Pakistan but also experienced worldwide (Fig. 1).

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Earthquake observations in Pakistan and worldwide have shown that substandard materials (low strength concrete, reduced size and low quality re-bars), reduction in longitudinal & transverse reinforcement, inadequate anchorage of longitudinal beam reinforcement in joints and joints lacking confining ties are major factors that lead to damage and early collapse of buildings during earthquakes. A low-cost, less invasive and easily implementable haunch retrofitting technique was adopted: a fully fastened stiffer steel haunch was used to stiffen the beam-column connections of RC frame and reduce shear demand on joint panels, and a buckling-restrained deformable steel haunch was used to dissipate the seismic energy. A total of nine 1:3 reduced scale RC frames of two story were built and tested dynamically on shake table employing 1994 Northridge earthquake record, linearly scaled to multiple intensity levels for testing the structure. The tested structure seismic response parameters (stiffness, strength, ductility and response modification factor) were retrieved for both the as-built and retrofitted frames.

**Description of the Test Frame Models**

A two story frame normally practiced for low-rise public buildings like hospitals, schools, apartment buildings and shopping malls is considered and designed using the lateral static force-based seismic design procedure specified in the Building Code of Pakistan – Seismic Provision BCP-SP 2007 [2] for high seismic hazard zone (Zone 4, 0.40g design PGA, site soil type B) and detailed as per the ACI-318-05 recommendations for special moment resisting frame (SMRF). Concrete with compressive strength of 3000 psi (21 MPa) and reinforcing steel bars with yield strength of 60,000 psi (414 MPa) were considered. The structure design was carried out in the finite element based software ETABS CSI, considering all the load combinations for dead, live and earthquake loads given in the BCP-SP 2007. Fig. 2 shows the geometric and reinforcement details of the designed structure, conforming to the code and SMRF detailing. Further, a total of five structural models were considered taking into account the construction deficiencies found in the field practice. Table 1 shows the characteristics of the as-built models considered for shake table testing and their seismic performance evaluation.

For shake table testing, 1:3 reduced scale simple model idealizations was adopted to prepare test models; all the linear dimensions of beams, columns and slabs and diameter of the re-bars were reduced by a scale factor $S_L$. A mix proportion of 1:1.80:1.60 (cement:sand:aggregate) with
w/c 0.48 is used to achieve 3000 psi (21 MPa) and mix proportion of 1:3.50:2.87 (cement:sand:aggregate) with w/c 0.80 is used to achieve 2000 psi (14 MPa).

Figure 2. Details of the considered RC frame structure, SMRF compliant model

Table 1. Details of the shake table tests models (disparities are highlighted) – as-built models

<table>
<thead>
<tr>
<th>S. No.</th>
<th>Member</th>
<th>Dimensions in (mm)</th>
<th>$f_c$</th>
<th>$f_y$</th>
<th>Long. Reinf.</th>
<th>Tran. Reinf.</th>
<th>Joint Ties</th>
<th>Hook</th>
</tr>
</thead>
<tbody>
<tr>
<td>Model-1</td>
<td>Beam: 12 x 18 (30 x 459)</td>
<td>3000 psi (21 MPa)</td>
<td>Beam: 6#6 (6$\phi$20 mm) Column: 8#6 (8$\phi$20 mm)</td>
<td>Beam: #3@ 3in (φ10mm @ 76mm) Column: #3@ 3in (φ10mm @ 76mm)</td>
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<td>Model-2</td>
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<tr>
<td>Model-3</td>
<td>Columns: 12 x 12 (304 x 304)</td>
<td>2000 psi (14 MPa)</td>
<td>60 ksi (414 MPa)</td>
<td>Beam: #4@ 6in (φ10mm @ 152mm) Column: #3@ 6in (φ10mm @ 152mm)</td>
<td>No -Ties</td>
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<tr>
<td>Model-4</td>
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It is worth to mention that the model and prototype uses essentially the same material type (concrete and re-bars), which have similar stress-strain behavior and material density (unit weight). Due to this, the reduced scale models was subjected to gravity and seismic mass less than the required as per the similitude requirements for prototype-to-model conversion:

\[ M_r = \frac{M_M}{M_P} = L_r^2; \quad L_r^2 = \frac{1}{S_L^2} \]  \hspace{1cm} (1)

where \( M_r \) is the ratio of model mass \( M_M \) to prototype mass \( M_P \), \( L_r \) is the reciprocal of linear scale factor \( S_L \). In order to satisfy the above requirements for model mass simulation, the additional required mass was applied to each floor of the model, calculated following the mass simulation model [3]:

\[ M_{M1} = \frac{M_P}{S_L^2} - M_{M0} \]  \hspace{1cm} (2)

where \( M_{M1} \) is the additional floor mass for model, \( M_{M0} \) is the floor mass of model. The total mass on each floor is, thus, the sum of additional mass \( M_{M1} \) and \( M_{M0} \). The additional floor mass (1200 kg for each floor) was simulated through two 600 kg steel blocks, that was prepared by stacking and welding steel plates together, which was mounted and fixed to the floor by means of fully secured \( \frac{1}{2} \) inch (13 mm) steel bolts.

**Input Excitation and Loading Protocols**

The test model was mounted on the shake table, firmly secured by means of 18 steel bolts of \( \frac{1}{2} \) inch (13 mm) diameter. The model is instrumented with six accelerometers with maximum capacity of \( \pm 10 \)g and five displacement transducers with maximum capacity of \( \pm 12 \) inch (305 mm). After careful analysis of number of accelerograms, a natural acceleration time history record of 1994 Northridge earthquake (horizontal component, 090 CDMG Station 24278 - PEER strong motion database) was selected as an input excitation. This record has maximum acceleration of 0.57g, maximum velocity of 518 mm/sec and maximum displacement of 90 mm (Fig. 3). The selected acceleration time history was linearly scaled to multiple intensity levels (5\%, 10\%, 20\%, 30\%, 40\%, 50\%, 60\%, 70\%, 80\%, 90\%, 100\%, 130\%), to push the structure from elastic to inelastic and severe damage state, in order to observe their progressive damage pattern. The tests were concluded when the model was found in the incipient collapse state.

![Figure 3: Input excitation for shake table test models, Northridge-1994 earthquake](image-url)
Observed Seismic Behavior of As-Built RC Frames

The code compliant model (Model-1) was observed with beam-sway mechanism; experiencing flexure yielding at the beam ends and slight flexure cracking at bottom end of columns on the ground story under test run with 100% intensity of excitation. This model was able to resist 130% of Northridge record for collapse limit state exceedance, deforming to 5.30% roof drift with maximum force resistance of 255 kN. Model-2 to Model-5 were observed with flexure cracking in both columns and beams and severe damages in joint panel regions under input excitation well below the design ground motions. Considering the ultimate limit state (incipient collapse state), Model-2 deformed to 5.0% drift with maximum force resistance of 180 kN, Model-3 deformed to 4.77% drift with maximum force resistance of 185 kN, Model-4 deformed to 3.45% drift with maximum force resistance of 152 kN, Model-5 deformed to 3.92% drift with maximum force resistance of 125 kN. Fig. 4 shows the typical damages observed in the deficient models. The use of low strength concrete lowered the structure resistance and altered the mechanism from beam-sway to column-sway and joint mechanism. In addition, the lack of confining ties in joint region resulted in the concrete cover spalling and core crushing under lateral excitations well below the 100% intensity of Northridge record. Further details on this can be found elsewhere [4].

Figure 4. Observed damages in deficient models under extreme loading, incipient collapse state
Haunch Retrofitting of RC Frames

Haunch Retrofitting Schemes
The haunch retrofitting technique for RC frames was proposed by Pampanin and Christopoulos [5]. The technique involves installing a metallic haunch type element at the beam-column connection that, under lateral loading, controls the hierarchy of strength within the beam-column members. It is with the possibility of using haunch of stiffer material to remain elastic during loading or deform, yield during loading to provide energy dissipation under cyclic response. The technique was investigated numerically and experimentally for 2D beam-column connections and frame that showed good performance in avoiding joint damageability of frames subjected to earthquake loading, and the technique also increased the energy dissipation of connections.

This technique was later on further developed, tested and validated through experimental tests on beam-column joint connections with further modification to make it more applicable in the field. Researchers at the University of Stuttgart Germany carried out quasi-static cyclic load tests on beam-column connections, installing haunch made of double-angle steel sections placed back-to-back used as the haunch element and designed in both tension and compression as per the capacity design principles. The haunch retrofitting increased the strength, ductility and energy dissipation of the connection, which are essential for better seismic performance of frame structures.

The technique was further explored and developed by Genesio and Sharma [6,7]. Genesio proposed a fully fastened haunch to increase the stiffness of the haunch with optimized design that was also tested experimentally, using quasi-static cyclic tests on beam-column connections. Further, Genesio also developed a numerical modelling technique for RC frames retrofitted with haunch. Shake table tests were conducted on 2D portal frame [7] to further investigate the fully fastened rigid haunch retrofitting.

In the present research both the rigid and deformable, energy dissipating, haunch types were tested adopting shake table testing for RC frames that involved more realistic frame structure models (including transverse beams and effects of slab), to assess the performance of technique in more realistic field condition and also studying wide cases of frames with construction deficiencies, common in the construction industry in developing countries. More focus was given to modifying the design scheme of haunch and scheme of application. Additionally, a hysteretic deformable haunch with restrained buckling was further included in the research to explore possibility of energy dissipation through deformable haunch that add supplemental damping to the structure. Both the stiffer and energy dissipating haunch were designed and verified through nonlinear static and time analysis using the FE based software SeismoStruct (SeismoSoft, 2016). The stiffer haunch were fabricated from the steel plates while the dissipating haunch was fabricated and encased in stiffer circular steel tubes that was filled with concrete to avoid buckling of the deformable haunch element under compression loading. In present research, the technique was tested for enhancing the seismic resistance of Model-3 and Model-5 (Table 1). Fig. 5 shows the retrofitting application schemes considered herein.
Figure 5. Details of stiffer and energy dissipating haunch and application schemes. The geometric details are shown for the 1:3 reduced scale model (in inches).
Observed Seismic Behavior of Haunch Retrofitted RC Frames

The installation of haunch at the beam-column connection altered the initial mechanism, forming flexure hinging in beams and columns at distance from the beam-column interface; the flexure cracking in beams and columns distributed over significant length (Fig. 6). However, damages were experienced also in the joint panels upon subjecting the structure to larger lateral displacement under extreme shaking. This happened following the pullout of haunch from column during shaking, causing connection opening, this took place following the detachment of a wedge like concrete from column due to the low strength of concrete. However, the retrofitted structures have shown increase in stiffness, strength and ductility. Fig. 7 shows the derived lateral force-deformation response of the tested frame structures, both for as-built and retrofitted. Model-3 stiffness was increased by 50% using stiffer haunch and 65% using dissipating haunch, the corresponding strength was increased by 10% and 30% respectively. Model-5 stiffness was increased by 90% using stiffer haunch and by 80% using dissipating haunch, the corresponding strength was increased by 30% and 60% respectively.

Figure 6. Observed damages in deficient models retrofitted with Haunch
Seismic Response Modification Factor

In the present research the seismic response modification factor $R$ is calculated, which is employed in the code-based seismic design of structures, for all the models using the analytical procedure, as adopted earlier [8]. Generally, $R$ factor for a structure can be calculated knowing the inelastic lateral force-deformation behavior of the structure.

$$ R = \frac{V_e}{V_s} = \frac{\frac{V}{V_y}}{V_x} = R_\mu \times R_S $$

where $V_e$ represents the elastic force the structure will experience, if respond elastically under earthquake demand; $V_y$ represents the idealized yield strength of the structure; $V_s$ represents the design base shear force; $R_\mu$ represents the ‘ductility factor’, structure ductility dependent factor, $R_S$ represents the ‘overstrength factor’, structure overstrength dependent factor. The overstrength factor $R_S$ is calculated directly from the lateral force-deformation capacity curve of the structure (i.e. dividing the idealized yield strength over the structure design strength), however, the ductility factor $R_\mu$ is related to the structural ductility Newmark and Hall [9] as given:

- Short Period: $T < 0.20$ sec. \hspace{1cm} $R_\mu = 1.0$
- Intermediate Period: $0.2 \text{ sec.} < T < 0.5 \text{ sec.}$ \hspace{1cm} $R_\mu = \sqrt{2\mu - 1}$
- Long Period: $T > 0.5 \text{ sec.}$ \hspace{1cm} $R_\mu = \mu$

where $T$ is the pre-yield vibration period of idealized single degree of freedom system. The weight of the considered prototype frame is 28 ton, and considering the yield stiffness obtained from the experimental idealized capacity curves, the structure vibration period was calculated using the classical formula of vibration period. The code specified ultimate drift limit of 2.50% is considered as the ultimate drift capacity that corresponds to displacement capacity of about 183 mm (7.20 inch). The frame ductility $\mu$ was obtained dividing the ultimate displacement capacity over the idealized yield displacement capacity of each structure model, which gives also an estimate of $R_\mu$. The response modification factor $R$ of prototype structures was calculated by multiply the ductility dependent $R_\mu$ factor with the overstrength factor $R_S$. The calculated $R$ factor
for as-built frame is 3.5 for Model-3 and 2.5 for Model-5. In case of retrofitted frame the calculated R factor increased to 5.5 for both Model-3 and Model-5 using stiffer haunch whereas R factor increased to 5.5 for Model-3 and 5.0 for Model-5 using energy dissipating haunch.

Conclusions

Shake table testing of as-built and retrofitted RC frames have revealed that both the stiffer and energy dissipating haunch types can significantly enhance the stiffness, strength and ductility of structures. This also made the structures able to resist larger shaking intensity as compared to the as-built frame models; the as-built models were able to resist only 40%-to-50% of intensity whereas the haunch retrofitted RC frames were able to resist 80%-to-90% of intensity, before exceeding the collapse limit state. The dissipating haunch performed somewhat better than the stiffer haunch due to its nature of adding supplemental damping to the structures besides stiffening the connection. The haunch retrofitting technique seems to perform efficiently well under frequent and rare earthquake events, hence it is promising for seismic risk mitigation.

Acknowledgments

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References