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Seismic performance of RC frames retrofitted with haunch technique

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Abstract. Shake table tests performed on five 1:3 reduced scale two story RC moment resisting frames having construction defects, have shown severe joint damageability in deficient RC frames, resulting in joint panels' cover spalling and core concrete crushing. Haunch retrofitting technique was adopted herein to upgrade the seismic resistance of the deficient RC frames. Additional four deficient RC frames were built and retrofitted with steel haunch; both axially stiffer and deformable with energy dissipation, fixed to the beam-column connections to reduce shear demand on joint panels. The as-built and retrofitted frames' seismic response parameters are 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's overstrength. The retrofitting increased response modification factor R by 60% to 100%. Further, the input excitation PGA was correlated with the lateral roof displacement to derive structure response curve that have shown significant resistance of retrofitted models against input excitations. The technique can significantly enhance the seismic performance of deficient RC frames, particularly against the frequent and rare earthquake events, hence, promising for seismic risk mitigation.

Keywords: seismic resistance; haunch retrofitting; shake-table; joint damage; response modification factor

1. Introduction

RC structures in Pakistan are on rise due to the rapid increase in urbanization, particularly in the commercial sectors (multistory storys Plaza's, Flats) and critical public facilities (Hospitals, Banks and Schools). Field surveys of more than 40 sites in Pakistan have shown that proper execution of design specifications in field is still a challenge and many disparities can be found in actual constructions (Badrashi *et al.* 2010). Despite the modern nature of concrete constructions, RC structures in Pakistan have shown very poor performance in past earthquakes. 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/or non-compliant nature of constructions (Waseem and Spacone 2017, Naseer *et al.* 2010, Rossetto and Peiris 2009). This is not only common in Pakistan but also experienced worldwide during moderate to large earthquakes (see Fig. 1), also Chaulagain *et al*. (2015).

Earthquake observations in Pakistan and worldwide have shown that substandard materials (low strength concrete, reduced size and low quality rebars), 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. Further, these deficient structures are regularly subjected to moderate to large earthquakes in high seismicity region, due to the active tectonics of the region, and high expected seismic hazard (Danciu *et al.* 2017, Waseem *et al.* 2018, Zare *et al.* 2014), resulting in the damage of structures and subsequent economic losses due to the routine reparability that may represent a huge burden on the shoulder of clients. Various advanced strengthening techniques exists (Shiravand *et al.* 2017), a low-cost, less invasive and easily implementable haunch retrofitting technique was adopted, as proposed earlier by Pampanin *et al*. (2006). 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 add supplemental damping to the structure and dissipate the seismic energy. The proposed solutions were visualized as a viable mean to enhance the seismic resistance of deficient structures (those with vulnerable joint panels) and avoid the structure damageability in frequent and rare earthquakes, thereby, reducing the economic losses and achieving the objective of seismic risk mitigation.

Nine 1:3 reduced scale frames of two-story were built including five as-built models and four models strengthened with haunches. The five as-built models were built including a fully code confirming model and, further, incorporating the construction deficiencies common in developing countries. Additionally, four models were built in present research and retrofitted with different schemes of haunch. The models were tested on shake-table in

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Fig. 1 Joint damageability and collapse of structures: from left to right: 1999 Izmit earthquake, Turkey, 1999 Chi-Chi earthquake (Taiwan), 1994 Northridge earthquake (USA) (Sharma 2013)

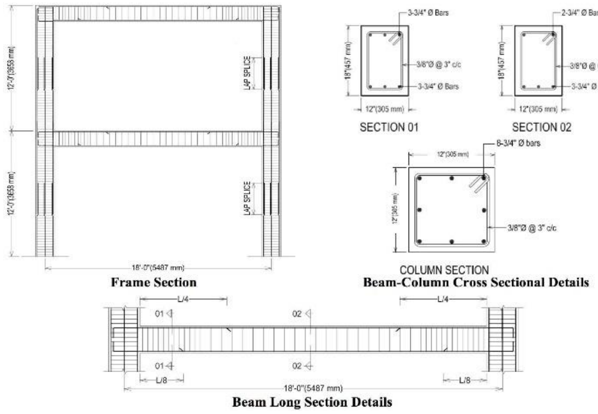


Fig. 2 Details of the considered RC frame structure, SMRF compliant and code conforming model

Earthquake Engineering Center in Peshawar, employing 1994 Northridge earthquake record. The acceleration time history is linearly scaled to multiple intensity levels (5%-to-100% and 130%) to test the structure progressively. The tested data is analyzed to obtain the structure seismic response parameters (stiffness, strength, ductility, response modification factor) for both the as-built and retrofitted frames.

1.1 Literature review on haunch retrofitting

The haunch retrofitting technique for RC frames was proposed first by Pampanin and Christopoulos (Pampanin *et al.* 2006) and comes from the University of Canterbury in New Zealand in collaboration with the University of Toronto, Canada. The idea was to install a metallic haunch type element at the beam-column connections that control the hierarchy of strength within the beam-column members. This technique was envisaged to strengthen the deficient RC frames, particularly those experiencing joint damageability upon subjecting to earthquake excitation. Since, the joint panels in real structures are difficult to strengthen with the commonly adopted techniques (Engindeniz *et al.* 2005), application of an intervention that divert (or reduce) shear demands on the panel, and instead allow members to deform inelastically under earthquake imposed deformation cycles.

A possibility of haunch with stiffer material to remain elastic during loading or deform, yield during loading and provide energy dissipation under cyclic response. The concept was investigated numerically and experimentally for 2D beam-column assemblies and frame that showed good performance in avoiding joint damageability of frames subjected to earthquake loading and increases the energy dissipation of connections (Pampanin *et al.* 2006, Wang *et al.* 2017).

This concept was later on further developed, tested and validated through experimental tests on beam-column joint assemblies 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 assemblies, with the 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 the better seismic performance of frame structures.

The idea was further explored and developed by Genesio and Sharma at the University of Stuttgart Germany, during their doctoral studies (Genessio 2012, Sharma 2013). Genesio proposed a fully fastened haunch to increase the stiffness of the haunch with optimized design and was tested experimentally using quasi-static cyclic tests on beam-column assemblies. Further, Genesio developed numerical modelling technique for RC frames retrofitted with haunch. The fully fastened rigid haunch retrofitting was further investigated (Appa-Rao *et al.* 2013), and was also tested dynamically on the 2D portal frame (Sharma *et al.* 2011, 2014).

2. 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 to the lateral static force-based seismic design procedure specified in the Building Code of Pakistan-Seismic Provision BCP-SP (2007) for high seismic hazard zone (Zone 4, 0.40 g design PGA on hard rock type B, as per the NEHRP classification) 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 detailings. Further, a total of five structural models were considered taking into account the construction defects found in the field practice. Table 1 shows the characteristics of the models considered for seismic performance evaluation by Rizwan *et al*. (2017).

For shake table testing, 1:3 reduced scale simple model idealizations was adopted to built test models; all the linear dimensions of beams, columns and slabs and diameter of the rebars were reduced by a scale factor S_L 3. A mix

Table 1 Details of the shake-table tests models (disparities are highlighted), as-built models tested by Rizwan *et al.* (2017)

S. No.	Member Dimensions in (mm)	f_c'	f_y	Long. Reinf.	Tran. Reinf.	Joint Ties	Hook
Model-1	Beam: 12 x 18 (30 x 459) Columns: 12 x 12 (304 x 304)	3000 psi (21 MPa)	60 ksi (414 MPa)	Beam: 6#6 (6ϕ20 mm) Column: 8#6 (8ϕ20 mm)	Beam: #3@ 3in (ϕ10 mm @ 76 mm) Column: #3@ 3in (ϕ10 mm @ 76 mm)	With Ties	135^0
Model-2		2000 psi (14 MPa)					
Model-3						No -Ties	
Model-4					Beam: #3@ 6in (ϕ10 mm @ 152 mm) Column: #3@ 6in (ϕ10 mm @ 152 mm)		
Model-5				Beam: 4#6 (4ϕ20 mm) Column: 6#6 (6ϕ20 mm)	Beam: #3@ 9in (ϕ10 mm @ 228 mm) Column: #3@ 9in (ϕ10 mm @ 228 mm)		90^0

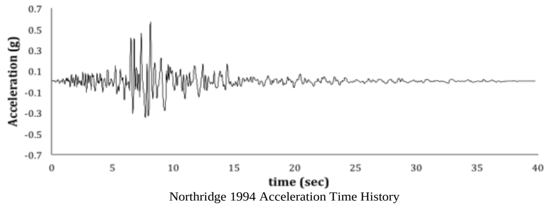


Fig. 3 Input excitation for shake table test models

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).

It is worth to mention that the model and prototype uses essentially the same materials type (concrete and steel 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

$$M_r = \frac{M_M}{M_P} = L_r^2 \quad ; \quad L_r^2 = \frac{1}{S_L^2} \tag{1}$$

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 (Quintana-Gallo *et al.* 2010)

$$M_{M1} = \frac{M_P}{S_L^2} - M_{M0} \tag{2}$$

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 ½ inch (13 mm) steel bolts.

The model was placed on the shake-table by means of 20 ton overhead crane. Using ½ inch bolts, the model pad was mounted to the shake table top. Shear capacity and model over-turning moment were checked and satisfied. Specially fabricated steel stool was fixed to the ground, and raised to the table top level, with ½ inch clearance, having 25 mm dia steel bars (placed between model pad and stool) to provide roller support to the projected pad of the model.

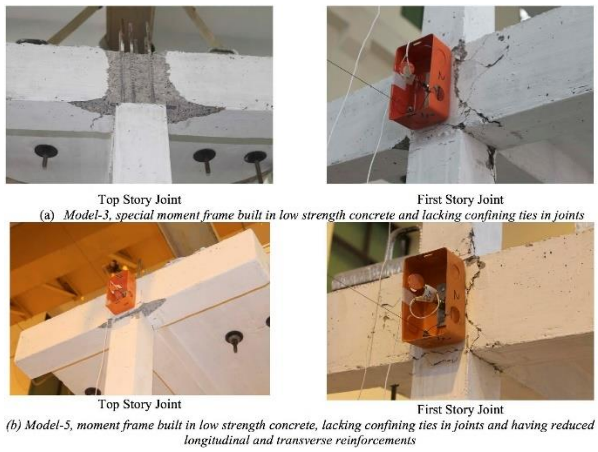


Fig. 4 Observed damages in deficient models under extreme loading, incipient collapse state (Rizwan *et al.* 2017)

3. Input excitation and loading protocols

Each of the test model was mounted on the shake table top, firmly secured by means of ½ inch (13 mm) diameter 18 steel bolts and instrumented with five accelerometers with maximum capacity of ±10 g and five displacement transducers with maximum capacity of ±12 inch (305 mm). 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 0.57 g, maximum velocity of 518 mm/sec and maximum displacement of 90 mm, and can laterally excite the structure roughly symmetrically in both positive/negative directions (Fig. 3). The selected acceleration time history was applied to the test model with multiple excitations (5%, 10% and incremented further with 10% increase), to push the structure from elastic to inelastic and severe damage state and to observe their progressive damage pattern, the tests were concluded when the model was found in the incipient collapse state.

4. Observed seismic behaviour 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 max. force resistance of

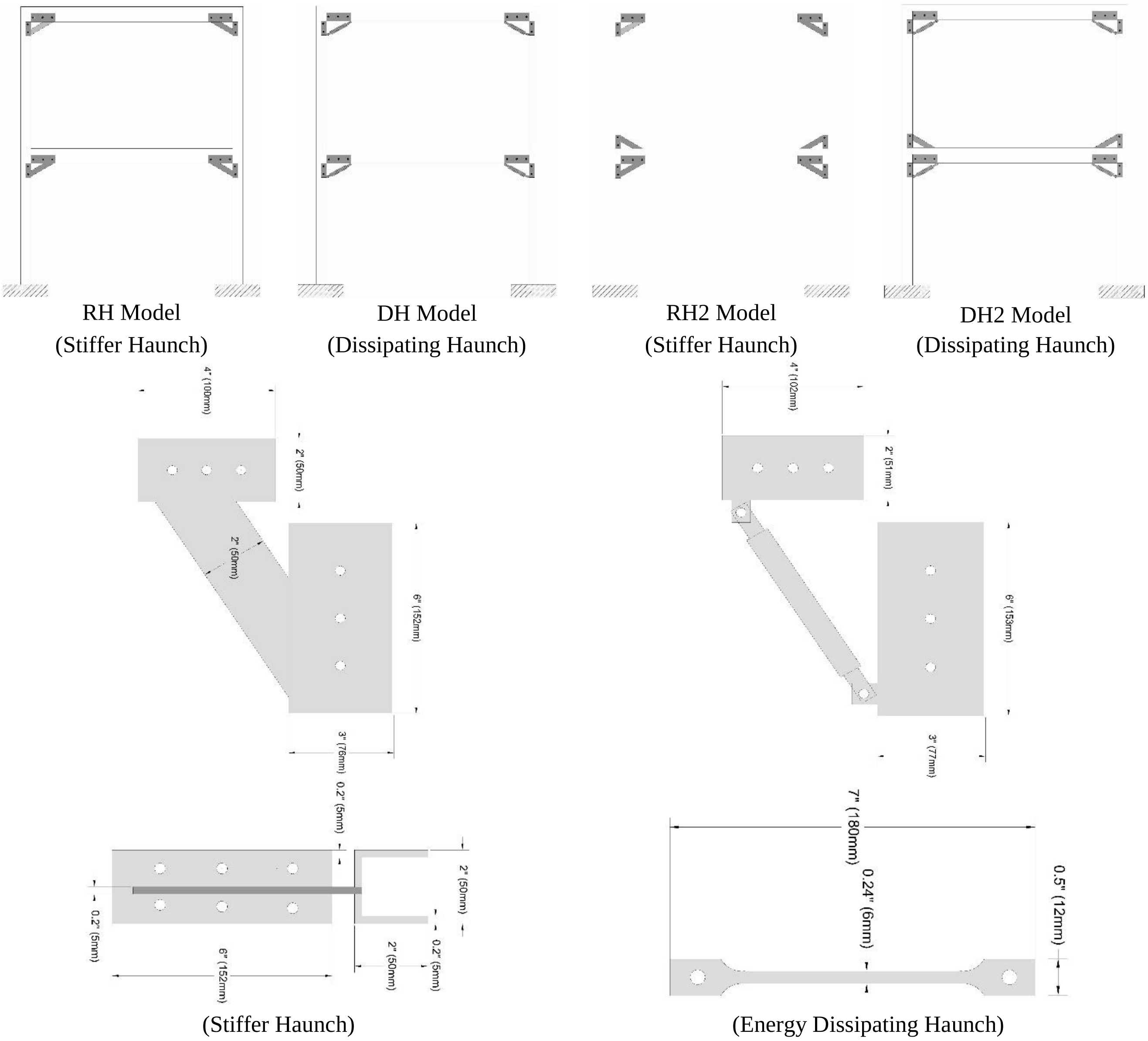


Fig. 5 Details of stiffer and energy dissipating haunch and application schemes. The geometric details are shown for the 1:3 reduced scale model

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. Considering the ultimate limit state (incipient collapse state), Model-2 deformed to 5.0% drift with max. force resistance of 180 kN, Model-3 deformed to 4.77% drift with max. force resistance of 185 kN, Model-4 deformed to 3.45% drift with max. force resistance of 152 kN, Model-5 deformed to 3.92% drift with max. 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 the observed damage mechanism of the tested models can be found in Rizwan *et al*. (2017).

5. Haunch retrofitting of RC frames

5.1 Haunch retrofitting schemes

In the present research both the rigid and deformable, energy dissipating, haunch types were tested under 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 defects, common in the construction industry in developing countries. Focus was particularly made on modifying the design scheme of haunch itself and scheme of application on structure, additionally a nonlinear 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

Fig. 6 Observed damages in deficient models retrofitted with Haunch

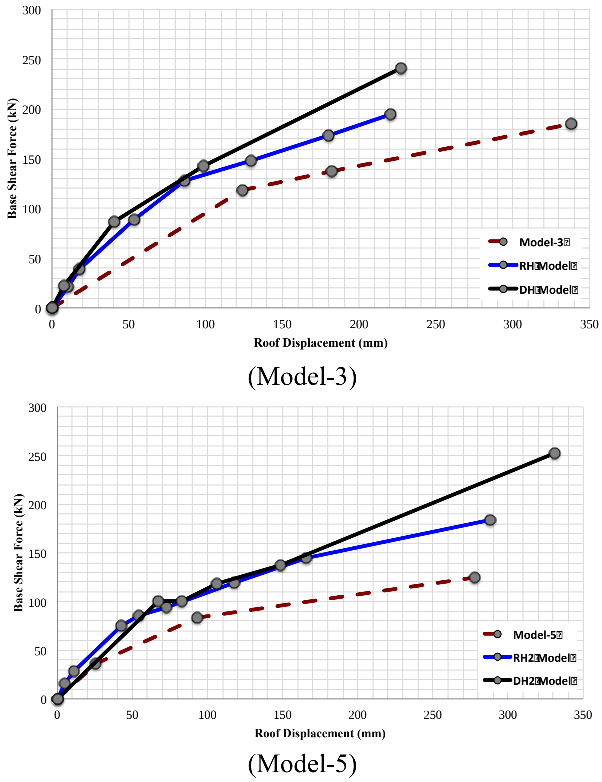


Fig. 7 Lateral force-deformation response of as-built and retrofitted RC frame

stiffer and energy dissipating haunch were designed by trial and verified through nonlinear time analysis using the finite element software SeismoStruct and the modelling procedure of Genesio, employing the 1994 Northridge earthquake acceleration time history. The stiffer haunch were fabricated from the steel plates whereas the dissipating haunch were fabricated and encased in stiffer circular steel tubes that were 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 application schemes considered herein. On

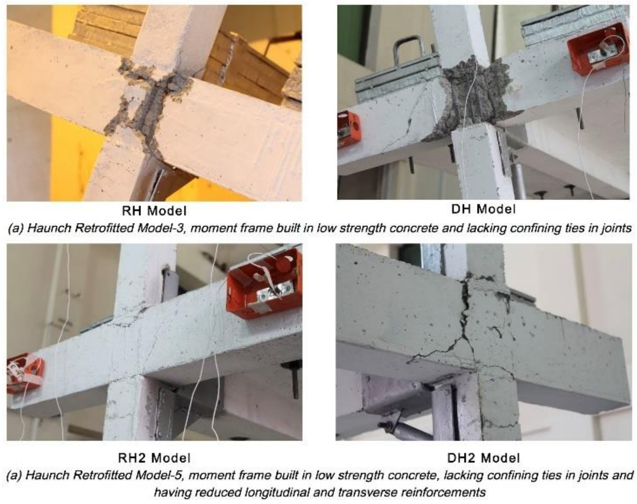


Fig. 8 Observed damages in haunch retrofitted RC frames with different application scheme

average, the overall cost of haunch retrofitting per location (per haunch) is about Rs. 950.0 (≈ 10.0 USD) for the model, which can reach to about Rs. 2500 (≈ 25.0 USD) for the prototype. The indicated cost included cost for all material types (steel plates, weld, epoxy, nails) and accessories, and also included labor cost.

5.2 Observed seismic behaviour of haunch retrofitted RC frames

The application 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 resulted due to the pullout of haunch from column during shaking causing connection opening, which happened due to 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 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.

Fig. 8 shows the extent of damage observed in beam-column joint regions upon subjecting the model to extreme level shaking. This damage is relatively more severe in model where haunch is applied only at the top ends of column at the connection. Because, the strain in the longitudinal bars of columns at the bottom ends penetrate through the joints under tension loading, resulting in stress demand on panel zone. The shaking induced stress demand in joint panel can result into joint damage upon exceeding the joint principal tensile strength (Priestley 1997, Pampanin *et al.* 2002).

The present research has shown that the application of haunch at both the top and bottom ends of column can although retard the joint damageability, but, additional measures will be required to avoid damage in the exterior joints. We propose to affix a steel plate on the exterior face of the joint, and extend over plastic hinge region i.e., 1*h*, h represents the depth of column. However, experimental validation of this proposal may be required, to further increase the confidence level.

6. Seismic response modification factor

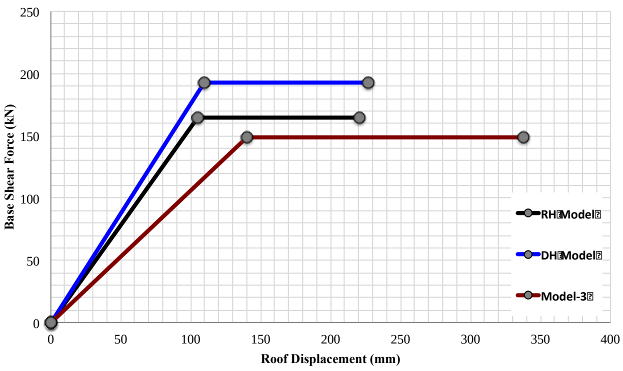
In the present research the seismic response modification factor R, which is employed in the code-based seismic design of structures, is calculated for all the models using the analytical procedure used also in earlier research (Ahmad *et al.* 2017). Generally, R factor for a structure can be calculated knowing the inelastic lateral force-deformation behavior of the structure.

$$R = \frac{V_e}{V_s} = \frac{V_e}{V_y} \times \frac{V_y}{V_s} = R_\mu \times R_s \tag{3}$$

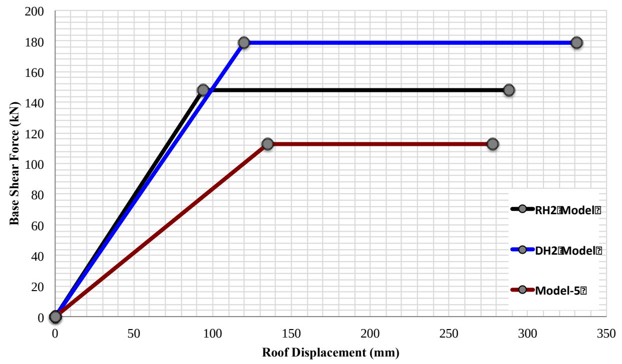
where V_e represents the elastic force the structure will experience, if respond elastically under earthquake demand; V_y represents the idealized yield strength of the structure; V_s represents the design base shear force; R_μ represents the 'ductility factor', structure ductility dependent factor, 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_μ is related to the structural ductility Newmark and Hall (1982) as given:

Short Period	T < 0.20 sec.	$R_\mu = 1.0$	Structure Vibration Period $T = 2\pi\sqrt{\frac{m}{k_y}}$
Intermediate Period	0.2 sec. < T < 0.5 sec.	$R_\mu = \sqrt{2\mu - 1}$	
Long Period	T > 0.5 sec.	$R_\mu = \mu$	

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 μ was obtained dividing the ultimate displacement capacity over the idealized yield displacement capacity of each structure model, which gives also an estimate of R_μ. The response modification factor R of prototype structures was calculated by multiply the ductility dependent R_μ factor with the overstrength factor R_S.



(Model-3)



(Model-5)

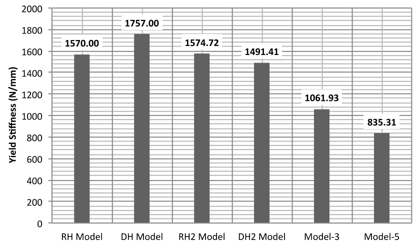
Fig. 9 Bi-linearized lateral force-deformation response of as-built and retrofitted RC frames

For this purpose, the actual lateral-force deformation capacity curve derived herein experimentally was idealized as bi-linear elasto-plastic curve (Fig. 9) to identify the yield strength, yield displacement and ultimate displacement capacity. The idealization was carried out using the energy balance rule; to make the energy under the curve equivalent for both the actual and the idealized capacity curve. These idealized elasto-plastic curves were used to calculate the seismic response parameters of the structure (Fig. 10). 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.

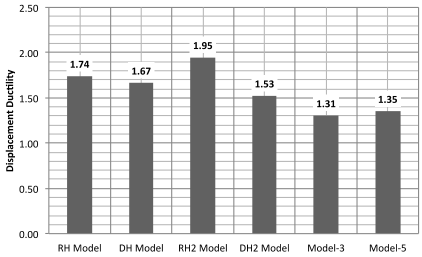
It can be observed from the seismic response parameters (Fig. 10) that the haunch retrofitting increases stiffness, strength and ductility of the structures. Increase in the response modification factor R of the retrofitted structures is largely due to the structural overstrength.

7. Seismic response curves

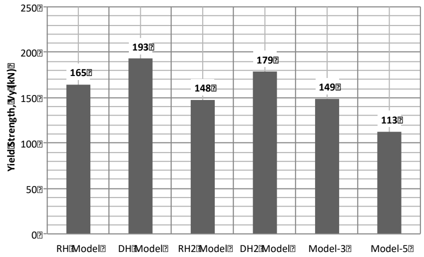
Further, the input excitation intensity, in terms of peak horizontal acceleration *referred as PGA*, was correlated with the roof displacement demand to derive structure response curve (Fig. 11). These curves show the structural lateral deformation against the shaking intensity severity.



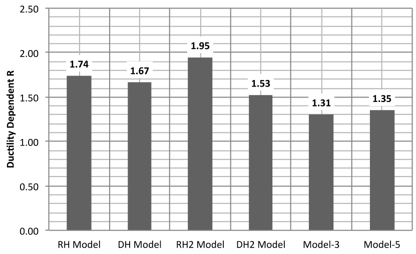
(Yield Stiffness)



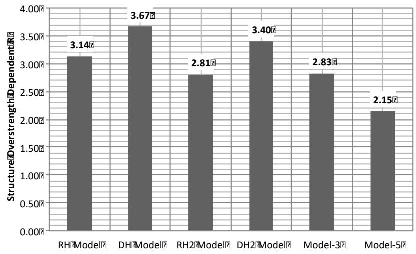
(Structure Ductility)



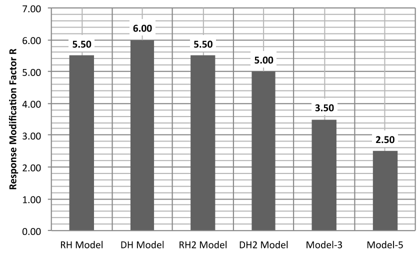
(Yield Strength)



(Ductility Dependent R)

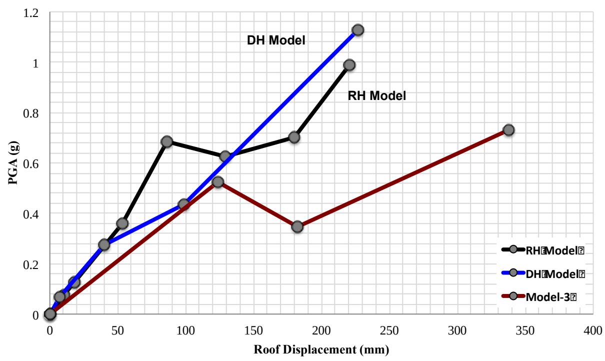


(Overstrength Dependent R)

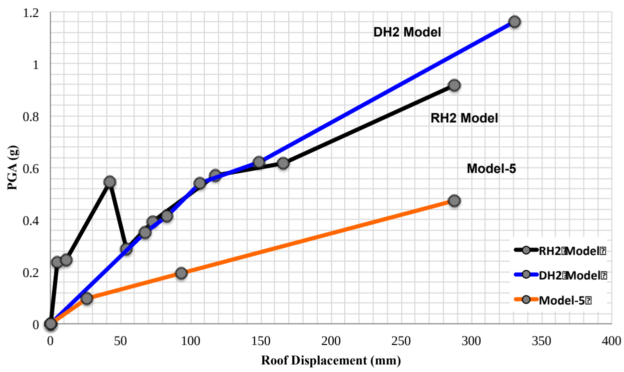


(Response Modification Factor R)

Fig. 10 Seismic response parameters of as-built and retrofitted RC frames



(Model-3)



(Model-5)

Fig. 11 Seismic response curve (PGA vs roof displacement)

As can be seen the curves behave linearly, initially, but deviate due to structural damages and tends to flatten for extreme level shaking that corresponds to the development of inelasticity in structural resisting members.

It can be observed that both the stiffer and energy dissipating haunch increases the structural resistance against the input excitation. Thereby, making the structure able to resist higher ground shaking. Further, the retrofitting technique enabled the structure control lateral deformation under earthquake ground motions, which is primarily due to the high stiffness of the structure and the energy dissipation capacity of the structure.

8. Conclusions

Shake table tests on 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 with construction defects (moment frame built in low strength concrete, lacking confining ties in joints and having reduced longitudinal and transverse reinforcements). 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 relatively 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, the technique seems promising for seismic risk mitigation of deficient RC frame structure.

It is worth to mention that the haunch application scheme significantly affects the seismic response of the structure; haunch applied on both top and bottom of column ends at the connection can better retard the joint damageability. This seems to work better for interior joints, however, the exterior joints will still need additional measures to avoid joint damageability under extreme shaking. A steel plate fixed to the beam-column exterior joint on the outside face, and extended up to the haunch edges (plastic hinge region), will ensure beam-column connection rigidity and will control the hierarchy of strength within the beam-column members. However, this will require further experimental tests for validation to increase the confidence level.

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References

ACI 318 (2005), *Building Code Requirements for Structural Concrete and Commentary*, American Concrete Institute, Farmington Hills, Michigan, U.S.A.

Ahmad, N., Shahzad, A., Rizwan, M., Khan, A.N., Ali, S.M., Ashraf, M., Naseer, A., Ali, Q. and Alam, B. (2017), "Seismic performance assessment of non-compliant SMRF reinforced concrete frame: Shake table test study", *J. Earthq. Eng.*, 1-19.

Appa-Rao, G., Navya, V. and Eligehausan, R. (2013) "Strengthening of shear deficient rc beam-column joints in mrfs under seismic loading", *Proceedings of the 8th International Conference on Fracture Mechanics of Concrete and Concrete Structures*, Spain.

Badrashi, Y.I., Ali, Q. and Ashraf, M. (2010), *Reinforced Concrete Buildings in Pakistan-Housing Report*, Housing Report No. 159, Earthquake Engineering Research Institute, California, U.S.A.

BCP-SP (2007), *Building Code of Pakistan: Seismic Provision*, Ministry of Housing and Works, Islamabad, Pakistan.

Chaulagain, H., Rodrigues, H., Spacone, E. and Varum, H. (2015), "Seismic response of current RC buildings in Kathmandu valley", *Struct. Eng. Mech.*, **53**(4), 791-818.

Danciu, L., Sesetyan, K., Demircioglu, M., Gulen, L., Zare, M., Basili, R., Elias A., Adamia, S., Tsereteli, N., Yalçın, H., Utkucu, M., Khan, M.A., Sayab, M., Hessami, K., Rovida, A.N., Stucchi, M., Burg, J.P., Karakhanian, A., Babayan, H., Avanesyan, M., Mammadli, T., Al-Qaryouti, M., Kalafat, D., Varazanashvili, O., Erdik, M. and Giardini, D. (2017), "The 2014 earthquake model of the middle east: Seismogeneic source", *Bullet. Earthq. Eng.*, 1-32.

EERI (2005), *Learning from Earthquakes: First Report on the Kashmir Earthquake of October 8, 2005*, Technical Report, Earthquake Engineering Research Institute (EERI), Oakland, California, U.S.A.

Engindeniz, M., Kahn, L.F. and Zureick, A.H. (2005), "Repair and strengthening of reinforced concrete beam-column joints: State of the art", *ACI Struct. J.*, **102**(2), 1.

Genesio, G. (2012), "Seismic assessment of rc exterior beam-column joints and retrofit with haunches using post-installed anchors", Ph.D. Dissertation, Stuttgart University, Germany.

Naseer, A., Naeem, A., Hussain, Z. and Ali, Q. (2010), "Observed seismic behavior of buildings in northern Pakistan during the 2005 Kashmir earthquake", *Earthq. Spectr.*, **26**(2), 425-449.

Newmark, N.M. and Hall, W.J. (1982), *Earthquake Spectra and Design*, Earthquake Engineering Research Institute, Oakland, California, U.S.A.

Pampanin, S., Christopoulos, C. and Chen, T.H. (2006), "Development and validation of a metallic haunch seismic retrofit solution for existing under-designed RC frame buildings", *Earthq. Eng. Struct. Dyn.*, **35**(14), 1739-1766.

Pampanin, S., Calvi, G.M. and Moratti, M. (2002), "Seismic behavior of R.C. beam-column joints designed for gravity only", *Proceedings of the 12th European Conference on Earthquake Engineering*.

Priestley, M.J.N. (1997), "Displacement-based seismic assessment of reinforced concrete buildings", *J. Earthq. Eng.*, **1**(1), 157-192.

Quintana-Gallo, P., Pampanin, S., Carr, A.J. and Bonelli, P. (2010), "Shake table tests of under designed RC frames for the seismic retrofit of buildings-design and similitude requirements of the benchmark specimen", *Proceedings of the New Zealand Society of Earthquake Engineering*.

Rizwan, M., Ahmad, N. and Khan, A.N. (2017), "Seismic performance of smrf compliant and non-compliant rc frames", *ACI Struct. J.*, Accepted.

Rossetto, T. and Peiris, N. (2009), "Observations of damage due to the Kashmir earthquake of October 8, 2005 and study of current seismic provisions for buildings in Pakistan", *Bullet. Earthq. Eng.*, **7**(3), 681-699.

Sharma, A., Reddy, G.R., Eligehausen, R., Genesio, G. and Pampanin, S. (2014), "Seismic response of reinforced concrete frames with haunch retrofit solution", *ACI Struct. J.*, **111**(3), 673-684.

Sharma, A. (2013), "Seismic behavior and retrofitting of rc frame structures with emphasis on beam-column joints-experiments and numerical modeling", Ph.D. Dissertation, Stuttgart University, Germany.

Sharma, A., Reddy, G.R. and Vaze, K.K. (2011), "Shake table tests on a non-seismically detailed RC frame structure", *Struct. Eng. Mech.*, **41**(1), 1-24.

Shiravand, M.R., Nejad, A.K. and Bayanifar, M.H. (2017), "Seismic response of RC structures rehabilitated with SMA under near-field earthquakes", *Struct. Eng. Mech.*, **63**(4), 497-507.

Wang, B., Zhu, S., Xu, Y.L. and Jiang, H. (2017), "Seismic retrofitting of non-seismically designed rc beam-column joints using buckling-restrained haunches: Design and analysis", *J. Earthq. Eng.*, 1-21.

Waseem, M. and Spacone, E. (2017), "Fragility curves for the typical multi-span simply supported bridges in northern Pakistan", *Struct. Eng. Mech.*, **64**(2), 214-223.

Waseem, M., Lai, C.G. and Spacone, E. (2018), "Seismic hazard assessment of northern Pakistan", *Nat. Haz.*, **90**(2), 563-600.

Zare, M., Amini, H., Yazdi, P., Sesetyan, K., Demircioglu, M.B., Kalafat, D., Erdik, M., Giardini, D., Khan, A. and Tsereteli, N. (2014), "Recent developments of the Middle East catalog", *J. Seismol.*, **18**(4), 749-772.

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