Magazine of Civil Engineering. 2022. 116(8). Article No. 11601

Magazine of Civil Engineering

ISSN 2712-8172

journal homepage: http://engstroy.spbstu.ru/

Research article

UDC 699.841

DOI: 10.34910/MCE.116.1

Seismic damage mechanism of weak beam-column joint frames

M. Rizwan [1] ***, N. Ahmad*** [2] ***, A. Naeem Khan*** [2]

[1] Sarhad University of Science and Information Technology, Peshawar, Pakistan

[2] Department of Civil Engineering, University of Engineering and Technology Peshawar, Peshawar, Pakistan

rizwan.civil@suit.edu.pk

Keywords: reinforced concrete, structural concrete, special moment resisting frame, building frame, beam-column joints, shake table test, progressive collapse

Abstract. The use of old building design codes and improper execution of recent seismic design practices resulted in substandard and vulnerable reinforced concrete building stock, the majority of which is built with weak beam-column joint connections (i.e. joint panels having no transverse reinforcement and built in low strength concrete). In order to understand the seismic response and damage behavior of recent special moment resisting frame (SMRF) structures with the defect of weak beam-column joints, shake table tests were performed on two 1:3 reduced scale two storey and one bay RC frame models. Reference code design and weak beam-column joint frame models were subjected to unidirectional dynamic excitation of increasing intensities using the natural record of 1994 Northridge Earthquake. The input scaled excitation were applied from 5 % to 130 % of the maximum input peak ground acceleration record, to force the test models from elastic to inelastic stage and then to fully plastic incipient collapse stage. The weak beam-column frame experienced flexural cracking of the columns, longitudinal bar-slip in beam members, cover concrete spalling and severe damageability of the joint panels under multiple dynamic excitations. The deficient frame was able to resist only 40 % of the maximum acceleration record as compared to the code design frame, which was able to resist up to130 %. Based on the experimental observations, a drift-based damage scale was developed for different performance limits states that can be employed for the global performance assessment of deficient weak beam-column joint RC frames.

Acknowledge: The authors are thankful to the Earthquake Engineering Center (EEC), Department of Civil Engineering, UET Peshawar, for supporting the experimental research presented herein.

Citation: Rizwan, M., Ahmad, N., Naeem Khan, A. Seismic damage mechanism of weak beam-column joint frames. Magazine of Civil Engineering. 2022. 116(8). Article no. 11601. DOI: 10.34910/MCE.116.1

1. Introduction

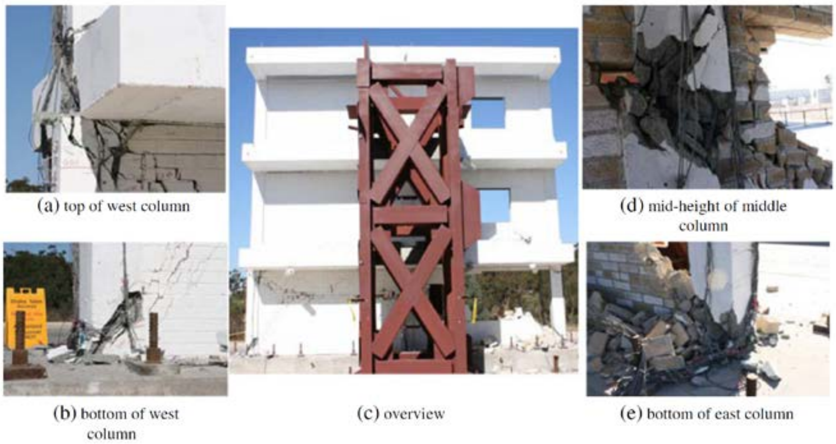
Reinforced concrete construction is on the peak in many urban areas of the world. This type of construction is particularly used for multi-storey buildings, schools, hospitals, and residential type buildings because of the vast availability of constituents of concrete around the world and the ease of construction. Although the design and construction of RC buildings in the developing countries are mostly based on the locally available or adopted international building codes, the proper construction execution of the specified design in the field is still a big challenge. The improper construction practices and use of old seismic code provisions resulted in a widespread substandard and vulnerable building stock [1–2], with recent surveys showing a number of construction/designs deficiencies [1, 3]. The commonly available construction/design defects include: substandard quality of concrete (low strength concrete), joint panels without transverse reinforcement, beam column members with reduced flexural reinforcement along with shear reinforcement spacing larger than specified by the code, and non-seismic hooks among others. In all these typical construction or non-seismic design practices, the weak beam-column joint connections i.e. joint panels having no transverse ties reinforcement and built with low strength concrete, are very common. An RC building with joints having such non-seismic provisions can cause shear failures of the critical panel regions; in turn, such failures can result in partial or full collapse of the structures. It is also worth to mention, majority of these RC buildings in the developing countries are located in high seismic regions and can be subjected to large earthquakes of extreme excitations in the future [4]. RC structures if not built properly can result in catastrophic failure and subsequent human and economic losses due to strong earthquake induced ground motions [5–9].

In recent decades, many researchers performed experimental investigations through quasistatic tests on weak beam-column connections and shake table tests on deficient frame structures, in order to assess their seismic performance [10–28]. In most of these experimental studies, either full scale and/or reduced scale beam-column connections and frame structures were employed. In these experimental investigations, different types of substandard and non-seismic parameters and diverse structural configuration were considered, in order to assess the seismic performance and damage mechanism of deficient of RC structures. Quintana et al. [10] tested a 1:2.5 reduced scale three-storey deficient frame structure (Fig. 1 a) on shaking simulator, in order to assess the damage mechanism and to provide bench mark test data for the retrofitting of old type, non-ductile and non-seismic design buildings. The test model was built with plain rebars, without joint transvers reinforcement provisions and with 90° non-seismic hooks. The test models were subjected to increasing PGA excitation of several earthquake records in order to observe the damage behavior of the test frame. During the initial excitations it was observed that the model experienced a lap splice failure at the top storey joints because of the use of plain rebars. At high PGA intensity most of the damage was concentrated at lower storey column bases and extensive damage was observed at beam-column connections; this showed high vulnerability of these deficient RC structures. Stavridis et al. [11] conducted shake table tests on a 2:3 scale three-storey and two-bay RC frame structure (Fig. 1 b) with older code design parameters and non-seismic detailing. The effects of infill panels were also investigated during the experimental program. Increasing inputs excitations were applied to test specimens in order to investigate the damage mechanism with low, moderate and high-level shaking intensities. It was observed during the testing program that at low level intensity, minor cracks were appearing in the infill panels, with the development of plastic hinge formation at column bases, i.e. cracking and joint shear failures at high level excitations. Yavari et al. [12] tested four 1:2.25 scaled two-storey and two-bay RC frames (Fig. 1 c) with non-seismic detailing in the beam and column members and no transverse reinforcement in the beam-column joints. The objectives of the experimental program were to investigate the collapse mechanism and gravity load redistribution with increasing dynamic excitations. Different factors affecting the behavior of these deficient RC frame structures during an earthquake loading including axial load demand on the column members and shear failure pattern of weak beam-column connections were evaluated. It was observed from the experimental study that the collapse of deficient RC frame structures may be a result of the plastic hinging mechanism produced at the base of non-ductile columns ends and critical shear failures of beam-column connections. Sharma et al. [13] conducted shake table test on a 3D, three-storey RC frame model (Fig. 1 d) with design deficiency of having no transverse reinforcement in beam-column connection and poor anchorage provision. The objectives of the testing program were to investigate the effectiveness of a tuned mass damper (TLD) retrofitting scheme and to observed the inelastic behavior of a non-seismic design RC frame structure under increasing dynamic excitations. It was observed from the test results that under increasing dynamic shaking, the model damage behavior is mostly concentrated at the lower storey columns base and beam-column panels. Most of the experimental studies available in the literature were focused on the vulnerability assessment of gravity type/old type RC buildings and/or the performance of a specific retrofitting, strengthening or isolation technique [14–28]. However, experimental shake table investigations on the recent special moment resisting frames (SMRF) structures (seismic code compliant beam and column members), but having weak beam-column joints (no transverse ties reinforcement and built with low strength concrete) and their seismic behavior at the ultimate damage state, are lacking.

In order to assess the ultimate capacity and damage mechanism of frames with weak beam-column joints, shake table tests were performed on representative frames structures. The shake table tests investigations have been performed on two 1:3 reduced scale two-storey RC frame specimens. Model-1 (reference model) was a code compliant model design based on the seismic building code. Whereas, Model-2 had similar characteristics but was provided with no ties in beam-column panel zone along with concrete strength less than the design specification (33 % less) to consider the effects of the most commonly available defects in the existing building stock [1, 3]. The test specimens were subject to multiple excitations using the natural accelerogram of 1994 Northridge Earthquake ranging from 5 % to 130 % of peak ground acceleration. These multiple scaled excitations were used in order to force the test models from elastic stage to inelastic and finally near collapse stage. The damage mechanism of each specimen was observed and reported. Acceleration and displacement response of the structure was recorded and analyzed to obtain the inter-storey drift demand, inter-storey shear and displacement profile of the structure and to develop deformation-based damage scale for seismic assessment of the considered structures.

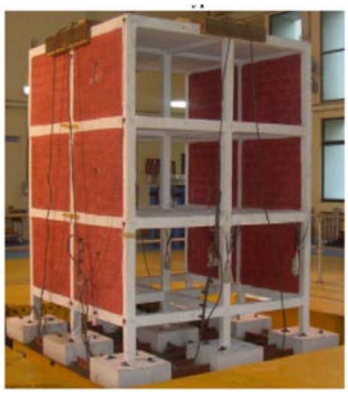
The objectives of the current experimental program were to understand the dynamic response, damage mechanism at ultimate capacity and to develop drift-based damage scales for RC frame structures with weak beam-column joint connections.

(a) 1:2.5 scale three storey two bays by one bay deficient RC model tested by Quintana et al. [10]



(b) 2:3 scale three storey RC frame tested by Stavridis et al. [11]

(c) 2:2.25 scale two storey and two bay RC frame tested by Yavari et al. [12]



(d) 1:3 scale three storey RC model tested by Sharma et al. [13]

Figure 1. Shake table tests investigations on deficient RC frame models.

2. Methods

2.1. Experimental Program

2.1.1 Test Specimens

The current research focused on low rise RC frame structures with weak beam-column joints (non-seismic detailing along with low strength concrete), in order to quantify their seismic performance and observe their damage response. In particular, a RC frame with a two storey structure was considered, typically employed for low rise schools, hospital or apartment type public buildings. The considered structure consisted of a 2 by 1 bay frame, with each bay length of 5487 mm (18 feet) and storey height of 3658 mm (12 feet) for both the stories, as shown in Fig. 2. The representative frame structure was designed according to static force-based procedures (BCP-SP 2007/UBC 97) [29–30], considering high seismic zone of 4.0 (0.40 g design peak ground acceleration) with stiff soil type B (NEHRP classification). The modeling and design were carried out using the design software CSI ETABS considering all load combinations as per the code. For the material properties, 21 MPa (3000 psi) concrete and 414 MPa (60,000 psi) rebar yield strength were considered in the design process. The design model was detailed as per the ACI-318-14 [31] recommendations for the SMRF provisions. Fig. 1 also shows the design details of the prototype structure.

2.1.2 Preparation of One-Third Reduced Models

The seismic shake table simulator in Earthquake Engineering Center, Department of Civil Engineering, UET Peshawar, has a table size of 1.5 m × 1.5 m (5 feet by 5 feet) and can operate with loading capacity of about 5 tons (11.01 kips) only in the unidirectional excitation. Due to the size and loading capacity limitations, only the interior critical frame was extracted from the prototype design building and reduced to one-third scale for seismic excitation as shown in Fig. 2. For scaling between the prototype and model dimension, a simple linear model idealization was considered as shown in Table 1. Such simple modeling allowed simplicity and reduced the cost as well as complexities associated with scaling the stress-strain properties of the materials (concrete and reinforcing bar) in the model domain. As shown in Table 1, all the members of the extracted prototype frame, i.e. beam, column and slab, and reinforcing rebar were reduced by a scale factor of S_L 3. For the constituents of concrete, 8.52 mm (3/8 in) down aggregate size was taken for model preparation, in order to use scaled coarse aggregate, whereas cement and fine aggregate were used with no scaling. As mentioned earlier two types of concrete strength were employed in this study, i.e. 21 MPa (3000 psi) and low strength test model with a 14 MPa (2000 psi) concrete compressive strength. For this purpose, ACI concrete mix design methods were followed for the preparation of concrete constituent mix.

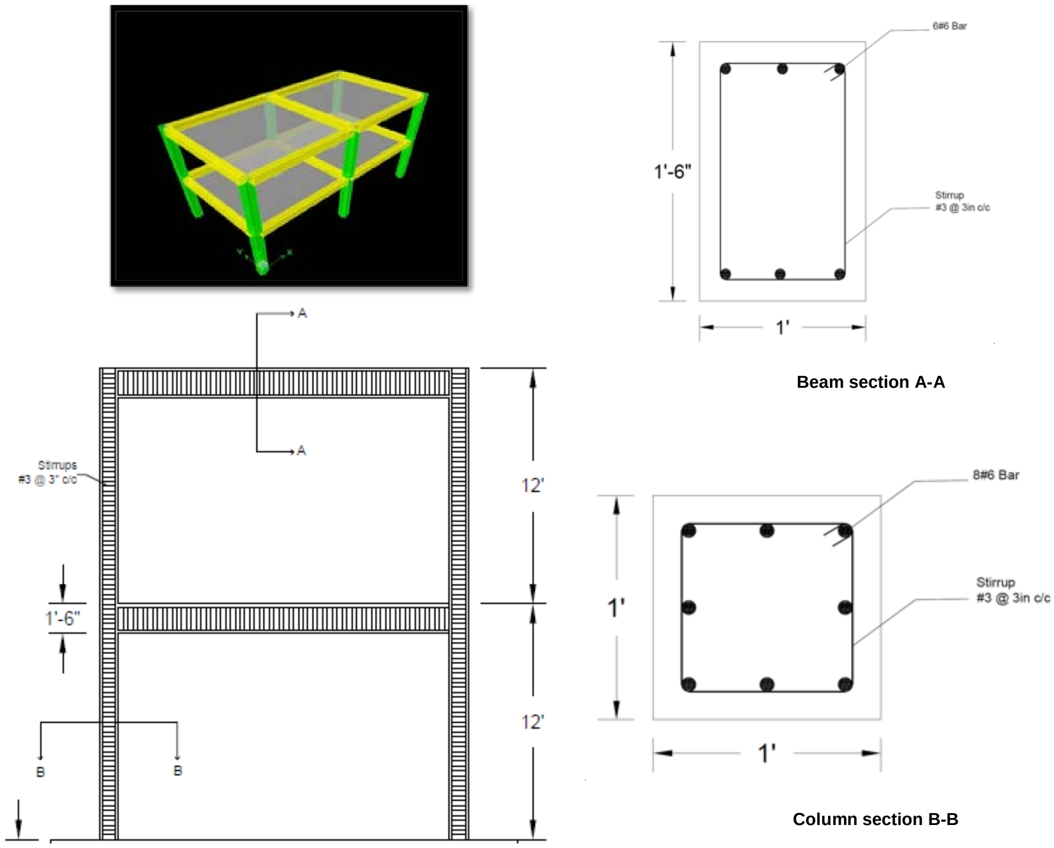


Figure 2. Geometric and design layout of prototype RC frame.

Table 1. Prototype and reduced models' dimensions and similitude conversion factors.

Structural Properties				
Prototype Frame	Test Models (Scale 1:3)	Simple Model Similitude Requirement		
		Physical Quantity	Relationship	Scale Factor
Beams: 304 mm × 459 mm (12 in × 18 in)	*Beams:* 102 mm × 153 mm (4 in × 6 in)	Length	$S_L = L_p/L_m$	3
Columns: 304 mm × 304 mm (12 in × 12 in)	*Columns:* 102 mm × 102 mm (4 in × 4 in)	Stress	$S_f = f_p/f_m$	1
Slab: 153 mm (6 in)	*Slab:* 51 mm (2 in)	Strain	$S_\epsilon = \epsilon_p/\epsilon_m$	1
Concrete strength: 21 MPa (3000 psi) 14 MPa (2000 psi)	*Concrete strength:* 21 MPa (3000 psi) 14 MPa (2000 psi)	Specific Mass	$S_\rho = \rho_p/\rho_m$	1
		Displacement	$S_d = d_p/d_m = S_L$	3
		Force	$S_F = F_p/F_m = S_L 2 S_f$	9
Aggregate size: 25.4 mm (1 in)	*Aggregate size:* 9.52 mm (3/8 in)	Time	$S_t = t_p/t_m = = S_L\sqrt{(S_\epsilon S_\rho/S_f)}$	3
Steel strength and dia: 414 MPa (60000 psi) 19 mm (#6 Rebar) 10 mm (#3 Rebar)	*Steel strength and dia:* 60000 psi (414 MPa) 6.33 mm (#2 Rebar) 3.33 mm (#1 Rebar)	Frequency	$S_\Omega = \Omega_p/\Omega_m = 1/S_t$	1/3
		Velocity	$S_v = v_p/v_m = = \sqrt{(S_\epsilon S_\rho/S_f)}$	1
		Acceleration	$S_a = a_p/a_m = S_f/S_L S_\rho$	1/3

The concrete mix proportion of cement:sand:coarse aggregate, for both design specified strength of 21 MPa (3000 psi) and reduced strength of 14 MPa (2000 psi) are shown in Table 1. Fig. 3 shows the sequence of construction for the 1:3 scaled frame models. Due to scaling and similitude requirements the reduced scale models were subjected to gravity and seismic mass less than the required. To satisfy the condition of mass simulation for the reduced scale test models, additional floor mass was applied following the mass simulation model [10, 32]. On each floor level extra mass of 1200 kg was applied through steel blocks mounted and fixed to the floor by means of fully secured 13 mm (½ inch) steel bolts as shown in Fig. 4.

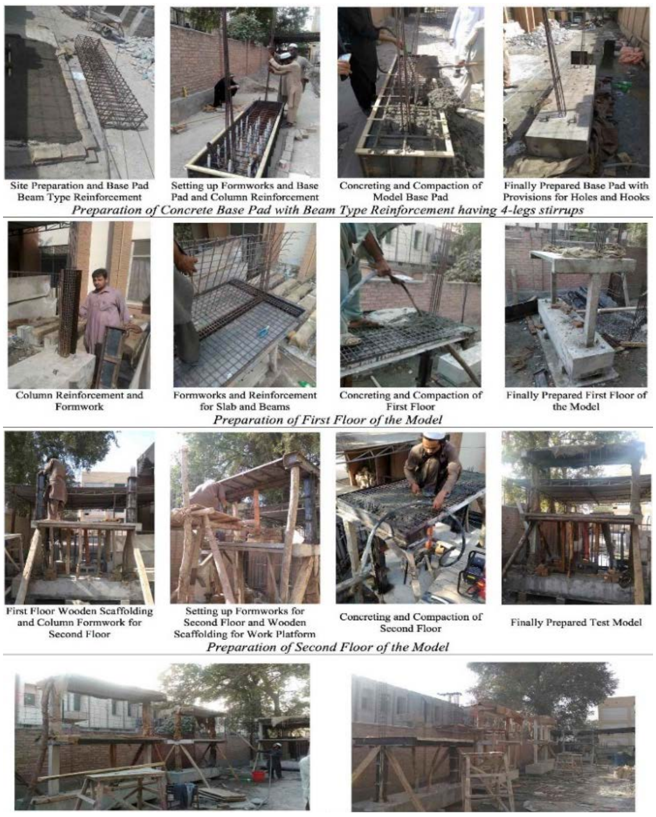


Figure 3. Test frames construction stages.

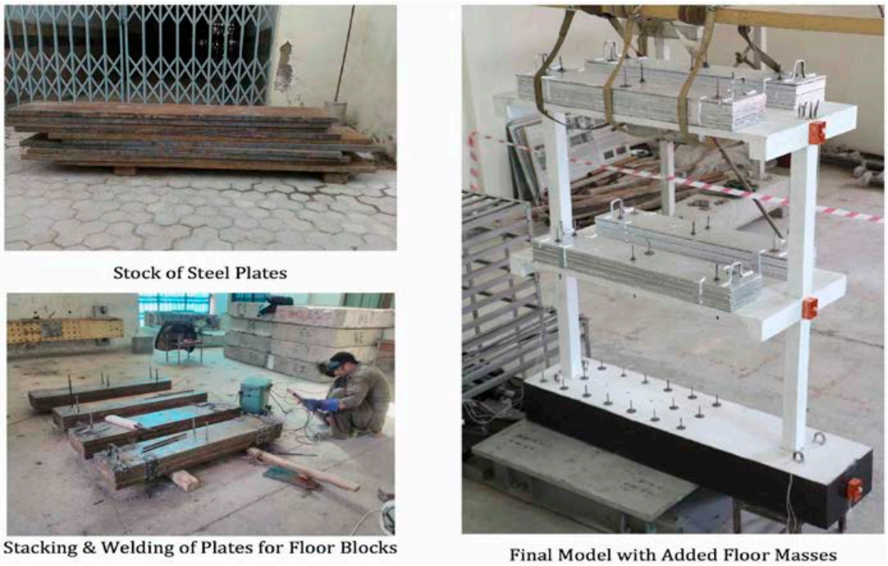


Figure 4. Added floor mass preparation and setup for artificial mass simulation.

2.2. Shake Table Test of RC frames Specimens

2.1.3 Testing setup and model instrumentation

The test model's setup on the shaking simulator and model instrumentation are shown in Fig. 5. The objective of experimental testing was to observe the global response of the test frames, so only external instrumentation was employed in the form of six accelerometers and three linear displacement transducers. The instruments sensitivity, maximum capacity and conversion coefficient are reported in Table 2. On each floor level (mid position of joint panel region) and at the base pad level, three accelerometers were installed on the front and back sides of the test models in order to record floor and base pad accelerations. For recording the displacement at each floor and pad level, three displacement transducers were attached to a fixed steel frame, which was installed in-line with the model in-plane position.

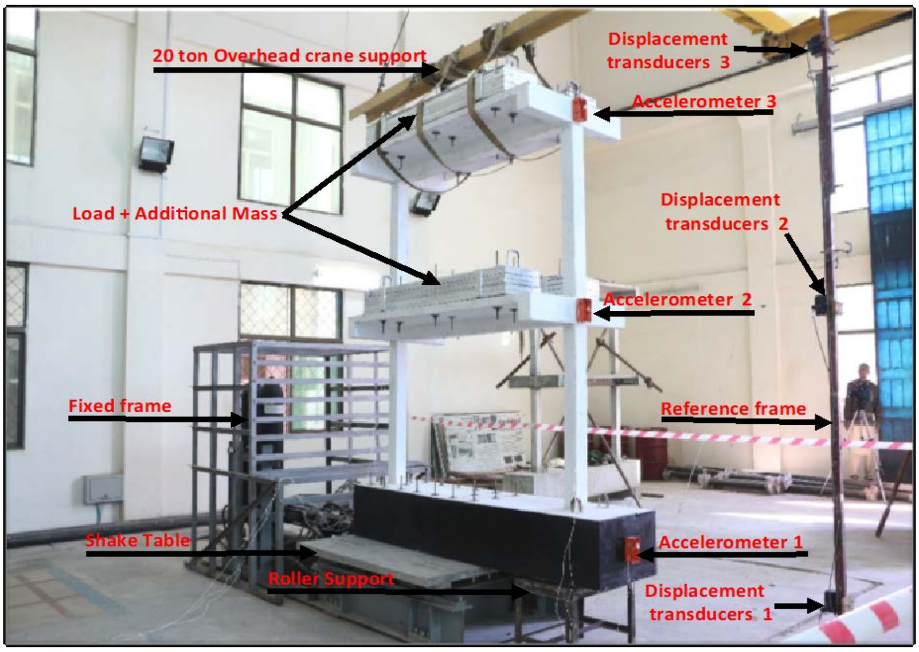


Figure 5. Test frame instrumentation and shaking setup.

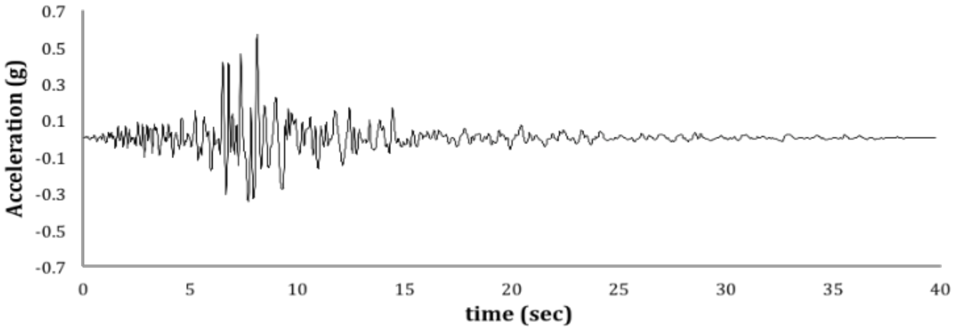


Figure 6. Input time history record of 1994 Northridge Earthquake.

2.1.4 Shake table Input Loading Protocols

The test models were tested by the Earthquake Engineering Center (EEC)'s seismic simulator in a unidirectional motion. Table 3 reports the characteristics limit values of the shake table. In order to excite the test models from elastic stage to full ultimate collapse stage, and also to be within the shaking range of shaking simulator, the 1994 Northridge Earthquake natural acceleration record was selected. This selection was made after careful analysis of a number of natural acceleration records to be within the range of shake table limiting acceleration, velocity and displacement limits. Fig. 6 shows the horizontal component of 1994 Northridge Earthquake record acceleration time history. The record was obtained from PEER strong motion data base (090 CDMG Station 24278) and had a peak acceleration, velocity and displacement values of 0.57 g, 518 mm/sec (20.39 in/sec) and 90 mm (3.54 in), respectively. The acceleration record time step was reduced by 1/3 to satisfy the input frequency requirement for the model as mentioned in Table 1. Table 4 and 5 reported the input multiple excitations and test sequence for both code compliant Model-1 and Model-2. Both frame models were excited with incremental excitations of 5 %, 10 %, 20 %, 30 %, 40 %, 50 %, 60 %, 70 %, 80 %, 90 %, 100 % and 130 % depending on the ultimate capacities of the tested frame. The idea of using multiple scaled excitation, i.e. from low to high level, was to deform and force the test structures from elastic to inelastic stage and then to attain full ultimate incipient collapse stage. The EEC's seismic simulator performs a self-adjustment motion called Self-Check, once the input time history is given to the shake table. After the simulator self-check adjustment, the test models were excited with scaled incremental excitations of the maximum acceleration records. After each run the damage mechanism of the test models was observed and documented with snapshots. The test sequence was progressing until the test models reached the near incipient collapse stage, after which the test was concluded. The recorded acceleration and displacement response time histories were obtained for each test run in the form of voltage values.

2.1.5 Recorded Data Processing

The accelerometers and displacement transducers recorded the data in the form of voltage values (mV). To get the time histories values in the form of accelerations (g) and displacements (mm), the recorded raw data needed to be divided by the instruments conversion coefficient as reported in Table 2. Once the recorded data were corrected for the respective instrument coefficient, the raw data were further processed for base line correction and signal filtering. This data processing correction and filtering was done to remove any noise in the actual recorded data. For this purpose, the SeismoSignal (SeismoSoft 2018) data processing software was employed with a linear category base line correction and filter type of Butterworth with frequency range of 0.10 Hz to 25 Hz considered.

Table 2. Test model instrumentation positions and characteristics.

Channel	Position	Direction	Maximum Capacity	Parameter	Coefficient	Unit
A1	Pad level	Front	±10g	Acceleration	492.20	mv/g
A2	First Floor	Front		Acceleration	501.10	mv/g
A3	Second Floor	Front		Acceleration	510.10	mv/g
A4	Pad level	Back		Acceleration	508.90	mv/g
A5	First Floor	Back		Acceleration	490.10	mv/g
A6	Second Floor	Back		Acceleration	502.00	mv/g
D7	Pad level	Front	24 inches (610 mm)	Displacement	1000.00	mv/inch
D8	First Floor	Front		Displacement	1000.30	mv/inch
D9	Second Floor	Front		Displacement	1000.20	mv/inch

Table 3. Earthquake Engineering Centre (EEC)'s seismic simulator (shake table) limits.

Shaking simulator characteristics	Limits
Excitation direction	Single degree of freedom / unidirectional
Physical dimension	1.5m × 1.5m (5 feet × 5 feet)
Pay load capacity	5 tones
Maximum acceleration	1.1 g
Maximum velocity	±1.1 m/s
Maximum displacement	±125 mm

Table 4. Input protocol and testing sequence for Model-1.

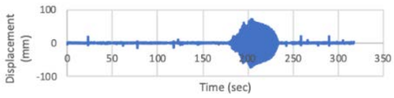
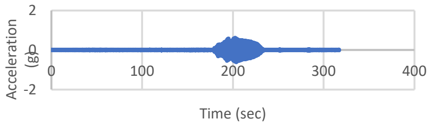
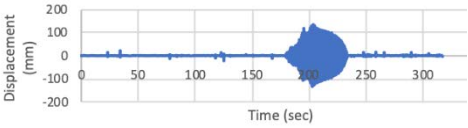
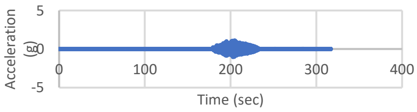
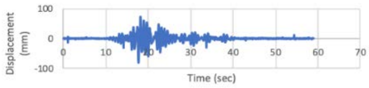
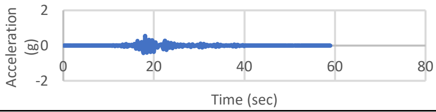
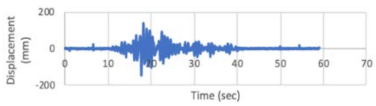
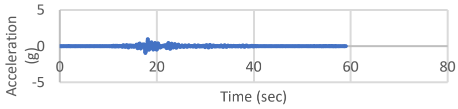
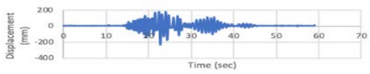
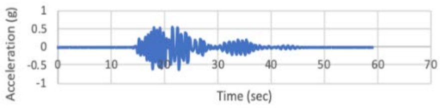
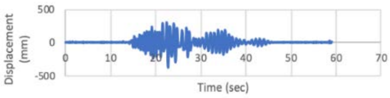
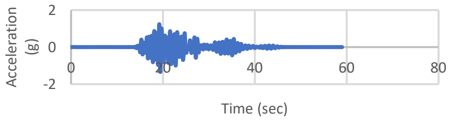
Time History	Run (%)	Observed Input PGA (g)
1994 Northridge Earthquake	Self-Check	0.60
	5% Run	0.033
	10% Run	0.06
	20% Run	0.12
	30% Run	0.16
	40% Run	0.19
	50% Run	0.25
	60% Run	0.31
	70% Run	0.36
	80% Run	0.41
	90% Run	0.49
	100% Run	0.62
	Self-Check 130%	0.62
	130% Run	1.06

Table 5. Input protocol and testing sequence for Model-2.

Time History	Run (%)	Observed Input PGA (g)
1994 Northridge Earthquake	Self-Check Run	0.015
	5% Run	0.52
	10% Run	0.25
	20% Run	0.31
	30% Run	0.35
	40% Run	0.73

Once the processed data were obtained, the displacement and acceleration histories were converted from the model domain to the prototype domain using the scaling conversion factors as mentioned in Table 1. Table 6 shows the experimentally obtained, first and second floor level acceleration and displacement time histories for the Model-1. Table 6 shows the time histories records for the selected significant runs in the prototype domain. For obtaining the relative displacements of each floor relative to the base of the pad, the displacement histories of the base pad were subtracted from each floor level displacements. For each test run, the peak values of displacement were obtained and normalized by the height of the storey to obtain the corresponding first and second floor drifts. To calculate the floor inertial forces at each floor level as well as the total base shear force at base of the porotype model, each of the floor accelerations were multiplied by each floor total mass, which included: the additional block mass, self-weight of the slab, self-weight of the beam and half column above and below the floor level. The inertial forces at each floor level were added to obtain the total base shear force at the base of frames.

Table 6. Model-1 experimental observed displacement and acceleration histories.

Run	Floor level	Displacement History	Acceleration History
Self-Check Run	Ground floor level		
	First floor level		
100% Run	Ground floor level		
	First floor level		
130% Run	Ground floor level		
	First floor level		

3. Results and Discussions

Table 7–8 shows Model-1 and Model-2 maximum roof displacement, maximum drift ratio, maximum base shear force and the observed damage mechanism for the selected significant runs. Fig. 7 shows the test model's comparison at the final run, i.e. incipient collapse stage and joint panel damage mechanisms. The code design Model-1 was initially excited by shake table self-check run which forced the structure to a drift of about 1.88 % and with shaking intensity of 0.60 g. During this first run the model developed significant beam flexural cracks at the first storey level. This flexural cracking was due to the reinforcement rebar yielding and plastic hinge mechanism. Minor vertical cracks were observed in the beam on the ground storey at the beam-column interface, which was due to the beam's longitudinal steel bars slip. This longitudinal beam's bar slip was observed also in full scale special moment resisting beams tested under quasi-static cyclic loading [33, 34]. During this run, there were flexural cracks observed on the ground storey at columns bases and on the first storey at beam ends. The model was then subjected to multiple excitations from 5 % to 100 %, with slight increase in cracking pattern. After this the test model was subjected to 130 % of the maximum acceleration record during which the model experienced shaking intensity of about 1.06 g. During this run the previous damage got significantly aggravated. The test model experienced concrete crushing and core spalling at the base and top ends of the columns on the ground storey due to excessive compressive strain demand on the cover concrete. Minor spalling was also observed at the base of columns on the first storey. Additionally, the model was observed with severe diagonal cracks in the joint panel region on the ground storey and slight diagonal cracks in the joint region on the first storey, which was due to transferring moments from beam-ends to columns' ends.

Table 7. Observed damage in Model-1.

Run	Top storey maximum displacement mm (Inch)	Top storey maximum Drift (%)	Maximum Base shear force kN (kips)	Observed damage
Self-Check – 0.60 g	61.45 (2.42)	0.87	151.08 (33.96)	 Flexure Cracks at Base of Columns  Flexure Horizontal & Vertical Cracks in Beam
100% – 0.62 g	133.56 (5.26)	1.88	188.90 (42.47)	 Concrete Crushing at Ground Storey Column Top  Cover Spalling at Ground Storey Column Base
130% – 1.06 g	373.03 (14.69)	5.26	254.73 (57.27)	 Diagonal Cracks in Joint Panel, Ground Storey  Diagonal Cracks in Joint Panel, First Storey

This damage pattern testified to the existence of materials' over-strength in beams that resulted in plastic section moment capacity higher than the yield moment capacity, consequently, increasing demands on the joint region [12, 14].

Table 8. Observed damage in Model-2.

Run	Top storey maximum displacement mm (Inch)	Top storey maximum Drift (%)	Maximum Base shear Force kN (kips)	Observed Damage
5%– 0.52g	123.69 (4.87)	1.75	117.92 (26.51)	 Flexural Cracks in Beams and Columns, Ground Storey  Slight Cracks in Joint Panel on Ground Storey
30% – 0.35 g	182.37 (7.18)	2.57	137.48 (30.91)	 Severe Bat-Like Cracks in Joints on First Storey  Damage Progress in Joint Panel on Ground Storey
40% – 0.73 g	338.19 (13.31)	4.77	184.50 (41.48)	 Cover Detachment and Damage in Joint on First Storey  Severe Damage to Joint Panel on Ground Storey

In comparison to Model-1, weak beam-column deficient Model-2 deformed laterally to larger roof drift under similar input excitations and damage to structural members (beam and columns) and beam-column joint panels occurred at comparatively lower excitation demand. Model-2 was able to only resist 40 % of the PGA value (0.73 g) as compared to Model-1. Unlike Model-1, Model-2 experienced damage in joints much earlier and to extreme extent under significantly lower excitations. This is due to the fact of using low strength concrete in SMRFs. This reduces the steel-to-concrete bond strength and allows steel bars slip through concrete, consequently resulting in larger displacement of the model. Unlike Model-1, the damage evolution showed that damage in Model-2 was more limited to the joint region than the columns and beams. Furthermore, the joint panels were damaged under less shear demand (in transferring beam moments to columns) due to the lower principal tensile strength of the joint panel, since the joint principal strength capacity primarily depends on the strength of core concrete that is related to the compressive strength of concrete [13, 35]. Joint cracks in Model-1 spread over larger area (joint panel core and transverse beams), whereas joint cracks in Model-2 spread primarily within the joint panel core.

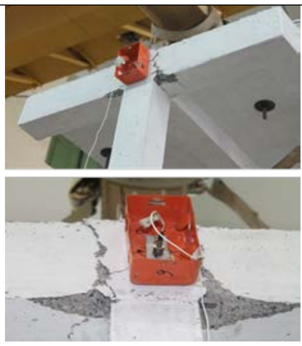
		Ground storey Joints Front and back Joints
		First storey Joints Front and back Joints
Model-1 at 130% Run	Model-2 at 40% Run	Final/Incipient collapse Run
1.06 14.69 (373.03) 5.26 57.27 (254.73)	0.73 13.31 (338.19) 4.77 41.48 (184.50)	PGA, g Displacement, in (mm) Drift, % Base shear, kips (kN)

Figure 7. Comparison of test models at final/incipient collapse stage.

Most of the global performance assessment studies require development of damage scale that specify strength or deformation limits and expected damage conditions of structure for various performance levels. The present study developed a drift-based damage scale for the considered deficient frame as per the damage levels: slight damage, moderate to heavy damage and critical damage, compatible with FEMA (2003) [36] for building seismic performance limit state, i.e. operational, life safety and collapse prevention. The developed drift-based performance level and corresponding damage for deficient weak beam-column joint frame model are reported in Table 9. As it can be seen from Table 9, the performance limit state for the case of operational level corresponds to drift of about 1.75 % with slight damage at the column bases and beam ends. With the drift demand of about 2.57 %, the test model was in the life safety limit state with moderate to heavy damages at column beam members ends and in the joint panel regions. At a drift demand of about 4.77 % the frame was in the collapse prevention limit state, with heavy non-repairable damages at members end and severe damage at joint panel regions. The drift-based performance levels and damage mechanism can be used for the damage models and fragility functions development in the context of performance based seismic assessment and economic loss estimation studies for deficient RC frames structures.

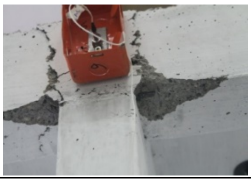
4. Conclusions

Recent reinforced concrete SMRF structures with weak beam-column joints defects were evaluated through dynamic shake table testing, in order to assess the seismic performance and ultimate damage mechanism of such class of deficient RC structures. The test models consisted of a code design SMRF model and a deficient model without transverse ties reinforcement in the beam column panels and built in low strength concrete. Unidirectional dynamic shake table tests were used to shake the test model under various scaled excitations in order to observe seismic damage mechanism and develop drift-based performance limit states.

The following are the main conclusions from this experimental research work:

1. The code design SMRF model experienced beam and column plastic hinge formation at the member ends under extreme ground shaking and complied with the code well (1.3 times (1.06 g) more than the input design level shaking).

Table 9. Drift based performance levels and damage scales for the weak beam-column joint frame.

Limit State	Damage State	Damage Description	Experimental (Drift %)
LS-1 Operational	Slight Damage	- Slight cracks at the base of columns on the ground storey. - Flexural cracks and slight vertical cracks at the beam-ends on both the ground storey and first storey. 	1.75%
LS-2 Life Safety	Moderate to Heavy Damage	- Flexural cracks at the base of columns widened on both the ground storey and first storey. - Flexural and vertical cracks at beam-ends significantly widened in beam on ground storey. - Damage in the joint panels on ground storey aggravated. 	2.57%
LS-3 Collapse Prevention	Critical Damage to Incipient Collapse	-Significant flexural cracks at top end and also at the bottom end of all columns and beam ends on both ground storey and first storey. -Joint regions on both ground storey and first storey with extreme damage and diagonal bat-like cracks. -Bat-like concrete cover wedge detachment from joint regions. - Heavy Concrete crushing and cover spalling at column base and beam ends. 	4.77%

2. We observed in the current study that beam longitudinal reinforcement was subject to bar slip and bar pullout due to the use of low-strength concrete that resulted in steel-concrete bond failure.

3. Concrete structures in which joints were not provided with confining ties showed joint damageability, while cover and core concrete showed spalling upon being subject to large lateral deformation.

4. The experimental testing on the deficient weak beam-column joint RC frame revealed that such frame building typology exhibited a mixed mechanism of column and beam members hinging followed by severe beam-column joint panel damage. This also resulted in altering the damage mechanism of the structure from beam-sway to column-sway and joint panel mechanism that consequently resulted in lowering the lateral strength and displacement ductility of the structural frames. This can result in significant vulnerability of the considered frame structures. The performance states of this structure type are dictated

by the joint damage state and their reparability issues that the engineers may face following damaging earthquake events.

5. The weak beam-column joint SMRF model was able to only resist 40 % of the PGA value (0.73 g) as compared to reference code design SMRF model, which was able to resist 130 % (1.06 g).

6. The experimental tests showed that the deficient weak beam-column joint frame reached the operational limit state at a drift demand of 1.75 %, life safety limit state at a drift demand of 2.57 %, and was found to be in the incipient collapse state at a drift demand of about 4.77 %.

References

1. Badrashi, Y.I., Ali, Q., Ashraf, M. Reinforced Concrete Buildings in Pakistan – Housing Report. 159. Earthquake Engineering Research Institute. 2010. Oakland, CA. Pp. 1–16.
2. Ahmad, N., Shahzad, A., Rizwan, M., Khan, A.N., Ali, S.M., Ashraf, M., Naseer, A., Ali, Q., Alam, B. Seismic Performance Assessment of Non-Compliant SMRF Reinforced Concrete Frame: Shake Table Test Study. Journal of Earthquake Engineering. 2017. Vol. 21. DOI: 10.1080/13632469.2017.1326426
3. Badrashi, Y.I., Ali, Q., Ashraf, M., Rashid, M. Seismic design characterization of RC special moment resisting frames in Pakistan-field survey to laboratory experiments. Journal of Engineering and Applied Sciences. 2016. 35(2). Pp. 25–32.
4. Bilham, R. Earthquakes in India and in the Himalaya: tectonic. Geodesy, and history. Ann Geophysics 2004. 47(2/3). Pp. 839–858.
5. Naseer, A., Khan, A.N., Hussain, Z., Ali, Q. Observed Seismic Behavior of Buildings in Northern Pakistan during the 2005 Kashmir Earthquake. Earthquake Spectra. 2010. Vol. 26. No. 2. Pp. 425–449. DOI: 10.1193/1.3383119
6. Arslan, M.H., Korkmaz, H.H. What is to be Learned from Damage and Failure of Reinforced Concrete Structures during Recent Earthquakes in Turkey. Engineering Failure Analysis. 2007. Vol. 14. No. 1. Pp. 1–22. DOI: 10.1016/j.engfailanal.2006.01.003
7. Erdil, B. Why RC Buildings Failed in the 2011 Van, Turkey, Earthquakes: Construction versus Design Practices. Journal of Performance of Constructed Facilities. 2016. Vol. 31. No. 3. DOI: 10.1061/(ASCE) CF.1943-5509.0000980
8. Ruiz-Pinilla, J.G., Adam, J.M., Perez-Carcel, R., Yuste, J., Moragues, J.J. Learning from RC Building Structures Damaged by the Earthquake in Lorca, Spain, in 2011. Engineering Failure Analysis. 2011. Vol. 68. 2016. Pp. 76–86. DOI: 10.1016/j.engfailanal.2016.05.013
9. Ates, S., Kahya, V., Yurdakul, M., Adanur, S. Damages on Reinforced Concrete Buildings due to Consecutive Earthquakes in Van. Soil Dynamics and Earthquake Engineering. 2013. Vol. 53. Pp. 109–118. DOI: 10.1016/j.soildyn.2013.06.006
10. Quintana-Gallo, P., Pampanin, S., Carr, A.J., Bonelli, P. Shake Table Tests of Under Designed RC Frames for the Seismic Retrofit of Build- ings – Design and Similitude Requirements of the Benchmark Specimen. Proceedings of the New Zealand Society of Earthquake Engineering. 2010. Paper No. 39. 12 p.
11. Stavridis, A., Koutromanos, I., Shing, P. S. Shake-Table Tests of a Three-Storey Reinforced Concrete Frame with Masonry Infill Walls. Earthquake Engineering & Structural Dynamics. 2012. Vol. 41. No. 6. Pp. 1089–1108. DOI: 10.1002/eqe.1174
12. Yavari, S., Elwood, K.J., Wu, C.L., Lin, S.H., Hwang, S.J., Moehle, J.P. Shaking Table Tests on Reinforced Concrete Frames without Seismic Detailing. ACI Structural Journal. 2013. Vol. 110. No. 6. Pp. 1000–1012.
13. Sharma, A., Reddy, G. R., and Vaze, K. K. Shake Table Tests on a Non-Seismically Detailed RC Frame Structure. Structural Engineering and Mechanics. 2012. Vol. 41. No. 1. Pp. 1–24. DOI: 10.12989/sem.2012.41.1.001
14. Elwood, K.J., Moehle, J.P. Shake Table Tests and Analytical Studies on the Gravity Load Collapse of Reinforced Concrete Frames. PEER Report 2003/01. Pacific Earthquake Engineering Research Center. 2003. University of California, Berkeley, Berkeley, CA. 364 p.
15. Calvi, G.M., Magenes, G., Pampanin, S. Experimental Test on a Three-Story R.C Frame Designed for Gravity Only. Proceedings of the 12th European Conference on Earthquake Engineering. 2002. London, UK.
16. Benavent-Climent, A., Morillas, L., Escolano-Margarit, D. Seismic Performance and Damage Evaluation of a Reinforced Concrete Frame with Hysteretic Dampers through Shake-Table Test. Earthquake Engineering & Structural Dynamics. 2014. Vol. 43. No. 15. Pp. 2399–2417. DOI: 10.1002/eqe.2459
17. Magliulo, G., Petrone, C., Capozzi, V., Maddaloni, G., Lopez, P., Manfredi, G. Seismic Performance Evaluation of Plasterboard Partitions via Shake Table Test. Bulletin of Earthquake Engineering. 2014. Vol. 12. No. 4. Pp. 1657–1677. DOI: 10.1007/s10518-013-9567
18. Koutromanos, I., Kyriakides, M., Stavridis, A.; Billington, S., Shing, P.B. Shake-Table Tests of a 3-Story Masonry-Infilled RC Frame Retrofitted with Composite Materials. Journal of Structural Engineering, ASCE. 2013. Vol. 139. No. 8. 2013. Pp. 1340–1351. DOI: 10.1061/(ASCE) ST.1943-541X.0000689 Earthquake Engineering, V. 11, No. 1, pp. 69–91. DOI: 10.1007/s10518-012-9404-5 ????
19. Garcia, R., Hajirasouliha, I., Pilakoutas, K. Seismic Behaviour of Deficient RC Frames Strengthened with CFRP Composites. Engineering Structures. 2010. Vol. 32. No. 10. Pp. 3075–3085. DOI: 10.1016/j. engstruct.2010.05.026
20. Dolce, M., Cardone, D., Ponzo, F. Shaking-Table Tests on Reinforced Concrete Frames with Different Isolation Systems. Earthquake Engineering and Structural Dynamics. 2007. Vol. 36. No. 5. Pp. 573–596. DOI: 10.1002/eqe.642
21. Hashemi, A., Mosalam, K.M. Shake-Table Experiment on Reinforced Concrete Structure Containing Masonry Infill Wall. Earthquake Engineering and Structural Dynamics. 2006. Vol. 35. No. 14. Pp. 1827–1852. DOI: 10.1002/eqe.612
22. Pinto, A., Varum, H., Molina, J. Experimental Assessment and Retrofit of Full-Scale Models of Existing RC Frames. Proceedings of the 12th European Conference on Earthquake Engineering. 2002. London, UK.
23. Hakuto, S., Park, R., Tanaka, H. Seismic Load Tests on Interior and Exterior Beam-Column Joints with Substandard Reinforcing Details. ACI Structural Journal. 2002. Vol. 97. No. 1. Jan.-Feb. Pp. 11–25.
24. Zarnic, R., Gostic, S., Crewe, A.J., Taylor, C.A. Shaking Table Tests of 1:4 Reduced-Scale Models of Masonry Infilled Reinforced Concrete Frame Buildings. Earthquake Engineering & Structural Dynamics. 2001. Vol. 30. No. 6. DOI: 10.1002/eqe.39
25. Beres, A., Pessiki, S., White, R., Gergely, P. Implications of Experiments on the Seismic Behaviour of Gravity Load Designed RC Beam-to-Column Connections. Earthquake Spectra. 1996. Vol. 12: No. 2. Pp. 185–198. DOI: 10.1193/1.1585876

26. Bracci, J., Reinhorn, A., Mander, J. Seismic Retrofit of Reinforced Concrete Buildings Designed for Gravity Loads: Performance of Structural Model. ACI Structural Journal. 1995. Vol. 92. No. 6. Nov.-Dec. Pp. 711–723.
27. Aycardi, L.E., Mander, J.B.; Reinhorn, A.M. Seismic Resis- tance of Reinforced Concrete Frame Structures Designed Only for Gravity Loads-Experimental Performance of Subassemblages. ACI Structural Journal. 1994. Vol. 91. No. 5. Pp. 552–563.
28. El-Attar, A.G., White, R.N., Gergely, P. Shake Table Test of a 1/6 Scale Two Story Lightly Reinforced Concrete Building. Technical Report NCEER-91 0017I, National Center for Earthquake Engineering Research (NCEER). 1991. Buffalo, NY. 118 p.
29. BCP (Building Code of Pakistan: Seismic Provisions). 2007. Ministry of Housing and Works, Islamabad, Pakistan. 302 p.
30. UBC (Uniform Building Code). International Conference of Building Officials. 1997 Whittier, CA.
31. ACI (American Concrete Institute). Building code requirements for structural concrete. ACI 318. Farmington Hills. 2014. MI: ACI.
32. Morcarz, P., Krawinkler, H. Theory and Application of Experimental Model Analysis in Earthquake Engineering. 1981. Technical Report Report No. 50, CA, 1981, 272 pp.
33. Ahmad, N., Rashid, M., Waqas, A. Quasi-static cyclic tests on special moment resisting beams. Technical Report, Earthquake Engineering Center. 2016. UET Peshawar, Khyber Pakhtunkhwa, Pakistan.
34. Rashid, M. 2016 "Earthquake Induced Damages in Modern Reinforced Concrete Special Moment Resisting Frames: Repair-ability and Cost Implications" MSc Thesis, Civil Engineering Department University of Engineering and Technology Peshawar, Pakistan.
35. Pampanin, S., Calvi, G.M., Moratti, M. "Seismic Behavior of R.C. Beam-Column Joints Designed for Gravity Only. Proceedings of the 12th European Conference on Earthquake Engineering. 2002. Paper No. 726, London, UK. Pp. 1–10.
36. FEMA. "NEHRP Recommended Provisions for Seismic Regulations for New Buildings." FEMA 450, Federal Emergency Management Agency. 2003. Washington, DC.

Information about authors:

***Muhammad Rizwan**, PhD*
E-mail: rizwan.civil@suit.edu.pk

***Naveed Ahmad**, PhD*
ORCID: https://orcid.org/0000-0003-1275-8380
E-mail: naveed.ahmad@uetpeshawar.edu.pk

***Akhtar Naeem Khan**, PhD*
E-mail: drakhtarnaeem@uetpeshawar.edu.pk

Received 27.03.2020. Approved after reviewing 28.06.2022. Accepted 01.07.2022.