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SEISMIC EVALUATION OF DEFICIENT REINFORCED CONCRETE WEAK BEAM-COLUMN JOINT FRAMES

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The use of old building design codes and improper execution of recent seismic design practices have caused large amount of substandard and vulnerable reinforced concrete RC building stock majority of which are built with weak beam-column joint connections defect (i.e. joint panel having no transverse reinforcement and built in low strength concrete). In order to understand the seismic response and damage behaviour of recent special moment resisting frame SMRF structures with the defect of weak beam-column joints, shake table tests have been performed on two 1:3 reduced scaled, two story RC frame models. The representative reference code design and weak beam-column joint frame models were subjected to uni-directional dynamic excitations of increasing intensities using the natural record of 1994 Northridge Earthquake. The input scaled excitations were applied from 5% to 130% of the maximum input peak ground acceleration record, to deformed the test models from elastic to inelastic stage and then to fully plastic incipient collapse stage. The weak beam-column frame experienced column flexure cracking, longitudinal bar-slip in beam members and observed with cover concrete spalling and severe damageability of the joint panels upon subjected to multiple dynamic excitations. The deficient frame was only able to resist 40% of the maximum acceleration input as compared to the code design frame which was able to resist about 130%. The seismic performance of considered RC frames was evaluated in terms of seismic response parameters (seismic response modification, overstrength and displacement ductility factors), for critical comparison.

Keywords: reinforced concrete, special moment resisting frame, response modification factor, beam-column joints

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1. INTRODUCTION

The construction of reinforced concrete (RC) frame buildings are growing abundantly in many developing countries of the world and mostly used for commercial, residential, office or school type building. The prime factors for this type of construction material and building system is the availability of vast resources of the constituents of concrete and easy adoptable construction methods. In the developing low-income countries, the seismic codes are mostly adopted based on the international building guidelines and code, with some regional adaptation of the region-specific seismicity characteristics. Due to the use of old building codes and also the improper use and execution of the current special moment resisting frame (SMRF) structures, seismic code detailing has caused majority of these RC building having different seismic defects or deficiencies. Different field surveys conducted recently in order to find the most typical construction and seismic deficiencies in the general RC building stock [1-2]. These field surveys have revealed that mostly the design or construction defects consist of substandard quality of building materials (i.e. low strength concrete and reinforcement material) along with different non-seismic provisions. The typical non-seismic practices that are currently available in the existing building stock consist of; not providing shear confining ties reinforcement in the beam-column joint panel regions, use of larger ties spacing in the RC structural member such as beam and columns, practicing reduced flexural reinforcement requirement for the beam and column members and provision of 90^{o} non-seismic hook for the shear ties and stirrups along with others [1,3]. In most of these construction defects the weak beam-column connection deficiency i.e. not having any confining shear ties in the joint panel critical regions and constructed with substandard low strength concrete as required per the code specifications, is very common. As also seen from past earthquakes damage observation of RC building typology, such defects can cause severe joint failures, thus reducing the load carrying capacity of the connecting members (beams and columns) and can cause partial or even complete collapse of the RC building systems [2]. Furthermore, majority of such RC buildings typology are currently existing in areas with very has high seismicity and which can be excited with the coming future earthquake demands [4]. It is also evident from the past earthquake observations that if reinforced concrete buildings having older non-seismic provisions or not design and constructed as per the proper design provisions will cause significant failure upon subjected by design level earthquake demands and which can cause huge human and economic losses [5-9]. All these shows the importance of the seismic evolution of RC frame system with design or construction defects with in context of seismic feasibility assessment of such building typology.

The present work aims to evaluate the seismic performance of RC special moment resisting frame (SMRF) structures with the weak beam-column connection defects (having no transverse confinement in beam-column panel regions and built with a low strength concrete of 14 MPa (2000 psi) which is about 33% less than the code specified concrete strength) using experimental dynamic shaking testing. Different experimental studies (Quasi-static and dynamic shake table tests) have been reported in the literature in order evaluate the seismic assessment of older code design buildings i.e. not having capacity design principles and non-seismic provisions in the structural members [10-20]. Various shaking test studies have also been reported on feasibility assessment of different retrofitting and strengthening technique provided to enhanced the performance of deficient RC frame buildings [21-24]. Most of these studies only focus on the older code design provisions, while the seismic assessment with the recent SMRF frame structures (i.e. beam and column members detailing as per SMRF) and having no confinement ties in the beam-column connections panel regions and also built with low strength concrete is not available. Similarly, the seismic response to real acceleration shaking record and as well the, the ultimate damage mechanism and seismic performance is also not well understood in the literature. In order evaluate the seismic performance of RC SMRF frames with weak beam-column connections, shaking table test on representative reduced scaled models have been conducted with multiple shaking excitations. The test structures consist of a code design frame Model-1, a representative of a critical frame of a two-story reinforced concrete building with typical geometry and design specification. For the seismic evolution of weak beam- column connection defects, a similar model but built with weak beam-column connections i.e. no confining shear ties and built in low strength concrete (33% less than the code specified compressive strength) has been considered as Model-2. The representative frame models were shake in a uni-directional movement using the 1994 Northridge Earthquake acceleration record with increasing shaking inputs from 5% to 130% of the peak acceleration record. The damage response and the experimental observed data in the form of test model acceleration and displacement histories have been recorded for each run and have been further analyze for the computation of seismic responses evolution in terms of force-deformation relationships and seismic response parameter (response modification factors) for critical comparisons.

2. Description of considered RC frame structure

For the seismic assessment of RC frame structures with the defect of having weak beam-column connections, this study considered a typical low-rise building. For this purpose, a two-story regular frame building structure with two bays by one bay configuration and with bay length of 5486 mm (18 feet) and story height of 3657 mm (12 feet), typically used for school, commercial and residential type building was considered. The seismic analysis of the RC frame building was performed using the static lateral force procedures for a region of high seismicity (0.40g peak ground acceleration) and with stiff soil characteristics. For the concrete material, a compressive strength of 21 MPa (3000 psi) and for reinforcement material yield strength of 413 MPa (60,000 psi) was used. The analysis and design were performed using the CSI ETABS design software considering the load combinations as per the Building Code of Pakistan (BCP-SP 2007) and Uniform Building Code (UBC-97) [25-26] and the seismic detailing was performed as per the American Concrete Institute ACI 318-14 [27] special moment resisting frame SMRF seismic detailing provisions. Fig. 1 shows the frame geometry and design details of the beam and column members. In order to evaluate the seismic assessment and observed damage response of frame structures with weak beam-column connections, a second deficient Model-2, having similar properties to that of code design Model-1, but, was not provided with any ties in the beam-column joint panels and the construction was performance using low strength concrete of 14 MPa (2000 psi), was considered. The low strength concrete used in the deficient Model-2 construction is about 33% less than the minimum specified code value of 21 MPa (3000 psi). For the steel rebar the similar yield strength of 413 MPa (60000 psi) was considered for Model-2. For the dynamic shake table testing the interior frame (critical direction) was selected from the prototype building and was reduced by a factor of 3, in order to be with in the shaking simulator limitations. The scaling and mass simulation of the representative interior frame was performed based on the principals of simple model idealization [28,29]. Table 1 report the scaling criterion selected for the preparation of reduced scale frame models. In the simple scaling model idealization, the stress-strain relations of the materials remain the same in both the prototype and model domain. Such simple idealization was adopted because of its simplicity, low cost and less associated complexity in modeling the material behavior in model domain [10,28]. The frame members dimensions were scaled by a factor of SL = 3. For the model frame, a reduced size beam dimension of 101.6 mm x 152.5 mm (4 in x 6 in), column dimension of 101.6 mm x 101.6 mm (4 in x 4 in) and slab thickness of 50.8 mm (2 in) was used. Similarly, for story height and bay length, a 1/3rd reduced length of 1219.2 mm (4 feet) and 1828.8 mm (6 feet) was used for the test model preparation.

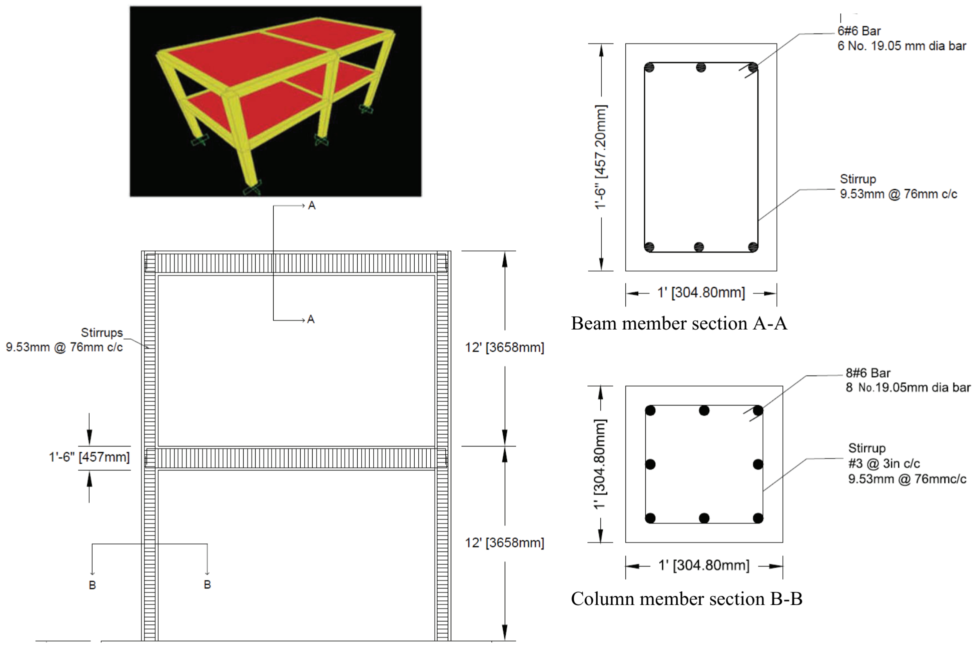


Fig. 1. Layout of two-story frame model, geometry and members design details

Table 1 Simple model similitude scaling factors

Physical Quantity	Scale Factor
Length	3
Stress	1
Strain	1
Specific Mass	1
Displacement	3
Force	9
Time	$\sqrt{3}$
Frequency	$\sqrt{1/3}$
Velocity	1
Acceleration	1/3

For the concrete constituents scaling, only the coarse aggregate size scaling has been performed with 8.52 mm (3/8 in) down size aggregates were used in the model preparation, whereas the fine aggregate and cement have been used without any scaling. The ACI mix design procedures were used in order to obtain different batches of concrete constituents for a code specified concrete strength of 21 MPa (3000 psi) and low strength concrete of 14 MPa (2000 psi). Two reduced scaled models i.e. code design reference Model-1 and a deficient weak beak-column joint Model-2 were constructed after the specified code design and low strength concrete cylinders were tested and verified. Initially a strong reinforced concrete pad base (strong beam having dimensions of 558.8 mm (22 in) depth, 406.4 mm (16 in) width and 2438.4 mm (8 feet long)) was constructed, and was provided with holes at regular intervals to be used to mount and fixed the test frames on shake table steel platform through steel bolts. After the construction of the fixed supported pad, the ground and first story were constructed sequentially. The SMRF code design reference frame model was constructed as per the design details and using concrete compressive strength of 21 MPa (3000 psi). The deficient frame model was constructed with same details of Model-1 but in the joint panels regions no confinement transverse ties were provided and the frame was built with low strength concrete of 14 MPa (2000 psi). After the construction, the as built models were water cured for 14 days and then left for 28 days for concrete maturity. After that the test models were shifted to the testing facility and mounted on the shaking simulator for testing purpose. For seismic mass simulation [10, 28], a total mass of 1200 kg (1.2 tone) was provided on each story level in the from steel plates. Each steel plate was firmly fixed to the slab using 06 numbers, 6.35 mm (0.25 in) diameter steel bolts.

3. DYNAMIC SHAKE TABLE TESTS

3.1. TEST SETUP AND INSTRUMENTATION DETAILS

The 1/3rd scaled two-story frame model testing setup is shown in Fig. 2. Both frame models were tested using the uni-directional seismic simulator of Earthquake Engineering Centre (EEC) Peshawar. After shifting of the test models to the testing laboratory, a 30-tone overhead crane was used to mount the test frame on the top of the shake table. The strong base beam of the test models was firmly fixed to the top of the shake table using 15 number and 6.35 mm (0.25 in) diameter steel screws to provide a fixed support in the in-plane direction as well as to stop the toppling of the frame in the out of plane direction (perpendicular to direction of plane of the test frame). For obtaining model response parameters (such as displacement and acceleration histories) during the shake table test, the test frames were instrumented with six accelerometers and three linear variable displacement transducers (LVDT). Three accelerometers on the front side, and three accelerometers on the back side were installed at the base, first story and second/roof story level mid position. Similarly, three LVDTs were mounted on a steel frame installed in front of the test frame (in-plane direction) at base, first story and roof story level mid position. The thread wire from each LVDT was mounted on the base, first story and roof story mid position level, in order to measure the displacement of each story level.

3.2. Input Excitation and Testing Protocols

For the shake table input shaking excitations a number of accelerogram were first examine, that can be operated with in the displacement, velocity and acceleration limitations of the shake table. After a number of analysis and trial shaking of equivalent weights on the shake table with different natural acceleration records, the 1994 Northridge Earthquake natural acceleration record as shown in Fig.3, was selected for the dynamic shaking of test frames.

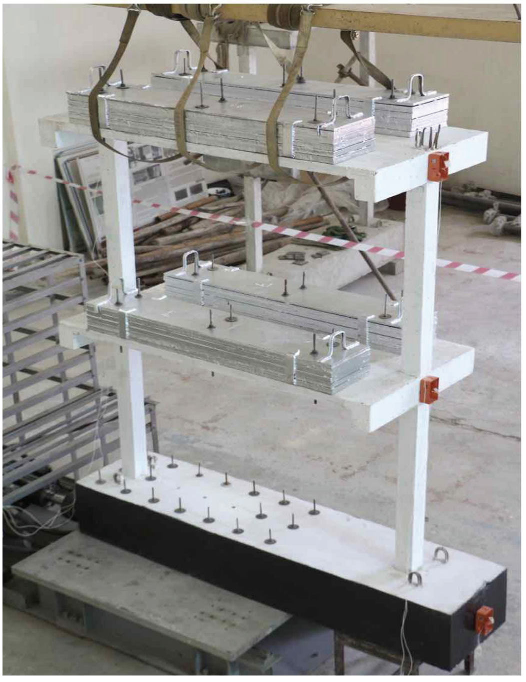


Fig. 2. Test models setup on shake table

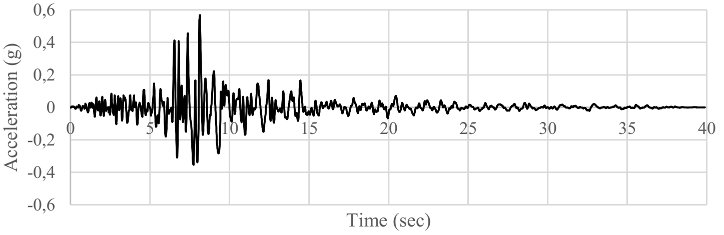


Fig. 3. Input shaking record of 1994 Northridge Earthquake

The acceleration record has been obtained from the PEER strong motion data base (Recording station: 090 CDMG STATION 24278, Frequency range: 0.12-23.0 Hz) and has a peak ground acceleration (PGA) of 0.57g. Time scaling of the input acceleration record was performed as per the conversion factors reported in Table 1. The testing sequence and applied loading protocols in terms of scaled excitations, for the code design Model-1 and deficient Model-2, are reported in Table 2 and Table 3. Both reduced frames were excited with linearly scaled and increasing shakings from 5% to 130% of the input acceleration record PGA, depending on the resisting test frames capacity. The idea of using low intensity shaking and, increasing the shaking to high intensity was to force the frame models from elastic to full collapse condition. For each scaled excitation, the response parameters (story displacements and accelerations) and observed damage behavior have been recorded through respective instrumentation and visual observations (photographs and video recording). For displacement and acceleration time histories a data logger system was used to records the response values.

Table 2 Input loading protocols for code design Model-1

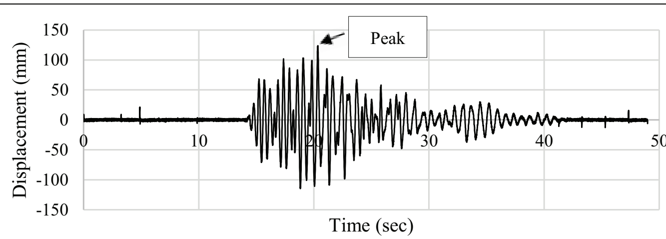
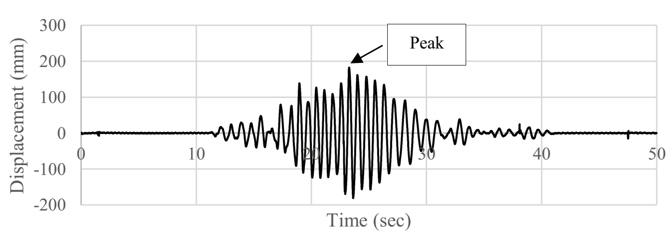
Input acceleration record	Run excitation (%)	PGA (g) Experimental recorded
Natural acceleration record of Northridge Earthquake	Self-Check	0.60
	5%	0.033
	10%	0.06
	20%	0.12
	30%	0.16
	40%	0.19
	50%	0.25
	60%	0.31
	70%	0.36
	80%	0.41
	90%	0.49
	100%	0.62
	Self-Check 130%	0.62
	130% (Final run)	1.06

Table 3 Input loading protocols for deficient Model-2

Input acceleration record	Run excitation (%)	PGA (g) Experimental recorded
Natural acceleration record of Northridge Earthquake	Self-Check	0.015
	5%	0.52
	10%	0.25
	20%	0.31
	30%	0.35
	40% (Final run)	0.73

The data logger records the accelerometers and LVDTs data in the form current (voltage mv/in or mv/g) values. For each installed accelerometer and LVDTs, instrument coefficient factors have been used to obtain the displacement and acceleration values in the units of length (mm or in) and acceleration (g) respectively. The raw data was further process for base line correction and data filtering in order remove any noise observed during experimental testing. For the base line correction and noise data filtering the data processing software SeismoSignal (2018) was utilized. Table 4 and Table 5 reports the deficient weak beam column joint Model-2 experimentally recorded top/roof story displacement and acceleration histories for the 5% run, 30% run and final 40% run excitations, with corresponding peak values.

Table 4 Experimentally observed roof displacement responses for deficient Model-2

Run	Story level	Displacement history (mm)
5% Peak Value 123.68mm	Roof story level	
30% Peak Value 182.36mm	Roof story level	

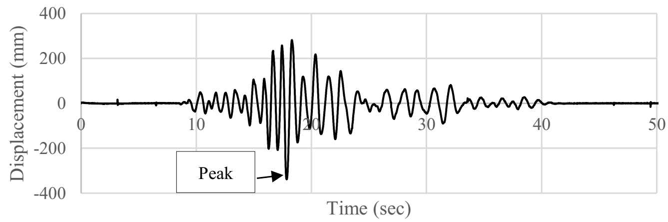
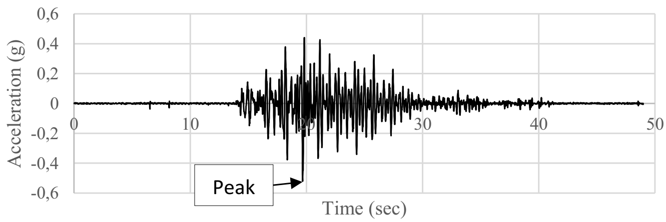
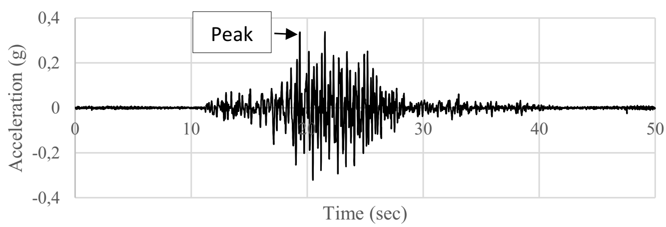
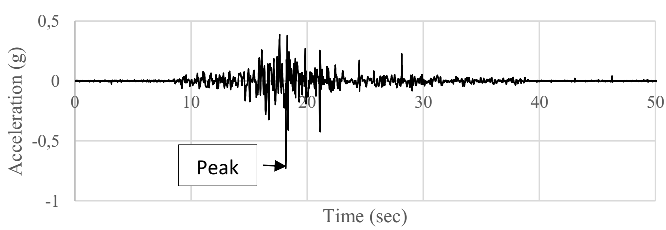
40% Peak Value 383.19mm	Roof story level	

Table 5 Experimentally observed roof acceleration responses for deficient Model-2

Run	Story level	Acceleration history (g)
5% Peak Value 0.52g	Roof story level	
30% Peak Value 0.35g	Roof story level	
40% Peak Value 0.73g	Roof story level	

3.3. Observe Damage Response

Fig. 4 shows Model-1 observe damage response for the selected significant runs. Model-1 was initially excited with shake stable automatic self-check excitation, which has displaced the test frame to a drift of about 0.87%. The peak ground acclamation value during this excitation was observed to be 0.60g. After the self-check run the test specimen was visually inspected for any damages and cracks. It was observed that horizontal and vertical flexural cracks were observed at ends of beams members at ground story, which was due the yielding of reinforcement and bar slippage. On the ground story at the beam column interface, few minor vertical cracks were also observed. Slight flexural cracks were also observed at column bases of ground and first story. After the self-check excitation, the test models were subjected to multiple scaled excitations ranging from 5% to 100% of the maximum input record. During each run the visual inspection of test model were conducted for any aggravation of existing and creation of new cracking response. Up to 100% excitation run, the already existing damages were slightly aggravated. When the test model was subject to 130% of the input peak ground acceleration record, the test model was found to be at a drift of about 5.26%. The PGA value record during this run was 1.06g. During this run the existing cracks of the specimen were significantly aggravated. It was observed that the ground story column bases and top ends were significantly damaged with concrete spalling and crushing. The first story column base was also observed with moderate cracking and spalling.

Fig. 4. Code design Model-1 damage response

Severe diagonal cracking was also observed on the joint panel regions and transverse beam of the ground story. Some minor cracking in the diagonal direction were also observed at the joint panel of the first story. The test model was found to be in the near incipient collapse stage and testing was stop after this run.

Fig. 5 shows the weak beam-column joint frame Model-2 observed damage response for the selected significant runs. Initially this model was excited with a self-check excitation which has displaced the test model to a drift of about 0.2% and the PGA value observed to be 0.05g. No visible cracks were observed during the self-check excitation. After this the test models were excited with 5% run during which the test model was laterally displaced to drift of about 1.75% and the observed PGA was about 0.52g. During this run the deficient model was observed with slight flexural cracks at the column bases of the ground story. Significant flexural cracks were also observed at the bases of the first story column. Diagonal cracks were also observed at joint panel regions of the first story and slight bat like cracks were observed at the panel region of the ground story. During this run, minor vertical and significant horizontal cracks were also observed at the beam member ends, both at the ground and first story. After this test model was excited with 10% and 20% shaking, during which the existing cracks were observed with minor aggravations.

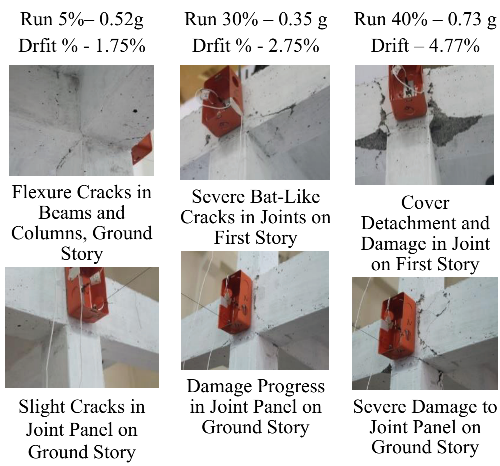
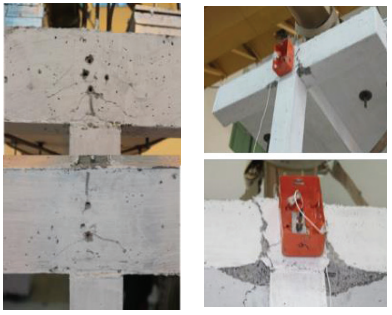


Fig. 5. Deficient weak beam-column joint Model-2 damage response

When the test model was exited with the 30% input acceleration record, the test model was laterally displaced to a drift of about 2.75% and the observed PGA was 0.35g. During this run the already existing cracking pattern were significantly aggravated with flexural cracks were further widens at the end of the beam and column members. The cracks on the ground and first story were further widens and spread over the panel regions. Significant detachment of the cover concrete was also observed at the ground story panel and transverse beams region. When the test model was further excited to 40% of the input excitation, the test specimen was laterally displaced to a drift of about 4.77% and the observe PGA was about 0.73g. During this run the test models experienced severe damages at beams and columns end with cover spalling and crushing of concrete. Joint panels on the first story were severely damaged with cover concrete wedge detachment and spalling. The model after this run was found to be in the incipient collapse state. Fig. 6 shows the test model joint panel observed damage responses at final runs i.e. at the incipient collapse state.

Model-1 at final 130% Run | Model-2 at final 40% Run



First story front and back side joint panels

Ground story front and back side joint panels

Fig. 6. Joint panel damage response at incipient collapse state

4. Seismic Evolution of Considered Frames Structures

4.1. Seismic Response Curves

Seismic response curves which basically shows the relationship between peak displacement (or drift) demands and peak ground acceleration PGA, have been obtained for the reference prototype Model-1 and deficient Model-2 using the experimental observed data. For this purpose, first, the shake table test data have been transferred using the scaling factors as shown in Table 1. The peak values of top/first story displacement have been obtained for each run and, correlated with the corresponding peak PGA in order to develop the seismic response curves as shown in Fig. 7. As observed from Fig. 7, the reference prototype frame structure was able to withstand higher intensity level shaking both in the elastic and at the final near collapse state as compared to the deficient frame structure. The code design frame was in near incipient collapsed condition at intensity shaking of about 130% i.e. at an input PGA level of about 1.06g. Whereas, the substandard weak beam-column frame with no shear reinforcement in the panel region as well built in low concrete strength, has resisted about 40% i.e. at an input PGA level of about 0.73g of input acceleration record.

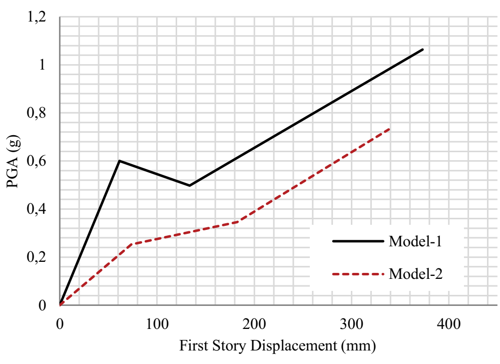


Fig. 7. Prototype frame seismic response curves

4.2. Force-Deformation Relationships

For the prototype frame structures, force-displacement curves have been developed by corelating the peak values of displacement and total base shear force for each run. In order to calculate the maximum values of base shear forces, the peak value of acceleration for each run have been obtained from the experimental testing. The peak values of the acceleration records for each story level were multiplied with the corresponding mass of that story. For the total mass/weight on each story; the self-weight of beam, column, slab portion and the extra seismic mass used during the dynamic testing was employed. The total base shear force has been computed using Eq. (1) to Eq. (3):

$$V = F_1 + F_2 \tag{1}$$

$$F_1 = M_1 \, x \, a_1 \tag{2}$$

$$F_2 = M_2 \, x \, a_2 \tag{3}$$

Where V is the total base shear force at base of the frame, F_1 and F_2 are maximum story shear at the first and roof story level, M_2 and M_2 are masses at the first and roof story level, a_1 and a_2 are the recorded peak values of acceleration at the first and roof story level. The story shear forces for both story levels are added to calculate the total base shear force for the prototype structure. The total base shear force for each run is further corelated with the corresponding peak values of displacement to develop the force-deformation relations for the code design Model-1 and deficient Model-2 prototype structure as shown in Fig. 8. In order to have a quantitative comparison between the reference code design and weak beam-column joint frames and also, to compute different seismic response parameters, the nonlinear force-displacement correlations were further converted to simple bilinear relationships using the energy balance technique. For this reason, the area under the nonlinear force-displacement were normalized to an equivalent bilinear force-displacement with well define yield displacement, ultimate displacement and yield force. Fig. 9 and Fig. 10 shows the developed idealized bilinear force-displacement correlations for the considered Model-1 and Model-2. For the ultimate displacement of the bilinear force-displacement relationships, the maximum displacement demands obtained from the shake table tests (i.e. for Model-1 375 mm and for Model-2 280 mm) have used. As shown in Fig. 11, the yield stiffness, strength and also the peak displacement capacity or ductility generally decreases for the case of deficient weak beam-column Model-2. In case of deficient weak beam-column joint frame Model-2, the yield displacement increased by 40%, ultimate displacement decreased by 9%, the yielding force decreased by 31.5% and the yield stiffness decreased by about 50% as compared to the code design Model-1.

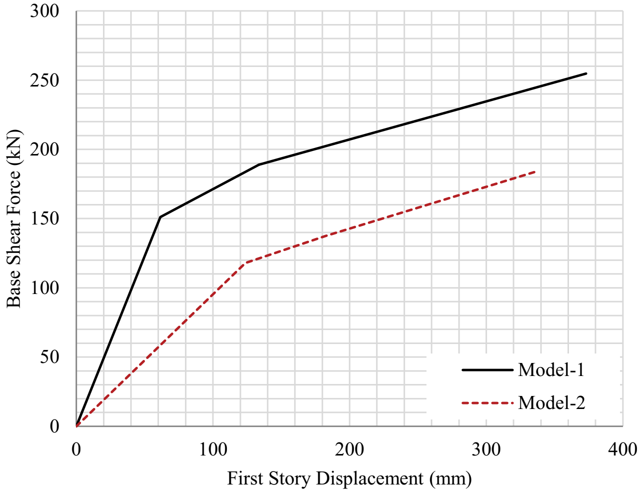


Fig. 8. Prototype frame force-deformation capacity curves

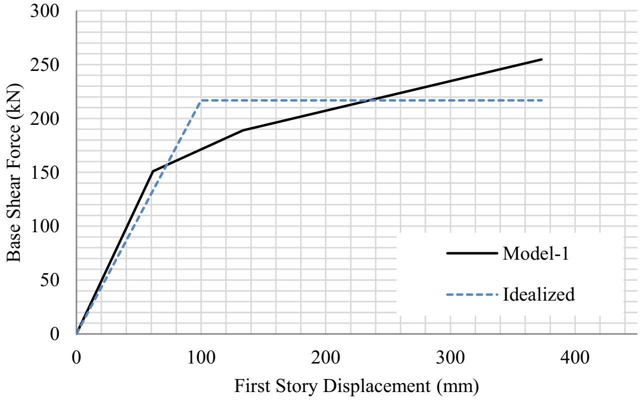


Fig. 9. Model-1 Idealization of force-deformation capacity curve

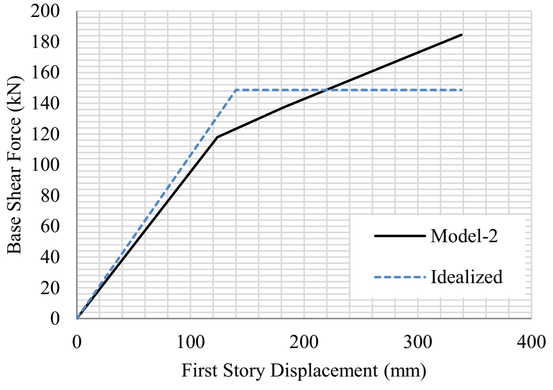


Fig. 10. Model-2 Idealization of force-deformation capacity curve

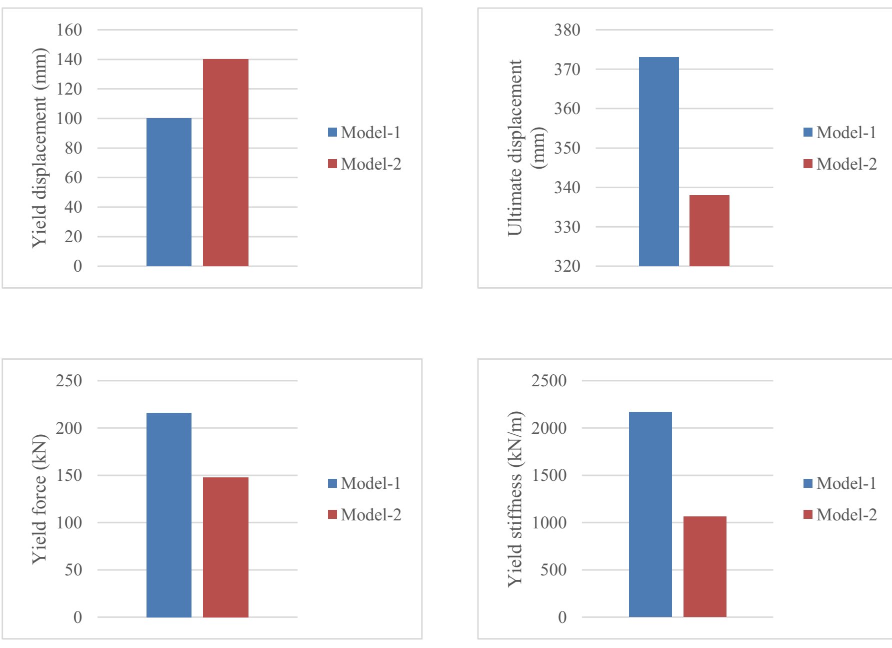


Fig. 11. Response parameter from idealized force-displacement relationships

4.3. Seismic Response Modification Factors

The seismic evolution of the considered frame models was also investigated in term of the structural seismic response modification factors. These factors are used in the building codes to allow the use of linear elastic seismic analysis of structural systems (lateral force procedure and response spectrum methods), and are used to account for the structural nonlinear response and energy dissipation behavior [30-35]. Generally, the response modification factors can be obtained from the inelastic force-deformation capacity curves as shown in Fig. 12 and can be computed mathematically using Eq. (4) [31-32].

$$R = \frac{V_e}{V_d} = \frac{V_e}{V_y}\, x\, \frac{V_y}{V_d} = R_\mu\, x\, R_S \tag{4}$$

Where, V_e is force required for the structural system to remain within the elastic limits; V_y is the force values at the yield strength from the idealize bi-linear curves; V_d is the force requiring for the design level demand; R_μ is the ductility factor and R_s is the overstrength factor. R_s can be obtained using force-deformation capacity curves from the ratio of the idealized yield strength V_y to the design base shear force V_d . In the current study the ductility factor R_μ is couple with the structural ductility and is obtained from the analytical procedure [36] and as given in Eq. (5) to Eq. (7).

$$R_\mu = 1.0, \qquad for\ short\ period\ strucutrs, T < 0.2\ sec \tag{5}$$

$$R_\mu = \sqrt{2\mu - 1} \qquad for\ intermediate\ \ period\ strucutrs, 0.2\ \mathrm{sec} < T < 0.5\ sec \tag{6}$$

$$R_\mu = \mu \qquad for\ long\ period\ strucutrs, T > 0.5\ sec \tag{7}$$

For short period structures having fundamental time period T less than 0.20 sec the value of R_μ can be taken as 1, for intermediate period structures having the fundamental time period in the range of 0.2 to 0.5 sec, the value of R_μ can be computed as √((2μ -1)) and similarly, for the longer period structures with a fundamental time period of more than 0.5 sec, the value of R_μ can be equal the ductility μ of the system [36].

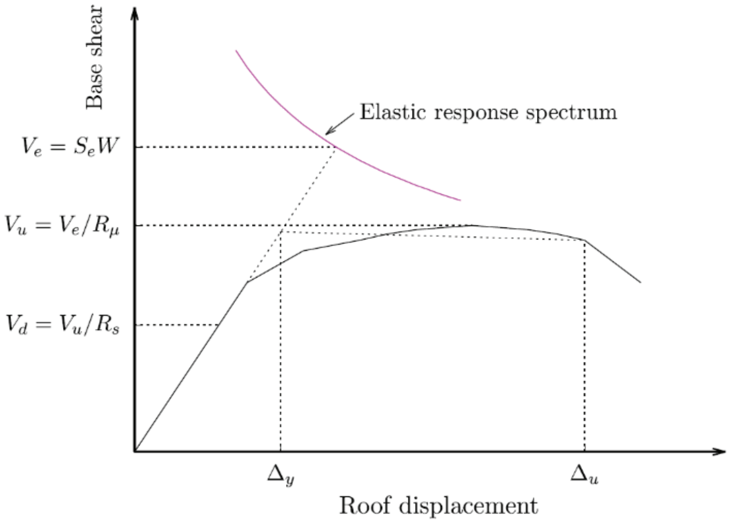


Fig. 12. Response modification factor parameters from capacity curve [31]

The response modification factor for the considered RC prototype code design and deficient frame structures were obtained by multiplying the ductility dependent factor Rμ with the over strength reduction factor. Fig. 13 report the seismic parameters and response modification factor for considered prototype frame structures. It can be seen that the response modification factor for the code design frame Model-1 is approximated as 7.5 which about 10% less than the UBC/BCP-SP 2007 code specified value of 8.5 for reinforced concrete SMRF structures. The response modification factor for the deficient frame structure with no provision of confinement ties in the joint panel regions and built with low strength concrete has been obtained approximately as 3.50, which about 53.5% less than Model-1 value and about 59% less than code specified value of 8.5. It has been observed from the current studies that due to prevailing defect of having weak beam-column connections the seismic parameters of the prototype frame structures in the form of yield stiffness reduces by 51.5%, the yield deformation reduces by 15.5%, the ultimate deformation reduces by 10.5%, the overstrength factor R_s reduces by 31% , ductility factor R_μ reduces by 28.5% and the seismic response modification factor reduces by 53.5% as compared to code design reference frame stricture.

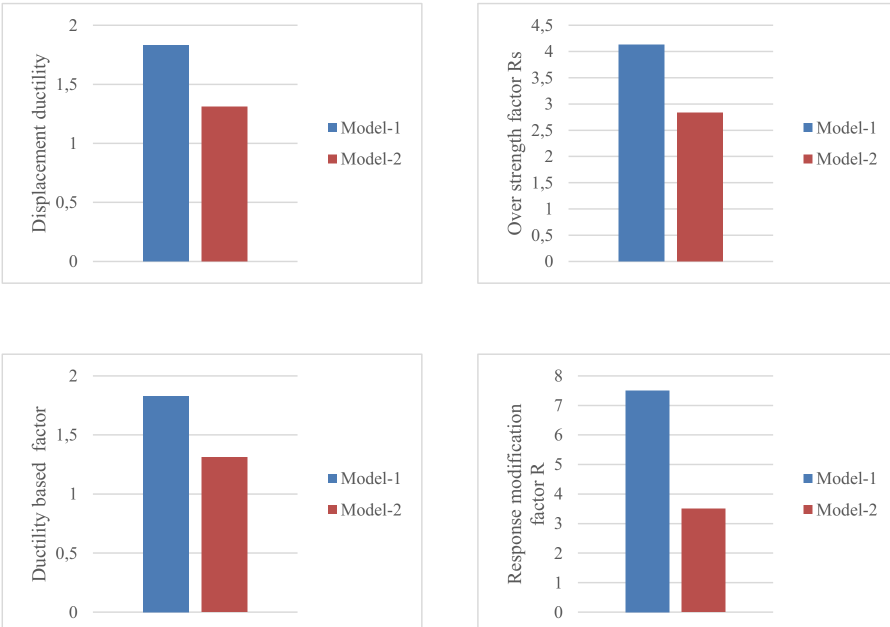


Fig. 13. Structural seismic response parameters

4. Conclusion

Due to the use of old seismic design building codes and the improper construction executions of the recent SMRF seismic detailing, numerous defects are currently existing in the RC building stock more specifically in the developing countries. The construction or design defects of having a weak beam-column connection (i.e. the joint panel not provided with the confining shear ties and constructed with low strength concrete of about 33% less the code specified value for concrete compressive strength) is very common and can cause partial or full collapse of RC buildings. In order to evaluate the seismic performance of current reinforced concrete special moment resisting frame SMFR building with the construction deficiency of having weak beam-column connections, dynamic shaking tests were performed on representative frame models.

The following conclusion have been summarized from the current experimental studies and seismic evaluation of the considered RC frame structures:

-The damage evaluation of code design frame model with multiple scaled excitations shows that this model upon subjected with high shaking have developed flexural cracking and plastic hinge mechanism at the base of columns and as well as, at the end of the beam members. The code design frame structure was able to resist about 130% of the input excitation before found in the near incipient collapse stage.

-The damage evolution of the deficient weak beam-column frame structure shows that such framing system exhibit a mixed damage response mostly concentrated at the column and beam ends and as well as with severe damage in the joint panel region with concrete crushing and detachments, due to non-provision of the confining ties and reduced concrete strength. Due to low strength concrete the longitudinal reinforcement have shown the bar slip mechanism and the reinforcement pullout. This also cause the reduction in stiffness, strength, deformation capacities and the seismic response modification factor for such type of building typology which can cause joint hinging mechanism at local level and soft story response at global level. The deficient frame structure was only able to resist 40% of the input shaking before near the collapse stage. The performance of such class of frame structure will be primarily depends on the joint damage state and the corresponding repair cost required after a design level earthquake shaking.

-It has been observed from the current studies that due to prevailing defect of having weak beam-column connections the seismic parameters of the prototype frame structures in the form of yield stiffness reduces by 51.5%, the yield deformation reduces by 15.5%, the ultimate deformation reduces by 10.5%, the overstrength factor R_s reduces by 31% , ductility factor R_μ reduces by 28.5% and the seismic response modification factor reduces by 53.5% as compared to code design reference frame stricture.

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LIST OF FIGURES AND TABLES:

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