

SEISMIC DAMAGE ASSESSMENT OF DEFICIENT REINFORCED CONCRETE FRAME STRUCTURES

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Abstract

In order to assess the seismic damage response of reinforced concrete deficient, weak beam-column connection frame structures, this study presents experimental shake table testing on representative 1/3rd reduced size and double story frame models. Two test models were considered for experimental testing, including a reference code design specimen and another frame with a similar characteristic, but was not provided with any shear reinforcement in the critical joint connection region and, was constructed with concrete having a compressive strength of 33 % less than the code specified value. 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 sustained plastic hinging at column bases and beam ends, with longitudinal reinforcement bar-slip and severe damageability of the joint panels upon subjected to multiple dynamic excitations. The deficient frame model was only able to resist 40 % of the maximum acceleration record as compared to the code design frame which was able to resist about 130 %.

Keywords:

Special moment resisting frame;
Building frame;
Beam-column joints;
Shake table test;
Progressive collapse.

1 Introduction

Due to the use of old building codes and also, the improper use and execution of the current special moment resisting frame SMRF seismic code detailing, have caused the majority of these RC building having different seismic defects or deficiencies. Different field surveys were 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 the low quality of building materials (i.e. low strength concrete constituents and reinforcement material) along with different non-seismic provisions. The most common construction defect is the weak beam-column connection deficiency i.e. not having any confining shear reinforcement in the joint panel area and build with low strength concrete as required per the code specifications, is very common [2,3]. As also seen from past earthquakes damage observation of RC building typology with 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 [3]. Furthermore, the majority of such RC buildings typology currently exists in areas which has very 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 show the importance of the seismic evolution of RC frame system with design or construction defects within the context of seismic feasibility assessment of such building typology.

From the last few decades, researchers have conducted experimental studies through quasi-static tests on weak beam-column connections and shake table tests on deficient frame structures, to

access their seismic performance. In most of these experimental studies, either full scaled and/or reduced scaled beam-column connections and frame structures have been employed. In these experimental investigations, different types of substandard and non-seismic parameters and diverse structural configuration have been considered, in order to access the seismic performance and damage mechanism of deficient RC structures. Quintana et al. [10] tested 1:2.5 reduced scaled three-story deficient frame structure on a shaking simulator, in order to access the damage mechanism and to provide bench mark test data for the retrofitting of the old type, non-ductile and non-seismic designed buildings. The test model was built with plain rebars, without joint transverse 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 experience lap splice failure at the top story joints because of the use of plain rebars. At high PGA intensity most of the damages were concentrated at lower story column bases and extensive damages were observed at beam-column connections, shows the high vulnerability of these deficient RC structures. Stavridis et al. [11] conducted shake table tests on 2:3 scaled three story and two bay RC frame structures, 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 damages mechanism with low, moderate and high-level shaking intensities. It was observed during the testing program that at low level intensity, minor cracks appeared 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 story and two bay RC frame 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 that effects the behavior of these deficient RC frame structures during an earthquake loading such as; axial load demand on the column members and shear failure pattern of weak beam-column connections, have been evaluated. It was observed from the experimental study that the collapse of deficient RC frame structures would be likely be the 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 a shake table test on a 3D, 3 story RC frame model 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 from test results that under increasing dynamic shaking, the model damage behavior is mostly concentrated at the base of the lower story columns base and beam-column panels. Most of the experimental studies available in the literature were reported on the seismic assessment of non-ductile gravity type RC building typologies [14-16]. However, seismic assessment studies on the current special moment resisting frames SMRF structures (beam and column members are as per the seismic code requirement), 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 have been conducted on representative frames structures. The experimental study has been performed on two, two story RC frame specimens with 1/3rd reduced size scaling. Model 1 (reference model) was a code design specimen, whereas, Model 2 was similar to Model 1, but was provided with no ties in the beam-column panel zone along with concrete strength less than the design specified strength 33 % less, to consider the effects of the most commonly available defects in the existing building stock [1,3]. The reduced size frame specimens were excited with multiple excitations in the un-directional shaking, using 1994 Northridge Earthquake multiple scaled input records. The test specimen observed damage mechanism, response parameter (such as acceleration and displacement histories) and force-deformation relationship have been developed and compared. The objectives of the current experimental program are 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.

2 Experimental program

2.1 Test specimens description

In order to investigate the seismic performance of non-ductile and code non-compliant weak beam-column connection RC frames through shake table testing, the current study considers a low rise, regular two-story prototype building [17], as shown in Fig. 1. The considered prototype building consists of two bays by one bay configuration, with all bays length of 5487 mm (18 feet) and typical story height of 3658 mm (12 feet). The analysis and design of the prototype RC building were performed as per the UBC97/BCP-SP07 [18,19] building codes and the seismic detailing of the framing system was based on the provisions of the ACI-318-14 [20] for special moment resisting frame SMRF systems. For seismic analysis, equivalent static forced based procedure, with the stiff ground condition (soil type B) and design peak ground acceleration PGA of 0.40 g (high seismicity level) was used. Typical code-based load (dead and live load) and load combinations were considered during the design process. The modeling, analysis and design of the prototype building were performed using CSI ETABS analysis and design software. For concrete material, the compressive strength of 21 MPa (3000 psi), whereas, for the steel rebar yield strength of 414 MPa (60000 psi) was considered. Fig. 1 and Table 1 shows the geometry and design details of the structural members of the considered building.

2.2 Preparation of one-third reduced models

Due to the shake table simulator's size limitations and motion restrictions in terms of acceleration and displacement response, it was decided to select only the critical interior frame from the prototype building. The selected interior frame was then reduced by a factor of 3 in order to test the frame models within the range of shaking limitations. For the simulation and conversion of 1/3rd reduced model, a simple linear scaling model was adopted [10,17]. All the linear geometrical dimensions of the prototype building interior frame and structural members were divided by a factor of 3, as reported in Table 1. Similarly, for reinforcing rebars, 1/3rd reduced diameter rebars were used in the test model construction. The simple model idealization is generally adopted based on its simplicity in simulating the prototype characteristics and associated low construction cost of the test specimens. Similarly, for concrete material preparation, 1/3rd reduced coarse aggregate size 9.53 mm (0.375 in) was used in the test model preparation. Fig. 2 shows the test model construction phases. Initially, a strong reinforced concrete pad (strong beam) with a size of 558.8 mm (22 in) depth, 406.4 mm (16 in) width and 2438.4 mm (8 feet) long was constructed, which will act as strong fixed foundation support for the frame model. The strong concrete pad was provided with a 25.4 mm (1 in) size hole as regular interval, which are going to use to anchor the test model on the shake table top, using steel screws. During the construction of the pad, the longitudinal reinforcement of the columns was placed at their required position. After the construction of concrete pad and water curing for 7 days, the first story was constructed using special fabricated steel formwork for the column, beam and slab members. The concrete laying for the first story column, beam and slab was performed on the same day. Similarly, after the construction of the first story and curing for 14 days, the second/roof story was constructed in the same manner. The test models were then water cured for 14 days and were provided with moist bags for a period of 28 days. After 28 days, the test models were shifted from the casting yard to the testing laboratory, after which the test specimen was mounted on the shaking simulator for testing.

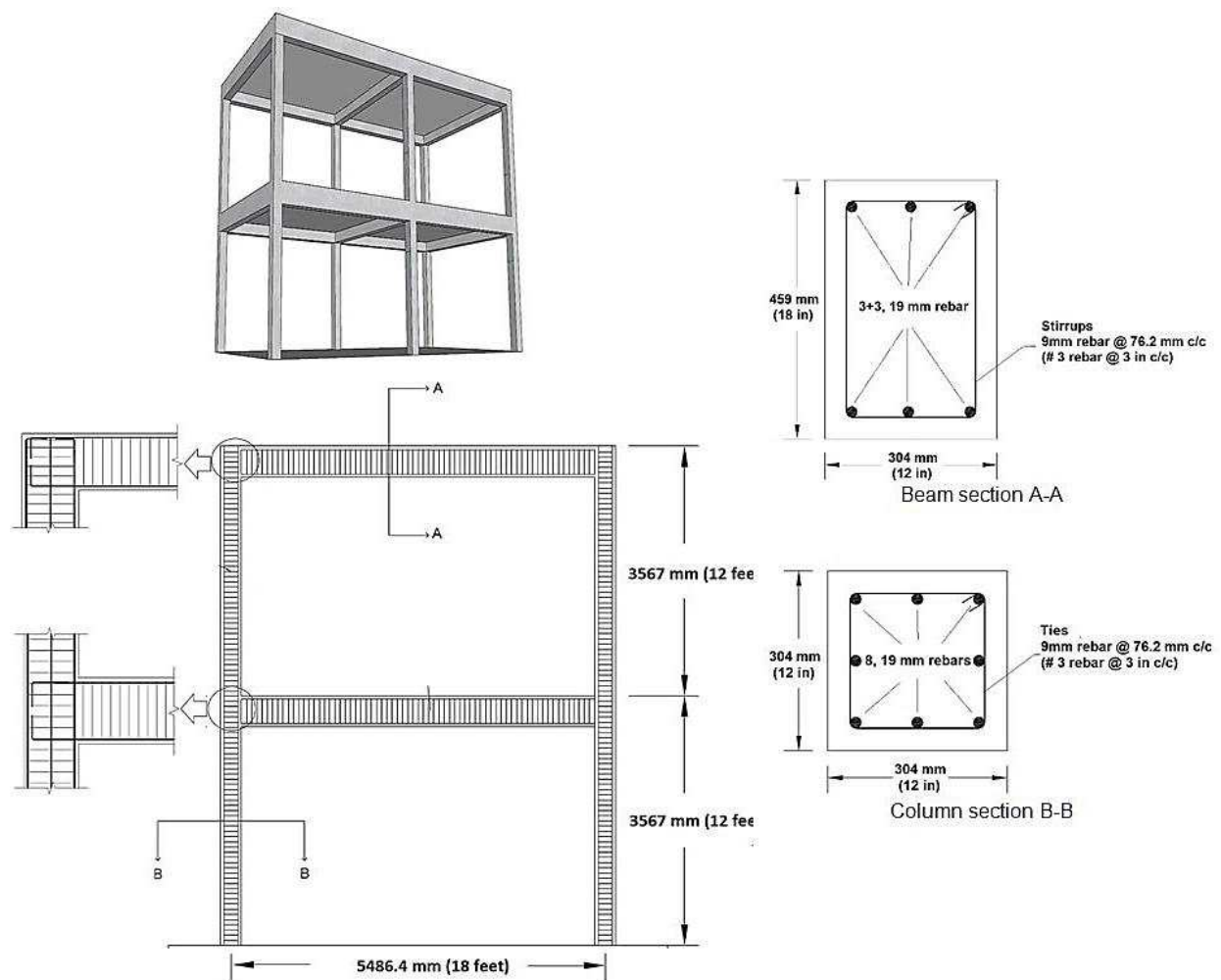


Fig. 1: Geometric and design layout of prototype RC frame, Model 1.

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

Member geometry / material properties	Prototype geometry	1/3rd scaled model geometry
Bay length	5487 mm (18 feet)	1828.8 mm (6 feet)
Story height	3658 mm (12 feet)	1219.2 mm (4 feet)
Beams	304 mm x 459 mm (12 in x 18 in)	102 mm x 153 mm (4 in x 6 in)
Columns	304 mm x 304 mm (12 in x 12 in)	102 mm x 102 mm (4 in x 4 in)
Slab	153 mm (6 in)	51 mm (2 in)
Reinforcement rebar	414 MPa (60000 psi)	414 MPa (60000 psi)
	19 mm (#6 Rebar)	6.33 mm (#2 Rebar)
	10 mm (#3 Rebar)	3.33 mm (#1 Rebar)
Concrete	21 MPa (3000 psi)	21 MPa (3000 psi)
		1:1.80:1.60 with w/c of 0.48
	14 MPa (2000 psi)	14 MPa (2000 psi)
		1:2.5:2.87 with w/c of 0.80
Coarse Aggregate	28.58 mm (1.125 in)	9.53 mm (0.375 in)

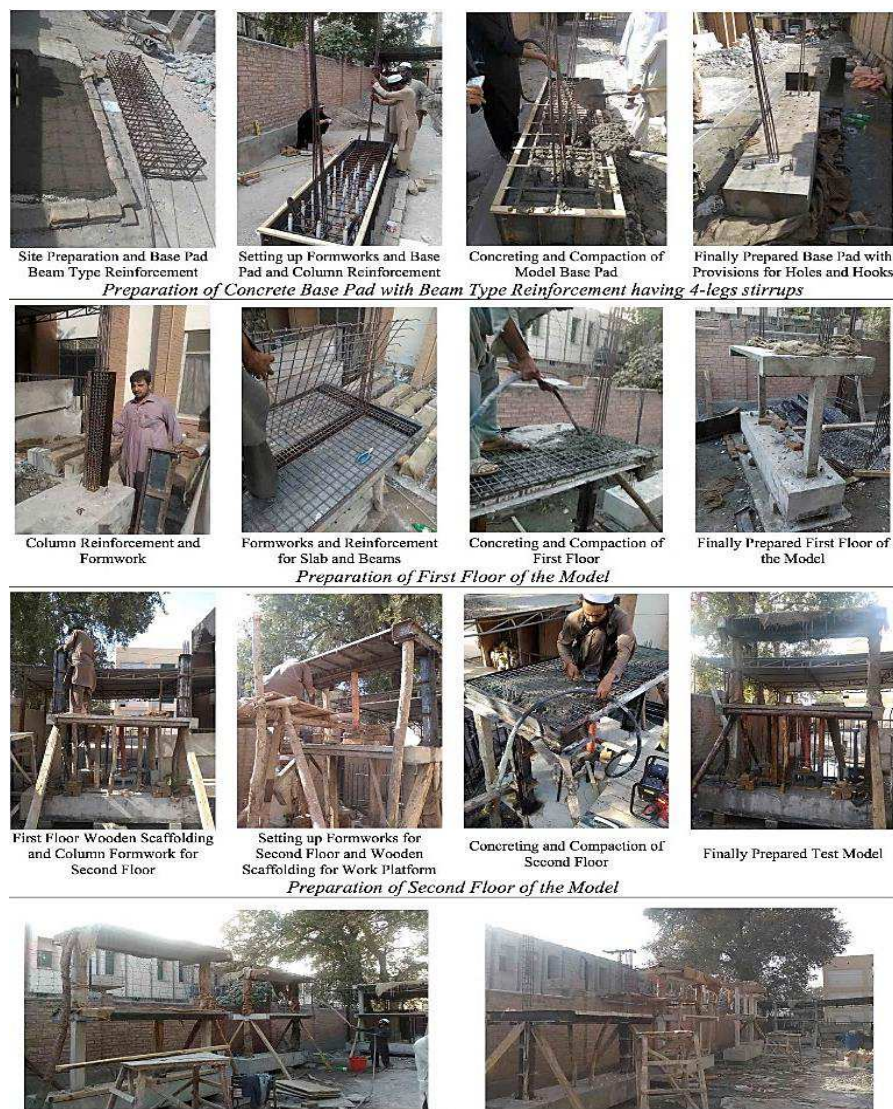


Fig. 2: 1/3rd scaled specimen construction.

It is also worth mentioning that, due to reduced scaling, the test model will be subjected by reduced force due to mass simulation. In order to simulate the seismic mass during the dynamic testing, extra seismic mass 1200 kg in the form of steel weights were provided on each story, as shown in Fig. 3. Table 1 also reports the concrete mix proportion based on the ACI mix proportion procedure, for the code specified compressive strength of 21 MPa (3000 psi) and reduced concrete strength of 14 MPa (2000 psi) using reduced size 9.53 mm (0.375 in) coarse aggregate.



Fig. 3: Extra weights in the form steel plates for seismic mass modeling.

2.3 Test model setup and instrumentation

Fig. 4 shows the experimental specimen testing setup, whereas, Table 2 and Fig. 5 shows the external instrumentation layout detail. In order to obtain acceleration responses, the test model was installed with one accelerometer on the base level and two pairs of accelerometers on the first and roof story level, one on the front and second on the back side. Similarly, to measure the displacement responses, three linear variable displacement transducer LVDT were installed on a steel reference frame, and were in line with the base, first and roof story level.

2.4 Shake table testing and input loading protocols

For shaking excitations, the 1994 Northridge Earthquake record shown in Fig. 6, was selected after careful analysis of a number of natural acceleration records that can excite the test specimen within shaking simulator limitations. The selected record has a peak acceleration of 0.57 g, peak velocity of 518 mm/sec and peak displacement of 90 mm. The code compliant and deficient RC frame test models were excited with an increasing scale value of peak ground acceleration i.e. from 5 % to 130 %, in order to force the specimen from elastic stage to yield and then, to full plastic collapse stage. Both test specimens were initially excited by the seismic simulator self-check run. After this run, the test frames were shaken with multiple incremental excitations. The test frames were inspected after each run for damage and crack formations, which were recorded through snapshots, video recording, and along with crack description for the structural members. The testing sequence was followed until the maximum capacity of the test models reached, after which the test was stopped. For each test run, the floor acceleration and displacement response were obtained using the installed accelerometers and displacement transducers.

2.5 Recorded data processing

For both test models, the acceleration and displacement raw data obtained from the installed instruments were further processed for instrument coefficients corrections, base line correction and data filtering in order to remove any noise. For the data analysis and correction purpose, the Seismosignal data correction package has been utilized with a linear type baseline process and Butterworth filter. The limits for the filter frequency were between 0.10 to 25 Hz. Once all the data has been processed, the acceleration and displacement response at the base, first and roof story level were obtained in the prototype domain using the simple model idealization scaling factors [10,17]. Tables 3 and 4 report the first and roof story level displacement and acceleration time histories, for both the code compliant and deficient prototype structure, only for the selected significant runs. In order to obtain the first and roof story relative displacement with reference to the base/pad level, the latter displacement values were subtracted from respective story displacement values. Similarly, in order to calculate the relative first and roof story drifts, the maximum displacement value of each run is normalized i.e. divided by the height of the story. The story level inertial forces were calculated by multiplying the maximum acceleration value of each run by the corresponding applied weights (which include self-weight and extra seismic mass). The total base shear at the base/pad level was then obtained by summing the first and roof story inertial forces.



Fig. 4: 1/3rd reduced RC frame model setup.

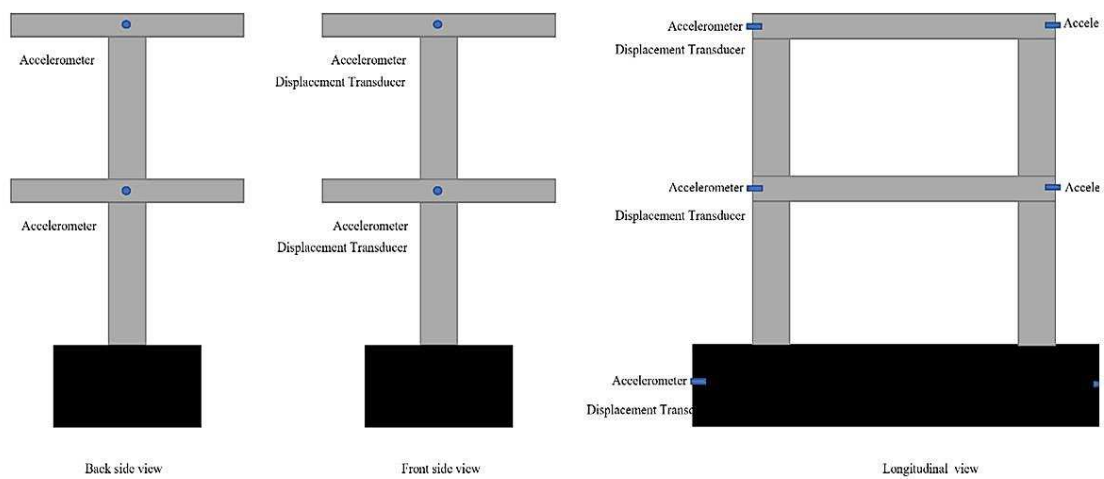


Fig. 5: Test specimen instrumentation detail.

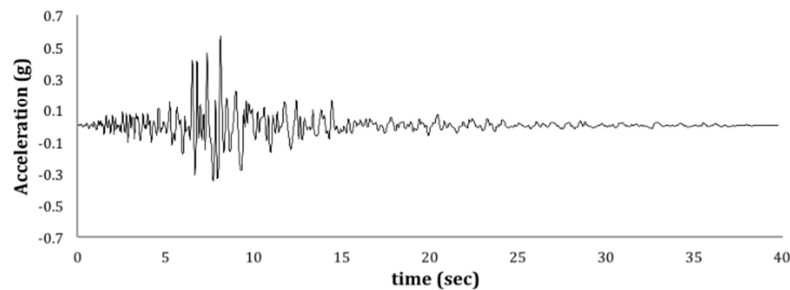


Fig. 6: 1994 Northridge earthquake acceleration record.

Table 2: Instrumentation position and characteristics.

Channel	Placement	Face	Peak limit	Response	Correction parameter
A1	Base level	Front	± 10 g	Acceleration [mv/g]	492.20
A2	First Floor	Front		Acceleration [mv/g]	501.10
A3	Second Floor	Front		Acceleration [mv/g]	510.10
A4	Pad level	Back		Acceleration [mv/g]	508.90
A5	First Floor	Back		Acceleration [mv/g]	490.10
A6	Second Floor	Back		Acceleration [mv/g]	502.00
D7	Pad level	Front	24 inches (610 mm)	Displacement [mv/inch]	1000.00
D8	First Floor	Front		Displacement [mv/inch]	1000.30
D9	Second Floor	Front		Displacement [mv/inch]	1000.20

Table 3: Model 1 experimental observed displacement and acceleration histories.

Run	Story level	Displacement history [mm]	Acceleration history [g]
1 st Run/ Self-check run	First story level		
	Roof story level		

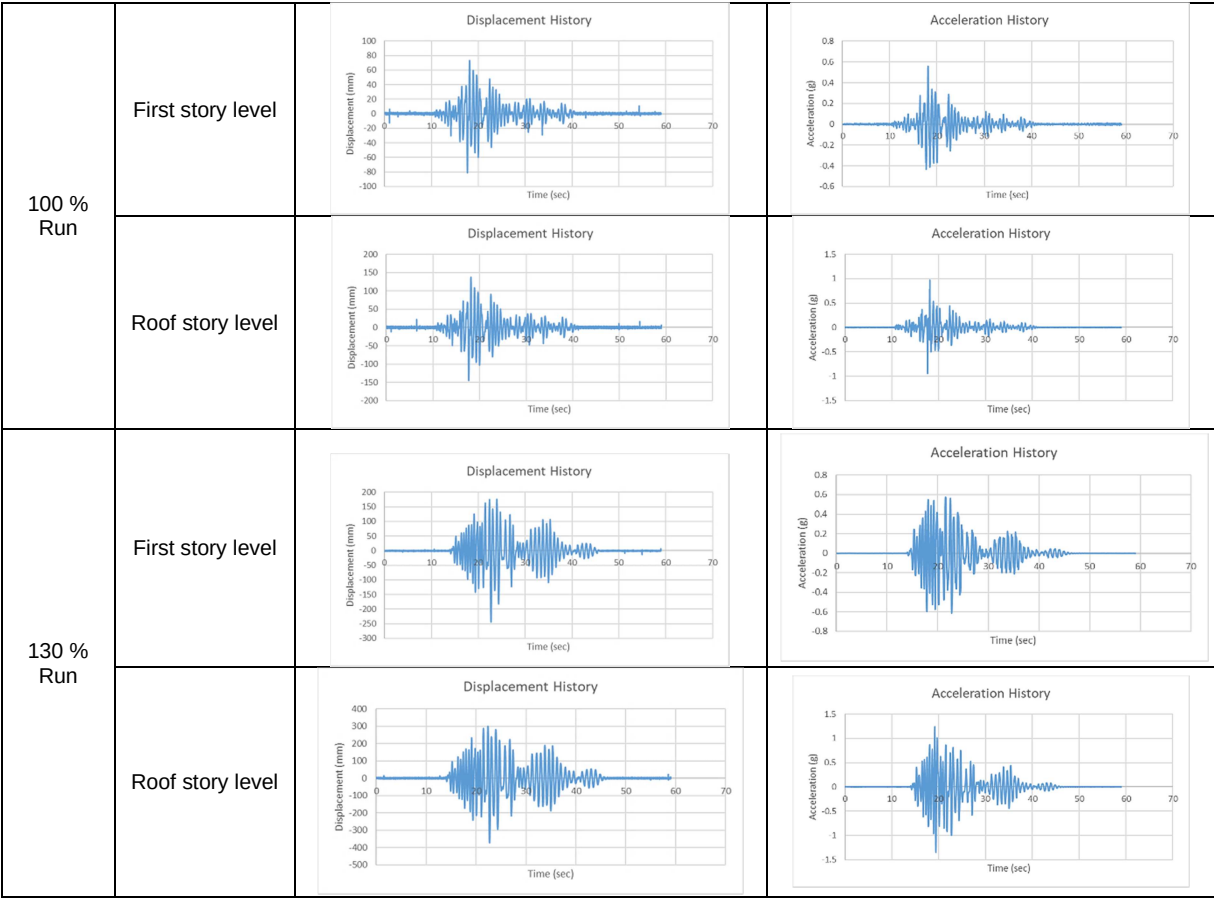
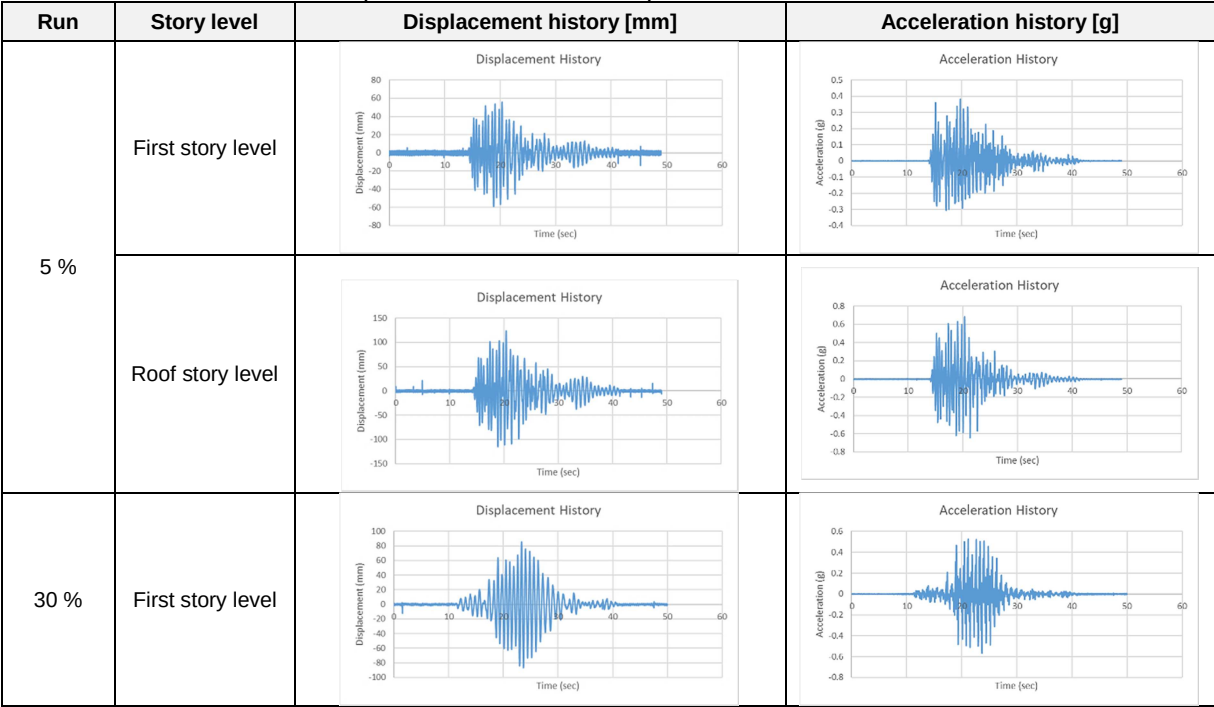
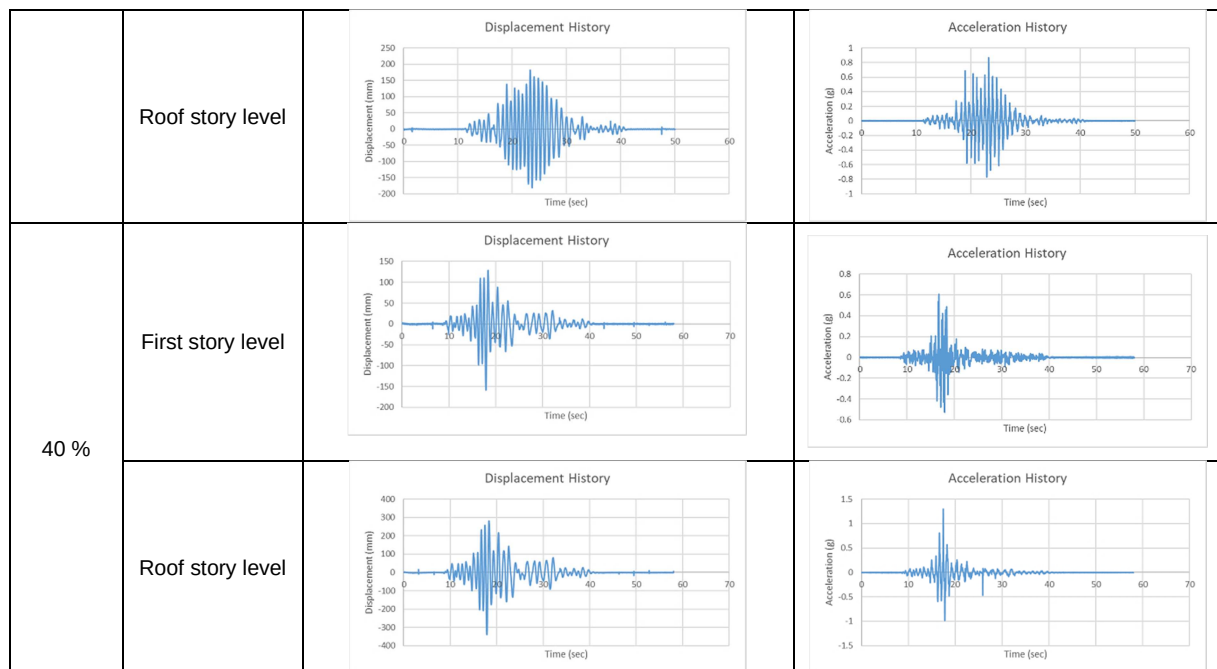


Table 4: Model 2 experimental observed displacement and acceleration histories.





3 Results and discussions

3.1 Observed damage response of test models

Tables 5 and 6 reports the code compliant and deficient test models peak values of roof story displacement, drift, base shear force and corresponding cracking patterns for significant excitations. Whereas, Figs. 7 and 8 shows the test frames global damage evaluation, for the same shaking excitations. In the start, both test frames were excited with shake table self-check run. During this run the code compliant specimen was displaced in the lateral uni-direction, with an excitation's intensity of 0.6 g (about 90 %) and the corresponding roof story drift was about 0.87 %. Sever flexural cracking was observed at the first story beam members during this run, which was due to the beam longitudinal reinforcement yield.

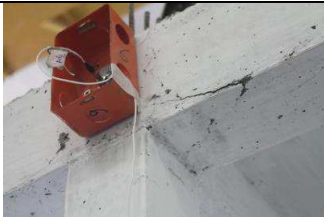
Table 5: Observe damage in Model 1.

Run	Roof story maximum displacement mm (Inch)	Roof story maximum drift [%]	Maximum base shear force kN (kips)	Observe damage
Self-Check– 0.60 g	61.45 (2.42)	0.87	151.08 (33.96)	 Flexure horizontal & vertical cracks in beam  Flexure cracks at base of columns

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

Table 6: Observe damage in Model 2.

Run	Top storey maximum displacement mm (Inch)	Top storey maximum drift [%]	Maximum base shear force kN (kips)	Observe damage
5 % – 0.52 g	123.69 (4.87)	1.75	117.92 (26.51)	 Visible cracks in beam and column elements, roof story level  Small horizontal cracks on joint panel area first story level

30 % – 0.35 g	182.37 (7.18)	2.57	137.48 (30.91)	 Joint panel area significant cracking, roof story level  Joint panel area significant cracking, first story level
40 % – 0.73 g	338.19 (13.31)	4.77	184.50 (41.48)	 Concrete spalling crack concentration in joint panel, roof story level  Crack concentration in joint panel, first story

The first story level beam was also observed with small vertical interface cracks. Few flexural cracks were also observed at the base of first story columns. After the self-check run, the code complaint model was further excited with increasing scaled excitations of 5 %, 10 %, 20 %, 30 %, 40 %, 50 %, 60 %, 70 %, 80 %, 90 % and 100 %. During all these runs, no significant damages were observed but only the existing cracking was widened. After the 100 % run, the test specimen was then excited with a 130 % which has displacement the test specimen to lateral drift of about 5.26 %, with an excitation intensity of about 1.06g. During this 130 % run, significant damages were observed in the test specimen. Already existing cracks were significantly spread and widened over the surface of members. Sever concrete cracking and crushing at the first story and with minor cracking roof story column bases were observed. Some major cracks on the joint panel region at the first story level and minor cracks at the roof story level were also observed during this run. After a 130 % run the testing sequence was stopped as the test model was in near incipient collapse state.

The code deficient test model was also tested with the same sequence but due to weak beam-column joint (no shear reinforcement in the joint panel regions) and reduced concrete strength deficiency, the test model was only able to resist about 40 % of the input excitation. The damages sustained by the beam, column and joint panel regions of the deficient model was quite earlier and significant as compared to the code design reference frame model. The damages in joint panel regions were mostly centered in case of deficient models, whereas, the minor joint cracking was mostly spread over a large area around the joint panel regions.

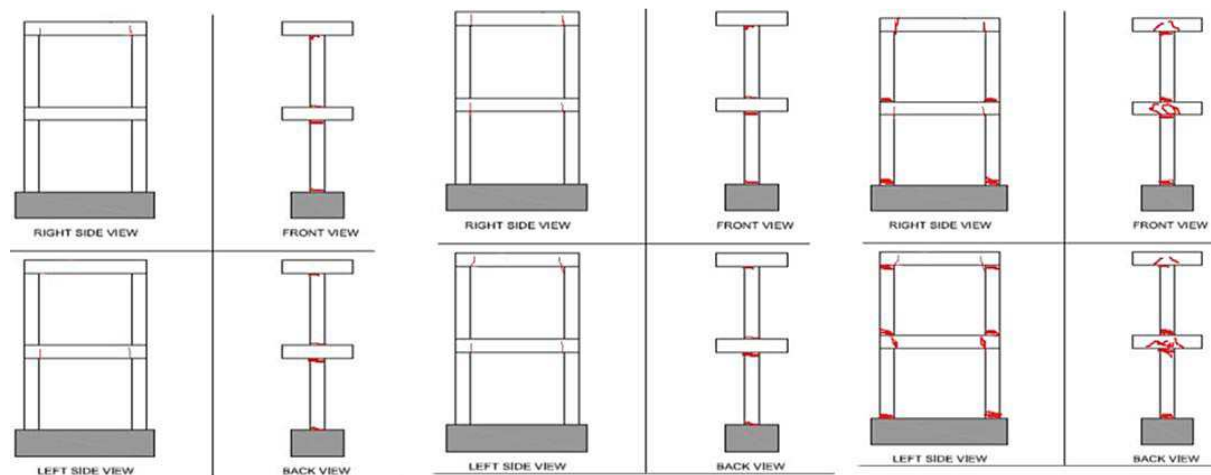


Fig. 7: Global damage response for Model 1.

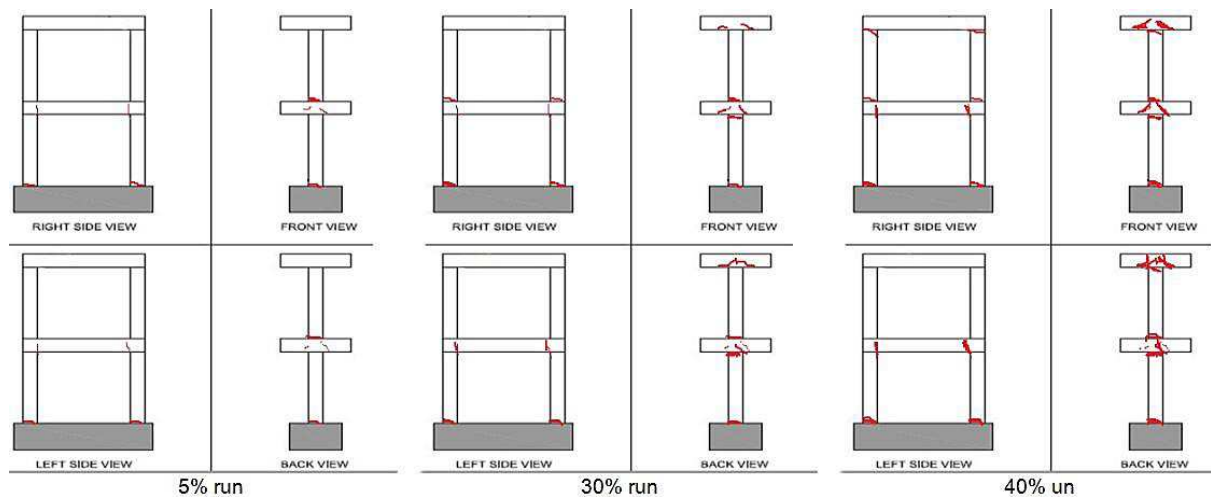


Fig. 8: Global damage response for Model 2.

3.2 Force - deformation relationships

The force-deformation relationships or capacity curves for the considered code design Model 1 and deficient Model 2 were obtained by correlating the peak values of displacement (drift) and base shear forces. At each floor level and for each run the corresponding peak value of accelerations was obtained and was multiplied with the respective floor masses (including floor, beam and column self-weight and extra steel weights) in order to obtain the story shear or inertial forces. The first and roof story level inertial forces were sum up in order to get the base shear force of the prototype frame. Fig. 9 shows the developed force-deformation capacity curves or envelope curves for code design frame Model 1 and deficient weak beam-column joint Model 2. The force-deformation relations obtained shown in Fig. 9 were further normalized by energy balance criteria in order to obtain bilinear idealized relationships, which can be employed for a comparable performance and computation of different seismic parameters. For this purpose, the actual capacity curves from the experimentally observed data were bi-linearized by equating the total area of real force-deformation relationships and the bilinear idealized capacity plots using iterative procedures. The peak values of displacement obtained during the final runs i.e. before the incipient collapse state, have been considered as the ultimate value of the displacement. By equating the energy, the corresponding value of the idealized yield deformation and strength has been evaluated from the developed bilinear curves as shown in Fig. 10. As it is depicted, the yield stiffness, strength and also the peak displacement capacity or ductility generally decreases for the case of deficient weak beam-column Model 2. It also observed that the yield stiffness of Model 2 decreases by about 50 % as compared to the code design reference Model 1. Similarly, in the case of Model 2 the yield strength has also been reduced by 31 % as compared to Model 1.

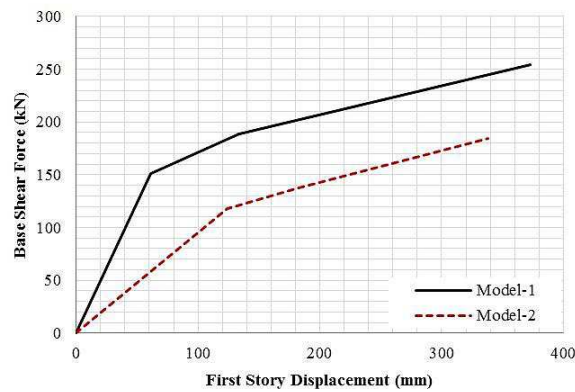


Fig. 9: Test models force-deformation capacity curves.

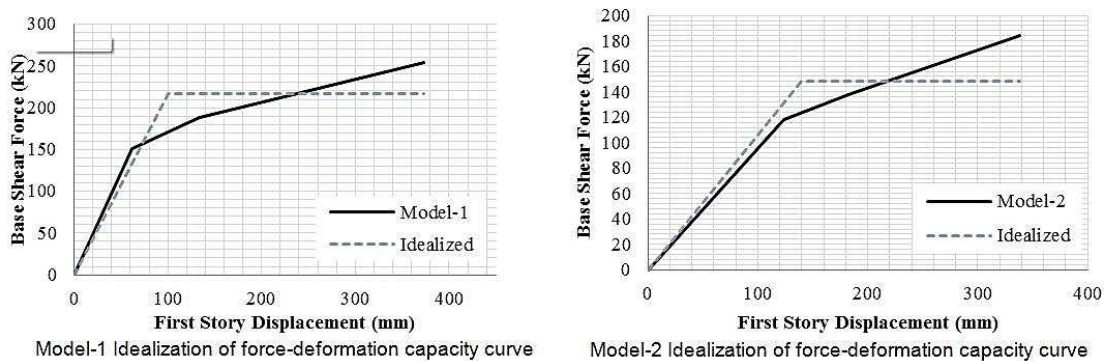


Fig. 10: Test models force-deformation capacity curves idealization.

4 Conclusions

Recent reinforced concrete special moment resisting frame SMRF structures with weak beam-column joints defects have been evaluated through dynamic shake table testing, in order to access the seismic performance and ultimate damage mechanism of such class of deficient RC structures. Unidirectional dynamic shake table tests have been used to the test model under various scaled excitations in order observe seismic damage mechanisms and develop drift-based performance limit states.

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

- 1) The code design SMRF model sustained, structural members flexural hinge formation at the ends, under extreme ground shaking and have response per the code very well i.e. about 1.3 times 1.06 g more than the input design level shaking.
- 2) The deficient weak beam-column frame model with no shear reinforcement provision in the joint panel region and built in low strength concrete, typically found in general construction, has only sustained up to 40 % of the input shaking. The weak beam-column joint SMRF model was able to only resist 40 % of the PGA value 0.73 g as compared to the reference code design SMRF model, which was able to resist 130 %, 1.06 g.
- 3) Due to the non-provision of joint panel region shear reinforcement and low strength concrete the damages response consists of flexural yielding of the beam and column members, followed by significant damages concentrations in the joint panel regions, thereby decreasing the load carrying capacity. Such damage can cause a story mechanism with a partial or full collapse of the building system.
- 4) It has been observed from the current experimental study that, the yield stiffness, strength and also the peak displacement capacity or ductility generally decreases for the case of deficient weak beam-column Model 2. It also observed that the yield stiffness of the Model 2 decreases by about 50 % as compared the code design reference Model 1. Similarly, in the case of Model 2 the yield strength has also been reduce by 31 % as compared to Model 1.

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