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PSEUDO-DYNAMIC TESTING OF LARGE-SCALE REINFORCED CONCRETE FRAME WITH OPEN GROUND STORY

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Abstract

Pseudo-dynamic (PsD) testing procedure is considered as one of the effective tool to investigate the seismic performance of structural components. In the PsD test, the results of numerical simulation of a prototype structure is used in experiment and vice-versa, which provides a flexibility of ascertaining the real-seismic performance of structures without shake-table tests. In this study, PsD test was carried out on a half-scale two-story reinforced concrete (RC) frame with open ground-story. The seismic performance of the test specimen was investigated for a scaled recorded ground motion. The main parameters investigated were the locations of damages, lateral load resisting capacity, and mode of failure. The results of PsD test were compared with the numerical results of a prototype frame under the same set of ground motions.

Keywords: Pseudo dynamic (PsD; Open ground story; Soft story mechanism

1. Introduction

Pseudo-dynamic (PsD) test is one of most economical and most effective test to assess the dynamic behavior of the structure. One of the most realistic test to assess all non-linear dynamic response of the structure is shaking table test. But this has limitation in term of strength, size and weight. On the other hand, the most economical and common method for obtaining information on the inelastic behavior of structures is through quasi-static tests, but it can't provide the real dynamic response of a test specimen [1]. Essentially in shake-table method, the three basic dynamic forces namely, inertial, elastic and damping forces are induced in the tested structure. Conducting such a realism experiment to evaluate seismic response of structures necessitates the use of highly sophisticated and expensive dynamic actuators and control systems. Also, designing a large shake-tables which capable of reproducing actual ground motions is to difficult, particularly when simulating multi-axial earthquakes. Among the reasons limiting the simulation of realistic effects are the deformability and inertia of the shake-table, its characteristic modes of vibration, the devices needed to carry the dead load of test structure and overturning moments without impeding the table's motion, the friction of the bearings, the physical capabilities of the hydraulic actuators, and to a lesser extent the limitations in the control devices [2].

In recent years PsD method is being adopted as an alternate to conventional shake-table method to evaluate the seismic performance of structures. PsD is the only one test which can combine both the realism of shaking table test and quasi-static test. In this method, varying displacement was applying slowly to the test structure. However, during testing, the motions and deformations observed in the test structure were used to infer the inertial forces that the structure would have been exposed to the actual earthquake; this information is then fed back into a control engine so as to determine and adjust the effective dynamic displacements that must be applied onto the structure. These pseudo-dynamic forces are typically accomplished by means of actuators pushing against a large reaction frame. This method has the advantage of testing large and tall structures with center of mass well above the base, which normally cannot be tested on a shake-table for evaluating their seismic performance [3]. As this method involves application of dynamic forces in an equivalent static mean through static actuators, close monitoring of the structural behavior including crack initiation, crack growth and stiffness degradation becomes possible. The draw back in such a hybrid method is the lack of simulation of strain rate effects which may not be critical under seismic loads. Also the method is time consuming due to its iterative nature. Fig. 1 shows clearly PsD test procedure.

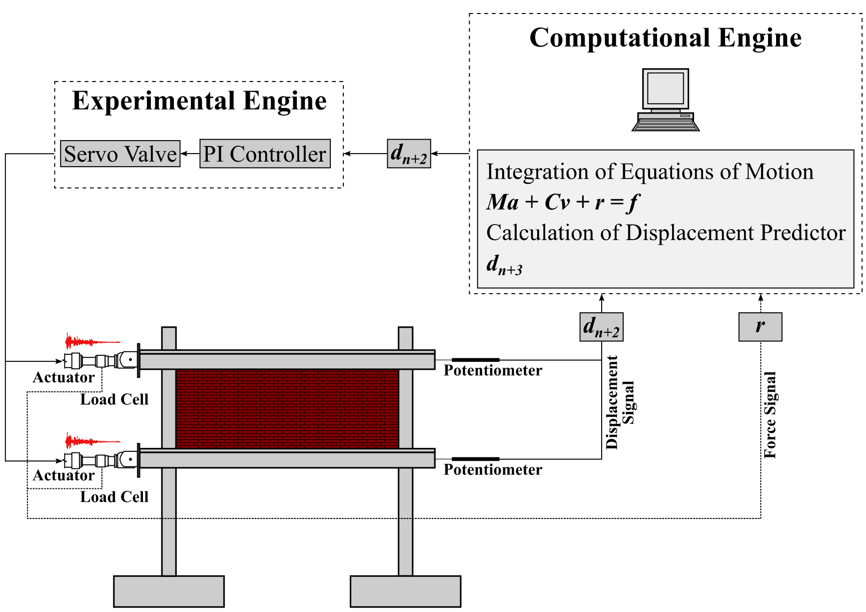


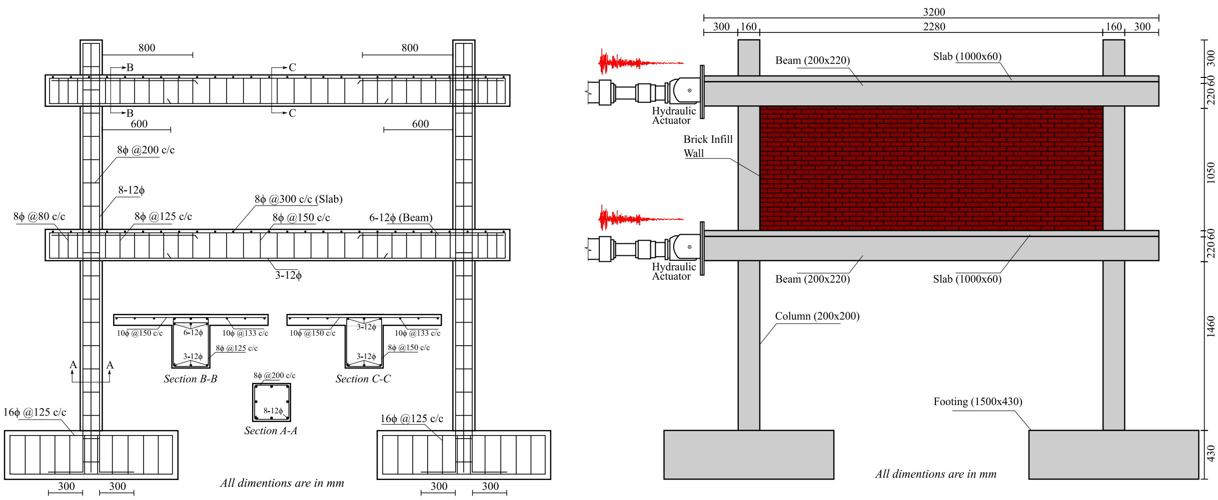
Fig. 1 – PsD Test Procedure

Generally, open ground story building is preferred for various reasons like garage and store purpose etc. Due to vertical irregularity of this building, during the earthquake performance is found to be not satisfactory. Major failure of this category of buildings are due to high displacement demand on first floor level [4,5]. To investigate such deficient building in terms of lateral load, in this study half scale open ground story building was tested using Elcentro ground motion with the help of PsD test. And the same parameters were investigated numerically using *Opensees* platform.

2. Prototype Study Frame

2.1 Description of Frame

A double-story single-bay reinforced concrete (RC) frame representing an interior bay of a prototype framed structure was considered as the test frame in this study. All the dimensions and percentage of reinforcement of test frame was simulated using 0.5 scale factor. The overall width and height of test frame was taken as 3200 mm and 3800 mm respectively. The cross sectional dimension of beam was taken as 200x220 mm, whereas both the columns dimension was 200x200 mm. To include the slab effect on the overall cyclic performance of test frames, a monolithic slab of 60 mm thick and 1000 mm wide was cast over the beam. Fig. 2 shows the details of the geometric properties and elevation of the test frame. In representing the existing non-earthquake resistant building, ductile detailing was not considered. Detailing was carried out as per Indian standard IS-456 [6] provisions.



(a) (b)

Fig. 2 – (a) Detailing of test frame, (b) elevation of test frame

2.2 Material Properties

The characteristic cube compressive strength of concrete used in the mix design was 25 MPa (referred as M25 grade of concrete), which resulted in a target mean cube compressive strength of 31.6 MPa as per IS: 10262 (2009) provisions. The compressive cylinder strength was used in the numerical modelling as input parameter. Table 1 summarizes the cube and cylinder compressive strengths of concrete used in the test frames. Thermo-mechanically treated (TMT) steel reinforcement bars were used as both longitudinal reinforcement and transverse stirrups in both the specimens. From bar tensile test, the minimum specified values of yield stress, ultimate stress, and elongation of TMT bars were found as 375 MPa, 540 MPa, and 10.0% respectively. Since these bars do not exhibit a definite yield point, the 0.2% proof stress values was considered as the yield stress. Sufficient development length of reinforcement bars was provided at the required locations.

Table 1 – Cube and cylinder compressive strengths of concrete at 28-days of curing

Specimen type	Cube comp. strength (MPa)	Mean cube comp. strength (MPa)	Cylinder comp. strength (MPa)	Mean cylinder comp. strength (MPa)
M25 Concrete	40.5	40.1	28.7	27.5
	37.9		27.5	
	41.8		26.2	

2.2 Loading on Test Frame

A constant gravity load of 46.8 kN and 33.3 kN were applied at the top floor and first floor slab level of the specimens by means of concrete cubes. Since the axial load ratio in columns was found to be nearly 10% in practice, the applied axial load does not truly represent the site condition. Elcentro ground motion was applied on the structure up to 20 seconds. Fig. 3 shows full scaled Elcentro ground motion which having PGA value of 0.35g.

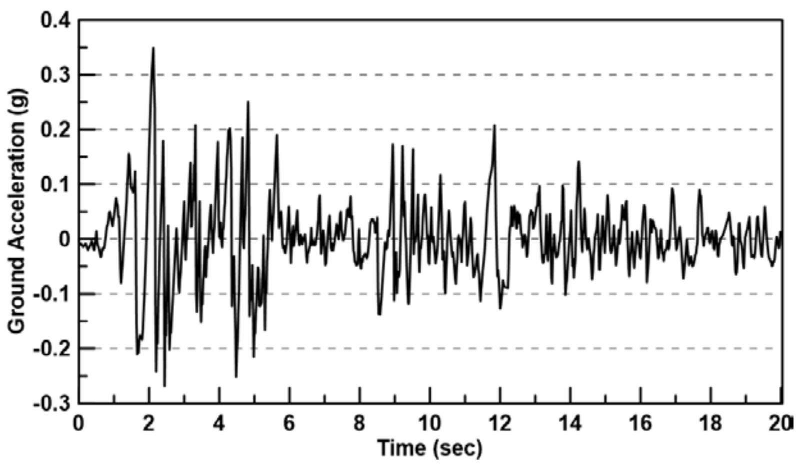


Fig. 3 – Full scale Elcentro ground motion up to 20 second

3. Numerical Modelling in *Opensees*

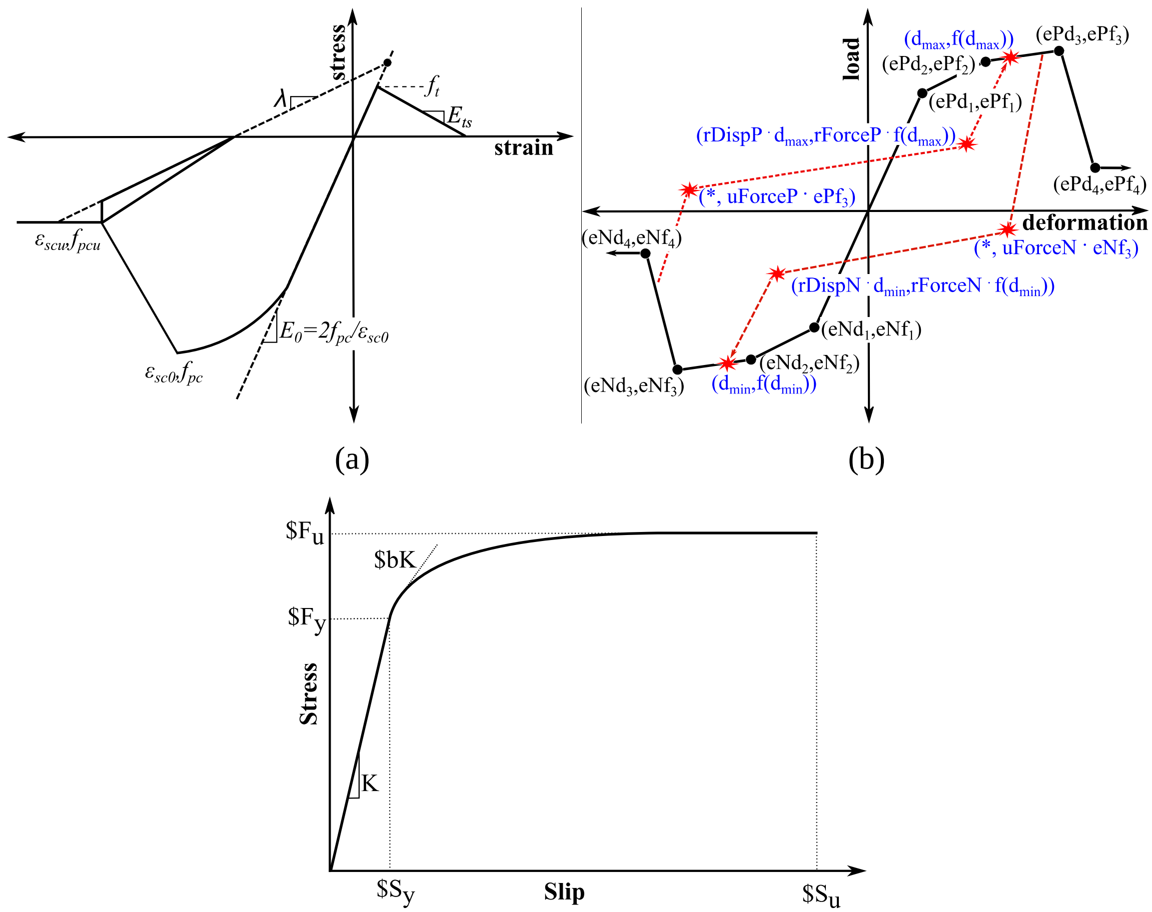
Test frames was modelled numerically in an analysis software *Opensees* [7] platform. The details of modelling technique used in this study are discussed in the following sections. Both columns and beams were modelled as nonlinear frame elements with fiber sections. And all the experimental feature were assigned on the numerical model i.e loading, boundary condition, material properties and ground motion etc. In the next section the experimental details are explained.

Beam column element was modelled using fiber sections and to predict accurately the inelastic behavior of concrete, “Force-Based Beam-Column Elements (*nonlinearBeamColumn*)” was used as an element. The respective material properties were assigned to these elements.

3.1 Concrete model

Uniaxial material model “*Concrete02”* was used to account for the compression as well as tensile behavior of concrete. The tensile strength of the concrete was not taken into account in the design of concrete members. But, for validating the experimental result tensile strength were not ignored. Direct tensile strength of M25 grade concrete was considered as 8-11 percent of compressive strength [8]. The value of concrete compressive strength in the numerical model was taken as 25 MPa only(See Table 1, cylinder strength). The tensile strength of the plain concrete was taken 10% of their mean compressive strengths [8]. The nonlinear monotonic behavior of concrete was characterized by a multi-linear curve, defined by seven parameters as shown in Fig. 4a. Which are

(a) maximum compressive strength (***f***$_{pc}$); (b) strain at maximum compressive strength (***ε*** $_{sc0}$); (c) crushing strength (***f***$_{pcu}$); (d) strain at crushing strength (***ε*** $_{scu}$); (e) ratio of unloading slope and initial slope (λ); (f) tensile strength of concrete (***f***$_t$); (g) tension softening stiffness (E_{ts}).



(c)

Fig. 4 – (a) Concrete02, (b) pinching4, (c) bond slip model

3.2 Pinching effect model

Pinching effect in the hysteretic response of the RC members at the higher cyclic excursions is a very common phenomenon. The hysteresis loops converging towards the origin point is termed as pinching. This is an inherent property of concrete, and it happens due to the combined effect of stiffness and strength degradation in the non-linear range while loading and unloading occurs. Due to this effect concrete members dissipate less energy compared to steel structures. To consider pinching effect in the modelling of the concrete members a uniaxial material called *pinching4* was used from *Openseess* library which is one of the most suitable material in the library. Fig. 4b shows the load deformation curve of *pinching4* material. The specialty of pinching4 material is that it considers material degradation (strength-stiffness). The strength-stiffness cycle degradation occurs in three ways; namely (a) unloading stiffness degradation; (b) reloading stiffness degradation; and (c) strength degradation.

3.3 Bond slip model

Bond slippage normally happens in column to foundation joint or beam to column joint. In this study bond slip model was applied to the column to foundation joint only. Due to lateral displacement imposes on beam level there was high bending forces developed at the column to foundation joint, so the probability of bond slip on this joint is was very high. In *Opensees* library there is a material called *Bond SP01*, which can model the effect of bond slippage. Fig. 4c shown deformation behavior of *Bond SP01* model. In this model strain penetration was counted at nonlinear deformation, which can generate the effect of bond slippage.

3.4 Masonry Model

To consider the masonry infill wall effect on test frame half scale masonry brick wall was used. Same wall was modeled in *Opensees* using three strut. For simplification in analysis code recommends a single strut for masonry modeling [9] but these will give conservative value, so in this study three strut model was adopted to distribute masonry wall effect uniformly on beam and column [10]. While modeling masonry strut, compressive strength was taken as 16 MPa and boundary condition on both end was kept as pin connection. Fig. 5 shows modeling parameters of masonry wall as a strut.

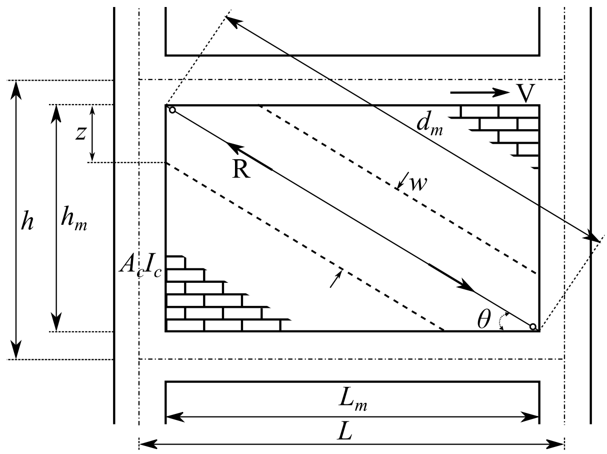


Fig. 5 – Masonry modeling details [11]

4. Experiment, Analytical Studies and Results

Stiffness of test frame structure was evaluated experimentally by applying a small displacement (2mm) at both beam level in different configuration [11,12] and the results obtained were closed to the theoretical calculation. Mass on the corresponding slab level were calculated as per static loading condition of the frame. Using stiffness and mass matrix, viscous damping values were calculated. The stiffness, mass and viscous damping matrix is as shown below.

$$K = \begin{bmatrix} 30 & -27 \\ -27 & 27 \end{bmatrix} kN/mm \; ; \; M = \begin{bmatrix} 3398 & 0 \\ 0 & 4768 \end{bmatrix} kg \; ; \; C = \begin{bmatrix} 0.03 & -0.02 \\ -0.02 & 0.02 \end{bmatrix} kN\,\mathrm{sec}/mm$$

4.1 Free Vibration Test

Free-vibration tests was carried out to find out the natural frequency and the first mode time period of the test frame. Natural frequency and time period of the test frame were found as 0.27 sec and 23.27 rad/sec. The response of the test frame were recorded through actuator at both slab level. This results were supported from the modal analysis with a time period of 0.28 seconds. Fig. 6 shows free vibration response of test frame structure. At the time of free vibration test, initial excitation displacement applied was 2mm.

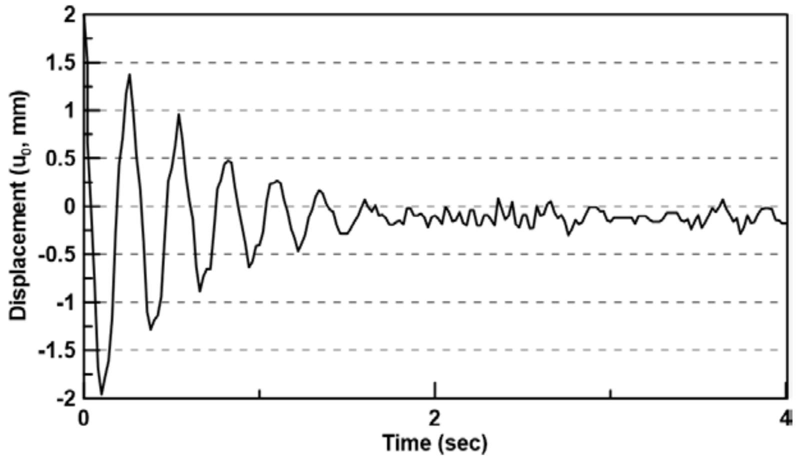


Fig. 6 – Free vibration response of test frame

4.2 Elcentro Ground Motion at 50%

Initially Elcentro ground motion was applied on the test specimen after reducing it to 50% and duration of ground motion was reduced to 20 seconds. At this level, experimental maximum peak lateral load resisting capacity of frame was observed as 32.2 kN and 30.6 kN in push and pull direction respectively. While numerical model predicted the same as 32.3 kN and 27.1 kN in push and pull direction respectively. Maximum displacement at the top beam level in push and pull direction were observed as 7.8 mm and 7.5 mm respectively, while numerically it was obtained 8.5 mm and 6.6 mm in push and pull direction respectively. Similarly, the maximum displacement at first floor beam level was obtained experimentally as 7.1 mm and 6.9 mm respectively. There was not much difference in experimental and numerical values in term of peak displacement and peck base shear in push and pull direction. Fig. 7 (a,b) shows the comparison of experimental and numerical study results in terms of displacement and base shear response during 50% Elcentro ground motion. Overall the performance of the frame under this earthquake intensity was elastic, so there was no damages on the ground column, beam and masonry panels.

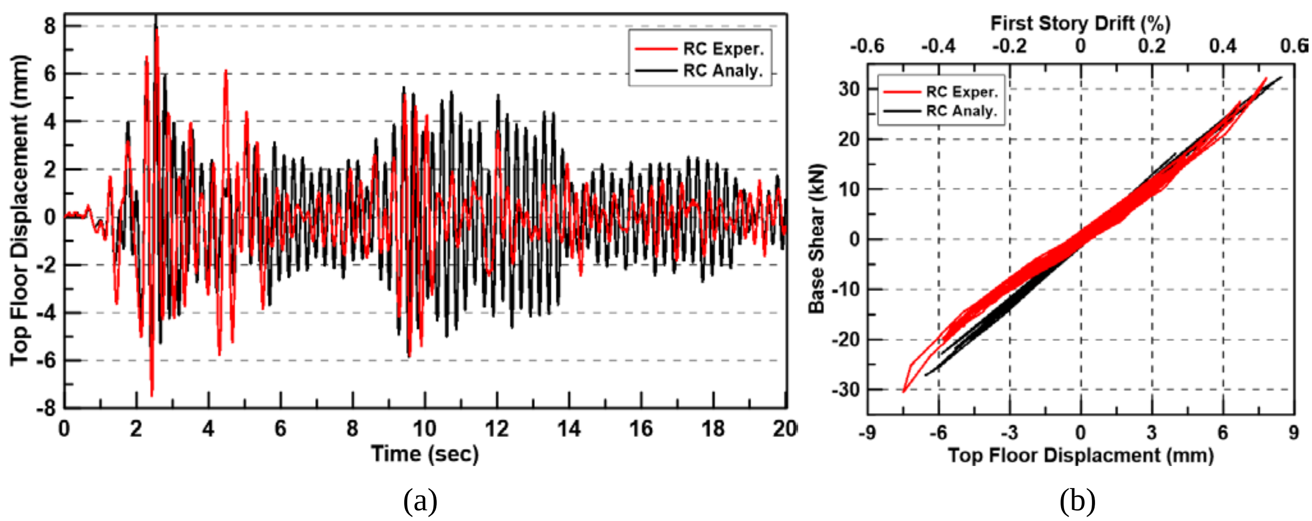
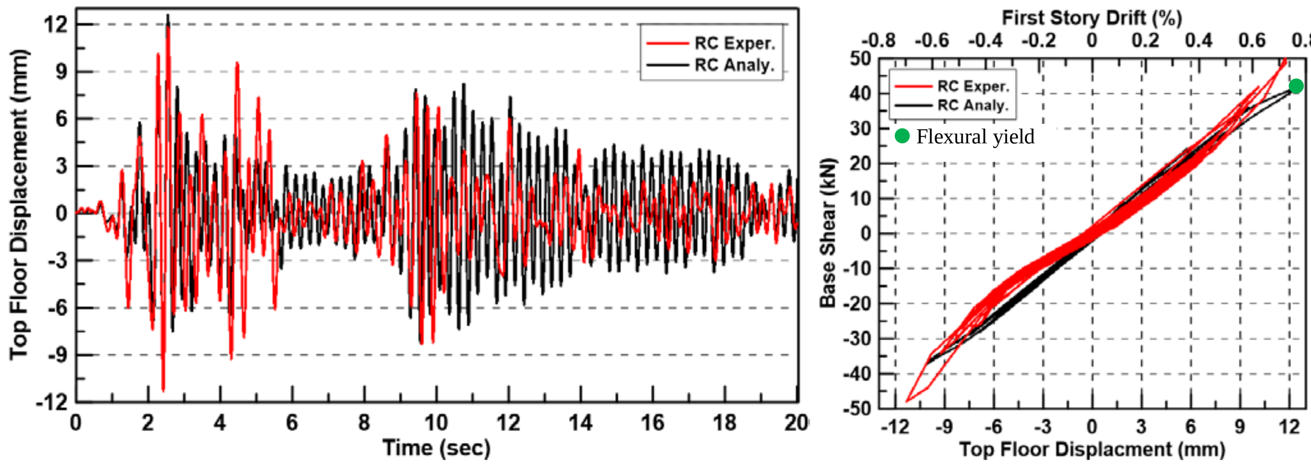


Fig. 7 – (a) Displacement and (b) base shear response of test frame under 50% Elcentro ground motion

4.2 Elcentro Ground Motion at 75%

After running 50% Elcentro ground motion, the intensity was increased to 75%. For this earthquake intensity, through PsD, the maximum peak load at top beam level in push and pull direction was observed as 50.1 kN and 30.6 kN corresponding to a displacement of 11.8 mm and 11.3 mm respectively. These values were supported by numerical studies, which had the peak values of base shear and displacement in push and pull direction at 42.5

kN, 37.6 kN and 12.6 mm, 10.2 mm respectively. Fig. 8 (a,b) shows comparison of experimental and numerical study results in terms of displacement and base shear response during 75% Elcentro ground motion. At this earthquake intensity level, micro cracks were observed at the plastic hinge zones of ground column while at the upper story there was no occurrence of micro cracks. Fig. 9 shows the state of the frame after applying 75% of Elcentro earthquake intensity. The cracks as seen in the figure appeared due to flexure dominated deformations.



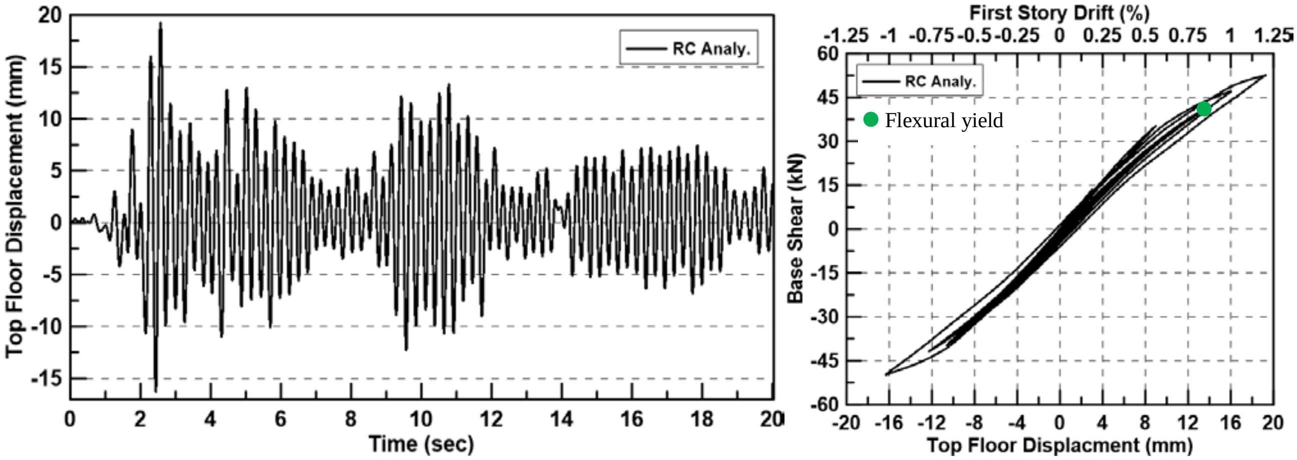
(a) (b)

Fig. 8 – (a) Displacement and (b) base shear response of test frame under 75% Elcentro ground motion

Fig. 9 – Test frame with loading condition and various damage on ground column in 75% Electro

4.2 Elcentro Ground Motion at 100%

After the 75% Elcentro impact on the test frame, there was minor damage on the ground column, so conducting experiment using 100% Elcentro impact was risky in terms of accident. Considering the safety aspect, this level of experiment was not conducted but from previous validated numerical model, this ground motion was studied. From the numerical studies, it was observed that the frame showed non-linear deformation with peak deformation of 19.3 mm and 16.3 mm in push and pull direction respectively. Fig. 10 (a,b) shows response of test frame under 100% Elcentro impact in terms of displacement and base shear.



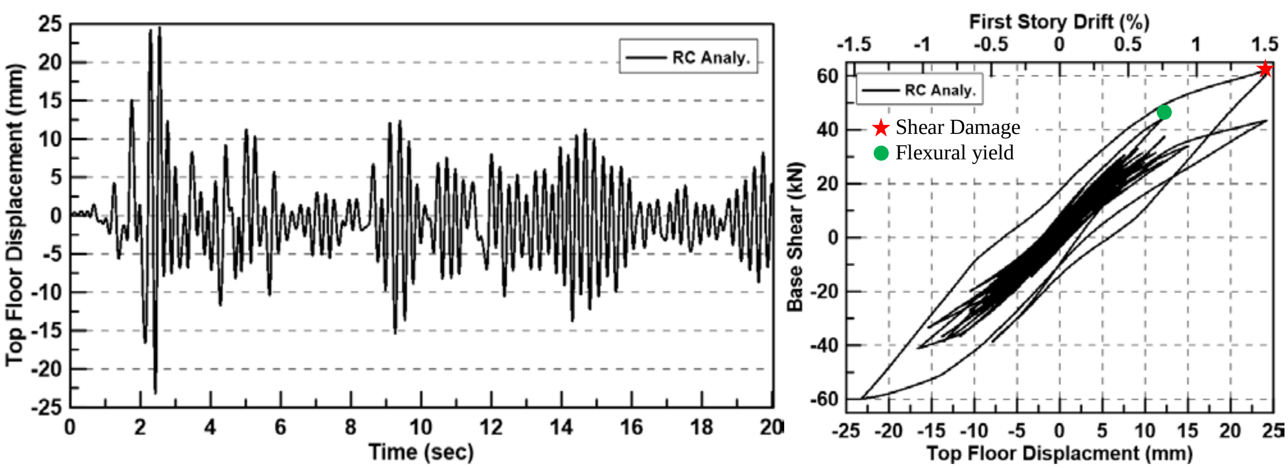
(a) (b)

Fig. 10 – (a) Displacement and (b) base shear response of test frame under 100% Elcentro ground motion

While peak base shear observed was 52.5 kN and 50.0 kN in push and pull direction respectively. This earthquake level didn't reach collapse level of this frame but the numerical model predicted flexural yielding in the ground columns.

4.2 Elcentro Ground Motion at 150%

To evaluate the collapse level of the test frame, Elcentro ground motion impact was increased up to 150%. At this earthquake level intensity, the test frame was about to collapse. Maximum displacement of top beam level in push and pull direction was 24.6 mm and 23.2 mm respectively. Story drift on first floor beam level was 1.53% and 1.45% in push pull direction respectively. And maximum base shear was recorded 62.5 kN and 60.0 kN in push and pull direction. Fig. 11 (a,b) shows response of test frame in 150% Elcentro ground motion impact in terms of displacement and base shear. Failure behavior was shear dominated at the ground column. In Fig. 11b, there was sudden drop down of load at 1.53% story drift level. This happened due to the ultimate shear deformation in ground column.



(a) (b)

Fig. 11 – (a) Displacement and (b) base shear response of test frame under 150% Elcentro ground motion

5. Conclusion

The major conclusion from this study are as follows:

- PsD test was successful in assessing the response of the frame for different levels of earthquake.
- The numerical model was able to predict the deformations and in-elastic response of the test frame as was seen during the experiment.
- For up to 75% of the ElCentro ground motion the response of the test frame was purely elastic with few micro cracks developing at the ground column. These cracks were developed due to the high flexural demands.
- Numerical model predicted flexural yielding of ground column at 100% of ElCentro ground motion without any signs of collapse of the frame.
- Numerical model predicted shear type failure of ground column at 150% of ElCentro ground motion highlighting the weakness of open ground story fames. So such type of structure need to be strengthened at the ground column for both flexural and shear demands.

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