ASPS Conference Proceedings 1: 373-378 (2022)

Proceedings of

12th Structural Engineering Convention - An International Event (SEC 2022)

Available at https://asps-journals.com/index.php/acp

Non-linear Dynamic Analysis of RC Structures Under Earthquake Sequences

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Paper ID - 040306

Abstract

Earthquakes in active seismic regions usually occur in a series of medium to strong intensity ground motions at small intervals of times. A large intensity mainshock (MS) is often followed by a series of aftershocks (AS) or even preceded by smaller foreshocks. This sequence type of mainshock-aftershock (MS-AS) ground motions with varying intensity pose major seismic hazardas there is limited scope of repair and retrofitting between seismic events. Due to continuous and repeated seismic ground motions over a brief period of time, the damages in the structure gets accumulated and structure collapse. This study navigates the behaviour of reinforced concrete structures under such seismic sequences. For this purpose, the nonlinear response of three 12 storey reinforced concrete buildings (regular plan, mass irregularity and diaphragm irregularity) is evaluated. The buildings are subjected to five real seismic sequences from previous earthquakes. Nonlinear dynamic analyses are carried out to study the response of buildings under MS and MS-AS sequences considering: a) material and geometric non linearities and b) irregularities. A single highest aftershock is considered in the present study. The results in this study indicates that MS-AS seismic sequence considering both material and geometric nonlinearities has significant effect on the response of structure. It also showed that seismic sequences significantly alter the response of irregular structure.

Keywords: Seismic sequences, mainshock-aftershock, nonlinearities, irregularities.

1. Introduction

In the past it has been observed that a large mainshock is followed by numerous aftershocks, these aftershocks occur because of the complex stress interactions in the fault systems. Generally, the stress on the earthquake fault drops drastically during mainshock and small distributions of stress and frictional strength cause that fault to produce most of the aftershocks [1]. Thus, buildings situated in active seismic regions are generally subject to mainshock-aftershock sequence. Aftershocks poses a major seismic hazard, damages of structures due to seismic sequences has been reported in recent earthquakes including Tohoku (Japan. 2011), Christchurch (New Zealand, 2010-2011), Chile (2010), Nepal (2015) and most recent in Kumamoto (Japan,2016) [2]. The 2012 East Azerbaijanearthquake hit northeast of Tabriz on August 11, 2012, and the strongest aftershock measured at M6.3 occurred 11 minutes after the M6.4 mainshock. The mainshock-aftershock sequence caused at least 327 deaths and more than 3000 other injuries [3]. The great Tohoku earthquake on March 11, 2011 in Japan triggered 60 aftershocks with magnitude 6.0 or greater and three over M7.0. The total economic loss in Japan is estimated at $309 billion [4]. The magnitudes of aftershocks are generally less than the mainshock but an aftershock may have a higher peak ground acceleration (PGA), longer duration than the mainshock. While the magnitude of aftershocks is smaller than mainshock, structures can be particularly vulnerable to aftershocks due to their high rate of occurrence and reduction in the lateral load-carrying capacity caused by damage induced by mainshock. Nonlinear behaviour of SDOF systems under multiple earthquakes indicates that the effects of multiple seismic events implied considerable accumulation of damage [5]. The behaviour of reinforced concrete structures under repeated ground motions of 2D bare frame models was investigated and found that multiple earthquakes were shown to require increased displacement demands in comparison with single seismic events. The study suggested that traditional seismic design procedure which is essentially based on single 'design earthquake' needed to be reconsidered [6].

It was also found that the use of real seismic ground motion data was crucial for evaluating the performance of structures under seismic sequences to consider the source mechanisms. Artificial seismic sequences lead to overestimation of responses [7]. Sequences of earthquakes increases the damage at structural members and the whole structure more than single seismic events when planar frames of reinforce concrete structures were analysed for five real seismic

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Proceedings of the 12th Structural Engineering Convention (SEC 2022), NCDMM, MNIT Jaipur, India | 19-22 December, 2022

Published online: December 19, 2022
doi:10.38208/acp.v1.524

sequences. Permanent deformation of structure also occurred under seismic sequences as compared with single seismic events [8]. The basic approach for seismic design of structures usually utilizes a single earthquake, but this approach has changed in the recent years. Due to subsequent exposure of ground motion over a brief period of time it is vital to analyse and observe the behaviour of structure under seismic sequences. In this study, linear and nonlinear time history analysis is carried out to observe the respective behaviour under mainshock (MS) and mainshock-aftershock (MS-AS) sequences.

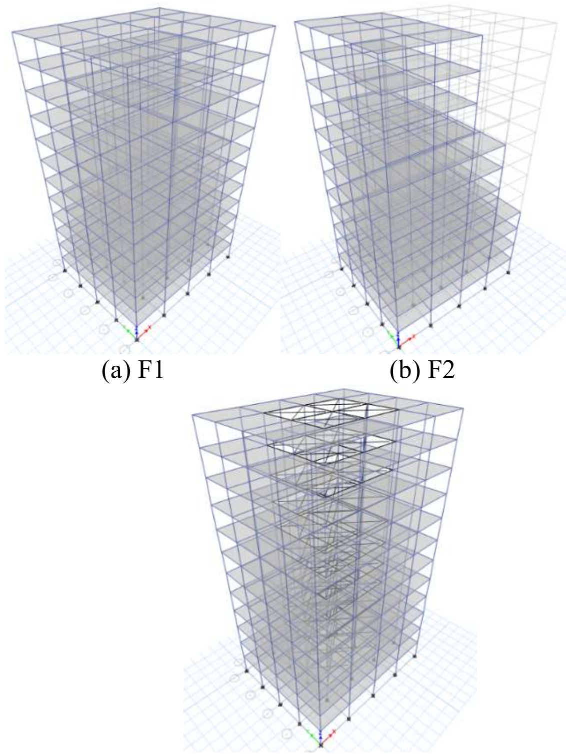
2. Modelling

In the present study, the response of three 3D reinforced concrete frames are examined for linear and nonlinear time history analysis under MS and MS-AS sequences.3D RC frames consideredare F1, F2 and F3, where F1 is a regular building, F2 has vertical irregularity (mass irregularity) and F3 has plan irregularity (diaphragm discontinuity).

The three frames considered has 4 bays in x-direction 5m in length for each bay, 4 bays in y-direction 4m in length for each bay. Beams and column have rectangular sections. F1 is a regular building with plan dimension 20m x 16m as shown in Table 1. F2 is an irregular building with irregular mass from height 16m to 36m. F3 has diaphragm discontinuity of plan dimension 10m x 8m which is hollow from top up to bottom of the building. While modelling, material nonlinearity is accounted by providing hinges and geometric nonlinearity by p-delta effects.Modelling of 3D RC frame and their analysis is carried out using commercial ETABS Version 2016 software. Fig.1 represents 3D schematic model of the three frames.

Table1: Modelling Parameters

Storey height	3 m
Size of beam	450 mm x 300 mm
Size of column	550 mm x 450 mm
Thickness of slab	125 mm
Dead load	1 kN/m^2
Live load	3 kN/m^2
Wall load	11.73 kN/m
Height of parapet	1.2 m
Load from parapet	3.45 kN/ m
Grade of concrete	M30
Longitudinal reinforcement	Fe500
Transverse reinforcement	Mild 250
Plan dimensions	20 m x 16 m



(a) F1 (b) F2

(c) F3

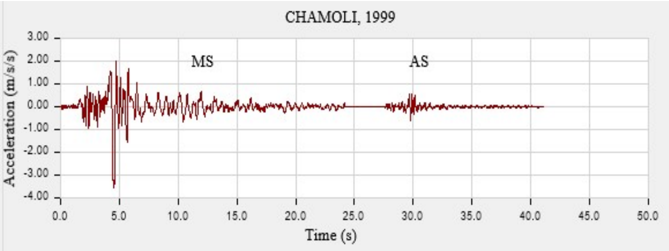
Fig. 1. Schematic 3D model of F1, F2 and F3

2.1 Seismic ground motion sequences

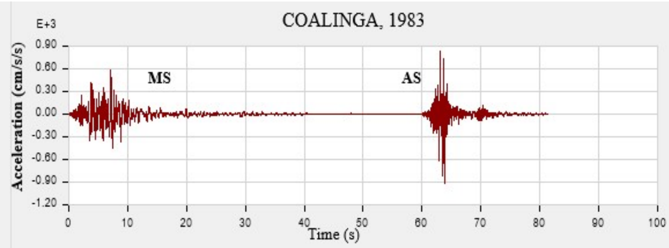
The seismic ground motion sequences are obtained from Strong-Motion Virtual Data Centre (VDC) facilitated by The Consortium of Organizations for Strong-Motion Observation Systems (COSMOS), California.The five seismic sequence records have a gap of 2 seconds between successive mainshock and aftershock seismic events which has zero ordinates of ground acceleration. The seismic ground motion sequences are tabulated in Table2. Fig. 2 shows the acceleration time history of MS-AS sequences of the five ground motion time histories. The ground motion acceleration is considered in one horizontal direction only i.e., x-direction for both MS and MS-AS sequence.

Table 2: Ground motion data

Ground motion	Time	Event	Magnitude	PGA (m/s^2)
Chamoli	19:05 (28/03/1999)	MS	6.6	3.528
	19:36 (28/03/1999)	AS	5.4	0.634
Coalinga	23:42 (02/05/1983)	MS	6.5	5.90
	02:39 (22/07/1983)	AS	6	9.2076
El Alamo	06:33 (09/02/1956)	MS	6.8	0.3242
	07:25 (09/02/1956)	AS	6.1	0.1545
Imperial Valley	22:16 (15/10/1979)	MS	6.5	7.704
	23:19 (15/10/1979)	AS	5	0.11267
Whittier Narrows	14:42 (01/10/1987)	MS	6.1	0.4682
	10:59 (04/10/1987)	AS	5.3	0.210



(a)



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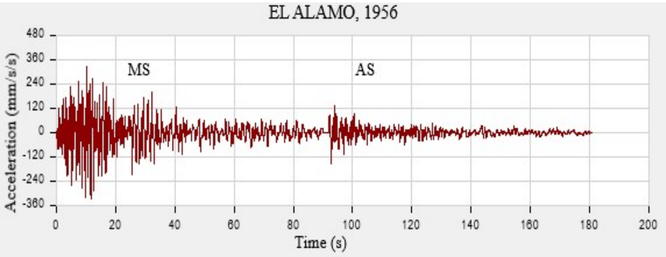

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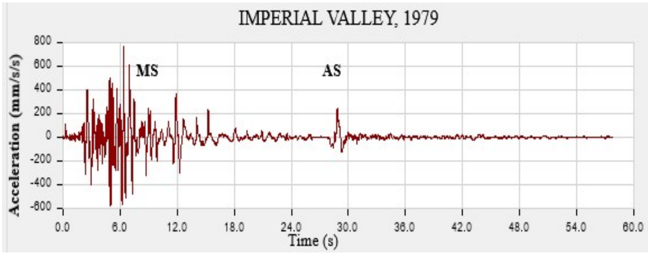

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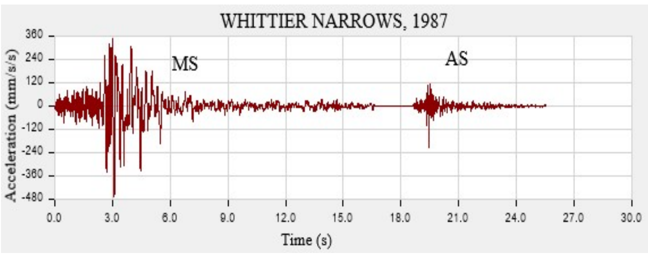


(e)

Fig. 2: Acceleration time history of seismic sequences

2.2 Time History analysis

In this study both linear and nonlinear time history analysis are carried out. Time history analyses is the dynamic response of a structure, step by step for a specified loading that vary with time. If the load includes ground acceleration, the displacements, velocities, and accelerations are relative to this ground motion.Firstly, linear time history analysis is carried out to observe the linear behaviour, nonlinear time history analysis is carried out considering both material and geometric non linearities to observe the nonlinear behaviour under MS and MS-AS seismic sequences. In the present study, p-delta effects are considered to account for geometric nonlinearity. To account for material nonlinearity automatic hinges were considered; Flexure M3 hinges and Flexure P-M2-M3 hinges were assigned for concrete beams and columns respectively. Secondly, for all the three frames with varying irregularities linear and nonlinear time history analysis is carried out under MS and MS-AS sequences. Finite element software ETABS 2016 is used to determinethe response parameters- maximum storey displacement, maximum storey drift, base shear and hinge formation.

3. Response of structure under MS and MSAS sequences

Terms used in the graphs and table indicates as under:

1. LMS – Linear time history analysis under MS
2. LMSAS- Linear time history analysis under MSAS
3. NLMS – Nonlinear time history analysis under MS
4. NLMSAS – Nonlinear time history analysis under MSAS
5. THA- Time History Analysis.

The results are presented graphically formaximum storey displacement, and maximum storey driftin Fig. 3 and Fig. 4 respectively.From Table 3 the base shear of each frame underMS andMS-AS sequences for both linear and nonlinear time history analyses are observed. The hinges formed at different levels of performance i.e., Immediate Occupancy (IO), Life safety (LS) and Collapse Prevention (CP) in all the three frames under MS and MS-AS are shown in Fig. 5.

From Fig.3 and Fig.4, it is observed that maximum storey displacements and maximum storey drifts under MSAS sequences arehigher compared to single MS under both linear and nonlinear THA.Also, the maximum storey storey displacements and drifts of all the structures under nonlinear THA are higher than linear THA by 45% and 20% respectively. This isdue to the addition of stresses and strains for considering geometric and material nonlinearities. The increase indisplacements and drifts under MSAS seismic sequences is due to successive ground motion that the structure undergoes within a small interval of time. This leads to accumulation of stresses, decrease in the stiffness and strength of structure which further results in permanent deformations.

It is also observed from Table 3 that as displacement increases under MSAS sequences, base shear decreases in both cases oflinear and nonlinear THA. The percentage decrease in base shear under MSAS sequences as compared to MS is24% and28% for linear and nonlinear THA respectively. The decrease in base shear under MSAS is due to loss of stiffness and strength under repeated seismic ground motion.Hence, nonlinear THA gives more realistic results while analysing RC structures as it takes into account the nonlinearities of the structure.

Table 3: Base shear (kN)

	Linear THA		Nonlinear THA	
Chamoli, 1999	**MS**	**MSAS**	**MS**	**MSAS**
F 1	6510.418	5371.095	3133.163	2763.831
F 2	5456.383	4851.850	2834.436	**2494.304**
F 3	5882.186	5483.996	3027.537	2640.012
Coalinga, 1983				
F 1	5142.464	4113.971	3085.793	2777.214
F 2	4984.304	4236.659	2688.000	**2284.800**
F 3	5080.320	4267.469	3190.930	2694.930
Imperial Valley, 1979				
F 1	8474.876	7472.457	6652.952	5976.573
F 2	7930.278	6409.000	6042.447	**4482.430**
F 3	8610.889	7266.244	6436.567	5020.320
Whittier Narrows, 1987				
F 1	3981.202	3583.082	2707.210	2263.379
F 2	3371.991	3037.835	1988.180	**1590.544**
F 3	4060.826	3099.832	2761.362	2153.862
El Alamo, 1956				
F 1	1222.711	1100.439	712.972	640.064
F 2	1089.837	980.853	584.760	**481.007**
F 3	1205.273	1084.746	631.192	596.399

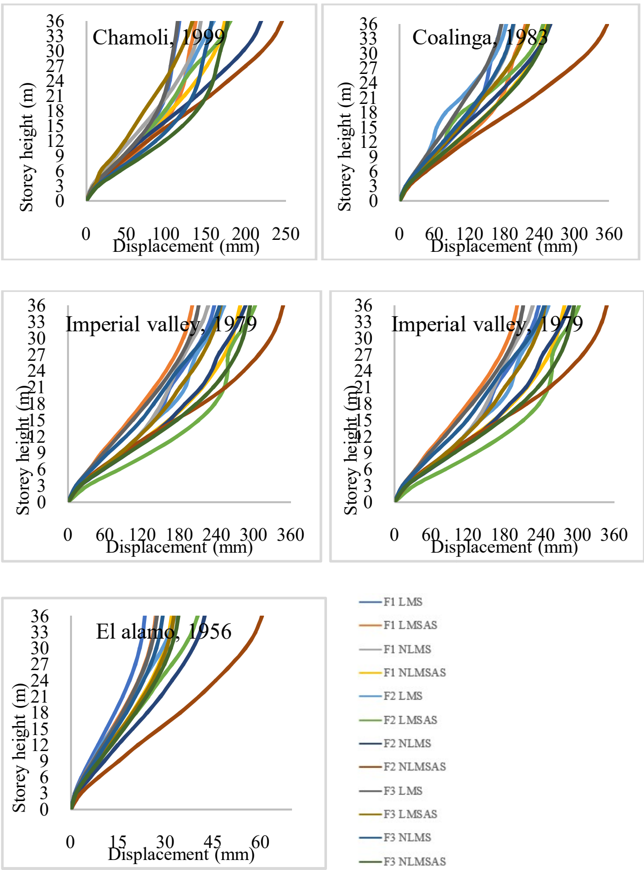


Fig. 3: Displacement of F1, F2 and F3 under MS and MSAS for linear and nonlinear THA

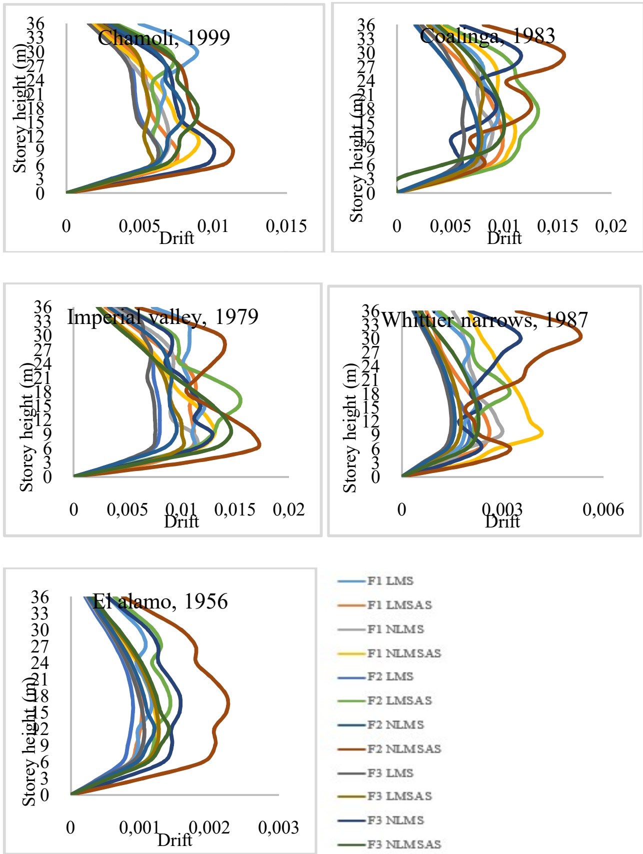
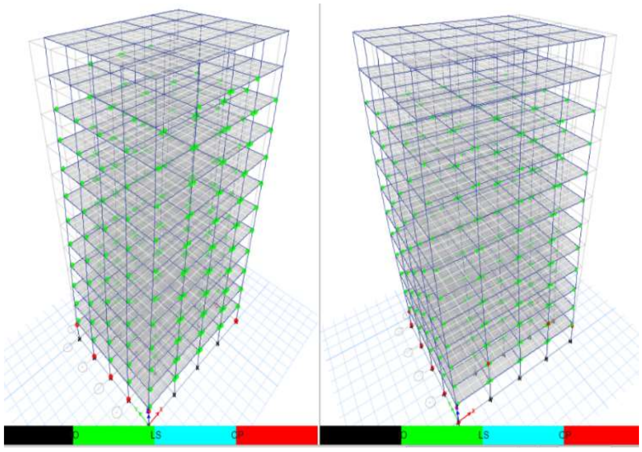
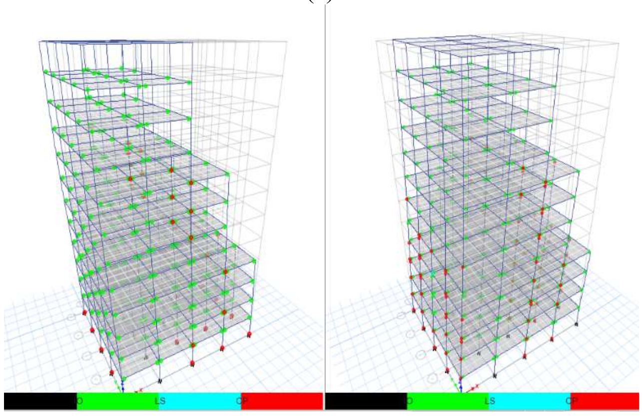


Fig. 4: Drift response of F1, F2 and F3 under MS and MSAS for linear and nonlinear THA



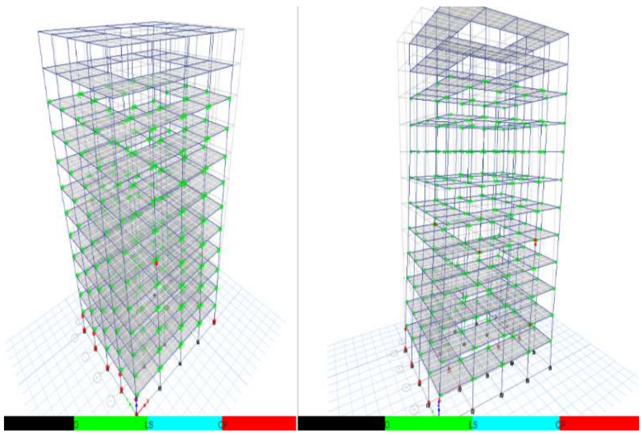
MS MS-AS

(a)



MS MS-AS

(b)



(c)

Fig.5: Hinge formation under MS and MSAS sequences

Results also indicated that, the maximum storey displacements and drifts of F2 is maximum for all the five ground motions. The maximum storey displacement is higher for F2 as compared to F1 and F3 by as much as 56% under severe earthquakes. Similarly, maximum storey drift for F2 is higher than F1 and F3 by as much as 65%. The increase in maximum storey displacements and drifts in F2 is due to the irregular distribution of mass in elevation. The uneven or irregular mass distribution in elevation causeshigher decrease in stiffness and strength of structure in F2 as compared to F1 which is a regular structure. F3 with plan irregularity undergoes higher displacements and drifts as compared to F1 butlower compared to F2. This is due to uniform stiffness present throughout the structure although the middle section is hollow in F3.

It is observed that Frame 2 has the least base shear, due to loss of stiffness and strength under successive ground motion and uneven distribution of stiffness resulting from mass irregularity in elevation.

Fig. 5 showsthe formation of hinges under collapse state is maximum in F2 which has mass irregularity. The hinges formed in F2 is almost doubled under MSAS sequences as compared to F1 and F2 which have marginal increase in number of hinges.

4. Conclusions

The present study examined the response of three frames which were regular and irregular structures. All frames were subjected to mainshock and mainshock-aftershock ground motion where only one aftershock i.e., the largest aftershock was considered. The frames were analysed for both linear and nonlinear time history analysis.The following conclusions are drawn from the present study: -

1. The response of buildings for nonlinear analysis are higher as compared to linear analysis as the nonlinear analysis considered both geometric and material nonlinearities which is the inelastic behaviour of the structure resulting in the increase in strains beyond the yield point.
2. The maximum storey displacements and drifts are higher for MSAS sequences as compared to corresponding single MS due to stiffness and strength degradation under successive ground motion.
3. The number of hinges formed under collapse prevention state for MSAS sequences is much higher as compared to single MS due to stiffness and strength degradation leading to permanent deformations under repetitive ground motion.
4. It is concluded that structure with mass irregularity i.e., F2has undergone maximum displacement, drift and hinge formation and the least base shear due to its irregular and uneven distribution of mass.
5. F3 with diaphragm irregularity exhibited higher responses in terms of maximum displacement, drift and hinge formation and lower base shear as compared to F1 due to reduction in mass.

Disclosures

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