Seismic Response of Shear Wall–Floor Slab Assemblage

Snehal Kaushik and Kaustubh Dasgupta

Abstract Reinforced Concrete (RC) structural walls are commonly used in tall RC frame–wall buildings in severe seismic zones for enhancing lateral strength and stiffness of the buildings. Conventionally, the structural walls in multi-storeyed buildings are designed in the same way as isolated shear walls. However, due to the presence of floor slabs at different levels, there is an increase of stiffness locally at each slab–wall junction, which may lead to a significantly different response of the assemblage as compared to the isolated shear wall. Also, the floor slabs tend to partition the slender wall into a number of smaller panels between successive floor slabs. The present study aims to investigate the seismic behaviour of such multi-storeyed slab–wall assemblage using non-linear time history analyses of three different models under ground motions recorded during a past earthquake.

Keywords Time history analysis · Shear walls · Slab–wall junction · RC buildings

1 Introduction

Reinforced Concrete (RC) structural wall is widely used in the lateral force-resisting system for multi-storeyed buildings located in the earthquake-prone regions. In such buildings, the wall is connected to the RC floor slab at every floor level. The junction region of shear wall and floor slab constitutes an important link in the load path from slab to the wall during earthquake shaking, thereby influencing the pattern of lateral load distribution in the various structural members of the system. Consequently, the behaviour of the lateral load resisting element affects the seismic performance of the overall building. During the Chile earthquake of 3 March 1985, many moderate-rise RC buildings got severely damaged. Most of them were designed

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with structural walls to resist both gravity and seismic loads. Walls and slab–wall connections sustained extensive cracking during that earthquake [1]. Also during the 27 February 2010 Chile earthquake, more than a hundred highrise RC shear wall buildings got severely damaged. Cracking of concrete walls and cracking of floor slabs occurred throughout the damaged corners of many buildings, causing significant building distortions. Damage caused to the buildings was mostly due to the compression failure of thin shear walls at the lower levels of the buildings [2].

From the past experimental research on a single-storey wall–slab assemblage, it was observed that the shear wall–slab junction experienced large stress concentration under combined axial and cyclic lateral loading [3]. The shear wall considered for analysis was squat in nature, and no study has been carried out on slender shear walls with connected floor slabs. Although various studies have been carried out on coupling action of the beam and floor slab in building with shear walls, the behaviour of shear wall–floor slab junction has not been studied extensively. Using finite element modelling and experimental studies, the bending stiffness of floor slab and its effects on the distribution of bending moments and stresses in slab have been investigated [4–7]. None of the past studies has focused on the detailed investigation of the behaviour of floor slab and shear wall junction under earthquake shaking. To investigate the behaviour of the shear wall–slab junction for rectangular walls in multi-storeyed buildings, non-linear time history analyses are carried out for three different models under ground motions recorded during a past earthquake.

2 Modelling Details and Parameters

A hypothetical five-storeyed RC frame–wall building is assumed to be located in Seismic Zone V as per the Indian Earthquake Code [8]. Three different models, namely, (a) five-storeyed building, (b) an exterior wall–slab assemblage (EWSC), and (c) coupled wall slab (CWSC) assemblage where two shear walls are coupled with the slab in between (Fig. 1), are considered for carrying out time history analysis. Beams and columns are modelled using 2-node linear beam elements (B31) while floor slab and shear wall are modelled using 4-node doubly curved thin or thick shell element with reduced integration (S4R) in the ABAQUS/Standard [9] finite element program.

The mentioned models are analysed under seven different ground motions, recorded during the 2011 Sikkim earthquake in India, using the dynamic implicit method. In the dynamic analysis, acceleration time history is applied at the base of each specimen and it is increased with a smooth amplitude curve varying over time (in seconds). Each ground motion is scaled to the arithmetic mean linear-elastic 5%-damped spectral acceleration of the ground motion ensemble at the fundamental period of the structure being analysed. The elastoplastic material properties are assigned to the beams and columns of the full building, while the Concrete Damaged Plasticity (CDP) [10] model for concrete is assigned to the shear wall and the floor slab of the sub-assemblage model. The various ground motions are first scaled with

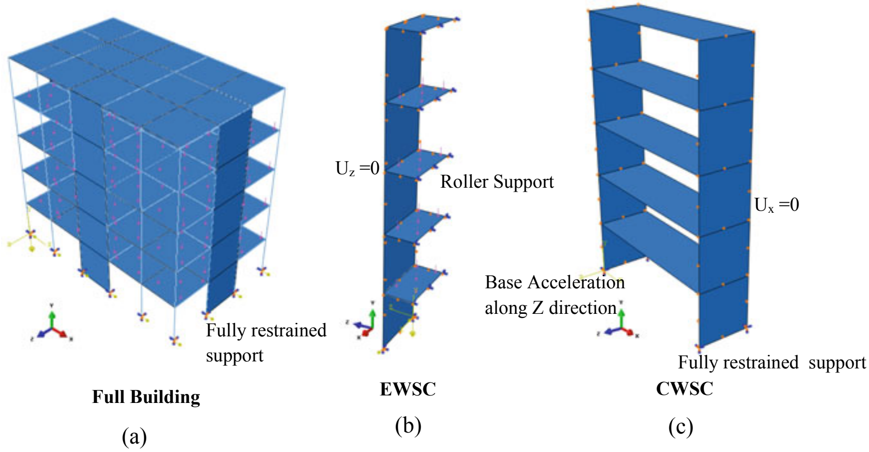

Fig. 1 Geometry and boundary conditions: **a** Full Building, **b** EWSC and **c** CWSC

the $\bar{S}_a(T_0)$ method (corresponding to the fundamental natural period) [11]. Then, the entire ground motion ensemble is scaled for the second time by different factors so the Peak Ground Acceleration (PGA) values become 1 g for all the records. Pseudo-acceleration spectra for the entire ground motion ensemble are developed using *SeismoSignal* [12] program.

The elastic modulus of concrete is considered as 25,000 MPa. Steel reinforcement is modelled with the material property assigned using the plasticity model in FE program. The yield stress, ultimate stress of steel are considered as 415 MPa and 527 MPa respectively. The modulus of elasticity for steel is considered as 2×10^5 MPa. The gravity loads (both dead and live loads) on the slab are assigned as pressure loads on the surface of solid elements. The total intensity of loading on slab including live load and floor finish is considered as 4 kN/m^2. The translational and rotational degrees of freedom are restrained at the bottom nodes of the wall. As the present study intends to investigate the non-linear behaviour for in-plane analysis of wall, the outer edges of slabs are supported on rollers, and the out-of-plane bending of the shear wall is prevented.

The assemblage analysed in the current study has a characteristic cube compressive strength of concrete as 25 MPa. The tensile strength of concrete is assigned as 3.5 MPa. In the current study, the dilation angle is assumed as 55°, eccentricity as 0.1, viscosity parameter as 0.01, shape factor (K_c) as 0.667 and stress ratio σ_{b0}/σ_{c0} as 1.16 [13].

2.1 Dynamic Analysis

The dynamic analysis procedure in the Abaqus program uses the implicit direct integration operator of Hilbert, Hughes and Taylor method [14]. The operator is an extension of the trapezoidal rule, with an additional parameter that can be varied to introduce different levels of numerical dissipation. In an implicit dynamic analysis, the integration operator matrix needs to be inverted and a set of non-linear equilibrium equations are solved at each time increment. The implicit operator options available in Abaqus/Standard are unconditionally stable and, thus, there is no limit on the size of the time increment for most of the analyses (accuracy of the results depends on the time increment in Abaqus/Standard).

2.2 Scaling of Ground Motion Records

Selection and scaling of strong ground motion time histories are critical and important to the time history analyses of structures. The scaling procedure used should be simple and should be implemented in such a way that the frequency content of the records is not changed. Real earthquake records are selected to match specific features of the ground motion, generally based on either response spectrum or an earthquake scenario with the minimum parameter being the magnitude, distance and site classification [15]. For the analysis and design, actual time histories are recommended to be used. The records should not be manipulated in the frequency domain but should be adjusted arithmetically in the time domain to match the desired spectral characteristics at the periods of most interest, or within a range around the period of interest [16]. To specify and predict the desired level of performance (degree of damage) of a structure for a specific level of ground motion intensity, non-linear time history analyses conducted using ground motion records that are scaled to adequately define the damage potential for the given site conditions and structural characteristics. Previous research describes many scaling methods of ground motion records. There are several methods that can be adopted to scale the ground motions in ensembles to produce a mean spectrum that satisfies the requirement of the studies.

In the current study, the various ground motions are first scaled to the arithmetic mean linear-elastic 5% damped spectral acceleration of the ground motion ensemble at the fundamental period of the structure being analysed, ($\bar{S}_a(T_0)$). The ($\bar{S}_a(T_0)$) method depends on the structural properties (i.e. (T_0)) as well as the ground motion characteristics. The ($\bar{S}_a(T_0)$) parameter is also referred to as the structure-specific ground motion spectral intensity. Thus, the entire ground motion ensemble is scaled for the second time by different factors so that the PGA value becomes 1 g for all the records. Pseudo-acceleration spectra for entire ground motion ensemble are developed using *SeismoSignal* program. The pseudo-acceleration spectra from the original ground motion record are shown in Fig. 2.

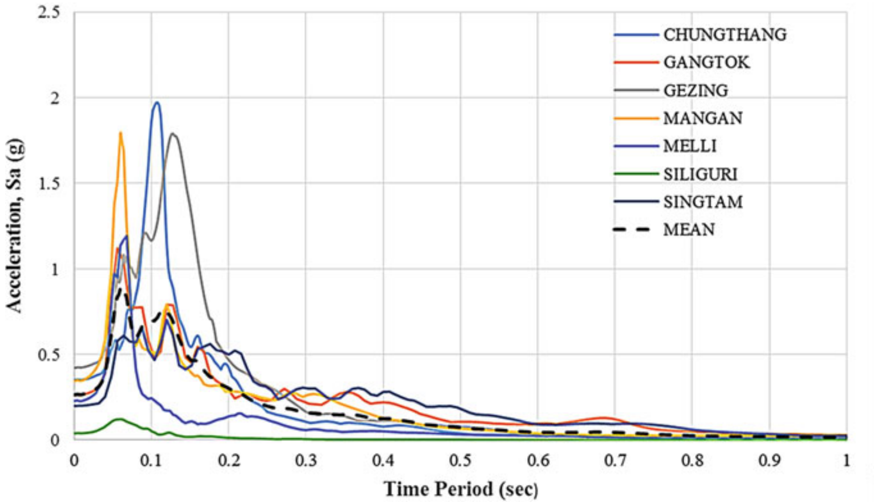

Fig. 2 Pseudo-spectral acceleration for original ground motion ensemble

The fundamental time periods for the full building, EWSC and CSWC models are obtained as 0.456 s, 0.041 s and 0.057 s, respectively. Each ground motion is scaled using ($\bar{S}_a(T_0)$) method of scaling, such that the spectral acceleration at the fundamental time period for all the specimens is equal to the mean spectral acceleration. The scaled pseudo-acceleration spectra for three above mentioned specimens are shown in Fig. 3. After scaling the original ground motions using ($\bar{S}_a(T_0)$), WAVGEN [17] program is used to generate seven (7) acceleration time histories to fit the scaled spectral acceleration. WAVGEN modifies a recorded accelerogram to make it compatible with a given Pseudo Spectral Acceleration (PSA) spectrum. Figure 4 shows the original ground motion records, the target pseudo spectra and the WAVGEN generated ground motion records developed from the target pseudo-acceleration spectra for 2011 Sikkim earthquake at *Chungthang* station. Similarly, for other stations, the ground motions are generated using WAVGEN. The specimens EWSC and CWSC are having very less fundamental time period, to achieve the expected damage in the specimens, these targeted ground motions require to scaling up for the second time. The entire targeted ground motion ensemble was scaled for the second time by different factors so the PGA value becomes 1 g for all the records.

The factors of scaling and various characteristics of the ground motion records are given in Table 1 for different recording stations. These scaled ground accelerations are employed to the two specimens, namely, (a) EWSC and (b) CWSC, to perform non-linear analyses in the time domain by using as input at the base of the specimens. The behaviours of the two models (EWSC and CWSC) are compared with the behaviour of the five-storied frame–wall building analysed under a single ground motion record.

Figure 5 represents the recorded and scaled Fourier amplitude spectrum of the recorded ground motions at *Chungthang* station of the 2011 Sikkim earthquake. It

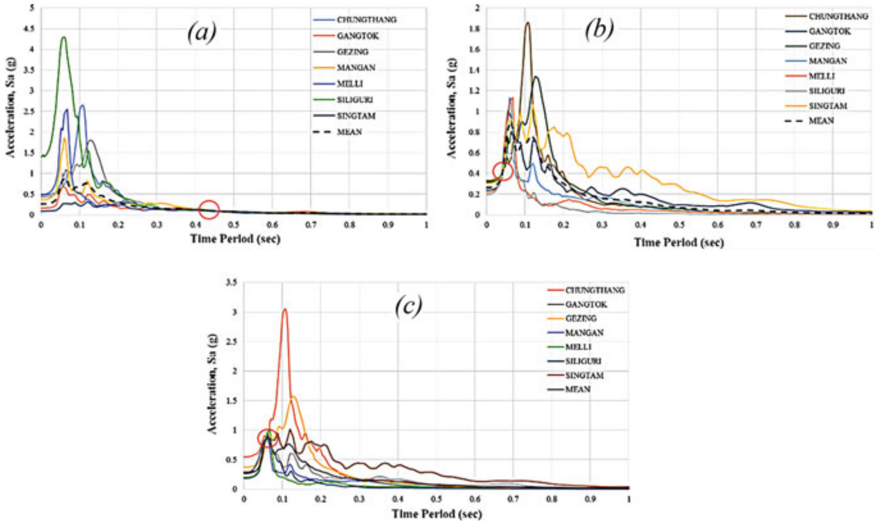

Fig. 3 Scaling of ground motion based on spectral acceleration: for **a** five-storeyed building **b** EWSC model and (c) CWSC model

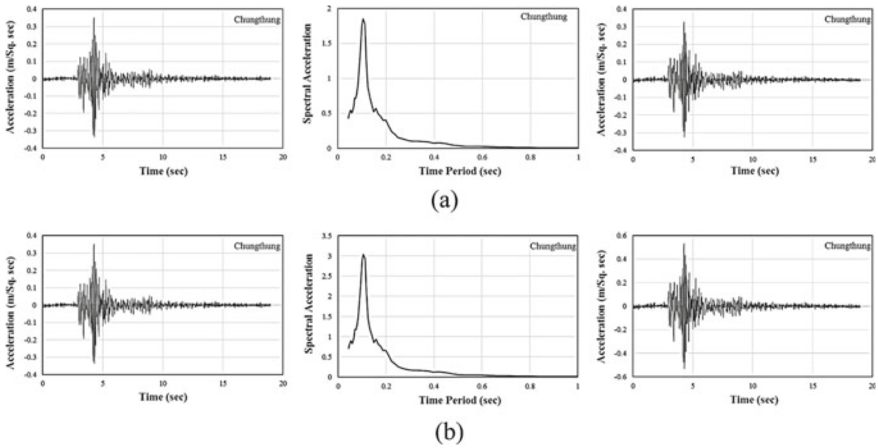

Fig. 4 Spectral response scaling at the fundamental natural period of the model for Chungthang station recorded during the 2011 Sikkim Earthquake; **a** EWSC model and **b** CWSC model

is seen that the energy content of the accelerogram gets changed after applying the fundamental period scaling procedure. After the application of the second scaling factor, the energy level increases significantly. It is also observed that the band for the frequency level where the energy is maximum is the same for the recorded and the scaled ground motions. The value of the Fourier amplitude changes with the scaling techniques.

Table 1 Characteristics of selected ground motion records for the 2011 Sikkim Earthquake

Station	PGA (g)	Scaling factor using ($\bar{S}_a(T_0)$)		Second time scaling factor	
		EWSC	CWSC	EWSC	CWSC
Chungthang	0.351	1.06	0.64	30.00	18.39
Gangtok	0.235	1.10	1.31	47.56	56.48
Geizing	0.422	1.33	1.13	30.92	26.31
Mangan	0.277	1.54	1.87	53.90	63.94
Melli	0.228	1.04	1.18	46.44	52.57
Silliguri	0.039	0.20	0.14	53.28	37.65
Singtam	0.200	0.66	0.69	34.20	35.85

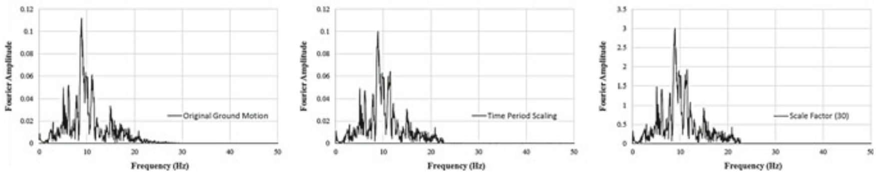

Fig. 5 Comparison of Fourier amplitudes for ground motions at Chungthang station

3 Finite Element Analyses Results

3.1 Five-Storey Shear Wall Building Model

The non-linear time history analysis of the five-storey building is carried out using a scaled acceleration ground motion of PGA value 1.12 g. The selected ground motion was recorded at Jellapur station during the 1997 Indo-Burma earthquake and the PGA was observed as 0.14 g. During the analysis, no damage was observed in the structure using the 0.14 g ground acceleration. To study the behaviour of the shear–wall slab junction, the PGA value is arithmetically scaled in the time domain up to 1.12 g. The input acceleration time history used for the analysis and the corresponding tensile damage pattern at the time instance of PGA are shown in Fig. 6a, b, respectively.

From the analysis, the damage is observed to initiate at the base of the shear wall first and then moves to the floor levels, mainly at the wall–slab junction. The damage starts at 0.2 s of the time interval at the base of the wall and reaches the wall–slab junction at the time instance of 1.46 s. The cumulative damage is determined and is observed to increase at the wall–slab junctions from the lower level. The cumulative damage, represented in Fig. 7a, shows that the damage starts earlier at the base of the shear wall as compared to the beginning of damage in the upper storeys. The maximum damage level is observed around the time instance of the PGA. It is observed that average tensile damage in the slab on the first floor level is more

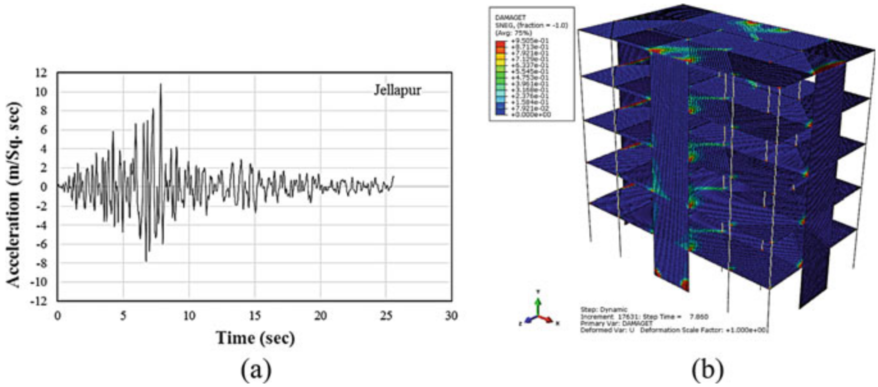

Fig. 6 **a** 1997 Indo-Burma (Jellapur) ground acceleration time history and **b** tensile damage pattern at the time instance of PGA

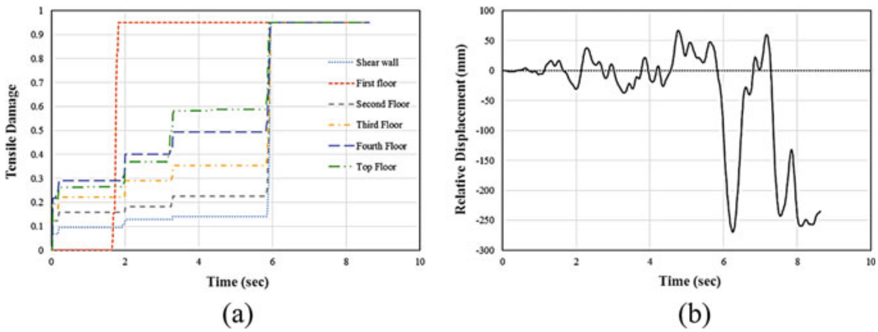

Fig. 7 **a** Comparison of average tensile damage parameter for five-storey building and **b** displacement at the top node of the five-storey building

severe than the other floor levels, mainly due to the stress concentration in the slab. Figure 7b represents the relative displacement at the top node of the building with respect to time. It is observed that the maximum displacement of 270 mm occurs in the negative X-direction, while 67 mm in the positive X-direction, around the time instance of PGA.

3.2 *Shear Wall–Slab Junctions for ESWC and CSWC Models*

Seven ground motion records from 18 September 2011 Sikkim earthquake of magnitude M_w 6.9 are selected to carry out the non-linear time history analysis of two different shear wall–slab assemblages (ESWC and CSWC). Figure 8 compares the tensile damage pattern of EWSC and CSWC specimens at the time instance of PGA.

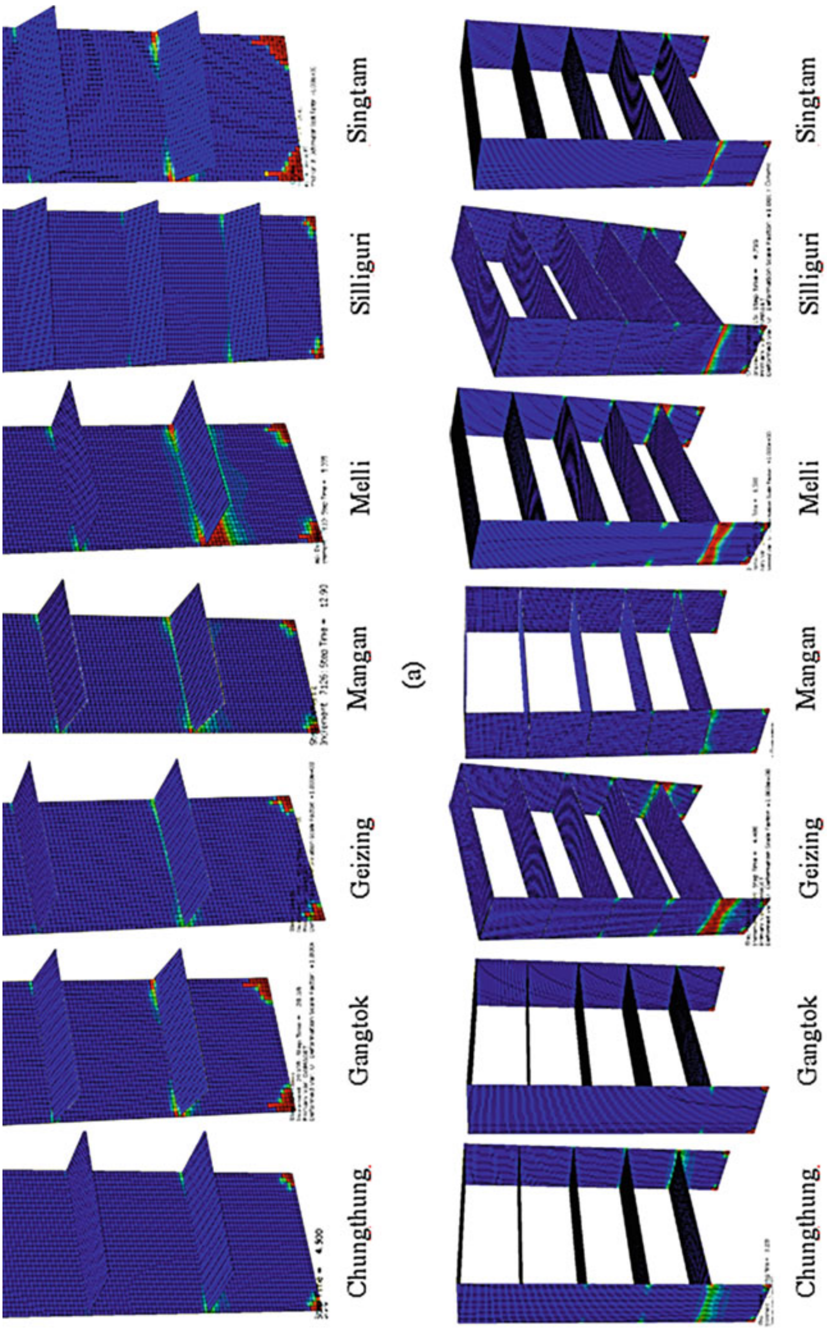

Fig. 8 Comparison of tensile damage pattern **a** ESWC model, **b** CSWC model at time instance of PGA for different earthquake stations

For both the models, it is observed that the damage at the base of the shear wall started well before the time instance of PGA. While the development of maximum tensile stress and tensile damage in the slab region started after reaching the PGA value, it can be concluded that the specimen undergoes maximum damage after the time instance of PGA. A similar damage pattern is observed at the wall–slab junction region considering different stations. The predicted tensile damage pattern at the shear wall–slab junction for both the specimens shows the effect of higher stress concentration at the junction region. The damage increases until the peak acceleration value is reached, thereafter it remains constant. It implies the achievement of maximum damage state in the structure.

For the CSWC model, with an elapsed time of shaking, the tensile damage moves to the upper floor levels. The damage reaches the fourth floor for the ground motion records at Geizing and Melli stations. For other ground motion records, the damage does not proceed beyond the second or third floor level.

4 Conclusions

Based on the present study, the following salient conclusions are drawn:

- For the scaled-up recorded ground motion of the 1997 Indo-Burma earthquake at station Jellapur (PGA 0.14 g), the cracking is observed to begin at the base of the wall and then get initiated at the slab–wall junctions. The tensile damage also gets propagated in the floor slabs due to the combination of slab displacement and flexural displacement of the shear wall.
- For the ESWC and CWSC specimens, analysed using ground motion ensembles of the 2011 Sikkim Earthquake, the maximum stresses and the tensile damage get developed at the base of the shear wall first and then get developed at the wall–slab junction region. Maximum stresses are developed at the first floor level. The damage increases till the attainment of the PGA value of the ground motion thereafter, it remains constant.

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