AFTERSHOCK FRAGILITY OF SHEAR WALL STRUCTURE WITH REPLACEABLE COUPLING BEAM

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Abstract

As an essential part of the lateral load resisting system, the coupled shear wall (CSW) structure in conformance with design method required in current seismic provisions may confront with the challenge of major damage or collapse in the aftershock occurred during the next several hours or days after the mainshock, even if the accumulated damage in the previous earthquake is slight. As more and more destructive aftershocks have been reported, the conception of resilience has become a research hotspot in civil engineering. The substitution of replaceable coupling beam (RCB) for the conventional RC coupling beam is proven to contribute significantly to energy dissipation and ductility characteristic in the CSW system by protecting the bottom of the shear wall from damage. Nevertheless, the aftershock performance of the CSW structure with the RCB has not been throughout investigated. In this study, a comparative study is conducted based on a 12-story prototype RC shear wall structure in condition of several RCBs. The aftershock seismic fragility of the CSW-RCB system is elaborated explicitly which is vitally important for the implementation of replaceable beam, recovery decision-making and its corresponding performance evaluation after replacement. The incremental dynamic analysis (IDA) is processed using an as-recorded earthquakes and the spectral acceleration with 5 percent damping ratio at the fundamental period of the structure is selected as the intensity measure. The three performance levels including immediate occupant, life safety and collapse prevention as specified in FEMA 356 are induced in CSW structures by utilizing mainshock IDA analysis, then the aftershock IDA is employed for the mainshock-damaged structures. Transient drift ratio is adopted as damage indicators to demonstrate the aftershock fragility. The study reveals that the CSW system contains RCBs may lead to a greater increasement of the vulnerability to aftershocks, and it emphasizes the importance of the aftershock effect in seismic design for the resilience-based structure.

Keywords: replaceable beam; shear wall structure; aftershock fragility; incremental dynamic analysis

1. Introduction

The Shear wall structure is of great significance to the seismic resistance in the lateral load resisting system as most buildings have gradually increased in height and slenderness with the development of urbanization. The traditional methods in improving the performance of the shear wall structure are typically accomplished by stiffening the structure through increasing the size of the structural component or the material strength and incorporating a vibration absorber damper. However, there are several limitations in adopting the above methods: the cost and self-weight increase but still not sufficient for reducing the structural response, significant damage in the bottom of the shear wall which is difficult or expensive to repair.

In recent years, the design of shear wall structure has been transformed to incorporating the replaceable coupling beam to concentrate deformation and dissipate earthquake energy to ensure the security of the shear wall [1,2]. As a new type of resilient structure, the shear wall structure with replaceable beams has been widely studied since it can achieve the purpose of decreasing the downtime and quickly recovering the normal function of the building by replacing the damaged coupling beam after an earthquake. Among the various types of replaceable coupling beam designed by many scholars, the metal coupling damper is considered as an effective form since its favorable energy dissipation ability, convenient manufacture process, low cost and easy constructions.

In the design of shear wall structure with replaceable coupling beam, only one earthquake or the mainshock is generally considered, and the aftershock effect is neglected assuming that the replacement process can be completed during the time interval. However, aftershock originates frequently typically accompanied by strong earthquakes in the following weeks, days or even hours. For example, after March 11, 2011 Tohoku earthquake, there were 3 aftershocks that had a magnitude greater than 7 and 50 aftershocks that had a magnitude larger than 6 in the following 2 weeks. It is not realistic to replace the damaged coupling damper and repair the shear wall structure to its undamaged state immediately after the mainshock.

Since the energy dissipation mechanism of metallic replaceable damper relies on the inelastic deformations of the metal, the sequence earthquake may lead to fatigue issues [3]. The replaceable beam acted as structural component, so its sudden failure may significantly affect the seismic capacity of the high-rise structures, which in turn may cause cumulative damage to the shear wall in the following aftershock. Therefore, it is necessary to consider the fatigue failure of the replaceable beam caused by aftershock in the inelastic analysis of shear wall structures.

In this paper, the aftershock fragility of CSW structure with RCBs is evaluated. First, the aftershock assessment method including the IDA analysis, demand parameter selection and fragility function are introduced. Then, a 12-story CSW structure designed according to ASCE 41-06 is untied in the case study. A 3D finite element model is constructed in the open-source software OpenSees and the fatigue issue of the metallic replaceable beam is considered in the model. Next, a mainshock IDA analysis is conducted to determine the scale factor to simulate the mainshock-damaged structure as well as the collapse capacity of the structure in the mainshock-only event. Finally, the aftershock IDA is carried out on the mainshock-damaged structure.

2. Aftershock fragility assessment method

In order to study the aftershock performance of CSW with RCBs at certain mainshock-damage states, the analysis method proposed in the literature [4] is utilized here. The mainshock IDA analysis is first conducted to determine the required scale factors of mainshock to achieve corresponding predefined damage states. The IDA is a parametric analysis method that has been proposed by Bertero [5] in 1977 to thoroughly estimate the structural performance under seismic loads. It involves a structure model subjecting to several ground motion records scaled to increasing levels of intensity, thus producing curves of response parameterized versus intensity level to analyze the influence of progressively enlarged seismic load on the nonlinear development of structures [6].

In this study, the 5% damped first mode spectral acceleration Sa(T1,5%) is selected as intensity measure (IM). Since the peak transient inter-story drift is directly related to the joint rotation, structural component failure and collapse prevention ability, so it is employed here as response parameters to identify the damage states in the mainshock and aftershock. The shear wall structure is subjected to gradually scaled of selected mainshock to generate the two selected damage states DS1 (0.5%), DS2 (1%) in CSW structure, and these two damage states correspond to immediate occupancy, life safety as specified in FEMA 356. In addition, 2% transient interstory drift is selected as collapse limit state according to the same specification, with the description that concrete shear wall structure reach this limit have following characters: major flexural and shear cracks and voids, sliding at joints, extensive crushing and buckling of reinforcement, failure around openings, severe boundary element damage, coupling beams shattered and virtually disintegrated.

After two particular damage states have been achieved, the aftershock IDA is applied to the mainshock-damage structures, respectively. To make sure the building returns to rest, an elapse time of 20s is added between mainshock and aftershock. The aftershock fragility curve can be calculated associated with the aftershock IDA results. The fragility function used in this study is derived from the lognormal cumulative distribution:

$$P(LS|S_a)=\Phi\left(\frac{\ln m_{D|S_a}-\ln m_C}{\beta_{D|S_a}}\right) \tag{1}$$

where LS represents the limit state, $P(LS|S_a)$ is the probability of a structure exceeding a certain limit state under an earthquake of intensity Sa, *D* is the engineering demand parameter, $\ln m_{D|S_a}$ is logarithmic mean of seismic demand *D* when ground motion intensity is *Sa*, $\ln m_C$ is logarithmic mean of seismic resistance *C*, equal to the logarithm of the median value m_C, $\beta_{D|S_a}$ is logarithmic standard deviation of seismic demand *D*.

The relation between engineering demand parameter D and IM (Sa) can be described as [7]:

$$\ln m_{D|S_a}=\beta_0+\beta_1\ln S_a \tag{2}$$

where β_0 and β_1 are regression coefficients

Combining the above relations to get the fragility function utilized in this study.

$$P(LS|S_a)=\Phi\left(\frac{\beta_1\ln S_a+\beta_0-\ln m_C}{\beta_{D|S_a}}\right) \tag{3}$$

3. Case study

3.1 Building model

The structure selected for a case study is a symmetric-plan 12-story RC shear wall that was developed with reference to ASCE 41-06. The detailed dimension is illustrated in Figure 1 which also depicts the layout of the floor plan and the elevation of the building structure. The shear core with coupling beams is assumed to resist the entire lateral load and the gravity framing system consists of slabs and columns that are proportioned to only resist the gravity load. Consequently, only the shear core was considered since ignoring the slab-column framing does not affect the response to any degree as studied in many literatures. However, the mass of the walls, floor slabs, columns, cladding, partitions and live load above these components are concluded in the seismic response analysis. The seismic masses used in this study are assigned as following: $2.5\times10^{6}kg$, $2.5\times10^{6}kg$ and $1.96\times10^{8}kgm$ in two orthogonal translational directions and torsional direction for each floor.

As shown in Figure 1, a total of 28 metallic CRBs are arranged in the building from story 2 to story 7, with 4 on each floor. The design of the metallic CRB should consider two parameters: yield displacement and yield strength. In order to achieve the purpose that the RCBs can reach maximum energy dissipation ability in frequent earthquakes while ensuring that stiffness and strength of replaceable component satisfied the seismic specification in China, the initial stiffness and yield displacement of the RCBs are defined as 450 kN/mm and 0.7mm, respectively.

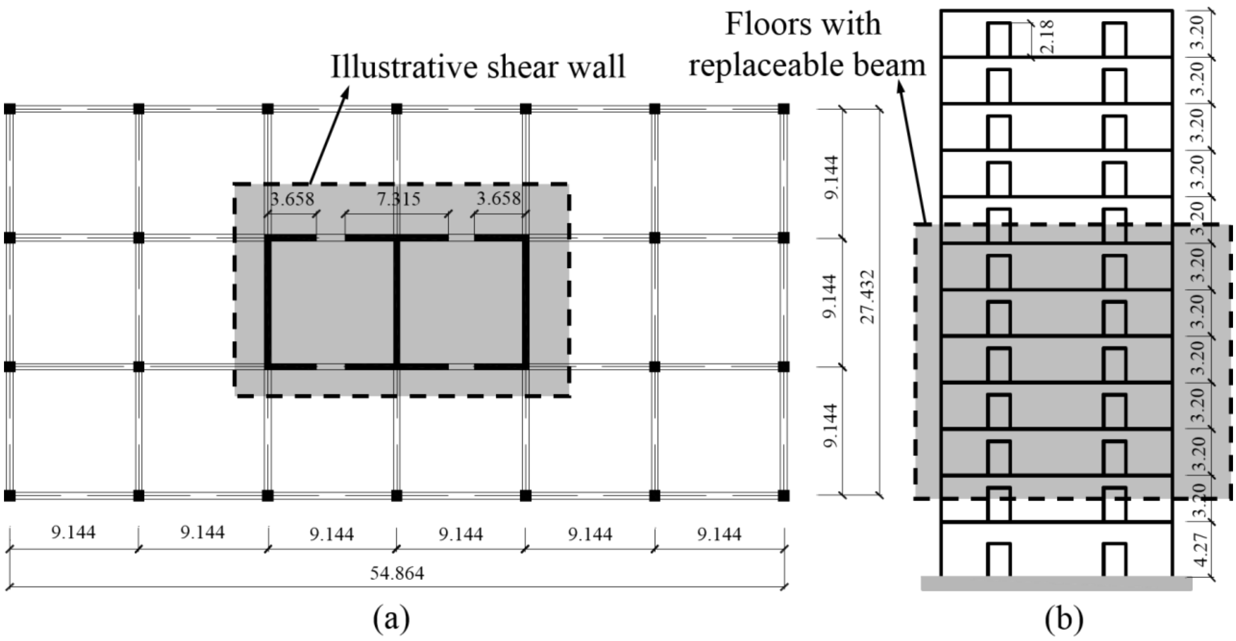

Fig.1 – Layout of the floor plan and the elevation of the building structure

A 3D structural analysis model is developed using the open-source finite element analysis platform OpenSees. The floor slab is modeled as a rigid diaphragm and the mass is lumped at each floor. The displacement-based fiber beam-column element is used to model the RC shear wall. The stress-strain behaviour of reinforced steel and concrete are represented using the uniaxial material steel01 and concrete01. Two elasticBeamColumn elements connected by a zero-length element is adopted to simulate the RCBs with a shear hinge in the middle. In addition, the fatigue material is utilized to account for the fatigue failure of metallic RCBs that may occur in the circumstance of mainshock-aftershock.

3.2 selection of ground motion sequences

Three earthquakes chosen from 22 far-field ground motion recommended by FEMA-P695 are selected to construct mainshock-aftershock sequences using the repeated method. The detailed information is shown in Table 1. Figure 2 shows the 5%-damped acceleration response spectrum of individual earthquakes and the fundamental period of the structure is indicated.

Table 1 – Ground motion records

NO.	ID	Earthquake			Recording station	
		Magnitude	Year	Name	Name	Owner
1	1	6.7	1994	Northridge	Beverly Hills - Mulhol	USC
2	3	7.1	1999	Duzce, Turkey	Bolu	ERD
3	6	6.5	1979	Imperial Valley	Delta	UNAMUCSD
4	8	6.9	1995	Kobe, Japan	Shin - Osaka	CUE
5	11	7.3	1992	Kocaell, Turkey	Yermo Fire Station	CDMG

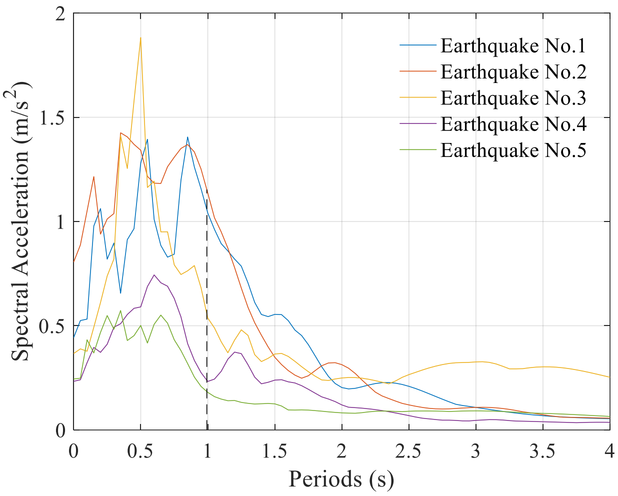

Fig.2 – The spectrum of the earthquake records

The basic assumption of using the repeated method is that the aftershock sequence has a similar frequency characteristic with the mainshock. The approach to constructing the repeated mainshock-aftershock based on the No.1 Northridge earthquake is shown in Figure 3. Two identical recorded ground motions are connected by a time gap of 20 seconds to make sure the stabilization of the system under free vibration.

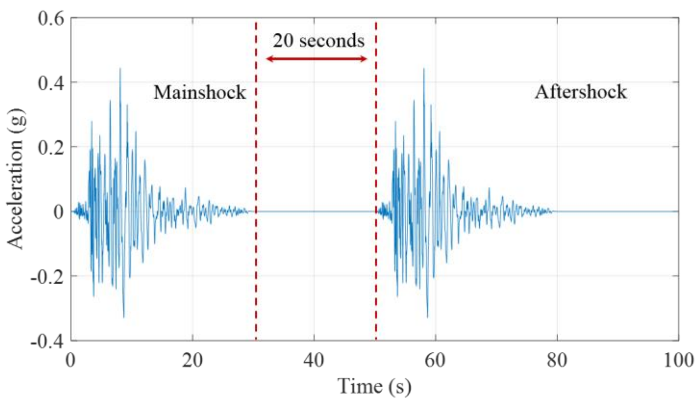

Fig.3 – Acceleration time history using the repeated method of No.1 earthquake

3.3 Aftershock fragility analysis based on transient interstory drift

The mainshock IDA analysis is conducted on the shear wall structure firstly to determine the scale factor to achieve the damage state DS1 and DS2, as well as the collapse capacity of the undamaged structure. The blue line in Figure 4 shows the median response of the mainshock-only IDA curve under five earthquake records. The collapse capacity of the undamaged structure is 1.75g.

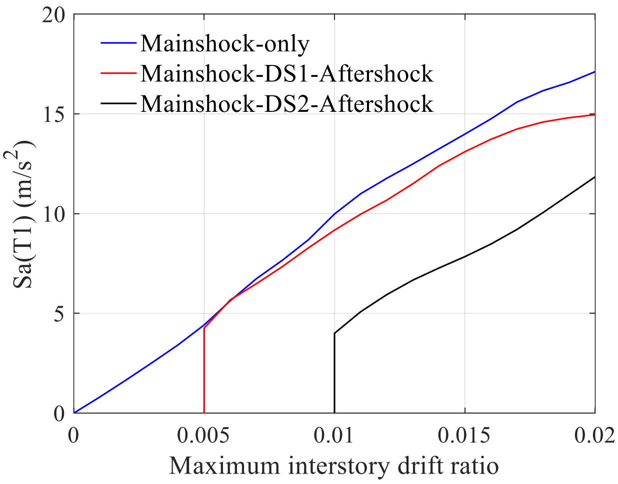

Fig.4 – Median mainshock IDA curve and aftershock IDA curves at two damage states

To develop the aftershock fragility function, the corresponding aftershock IDA is conducted on two mainshock-damaged structures using increasing intensity measure of aftershock while keeping the scale of the mainshock as constant until the system reaches collapse limit state or instability state, the aftershock IDA curve is illustrated in Figure 4 with red and black line corresponding to Mainshock-damage-DS1 structure and Mainshock-damaged-DS2 structure. The collapse capacity has decreased to 1.53g and 1.21g. It can be noted that the gap between the red line and the blue line is gradually increasing as the intensity measure increase, which suggests the mainshock dominates the behaviour when the aftershock intensity is smaller. When the structure reached the DS2 damage state in the mainshock, a slight increase in ground intensity will lead to a larger interstory drift than undamaged structure, as depicted by the black line.

According to the fragility calculation method listed in section 2, the aftershock fragility curve is drawn in Figure 5. The median value and logarithmic standard deviations of the seismic capacity to resist collapse failure is selected as 0.0186 and 0.45 [8]. As can be seen in the figure, the corresponding fragility curve becomes higher as the mainshock-damage gets more severe, which indicates a higher collapse failure. Comparing the red line with the blue line, the collapse probability of the mainshock-damaged-DS1 structure is slightly higher when the aftershock intensity is small, and the differences are increasing as the intensity of the aftershock increase. That suggested the structural damage state DS1 is small so it has little influence on structural collapse capacity. However, the collapse probability of the mainshock-damaged-DS2 structure represented by the black line is significantly higher than the undamaged structure even the aftershock intensity is small.

Table 2 lists the collapse capacity when the failure probability of the structures in three mainshock damage states is 50%. The collapse capacity has decreased from 1.8036g to 1.5736g and 1.0806 when the structure reached DS1 and DS2 in mainshock, respectively, indicating the structural collapse prevention ability decreased obviously. Therefore, it is necessary to consider the influence of aftershock on the fatigue failure of the replaceable component and the overall structural performance if the replacement process can be completed immediately.

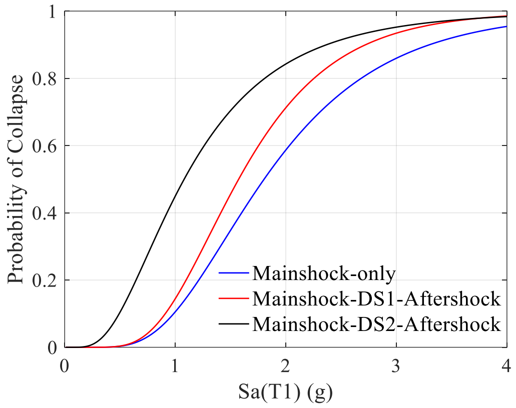

Fig.5 – Fragility curves for undamaged structure and two mainshock-damaged structure

Table 2 – Collapse capacity of the shear wall structure in three mainshock-damage state

	Mainshock-only	Mainshock-DS1-Aftershock	Mainshock-DS2-Aftershock
Sa(T1,5%) (g)	1.8036	1.5736	1.0806
Reduction	/	12.8%	40.1%

4. Conclusion

In this study, the aftershock fragility performance of CSW structure with RCBs considering the fatigue issue is investigated. A case study building is modeled with several RCBs on certain floors. The fatigue material is used to account for the fatigue failure of the RCBs. The mainshock IDA analysis is conducted to determine the scale factor to simulate the mainshock-damaged structure with two damage states as specified in FEMA 356. Since the sequence earthquake records are constructed by repeated methods, the mainshock IDA analysis also provides the collapse capacity of the undamaged structure. Results show that the collapse capacity of the overall structure reached DS1 in the mainshock decrease slightly compared to the undamaged structure. However, the collapse probability of shear wall structure reached DS2 in mainshock is significantly higher than the undamaged structure even the aftershock intensity is small. It is necessary to consider the influence of aftershock on the fatigue failure of the replaceable component in the design process.

5. Acknowledgements

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