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SEISMIC COLLAPSE MARGIN OF LINKED COLUMN STEEL BRACED FRAMES WITH LIFT COLUMN BASES

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Abstract

Linked column braced frame（LCBF）with lift column base is a new type of replaceable component structural system. Under the action of frequent earthquake and basic intensity earthquake, the foot of linked column can move upward, and the two ends of the link beam are staggered. The link beams enter the seismic energy dissipation stage preferentially, while the main frame members remain elastic state or rarely reach the yield state. After the earthquake, only replacing the link beam can quickly restore the structure's function, and will not have an obvious impact on the lateral stiffness, bearing capacity and ductility of the structure. Under the action of rare earthquake, the lifting height of foot reaches the limit, which improves the bearing capacity of the structure. Since the foot of the column can move upward and link beam enters plastic state, the dynamic response of the structure is reduced and the direct collapse of the building structure can be avoided. In order to evaluate the seismic collapse margin of LCBF, the collapse criteria of multi-story steel structures are summarized. Pushover analysis of LCBF is carried out to determine the displacement limit according to the overall deformation of the structure and the damage degree of the components, and then a criterion to judge the collapse of this structure is proposed. The anti-collapse capacity of LCBF structures is evaluated by CMR（Collapse Margin Ratio）method, and then obtained the corresponding collapse margin ratio and the collapse possibilities. The analysis results show that the collapse criterion is reasonable for LCBF when story drift reaches 2.6%, and collapse margin capacity of LCBF structure is strong. The collapse margin ration（CMR）is higher than the acceptable standard, and the collapse probability under the action of rare earthquake is less than 10%, satisfying CMR stipulated by FEMA695. Under the same strength level, the seismic response of LCBF is different under different ground motions. With the increase of earthquake strength level, the story drift ratio and the base shear of the structure will also increase.

Keywords: linked column steel braced structure; lifting column base; incremental dynamic analysis

1. Introduction

Conventional seismic design method can help structures provide acceptable life-safety performance, and avoid collapse of buildings in short time. In this case, inhabitants can earn valuable time to escape from the shaking buildings under the earthquake. However, earthquake still causes a great deal of economic loss and huge social panic, for example many seriously damaged structures need a reconstruction, and those relatively slightly damaged buildings requires high cost of retrofit and long construction time. Therefore, there is a need for systems that not only protect the lives of people, but are also easily repaired with less material cost, labor cost and construction time.

Linked column frame(LCF) system is a new lateral load resisting structure[1]. This new system consists of short link beams between dual columns shown in Fig 1. Under the action of earthquakes, the link beams will reach yield stage preferentially to other structural elements, and inelastic deformation or damage mainly focuses on replaceable links. The link beams in LCF is similarly to that in eccentrically braced frames, and it is easy to replace the links after a major earthquake to achieve the goal of repairing a building in a short period. In recent years, a number of experimental investigations and numerical studies have been conducted on LCF. The seismic performance can be divided into three levels by lateral deformation characteristics: elastic, rapid return to occupancy, and collapse prevention. The inelastic behavior of the system can be achieved without damaging the gravity moment frame for the range between 0.43% to 1.7%[2]. An experiment on steel moment frames with energy dissipation beams was conducted, and results indicated that the proposed system shows some features including the controllable yielding sequence, the development of energy dissipation mode, the transformation of hysteretic behavior and small residual story drift[3].

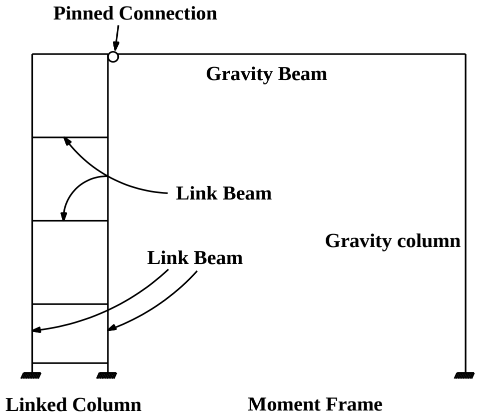

Fig.1-Linked column frame system

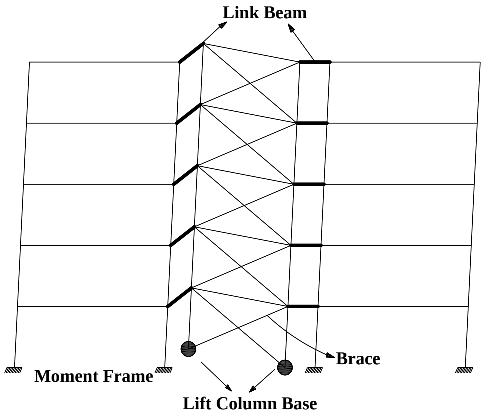

Fig.2-LCBF with lift column bases

However, when LCF system is subjected to later loading, forces are resisted only through deformations of the beams and columns without any other structural components, leading to low initial stiffness. In order to expand the application of LCF in highly seismic regions, linked column braced frame (LBCF) was proposed, which means LCF is coupled with concentrically braced frames to increase the stiffness[4].

Linked column bottom bases are generally fixed to a foundation, and a disadvantage is that the damage degree of link beams is not significant. Because the plastic response of link beams caused by flexural and bending deformation of adjacent columns is limited. By releasing the restraints between upper structures and foundations, only a compression capacity exists without a tension compression capacity, making the structure rock under the action of earthquake[5]. The study showed the rocking structure decreases ductile design demand of upper structures and save construction cost. Based on this rocking structure theory, in order to increase the plastic behavior of link beams and improve seismic performance, the connections between linked columns bases and foundations can be loosened, namely linked column steel braced frames with a lift column base, shown in Fig.2.

This paper introduces the definition of LCBF with a lift column base and describe the failure mode under the ground motions firstly. Then the collapse criteria of multi-story steel structures are summarized, pushover analysis of LCBF is carried out to determine the displacement limit according to the overall deformation of the structure and the damage degree of the components, and then a criterion to judge the collapse of this structure is proposed. Finally the anti-collapse capacity of LCBF structures is evaluated by CMR（Collapse Margin Ratio）method.

2. Linked Column Steel Braced Frame

A typical linked column steel braced frame (LCBF)is shown in Fig.2, and it consists of linked columns interconnected with replaceable link beams, a flexible secondary moment frame and a lift column base. The linked columns are unable to bear the tension, so if the overturning moment of buildings is larger than gravity resisting moment of that under strong earthquakes, one linked column base is likely to move upward, increasing the plastic deformation of link beams connected to close columns. The other column base will function as a hinge, and the structure rotates around this hinge, which is similar to rocking structure. Due to the release in constraints between the column base and the foundation, a certain lifting distance occurs at the bottom of the structure, allowing the sway movement of a structure, and reduce the dynamic response of main moment frame.

Fig.3 shows a practical lift column base and it is comprised of disc springs, an antifriction bearing, bolts and steel plates. The antifriction bearing is welded under the bottom steel plate, and disc springs are set above the steel plate through bolts. For example, under the action of lateral loads in the right direction, the left linked column base can move upward or lift, it lack the ability of bearing the tension. The left springs are compressed and the left antifriction will rise with bottom steel plates. Then right column base can bear compression and functions as a hinge, so the whole structure rotates around the hinge, highly similar to rocking structure. In addition, the bolt holes on the base plate need to be designed as short slot holes. Because short slot holes can provide a little more space for bolts, helping column base avoid being stuck by inclined bolts when opposite direction loads are applied on the frame.

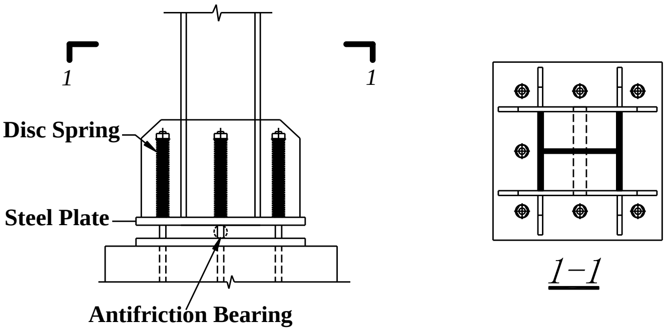

Fig.3 Lift column base with disc springs

3. A Collapse Criterion for LCBF

Previous earthquakes have shown that collapses of structures can cause significant casualties and economic losses. However, LCBF with lift column base is a new replaceable component structural system, and current studies about the anti-collapse capacity of LCBF structures are insufficient. In order to evaluate the anti-collapse capacity of LCBF system, a reasonable criterion for judging the collapse state of LCBF needs to be proposed firstly.

3.1 comparisons of criteria for steel moment frames

Several design codes have presented many criteria of different performance levels, including collapse prevention and immediate occupancy. In the code FEMA-351, the collapse prevention structural performance level is defined as the post-earthquake damage state in which the structure is on the verge of experiencing partial or total collapse. Substantial damage to the structure has occurred, potentially including significant degradation in the stiffness and strength of the lateral force resisting system, large permanent lateral deformation of the structure, and to a more limited extent, degradation in the vertical-load carrying capacity. However, all significant components of the gravity-load-resisting system must continue to carry their gravity-load demands. The structure may not be technically or economically practical to repair and is not safe for re-occupancy; aftershock activity could credibly induce collapse. Table 1 relates these structural performance levels to the limiting damage states for common framing elements of steel moment-frame buildings.

Table 1 Structure Performance Level in FEMA351

Elements	**Structural Performance Levels**	
	Collapse Prevention	Immediate Occupancy
Girder	Extensive distortion; local yielding and buckling. A few girders may experience partial fractures	Minor local yielding and buckling at a few places.
Column	Moderate distortion; some columns experience yielding. Some local buckling of	No observable damage or distortion

	flanges	
Beam-Column Connections	Many fractures with some connections experiencing near total loss of capacity	Less than 10% of connections fractured on any one floor; minor yielding at other connections
Panel Zone	Extensive distortion	Minor distortion
Column Splice	No fractures	No yielding
Base Plate	Extensive yielding of anchor bolts and base plate	No observable damage or distortion
Inter-story Drift	Large permanent	Less than 1% permanent

In code FEMA-356, the structural performance level of a building shall be selected from four discrete structural performance levels and two intermediate structural performance ranges. Structural performance level S-5, collapse prevention, shall be defined as the post-earthquake damage state that includes damage to structural components such that the structure continues to support gravity loads but retains no margin against collapse in compliance with the acceptance criteria specified in this standard. Table 2 shows the descriptions of different lateral load resisting systems, including steel moment frames and braced steel frames, for collapse prevention state. The table indicates that the deformation capacity of steel moment frames is stronger than that of braced steel frames.

Table 2 Collapse Prevention Level in FEMA356

Elements	Collapse Prevention S-5
Steel Moment Frames	Extensive distortion of beams and column panels. Many fractures at moment connections, but shear connections remain intact. 5% transient or permanent story drift ratio.
Braced Steel Frames	Extensive yielding and buckling of braces. Many braces and their connections may fail. 2% transient or permanent story drift ratio.

Both in the Code for Seismic Design of Buildings and the Design Code of Collapse Resistance of Building Structures, the limit value of inter-story drift ratio of multi-story and high-rise steel structure is 1/50, and several seismic fortification measures are essential for meeting deformation requirements or avoiding collapses. The specification does not consider the differences in the lateral force performance between the steel moment frame and the braced steel frame. So it is unreasonable to define the story drift ratio 1/50 as the collapse standard of multi-story and high-rise steel structures generally.

In General Rule for Performance-Based Seismic Design of Buildings, buildings are divided into four categories: I, II, III and IV according to the architectural function. The specification for life safety level of II buildings is that the basic function is affected significantly, severe damage in the main structure, and non-structural members may fall, but will not hurt people. This design code also presents the limit value of elastic-plastic story drift ratio for steel moment frames and braced steel frames: 0.029 and 0.020 rad separately. There is no clear requirement to prevent the collapse for these two systems, but it is reasonable to infer that the corresponding value of collapse performance level for two steel systems is higher than 0.029 and 0.020 rad separately.

A study on the collapse criteria for steel moment frames was carried, and more than 20 experimental investigations data was collected and summarized[6]. Parameter estimation method was used to determine the maximum story drift ratio of collapse level of steel moment frames,0.035rad.

From above discussions, it is found that different code designs have defined several collapse performance level criteria for steel lateral load resisting systems from many aspects, including a description of collapse state and the limited story drift ratio. The story drift ratio for steel moment frames and braced

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steel frames should be 0.02 and 0.035rad separately. The corresponding value for LCBF should vary between 0.02 and 0.035 rad, because this new component replaceable structure can be considered as braced steel frames with a lift column base, and the deformation capacity is weaker than braced steel frames but stronger than pure steel moment frames.

3.2 A collapse criterion for LCBF

In order to propose an appropriate collapse criterion for LCBF, pushover analysis of LCBF is carried out to determine the displacement limit according to the overall deformation of the structure and the damage degree of the components, by using software ABAQUS. A 3-span and 9-story frame finite element model with two dimensions is shown in Fig.4 and Fig.5, and Table 3 shows the structural members sizes. Shell elements S4R with reduced integration schemes are adopted in static analyses for both accuracy and efficiency. Given the regularity of nine story buildings, the lateral load distribution throughout the height adopts displacement-control mode based on equivalent lateral forces. The gravity loads include the dead load and 50% of the design live load as defined by Load Code for Building Structures. The middle span is set with lift bases, stimulated by Spring 2 elements, and the other column bases are restrained in all directions, similar to fixed connections with foundations. The load-deformation capacity of Spring 2 elements totally depends on that of discussed disc springs and bolts shown in Fig.6.

Table 3 Structural Members Size

Floor	KBZ	KZZ	ZKZ	ZC	KL1	KL2	LL
1~3	H650×500×24×30	H550×450×22×28	H300×300×16×20	H200×200×16×18	H650×350×12×18	H400×200×10×12	H350×150×8×12
4~6	H550×450×22×28	H450×450×20×24	H250×200×14×16	H160×160×12×14	H650×350×12×18	H400×200×10×12	H350×150×8×12
7~10	H450×400×20×26	H400×400×18×20	H200×200×12×14	H120×100×10×12	H650×350×12×18	H400×200×10×12	H350×150×8×12

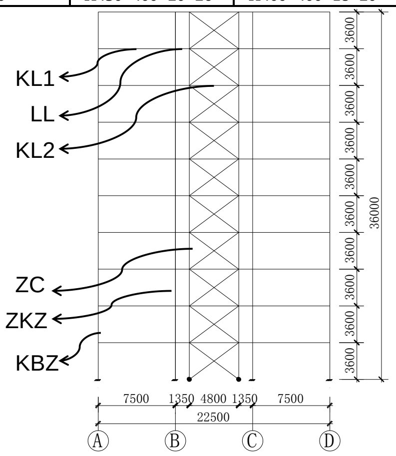

Fig.4-9-story and 3-span LCBF

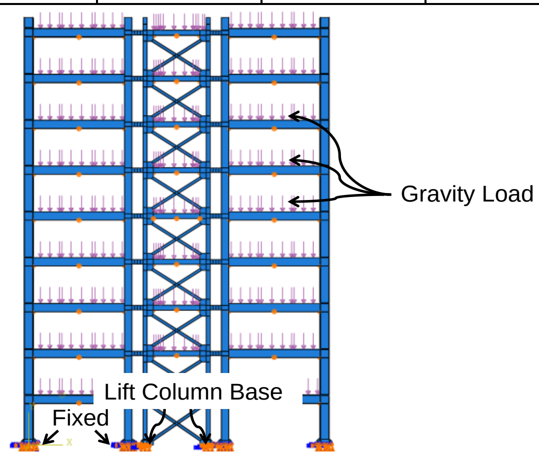

Fig.5-Finite element model

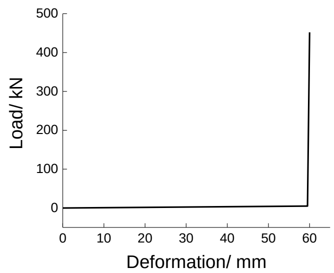

Fig.6-Load-deformation curve of disc springs

The stimulation results of the FE model in different story drift ratios are shown in Fig.7, 8 and 9, where the internal force transfer, yield sequences, and damage degrees of components are revealed.

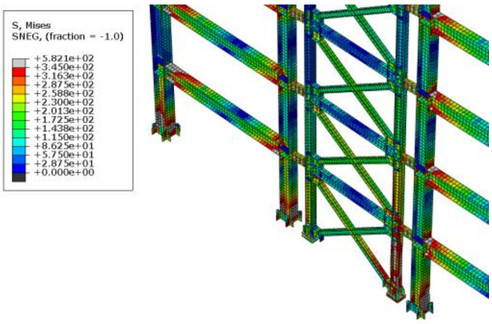

Fig.7-Story drift ratio 2%

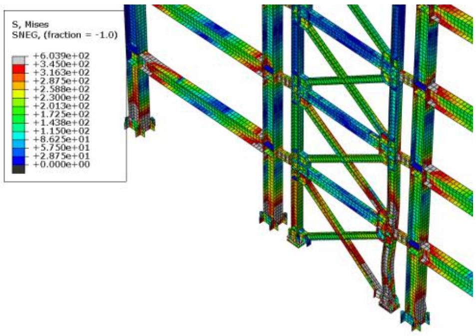

Fig.8-Story drift ratio 2.6%

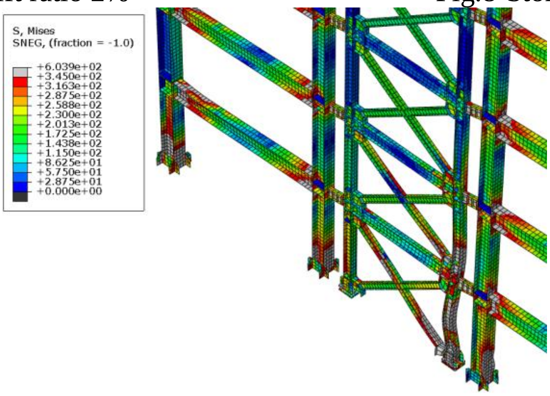

Fig.9-Story drift ratio 3.5%

Table 4 shows damage state descriptions of structural elements, including beams, columns, braces and connections, under different roof drift ratios: 2%, 2.6% and 3.5%. According to the compparsions between analysis results in Table 4 and above definitions of prevention collapse performance level, this paper defines roof drift ratio 2.6% as the collapse criterion for LCBF system.

Table 4 Pushover Analysis Results

Elements	**Roof Drift Ratio**		
	2%	2.6%	3.5%
Link Beams	Plastic hinges at the end plates; 90% yielding area of web	Further development in plastic areas. Partial flange entering into plastic state	Extensive yielding and buckling.
Frame Beams	50% beams yielding at ends. Large plastic deformation of the flange	80% beams yielding at ends. Plastic zone spreading into the middle section of beams	Extensive yielding and partial beam flange buckling.
Frame Columns	Large plastic deformation of the column flange, spreading to the web; the obvious flange buckling.	Increase in column flange buckling deformation. At least 70% yielding area of the column base web	Extensive yielding in column base, with the yielding length of 20% floor height
Linked Columns	Almost all of the bottom columns yield; no buckling deformation.	Bottom column starting to buck.	Extensive buckling of bottom column.
Braces	Maximum internal force in bottom braces; braces yielding from the first to the fourth floor nearly	50% area of braces in the first floor reaching yield stress, as well as 30% of joints between braces and close connections.	Extensive yielding and buckling of braces in the first and the second floor.
Beam-to-Column	70% area of the panel zones yielding; nearly the same	Yielding zones spreading further. Connections experiencing near	90% yielding area of the panel zones and

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connection	plastic deformation degree of connections in each floor	total loss of capacity.	extensive distortion

4. Collapse Margin Ratio for LCBF with Lift Column Bases

Incremental Dynamic Analysis (IDA) is a parametric analysis method that has recently emerged in several different forms to estimate more thoroughly structural performance under seismic loads. It involves subjecting a structural model to more ground motion records, each scaled to multiple levels of intensity, thus producing more curves of response parameterized versus intensity level. The relation curve of damage index DM (damage measures) versus seismic intensity index IM (intensity measures) is generated, i.e. IDA curve. Finally, the collapse reserve capacity of the structure is evaluated by the behavior points on the IDA curve.

4.1 Finite element model and ground motion records

In order to save calculation cost, only link beams adopt shell elements, column base spring adopts spring2 element, and other structural members adopt wire elements, shown in Fig.10. Because the problem of inconsistent freedom degrees between shell element and beam element occurs, MPC constraint is introduced, allowing to impose constraints between different freedom degrees in Fig.11.

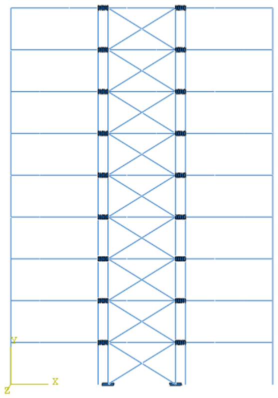


Fig.10-Finite element model

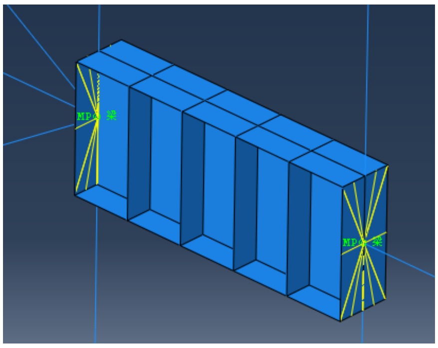
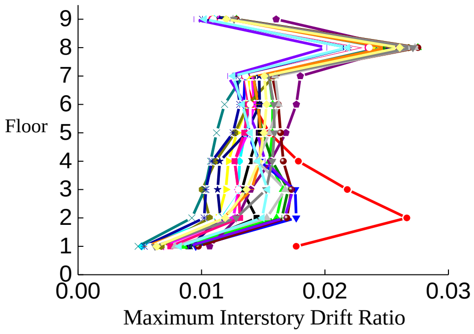


Fig.11-MPC constraints

The incremental dynamic analysis (IDA) method is essentially an elastic-plastic analysis of seismic time history, so a large number of appropriate ground motion records is necessary. Due to the difference between earthquake waves, the correct selection of ground motion records is the premise of obtaining ideal calculation results. In FEMA-P695 the ground motion record selection criteria are listed, including source magnitude, source type, site conditions, site- resource, number of records per event, strongest ground motion records, strong motion instrument capability, and strong-motion instrument location. Referring to these criteria, this paper selects 22 ground motions recommended in FEMA-695.

4.2 Seismic responses and collapse margin ratio

The peak value of 22 earthquake waves is continuously increased, and is input to the FE model, until the maximum interstory drift ratio reaches a collapse criterion for LCBF system 0.026 rad. Fig.12 shows the maximum interstory drift ratio of FE model under 22 ground motions, and Fig.12 shows the average maximum interstory drift ratio of 22 ground motions. It can be seen from the figure that the weak layer occurs in the second floor once, and occurs in the eighth floor for the other 21 ground motions. Even the intensity of seismic weaves is the same, the dynamic response of LCBF is different for different ground motions.



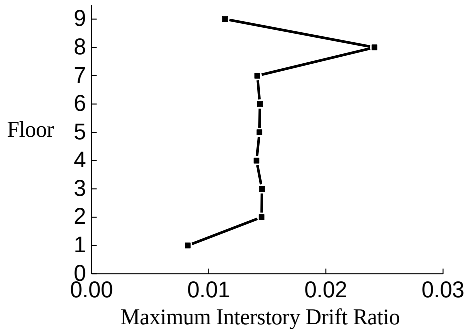
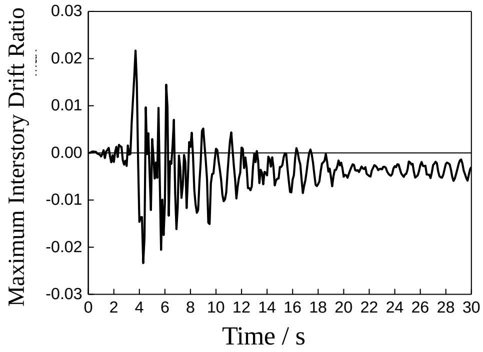


Fig.12-Maximum interstory drift ratio　　Fig.13-Average value of maximum interstory drift ratio

Fig.14 shows maximum interstory drift ratio-time curve and Fig .15 shows base shear force-time curve of the eighth floor for the condition of collapse level. It can be seen that the interstory drift ratio cannot recover back to zero value, which indicates that structure enters the plastic deformation stage. With the increase of earthquake intensity level, the story drift ratio and the base shear of the structure will also increase.



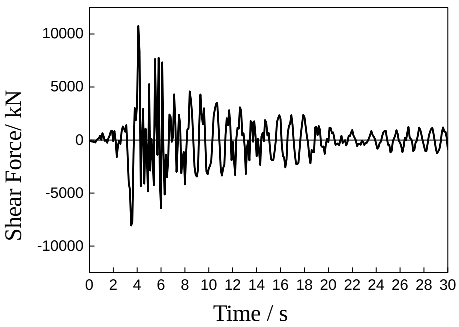


Fig.14-Drift ratio-time curve　　Fig.15-Shear force-time curve

The IDA results are shown in Fig.16, and it considers maximum interstory drift as damage index DM (damage measures) and considers $S_a(T_1;5\%)$(the ξ = 5% damped Spectral Acceleration at the structure's first-mode period) as intensity index IM (intensity measures). Referring to the equation

$$CMR = \frac{S_{CT}}{S_{MT}} \tag{1}$$

CMR is the primary parameter used to characterize the collapse safety of the structure, S_{CT} is the median collapse intensity, corresponding to a 50% probability of collapse and S_{MT} is the maximum considered earthquake ground motion intensity. For the 3-span and 9-floor LCBF, CMR=1.257g/0.353g=3.56.

In order to evaluate the seismic performance, this paper adjusts the total system uncertainty, and incorporate the effects in the collapse assessment process. The following sources of uncertainty are considered: Record-to-Record Uncertainty, Design Requirements Uncertainty, Test Data Uncertainty and Modeling Uncertainty. The total system uncertainty is 0.75 and the minimum acceptable standard of adjusted collapse margin ratio ACMR20%=1.88. This is less than the above calculated results CMR3.56, satisfying the seismic collapse safety requirements.

Using collapse data from IDA results, a collapse fragility curve can be defined through a cumulative distribution function, which relates the ground motion intensity to the probability of collapse. The maximum considered earthquake ground motion intensity S_{MT} is 0.353g, and the corresponding collapse probability is 2.06% from the Fig.17, less than 10%. It shows that this model is unlikely to collapse under strong ground motions, and can achieve the expected target of no collapsing in the strong earthquake.

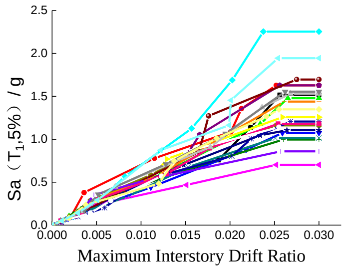


Fig.16-22 IDA curves

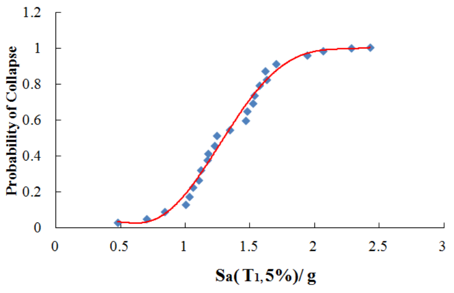


Fig.17- Vulnerability curve

5. Conclusion

LCBF with a lift column base is a new component replaceable load resisting system. The lift column base with disc springs can function as a hinge under seismic action, making the link beams reach yielding stress and undergo inelastic deformation preferentially, similar to rocking structures. An appropriate criterion of judging the collapse state is defined as the interstory drift ratio 2.6%. This criterion is used to evaluate the anti-collapse capacity of LCBF system by IDA method, it is found that collapse margin capacity of LCBF structure is strong, higher than the acceptable standard, and the collapse probability under the action of rare earthquakes is less than 10%. In addition, it is necessary to consider how other design parameters, including height-span ratio and the length of link beams, affect the collapse prevention performance in the future work.

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