Damage-control Seismic Design of Moment-resisting RC Frame Buildings

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Abstract

In this study, a new seismic design method for directly and efficiently controlling damage to structural and non-structural components of moment-resisting reinforced concrete (RC) frame buildings is developed. The global seismic damage index, which considers the effect of accumulative damage, and the inter-story drift ratio are applied as the performance indexes for structural and non-structural components, respectively. As the base of this method, the seismic damage model, the determination of the seismic strength demand, and seismic performance objectives are first introduced. Then, the detailed design procedures of this method are presented. Finally, as a case study, the seismic design of a typical 6-story moment-resisting RC frame building structure is provided following this new method. The seismic performance of the designed structure is evaluated under different levels of earthquake by conducting a nonlinear time history analysis. The results indicate that the predetermined seismic performance objectives of the structure as designed in accordance with the proposed method can be achieved with great efficiency.

Keywords: moment-resisting RC frame; damage control; performance-based seismic design; performance index

1. Introduction

The performance-based seismic design (PBSD) approach was first introduced in the early 1990s as a viable tool for designing structures with certain seismic performance under different earthquake severities. In implementing the PBSD approach, selecting a reasonable performance index is crucial. So far, the most widely applied seismic performance index is displacement or deformation. The displacement-based seismic design (DBSD) method for RC structures was first proposed by Qi and Moehle (1991). This method limits structural damage by controlling the displacement responses. Later, Priestley (2000) systematically discussed several issues in utilizing DBSD and indicated that the design process should be guided by using concrete strain as an important parameter. As one of the principal deformation indexes, the inter-story drift ratio, which is closely related to the damage state of structural and non-structural components in building structures, is generally applied in DBSD. Several previous studies (Gunturi and Shah, 1992; Pao, 2002) applied the inter-story drift ratio as the seismic performance index and investigated the relationship between the inter-story drift ratio and the damage states of non-structural components. However, the cumulative damage resulting from inelastic cycles has significant effects on the damage state of a structure. The performance index of displacement is not reliable enough to determine the strength demand of a structure (Warnitchai and Panyakapo, 1999). On the other hand, the seismic damage index, which includes the cumulative damage effect, has been regarded as one of the most efficient tools for controlling damage to structural components. A more reliable damage-control seismic design should be based on simultaneous and reasonable considerations of damage to structural and non-structural components.

In this study, a new damage-control seismic design method is developed, that enables us to more directly and efficiently control the seismic damage of structural and non-structural components for moment-resisting RC frame buildings. Two performance indexes, the global seismic damage index and the inter-story drift ratio, are applied to control the damage of structural and non-structural components, respectively. The detailed design procedures are introduced. Then the design of a 6-story moment-resisting RC frame building structure using this method is provided as a case study.

2. Seismic Damage Model

One of the best known and most widely used cumulative damage models that are appropriate for
RC members is the Park-Ang model (Park and Ang, 1985), based on a linear combination of the damage due to excessive deformation and cumulative plastic energy. A non-normalization problem exists in this model, that is, the damage index will be greater than 1.0 when the structure is loaded monotonically up to the ultimate limit state. In this study, a modified Park-Ang damage model with higher precision and smaller scatter, proposed by the authors (Jiang et al., 2011) to eliminate the non-normalization problem that existed in the original Park-Ang damage model, is thus used. The expression of the modified Park-Ang damage model is shown as follows:

\[ D = (1 - \beta) \frac{\mu_u - \mu_v}{\mu_u} + \beta \frac{1}{F} \int \delta (\mu_u - 1) \, dE \]  

(1)

where \( \mu_u \) and \( \mu_v \) are the ultimate displacement ductility factors under monotonic loading and the maximum deformation ductility factor experienced in the time history under earthquake ground motions, respectively; \( F \) and \( \delta \) are the yield strength and yield displacement of the system, respectively; \( \beta \) is the combination coefficient, which is taken as 0.1 here for flexure-dominant RC structures with good deformation capacity (Chen et al., 2010); and \( dE \) is the incremental hysteretic energy. The seismic damage suffered by the two ends of the member may be different. The seismic damage index of the structural member should be taken as the larger one of its two ends.

The seismic damage index of a story is determined by the weighted average of all the relevant members in the story, expressed as

\[ D_i = \sum_{j=1}^{l} \lambda_{ij} D^{(i)}_j \]  

(2)

where \( D_i \) is the seismic damage index of the \( i \)th story; \( l \) is the total number of the members in the \( i \)th story; and \( D^{(i)}_j \) and \( \lambda_{ij} \) are the seismic damage index and the weighting factor of the \( j \)th member in the \( i \)th story, respectively. \( D^{(i)}_j \) is determined by Eq. 1, and \( \lambda_{ij} \) is expressed as

\[ \lambda_{ij} = \frac{D^{(i)}_j}{\sum_{k=1}^{l} D^{(i)}_k} \]  

(3)

The global seismic damage index of the entire structure is taken as the maximum of the damage indexes of all stories, that is,

\[ D_G = \{D_1, D_2, \ldots, D_i, \ldots, D_n\} \]  

(4)

where \( D_g \) is the seismic damage index of the structure; and \( n \) is the total number of stories.

3. Seismic Strength Demand

The constant-damage yield strength spectrum (CDYSS), which is defined as the ratio of the yield strength of a single-degree-of-freedom (SDOF) system to the product of the mass and the elastic acceleration spectrum, is used to reflect the yield strength demand of a SDOF system of varying natural vibration period with the target constant damage index and specified ductility capacity. The CDYSS derived by the authors (Jiang et al., 2012a) is employed here. The expressions for CDYSS are as follows

\[ C_r = A + \frac{B}{T_w^c} \]  

(5)

\[ A = a_1 \cdot (\mu_u D_c)^{c_1} \]  

(6)

\[ B = b_1 \cdot (\mu_u D_c)^{c_2} \]  

(7)

\[ C = c_1 \cdot e^{-\frac{c_2}{\mu_u D_c}} \]  

(8)

where \( T_w \) is the natural vibration period; and \( a_1, a_2, b_1, b_2, c_1, \) and \( c_2 \) are constant coefficients related to the category of site soil condition.

4. Seismic Performance Objectives

Seismic performance objectives could be defined as the coupling of expected performance levels with expected levels of seismic ground motions. Four damage levels, fully operational, operational, repairable and collapse prevention, are considered in this study. The global seismic damage index and the maximum inter-story drift ratio, representing the damage levels of the structural and non-structural components respectively, are used as the indicators of performance level. The corresponding limit values of the performance indexes for all performance levels, which were determined based on statistical analyses of two performance indexes that attained the individual performance level, are shown in Table 1. (Chen, 2010; Jiang et al., 2012b). Four levels of seismic hazards, minor or frequent earthquake with the exceeding probability of 63.2% in 50 years (50 year return period), moderate or basic earthquake with the exceeding probability of 10% in 50 years (475 year return period), strong or rare earthquake with the exceeding probability of 2% in 50 years (2475 year return period), are considered. Compared to the current Chinese seismic design code, one additional level, the rare earthquake with a 975 year return period, is used.

Table 1. Limit Values for Different Performance Levels

| Performance Level       | Global Damage Index | Inter-story Drift Ratio |
|-------------------------|---------------------|-------------------------|
| Fully Operational       | 0.05                | 1/550                   |
| Operational             | 0.20                | 1/250                   |
| Repairable              | 0.40                | 1/100                   |
| Collapse Prevention     | 0.80                | 1/50                    |

According to Chinese standard (CMC, 2008), buildings can be classified into four types, A, B, C, and...
D, according to their importance and the consequences of their failure. Four grades of seismic performance objectives are proposed according to the four types of buildings as shown in Table 2. Many combined factors, including construction cost, importance of the building, expected earthquake loss (property and life), etc., could be taken into consideration when selecting the seismic performance objectives. For Example, Grade C can be used for ordinary buildings of Type C.

5. Design Procedures

As mentioned above, two performance indexes, the global seismic damage index and the inter-story drift ratio, are employed to control seismic damage of structural and non-structural components in RC frame buildings. The procedure for damage-control seismic design of RC frame buildings is as follows:

1. Perform preliminary design and determine seismic performance objectives

After the preliminary design is completed, the basic configuration and structural layout are selected, the initial parameters are input, and the seismic performance objectives are determined considering many combined factors.

2. Design structural components for required strength under frequent earthquakes by the current conventional strength-based method

Seismic effects under frequent earthquakes and the effects of other actions are determined based on linear-elastic behavior. The dimensions and steel reinforcement of structural components are derived by following the conventional strength-based method based on the current Chinese seismic design code (CMC, 2010). The general method for determining the seismic effects is the modal response spectrum analysis using elastic design spectra.

3. Transform a MDOF system into an equivalent SDOF system

The dynamic characteristics of the structure, such as the natural vibration periods and modes, are obtained, and then the original multi-degree-of-freedom (MDOF) system is transformed into an equivalent SDOF system by using normal equivalent principles.

4. Perform pushover analysis

The base shear versus top displacement relationship curve is obtained, and then the yield strength and the ultimate displacement ductility factor under monotonic loading are derived. The method for determining the yield and ultimate state from the base shear versus top displacement relationship curve is shown in Fig.1.

5. Check the inter-story drift ratio

Using the capacity spectrum method proposed in ATC-40 (1996), the performance points for each design earthquake level can be found by iterative trial calculations. The inter-story drift responses at the performance points are checked against the limit values that correspond with the selected performance objectives. The steel reinforcement should be adjusted if the requirement could not be met. Then, the process should be repeated from Step 4. The iteration should be complete until the limit is satisfied.

6. Check yield strength

By Eq. 5, the yield strength demand of an equivalent SDOF system can be determined. For the original MDOF system, the yield strength demand is derived by the following equation

\[ F_y^r = \Gamma_1 \cdot C_r \cdot M_{S_{ae}} \]  \hspace{1cm} (9)

where \( S_{ae} \) is the elastic acceleration response spectrum of the equivalent SDOF system determined according to current Chinese seismic design Code (CMC, 2010); \( M \) is the mass of the equivalent SDOF system; and \( \Gamma_1 \) is the modal participating factor of the first vibration mode. For simplicity, only the contribution of the first vibration mode to the distribution of lateral force is considered because the first mode usually has the most significant effect for moment-resisting RC frame structures.

The steel reinforcement of the structural components should be modified if the following expression could not be satisfied:

\[ |F_y - F_y^r | \leq 0.05 \cdot F_y \]  \hspace{1cm} (10)

where \( F_y \) is the actual yield strength of the structure. Then, that process is repeated from Step 4. The
required shear strength of the \( i \)th story is determined by the following equation:

\[
F_i = \frac{\sum_{j=1}^{n} m_j \phi_j}{\sum_{j=1}^{n} m_j} \cdot F_y
\]

where \( \phi_j \) is the amplitude at the \( j \)th floor of the first mode; and \( m_j \) is the mass at the \( j \)th floor. The iterative adjustment should be conducted until Eq. 10 is satisfied.

7. Conduct construction detail design

As the last stage of design, reasonable seismic construction details should be adopted, and the final shop drawings should be completed accordingly.

In conclusion, the structure is initially designed by the conventional strength-based method. Then, the two performance indexes, the inter-story drift ratio and the global damage index, are checked against the limits that correspond to the performance objectives. The inter-story drift alone is evaluated by the capacity spectrum method. The global damage index is evaluated indirectly by means of CDYSS, which builds the relationship between the yield strength and the damage index. Fig. 2. shows a flowchart of the damage-control seismic design procedure.

**Fig. 2. Flowchart of Damage-control Seismic Design Procedure**
6. Case Study

6.1 Structure Description

To verify the proposed design method, a typical 6-story moment-resisting RC frame building structure is selected as the structure to be designed following this method.

The building is assumed to be located in an area with a Chinese seismic intensity of 8. The PGA values are 70, 200, 264, and 400 gal, respectively for the four design earthquake levels. The category of the site soil condition is specified as Design Group 1 of Class IV according to current Chinese seismic design code (CMC, 2010). The characteristic period of the site soil is 0.65 s.

The typical structural plan layout is shown in Fig.3. The total structural height is 22.5m, with a story height of 4.5m for the first story and 3.6m for the upper stories. The cross-sectional dimensions of the columns are 500mm×500mm for the 1st to 3rd stories and 400mm×400mm for the 4th to 6th stories. All beams are 250mm×500mm in cross section. The material strengths chosen are as follows: the standard yield strength of steel reinforcement is 335 MPa, and the standard cubic compressive strength of concrete is 30 MPa. The live loads of the floors are taken as 2kN/m².

6.2 Damage-control Seismic Design

Grade C is selected as the seismic performance objectives for the target building. According to Table 1., the building should behave fully operationally under frequent earthquakes, operationally under basic earthquakes, be repairable under rare earthquakes, and serve as collapse prevention under extremely rare earthquakes. For the four levels of earthquakes, the corresponding limits of the global damage index are 0.05, 0.20, 0.40, and 0.80, and the limits of the inter-story drift ratio are 1/550, 1/250, 1/100, and 1/50.

According to the strength-based method specified in the current Chinese seismic design code, the initial steel reinforcement of structural members is determined. With the aid of OpenSees software, the computational model of the structure is established using fiber elements. The main parameters of the structure are shown in Table 3. The main parameters of the equivalent SDOF system are as follows: 0.98 s for the fundamental period, 207t of the total mass, and 1.309 as the modal participating factor of the first mode.

By pushover analysis, the base shear-top displacement relationship is obtained, and then the yield strength and the ultimate displacement ductility factor under monotonic loading are derived. The inter-story drift ratios and the yield strength are checked against the requirements according to the predetermined performance objectives. The steel reinforcement of the structural components is revised iteratively to meet the requirements. After four iterations of adjustment, the steel reinforcement is finalized, and the corresponding base shear-top displacement relationship is shown in Fig.4. By the method shown in Fig.1., the yield displacement $\delta_y$ and the ultimate displacement $\delta_u$ are found to be 0.117 m and 0.634 m, respectively; hence, the ultimate ductility factor $\mu_u$ is 5.42.

6.3 Verification by Time-history Analysis

To verify the efficiency of the proposed damage-control seismic design method for RC moment-resisting frame buildings, a nonlinear time-history analysis is carried out by inputting 83 sets of natural earthquake ground motion records in line with the site soil category of Design Group 1 of Class IV.

The global damage indexes of the structure under each level of earthquake are then calculated. The distributions of the global damage indexes under the
Table 3. Parameters of a 6-story RC Frame Structure

| Story | \( m_i \) (t) | \( \Phi_i \) | \( m_i \Phi_i \) | \( m_i \Phi_i \) |
|-------|----------------|-------------|-----------------|-----------------|
| 6     | 62.0           | 1           | 62.0            | 62.0            |
| 5     | 72.8           | 0.925       | 67.3            | 62.3            |
| 4     | 72.8           | 0.781       | 56.9            | 44.4            |
| 3     | 72.8           | 0.578       | 42.1            | 24.3            |
| 2     | 72.8           | 0.393       | 28.6            | 11.2            |
| 1     | 72.8           | 0.193       | 14.1            | 2.7             |
| \( \sum \) | 426.0         |             | 270.9           | 207.0           |

Table 4. Yield Strengths Satisfying the Performance Objectives for a 6-story RC Frame Structure

| Earthquake Design Level | Frequent Earthquake | Basic Earthquake | Rare Earthquake | Extremely Rare Earthquake |
|------------------------|---------------------|------------------|-----------------|---------------------------|
| Global Damage Index    | 0.05                | 0.20             | 0.40            | 0.80                      |
| \( a_1 \)              | 0.4448              | 0.4448           | 0.4448          | 0.4448                    |
| \( a_2 \)              | -0.5865             | -0.5865          | -0.5865         | -0.5865                   |
| \( b_1 \)              | 0.3719              | 0.3719           | 0.3719          | 0.3719                    |
| \( b_2 \)              | -0.6908             | -0.6908          | -0.6908         | -0.6908                   |
| \( c_1 \)              | 0.8747              | 0.8747           | 0.8747          | 0.8747                    |
| \( c_2 \)              | 2.6377              | 2.6377           | 2.6377          | 2.6377                    |
| \( A \)                | 0.9564              | 0.4242           | 0.2825          | 0.1881                    |
| \( B \)                | 0.9163              | 0.3517           | 0.2179          | 0.1350                    |
| \( C \)                | 0.0001              | 0.0768           | 0.2592          | 0.4762                    |
| \( C_\gamma \)         | 1.87                | 0.78             | 0.50            | 0.32                      |
| \( S_{ae} \) \((m/s^2)\) | 1.08               | 3.04             | 4.06            | 6.10                      |
| \( F_{y} \) \((kN)\) | 720                 | 836              | 722             | 702                       |

Fig. 5. Drifts and Inter-Story Drift Ratios of the Target Structure under Different Levels of Earthquake
four intensities of earthquake are shown in Fig.6. The mean values of the global damage indexes for the four intensities of earthquake are 0.035, 0.154, 0.307 and 0.572, and the corresponding coefficients of variation (COV) are 0.208, 0.167, 0.143 and 0.190. It can be concluded that all of the global damage indexes under each level of earthquake are within the allowable limits.

Fig.6. Distribution of Global Damage Indexes of the Target Structure under Different Levels of Earthquake.

Fig.7. Maximum Inter-Story Drift Ratios and Relative Errors
Fig. 7 illustrates the results of the average maximum inter-story drift ratio by time-history analysis compared to the corresponding results of pushover analysis. The relative error between the results of the two methods is defined as follows:

$$\text{Relative error} = \frac{\text{IDR}_{\text{TH}} - \text{IDR}_{\text{Push}}}{\text{IDR}_{\text{Push}}} \times 100\% \quad (12)$$

where \( \text{IDR}_{\text{TH}} \) is the average value of the maximum inter-story drift ratios by time-history analysis; and \( \text{IDR}_{\text{Push}} \) is the inter-story drift ratio at the performance points found by pushover analysis. It can be found that all the average values of the maximum inter-story drift ratios are within the allowable limits. The results of the time-history analysis are comparable to those found by pushover analysis because the distributions of inter-story drift ratios are similar and most of the relative errors are within 20%.

Both the global seismic damage indexes and the inter-story drift ratios are well controlled within the specified limits. Therefore, the structure designed by the proposed damage-control seismic design method is capable of satisfying all the requirements of the preset seismic performance objectives.

7. Conclusions

A new damage-control seismic design method is developed for moment-resisting RC frame buildings. Two performance indexes, the global seismic damage index and the inter-story drift ratio, are applied to control the damage of structural and non-structural components, respectively. Four performance levels, defined by two quantitative performance indexes and four grades of seismic performance objectives, corresponding to four types of buildings, are proposed. By this method, the seismic damage of structural as well as non-structural components can be controlled effectively. The validity of the method is tested using a case study of a typical 6-story RC frame building structure that is seismically designed following this new method. By nonlinear time-history analyses, the responses of the target structure under different levels of earthquake ground motions are evaluated and are found to be well within the preset limits according to the performance objectives. This method can be extended to apply to other types of RC building structures. However, the method proposed here is suitable for controlling seismic damage of non-structural components whose damage is closely related to inter-story drift, such as infilled walls and pipelines, rather than those whose damage is related to other responses, such as velocity or acceleration.

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