An improved displacement based seismic design method of prestressed fabricated concrete frame

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Abstract: Due to the randomness of ground motion and the variation of the material itself, it is unreasonable to regard the inter-layer target displacement angle of the structure as a certain amount in the direct displacement-based seismic design. In this paper, an improved displacement seismic design method is proposed, which is characterized by: (1) Considering the reliability when determining the target displacement angle between layers, and combining the probability characteristics of the target displacement angle between layers, the seismic design of the structure has a certain probability meaning. (2) According to the quantitative relationship between damage index and displacement ductility, combining structural damage performance target, the displacement ductility of the structure is determined by the damage index in the displacement-based design method, and the determination of displacement ductility is directly related to the damage performance target. Finally, it is applied to the design of prestressed precast concrete frame structure, then OpenSees modeling is used to verify the improved displacement design method, and at the same time compared with the original design method. The results show that the improved displacement seismic design method proposed in this paper can well control the elastoplastic response of the structure and meet the performance of the structure. The original displacement design method is more conservative than the improved displacement seismic design method.

1. Introduction
Based on the record on seismic damage and theoretical research, the shortfall in deformability is the main reason behind the collapse of the structure under the influence of major earthquake. Therefore, it is considered as an effective means to safeguard the structure by controlling over structural displacement \cite{1}. The fulcrum of direct displacement-based seismic design focuses on the idea that it deems the inter-story displacement of the part of the structure which most easily comes to buckle as the criteria of design control, and converts the multi-degree-of-freedom system into a single-degree-of-freedom system. According to the the single freedom system, the equivalent stiffness of the structure and the lateral bearing capacity are pined down, then the structural section reinforcement is able to be fixed. This method can provide a better control in the deformation of the structure under seismic excitation and serve to achieve the target of earthquake-resistance.
In 1984, Moehle et al. used the response spectrum method to equate the multi-degree-of-freedom system into a single free system and calculated the maximum displacement response of the multi-free system, which led to the rational for the direct displacement-based seismic design method[2]. In 1995, Kowalsky applied the direct displacement-based seismic design method to the bridge structure and performed the nonlinear time-history analysis[3]. In 2000, Priestley proposed the design process for the direct displacement-based seismic design based on the energy spectrum, in addition, parameter analysis of flexible foundation, intensity of ground motion, construction height, seismic pulse, etc as well as their effects on design result was carried out[4]. Priestley solved the basic period of structure through elastoplastic displacement spectrum and applied the displacement-based design method to the prestressed precast concrete shear wall structure, aiming to meet the requirements of different performance levels of prestressed precast concrete shear wall structure under earthquake[5]. In 2001, Qiang Que corrected the elastic response spectrum by strength reduction factor and obtained the seismic demand spectrum, and then the displacement demand of the structure was given by graphic illustration. In this way, the displacement seismic design method based on the capability spectrum was set up[6]. Rahman et al. proposed a direct displacement-based design method for self-centering shear wall structures, and suggested the base shear force obtained by displacement-based design method was smaller than that determined by force-based design method[7]. Chopra et al. conducted a comprehensive evaluation on structural nonlinear response through the idea of equivalent linearization, and found that the nonlinear reaction and time history analysis determined by equivalent method were not accurate enough. At the same time, Chopra also evaluated the single-degree-of-freedom structure designed by the equivalent linearization idea. The research showed that the real displacement of the designed structure under earthquake had a large gap with the target displacement, and the nonlinear response spectrum method not only well determined the nonlinear response of the structure, but also was consistent with the expectation[8]. In the direct displacement-based seismic design method of shear wall suggested by Tjen N T et al, the plastic displacement angle and vertex displacement angle of the shear wall were used to define the performance level of the shear wall structure, which could better describe the performance state of the structure[9]. Cardone D considered the nonlinearity of the upper frame structure and proposed a direct displacement-based seismic design method for the base isolation frame structure[10]. The prestressed fabricated concrete frame structure can provide the advantages of environmental protection, saving manpower and material. In order to promote its application in the seismic zone, its seismic design has been sent to the spotlight. Therefore, it is of importance to apply the displacement-based seismic design method to the prestressed fabricated concrete frame structure. In the seismic design of a direct displacement-based frame structure, the key issue is to determine the inter-story target displacement angle of the structure. Due to the randomness of ground motion and the variation of the material itself, the inter-story displacement response of the structure under the expected earthquake will also be random. However, the inter-story target displacement angle, as rule, is regarded as a certain value, which may lead to the strange inconsistency for the expected safety of earthquake resistance structure. Focusing on the above issues, this paper proposes an improved displacement-based seismic design method, which combines the reliability theory and the probability characteristics of the inter-story displacement angle to determine the inter-story target displacement angle.

In addition, in the direct displacement-based seismic design method, the displacement ductility coefficient of the structure is previously assumed according to the performance target in general, and then the equivalent damping ratio of the single free system is determined based on the ductility coefficient. This way cuts off the direct association between the displacement ductility coefficient and structural performance target, and is not intuitive enough. For this reason, the displacement-based seismic design method improved in this paper adopts the damage performance target, combining the quantitative relationship between the damage performance target and the displacement ductility coefficient established by Park-Ang[11] bi-parameter damage criterion, associates the determination of structural displacement ductility with damage performance objective.
2. Quantitative relationship between structural damage performance target and the coefficient of displacement ductility

2.1 Determination of displacement ductility coefficient based on damage

Seismic damage assessment is an indicator of the overall structural damage under an expected earthquake. At present, it mainly reflects the damage caused by different failure mechanisms to structures or components from three aspects: deformation, energy or combination of them. Li Hongnan et al. extended it according to Park-Ang bi-parameter damage model of the single-degree-of-freedom system to the multi-degree-of-freedom system by combining the equivalent principle, and established the damage calculation formula for each layer as follows:

\[
DM_i = \frac{\Delta_{im}}{\Delta_{iu}} + \lambda \frac{\gamma_h}{V_i \cdot \Delta_{iu}}
\]  

(1)

In formula (1), \(V_i\) is the yield shear force of the \(i\)th layer, \(\gamma_h\) refers to the accumulation hysteresis energy and \(\Delta_{im}\) represents the maximum of the elastoplastic story draft of the \(i\)th layer. What can be determined according to the simplified formula given in the Code for Seismic Design of Buildings as follows:

\[
\Delta_{iu} = \eta_p \Delta_{iem}
\]  

(2)

Where \(\Delta_{iem}\) is the maximum inter-story draft of the structure obtained by elastic analysis under rare earthquakes, \(\eta_p\) is the augmentation coefficient of elastoplastic inter-story draft. As for the value, See also the Code for Seismic Design of Building Structures. \(\Delta_{iu}\) is the limit value of the inter-layer draft of the \(i\)th layer, the limit value of the elastoplastic inter-layer draft angle specified by the specification may be obtained from that in the limit state.

Fajfar introduced the normalized cumulative energy consumption parameters to establish the relationship between maximum inter-story drift and the inter-story cumulative energy dissipation:

\[
\gamma_h = \frac{1}{\omega q_{max}} \sqrt{\frac{E_h}{m}}
\]  

(3)

In formula (3), \(q_{max}\) and \(E_h\) are the maximum seismic displacement and cumulative hysteretic energy of a single degree of freedom system, respectively; \(\omega\) and \(m\) are the elastic frequency and mass of a single degree of freedom system, respectively. Equation (3) is a statistical result of the SDOF system based on a large number of actual seismic records, which is inconvenient to apply. Consequently, it is sensible to establish the relationship between damage index and structural ductility coefficient by introducing the theory of single degree of freedom system.

\[
\Delta_{im} = \Psi_{i} \phi_{max} (\Phi_{i} - \Phi_{i-1})
\]  

(4)

\[
V_i(t) = \Psi_{i} \cdot \sigma(t) \cdot \frac{\sum_{j=1}^{n} m_j \Phi_i}{\sum_{j=1}^{n} m_j \Phi_i^2}
\]

\[
= \Psi_{i} \cdot b \cdot \sigma(t)
\]  

(5)
In formula (5), $\Psi_i$ is normalized first mode participation coefficient, $\Phi_{i1}$ is the normalized first mode value of $i$th floor, $\phi_{\text{max}}$ is the maximum displacement of an equivalent single degree of freedom system, $\sigma(t)$ is the restoring force time history.

Combined with the theory of single degree of freedom system, the generalized cumulative energy dissipation parameters are obtained:

$$
\gamma_h = \frac{1}{\omega \phi_{\text{max}}} \sqrt{\int_0^t \phi(t) \sigma(t) \frac{M}{M}}$

$$
= \frac{\sqrt{(\Phi_{i1} - \Phi_{i1-1})^2/b^{\phi_{\text{max}}}}}{\omega \Delta_{im}} \cdot \frac{\gamma_h}{M}$

(6)

Where $M$ is the mass of an equivalent single degree of freedom system.

Substituting the formula (6) into the formula (1), the damage index equation of the $i$th layer is:

$$
\mu = \frac{q_{\text{max}} \cdot k}{f(t)}
$$

$$
DM_i = \frac{\Delta_{im}}{\Delta_{iu}} + \lambda \frac{\gamma_h}{V_i \cdot \Delta_{iu}}$

$$
= \frac{\Delta_{im}}{\Delta_{iu}} + \lambda \frac{\gamma_h^2 \Delta_{im}^2 \cdot \Delta_{iu}}{(\Phi_{i1} - \Phi_{i1-1})^2 \cdot \Delta_{iu}^2}$

$$
= \frac{\Delta_{im}}{\Delta_{iu}} (1 + \lambda \gamma_h^2 \cdot \frac{q_{\text{max}} \cdot k}{f(t)})$

$$
= \frac{\Delta_{im}}{\Delta_{iu}} (1 + \lambda \mu \gamma_h^2)
$$

(8)

Where $\mu$ is the ductility coefficient and $k$ is the elastic stiffness of the equivalent single degree of freedom system.

The quantitative relationship between damage index and displacement ductility is established through the single degree of freedom system theory and Park-Ang bi-parameter damage model, and thus, the displacement ductility of the structure can be determined by the damage index.

2.2 Structural damage performance target

The direct displacement-based seismic design method needs to meet the performance requirements proposed by the owner as much as possible, and consider the repair cost to quantitatively describe the performance objectives of the structure. The structural performance target can be measured by the energy dissipation index, the displacement index, the ductility coefficient index, etc. At present, the displacement index measured by the draft angle between the structural layers is most widely used. In order to establish the association between the structural displacement ductility coefficient and the structural performance target, combined with the related literature[16], this paper uses the seismic damage performance target of the structure at the three-level seismic fortification to establish the relationship between the damage performance target and the damage index. Table 1 lists the seismic damage performance objectives of the structure under the three-level seismic design.
Table 1. Damage performance target under three level seismic design

| Fortification criterion of three level earthquake | Undamaged under minor earthquake | Repairable under moderate earthquake | No collapsing under strong earthquake |
|--------------------------------------------------|---------------------------------|---------------------------------|-----------------------------------|
| Performance level                                | Good application                | Personal safety                 | Collapse prevention                |
| Permissible value of damage index                | 0.00 ~ 0.15                    | 0.15 ~ 0.30                    | 0.30 ~ 1.00                       |

In the improved displacement-based seismic design method, the related damage index is firstly determined according to the structural damage performance target proposed by the owner, and then the displacement ductility coefficient satisfying the damage performance target is determined according to the inter-story damage calculation formula (8), thereby determination of the draft ductility factor is rational and can be directly linked to the damage performance objectives of the structure.

3. Improved design process of displacement-based seismic design method

3.1 Determination of target displacement angle of inter-story based on reliability theory

The target displacement can be considered as the displacement requirement of the corresponding performance point of the structure under the expected earthquake. In the improved displacement-based seismic design method, the functional function (9) is established to introduce the reliability concept, and the inter-story draft angle of the corresponding performance point under the corresponding seismic risk level is taken as the effect \( \theta_r \), and the limit of inter-story draft angle under the expected earthquake is seen as resistance \( \theta_s \). Once the mean value is determined, the inter-story target draft angle of the structure can be obtained.

\[
Z = \theta_r - \theta_s \tag{9}
\]

The reliability formula established by the functional function is as follows:

\[
\beta = \frac{\mu_{ln\theta_r} - \mu_{ln\theta_s}}{\sqrt{\sigma_{ln\theta_r}^2 + \sigma_{ln\theta_s}^2}} \tag{10}
\]

It is assumed that the limit of inter-story displacement angle obeys the law of lognormal distribution, and this is future available:

\[
\beta = \ln\left(\frac{\mu_{\theta_r}}{\mu_{\theta_s}}\right) \approx \ln\left(\frac{\mu_{\theta_s}}{\mu_{\theta_r}}\right) \tag{11}
\]

Where \( \mu \) and \( \sigma \) refer to mean value and standard deviation, respectively. And \( \delta \) represents the coefficient of variation.

It can be seen from equation (11) that as long as the mean value of the coefficient of variation and the limit displacement angle between the layers is determined, the quantitative relationship between the reliability index and the mean value of the target inter-story draft angle can be established. Wu Bo et al. performed statistical analysis of the variation coefficient and the probability characteristics of the limit inter-story draft angle from more than 40 domestic and foreign test samples, and indicated that \( \delta_{\theta_r} \) and \( \delta_{\theta_s} \) are 0.429 and 0.61 respectively, and based on this assumption that the ultimate deformation angle of the reinforced concrete column is approximately equal to the frame of the ultimate inter-story draft angle of the structure, then fitting the regression formula of the mean value of the limit of inter-story draft angle:
\[ \mu_{\theta_y} = 0.0086 \sqrt{\frac{\rho_s f_y}{f_c}} \left( \frac{2.491 + 0.219 m}{0.523 + 1.466 n} \right)^2 \]  
\[ \rho_s = \frac{A_p}{S h_c} \]  

In the formula, \( \rho_s \) is the pitch of the stirrup, \( h_c \) is the distance from the center of the outer stirrup to the center, \( f_y \) is the yield strength of the stirrup, \( m \) is the shear span ratio, \( n \) refers to the axial compression ratio and \( f_c \) represents the compressive strength of the concrete.

Taking the mean value of the extreme displacement angle between layers as the inter-story target drift angle of the weak layer of the designed structure makes the determination of the inter-story target displacement angle statistically more scientific and reasonable. Although the target inter-story displacement angle is still a certain value, the characteristics of structural random seismic response are considered in the probability sense.

### 3.2 Determination of equivalent parameters of single degree of freedom system

The direct displacement-based seismic design method needs to determine the relationship between structural deformation and earthquake effect. Firstly, the multi-degree-of-freedom system must be converted into a single-degree-of-freedom system. Three basic principles need to be followed: 1) The displacement mode of the multi-degree-of-freedom system under the earthquake effect is constant; 2) the original base shear of multi-degree-of-freedom system is equivalent to the equivalent single-degree-of-freedom system base shear; 3) the earthquake effect is equal in both systems. According to the above equivalent principle, the value of the equivalent parameter is determined.

See Eq (14) and Eq (15) for the equivalent displacement \( x_{eq} \) and equivalent mass \( m_{eq} \) of the structure.

\[ x_{eq} = \frac{1}{\sum_{i=1}^{n} m_i x_i^2} \sum_{i=1}^{n} m_i x_i \]  
\[ m_{eq} = \frac{1}{x_{eq}} \sum_{i=1}^{n} m_i \]  

Where \( m_i \) and \( x_i \) are the mass of storey \( i \) of the structure and the displacement of that respectively; \( n \) is the storey of the structure.

The reinforced concrete frame structures located in the seismic zone have fewer layers. The pattern of seismic performance of the frame structures is the mutual displacement between layers and the translation of floors, it is advisable to consider the lateral displacement mode as shear type. When the mass and the stiffness are evenly distributed along the height, the deformation curve of frame structure is similar with the cantilever beam with the upper end free and the lower end fixed. If it is assumed that the horizontal seismic force is in a type of an inverted triangle distribution, the lateral shift \( u(z) \) of the cantilever with equal section at any height \( z \) can be expressed as:

\[ u(z) = \frac{\delta q h^2}{6GA} \left[ \frac{3}{h} \frac{h}{h} - \left( \frac{h}{h} \right)^3 \right] \]
\[ x_i = \frac{\delta q h^2}{3GA} \]  
\[ \xi_i = \frac{h_i}{h} \]  
\[ x_i = \phi(\xi_i) x_t \]  
\[ \phi(\xi_i) = \frac{1}{2} (3\xi_i - \xi_i^3) \]

Where \( x_t \) is the vertex displacement of the structure, \( x_i \) is the displacement of the \( i \)th storey of the structure, \( h \) is the total height of the structure, \( h_i \) is the height of storey \( i \) of the structure, \( \phi(\xi_i) \) can be regarded as the shape of the first mode of vibration shape of the structure, \( GA \) is shear stiffness, \( \delta \) is the shear stress unevenness coefficient, \( q \) is the peak of the distributed load of the inverted triangle.

On the premise that the weak inter-story draft is known, the other inter-layer draft of the structure can be determined according to formula (20).

The structural equivalent period \( T_{eq} \) can be determined by the displacement spectrum. Under the premise that there is no actual displacement spectrum in China, the displacement spectrum can be calculated according to the acceleration response spectrum:

\[ S_d = \frac{S_p(\xi)}{4\pi^2 T_{eq}^2} \]  

In the formula, \( S_d \) refers to the acceleration response spectrum and \( S_p \) is the displacement response spectrum.

It should be pointed out that in order to ensure that the acceleration of the medium-long period structure is not unduly undersize under the action of earthquake, the current specification has a conservative value for the spectral acceleration value of the long-period part of the acceleration response spectrum. Although this causes inaccurate results in the medium-long-period structure, it is acceptable to calculate the displacement spectrum from the above formula and apply it to the frame structure because the frame structure with a small number of layers has a small period.

To establish the displacement spectrum, all the first is to determine the damping ratio \( \zeta_{eq} \) of the equivalent single degree of freedom system. Scholars \([20-23]\) have studied the equivalent linearization of inelastic systems and this paper uses the equivalent damping ratio formula based on energy method.

\[ \zeta_{eq} = k \cdot \frac{2 \cdot (\mu - 1) \cdot (1 - \alpha)}{\pi \mu \cdot (1 + \alpha \mu - \alpha)} + \zeta_0 \]

In the formula, \( k \) is the damping adjustment coefficient of the hysteretic energy of the reaction structure, \( \mu \) is the ductility coefficient of the structure, \( \alpha \) is the stiffness degradation coefficient of the structure, and \( \zeta_0 \) is the elastic damping ratio of the system.

The structural equivalent stiffness \( K_{eq} \) can be determined according to formula (23) according to the equivalent period \( T_{eq} \):

\[ K_{eq} = \frac{2\pi}{T_{eq}} m_{eq} \]  

The base shear force \( V_{eq} \) and the inter-story shear \( V_i \) can be determined as follows:

\[ V_{eq} = K_{eq} x_{eq} \]
It should be noted that if the structure is required to satisfy the established performance target under multiple earthquakes or basic seismic earthquakes, the equivalent shear force should be calculated according to the seismic parameters under the multiple intensity or basic intensity and the anti-collapse check of the structure under the rare earthquake should be conducted. If the structure is required to satisfy the established performance target under the rare earthquake, the seismic parameters under the rare earthquake should be used when calculating the equivalent shear force.

The flow chart of the improved displacement-based seismic design method is shown in Figure 1.

4. Design and verification of calculation example

4.1 Design of calculation example

For comparison, a four-layer two-span prestressed precast concrete frame structure is designed based on improved displacement-based seismic design and displacement-based seismic design method proposed in reference[5]. The structure is located in the 8 degree seismic zone and PGA is 0.20g. The beam is made of C40 concrete and the column is made of C30 concrete. The prestressed tendons in the beam are tensioned by \( \phi \)15.24 high-strength and low-relaxed steel strands. The height of the first layer of the structure is 6m, and the height of the other layers is 4.8m.

For the sake of comparison, the same performance targets were adopted: the structure does not collapse under the major earthquake. According to Table 1, the damage index of the structure is selected to be 0.7, and the ductility coefficient of the structure is determined by formula (8) and the value is \( u = 2.5 \).

The cross-sectional dimension of the bottom column is \( 700 \times 700 \) determined empirically, and the column axial pressure ratio \( n \) is approximately calculated to be 0.280. According to formula (12), the mean value of the limit of the weak inter-story draft angle can be obtained. Assuming the reliability of the weak inter-story draft angle is \( \beta = 2.0 \), the mean value of the target weak inter-story draft angle is \( \mu_{\theta_y} = 0.018 \), determined according to the inter-layer displacement reliability formula (11), so that this draft can be determined \( 0.018 \times 6000 = 108mm \).

After determining the target inter-story draft value, the equivalent parameters of the single degree of freedom system can be given by the steps of the flowchart. The equivalent parameters determined by the improved displacement-based seismic design method are shown in Table 2, and the comparison with the equivalent parameters determined by the displacement seismic design method used in the reference[5] is given.
Figure 1. Flowchart of improved displacement based seismic design

| Equivalent parameters       | The improved displacement-based seismic design method | The normal displacement seismic design method |
|-----------------------------|------------------------------------------------------|---------------------------------------------|
| Equivalent damping          | 21.5%                                                | 26.7%                                       |
| Equivalent displacement     | 209.6                                                | 288.4                                       |
| Equivalent mass             | 732.5                                                | 757.1                                       |
| Equivalent period           | 0.86                                                 | 0.96                                        |
| Equivalent stiffness        | 5.32                                                 | 4.97                                        |
| Equivalent shear            | 1115.07                                              | 1190.4                                      |

The determined equivalent shear force is combined with the gravity load to determine the area of prestressed tendons and energy consuming bars. Figure 2 shows the reinforcement of the prestressed precast concrete frame joints.
Figure 2. Reinforcement design

Note: $A_p$ is the area of the prestressed tendon, and $A_e$ is the area of the energy-consuming steel bar. The structure has the same cross-section reinforcement in the left and right side, and the standard unit is mm.

After converting the structure into a single degree of freedom system, the equivalent parameters determined according to reference[5] are compared with the equivalent parameters determined by the improved displacement-based seismic design method. It shows that, in the improved displacement based seismic design method, the determined equivalent damping, equivalent displacement, equivalent mass, equivalent period and equivalent shear force are smaller than those determined by the displacement seismic design method used in reference[5], but the value of equivalent stiffness is larger than that of the literature.

4.2 Verification of calculation example

4.2.1 Verification of finite element model

The finite element model was established by using the structural simulation software OpenSees[24], and the elastoplastic analysis of the prestressed precast concrete frame structure was carried out. Firstly, six prestressed precast concrete frame joint test models designed by Dong Tingfeng[25] et al. are taken as examples to illustrate the rationality of the modeling method. The joint finite element model is shown in Figure 3.
Figure. 3 Finite element model of joint

In the joint finite element model, the prefabricated beam-column members are simulated by fiber section based nonlinear element, and the concrete fiber is simulated by Concrete02 material. Using Concrete01 material that can only be pressurized simulates the mortar layer, and adopts a nonlinear beam-column element. Prestressed tendons and energy-consuming steel bars are simulated by truss elements that can only withstand axial forces. Steel02 material is used to simulate the prestressed tendons. BarSlip element, taking into account the slippage of the steel bars, is adopted to simulate the energy-consuming steel bars. The beam-column joints are simulated by setting the inner and outer joints, and the inner and outer joints are connected by two axial springs that reflect the axial deformation and a shear spring that reflects the shear deformation.

A six-node finite element model is established and a pushover analysis is performed. The comparison between the simulation results and the test results is shown in Fig. 4. The simulation results of the hysteresis curve of the joint specimen are in good agreement with the test results in terms of fullness and pinching tendency, and the simulations on the initial stiffness and ultimate bearing capacity are also relatively accurate. Although the simulated loading and unloading stiffness of the joint specimen are different from the experimental results, the difference is not significant. Therefore, the established joint finite element model can effectively predict the energy dissipation capacity of prestressed precast concrete frame joints.

4.2.2 Selection of ground motion

The finite element model of the whole structure is established by using the above-mentioned OpenSees joint finite element model, and elastoplastic analysis is carried out. According to the requirements of the Code for Seismic Design of Buildings[12], two natural waves and one artificial wave are selected. Table 3 lists the details of the three selected seismic waves.

Table 3 Earthquake Record

| SN    | Seismic waves | Time cell / (s) | Duration / (s) | Spot          |
|-------|---------------|-----------------|----------------|---------------|
| DZD1  | RSN54         | 0.005           | 25.64          | Borrego Springs|
| DZD2  | RSN86         | 0.005           | 52.47          | San Onofre    |
| DZD3  | REN           | 0.01            | 40             | -             |
4.2.3 Results of elastoplastic analysis

The obtained base shear-displacement curve of the structure is analyzed as shown in Fig. 5. The capacity curve is linearized to obtain the yield displacement and ultimate displacement of the structure. It can be seen from the figure that the yield displacement of the structure is 0.13m and the ultimate displacement is 0.3m. The calculated displacement ductility coefficient of the structure is 2.3, which is not much different from the design displacement ductility coefficient of 2.5. Consequently, the framework satisfies the ductility requirements.
Figure 6 shows the inter-story draft curves of layers under different rare ground motions. It can be seen that the inter-story drift of the designed structure under different ground motions is smaller than the limit inter-story draft specified by the specification\(^5\), and meets the damage performance requirements of “preventing collapse” under rare earthquakes. The inter-story draft of the designed structure under different ground motions is not only smaller than the inter-story target displacement used in the reference\(^5\), but also smaller than the target inter-story draft proposed in this paper except the top storey. Since the displacement of the top layer of the frame structure is relatively small, it is usually not the weak layer of the structure, and the structure can be considered safe on the top layer. Figure 7 shows the floor displacement curve of the designed structure under different ground motions. It can be found that under different ground motions, the lateral draft curves of each floor of the structure are remarkable different, but they have almost the same pattern as the assumed lateral shift curve, and are smaller than the proposed displacement mode in this paper and the displacement mode in reference\(^5\). Based on Figure 6 and Figure 7, it can be considered that the improved displacement-based seismic design method and the displacement seismic design method in reference\(^5\) are safer, and the displacement seismic design method adopted in literature\(^5\) is more conservative.

The elastoplastic analysis results show that the maximum inter-story draft of the structure is in the first storey, which is consistent with the initial assumed weak layer position, and no further calculation is needed. Calculate the maximum inter-story draft and cumulative hysteresis energy of the first layer of the structure under different ground motions. Figure 8 shows the cumulative hysteretic energy dissipation of the first layer of the design structure. Table 4 lists the damage indicators of the design structure obtained by the damage formula (8) under different ground motions. The averaged damage index is DM=0.72, which is exactly like the expected damage index 0.7, which remains favourable.
## Table 4 Damage index of weak story

| Seismic waves | $\gamma_h$ (kN·m) | $\Delta_{1m}$ (mm) | $\Delta_{1u}$ (mm) | $V_{1y}$ (kN) | $DM_1$ |
|---------------|------------------|------------------|------------------|--------------|--------|
| DZD1         | 672              | 90               | 120              | 2240         | 0.85   |
| DZD2         | 500              | 74               | 120              | 1911         | 0.71   |
| DZD3         | 480              | 62               | 120              | 1847         | 0.61   |

Note: $\gamma_h$ refers to the accumulated hysteretic energy of the first layer, $\Delta_{1m}$ is the maximum displacement between the elastoplastic layers of the first layer, $\Delta_{1u}$ is the limit displacement between the first layers, $V_{1y}$ is the yield shear of the first layer, and $DM_1$ is the damage index of the first storey.

Compared with the displacement-based seismic design method in reference[5], the improved displacement-based seismic design method proposed in this paper adopts a smaller displacement mode, but still can well control the elastoplastic response of the structure and meet the performance requirements.

### 5. Conclusion

In this paper, an improved displacement-based seismic design method is proposed for prestressed precast concrete frame structures. The seismic design of the structure can take a remarkable control in the elastoplastic response of the structure and achieve the performance target of the structure under the expected earthquake.

The improved displacement-based seismic design approach has the following innovations and deficiencies:

1. The ductility coefficient of the structure is determined by the damage index according to the Park-Ang double-parameter criterion and seismic damage performance target of the structure, not only the structural damage factor is considered in the seismic design process of the structure, but also the ductility coefficient of the structure displacement associates with damage performance goals and is more intuitive.

2. According to the quantitative relationship between its reliability and the inter-story draft angle and the probability characteristics of the inter-story draft angle, the target inter-story draft angle is determined, so that the determined inter-layer target displacement angle has a probability meaning. The process improves significantly than considering the inter-story displacement angle as a certain amount.

3. The structure designed according to reference[5] is more conservative than the structure designed according to the improved displacement-based seismic design method in this paper.

4. The improved displacement-based seismic design method is applicable to the planar and vertical regular structures, since the displacement mode of this paper can not consider the complex shape of the irregular structure.

### References

[1] Sozen M A. Review of earthquake response of reinforced concrete building with a view to drift control.

[2] Moehole J P. Strong motion drift estimation for R/C structures[J]. Journal of Structural Engineering, 1984, 119(9):1998-2001.

[3] Kowalsky M J, Priestley M J N and Macrae G A. Displacement-based design of RC bridge columns in seismic regions[J]. Earthquake Engineering and Structural Dynamics, 1995, 24(12):1623-1643.

[4] Priestley M J N. Performance based seismic design[J]. 12WCEE, New Zealand, 2000, Reference No.2831.

[5] Priestly M J N. Direct displacement-based design of precast/prestressed concrete building[J]. PCI Journal, 2002, 47(6): 66-79.

[6] Qiang Xue. A direct displacement-based seismic design procedure of inelastic structures[J]. Engineering Structures, 2001, 23:1453-1460.
[7] Rahman M A and Sritharan S. An evaluation of force-based design vs. direct displacement-based design of jointed precast post-tensioned wall systems[J]. Earthquake Engineering and Engineering Vibration, 2006, 5(2):285-296.

[8] Chopra A K and Goel R K. Direct displacement-based design: use of inelastic vs. elastic design spectra[J]. Earthquake Spectra, 2001, 17(1):47-64.

[9] Tjen N T, Aschheim M A and Wakkacec J W. Yield displacement-based seismic design of RC wall building[J]. Engineering Structure, 2007, 29(11):2946-2959.

[10] Cardone D, Palermo G, Dolce M. Direct displacement-based design of buildings with different seismic isolation systems[J]. Journal of Earthquake Engineering, 2010, 14(2):163-191.

[11] Park Y J and Ang H S. Mechanistic seismic damage model for reinforced concrete[J]. Journal of Structural Engineering, 1985, 111(4): 722-739.

[12] WANG Feng, LI Hongnan and YI Tinghua. Direct Damage Based Seismic Design Methodology for RC Buildings[J]. Journal of Vibration and Shock, 2009, 28(02):128-131+144+206.

[13] Code for seismic design of buildings: GB 50011-2010 [S]. Beijing: China Architecture & Building Press, 2010.

[14] Fajfar P and Vidic T. Consistent inelastic design spectra: hysteretic and input energy[J]. Earthquake Engineering and Structural Dynamics, 1994, 23:523-537.

[15] Fajfar P. Capacity spectrum method based on inelastic demand spectra[J]. Earthquake Engineering and Structural Dynamics, 1999, 28(7):979-993.

[16] Ghobarah A, Abou-Elfath H and Biddah A. Response based damage assessment of structures[J]. Earthquake Engineering and Structural Dynamics, 1999, 28:29-104.

[17] Wu Bo, Guo Anxin and Lin Shaoshu. Probabilistic characteristics of ultimate drift ratio of R.C. column and its application in reliability analysis of controlled structures[J]. Journal of Earthquake Engineering and Engineering Vibration, 2001, 03:36-40.

[18] Medhekar M S and Kennedy D J L. Displacement-based seismic design of building-theory[J]. Engineering Structures, 2000, 22:201-221.

[19] Ray Clough and Joseph Penzien. Dynamics of structures. CSI 1995.

[20] Miranda E and Garcia J G. Evaluation of approximate methods to estimate maximum inelastic displacement demands[J]. Earthquake Engineering and Structural Dynamics, 2002, 31:539-560.

[21] Hwang J S. Evaluation of equivalent linear analysis methods of bridge isolation[J]. Journal of Structural Engineering, 1996, 122(8):972-976.

[22] Lin Y Y, Tsai M H, Hwang J S, et al. Direct displacement-based design for building with passive energy dissipation systems[J]. Engineering Structures, 2003, 25:25-37.

[23] Applied Technology Council. ATC-40 seismic evaluation and retrofit of concrete buildings[R]. California: Report No. SSC96-01, 1996.

[24] Mazzoni S, Mckenna F, Scott M H. Berkeley: OpenSees user’s manual[R]. University of California, 2006.

[25] LI Zhenbao, DONG Tingfeng, YAN Weiming, et al. Research on seismic behavior of joints in hybrid jointed frame[J]. Journal of Beijing University of Technology, 2006, 10:895-900.