Periodica Polytechnica
Civil Engineering

60(1), pp. 127–134, 2016
DOI: 10.3311/PPci.7383

RESEARCH ARTICLE

Performance-Based Plastic Design Method for Steel Concentrically Braced Frames Using Target Drift and Yield Mechanism

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Received 03-03-2014, revised 09-12-2014, accepted 18-12-2014

Abstract

Under severe earthquakes, steel concentrically braced frames (SCBFs) will experience large inelastic deformations in an uncontrolled manner. According to the energy-work balance concept, a performance-based plastic design (PBPD) methodology for steel concentrically braced frames was presented here. This method uses pre-selected target drift and yield mechanism as key performance limit states. The designed base shear for selected hazard levels was derived based on work-energy balance equations. Plastic design was performed to design bracing members and connection nodes in order to achieve the expected yield mechanism and behavior. The method has been successively applied to design a six-storey steel concentrically braced frame. Results of inelastic dynamic analyses showed that the story drifts were well within the target values, thus to meet the desired performance requirements. The proposed method provided a basis for performance-based plastic design of steel concentrically braced frames.

Keywords

PBPD · steel concentrically braced frames · target drift · yielding mechanism · work-energy balance concept

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Introduction

As a main lateral force resisting system of building structures, a steel concentrically braced frame (SCBF) is characterized by a larger lateral stiffness, a relatively simple detail. Besides, it can effectively reduce the horizontal displacement of structure and can improve the internal force distribution of structure. But CBFs will be prone to lateral buckling under the horizontal earthquake. Especially when subjected to the repeated horizontal earthquake, the shear capacity of floor and the lateral stiffness will drop sharply, which will induce the excessive increase of storey drift and eventually cause the structures to fail in a manner of total instability [7, 15].

It is well known that steel concentrically braced frames will undergo large inelastic deformation under major earthquake. The current code for seismic design of buildings is usually based on elastic properties of the structure and accounts for the inelastic properties indirectly. It is traditionally assumed that the applied forces on CBFs are primarily resisted by the truss action, and the capacity of design for bracings is conducted by the use of directional force [2–6, 9, 11, 12, 14].

However, when struck by severe ground motions, the structures designed by such procedures have been found to undergo inelastic deformations in a somewhat 'uncontrolled' manner. The inelastic behavior, which may include severe yielding and buckling of structural members and connections, can be unevenly and widely distributed in the structure. This may result in a rather undesirable and unpredictable response, sometimes total collapse, or difficult and costly repair work at best.

In recent years, several strong earthquakes have caused tremendous losses in life and belongings of people. The strength-based seismic design method cannot meet the requirements, and naturally the performance-based seismic design thoughts have aroused enough attention. However, the present performance-based seismic design depends heavily on a repeated iterative process including "assess performance", "revised design", "assess performance" until the designed structure can attain the expected behavior [1, 13]. Above this, an energy-based seismic design of structures using yield mechanisms and target drift as key performance objectives, was developed and was applied to design six example steel moment frames [8]. In this paper, a performance-based plastic design (PBPD) is proposed for steel concentrically chevron braced frames In this method, according to the pre-selected yielding mechanism and target drift, the design base shear is obtained from the work-energy equation; the designated yielding component is designed by the PBPD method; the non-designated yielding component is designed by the capacity method. The performance-based plastic design method can directly consider inelastic properties without estimate and iteration. Due to its clear concept and simple procedure, the PBPD method can enjoy a wide application in the actual design process.

1 Performance-based plastic design method

Performance-based plastic design method uses pre-selected yielding mechanism and target drift as performance limit states and these two limit states are directly related to the degree and distribution of structural damage. During the severe earthquake, in order to avoid the structural collapse, to dissipate seismic energy at most, and to endow the structure with sufficient strength and ductility, a reasonable yielding mechanism should be chosen at the beginning of the design. The selected target yielding mechanism for the steel braced frame structure is shown in Fig. 1a. Assuming that the plastic hinges only occur at the column base, and the buckling and yielding only accrue to the bracings. The design base shear for a selected hazard level is calculated by equating the work needed to push the structure monotonically up to the target drift to that required by an equivalent elastic plastic single degree of freedom to achieve the same state (Fig. 2b). In order to achieve the expected yield mechanism and behavior, the plastic design was performed to design bracing members and joints.

1.1 Design base shear

For an earthquake level, the determination of design base shear is a key in the PBPD method. As mentioned above, the computation of the design base shear is based on the energy equivalency, namely pushing the structure monotonically up to the target drift to that required by an equivalent elastic plastic single degree of freedom (EP-SDOF) to achieve the same state. Assuming the system as an ideal elasto-plastic system, the work-energy equation is

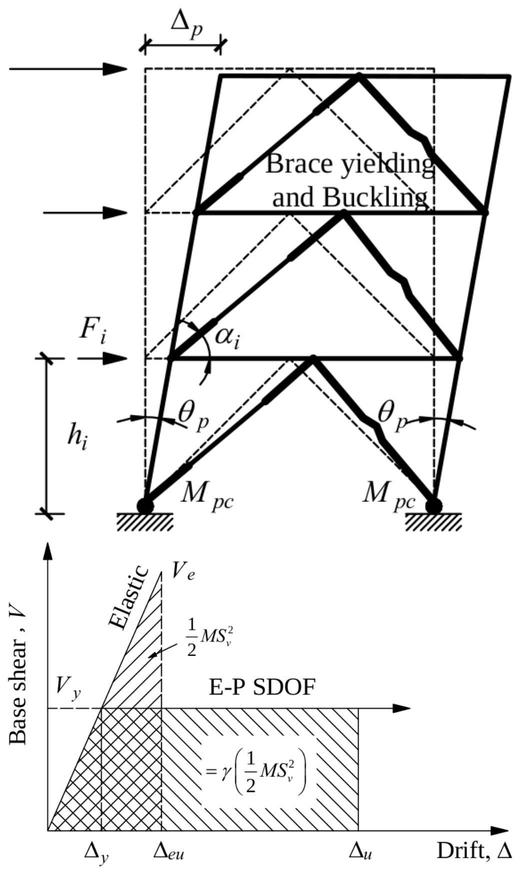
$$\left(E_e + E_p\right) = \gamma\left(\frac{1}{2}MS_v^2\right) = \frac{1}{2}\gamma M\left(\frac{T}{2\pi}S_a\right)^2 \tag{1}$$

$$\gamma = \frac{2\mu_s - 1}{R_\mu^2} \tag{2}$$

where,

E_e elastic component of energy required to make the structure achieve the target drift

E_p plastic component of energy required to make the structure achieve the target drift



(a) Yielding mechanism

(b) Performance-based plastic design

Fig. 1. PBPD Concept

S_v design pseudo velocity spectrum

S_a pseudo acceleration spectrum

T natural period of vibration

M total mass of system

g acceleration of gravity

γ energy modification factor)

μ_s structural ductility factor

R_μ ductility reduction factor

Elastic energy is:

$$E_e = \frac{1}{2}M\left(\frac{T}{2\pi}\frac{V_y}{G}g\right)^2 \tag{3}$$

where,

G the total gravity of structure

V_y yield base shear

The plastic energy is equal to the energy dissipated by the plastic hinge in the structure, as shown in Fig. 1. For the selected yield mechanism, the energy is:

$$E_p = V_y\left(\sum_{i=1}^{N}\lambda_i h_i\right)\theta_p \tag{4}$$

where,

λ_i lateral force distribution coefficient

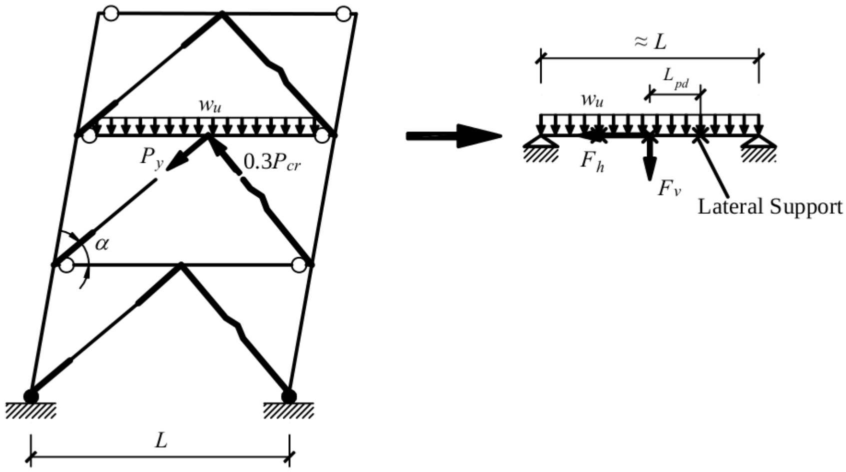


Fig. 2. Beam design forces for a chevron-type CBF

h_i height of story i from the base

θ_p plastic drift ratio

According to Equations (1), (3) and (4), work-energy equation can be rewritten as

$$\frac{1}{2}\left(\frac{G}{g}\right)\left(\frac{T}{2\pi}\frac{V_y}{G}g\right)^2 + V_y\left(\sum_{i=1}^{N}\lambda_i h_i\right)\theta_p = \frac{1}{2}\gamma\left(\frac{G}{g}\right)\left(\frac{T}{2\pi}S_a\right)^2 \quad (5)$$

or

$$\left(\frac{V_y}{G}\right)^2 + \frac{V_y}{G}\left(h\frac{8\theta_p\pi^2}{T^2 g}\right) = \gamma\left(\frac{S_a}{g}\right)^2 \quad (6)$$

The admissible solution of Equation (6) gives the required design base shear coefficient $\frac{V_y}{G}$:

$$\frac{V_y}{G} = \frac{-\alpha + \sqrt{\alpha^2 + 4\gamma(S_a/g)^2}}{2} \quad (7)$$

$$\alpha = h\frac{8\theta_p\pi^2}{T^2 g} \quad (8)$$

where,

α dimensionless parameter

$$h = \sum_{i=1}^{N}(\lambda_i h_i)$$

1.2 Lateral force distribution

For the performance design, the lateral force mode should be derived from the nonlinear dynamic structural analysis and should have been verified. Based on the structural nonlinear analysis, Lee [8] calculated the story shear distribution coefficient and regarded the coefficient as the lateral force distribution for the steel chevron braced frame structure in elastic-plastic state. The use of above lateral force distribution will permit the designed structure to experience a more uniform story drift ratio under major earthquakes. The distribution can accurately estimate the maximum required bending moment of column ends and can consider the effects of higher modes for tall steel structures.

$$\beta_i = \frac{V_i}{V_n} = \left(\frac{\sum_{i=1}^{n} G_j h_j}{G_n h_n}\right)^{0.75T-0.2} \quad (9)$$

$$F_i = (\beta_i - \beta_{i+1})\left(\frac{G_n h_n}{\sum_{j=1}^{n} G_j h_j}\right)^{0.75T-0.2} V_y \quad (10)$$

where,

G_j weight of story j

G_n weight at the top story

h_j the height of story j from the base

F_i the lateral force of story i

β_i shear distribution coefficient of story i

β_{i+1} shear distribution coefficient of story, $\beta_{n+1} = 0$

1.3 Member design of steel braced frame

1.3.1 Design of bracing members (Designated yield component)

In the PBPD method, the design of braced members needs to meet three criteria: strength criterion, fatigue life and compactness criterion. From the viewpoint of strength, when the strength distribution follows the design shear distribution along the height of building, it can reduce the inelastic deformations concentrated at few stories as possible. In the design, it is assumed that the bracing members resist the total shear of design story and the contribution of column is ignored. As the designated yield component, it is supposed that the chevron braces will reach the limit state under major earthquake. The tensile brace was designed by yield capacity and compressive brace was designed by post-buckling capacity under cyclic deformation,

$$\left(V_{storyshear}\right)_i \le \left(P_y + 0.3P_{cr}\right)_i \cos\alpha_i \quad (11)$$

where,

$V_{storyshear}$ story shear at story i shear of equivalent single span frame
P_y yield capacity of the brace member
P_{cr} buckling capacity
α_i the angle between the brace and horizontal plane

In order to prevent the steel brace premature fracture of braces due to the low-cycle fatigue cracking of the local yielding position, the fatigue life of braces should be checked. The literature [1] indicated that brace would meet the requirement of low-cycle fatigue fracture properties when the sectional compactness met the related requirements.

For the compactness requirement, the width-thickness ratio of plate can be checked by the Code for seismic design of buildings.

1.3.2 Design of non-designated yielding component

The non-yielding component (including the beam and column) should be proportioned through the capacity method, for example non-yielding component must resist design gravity load and the unbalanced force due to bracings in the limit state.

(1) Design of beam

Owing to the configuration particularity of V-type or chevron brace, the vertical unbalanced force between the tensile and compressive bracings will be applied to the transverse beam connected to the bracings, and should taken into consideration. For the V-type or chevron bracing, the beam should be designed to support the vertical and horizontal unbalanced forces resulting from the tensile and compressive brace. For this purpose, the supporting pressure and tension are assumed to be $0.3P_{cr}$ and P_y respectively (Fig. 2). Additionally, in the design of transverse beam intersected with bracings, supposing that the brace does not carry any gravity, the beam and column are connected by the shear splices, so the beam can be reduced to a simply supported beam. For the sake of large axial force, the beam should be designed to satisfy the design beam-column requirements. The lateral supports with the minimum spacing of L_{pd} should be placed. The unbalanced force induced by bracings is:

$$F_h = \left(P_y + 0.3P_{cr}\right)\cos\alpha \tag{12}$$

$$F_v = \left(P_y - 0.3P_{cr}\right)\sin\alpha \tag{13}$$

where,

F_h horizontal unbalanced force
F_v vertical unbalanced force

(2) Design of column

Due to the pinned beam-to-column connections, the beams cannot almost transfer any bending moment to column, so the design of column should just consider the axial load. The vertical axial force of column mainly comes from the gravity load and the vertical component of support force. The design of column needs to consider two limit states.

1) Pre-buckling limit state

Prior to brace buckling, no unbalanced force occurs in the beam. The design axial force for a typical exterior column (Fig. 3a) is:

$$P_u = (P_{transverse})_i + (P_{beam})_i + (P_{cr}\sin\alpha)_{i+1} \tag{14}$$

where,

$(P_{transverse})_i$ the tributary factored gravity load on columns from the transverse direction at level i;
$(P_{beam})_i$ the tributary factored gravity load from the beam at level i;
$(P_{cr})_{i+1}$ the buckling force of brace at i+1 level.

For a typical interior column, the axial force demand (Fig. 3b) is:

$$P_u = (P_{transverse})_i + \sum (P_{beam})_i + (P_{cr}\sin\alpha)_{i+1} \tag{15}$$

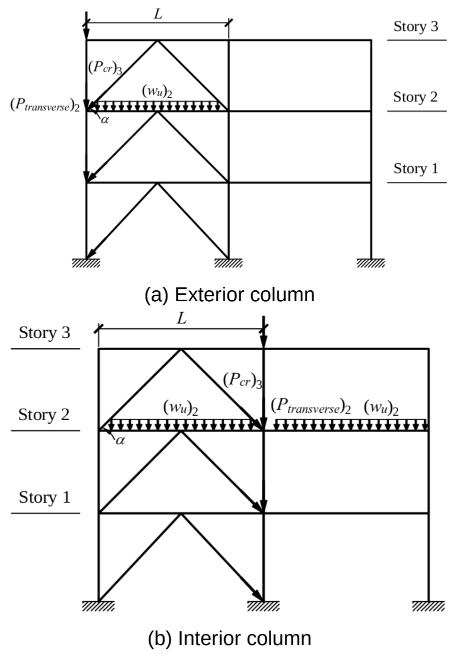


Fig. 3. Axial force components for brace pre-buckling limit state

2) Post-buckling limit state

When the chevron-type bracing attains its ultimate state, the unbalanced force occurs in the beam. The axial force demand of typical exterior columns (Fig. 4a) is:

$$P_u = (P_{transverse})_i + (P_{beam})_i + (0.3P_{cr}\sin\alpha)_{i+1} + \frac{1}{2}F_v \tag{16}$$

where,

F_v unbalanced vertical forces

Likewise, the axial force demand in a typical interior column (Fig. 4b) is:

$$P_u = (P_{transverse})_i + \sum (P_{beam})_i + (0.3P_{cr}\sin\alpha)_{i+1} + \frac{1}{2}F_v \tag{17}$$

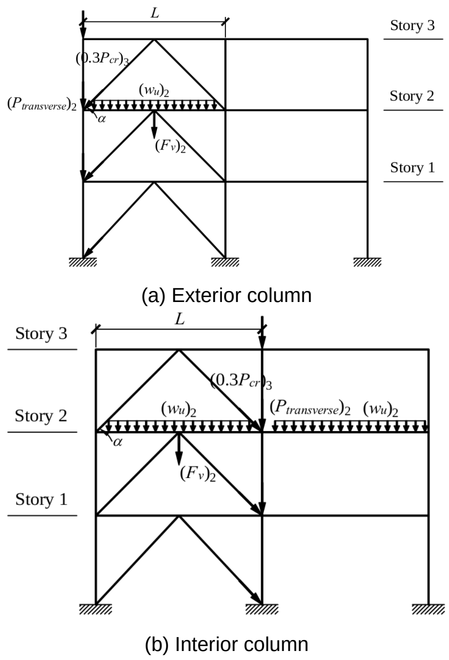


Fig. 4. Axial force components for brace post-buckling limit state

The axial force demand can be determined by pre-buckling and post-buckling limit state. It is noted that the above approach assumes that all bracings reach the limit state simultaneously. Maybe this assumption is conservative, especially for the design of low-story column in high-rise buildings.

The PBPD design flowchart of steel concentrically braced frames was shown in Fig. 5.

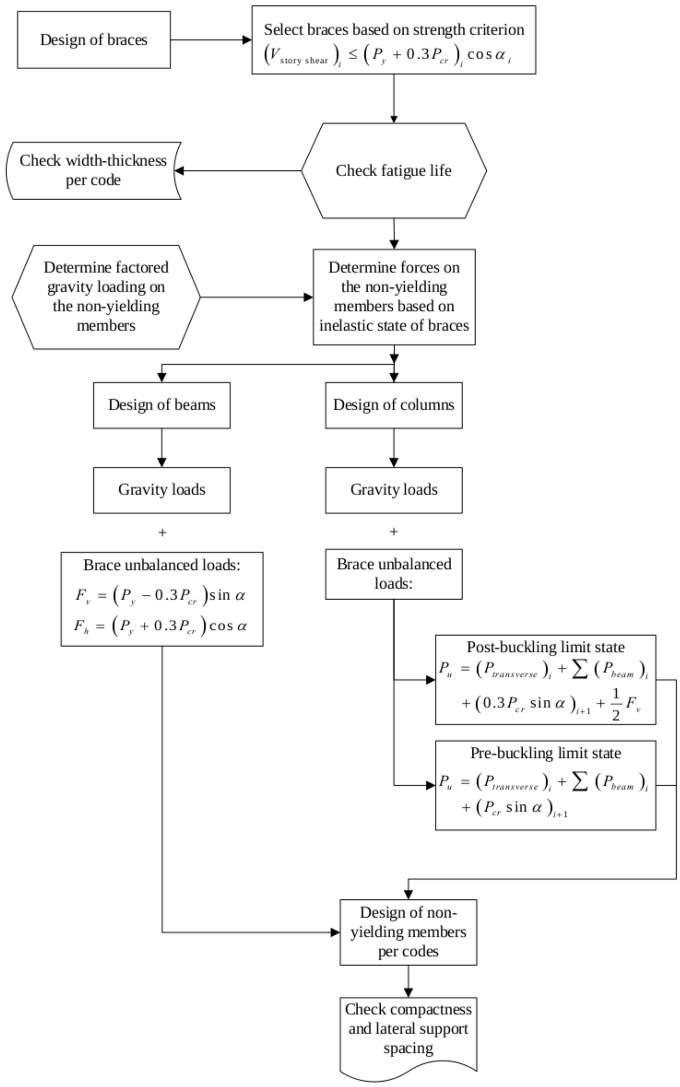


Fig. 5. Performance-based plastic design flowchart for CBF: element design

2 Example and analysis

2.1 Project overview

The project is a steel chevron braced frame structure with six stories and three spans. The story height is 3.3m. The floor dead (live) load is 4.0(2.0) kN/m^2. The roof dead (live) load is 4.5(2.0) kN/m^2. The snow load is 0.5 kN/m^2. The seismic fortification intensity is 8 degree (0.2 g). The site condition is type II. The design earthquake classification is the 2nd group. The plan of the braced frame is given in Fig. 6. The structural calculation diagram is shown in Fig. 7.

The welded H-shaped sections are selected for both beams and columns and the steel is Q235-B.F.

2.2 Design base shear and lateral force distribution

(1) Estimate fundamental period

According to load code for the design of building structures (GB50009-2012), the fundamental period of the structure can be estimated as:

$$T = 0.10\,n = 0.10 \times 6 = 0.60\,s$$

(2) Determine the yield drift ratio and target drift ratio

According to the document [1], the yield drift ratio of CBFs can be obtained by the shear and flexural component of yield drift ratio, namely,

$$\theta_y = \theta_{y,flex} + \theta_{y,shear} \tag{18}$$

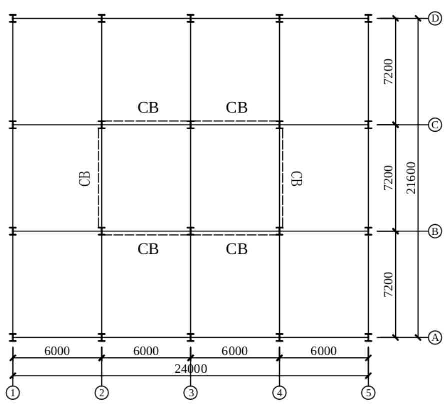


Fig. 6. Plan view of structure

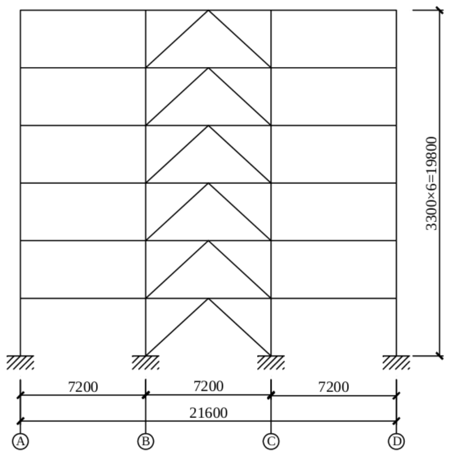


Fig. 7. Calculation chart of structure

$$\theta_{y,flex} = 0.42\varepsilon_y \frac{h}{L} \tag{19}$$

$$\theta_{y,shear} = \frac{2\varepsilon_y}{\sin(2\alpha)} \tag{20}$$

where,

- θ_y yield drift ratio
- $\theta_{y,flex}$ flexural component
- $\theta_{y,shear}$ shear component
- ε_y yield strain of steel
- h story height of single-story single span CBF
- L span length

According to Equations (18) to (20), the calculated yield drift ratio is shown in Table 1. According to the literature [1], for SCBF the design base shear was determined for two level performance criteria: 1) a 1% maximum story drift ratio for a ground motion hazard with 10% probability of exceedance in 50 years (moderate earthquake); 2) a 1.5% maximum story drift ratio for 2 / 50 event (Major earthquake).

Tab. 1. Design parameters for PBPD CBF

Design parameters	Moderate earthquake	Major earthquake
S_a / g	0.312	0.624
T / s	0.60	0.60
θ_y / %	0.34	0.34
$\theta_{y,flex}$ / %	0.11	0.11
θ_u / %	1.0	1.50
$\theta_{u,eff}$ / %	1.11	1.61
$\left(\theta_p = \theta_{u,eff} - \theta_y\right)$ / %	0.77	1.27
$\mu_s = \theta_{u,eff} / \theta_y$	3.26	3.73
R_u	3.26	3.73
γ	0.519	0.464
α	2.566	4.232
η	1.0	1.0
V / W	0.055	0.119
design base shear V / kN	459	993

(3) Determine the acceleration response spectrum

According to *Code for seismic design of buildings*, the acceleration response spectrum can be obtained as:

$$\begin{aligned}
&S_a = [0.45 + 10(\eta_2 - 0.45)T]\,\alpha_{\max} g \quad (a)\\
&S_a = \eta_2 \alpha_{\max} g, (0.1s \le T \le T_g) \quad (b)\\
&S_a = \left(\frac{T_g}{T}\right)^{\gamma} \eta_2 \alpha_{\max} g, (T_g \le T \le 5T_g) \quad (c)\\
&S_a = \left[0.2^{\gamma}\eta_2 - \eta_1\left(T - 5T_g\right)\right]\alpha_{\max} g, (5T_g \le T \le 6.0s) \quad (d)
\end{aligned} \tag{21}$$

$$\begin{aligned}
\gamma &= 0.9 + \frac{0.05 - \xi}{0.3 + 6\xi}\\
\eta_1 &= 0.02 + \frac{0.05 - \xi}{4 + 32\xi}\\
\eta_2 &= 1 + \frac{0.05 - \xi}{0.08 + 1.6\xi}
\end{aligned}$$

where,

- $\alpha_{\max}$ the maximum seismic coefficient, which can be specified from the current code (GB50011-2010).

(4) Calculate design base shear

On the basis of the above parameters, the design base shear can be calculated from Equation (7). The calculated values of all significant design parameters are listed in Table 1.

(5) Calculate lateral force distribution

The design lateral force distribution as calculated by using Equations (14) to (16) is shown in Table 2.

2.3 Design of components

As the designated yielding member, the bracings should be designed in accordance with strength criterion, fatigue life and compactness criterion as previously mentioned. As the non-designated yielding members, the beam and column should be designed by the capacity method, and the design of column is decided by the post-bulking limit state of bracings.

The braces of specified non-yield component are designed by the energy method. Design column is controlled by the post-buckling limit state. The design parameters and final cross sections are shown in Table 3 to Table 5.

3 Verification by nonlinear analysis

The above results are verified by dynamic time history analysis method. The peak of the earthquake accelerogram in the time history analysis is determined by the current code [3]. The selected earthquake waves are respectively Lanzhou wave 1, Artificial wave 2, Artificial wave 3, Elcentro, Cape Mendocino, Taft, Chichi, Coalinga, Loma and Landers as shown in Table 6. These ten waves vary in their frequency contents. Fig. 8 and Fig. 9 show comparison of maximum interstory drift ratio of SCBF from time-history analyses using appropriately scaled ground motion records representative of moderate earthquake and major earthquake.

Tab. 2. Lateral force distribution calculation

Floor	h_i (m)	G_i (kN)	G_ih_i kN · m	$\sum G_ih_i$ (kN · m)	β_i	$\beta_i - \beta_{i+1}$	$(\beta_i - \beta_{i+1})h_i$	F_i (kN)	V_i (kN)
6	19.8	1348	26690.4	26690.4	1.000	1.000	19.800	342.93	342.93
5	16.5	1400	23100	49790.4	1.679	0.679	11.196	232.70	575.62
4	13.2	1400	18480	68270.4	2.182	0.503	6.643	172.57	748.19
3	9.9	1400	13860	82130.4	2.544	0.362	3.584	124.16	872.35
2	6.6	1400	9240	91370.4	2.779	0.236	1.555	80.78	953.13
1	3.3	1400	4620	95990.4	2.896	0.116	0.384	39.87	993.00
$\sum$		8348	95990.4			2.896	43.162	993.00	

Tab. 3. Required brace strength and selected sections

Floor	α	V_i / cos(α) (kN)	Brace section	$P_y + 0.3P_{cr}$ (kN)	Area (cm^2)	$0.3P_{cr}$ (kN)	P_y (kN)
6	42.5	465.30	H125 × 125 × 4 × 6	498.81	19.5	40.56	458.25
5	42.5	781.04	H125 × 125 × 7 × 9	765.66	29.99	60.89	704.77
4	42.5	1015.19	H140 × 140 × 9 × 10	1004	38.8	92.18	911.8
3	42.5	1183.66	H150 × 150 × 10 × 11	1197.7	45.80	121.4	1076.3
2	42.5	1293.26	H150 × 150 × 10 × 12	1273	48.60	130.88	1142.1
1	42.5	1347.35	H160 × 160 × 10 × 12	1376.34	52	154.34	1222

Table 6 from Fig. 8 and Fig. 9, the maximum interstory drift ratios are comparatively uniform along the height of structure (except the first story) under all the seismic waves, which indicates that the inelastic activity is more evenly distributed over the height and the seismic energy can be dissipated simultaneously by each floor. Unlike the traditional design method, the seismic energy can be dissipated only by one story or several soft stories. The results show that the mean maximum interstory drifts of the PBPD frame are well within the corresponding target values.

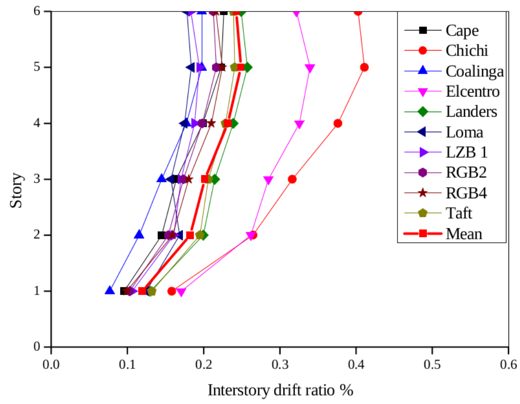


Fig. 8. Maximum interstory drift ratios of PBPD CBF under moderate earthquake

4 Conclusions

1 The PBPD method uses pre-selected target drift and yield mechanism as performance objectives and introduces the inelastic behavior and important performance criterion in design process. So CBFs designed by the PBPD method does not require tedious and repeated iteration performance evaluation.

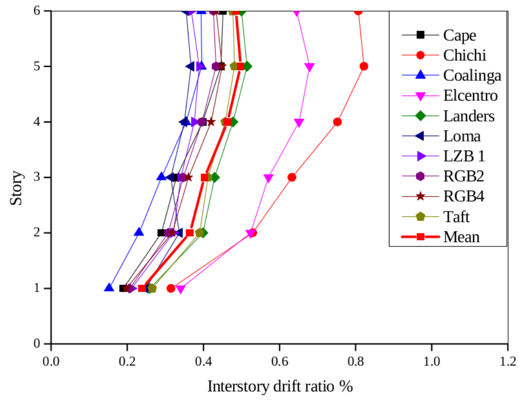


Fig. 9. Maximum interstory drift ratios under major earthquake

2 The seismic performance of PBPD CBFs was assessed by the dynamic time history analysis. The validity of the PBPD design method is demonstrated.

3 The method in this paper can be used to design CBFs under different performance levels and to control the performance of CBFs under frequent intensity, basic intensity, and infrequent intensity of the earthquakes.

Acknowledgements

The author would like to acknowledge the National Natural Science Foundation of China (Nos. 51108035, 51178388 and 10972168), the Natural Science Basic Research Plan in Shaanxi Province of China (No. 2010JQ7006), China Postdoctoral Science Foundation funded project (No. 20100481313) and the Special Fund for Basic Scientific Research of Central College (No. CHD2012ZD012).

Tab. 4. Design parameters for beams

Floor	w_u kN/m	P_y	$0.3P_{cr}$	F_h	F_v	P_u	M_u	Beam section	
								Interior	Exterior
6	28.5	458.25	40.56	367.62	282.36	183.81	656.68	H550 × 250 × 16 × 22	H350 × 260 × 6 × 10
5	30	704.77	60.89	564.29	435.26	282.15	929.37	H600 × 270 × 18 × 24	H350 × 260 × 6 × 10
4	30	911.8	92.18	739.93	554.06	369.97	1134.30	H650 × 300 × 22 × 24	H350 × 260 × 6 × 10
3	30	1076.3	121.4	882.70	645.51	441.35	1292.05	H700 × 300 × 22 × 24	H350 × 260 × 6 × 10
2	30	1142.1	130.88	938.19	683.58	469.09	1357.72	H750 × 300 × 22 × 24	H350 × 260 × 6 × 10
1	30	1222	154.34	1014.36	721.74	507.18	1423.54	H750 × 300 × 24 × 26	H350 × 260 × 6 × 10

Tab. 5. Design of columns

Floor	$P_{trans.}$	P_{beam}	$0.3P_{cr}\sin\alpha$	$0.5F_v$	P_u	P_u(Cumulative)	Column section
6	9	205.2	0	141.18	355.38	355.38	H250 × 250 × 6 × 8
5	24.6	216	27.42	217.63	485.65	841.03	H250 × 250 × 6 × 8
4	40.2	216	41.16	277.03	574.39	1415.42	H340 × 340 × 10 × 12
3	55.8	216	62.31	322.76	656.87	2072.29	H340 × 340 × 10 × 12
2	71.4	216	82.07	341.79	711.26	2783.55	H400 × 400 × 16 × 18
1	87	216	88.47	360.87	752.34	3535.89	H400 × 400 × 16 × 18

Tab. 6. Earthquake wave input

Records	Sequence name	Date	PGA (g)	Duration (s)
1	Lanzhou wave 1	——	0.200	20.000
2	Artificial wave 2	——	0.200	20.000
3	Artificial wave 3	——	0.200	20.000
4	Elcentro	1940.5.18	0.349	30.000
5	Cape Mendocino	1992.4.25	0.163	36.000
6	Taft	1952.7.21	0.225	54.360
7	Chichi	1999.9.20	0.173	60.000
8	Coalinga	1983.5.2	0.147	40.000
9	Loma	1989.10.18	0.195	39.950
10	Landers	1992.6.28	0.109	60.000

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