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A consistent seismic design concept of prestressed concrete buildings

F. Watanabe, M. Nishiyama & H. Muguruma
Architectural Engineering, Kyoto University, Japan

ABSTRACT: A consistent and rational seismic design method of prestressed, partially prestressed and reinforced concrete buildings which has been proposed by the AIJ Task-committee on seismic design of prestressed concrete is introduced. Four design categories are considered. Each is based on the intended structural system and its collapse mechanism of a building to be designed. Different base shear coefficients are given for the structural systems according to the energy dissipating capability of the systems.

1 INTRODUCTION

Ductility-based structural design method (capacity design of ductile structures) was developed for reinforced concrete (Park and Paulay 1974) and it has been adopted to New Zealand Design Codes (SANZ 1984, SANZ 1982). In Japan, the Design Guidelines (AIJ 1990) based on the similar philosophy was also proposed by AIJ (Architectural Institute of Japan) Sub-committee for Seismic Design of Reinforced Concrete.

The seismic design methods in these code and guidelines aim at a building surviving against a severe earthquake by energy absorption of plastic deformation at intended hinge regions. A beam yielding sidesway mechanism has been usually recommended as the most favorable collapse mechanism (see Fig.1). The columns of a building are so designed as to have enough strength to assure the beam hinging mechanism and to remain in elastic range during a strong earthquake motion. Plastic hinges can form in the bottom of the first layer columns and the upper critical part of the top layer columns as well as in the exterior columns subjected to high tensile forces due to overturning moment. Flexural overstrength at beam plastic hinges, two-way frame actions and dynamic effects on column moments or forces are taken into account to estimate the probable maximum column moments or forces to be induced during an earthquake.

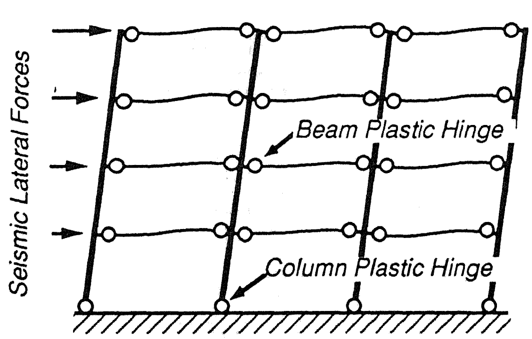


Fig.1 Beam sidesway mechanism

The design method which aims at the beam sidesway mechanism would be applicable even to prestressed concrete buildings. However, the direct application of the beam sidesway mechanism to the design of relatively low rise prestressed concrete buildings with fully prestressed concrete long-span beams would result in impractical and uneconomic design. Prestressing tendons are usually provided to cancel or reduce flexural moments due to dead and live loads. That results in much more strength of the beams than required for the actions due to design seismic loads.

The AIJ Task-committee on seismic design of prestressed concrete has been investigating and discussing a practical and reasonable seismic design method of prestressed concrete buildings. On the basis of the trials of seismic design for prestressed concrete buildings the committee has reached the conclusion that the design procedure to assure a beam hinging mechanism is not always reasonable. Recently the committee has proposed the seismic design method for prestressed concrete building structures, in which a column sidesway system is allowed. For a building which is predicted to collapse in a column sidesway mechanism, the structural safety under seismic load is assured by increasing the lateral seismic design force and by giving the smooth distribution of story shear strength and rigidity along the building height.

In this paper, the seismic design method for prestressed concrete buildings which has been proposed by the AIJ Sub-committee for Seismic Design of Prestressed Concrete is introduced.

2 PROPOSED SEISMIC DESIGN METHOD

2.1 Design category

Four design categories are considered. Each category is based on the intended structural system of a building to be designed.

a) BDF System (Beam-sidesway Ductile Frame Structures) ; The beam hinging sidesway mechanism is intended and should be assured.
b) GDF System (General Ductile Frame Structures) ; The beam hinging sidesway mechanism is not necessarily assured. The column sidesway mechanism is assumed as a probable collapse mechanism. Column critical sections should be designed as potential plastic hinge regions.
c) DWF System (Ductile Wall Frame Structures) ; The collapse mechanism should be initiated by either the flexural yielding at the bottom of walls or the rotational yielding at wall foundations.
d) SRS System (Strength Resisting Structures)

It is assumed that the allowable maximum interstory drift during a strong motion earthquake for BDF, GDF and DWF systems is approximately 0.01 radian. The corresponding member rotation angle of beams and columns is assumed to be 0.02 radian. Thus, plastic hinge regions are required to achieve the rotation of 0.02 radian without significant reductions of load carrying capacities; the elastic deformation of non-hinge region can be ignored as a conservative assumption. Hinge lengths are assumed to be 1.5 times the total depth of beam or column sections.

2.2 Material strength used in structural design

Reliable strength, average strength and over-strength of sections or members are used in the design procedure. The contribution of slab reinforcements to negative moment resistance is taken into account in the calculation of beam flexural over-strengths at plastic hinges. The material strengths for reinforcements specified in Table 1 are used in the calculation of section or member strengths. For concrete, a specified design strength (including an adequate margin) is used in strength calculation.

2.3 Seismic design load

The standard base shear coefficients for ordinary reinforced concrete buildings are assumed as below. In GDF system, the larger base shear coefficient than that for other ductile systems is specified to avoid the excessive interstory drift angle greater than 0.01 radian at the critical story.

a) 0.25 : Beam-sidesway Ductile Frame Structures (BDF System)
b) 0.35 : General Ductile Frame Structures (GDF System)
c) 0.30 : Ductile Wall Frame Structures (DWF System)
d) 0.50 : Strength Resisting Structures (SRS System)

On a basis of the dynamic response analyses (Thompson and Park 1980, Okamoto 1986) and engineering judgements, the standard base shear coefficients for prestressed concrete building structures are given as shown in Table 2 and 3. For example, the base shear coefficient for GDF system with fully prestressed concrete beams is 20 % higher than that for an ordinary reinforced concrete GDF system.

In the current New Zealand design code (SANZ 1984), structural material factor M for ductile prestressed concrete structures is 25 % higher than that for ductile reinforced concrete structures to allow for larger responses of prestressed concrete structures than reinforced concrete. This results in 25 % higher total horizontal seismic design force.

For strength resisting prestressed concrete structures (SRS System), the base shear coefficient of 0.50 is given regardless of a type of the beams, because the primary concern is the strength and no ductile behavior is expected.

Table 1 Material strength to be used in strength calculation

Classification of Strength	Grade 30 Steel Grade 35 Steel	Grade 40 Steel	Prestressing Steel
Reliable Strength	σ_y	σ_y	σ_{py}
Average Strength	$1.10\sigma_y$	$1.10\sigma_y$	$1.15\sigma_{py}$
Over-strength	$1.30\sigma_y$	$1.25\sigma_y$	$1.18\sigma_{py}$

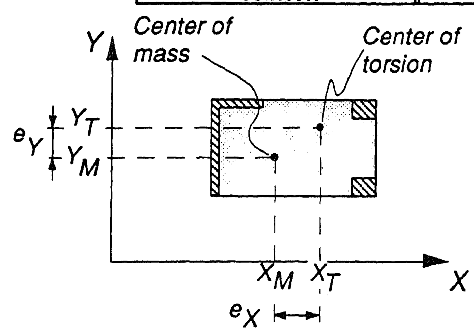
Numerical numbers of the steel grade indicates the specified yield strength in kgf/mm^2
σ_y : Specified yield strength of ordinary reinforcing steel
σ_{py} : Specified yield strength (0.2% off-set) of prestressing steel

Table 2 Base shear coefficient for prestressed concrete structures

Structural System	BDF	GDF	DWF	SRS
C	$0.25\eta_{BDF}$	$0.35\eta_{GDF}$	$0.30\eta_{DWF}$	0.50

Table 3 Magnification factors

Constituent beams	BDF system η_{BDF}	GDF system η_{GDF}	DWF system η_{DWF}
Reinforced concrete	1.0	1.0	1.0
Prestressed concrete	1.2	1.1	1.05
Partially prestressed concrete	1.1	1.05	1.02



$$e_x = [X_M - X_T] \qquad e_y = [Y_M - Y_T]$$

$$R_{ex} = \frac{e_x}{\sqrt{K_T / K_{hx}}} \qquad R_{ey} = \frac{e_y}{\sqrt{K_T / K_{hy}}}$$

$$K_{hx} = \sum_i J_{xi} \qquad K_{hy} = \sum_i J_{yi}$$

$$K_t = \sum_i J_{xi} \cdot y_i^2 + \sum_i J_{yi} \cdot x_i^2$$

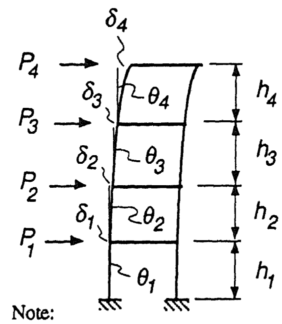
Note:

J_{xi}, J_{yi} : Lateral stiffnesses of vertical structural element i in X and Y directions, respectively.

X_i, Y_i : Coordinates of i-th element measured from the center of torsion.

R_{ex}, R_{ey} : Eccentricity ratios in X and Y directions, respectively.

Fig.2 Eccentricity ratio, Re



$$\theta_j = \frac{\delta_j}{h_j}$$

$$R_{sj} = \frac{\dfrac{1}{\theta_j}}{\dfrac{1}{n}\left[\sum_{i=1}^{n} \dfrac{1}{\theta_i}\right]}$$

Note:

θ_j : Interstory drift of j-th story under seismic design load of the first phase design load.

δ_j : Interstory displacement.

h_j : story height.

R_{sj} : stiffness ratio.

n : Number of stories.

Fig.3 Stiffness ratio, Rs

Table 4 Coefficients Fe and Fs with regard to the eccentricity ratio Re and stiffness ratio Rs

Re	Fe	Rs	Fs
≤0.15	1.0	≥0.60	1.0
0.15~0.30	Linear interpolation	0.30~0.60	Linear interpolation
≥0.30	1.5	≤0.30	1.5

Design seismic story shear force at k-th story, Q_k, is given by Eq.1.

$$Q_k = C_k \cdot W_k \qquad \text{-(1)}$$

$$C_k = Z \cdot R_t \cdot A_k \cdot F_e \cdot F_s \cdot C \qquad \text{-(2)}$$

$$R_t = 1\text{-}0.2(T/0.8\text{-}1)^2 \text{ : soft subsoil} \qquad \text{-(3a)}$$

$$R_t = 1\text{-}0.2(T/0.6\text{-}1)^2 \text{ : medium subsoil} \qquad \text{-(3b)}$$

$$R_t = 1\text{-}0.2(T/0.4\text{-}1)^2 \text{ : hard subsoil} \qquad \text{-(3c)}$$

$$A_k = 1+2T\left(\frac{1}{\sqrt{\alpha_k}} - \alpha_k\right)(1+3T) \qquad \text{-(4)}$$

$$\alpha_k = W_k/W \qquad \text{-(5)}$$

Q_k : design seismic story shear force at k-th story
W : total weight of the building
W_k : weight of the building above k-th story
Z : seismic hazard zoning coefficient, $0.7 \le Z \le 1.0$
T : fundamental period of vibration of the building in second
R_t : design spectral coefficient which depends on the subsoil profile and the period of vibration of the building, and $R_t \ge 0.25$ (see Fig.2)
A_k : lateral shear distribution factor at k-th story
F_e : coefficient of structural eccentricity at k-th story, $1.0 \le F_e \le 1.5$ (see Fig.2 and Table 4)
F_s : lateral stiffness coefficient at k-th story, $1.0 \le F_s \le 1.5$ (see Fig.3 and Table 4)
C : base shear coefficient given by Table 2

2.4 Structural analysis

Linear elastic analysis for the specified static seismic design story shear force given by Eq.1 can be applied to evaluate the maximum effects. In the frame analysis, an adequate relative stiffness for each member shall be assumed. If necessary the effect of flexural cracking is considered. Moment re-distribution can be considered. However, the amount should be limited in slightly smaller than that for ordinary reinforced concrete structures, because rotational capacity of prestressed concrete beams is considered to be smaller than that for ordinary reinforced concrete.

2.5 Design of members

In the seismic design, the most unfavorable load combinations should be considered. The loads to be considered are secondary stresses due to prestressing force (U), dead load (D), live load (L) and specified seismic design load (E) given by Eq.1.

a) Beams

Design for flexure
Reliable flexural strength of a beam section shall be equal to or greater than the design moment as given by Eq.6 for all structural systems.

The potential beam hinge regions shall behave in ductile manner without significant reduction of load carrying capacity up to the required rotation angle of 0.02. Therefore, an adequate amount of transverse reinforcement should be provided in potential plastic hinge regions to avoid the buckling of compression reinforcements and the premature crushing of concrete under reversed cyclic earthquake loading.

$${}_BM_R \geq {}_BU_M + {}_BD_M + {}_BL_M + {}_BE_M \qquad \text{-(6)}$$

${}_BM_R$: Reliable flexural strength of beam section
${}_BU_M$: Related beam moment due to prestressing
${}_BD_M$: Related beam moment due to dead load
${}_BL_M$: Related beam moment due to live load
${}_BE_M$: Related beam moment due to specified earthquake load

Design for shear
In the shear design of beams, following equations shall be satisfied.

Beams without hinges:

$${}_BQ_R \geq {}_BU_Q + {}_BD_Q + {}_BL_Q + {}_BE_Q \qquad \text{-(7)}$$

Beams with hinges:
For hinge region

$$\phi_B \cdot {}_BQ_R \geq ({}_BM_O + {}_BM_O')/L_B + {}_BU_Q + {}_BD_Q + {}_BL_Q \qquad \text{-(8)}$$

For non-hinge region

$${}_BQ_R \geq ({}_BM_O + {}_BM_O')/L_B + {}_BU_Q + {}_BD_Q + {}_BL_Q \qquad \text{-(9)}$$

${}_BQ_R$: Reliable shear strength of beams
${}_BU_Q$: Related beam shear due to prestressing
${}_BD_Q$: Related beam shear due to dead load
${}_BL_Q$: Related beam shear due to live load
${}_BE_Q$: Related beam shear due to specified earthquake load
ϕ_B : Shear strength reduction factor for hinge regions of beams to assure the required hinge rotation indirectly : tentatively ϕ_B = 0.90
${}_BM_O$, ${}_BM_O'$: Flexural overstrengths of plastic hinges at either of beam ends
L_B : Clear span length of beams

b) Columns

Design for flexure
In a building to be designed as BDF system, reliable flexural strength of column section shall be equal to or greater than the design moment as given by Eqs.10 and 11.

The potential hinge regions in columns shall behave in a ductile manner without significant reduction in load carrying capacity up to the required rotation angle of 0.02. Thus, an adequate amount of confining reinforcement should be provided in potential plastic hinge regions to avoid the buckling of compression reinforcements and the premature crushing of concrete under reversed cyclic earthquake loading.

For hinge region:

$${}_CM_R \geq {}_CU_M + {}_CD_M + {}_CL_M + {}_CE_M \qquad \text{-(10)}$$

For non-hinge region:

$${}_CM_R \geq \beta \cdot \lambda \cdot \ {}_CM_O \qquad \text{-(11)}$$

In a building to be designed as GDF system, Eq.12 shall be satisfied. Eq.12 gives not only the minimum required flexural strength of column sections but also the maximum allowable flexural strength of column sections. Eq.12 is intended to give the building the smooth distribution of lateral story shear strength along the building height. In GDF system, column sidesway mechanism might be formed, which can result in the damage significantly concentrating into the weakest story. To avoid such an unfavorable response, both upper and lower limits of story shear strength needs to be introduced. The maximum allowable value of γ is tentatively 0.30 on the basis of the dynamic response analyses on several types of buildings in which column sidesway mechanism was predicted.

To assure the required ductility of columns (0.02 radian in a member rotation angle) adequate amount and arrangement of transverse reinforcement should be provided in potential plastic hinge regions according to design axial load.

$$_{C}U_{M}+{}_{C}D_{M}+{}_{C}L_{M}+(1+\gamma){}_{C}E_{M} \geq {}_{C}M_{R}$$
$$\geq {}_{C}U_{M}+{}_{C}D_{M}+{}_{C}L_{M}+{}_{C}E_{M} \qquad -(12)$$

$_{C}M_{R}$: Reliable flexural strength of column section
$_{C}U_{M}$: Related column moment due to prestressing
$_{C}D_{M}$: Related column moment due to dead load
$_{C}L_{M}$: Related column moment due to live load
$_{C}E_{M}$: Related column moment due to earthquake load
$_{C}M_{O}$: Column moment induced by the beams framing into the beam-column joint. The beam bending moments are assumed to reach the flexural over-strength and the moments induced by the beams are transferred into the columns on the basis of the results of the elastic linear frame analysis
β : Coefficient for two way frame action (AIJ 1990)
λ : Dynamic magnification factor (AIJ 1990)
γ : Coefficient to limit the distribution of story shear strength along building height (a tentative value is 0.3)

Design for shear
In shear design of columns in a building to be designed as BDF system, the following equations shall be satisfied.

Columns without hinges:

$$_{C}Q_{R} \geq \beta \cdot \lambda \cdot \ {}_{C}Q_{U} \qquad -(13)$$

Columns with hinges:

For hinge region

$$\phi_{C} \cdot {}_{C}Q_{R} \geq ({}_{C}M_{O}+{}_{C}M_{O}')/L_{C} \qquad -(14)$$

For non-hinge region

$$_{C}Q_{R} \geq ({}_{C}M_{O}+{}_{C}M_{O}')/L_{C} \qquad -(15)$$

In shear design of columns in a building to be designed as GDF systems, the following equations shall be satisfied.

For hinge region

$$\phi_{C} \cdot {}_{C}Q_{R} \geq ({}_{C}M_{O}+{}_{C}M_{O}')/L_{C} \qquad -(16)$$

For non-hinge region

$$_{C}Q_{R} \geq ({}_{C}M_{O}+{}_{C}M_{O}')/L_{C} \qquad -(17)$$

For SRS system, the following equation shall be satisfied.

$$Q_{R} \geq {}_{C}U_{Q}+{}_{C}D_{Q}+{}_{C}L_{Q}+{}_{C}E_{Q} \qquad -(18)$$

$_{C}Q_{R}$: Reliable shear strength of columns
$_{C}Q_{U}$: Shear forces induced in columns by adjacent beam end moments through beam-to-column joints, where moments of resistance at beam ends should be assumed to reach flexural over-strengths
$_{C}U_{Q}$: Related shear force due to prestressing
$_{C}D_{Q}$: Related shear force due to dead load
$_{C}L_{Q}$: Related shear force due to live load
$_{C}E_{Q}$: Related shear force due to earthquake load
$_{C}M_{O}$, $_{C}M_{O}'$: Flexural over strengths of plastic hinges at column top and bottom sections
ϕ_{C} : Shear strength reduction factor for hinge regions of columns to assure the required hinge rotation indirectly ; tentatively $\phi_{C}=0.85$
L_{C} : Clear height of columns

Design axial load
Design axial loads for columns are obtained from a linear frame analysis described in section 2.4, excepting BDF system.

In a structural design of BDF system, design axial loads of columns are basically evaluated on the basis of beam flexural over-strengths. The flexural over-strength is assumed to be developed at each potential plastic hinge region of beams. However the full development of beam over-strength at every potential plastic hinge region in a high-rise building is not so determinate due to the effects of higher mode vibrations. In such case, some reduction in design column axial load can be introduced. That is because the flexural strengths at critical column sections are likely to be overestimated when design axial load greater than the actual axial force responses is estimated. Thus, the reduction factor R_{V} in New Zealand Code (SANZ 1982) can be referred. When two way frame action has to be considered, 50 % of axial load induced by perpendicular adjacent beams through beam-column joints shall be added to design column axial load.

2.6 Design of beam-column joint

Premature shear failure of beam-column joints and excessive slippage of beam longitudinal bars from beam-column joints shall be avoided. Therefore, the careful detailing of tendon anchorages and the adequate development length and anchorage details of beam longitudinal reinforcement shall be considered in addition to the assurance of enough shear strength of beam-column joint.

2.7 Design Requirements for Walls in DWF System

In a building to be designed as DWF system, the development of collapse mechanism must be associated with the flexural yielding at wall bottom or the rotational yielding of wall foundations. The rotational yielding means that the overturning moment at the wall foundations reaches the maximum moment of resistance of the wall foundations. The flexural yielding moment and corresponding shear at the end sections of the adjacent foundation beams must be considered. Soil pressure or pile resistances and the gravity load at the wall foundations should be also considered in addition to the flexural strength of the wall itself in the calculation of the maximum rotational resistance of the wall foundations.

Resultant lateral shear resistance of the walls at the first story is assumed to range between 30 % and 70 % of the specified seismic shear force at the first story. When the overturning moment at the wall foundation computed by a linear frame analysis exceeds the rotational yielding moment of the wall foundations, an additional lateral shear strength shall be allocated to the boundary frames. However, the walls shall not yield at the first story when the whole frame is subjected to the seismic design load resulting from the base shear coefficient of 0.2.

The observations of past earthquake damages have indicated non-ductile shear failure of the boundary beams in wall frame structures or coupling beams in coupled shear wall structures due to insufficient web reinforcements. Therefore, for the shear design of boundary beams or coupling beams, the following equation is applied to assure ductile behaviors of them.

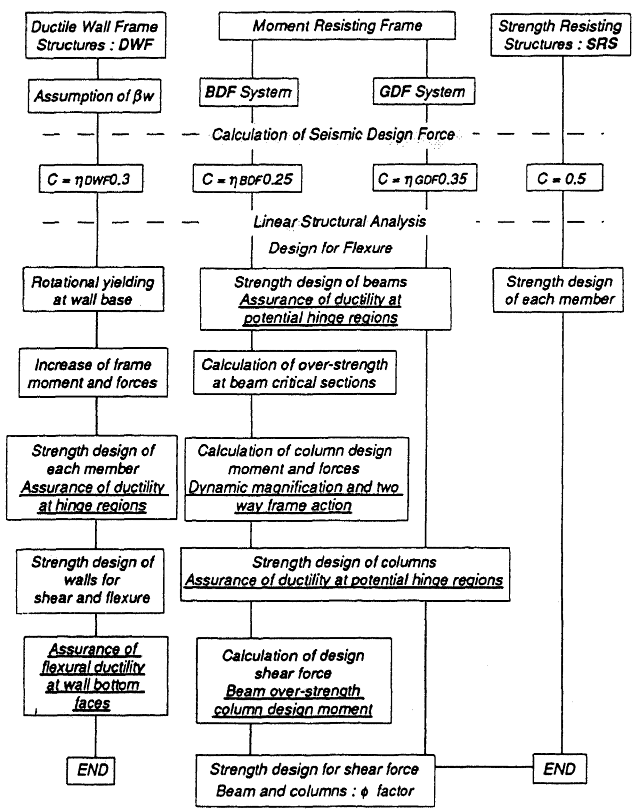
For hinge region

$$\phi_{W1} \cdot {}_{B}Q_{R} \geq ({}_{B}M_{O} + {}_{B}M_{O}')/L_{B} + {}_{B}U_{Q} + {}_{B}D_{Q} + {}_{B}L_{Q} \quad \text{-(19)}$$

For non-hinge region

$$\phi_{W2} \cdot {}_{B}Q_{R} \geq ({}_{B}M_{O} + {}_{B}M_{O}')/L_{B} + {}_{B}U_{Q} + {}_{B}D_{Q} + {}_{B}L_{Q} \quad \text{-(20)}$$

ϕ_{W1}, ϕ_{W2} : Shear strength reduction factors for boundary or coupling beams to assure the required member ductility indirectly; tentatively $\phi_{W1} = 0.8$, $\phi_{W2} = 0.9$



β_w : A ratio of shear resistance of walls to specified seismic story shear at first story. Assumed value of β_w shall be greater than 30 % and smaller than 70 % of specified seismic shear at first story.

Fig.4 Seismic design flow chart for prestressed concrete buildings

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