SEISMIC DESIGN OF PRESTRESSED CONCRETE BUILDINGS

Minehiro Nishiyama[1]

ABSTRACT

The current seismic design procedure for prestressed concrete buildings in Japan is described. The design seismic loads for prestressed concrete buildings provided in NZS 4203:1984 are compared with those in the corresponding Japanese code. Comparisons between prestressed concrete and ordinary reinforced concrete buildings are discussed with regard to design seismic load, dynamic response during earthquake motions and the performance of beam-column joints under reversed cyclic loading. The results of several tests are summarised.

INTRODUCTION

Nowadays higher strength structural materials are being used at construction sites. These materials enable smaller dimensions of members and make possible the design of taller reinforced concrete buildings. However, conventional reinforced concrete cannot make the most use of higher strength materials because serviceability (for example, crack widths of beams and slabs under gravity load) cannot be improved although the ultimate strength of buildings can be higher. In contrast, prestressed concrete can extract the full potential of those materials. Design in prestressed concrete can result in much longer spans of beams without cracking or with small crack widths, and smaller deflections than designs in reinforced concrete.

In this paper, the current seismic design procedure for prestressed concrete buildings in Japan is introduced. The design procedure itself is not a novelty to those who are familiar with the capacity design method developed in New Zealand. In fact they may rather feel it lags behind New Zealand. However, more prestressed concrete buildings have been constructed in Japan than in New Zealand. This paper is also intended to bring the attention of structural designers and other engineers in New Zealand to the option of using prestressed concrete in buildings. Some advantages of prestressed concrete buildings over ordinary reinforced concrete buildings will be illustrated.

The seismic design procedure introduced in this paper is applied to the design of prestressed concrete buildings, not structures like bridges, although they are essentially the same. In fact, they are designed using the same design code in most countries except Japan. In Japan, a separate code exists for the design of prestressed concrete bridges.

[1] Research Associate,
Department of Architectural Engineering,
Kyoto University, Japan

CURRENT SEISMIC DESIGN PROCEDURE FOR PRESTRESSED CONCRETE BUILDINGS IN JAPAN

Code Approaches

The Standard for Structural Design and Construction of Prestressed Concrete Structures [1] was issued by Architectural Institute of Japan (AIJ) in 1961 and the structural design of prestressed concrete buildings has been based on the strength design method. The design of reinforced concrete buildings had been based on allowable stress design until a drastic revision was made in 1981. The current seismic design method for prestressed concrete buildings was also revised in 1981 by taking into account some innovation provisions of the revised design and loading code for reinforced concrete buildings.

The design procedure for prestressed concrete structures issued in 1961 and revised in 1981 is divided into some options with respect to the height of the building. For a building higher than 60m the special permission of the Minister of Construction is required. For a building height equal to or less than 60m, there are six options which a designer can consider. In this paper, two typical options which can be applied to a building lower than or equal to 60m are described.

Figure 1 is a flow diagram of the design procedure, showing two alternatives. The structural design of both alternatives is divided into two phases. One is the first phase design based on the allowable stress design concept with linear elastic analysis. The other is the second phase design based on the strength design method with specified strengths of materials or capacity design based on overstrength of materials. The first phase design is for checking

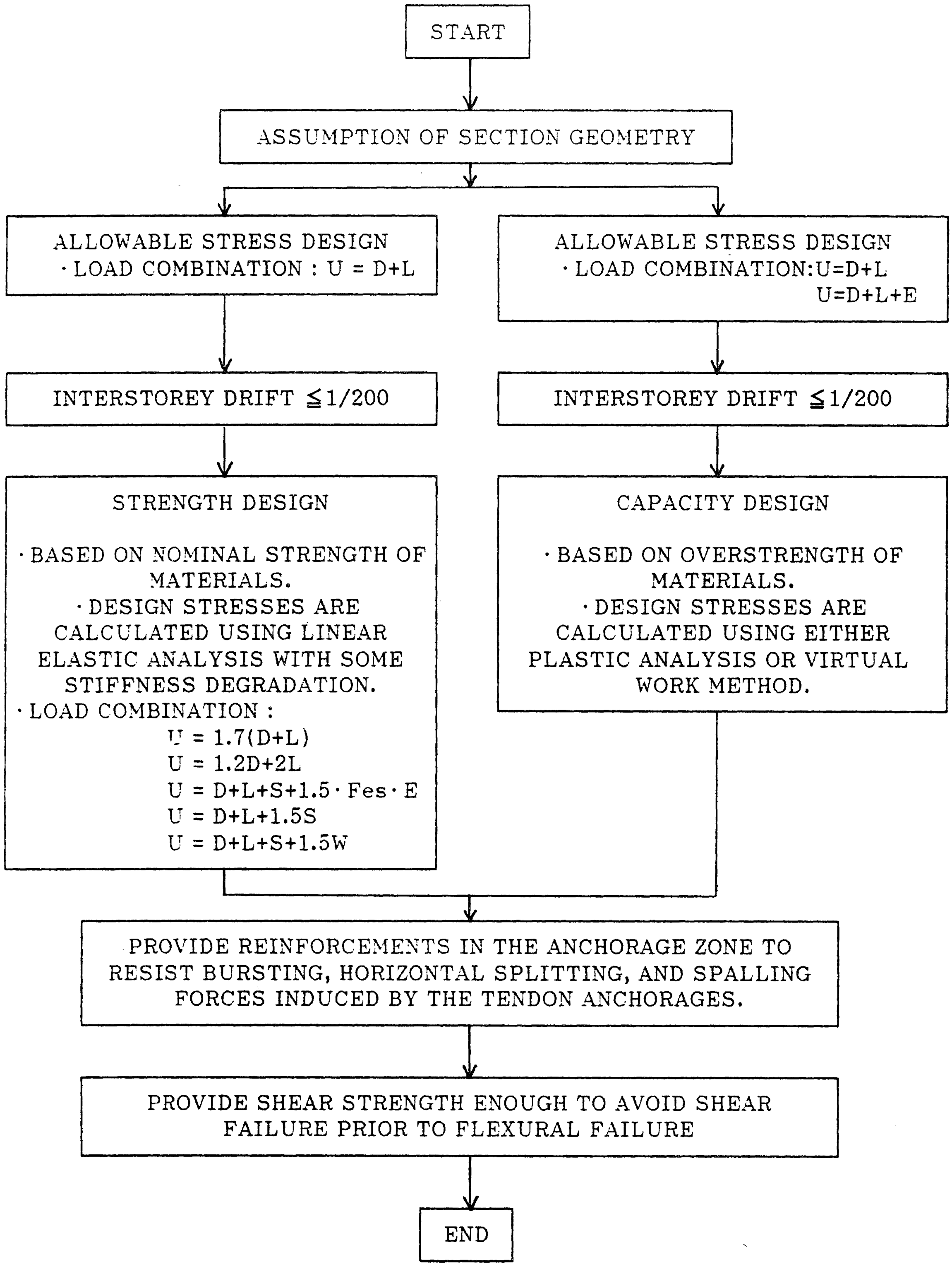


FIGURE 1 FLOW DIAGRAM FOR THE DESIGN PROCEDURE

serviceability, and the second is for confirming that the building has a greater ultimate strength than is required by the design seismic load. Prestressing is primarily introduced to improve serviceability by reducing the crack widths and deflections of beams and slabs. However, excessive prestressing may result in unsatisfactory behaviour at service loads such as excessive camber. Therefore, the checking of serviceability is inevitable.

In the first phase design, the service load combination is D+L, where D and L denote the service dead and live load, respectively. The service snow load, S, may be added as well, if necessary. Members are designed either not to crack (fully prestressed concrete) or to meet the specified crack width limit (partially prestressed concrete) under this load combination. Deflections of members are expected to be within acceptable limits. Interstorey drift calculated for the design earthquake load of D+L+E by elastic analysis shall be smaller than or equal to 1/200, where E denotes the seismic design load due to lateral shear force Q_i given by Eq.6. This limitation may be eased to 1/120 where it is confirmed that non-structural elements may not be damaged so seriously. When the second phase design is carried out using the capacity design method, an allowable stress design is also required for the seismic load combination of D+L+E. However, this allowable stress design can be skipped over when the second phase design by the strength design method is chosen because a building which is designed according to this method always meets this allowable stress design requirement.

The second phase design is supposed to be carried out to confirm that a building designed according to the first phase design has the ultimate strength equal to or greater than the seismic design load based on possible severe earthquake motions. Practically, when designing relatively large buildings in which earthquake loading is dominant, structural designers first may conduct the second phase design and then confirm that the members meet the first phase design requirements.

Second Phase Design by Strength Design

This design method is applied to a building lower than or equal to 31m. In the ultimate strength design procedure, the flexural and shear strengths of members are calculated using nominal strengths of materials. The design actions in the members under the factored design load are calculated using linear elastic analysis. The design loads, U, shall be not less than whichever of the following load combinations is applicable and gives the greatest effect:

$$U = 1.7(D+L) \qquad (1)$$
$$U = 1.2D+2L \qquad (2)$$

$$U = D+L+(S)+1.5.F_{es}.E \quad \text{(with earthquake)} \qquad (3)$$
$$U = D+L+1.5S \quad \text{(with snow)} \qquad (4)$$
$$U = D+L+S+1.5W \quad \text{(with wind)} \qquad (5)$$

In Equation (3), E denotes design load due to lateral shear force Q_i, which is given by

$$Q_i = W_i . C_i \qquad (6)$$

where W_i = gravity load above the i-th storey,
C_i = lateral seismic shear coefficient of the i-th storey which is given by the following equation.

$$C_i = Z.R_t.A_i.C_o \qquad (7)$$

where Z = seismic hazard zoning coefficient and varies between 0.8 and 1.0,
R_t = design spectral coefficient which depends on a subsoil profile and a natural period of vibration of a building, and R_t is given by Equation (8).

$$R_t = \begin{cases} 1 & T<T_c \\ 1 - 0.2\,(T/T_c-1)^2 & T_c \le T \le 2T_c \\ 1.6T_c/T & 2T_c<T \end{cases} \qquad (8)$$

where T is a period of first mode of a building and T_c is a factor with respect to a subsoil profile. A longer natural period results in a smaller R_t. R_t ranges between 1.0 to 0.25, and is expressed schematically in Figure 2. Also, A_i = the distribution factor of lateral shear forces along the height. A_i is given by Equation (9).

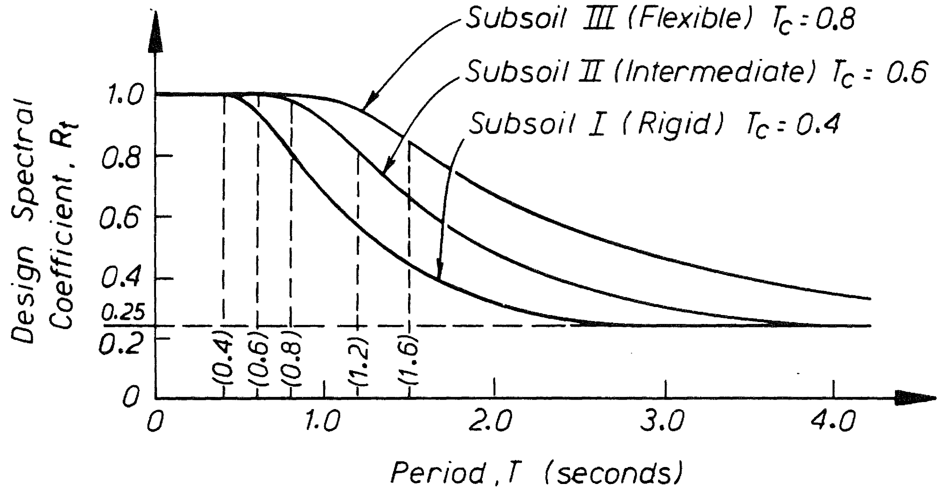
$$A_i = 1 + \left(\frac{1}{\sqrt{\alpha_i}} - \alpha_i\right) \frac{2T}{1+3T} \qquad (9)$$

where α_i is the ratio of the reduced gravity load above the i-th layer to the total reduced gravity load above the level of imposed lateral ground restraint. Also, C_o is the basic seismic coefficient of 0.2, and corresponds to a ground acceleration of about 0.08 - 0.10 g. Recently, it has become popular to express the intensity of an earthquake in terms of the velocity, because the velocity is related directly to the energy. Thus the above acceleration corresponds to a velocity of 25-30 cm/s.

In Equation (3), $F_{es} = F_e . F_s \qquad (10)$

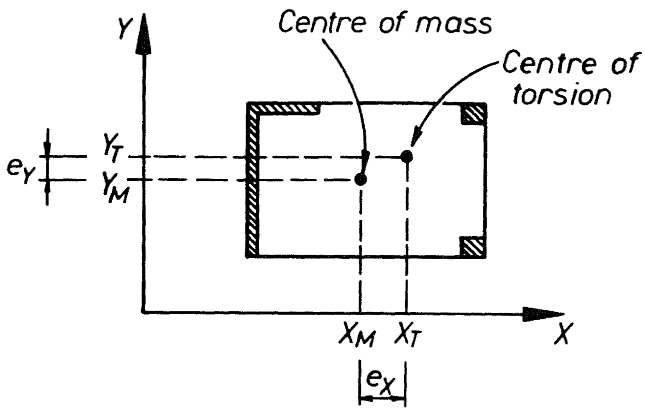
where F_e is a coefficient that is related to the eccentricity ratio R_e in each storey (see Figure 2) and ranges between 1.0 and 1.5. The arrangement of the seismic load resisting elements in a building should be as symmetrical as possible about the centre of mass of the building in order to minimise the torsional response of the building during an earthquake. Also, F_s is a coefficient that is dependent on the stiffness ratio R_s in each storey (see Figure 3) and ranges between 1.0 and 1.5. F_s is introduced because the existence of an extremely flexible storey can lead to a dangerous concentration of damage into the storey. Values for F_e and F_s with regard to R_e and R_s, respectively, are given in Table 1.

F_{es} from Equation (10) varies between 1.0 and 2.25. F_{es} was introduced to provide an extra strength in the case of buildings with an unsymmetrical arrangement of the seismic load resisting elements and/or with extremely flexible stories.



$$R_t \begin{cases} 1 & T < T_C \\ 1-0.2\,(T/T_C-1)^2 & T_C \leq T < 2T_C \\ 1.6\,T_C/T & 2T_C \leq T \end{cases}$$

FIGURE 2 DESIGN SPECTRAL COEFFICIENT, R_t



$$e_X = [X_M - X_T] \qquad e_Y = [Y_M - Y_T]$$

$$R_{ex} = \frac{e_X}{\sqrt{K_t / K_{hx}}} \qquad R_{ey} = \frac{e_Y}{\sqrt{K_t / K_{hy}}}$$

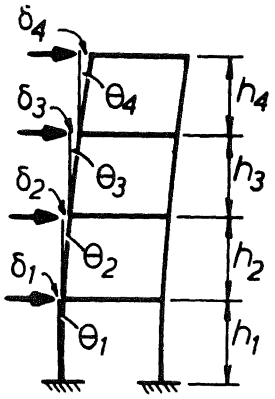
$$K_{hx} = \sum_i J_{xi} \qquad K_{hy} = \sum_i J_{yi}$$

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

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

FIGURE 3 ECCENTRICITY RATIO, R_e



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

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

NOTE:

θ_j : Interstorey drift of j-th storey under seismic design load of the first phase design load.
δ_j : Interstorey displacement.
h_j : Storey height.
R_{sj} : Stiffness ratio.
n : Number of stories.

FIGURE 4 STIFFNESS RATIO, R_s

TABLE 1 COEFFICIENTS F_e AND F_s WITH REGARD TO THE ECCENTRICITY RATIO R_e AND STIFFNESS RATIO R_s, RESPECTIVELY

R_e	F_e	R_s	F_s
≤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

The code does not rely on the plastic deformation of prestressed concrete members and there is no provision for the content of prestressed plus non-prestressed flexural steel to ensure ductile behaviour in plastic zones and on moment redistribution. Prestressed concrete members usually have higher flexural crack loads than reinforced concrete, and they recover to their original states and respond elastically even after they are loaded up to near their ultimate strengths. However, in practice it is unusual for a member section to contain prestressed steel without non-prestressed flexural steel. A reasonable amount of non-prestressed steel in the member section results in hysteresis loops and energy dissipation characteristics which are similar to those obtained from reinforced concrete members. Besides, compression failure in concrete followed by critical reduction in load capacity, which defines the ultimate state typical for prestressed concrete members, can be avoided by appropriate confining of the concrete. A seismic design procedure which accounts for the ductility of members is given in a draft seismic design procedure proposed by the AIJ sub-committee on seismic design of prestressed concrete [2].

Second Phase Design by Capacity Design

The more recent seismic design procedure for reinforced concrete buildings was introduced in 1981, soon after the severe Miyagiken-oki earthquake in 1978. The design procedure is divided into two phases: one is the seismic design for moderate earthquakes and the other is for severe earthquakes. A moderate earthquake is defined as an earthquake which is assumed to happen a few times within the service life of buildings. Buildings are expected to respond to it in an elastic manner and not to be damaged. A severe earthquake is defined as a devastating earthquake which is assumed to possibly happen once in the service life of buildings. Buildings are expected not to collapse but possibly undergo some structural and nonstructural damage.

After the 1981 Code came into force, prestressed concrete buildings could be designed according to either the standard for design and construction of prestressed concrete described before or the new seismic design procedure issued in 1981. This is a reason why there are some options in the design of prestressed concrete buildings. A building higher than 31m and lower than or equal to 60m shall be designed according to this more recent design method.

In Japan, the design procedure issued in 1981 is called capacity design, but this capacity design is quite different from the one described in NZS 3101:1982 [3]. In NZS 3101:1982, energy-dissipating elements of mechanisms are chosen and suitably detailed and other structural elements are provided with sufficient reserve strength capacity to ensure that the chosen energy-dissipating mechanisms are maintained at near their full strength throughout the deformations that may occur. However, in the Japanese design procedure issued in 1981, it is not necessary to consider favourable energy-dissipating mechanisms, although a ductile moment resisting frame is so designed as to avoid brittle failure like shear failure. The building is required to have sufficient ultimate strength to resist severe seismic actions whatever the collapse mechanism may be.

The lateral load resistance in each storey is calculated using inelastic analysis or virtual work method based on the overstrengths of materials. The building is required to have a lateral shear strength greater than the shear force at each storey corresponding to the load combination of $D+L+F_{es}.E'$, where, E' is due to seismic storey shear Q_i', which is given by

$$Q_i' = D_s.W_i.Z.R_t.A_i.C_o \qquad (11)$$

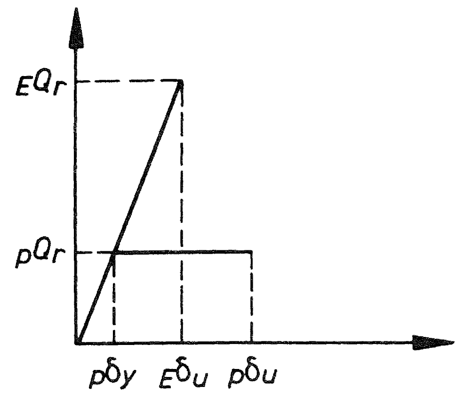
where C_o is the standard base shear coefficient and for the second phase design C_o=1.0. This corresponds to a ground acceleration of 0.30 - 0.40 g and a velocity of approximately 50 cm/s. D_s is the reduction factor which depends on the type and the ductility of the structure. This factor is based on the equal energy concept in which the energy absorbed by a building which yields with elasto-plastic characteristics is assumed to be equal to that of a building which is strong enough to respond elastically, From Figure 5,

$$D_s = 1/\sqrt{(2\mu - 1)}$$

where μ is the allowable displacement ductility factor in each storey. D_s ranges from 0.3 for ductile frames to 0.55 for a building in which a large portion of lateral load is assigned to walls and braces. Other coefficients are the same as described in the previous section.

Comparing the seismic design load of the strength design method in the previous section with that of this alternative design method, it is found that they are identical in the case of prestressed concrete ductile frames. However, the strength design method is based on a linear elastic analysis and nominal material strengths, while the capacity design method is based on plastic analysis and material overstrengths.

In case a designer chooses this option, he has to conduct an allowable stress design for the load combination of U=D+L+E, where E is identical to the load used in Equation (3).



$$D_s = \frac{{}_pQ_r}{{}_EQ_r} = \frac{1}{\sqrt{2\mu-1}}$$

$$\mu = \frac{{}_p\delta_u}{{}_p\delta_y}$$

NOTE:

${}_EQ_r$, ${}_pQ_r$	:	Lateral inertia load responses in structures with elastic and elasto-plastic characteristics, respectively.
${}_EQ_u$, ${}_pQ_u$	:	Maximum horizontal deflections.
${}_p\delta_y$	:	Horizontal deflection at first yield.
μ	:	Ductility ratio.

FIGURE 5 EQUAL ENERGY CONCEPT AND THE REDUCTION FACTOR, D_s

COMPARISON OF SEISMIC DESIGN LOAD, NZ vs JAPAN

It is said that the seismic design load specified in the Japanese code is much larger than that specified in NZS 4203:1984 [4], although the seismicity of both countries is supposed to be almost the same. However, no available information on this matter, to the author's knowledge, has been reported. The NZ Code is based on the equal displacement concept in which the maximum horizontal deflection reached by a building which yields with elasto-plastic characteristics is assumed to be the same as that of a building which is strong enough to respond in the elastic range. The Japanese code is based on the equal energy concept in which the energy absorbed by a building which yields with elasto-plastic characteristics is assumed to be the same as that of a building which is strong enough to respond elastically, but for buildings with a relatively long period of vibration, the Japanese code uses the equal displacement concept. If available ductilities are expected to be the same, the yield force, Q_{yd} and Q_{ye}, required by the equal displacement and equal energy concepts, respectively, are given by the following equations.

$$Q_{yd} = Q_e/\mu \tag{4}$$

$$Q_{ye} = Q_e/\sqrt{(2\mu-1)} \tag{5}$$

where, Q_e is the required strength of an elastically responding building. Therefore, $Q_{ye} \geq Q_{yd}$ for $\mu \geq 1$.

For example, consider a prestressed concrete ductile frame with a symmetrical arrangement of the seismic load resisting elements about the centre of mass and without extremely flexible stories. Let the frame be built on a rigid subsoil in the most hazardous zone. The design loads U in NZS for the strength method with earthquake are given by the following load combinations.

$$U = 1.0D+1.3L_R+E_N \tag{15}$$
$$U = 0.9D+E_N \tag{16}$$

The Japanese code gives the following load combination.

$$U = D+L+1.5.F_{es}.E \tag{17}$$

Here, E_J=1.5.F_{es}.E is regarded as the seismic design load in the Japanese code, while in NZS, E_N is the seismic design load. The effects of seismic loads in relation to the total design loads are different in the two codes because the load factors for dead load and live load are different.

In NZS, the seismic design load is given by the total horizontal seismic force V, while in the Japanese code it is given by the shear force acting on each storey. The distribution of the seismic load along the height of a building in NZS 4203:1984 is a triangular shape with a concentrated load at the top of the building. The Japanese code gives the A_i distribution described in the section on second phase strength design. The Design Guideline for Earthquake Resistant Reinforced Concrete Buildings Based on Ultimate Strength Concept published by AIJ in 1988 [5] have adopted the same distribution as NZS.

The total horizontal seismic force, V, in NZS, and the shear force acting on the first storey, Q_1, in the Japanese code are given by C_d.Wt and C_1.W_1, respectively. Therefore, the seismic design coefficient C_d in NZS can be compared with the base shear coefficient C_1 of the Japanese code, if the total reduced gravity loads above the level of imposed lateral ground restraint W_t in NZS and W_1 in the Japanese code are assumed to be identical. C_d is given by the following equation:

$$C_d = C.R.S.M = 0.8C \tag{18}$$

where, the risk factor R is assumed to be the smallest value, 1.0, because the Japanese code, which does not have a factor corresponding to the risk factor, is supposed to be at least satisfied and a designer can provide further load capacity to a building within the budget if he predicts its failure would lead to an unusually high level of loss. The structural type factor S is taken as 0.8 for a ductile frame and the structural material factor M is taken as 1.0 for a prestressed concrete building. The basic seismic coefficient C depends on the period of vibration of the building, the seismic zone and the subsoil.

C_1 is given by the following equation:

$$C_1=1.5.F_{es}.Z.R_t.A1.C_o = 0.3.R_t \qquad (19)$$

where, F_{es} is taken as 1.0 for a building with a symmetrical arrangement of seismic load resisting elements about the centre of mass and without extremely flexible storeys, Z=1.0 for the most hazardous zone, A1=1.0 for the first storey and C_o=0.2. The design spectral coefficient R_t depends on the period of vibration of the building and the subsoil.

In NZS, the period T shall be established from properly substantiated data, or computation, or both. T may be calculated by the following Rayleigh Formula:

$$T = 2\pi\sqrt{\frac{\Sigma(w_z d_x^2)}{g\Sigma(F_x d_x)}}$$

Therefore, T can be calculated from the real structural stiffness and mass. On the other hand, in the Japanese code, the period in seconds is calculated using the following equation for reinforced concrete buildings:

$$T=0.02.h \qquad (21)$$

where, h is the height of the building in metres. Hence T depends on the height only, and this may lead to a much shorter period than the real period and a larger R_t, although R_t can be reduced by 25% if the period is established from properly substantiated data or computation. This equation indicates that, for example, a 30m-high reinforced concrete building has the period of only 0.6 seconds.

For a comparison of seismic loads, it is assumed that the same periods T are obtained for a building designed to both codes. Then using C=0.15 and R_t=1.0 for buildings with a short period of vibration ($T \le 0.4$ seconds) result in giving C_d=0.12 and C_1=0.3, which means that Q_1 in the Japanese code is 2.5 times larger than V in NZS. For buildings with a long period ($T \ge 3.0$ seconds), C=0.075 and R_t=0.25 result in giving C_d=0.06 and C_1=0.075, which means that Q_1 is 1.25 times larger than V. In the Japanese Code, R_t can be reduced to 0.25 for longer periods than 2.56 seconds in case of rigid subsoil, while in NZS C for a longer period than 1.2 seconds can be reduced to half of the value for a short period.

COMPARISON OF PRESTRESSED CONCRETE WITH REINFORCED CONCRETE

Seismic Design Load

In NZS 4203:1984, the structural material factor M for ductile prestressed concrete buildings is 25% larger than that for ductile reinforced concrete buildings. This 25% higher total horizontal seismic design load for prestressed concrete buildings is to allow for the larger response of prestressed concrete buildings.

However, this provision does not result in a 25% larger seismic design load for prestressed concrete buildings than that for ordinary reinforced concrete unless the prestressed and ordinary reinforced concrete buildings have exactly the same structural configuration of frames and dimensions of members. Normally, prestressed concrete as a structural type can lead to longer spans. Hence prestressed concrete frames may be more flexible than ordinary reinforced concrete frames designed to sustain the same gravity load. Therefore, the natural period of vibration of prestressed concrete frames may be longer than that of reinforced concrete frames of the same height. That may result in smaller seismic design loads for prestressed concrete frames since in current seismic design codes, a smaller seismic design load can be applied to buildings with longer periods.

Consider a simple example involving two buildings. One is a four-storey prestressed concrete building with beams of 21 m span, shown in Fig.6(a). The other is a four-storey three-bay reinforced concrete building with beams of 7 m span, shown in Fig.6(b). The latter has one-third span length of the former, because it is not practical to build a reinforced concrete beam of 21 m length and a 7 m span is considered to be appropriate. These two frames are designed to sustain the same gravity loads and are supposed to be "equivalent" to each other. The member dimensions for these two buildings are summarised in Table 2. The overall depths of the prestressed and reinforced concrete beams are assumed to be 1/20 and 1/10 of span length, respectively. Columns of the prestressed concrete building have larger dimensions than those of the reinforced concrete building because they are subjected to larger actions due to both gravity load and horizontal load than those of the reinforced concrete building. The prestressed concrete beams have a larger modulus of elasticity because they are usually constructed of concrete with higher compressive strength. Here, the prestressed concrete beams are assumed to have concrete with a compressive strength of 30MPa, while the reinforced concrete beams and columns are assumed to have a compressive strength of 20 MPa. The modulus of elasticity is calculated using $E_c=4700\sqrt{f'_c}$. Total gravity load is assumed to be 9.8 kN/m^2 on each floor. The span length of the plane frame perpendicular to the frame considered is assumed to be 7 m. These frames appear to have larger dimensions for the members than is needed in New Zealand, but from a Japanese structural designers' point of view these are considered to be appropriate. The effective member stiffnesses are assumed to be based on 50% of the area and moment of inertia of the gross section of the beams and 80% for the columns. Assumed stiffnesses are listed in Table 3.

Natural periods of the first mode of these

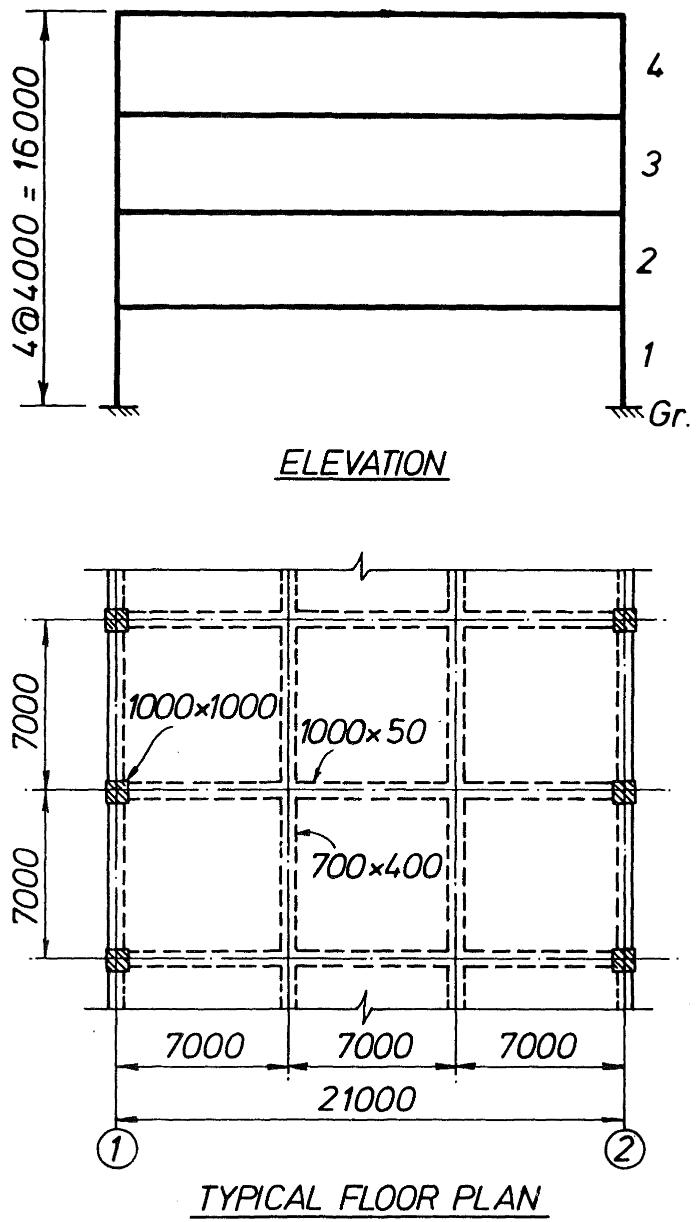


FIGURE 6(a) FOUR STOREY PRESTRESSED CONCRETE BUILDING

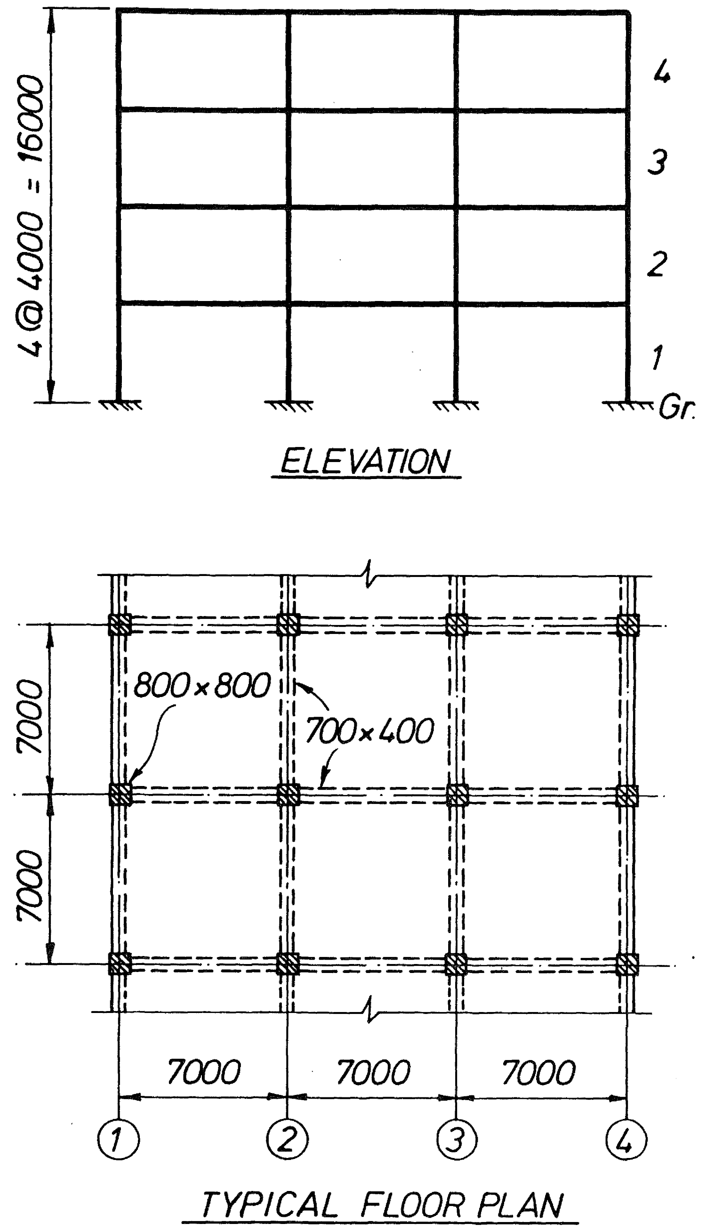


FIGURE 6(b) FOUR STOREY REINFORCED CONCRETE BUILDING

TABLE 2 MEMBER DIMENSIONS FOR FOUR-STOREY PRESTRESSED AND REINFORCED CONCRETE BUILDINGS

Members	Prestressed Concrete Frame (mm)	Reinforced Concrete Frame (mm)
Main Beams	1000x500	700x400
Secondary Beams	700x400	
Columns	1000x1000	800x800

TABLE 3 EFFECTIVE MEMBER STIFFNESSES

Properties	Beams	Columns
Area	$0.5A_g$	$0.8A_g$
Moment of Inertia	$0.5I_g$	$0.8I_g$

TABLE 4 MEMBER DIMENSIONS FOR SEVEN-STOREY FRAME

Members	Floor 1 (mm)	Floors 2 to 3 (mm)	Floor 4 (mm)	Floors 5 to 7 (mm)
Beams	1100x750	1100x750	900x650	800x650
Columns	1080x550	880x550	880x550	880x550

frames are 0.992 second for the reinforced concrete and 1.214 seconds for the prestressed concrete. These result in basic seismic coefficients of C=0.096 for the reinforce concrete frame and C=0.075 for the prestressed concrete, using Figure 3 of Chapter 3 of NZS 4203:1984. Products of these seismic coefficients C and the structural material factor M are 0.077 for the reinforced concrete frame and 0.075 for the prestressed concrete frame. Therefore, if they have the same total gravity load above the level of imposed lateral ground restraint, the seismic design load for the prestressed concrete frame is smaller than for the reinforced concrete frame even after structural material factor M is taken into account. However, it should be noted that a too flexible frame may not meet the interstorey drift limitation of the design code. No allowance of this kind can be made in the Japanese code because the period depends on only the height of the frame.

Dynamic Response

Is the response of prestressed concrete buildings to an earthquake motion really 25% larger than that of reinforced concrete? Past research [6,7] showing a larger response for prestressed concrete by as much as 25% has used a single- or multi-mass shear system. The dynamic analysis using a multi-mass shear system, which can save time and expense, is very useful if the restoring force characteristic of each storey can be modelled appropriately. However, it is difficult to suitably model the restoring force characteristic in each storey because this fluctuates widely with the ratio of the stiffness of the prestressed concrete beam to that of the reinforced concrete column, and the combination of their flexural capacities. Moreover, the restoring force characteristic of a prestressed concrete beam itself fluctuates widely with the amount of prestressing force and the amount of nonprestressed longitudinal reinforcement and prestressing steel bars. The response of prestressed concrete, derived by analysis using a single- or multi-mass shear system appears to be overestimated, although it may be the upper limit.

Two-dimensional dynamic response analyses of seven-storey single-bay buildings of prestressed and reinforced concrete have recently been carried out by the author. The basic dimensions of the buildings are given in Table 4 and Figure 7. These frames were designed according to the design category BDF system, aimed at a beam-sidesway mechanism, in the draft seismic design provision for prestressed concrete buildings proposed by the AIJ Sub-committee [2]. The two frames are identical except that the moment-curvature idealisations of the prestressed and reinforced concrete plastic hinges required were obtained using the modified PS model proposed by Okatomoto [7]. In this model, a coefficient α indicates the degree of prestress where α ranges between 0.2 and 0.8. The values of α = 0.2 and 0.8 are considered to correspond to the moment-curvature models for reinforced and prestressed concrete, respectively.

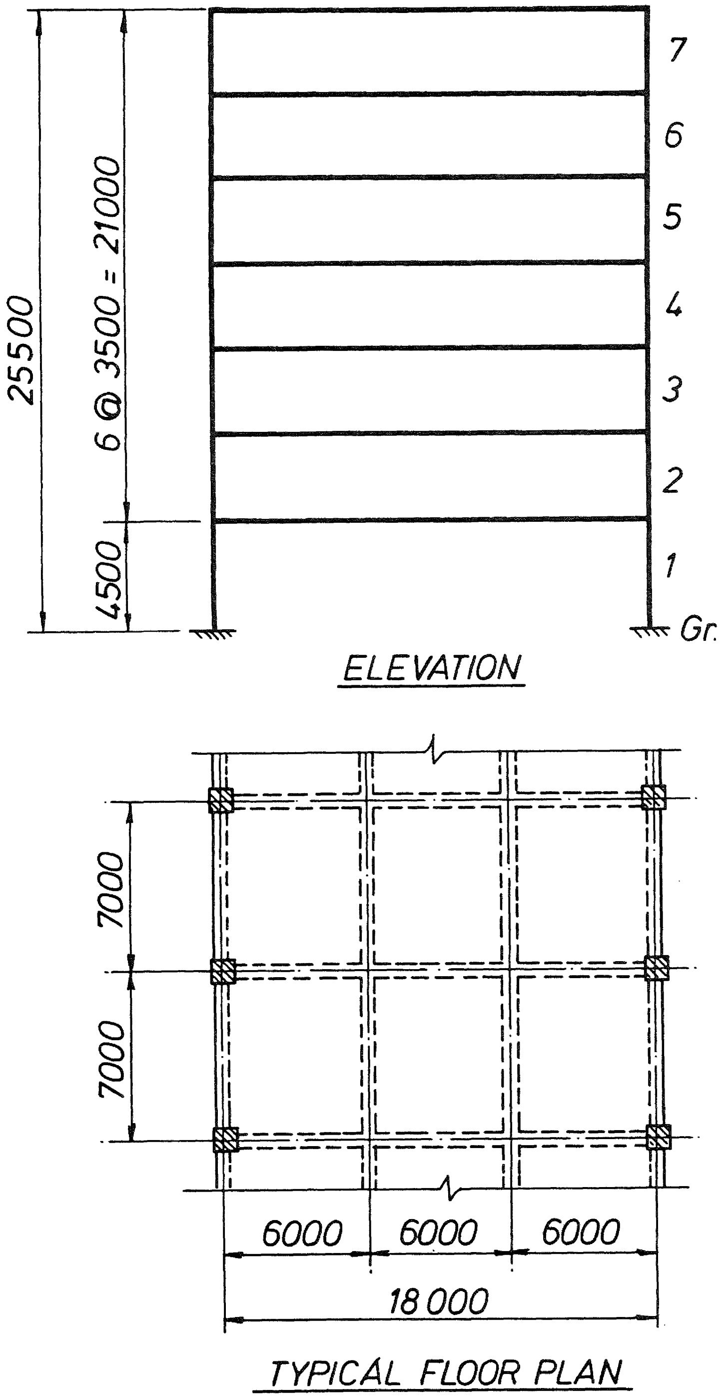


FIGURE 7 SEVEN STOREY MODEL

The ground motion used was that of the first 12 seconds of motion recorded at the ground floor of the building at Tohoku University during the Miyagiken-oki earthquake in 1974. It was amplified to a maximum velocity of 50 cm/s, that is, the maximum acceleration was 357.0 cm/s^2. The time increment used in the analysis was 0.02 seconds.

During the earthquake shaking, plastic hinges formed at the beam ends, the bottom of the first storey column, and the top of the highest layer column, that is, a beam-sidesway mechanism was realised. The following results reveal the differences in the dynamic response of reinforced concrete and prestressed concrete frames, because all the beam ends entered the inelastic range.

Figure 8 shows a time history plot of displacement for the top of the building. The maximum displacement appeared at around 6 second after the ground motion started. The maximum displacement of the reinforced concrete building was 277 mm and was 286 mm for the prestressed concrete building. It

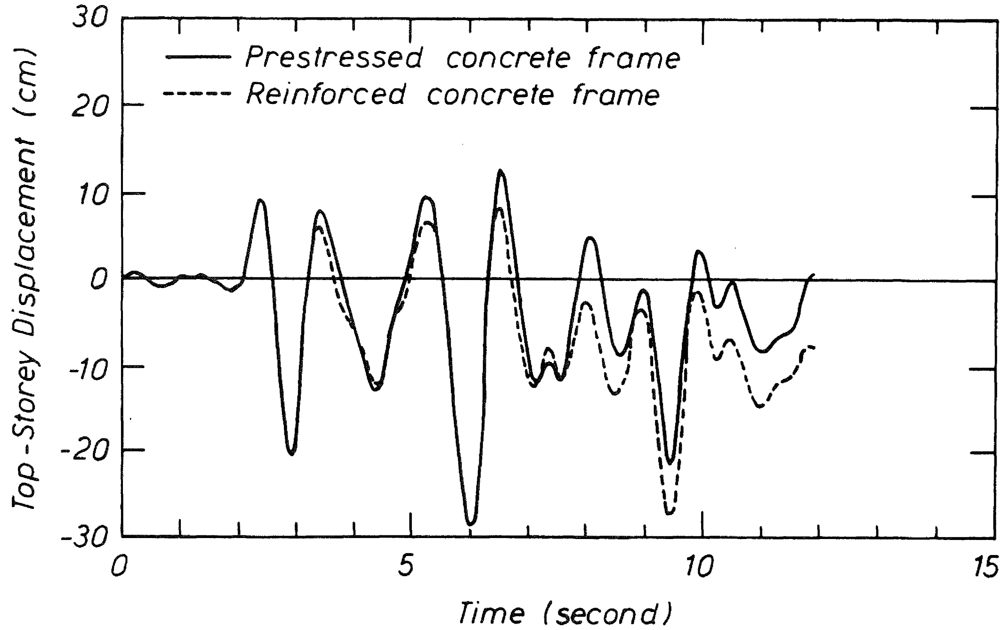


FIGURE 8 DISPLACEMENT TIME-HISTORY FOR THE TOP OF THE BUILDING

is evident that the maximum displacements that occurred were almost the same. The maximum interstorey drift, shown in Fig.9, occurred in the fourth storey and was 53 mm (an interstorey drift angle of 1/66) for the reinforced concrete building and 55 mm (1/63) for the prestressed concrete building.

The reason why their responses were not so different may be that the response of the building was largely dominated by the moment-rotation characteristics in the plastic hinges of the first storey reinforced concrete columns. However, it is apparent that the results of the dynamic response analysis depend very much on the structural configuration and the selected ground acceleration record. Hence a sweeping generalisation should not be made based on only one analytical result. More research in this area should be carried out.

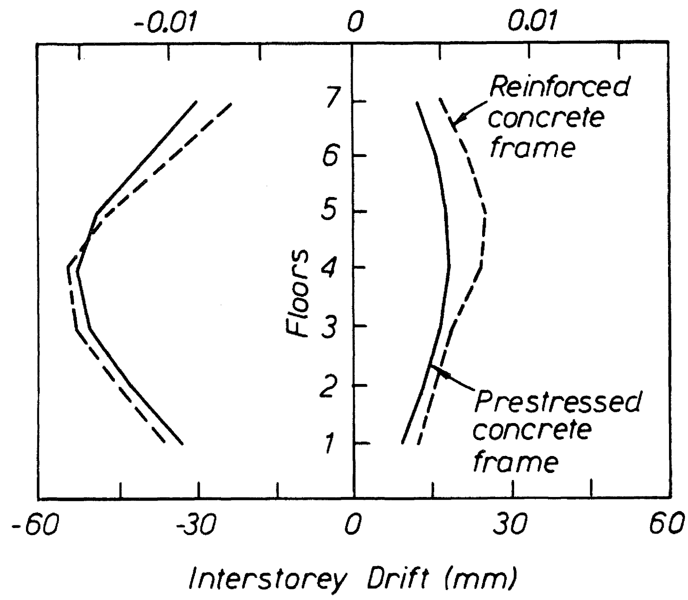


FIGURE 9 MAXIMUM INTER-STOREY DRIFT

Beam-column Joints

Prestressed concrete members usually have higher moments at the commencement of flexural cracking than do reinforced concrete members, and they recover to their original states and respond elastically even after they are loaded up to near their ultimate strengths. That is a reason why the dynamic response of prestressed concrete frames to ground motions can be larger than that of reinforced concrete. However, the restoring force characteristics of a frame are not determined only by those of the beams. The moment-rotation relationships of the potential plastic hinge regions of the columns also have a large effect on the performance of the frame as shown in the previous section. In addition, the performance of the beam-column joints can have a large influence on the response of the frame to ground motions, although joint cores are usually assumed to have high rigidity.

Prestress introduced into a beam has been shown to improve the shear resistance of beam-column joints. The reason is that the joint core is compressed in the horizontal direction as well as in the vertical direction due to the axial force on the column. Hence the joint core is subjected to biaxial compression. Also, prestressing is considered to increase the diagonal compression strut action in the joint core because the prestress results in a larger neutral axis depth. In addition, prestress can help preserve the rigidity of the joint cores. Stiffness degradation in joints can result in pinched hysteresis loops with reduced energy dissipation.

The results of tests which were conducted by the author et al on four prestressed (PC1-PC4) and two reinforced (RC1 and RC2) concrete beam-column subassemblies, which represented the joint region at the exterior columns of one-way moment resisting frames with plastic hinging occurring in the beams at the column faces, are summarised briefly below. The beam cross sections were 200 x 300 mm and the column cross sections were 300 x 300 mm, as shown in Figure 10. The beams were designed to have the same flexural capacities. The properties of the test units are summarised in Table 5.

The main features of the designs were as follows:

PC1: Prestressing steel bars were located within the central third of the beam depth. The nonprestressed longitudinal beam bars were welded to an anchorage plate on the outer side of the column.

PC2: Prestressing steel bars were located near the extreme fibre of the beam section. The nonprestressed longitudinal beam bars were welded to an anchorage plate.

PC3: Prestressing steel bars were located within the central third of the beam depth. Both the nonprestressed longitudinal top and bottom beam bars were bent downwards into the joint core or the column core. This is an anchorage detail which is common practice in Japan, although the Design Guidelines for Earthquake Resistant Reinforced Concrete Buildings Based on Ultimate Strength Concept [5] published in 1988 recommends that top bars should be bent downwards and bottom bars be bent upwards into the column core.

PC4: Prestressing steel bars were located near the extreme fibre of the beam section. Both the nonprestressed longitudinal top and bottom bars were bent downwards into the joint core or the column core.

RC1: Reinforced concrete beam - external column joint with nonprestressed longitudinal beam bars welded to an anchorage plate on the outer side of the column.

RC2: Similar to RC1 except that both the nonprestressed longitudinal top and bottom beam bars were bent downwards into the column core.

All test units except RC2 were able to be loaded to well beyond a beam rotation angle of 1/15. In unit RC2, after the maximum moment had been reached in each direction, it was observed that the subsequent reduction in stiffness and strength was due to damage concentrating in the joint core.

TABLE 5 PROPERTIES OF TEST UNITS

SPECIMEN	f'_c (MPa)	COLUMN			JOINT	BEAM		
		Longitudinal Rebar	P (kN)	$P/(A_g.f'_c)$	P_{wj} (%)	Longitudinal Rebar	e (e/D) (mm)	Pe (kN)
PC1	41.2	10-D19	264.6	0.0714	0.960 (ϕ9)	4-D19	30(1/10)	276.8
PC2							80(1/3.8)	283.9
PC3	29.8		98.0	0.0364			30(1/10)	303.3
PC4							80(1/3.8)	294.2
RC1	41.2			0.0714		6-D19,4-D16		
RC2	29.8			0.0364				

NOTES:

f'_c = Compressive strength of concrete
P = Axial force on column
A_g = Gross area of column section
e = Eccentricity of prestressing steel measured from the centroidal axis of the beam section
D = Total depth of section
P_e = Effective prestress
P_{wj} = $A_{sh}/(b_c \cdot j_b)$
A_{sh} = Total cross-sectional area of horizontal reinforcement within the joint core
b_c = Width of column
j_b = Distance between centroids of non-prestressed compression and tension reinforcing bars

Measured yield strenth of reinforcing: D-19 → 426 MPA
D-16 → 359 MPa
ϕ9 → 335 MPa
Specified yield strength (0.2% offset): 1275 MPa; and
Specified tensile strength of prestressing tendon: 1422 MPa

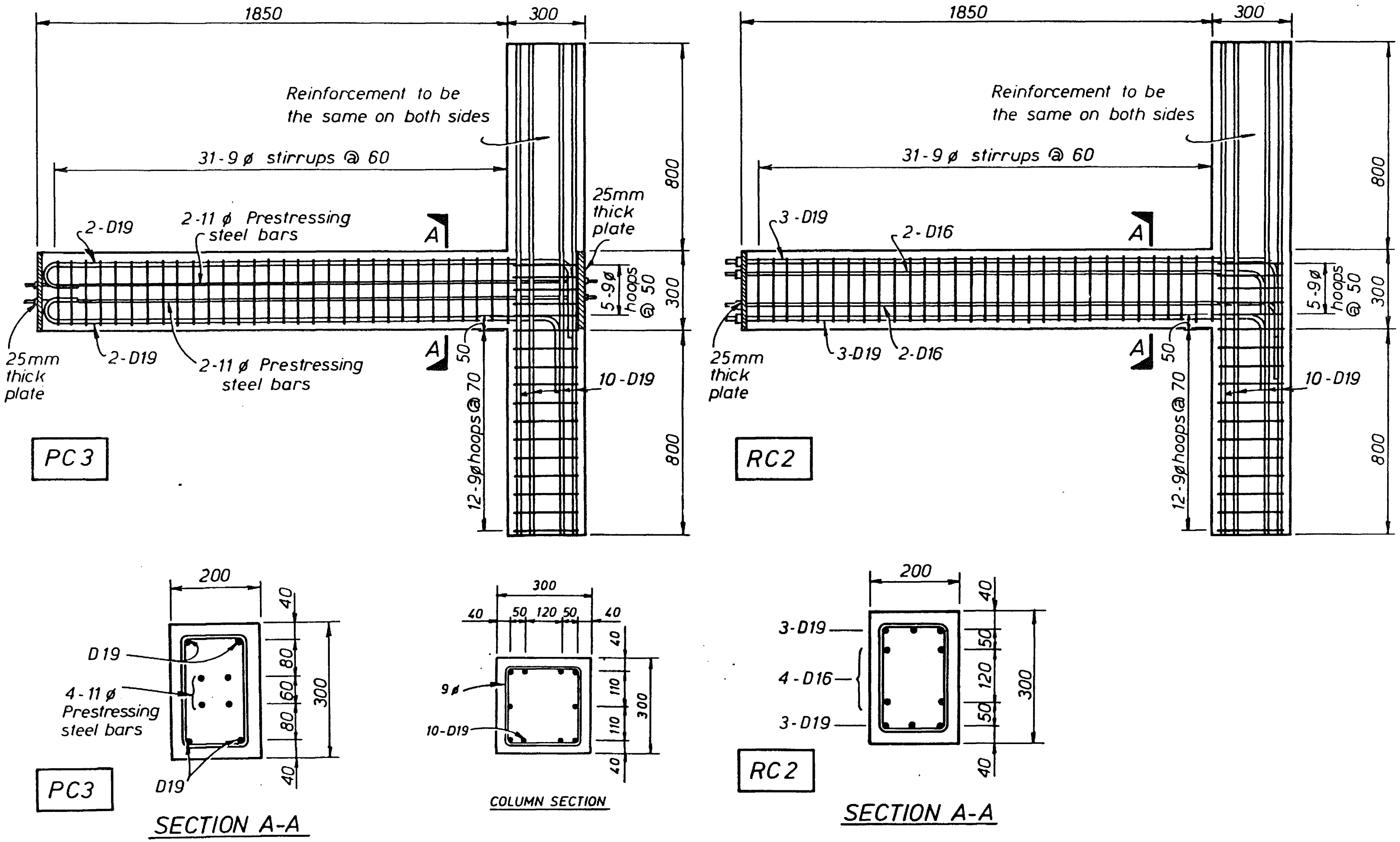
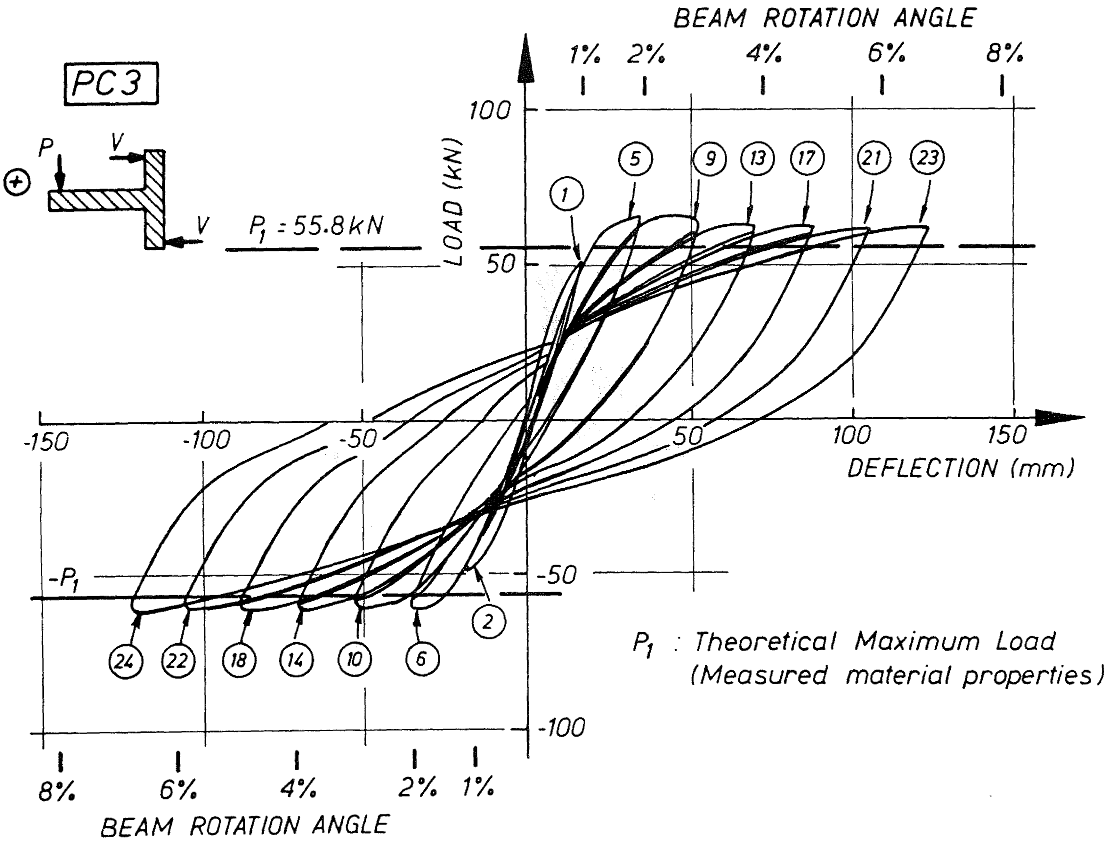


FIGURE 10 BEAM-COLUMN JOINT TEST UNITS



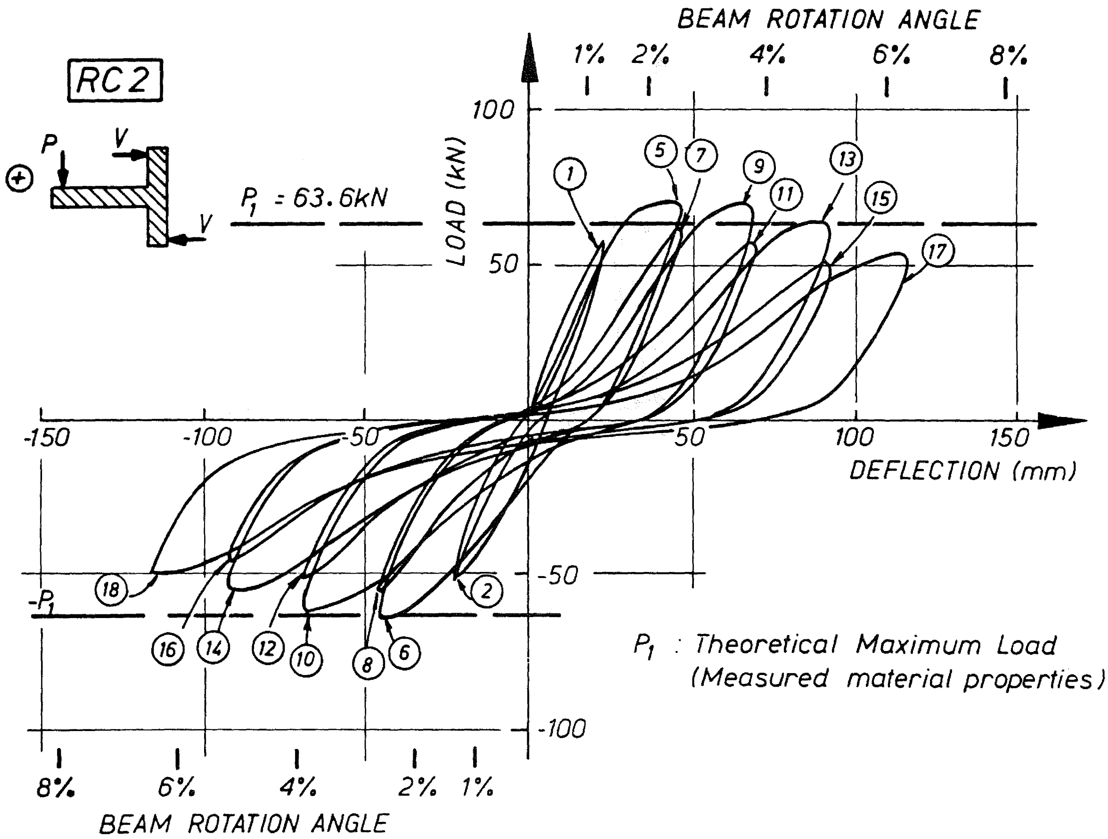


FIGURE 11 LOAD-DEFLECTION HYSTERESIS LOOPS MEASURED AT THE BEAM END FOR: (TOP) TEST UNIT PC3; (BOTTOM) TEST UNIT RC2

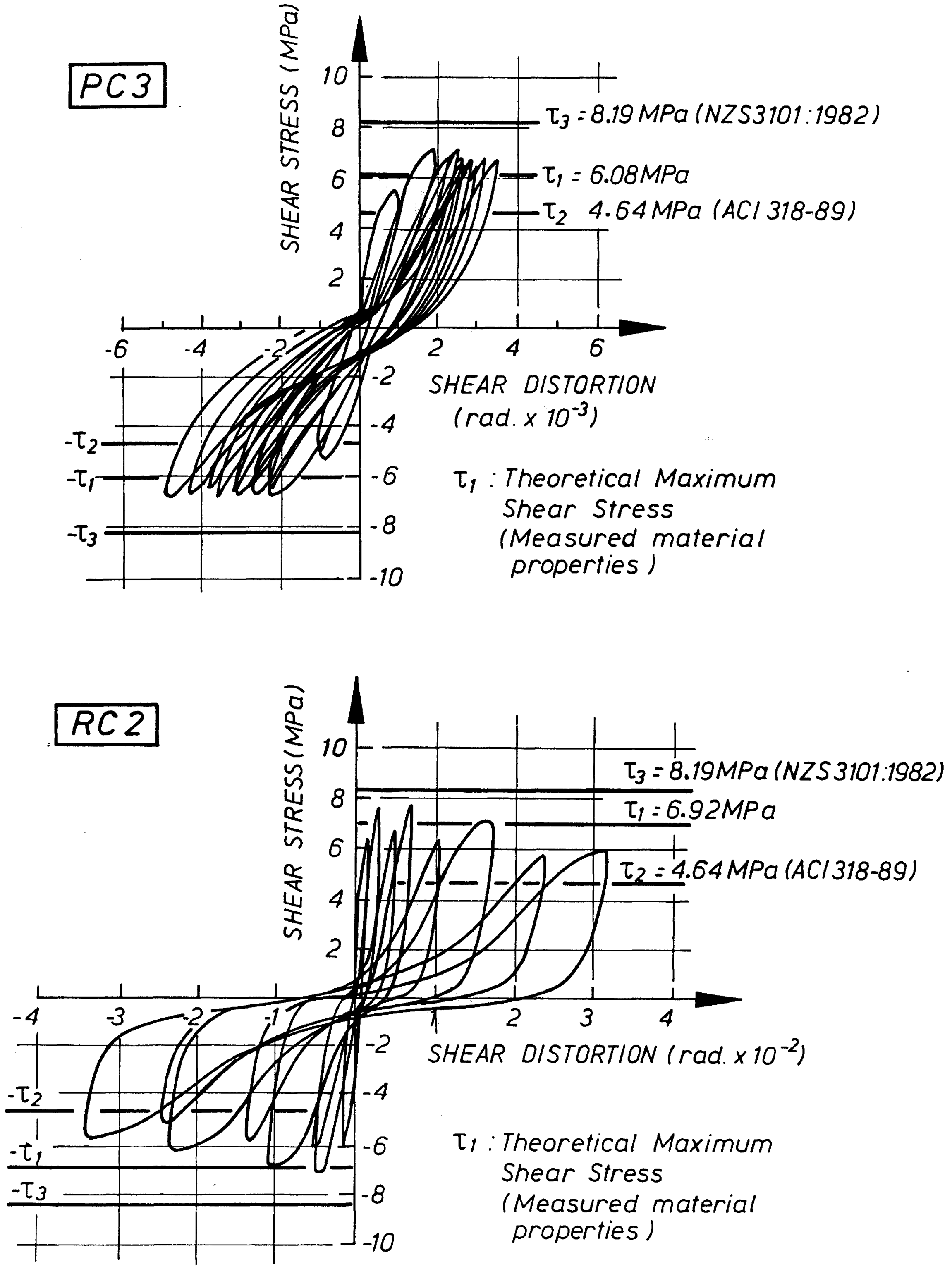


FIGURE 12 SHEAR STRESS - SHEAR DISTORTION RELATIONSHIPS MEASURED IN THE JOINT CORE FOR: (TOP) TEST UNIT PC3: (BOTTOM) TEST UNIT RC2

The load-deflection hysteresis loops measured at the beam end are shown in Figure 11 for PC3 and RC2, in order to compare the performance of prestressed and reinforced concrete beam-column joints. The results of these units gives a good comparison of the typical performance of prestressed and reinforced concrete beam-column joints. Figure 12 shows the shear stress-shear distortion relationships measured in the joints of PC3 and RC2. Brief conclusions reached from these experiments are:

(1) The performance of PC3 was significantly good with little reduction in the strength and little pinching. Contrarily, in RC2 pinching and strength reduction of the load-deflection loops was noticeable even in the loading cycles to lower ductility values.

(2) Most of the shear reinforcement in the joint core of RC2 yielded in the loading cycles to lower ductility values, while those stresses in PC3 remained below the yield stress. Figure 12 shows the damage concentrated in the joint of RC2.

(3) The prestressing force in the beam had a great effect on the performance of the beam-column joint. In other words, the prestressing force prevented the joint core from failing in shear and improved the behaviour of the beam-column joint assembly.

CONCLUSIONS

The current seismic design procedure for prestressed concrete buildings in Japan is introduced in this paper. The design procedure is fairly complicated because of the options which a designer can choose.

The following aspects of the design of prestressed concrete buildings are described and compared with those for reinforced concrete buildings:

(1) In NZS 4203:1984, the structural material factor M for prestressed concrete is 25% larger than that for reinforced concrete. However, this does not necessarily lead to 25% greater design seismic load. As shown in the section on the comparison of the seismic design loads, the design seismic load required for a prestressed concrete building may be smaller than that for an equivalent reinforced concrete building because of the longer period of vibration of the prestressed concrete building.

(2) The 25% larger structural material factor for prestressed concrete than for reinforced concrete specified in NZS 4203:1984 was determined based on the basis of dynamic response analyses of single- or multi-mass shear systems to severe seismic motion records. However, two-dimensional dynamic analysis of building frames conducted recently reveals a less significant difference between prestressed and reinforced concrete frames.

(3) The test results obtained from prestressed and reinforced concrete beam-column joints revealed that prestressing tendons in the beam and passing through the joint core improved not only the shear behaviour but also the energy absorption capacity of the beam-column joint subassemblage.

(4) Finally, in author's view, there seem to be advantages in the use of prestressed concrete buildings frames in both New Zealand and Japan, particularly to improve serviceability where high strength materials are incorporated.

ACKNOWLEDGEMENTS

This paper was written while the author was a visiting scholar at the Department of Civil Engineering at the University of Canterbury during 1990.

NOTATION

A_i : Distribution factor for lateral shear forces along the height of a building
C : Basic seismic coefficient in NZS 4203:1984
C_d : Seismic design coefficient
C_i : Lateral seismic shear coefficient in the i-th storey
C_o : Basic seismic coefficient in the Japanese code
D : Dead load
D_s : Reduction factor
E : Design load due to earthquake load in the second phase design by strength design
E' : Design load due to earthquake load in the second phase design by capacity design
E_c : Modulus of elasticity
E_J : Design load due to earthquake load of the Japanese code
E_N : Design load due to earthquake load of the New Zealand code
f'_c : Ultimate compression strength of concrete
F_e : Coefficient with regard to R_e
F_{es} : $F_e.F_s$
F_s : Coefficient with regard to R_s
h : Height of a building
L : Live load
M : Structural material factor
Q_e : Required strength of a elastically responding building
Q_i : Lateral shear force imposed on the i-th storey in the second phase design by strength design
Q_i' : Lateral shear force imposed on the i-th storey in the second phase design by capacity design
Q_{yd} : Yield force required by the equal displacement concept
Q_{ye} : Yield force required by the equal energy concept
R : Risk factor
R_e : Eccentricity ratio
R_s : Stiffness ratio
R_t : Design spectral coefficient
S : Snow load or structural type factor
T : Period of first mode of a building
T_c : Factor with respect to a subsoil profile

U : Design load
V : Total horizontal seismic force
W : Wind load
W_i : Gravity load above the i-th storey
Z : Seismic hazard zoning coefficient
α : Coefficient in the modified PS model
α_i : Ratio of reduced gravity load above the i-th layer to total reduced gravity load above the level of imposed lateral ground restraint
μ : Ductility factor

REFERENCES

1. "The Standard for Structural Design and Construction of Prestressed Concrete Structures", Architectural Institute of Japan, 1987.

2. AIJ Sub-committee on Seismic Design of Prestressed Concrete, "A Proposal for Seismic Design Method of Prestressed Concrete Buildings, FIP Commission 9 : Seismic Structures, Singapore, Sept 30, 1989.

3. "Code of Practice for the Design of Concrete Structures NZS 3101 Part 1 : 1982, and Commentary NZS 3101 Part 2:1982, Standards Association of New Zealand, 1982.

4. "Code of Practice for General Structural Design and Design Loadings for Buildings NZS 4203:1984", Standards Association of New Zealand, 1984.

5. "Design Guideline for Earthquake Resistant Reinforced Concrete Buildings Based on Ultimate Strength Concept", Architectural Institute of Japan, 1988.

6. Thompson, K.J. and Park, R., "Seismic Response of Partially Prestressed Concrete", Journal of the Structural Division, Proceedings of the ASCE, No. ST8, August 1980, p.p. 1755-1775.

7. Okamoto, S., "Fundamental Study on Earthquake Resisting Behaviours of Prestressed Concrete Frame Structures, Chapter 6: Seismic Responses of Prestressed Concrete Buildings:, Ph.D Thesis, Kyoto University, Japan 1986, p.p. 250-281.