DEVELOPMENT OF THE STRUCTURAL DESIGN AND CONSTRUCTION GUIDELINE FOR HIGH-RISE PC BUILDINGS - JAPANESE PC PROJECT

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SUMMARY

Prestressed concrete has been believed that the seismic performance is questionable due to its low energy dissipation capacity. However recent research and earthquake experience revealed that the seismic performance of prestressed concrete building is not necessarily inferior to other structural materials. The structural damage to prestressed concrete buildings during 1995 Kobe earthquake was very little although more than 150 prestressed concrete buildings were in the most severely damaged area. Therefore the high seismic performance of prestressed concrete was recognised once again. In 1996, Japanese PC project started to take advantages of prestressed concrete and completed in 1999. The main products were the design guideline and the construction guideline for prestressed concrete buildings up to 60 meters high. In the design guideline, the displacement based seismic design method is adopted. The construction guideline includes the structural planning, construction procedure of precast systems, required quality of materials, quality control and inspection. Typical connection details for precast construction were also summarised. Pseudo dynamic test was conducted on a three storied sub-structure frame (37% scaled model) as a part of the project. Test result indicated that the prestressed concrete high rise building is able to be designed to satisfy the seismic performance required in the design guideline.

INTRODUCTION

Brief history of prestressed concrete in Japan

Research on prestressed concrete first started in 1941 at the railway technology institute of the Ministry of Transport, Japan. After the Second World War, the committee on prestressed concrete at the Ministry of Commerce and Industry reopened the research to utilise the surplus of military materials. In early 1950s, high strength twisted strands and steel bars were developed in succession. This made the practical use of prestressing technology possible. In 1957 the Ministry of Construction circulated the Notification for the design and construction of prestressed concrete buildings. The code of practice for the design and construction of prestressed concrete buildings was published from the Architectural Institute of Japan in 1961. This AIJ code adopted the ultimate strength design method although the design code for reinforced concrete was based on the allowable stresses of materials. After then the construction of prestressed concrete buildings showed rapid progress. In 1986 the design and construction guideline for partially prestressed concrete was published from AIJ to promote the more utilisation of prestressed concrete. Recently the prestressed concrete is mainly applied to precast constructions assembled by post tensioning due to short construction period, simple joint mechanism and high quality control. January 1995, the Hyougoken-nanbu Earthquake hit the city of Kobe and surrounding area. However the damage to prestressed concrete buildings was quite small although more than 150 prestressed concrete buildings existed in most severely damaged area. The high seismic performance of prestressed concrete is being recognised once again.

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Seismic performance, durability and space flexibility

The specific feature of prestressed concrete is non-linear elastic behaviour. Therefore it has been believed that the seismic performance of prestressed concrete is inferior to ordinary reinforced concrete due to its smaller energy dissipation. However it should be remembered that the larger energy dissipation means the larger damage to materials and members, such as damage to concrete, permanent plastic deformation of re-bars, larger residual deflection and degradation of member stiffness. Smaller residual deformation of prestressed concrete after the earthquake is one of the advantages and should be looked at it in a new light. Recently, several researches are being conducted to take this advantage into seismic design. Main focuses are placed on the assurance of deformation capability, high restorability and larger energy dissipation. Several methods have been proposed such as partial de-bonding of prestressing tendons, use of graded composite strand and combined use of non-stressed mild steel and unbonded tendon. These technologies make the construction of prestressed concrete buildings possible in high seismicity regions. On the other hand the durability of buildings is important design target to save energy and to save finite natural resources. During more than 40 years experience of the construction of prestressed concrete buildings in Japan the durability problem has been never hard. This is due to the use of high strength concrete, high quality control at the construction site and the crack control ability. The other advantage of prestressed concrete is space flexibility. The usage of buildings could be changed as the change of social and economical situation. Prestressed concrete building is able to satisfy this due to its comparatively large unit space.

Application to precast construction

The lack of skilled workers at the construction site is a problem we have got to face so that we have no choice but depend on the precast construction. Key of the precast construction is the joint. In the reinforced concrete precast construction the complicated wet joints are widely used to assure the equivalent monolithic structural behaviour. On the other hand the precast construction assembled by post tensioning has an advantage due to its construction easiness, simple detailing of joints and clear stress transfer mechanism at joints. From this point the precast prestressed concrete has a great future as a construction method of buildings.

JAPANESE PC PROJECT

Purpose of Japanese PC Project

The prestressed concrete has several advantages and more practical use for buildings is expected. However the construction of prestressed concrete buildings has been limited to low or medium height buildings by the control of the Building Standard Law Enforcement, Notifications and Circular Notices by the Ministry of Construction. The reason is that the design guideline for high rise prestressed concrete buildings has not been well-established and some suspicious opinions in seismic performance of prestressed concrete buildings as well. If this legal control is relaxed, the high-rise high-quality prestressed concrete buildings can be supplied to the society and results in the successful formation of social stock. Therefore the co-operative research project on prestressed concrete (JAPANESE PC PROJECT) was planned and started in 1996 under the leadership of the Building Research Institute, Ministry of Construction Japan. The Project completed in April 1999.

Project Organisation

The project was funded by the Ministry of Construction, Building Contractors Society, Japan Prestressed Concrete Contractors Association, Japan Association of Representative General Contractors, Housing and Urban Development Corporation, Japan Structural Consultants Association and Building Centre of Japan. The secretariat of the project was established in the Japan Association for Building Research Promotion. The project founded three committees. Under the Working Committee, three task groups were established to conduct the substantial research works in the project.

1) Promotion Committee:	Promotion and funding (chaired by K. Nakano)
2) Technical Co-ordinating Committee:	Technical co-ordination (chaired by S. Okamoto)
3) Working Committee:	Technical co-ordination and mutual exchange of information between three task groups (chaired by F. Watanabe)
a) Task Group on Design Guideline	(group leader: S. Machida)
b) Task Group on Structural Performance	(group leader: F. Watanabe)
c) Task Group on Construction Guideline	(group leader: S. Sugano)

Output of the Task Group on Design Guideline

The Task Group completed the design guideline based on the performance based design method. Covered structural systems are pure frames, multi-storey wall systems and dual systems, where the building height was limited less than 60 meters. Design for precast construction is also involved in the guideline. Possible member types in a building are listed on Table 1. Structural designer decides the combination of members based on Table 1.

Table 1 Possible member types

	Cast-in-situ construction		**Precast construction**	
	Prestressed concrete	**Reinforced concrete**	**Prestressed concrete**	**Reinforced concrete**
Columns	Yes	Yes	Yes	Yes
Beams	Yes	No	Yes	No
Walls	No	Yes	No	Yes

The minimum value of compressive strength of concrete is 24 N/mm^2 for prestressed and reinforced concrete members. For pre-tensioned prestressed concrete members it is specified as 35 N/mm^2. The upper limit value of compressive strength of concrete is not clearly specified in the guideline but it may be approximately 60 N/mm^2. Use of unbonded prestressing steel for primary earthquake resistant members is not permitted in the guideline due to the lack of enough data. Seismic performances are classified into three categories as safety, reparability and serviceability. Design criteria are indicated in Table 2.

Table 2 Summary of design criteria

Condition	**Serviceability**		**Reparability**		**Safety**	
	Load combination	**Limit state**	**Load combination**	**Limit state**	**Load combination**	**Limit state**
Gravity load	D+L+ α S	Deflection Crack width	-	-	1.7(D+L+S) 1.2D+2(L+S)	Collapse mechanism
Wind load	D+L+W+ β S		-	-	(D+L+S)+1.5W	Collapse mechanism
Earthquake load	D+L+ β S +L1	Elastic response	D+L+ β S +L2	Engineer's choice	D+L+ β S +L3	Collapse

D: dead load, L: code specified live load, S: maximum snow load for regions with heavy snowfalls, α: snow load reduction factor and can take 0.7, β: snow load reduction factor and can take 0.35, W: code specified wind load, L1: code specified moderate earthquake, L2: code specified major earthquake, L3: maximum earthquake.

Design for gravity load and wind load

Calculated stresses of materials, member deflections and flexural crack widths should be smaller than the limit state values to assure the serviceability for gravity load and wind load. For the structural safety the formation of collapse mechanism should be avoided under the factored load conditions indicated in Table 2. Moment and forces are computed by linear elastic analysis and the ultimate strength design method is applied to beams, columns, structural-walls, beam-column joints and precast connections.

Design for earthquake load

Design for earthquake load should assure serviceability for the moderate earthquake (L1), reparability for the major earthquake (L2) and safety for the maximum earthquake (L3) as indicated in Table 2. Japanese building standard law gives the standard response spectrum at the bedrock for the moderate earthquake (L1) and for the major earthquake (L2). Therefore these spectrum should be modified to take into account the magnification due to soil foundation interaction and the seismicity at the construction site. The L3 earthquake is the possible maximum earthquake at the site and is predicted by the engineer. However it should be greater or equal to L2.

For L1 earthquake the building should show elastic response without any damage. For L3 earthquake the building should not collapse. Reparability limit state for L2 earthquake is chosen between serviceability limit state and safety limit state according to the agreement between the engineer and the client. The responses of members, precast connections and materials can be converted from the displacement response.

To predict the lateral displacement response the equivalent linearisation method is applied using the capacity spectrum and the demand spectrum. The intersection point of a capacity spectrum and a demand spectrum gives the performance point (displacement response). Demand spectra are given for three levels of earthquakes (L1, L2 and L3) and expressed by the acceleration response and the displacement response. The capacity spectrum is expressed by the lateral load-lateral deflection relationship at the equivalent centre of mass of multi-degree-of-freedom system, where the lateral load-lateral deflection relationship is computed by the non-linear push over analysis.

When the equivalent linearisation method is applied the equivalent damping factor is needed. For L1 earthquake it can take 3 %. For L2 and L3 earthquakes the equivalent damping factors are computed based on the non-linear push over analysis. It is given as the weighed mean value of damping factors of members when the lateral displacement at the equivalent centre of mass reaches the target limit state displacement.

Design Trials

Design trials of three types of buildings were conducted to demonstrate the advantages of prestressed concrete buildings. Type 1 is a 10 storied office building with a structural core at its centre. Beams are constructed by precast partially prestressed concrete and columns are ordinary cast-in-situ reinforced concrete. Type 2 is an 11 storied office building with structural cores at both sides of the plan. Beams are precast prestressed concrete. Columns and walls are ordinary cast-in-situ reinforced concrete. Type 3 is a 20 storied precast prestressed concrete condominium building with an open space at its centre of the plan. The design trials indicated that the prestressed concrete construction is successfully applied to high rise buildings even in high seismicity regions.

Output of the Task Group on Structural Performance

The task was to provide the necessary data for the design guideline. Research focuses were placed on a) relationship between member deformation and materials damage, b) idealisation of load deflection behaviour of prestressed concrete members, c) equivalent damping factors and d) strength and ductility of members, beam-column joints and precast connections. Main outputs are introduced below.

Idealisation of load deflection behaviour of prestressed concrete members

The idealised load deflection curves for prestressed concrete members are applied for the push over analysis and for the dynamic time history analysis when it is required. The skeleton curve was given by the load deflection curve for monotonic loading which consisted of four characteristic points such as cracking, flexural yielding, maximum capacity and ductility limit. The evaluation method of load deflection co-ordinate for each characteristic point was well established. Rules for cyclic behaviour was then idealised where considered influencing parameters were amount of prestressing steel, amount of non stressed reinforcement, level of prestressing, bond characteristics of prestressing steel and loading history.

Design for beam-column joint

There have been several discussions on the design of beam-column joint in reinforced concrete frames. However the joint design method for prestressed concrete is only indicated in the New Zealand Design Code (NZS3101: 1995). In the NZ3101, 70 % of prestressing force can be deducted from the joint shear. On the other hand there is a discussion that the joint shear strength does not increase due to prestressing in case of interior joints. Therefore the loading tests and FEM analysis were conducted on interior beam-column joints with prestressed concrete beams and reinforced concrete columns. The results indicated that the effect of prestressing was not clearly evaluated due to very limited tests and analysis. The Task group could not reach the final agreement and only indicated the calculation method for joint shear force. That is, the design method for ordinary reinforced concrete beam-column joint is to be applied. For exterior joint NZS3101 could be used. Future research is needed to examine the effect of prestressing force to joint shear strength.

Shear transfer at the concrete interface

Shear design of the interface between precast concrete and cast-in-situ concrete has been conducted based on the famous shear friction theory. However the data for high strength concrete and high strength connection reinforcement is not enough. Therefore the direct push-off shear tests were conducted to derive the design equation for concrete joint with high strength materials. In the tests the shear strength was taken at the maximum capacity when it was reached at the slip less than 2 mm. Otherwise it was taken at the slip of 2 mm. This is to avoid the excessive shear slip at the interface. From the experimental observation and analysis the following equation was derived for the design. Applicable range of variables is indicated in Table 3. Equation 1 was derived for concrete interface with high strength materials so that the shear friction equation proposed in the past is applied for out of these ranges.

Table 3 Applicable range of variables

	Range of variables
Compressive strength of concrete	$25N/mm^2 \le f_c' \le 98N/mm^2$
Yield strength of connection reinforcement	$300N/mm^2 \le f_y \le 999N/mm^2$
Ratio of connection reinforcement	$0.4\% \le \rho_s \le 1.95\%$
Diameter of connection reinforcement	$10mm \le d_s \le 16mm$
Value of $\rho_s f_y$	$\rho_s f_y \le 7N/mm^2$
Interface conditions	Aggregate exposure, uniformly distributed small shear keys

Exceptions: 1) Smooth concrete surface, 2) Combination of $f_c' \le 50N/mm^2$ and $f_y \le 500N/mm^2$

$$\tau_u = k(0.67\rho_s f_y + 2.84) \tag{1}$$

$$k = 0.02 f_c' + 0.2 \tag{2}$$

τ_u: shear strength (N/mm^2), ρ_s: connection reinforcement ratio, f_y: yield strength of connection reinforcement (N/mm^2), f_c': compressive strength of concrete (N/mm^2)

In the precast prestressed concrete construction, beams and columns are assembled by post-tensioning. The gap between beam end section and column side face is filled with non-shrink mortar and is bound by prestressing force. The joint should be designed by Eq. 3 where the friction coefficient, μ, is specified to be 0.5 because the form produced smooth surface is generally used in Japan.

$$Q_d \le \mu P \tag{3}$$

Q_d : design shear for safety limit state (see Table 2), μ : friction coefficient (=0.5), P : prestressing force

Another point of joint design is the residual shear strength after experienced the earthquake. Because the joint shear strength may reduce due to the loss of prestressing force and the damage to joint materials. The joint should have residual shear strength to resist the shear due to gravity load. To investigate the residual shear strength and the loss of prestressing force, loading tests and FEM analysis were conducted on beam-column sub-assemblages which satisfied Eq. 3. Results indicate that if the maximum inter-story drift response is suppressed less than the certain values the required residual joint shear strength is automatically assured. In Table 4 permissible maximum inter-story drift response and required residual shear strength are summarised.

Table 4 Permissible response limit and requirement for residual joint shear strength

Required limit of earthquake response			Requirement for residual joint shear strength
Limit state	Target earthquake	Inter-story drift R	
Serviceability	Moderate (L1)	$R \le 1\%$	$1.7(D+L+S) \le Q_{ur}$ $1.2D + 2(L+S) \le Q_{ur}$
Reparability	Major (L2)	$R \le 2\%$	$D+L+S \le Q_{ur}$
Safety	Maximum (L3)	$R \le 3\%$	$D+L+S \le Q_{ur}$

Q_{ur} : required residual joint shear strength

Shear strength of prestressed concrete members

Design of prestressed concrete members for shear has been conducted by very simple method in Japan. Ten percent of axial compressive stress due to prestressing is added to the contribution of concrete. This is based on the experimental observation in the past. This Task Group intended to propose the shear design method based on the superposition of truss and arch mechanism. The proposed shear design equation was compared with test data and showed enough accuracy. Shear strength is given by Eq. 4.

$$V_u = b_o j_o p_w \sigma_{wy} + 0.5 b_o h (\nu f_c^{'} - 2 p_w \sigma_{wy}) \tan\theta \tag{4}$$

$$\tan\theta = \sqrt{\left(\frac{2M}{Qh}\right)^2 + 1} - \frac{2M}{Qh} \tag{5}$$

$$\nu = \alpha L_r \left(1 + \frac{\sigma_p + \sigma_o}{f_c^{'}}\right) \tag{6}$$

$$\alpha = \sqrt{60 / f_c^{'}} \le 1.0 \tag{7}$$

$$L_r = \frac{M}{2Qh} \le 1.0 \tag{8}$$

b_o = section width at the centroidal axis, h = total section height, j_o = distance between compression and tension reinforcement, p_w = shear reinforcement ratio for b_o, σ_{wy} = yield strength of shear reinforcement (N/mm^2), $f_c^{'}$ = compressive strength of concrete (N/mm^2) ν = effectiveness factor of concrete and given by Eq. 6 and $1 \ge \nu \ge 0.65$, σ_p = average compressive stress due to prestressing, σ_o = average compressive stress due to external axial load, M and Q =maximum moment and shear force of member

Shear design method for beams with an opening is also proposed. The top part and the bottom part of an opening (circular or rectangular) are separately designed for shear as reinforced concrete members which are subjected to axial force due to external bending moment and prestressing force.

Ductility assurance of plastic hinge regions

Ductility assurance of plastic hinge region is primarily important in seismic design of ductile frames. In this project several number of prestressed concrete beams were tested to derive the design method for ductility. The following design equations were proposed for plastic hinge region. The amount of longitudinal reinforcement should satisfy Eq. 9. Required amount of transverse reinforcement is given by Eq. 12 where $p_w \sigma_{wy}$ should be greater or equal to $0.8 N/mm^2$. The unsupported length of transverse reinforcement should be smaller than 200 mm and the spacing of a set of transverse reinforcement should not be greater than 100 mm.

$$q_{sp} \le 0.5 - 10R_u / F \tag{9}$$

$$F = 1.4 - f_c^{'} / 100 \tag{10}$$

$$q_{sp} = \frac{A_{sp} f_{py} + (A_t - A_c) f_y}{bhf_c^{'}} \tag{11}$$

$$p_w \sigma_{wy} \ge 0.8 + 6.67 \left[\frac{10R_u}{F(0.5 - q_{sp})} - 1 \right] \tag{12}$$

$f_c^{'}$: compressive strength of concrete ($24N/mm^2 \le f_c^{'} \le 60N/mm^2$), R_u : member rotation angle at the ultimate limit state and $R_u \le 4\%$, A_{sp} and A_t : sectional area of prestressing steel and non-stressed ordinary reinforcement in tension side, respectively, A_c : sectional area of non-stressed ordinary reinforcement in compression side. reinforcement, f_{py} : yield strength of prestressing steel, f_y : yield strength of ordinary reinforcement, b : section width, h : total section height, p_w : transverse reinforcement ratio, σ_{wy} : yield strength of transverse reinforcement and $\sigma_{wy} \le 800N/mm^2$

Output of the Task group on construction guideline

Construction guideline consists of "Volume 1: Required Quality, Construction and Quality Control Program" and "Volume 2: Construction and Inspection".

Volume 1 indicates the basic concept for the structural planning of precast or cast-in-situ prestressed concrete buildings and the construction planning (production and transportation of precast elements, election, assembling of precast element, prestressing, grouting, concrete casting, site safety and quality control). In Volume 2 practical method for construction and quality control are indicated for several items such as a) materials, b) mix proportion of concrete, grout mortar and dry mortar, c) ready mixed concrete, d) prestressing steel and introduction of prestressing, e) grouting, f) production and quality control of precast elements, g) storage and transportation of precast elements, h) election and assemble of precast elements and i) inspection.

Typical connection details for precast construction were also summarised in generic style.

Pseudo dynamic test on a model frame

The pseudo dynamic test was conducted on a two-bay sub-structure frame of lower three stories extracted from 11 storied precast prestressed concrete buildings (45 meters high). The sub-structure frame was 37 % scaled model and designed to show beam hinging collapse mechanism. Precast beams and columns were assembled by post tensioning and then floor concrete was cast. The tested frame was designed based on the design guideline. The criteria of seismic design are summarised in Table 5. Input ground motions are listed in Table 6.

Table 5 Design Criteria for seismic load

Level of earthquake	Performance	Design base shear	Limit state	
			Inter-story drift	Member ductility
L1: Moderate earthquake	Serviceability	0.2	0.5%	1.0
L2: Major earthquake	Reparability	0.3	1.0%	1.0
L3: Maximum earthquake	Safety	0.333	2.0%	2.0

Table 6 Input ground motion

Level of earthquake	Input ground motion
L1: Moderate earthquake	Hachinohe EW (standardised), maximum velocity =25 cm/sec
L2: Major earthquake	Hachinohe EW (standardised), maximum velocity =50 cm/sec
L3: Maximum earthquake	Hachinohe EW (standardised), maximum velocity =75 cm/sec
	JMA-Kobe NS (original), maximum accrelation=820.6 gal.

After the pseudo dynamic test, static cyclic loading test was conducted up to the inter-story drift angle of 5 %. From the tests following results were obtained.

1) Frame showed fully ductile behaviour up to the inter-story drift angle of 5%.
2) Maximum inter-story drift responses for L1 and L2 earthquakes were 0.66 % and 1.16%, respectively. The residual drifts were quite small although these values slightly exceeded the design values.
3) For L3 earthquake the maximum inter-story drift response was 1.92% without any notable capacity reduction.
4) Equivalent damping factor was about 5% for the drift less than 1% and 6% to 8% for the larger deflection.

CONCLUDING REMARKS

For the more utilisation of prestressed concrete to building structures, the Japanese PC Project was carried out. Then the design guideline and the construction guideline for high rise prestressed concrete buildings were well established. Current legal control will be relaxed in the future and result in the spread of the construction of high rise prestressed concrete buildings.

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