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# STRUCTURAL DESIGN OF MULTISTORY PRECAST CONCRETE APARTMENT BUILDINGS

Toshikazu KOMURA[1], Noritoshi YAMAGUCHI[1]
and Norio NAGATA[1]

[1]Structural Engineer, Mitsubishi Estate Co., Ltd,
Chiyoda-ku, Tokyo, Japan

## SUMMARY

The structure of Nishi-Toyama Tower Homes (located in Tokyo) is the first example where precast concrete construction has been adopted for the multistory RC frame buildings exceeding 60 meters in height. The building height is 75 meters (25 stories) and the precast concrete construction was used in the beams and the floors. Earthquake resistance has been confirmed with dynamic analysis, but also the pertinency of the analysis is being studied with experiments on parts of the frame (full scall model test).

## INTRODUCTION

At present there are nine completed RC frame buildings in Japan with a height of more than 60 meters. All of them are apartment buildings, and eight were completed during the past two years. High-rise RC frame building is now on the increase. The Nishi-Toyama Tower Homes introduced in this paper was completed in March 1983 and was the first building for which the precast concrete construction was used.

## STRUCTURAL DESIGN AND CONSTRUCTION

Structural Planning  Nishi-Toyama Tower Homes consists of three identical, multistory RC frame buildings and two annexes. The three RC frame buildings share the same structural planning. The buildings are 75 meters at major structure, with 25 stories above ground and one basement. A typical floor has eight apartments, with all of the doors opening towards the light court in the center of the building. The north wall of the building is opened to allow light to enter the court. The total span on each side is 32.1 meters with a six-span frame.

The north-east and the south-west corners are rounded and adjacent two columns are arranged at the north-west and the south-east corners. That two columns have then been connected to each other with small beam in height.

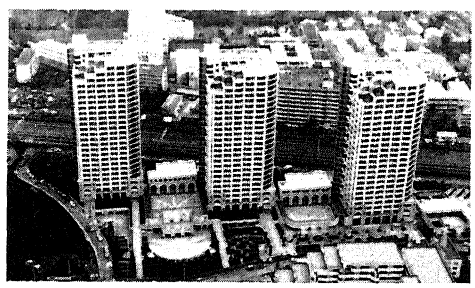
Photo. 1 The Whole View

Materials  The concrete is normal weight concrete with a maximum specified design strength of 270 - 420 kg/$cm^2$, main reinforcement is SD 40 grade (max. diameter 41 mm), stirrups and hoops are deformed prestressing bars SBPD 130/145. Fig 2 shows the places where concrete has been used.

<ins>Construction</ins> Fig. 3 shows the PC construction cycle. One cycle is seven days. Peripheral beams are precast concrete, internal beams and floors are partially precast concrete, and the columns and the beam-column connections are cast-in-place concrete. The method used in the joints is semiautomatic enclosed arc welding (NKE method). Fig. 4 shows the detail plan for the bar arrangement in the peripheral beam-column connection. Fig. 5 and Fig. 6 show the beam PC unit.

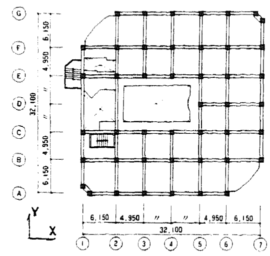

Fig. 1 Typical Framing Plan

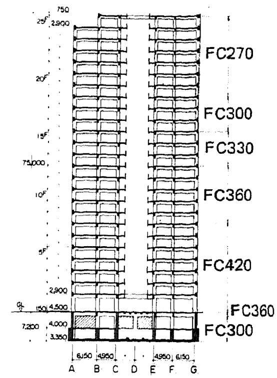


Fig. 2 Section and Specified Design Strength of Concrete

## SEISMIC DESIGN

The principle of the seismic design is to confirm the earthquake resistance of the frame by earthquake response analysis. Table 1 shows the criteria used. The designed base shear coefficient ($C_B$) is 0.130, and every member is sufficiently ductile. The collapse mechanism of the frame is beam sideway. Table 2 shows the criteria for member design.

<ins>Dynamic Analysis</ins> Dynamic analysis is conducted on the 25 stories above ground to confirm that the criteria in Table 1 are met.

The analysis models are the elasto-plastic response model of multi-mass equivalent shear type (X and Y directions) and the torsional elasto-plastic response model of multi-mass equivalent shear type.

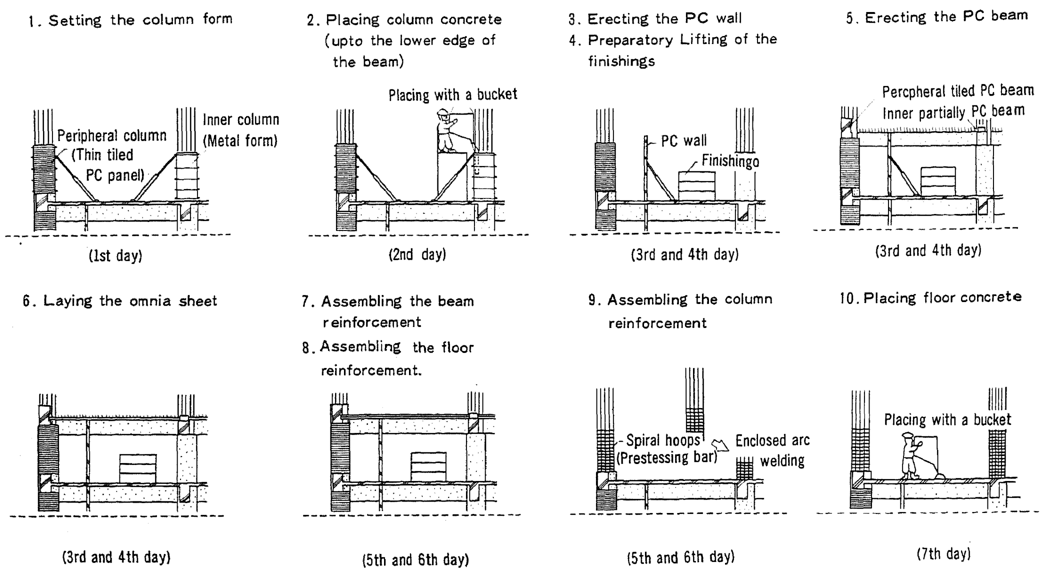


Fig. 3 Stages of The PC Construction Cycle

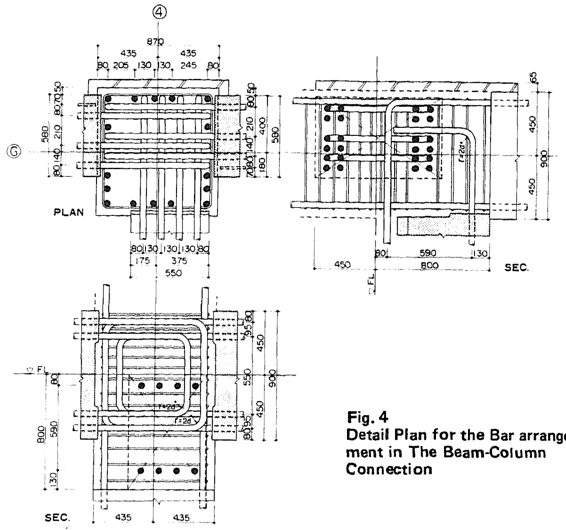


**Fig. 4**
**Detail Plan for the Bar arrangement in The Beam-Column Connection**

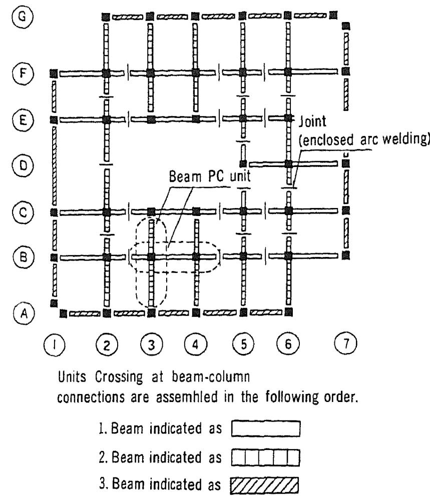


**Fig. 5 Beam PC Unit (plan)**

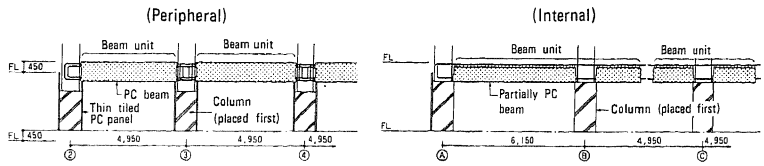


**Fig. 6 Beam PC Unit (elev.)**

Restoring force characteristics were found as follows: The analysis is performed by first taking all planes of the structure apart and then adding step by step load statically. As a result of this analysis, it can be seen that the restoring force characteristics in all planes on every story resemble the Tri-linear form. In the case of the torsional response model, they are changed into skelton curves. In the case of the multi-mass shear type model, they are further condensed, and resemble the Tri-linear form. The hysteresis rule is the degrading Tri-linear type (Takeda model). A part of results derived from the responses is shown on Fig. 7.

Below is explained the method of ensuring that the yield strength of every member is sufficient for the 25 kine response analysis. As shown in Fig. 8, plot on the chart the relation of Q and $\delta$ of each plane of structure (from the step by step analysis' results), the largest value received in the 25 kine response analysis and the load where the columns and the beams first yielded. By the order to the plotted lines you can estimate whether the risk of yielding in each member exists. This analysis confirms that the ductility factor of each member is within 1.0. The ductility factor of members is estimated in the same way also in the case of 40 kine response analysis.

Presently, the input level generally used in Japan in level 2 is 50 kine. Our buildings meet the criteria in table 1 even with an input level of 50 kine.

**Table 1 Dynamic Design Criteria**

| | Input Level | Story Drift | Member Duc. Factor | Story Duc. Factor |
|---|---|---|---|---|
| Level 1 | 25 kine | 1/200 max. | 1.0 max. | 1.0 max. |
| Level 2 | 40 kine | 1/100 max. | 4.0 max. | 2.0 max. |

**Table 2 Criteria for Member Design**

| Items | | Criteria |
|---|---|---|
| Allowance in the column bending strength | Inner column | ≧ 1.3 |
| | Peripheral col. (Tens.) | ≧ 1.0 |
| | Peripheral col. (Comp.) | ≧ 1.4 |
| Allowance in the column shear strength | Inner column | ≧ 1.3 |
| | Peripheral col. (Tens.) | ≧ 1.1 |
| | Peripheral col. (Comp.) | ≧ 1.4 |
| Limit of the axial forces under mechanism | Compressed side | ≦ 0.6 FcAe |
| | Tension side | ≧ −0.3 FcAe |
| Tension reinforcement ratio of beam | | ≦ 0.75 Ptb |
| The length of straight anchorage of bar | | ≧ 20 d |

Table 3 Seismic Waveforms Adopted

| Seismic waveform | Max. acc. (gal) | | |
|---|---|---|---|
| | Original | 25 kine | 40 Kine |
| EL CENTRO 1940NS | 341.7 | 259 | 414 |
| TAFT 1952EW | 175.9 | 257 | 411 |
| TOKYO101 1956NS | 74.0 | 256 | 410 |
| SENDAI038 1978EW | 240.9 | 160.5 | 257 |

Table 4 Natural Periods (elastic)

| | X-derec. | Y-derec. |
|---|---|---|
| T1 (sec) | 1.253 | 1.362 |
| T2 (sec) | 0.484 | 0.532 |
| T3 (sec) | 0.297 | 0.327 |

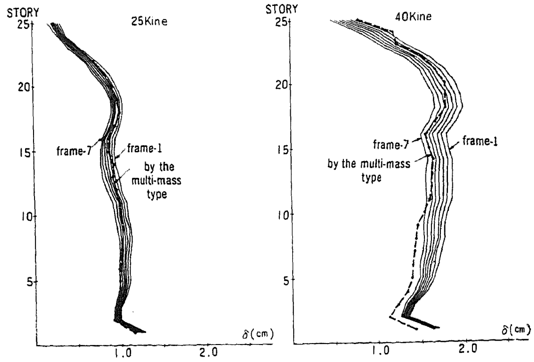


Fig. 7 Results of Respons Analysis by The Torsional Elasto-Plastic Model (story drift – Y direc., TAFT)

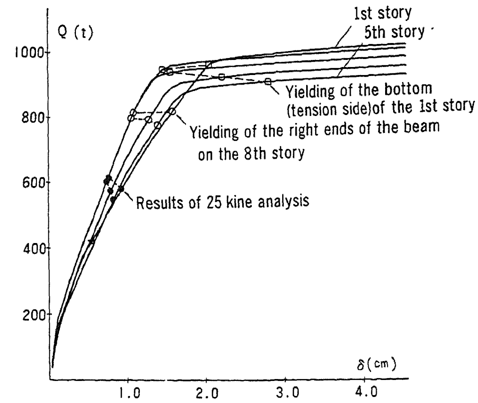


Fig. 8 Observation of The Yielding of Members after The 25 Kine Analysis

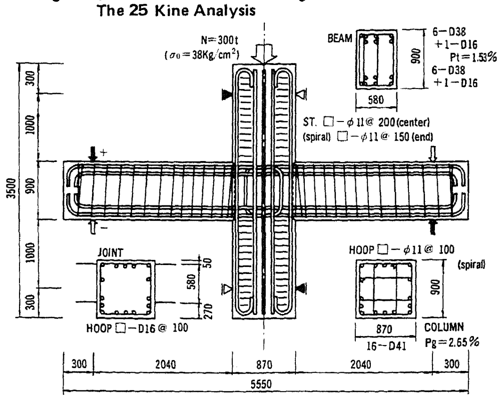


Fig. 9 Specimen A

## CONFIRMING EXPERIMENT

Objective

(1) Testing the strength and ductility of the joint panels used to the peripheral frame when using the U-shaped anchorage (specimen A).

(2) Testing the strength and ductility of the joint panels under high level shear stress from the mechanism of the frame. The panels tested are the ones in the internal beams where the main reinforcement of the precast beams have straight anchorages (specimen B).

Full Scall Specimens The main principles in designing and producing the specimens are as follows:

(1) The production of the specimen is conducted in the construction sequence.

(2) Material dimensions, strength of the concrete, diameter of the reinforcement bars and the details of the bar arrangement are decided by the design drawings.

The Load Applying Method The specimen was placed on the reaction floor, and holded both ends of a column by sliding and revolving bearings. Under this condition, the tests were conducted in repeated and cyclic reversed loads to both ends of the beam. Displacement was controlled by load. The axial force was constant at N = 300 tf ($\sigma_0$ = 38 or 42 kgf/cm$^2$).

The Collapse Process Table 8 is the summary of different loads and Fig. 11 shows the relation between each specimen's shear force (Q) and the angle of story drift (δ). With both specimens, the bending crack and flexural shear crack of the beam occurred first. Following them were the bending crack of column, the shear crack of beam and the shear crack of column. The yielding to bend by beam in the case of specimen A occurred at $6 \times 10^{-3}$ rad and in the case of B, at $9 \times 10^{-3}$ rad. In the case of specimen A, the width of the cracks widened, and at repetition of $R = 20 \times 10^{-3}$ rad, the breaking under compression and the swelling of the concrete on the panel part became noticeable.

In the case of specimen B, the occurrence of cracks in the projective beam would not be observed sufficiently, but the repturing process was approximately the same as with the specimen A.

Maximum Strength As shown in Table 8, the maximum strength is more than the calculated values derived from the bending strength, because the end of the beam is sufficiently connected inside the panel, and it is confirmed that the bending strength is fully exhibited. Table 9 shows the shear stress.

After surpassing the maximum stress as a result of the increased width of the cracks in the panel part's concrete, and the collapse by compression, the strength dropped 10 to 15% around the level of $R = 50 \times 10^{-3}$ rad in the end of the experiment.

**Table 8 Cracking Load and Maximum Strength**

(unit: ton)

| Load | | Specimen A | Specimen B |
|---|---|---|---|
| Bending crack | Beam | 23.9 | 15.4 |
| | Column | 111.4 | 61.1 |
| Flexural shear crack | Beam | 56.9 | 56.6 |
| | Column | – | – |
| Shear crack | Beam | 111.4 | 103.1 |
| | Column | 161.2 | 168.0 |
| Max. strength | Experiment | 201.8 | 186.3 |
| | Calculated | 180.6 | 175.2 |
| | (Mu)*1 | (1.12) | (1.06) |

*1 $Mu = 0.9 \times a_t \times s\sigma_y \times d$

**Table 9 Comparison of The Shear Stress Levels Inside The Panel Part**

(unit: kg/cm²)

| Load | | Specimen A | Specimen B |
|---|---|---|---|
| Cracking stress | Experiment | 31.6 | 15.4 |
| | Calculated | 52.4 | 41.5 |
| | ($p\tau_c$)*1 | (0.60) | (1.02) |
| Max. stress | Experiment | 91.7 | 118.4 |
| | Calculated | 103.6 | 109.3 |
| | ($p\tau_u$)*2 | (0.89) | (1.08) |

*1 $p\tau_c = F_t\sqrt{1 + \sigma_0/F_t}$, $F_t = 1.6\sqrt{F_c}$

*2 $p\tau_u = \tau_c + \tau_s$ (Kamimura's equation)
$\tau_c = 95$ ($F_c \geqq 244$ kg/cm²)
$\tau_s = 0.5 \times P_w \times s\sigma_y$

**Table 5 Specimen Factors**

| Factors \ Specimen | | Specimen A | Specimen B |
|---|---|---|---|
| Subject part | | Peripheral beam –column connections | Internal beam –column connections |
| Column | Section dimensions | 90 x 87 (cm) | 85 x 85 (cm) |
| | Ratio of all main reinforcement | 2.65% (16-D41) | 2.97% (16-D41) |
| Beam | Section dimensions | 58 x 90 (cm) | 60 x 80 (cm) |
| | Tension reinforcement ratio (Pt) | 1.53% (6-D38) | 1.85% (4-D41, 2-D38) |
| Connection part | Anchorage method for beam's main reinforcement | Double U-shaped anchorage | Straight anchorage |
| | Anchorage length | The length of horizontal part 1 step bar: 18.3d 2 step bar: 15.4d | 20.7 d |
| Orthogonal beam | | None | Only on one side |
| Slab reinforcement | | None | 2-D22 reinforced |
| Reinforcement of the connection part against shear | | D16 @100 (SD35) | $\phi$11 @100 (SBPD 130/145) |

**Table 6 Mechanical Property of Bars**

| Diameter and grade | Yield strength (kg/cm²) | Tensile strength (kg/cm²) |
|---|---|---|
| D41 (SD40) | 4370 | 6520 |
| D38 (SD40) | 4330 | 6400 |
| D16 (SD35) | 3890 | 5920 |
| $\phi$11 (SBPD130/145) | 13500*1 | 15000 |

*1 0.2% offset strength

**Table 7 Concrete Strength**

| Specimen | Compression strength (kg/cm²) | |
|---|---|---|
| | Connection part | Average value in other ports |
| A | 525 | 491 |
| B | 502 | 430 |

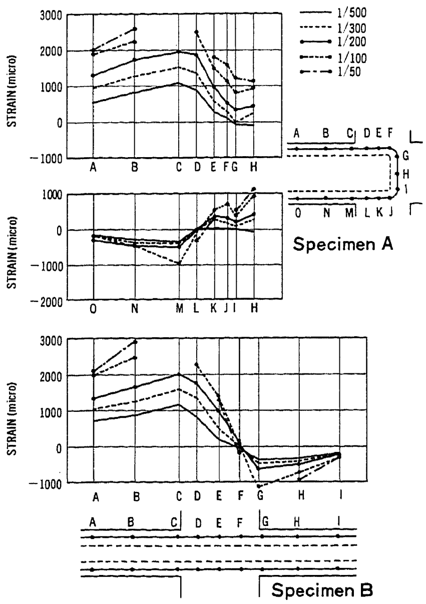


**Fig. 10 Distribution of Strain in The Beam's Main Reinforcement**

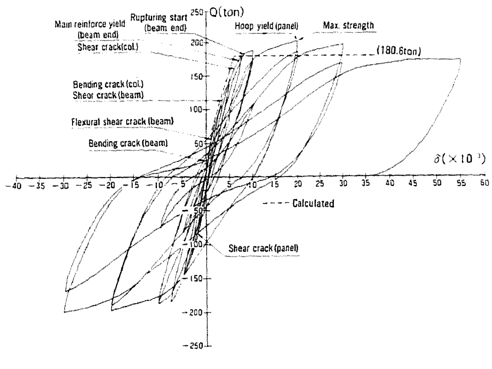

Fig. 11 Relationship Between Q and δ (Specimen A)

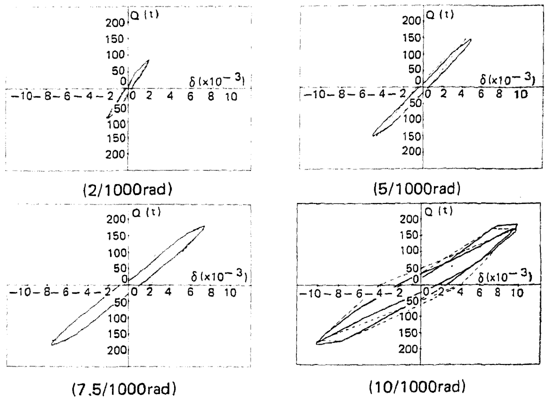

Fig. 12 Q-δ Relation in Each Cycle

The Distribution of Strain in The Beam's Main Reinforcement Fig. 10 shows the distribution of strains inside the joint part of the beam's main reinforcement. In both cases the tensile force didn't reach the reinforcement of the beam on the compressed side until R = 1/50 rad, and the anchorage had sufficient strength.

Comparison of The Experiment Results and The Analysis The dotted line in Fig. 11 shows the skeleton curve used in the analysis. Fig. 12 shows a comparison between the hysteresis loop used in the analysis and the experiment values in each stage of the load applying test on specimen A. From these figures it can be evaluated that the Takeda Model used in the analysis is relatively close to the results received from the experiments.

## CONCLUSION

The adoption of precast concrete improved accuracy in member fabrication and assembling, simplified field work, and reduced construction time. On the other hand, it was necessary to decide the bar arrangement of the beams and columns giving continuous consideration to details unique to this construction method.

Also on the beam-column connections with these details, the earthquake resistance and the appropriateness of the hysteresis rule used in the dynamic analysis were supported by the full scall model tests.


## ACKNOWLEDGEMENTS

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## REFERENCES


1. Nagashima, T. et al, "An Experimental Study on Beam-Column Joints for High-Rise Reinforced Concrete Buildings" Technical Papers of Annual Meetings of AIJ, 347-348, (1986).