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# Structural Safety of Precast Concrete Apartment Houses against Earthquake

La sécurité structurale de bâtiments d'habitation en béton précontraint vis-à-vis des tremblements de terre

Die konstruktive Sicherheit von Wohnhäusern aus Betonfertigteilen gegen Erdbeben

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## 1. Introduction

This report discusses the structural safety of publicly operated precast concrete apartment house develped in Japan, based upon the tests on full-size housing structures, carried out in the Large Size Structures Testing Laboratory.

This laboratory has a double decked testing bed of 9.5 m x 20 m in plan, and a vertical reaction wall of 16.1 m in height as shown in Fig. 1. Horizontal loads can be applied both in pushing and pulling direction against the reaction wall by means of 20 oil jacks of 50 ton capacity each. Three dynamic exciters of the maximum capacity of 10 ton power each were also furnished to perform forced vibration tests.

## 2. Construction systems and test specimens

Four tests have been carried out in the above laboratory since 1967 to provide the basic data in regard to the rationalization and the economization of the earthquake resistant design of precast concrete structures.

The outline of the test specimens and some connection details are summarized in Fig. 3 to 7.

The first test was concerned with the 5-story precast concrete large panel construction. Horizontal connections between concrete panels were made by welding of steel plates anchored in these panels. Vertical connections were cast-in-place connections. Each panel had six shear keys and reinforcing bars of 9 mm diameter anchored in the center of each shear key were welded to the vertical reinforcing bars placed through vertical joints. The test specimen consisted of ten housing units as shown in Fig. 7-1.

The second test was carried out in order to realize an 8-story apartment buildings with precast concrete large panel constructions. In this case, the splice sleeves filled with non-shrink mortar were used to connect vertical reinforcing bars cast in precast wall panels instead of welding connections. The test specimen shown in Fig. 7-2 represented the transversal shear wall structure of real 8-story apartment building.

The third test was concerned with the precast concrete structure assembled with post-tensioning method. This housing structure consisted of following precast reinforced concrete elements: L, +, T, I shaped wall columns, shaped girders and slabs. Prestressing bars anchored in underground girders cast-in-place were extended through wall columns and girders and tensioned and grouted after completion of assembly works of every five stories. The outline of this construction system is summarized in Fig. 5. Specimen 3-1 was the 4-story wall frame structure consisting of full size structural elements almost same as those of the lower 4-story of a real 8-story apartment building. Specimen 3-2 corresponded to transversal wall frame structure full-filled with bearing wall.

The last test was intended to realize an 11-story precast concrete apartment building which had 14-bay rigid frame structures in the ongitudinal direction and shear wall structures in the transversal direction. The typical precast reinforced concrete member in the rigid frame structure had double cross shape as shown in Fig. 6-a). These members were connected each other at the ends of the each member, that is at the mid-span of beams and columns of the frame structure. The longitudinal reinforcing bars were connected with screws in the beams and with splice sleeves filled with non-shrink mortar in the columns. The transversal shear wall consisted of precast wall panels with top beam. Connection details between wall panels and columns are shown in Fig. 6-d). The specimen 4-1 was of a 4-story full size model representing the rigid frame of an 11-story building, and specimen 4-2 was of an 8-story half size model representing the transversal shear wall structure.

## 3. Test procedure

In each test, a specimen was fixed to the testing bed and horizontal loads were applied to the specimen using oil jacks fixed to the vertical reaction wall. The distribution of horizontal loads in each test is shown in Fig. 8. Except the first test, vertical axial loads representing the dead and live loads of the upper portion of an actually designed building were also applied during horizontal loading as shown in Fig. 2.

The test specimen was loaded staticaly up to collapese in the horizontal direction. In the test No.1 and No.3 some supplemental forced vibration tests were carried out to clarify the change of dynamic properties such as natural period and damping, according with the progressing of failure.

## 4. Test result

The principal test results on static horizontal loading are summarized in Table 1 and Fig. 8. The ordinate and abscissa of Fig. 8 show the ratio of total applied loads in each test to the sum of dead weight and additional live loads supposed to be beared in the corresponding portion of the actually designed building and the translation angle defined as the ratio of total horizontal displacements measured at the top of the specimen to the total height of it, respectively. The design load was defined, hereafter, as the loads corresponding to 20% of the above mensioned gravity loads.

Specimen 1 : Flexure cracks appeared in beams just at the design load. As the load increased, flexure cracks in beams and shear cracks in walls progressed successively. The maximum load carrying capacity attained to 7.6 times design load with the translation angle of $13.5 \times 10^{-3}$ at the second floor and of $2.7 \times 10^{-3}$ at the top. Considerable decreasing of load carrying capacity due to shear failure of wall panels was observed after the maximum load. The natural period and damping obtained from the forced vibration tests were 0.1 sec. and 6.3%, respectively before static loading test and 0.17 sec. and 10.6%, respectively after maximum load attained.

Specimen 2 : Initial shear cracks appeared in the coupling beams at 2.5 times design load. The maximum load carrying capacity was attained at the 4.8 times design load with the translation angle of $5.1 \times 10^{-3}$ at the top. The coupling beams had heavily failed in shear before reached to the maximum load.

The final translation angle at the top was $21.7 \times 10^{-3}$.

Specimen 3-1 : The load displacement curves indicate a fairly linear relation up to 1.2 times design load. At this stage, no column cracks but slight cracks of girders were observed. The initially linear stiffness decreased gradually beyond this loading stage with gradual increase of horizontal slippage and progressing of bending cracks of wall columns and girders. The maximum load carrying capacity attained to 3.9 times design load, with the translation angle of $12.2 \times 10^{-3}$ at the 2nd floor and of $9.7 \times 10^{-3}$ at the top. The final failure mechanism was the shear failure of 2nd and 3rd floor beams and further repeating of load did not cause significant decrease of load carrying capacity up to the translation angle of $24.4 \times 10^{-3}$ at the 2nd floor and of $16.1 \times 10^{-3}$ at the top. The natural period and damping obtained from forced vibration tests were 0.12 sec. and 2% before applying any static load and 0.28 sec. and 7 to 9% after failure, respectively.

Specimen 3-2 : The load-displacement curve show linear relation up to 2 times design load. Beyond this loading stage, the vertical slippage between wall columns and bearing walls occured. The stiffness of the structure decreased rapidly after the applied load reached to 3.5 times design load due to occurreuce of shear cracks of wall columns and progressing of shear cracks of beams just above the vertical joint between wall columns and bearing walls. The maximum load carrying capacity was about 4 times design load and the translation angle at this loading stage was $5.3 \times 10^{-3}$ at the 2nd floor and $3.8 \times 10^{-3}$ at the top. The decreasing of load carrying capacity was about 36% of the maximum value at the translation angle of $10.2 \times 10^{-3}$ at the top.

Specimen 4-1 : Before reached to the design load, bending cracks of beams and columns had progressed extensively. Diagonal shear cracks in the panel zone of the beam-column connection occurred almost at the design load. At 1.7 times design load yielding of the main reinforcing bars at the ends of columns and beams and compression failure of concrete at the top of the 4-th story column and at the bottom of the 1st story column started. The maximum load carrying capacity was 1.8 time design load. The final displacement reached to 4.5 times the displacement at the yield load not showing any decreasing of load carrying capacity. This frame structure showed very ductile behaviour under horizontal loading, because the flexural failure at beams and columns was dominative instead of shear failure of the members in the other specimens.

Specimen 4-2 : The bending cracks at the 1st story column-column connection appeared at 1.3 times design load and considerable increase of crack width at this portion was observed in successive loading. The maximum load carrying capacity was 2.1 times design load at the translation angle of $4 \times 10^{-4}$ at the top. Beyond the maximum load, significant decrease of load carrying capacity was observed, because of the slipping out of longitudinal reinforcement at the 1st story column-column connection.

## 5. Concluding remarks

In case of precast concrete structures, it is not always easy to estimate the structural behaviour as a whole up to failure with analytical procedures, because of not only the difficulty of making model representing the behaviour of connection, but also the difficulty of considering the soundness of the structure affected by construction works. Tests on full scale structure as mentioned above give us the direct informations for estimating the soundness of structures as a whole under the earthquake loading. Further accumulation of such test datum will make us possible to estimate the extent of damage caused by earthquake more precisely and bring us the economization and the rationalization of earthquake resistant design of such the structures.

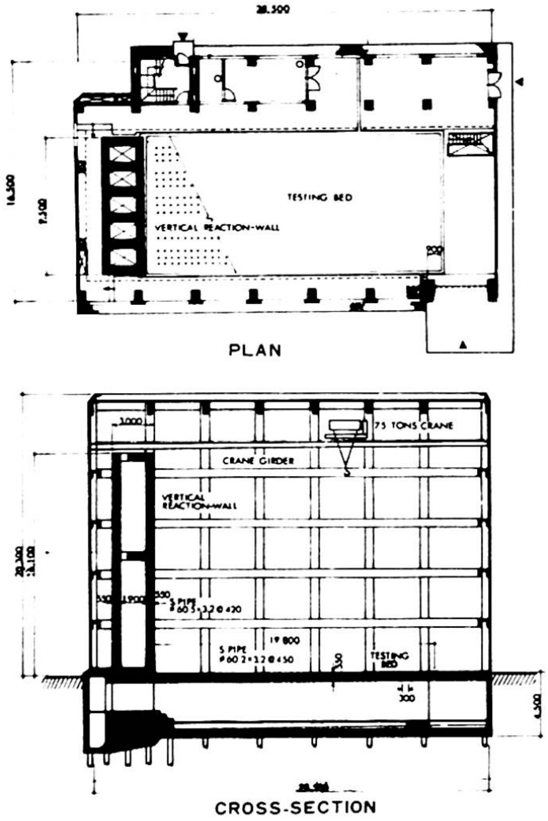


Fig. 1 Outline of Large Size Structure Testing Laboratory

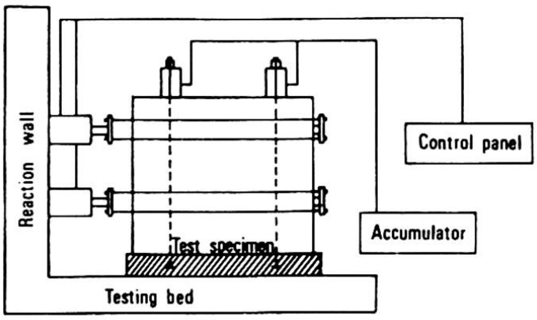


Fig. 2 Block Diagram of Loading Facilities

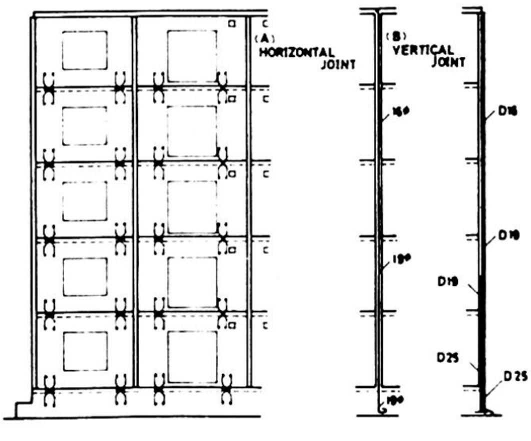


Fig. 3 Detail of Construction System of Specimen 1

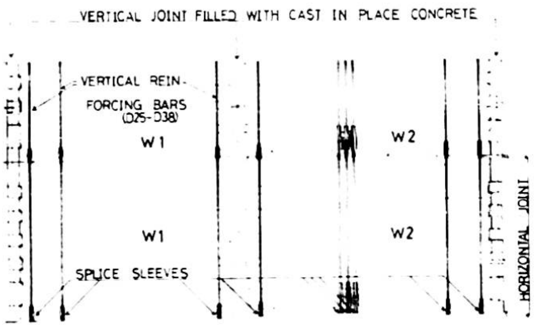


Fig. 4 Detail of Construction System of Specimen 2

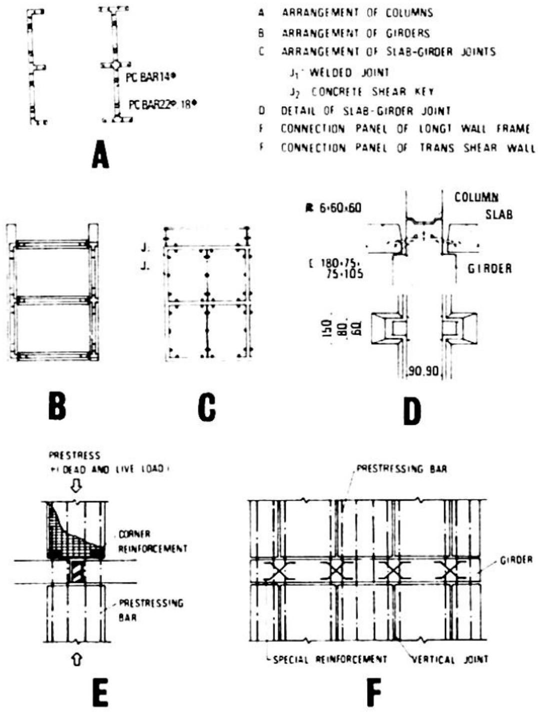


Fig. 5 Detail of Construction System of Specimen 3

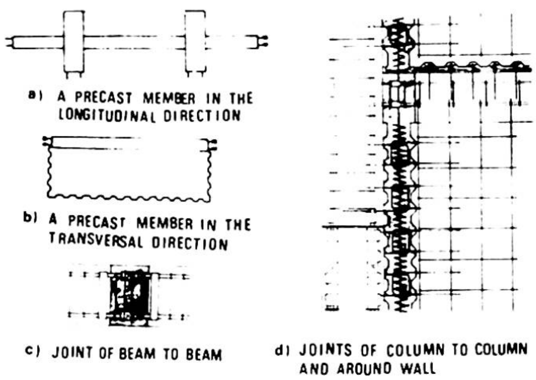


Fig. 6 Detail of Construction System of Specimen 4

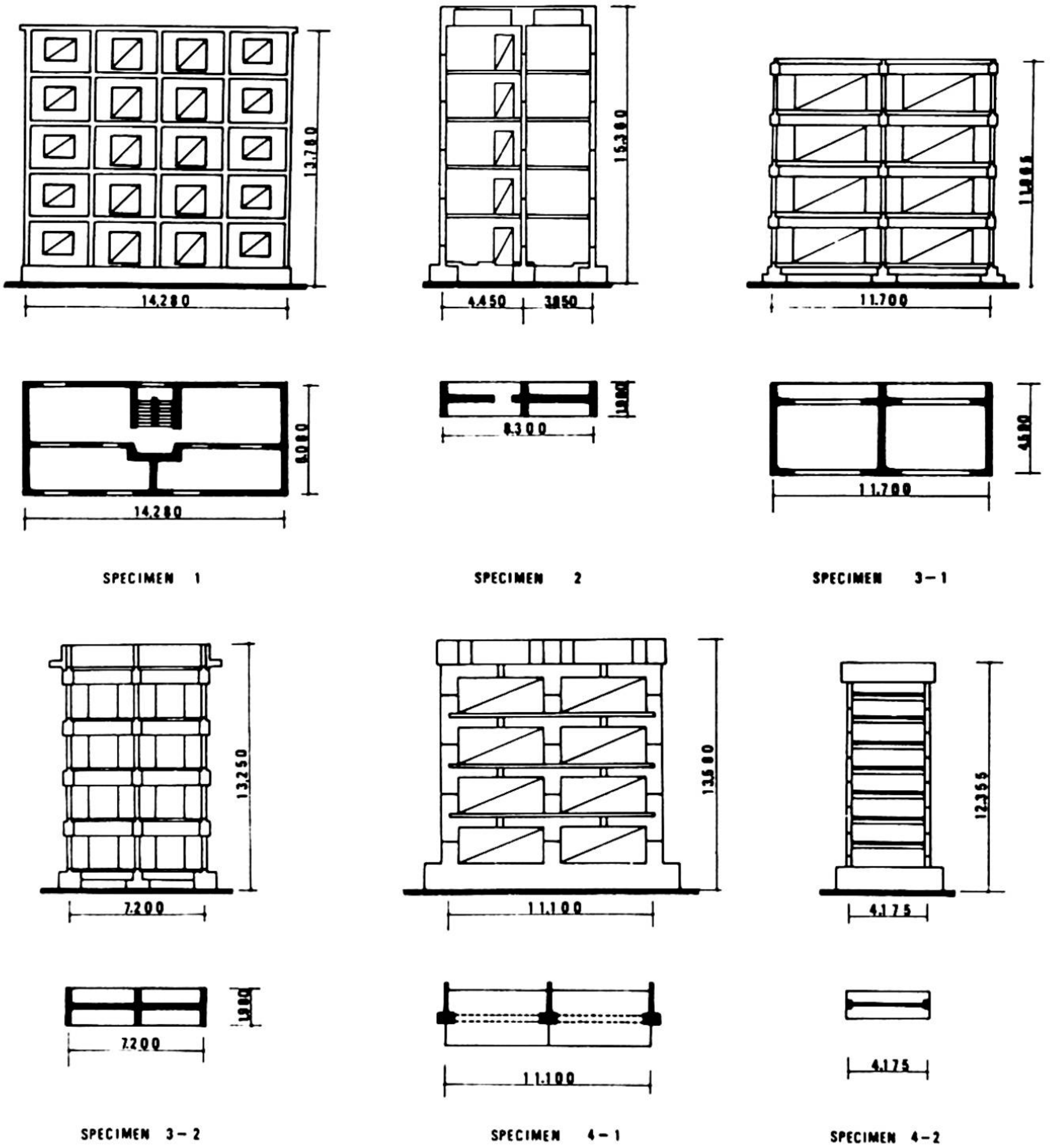


Fig. 7 Outline of Test Specimen

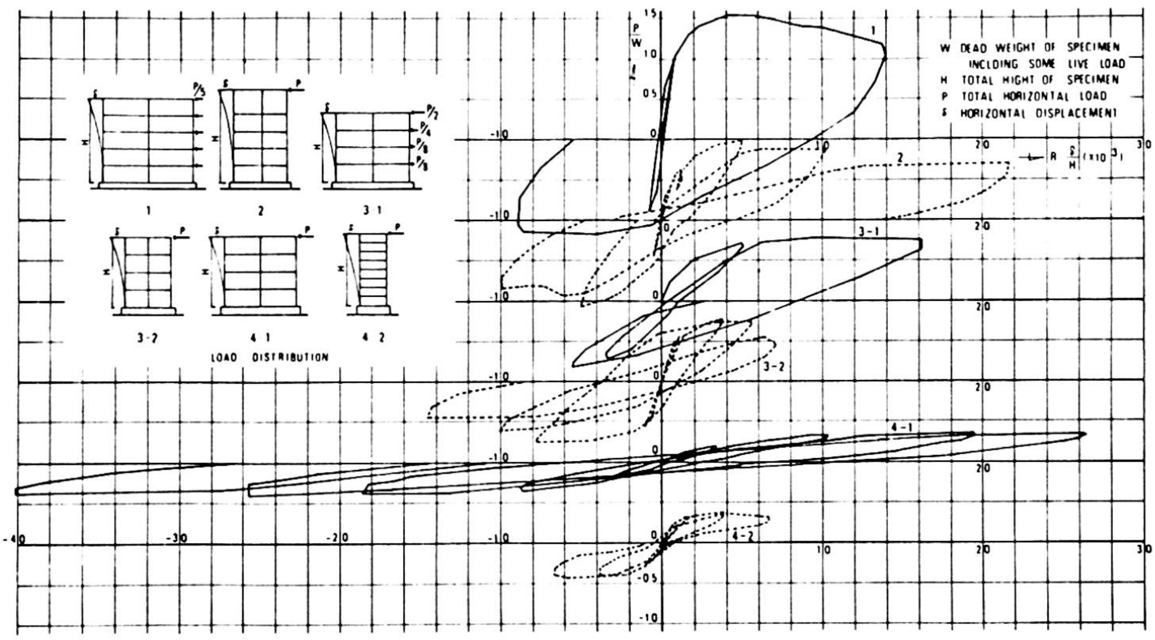


Fig. 8 Dimensionless Load-displacement Curves

Table 1 Result of Test

| NO. | | 1 | 2 | 3 | | 4 | |
|---|---|---|---|---|---|---|---|
| | | | | 3 - 1 | 3 - 2 | 4 - 1 | 4 - 1 |
| CONSTRUCTION SYSTEM | | R.C. LARGE PANEL CONSTRUCTION | R.C. LARGE PANEL CONSTRUCTION | P.C. CONSTRUCTION | | R.C. FRAME CONSTRUCTION | |
| STRUCTURAL TYPE OF SPECIMEN | | SHEAR WALL | SHEAR WALL | WALL FRAME | SHEAR WALL | RIGID FRAME | SHEAR WALL |
| NUMBER OF FLOORS OF TEST SPECIMEN (OF REAL BUILDING) | | 5 | 5 (8) | 4 (8) | 4 (10) | 4 (11) | 8 (11) (HALF SCALE MODEL) |
| THICKNESS OF WALL (cm) | | 15 | 18.25 | 18 | 18 | – | 7.5 |
| SECTION (cm) | BEAM | – | – | 30 × 74 | 30 × 74 | 40 × 55 | 20 × 27.5 |
| | COLUMN | – | – | – | – | 55 × 87 | 27.5 × 43.5 |
| SECTIONAL AREA OF WALL OR COLUMN PER UNIT FLOOR AREA ($cm^2/m^2$) | | 425 | 317 | 247 | 244 | 207 | 421 |
| LENGTH OF WALL PER UNIT FLOOR AREA ($cm/m^2$) | | 28.3 | 16.7 | 13.7 | 13.6 | – | 18.0 |
| REINFORCEMENT RATIO FOR SHEAR (%) | | 0.34 | 0.44 | 0.63 | 0.64 | 1.27 | 0.56 |
| CONCRETE STRENGT ($kg/cm^2$) | | 230 | 335 | 380 | 414 | 400 | 320 |
| MEAN SHEAR STRESS AT THE DESIGN LOAD BASED ON $k=0.2$ ($kg/cm^2$) | | 2.4 | 3.8 | 8.1 | 7.8 | 9.1 | 4.7 |
| MEAN SHEAR STRESS OF WALL AT THE MAXIMUM LOAD ($kg/cm^2$) | | 17.1 | 18.8 | 31.5 | 30.3 | 16.3 | 9.5 |
| $\tau_u/\tau_D$ | | 7.2 | 3.3 | 3.9 | 3.9 | 1.8 | 2.0 |
| COLLAPSE MECHANISM | | SHEAR FAILURE OF WALL COLUMN | SHEAR FAILURE OF COUPLING BEAMS | SHEAR FAILURE OF BEAM | SHEAR FAILURE OF BEAM | FLEXURAL FAILURE OF COLUMN AND BEAM | FAILURE CAUSED BY SLIPPING OUT OF REINFORCEMENT AT JOINT OF COLUMN |
| TRANSLATION ANGLE OF THE FIRST STORY AT MAXIMUM LOAD ($10^{-3}$) | | 7.8 | 4.2 | 20.8 | 22.2 | 20.0 | 5.9 |

* R.C.; Reinforeced concrete, P.C.; Prestressed concrete


## SUMMARY

This report discusses the structural safety against earthquake of precast concrete apartment houses developed in Japan. The evaluation is based upon tests on full-size model structures. The results show that the construction system of precast elements have a sufficient load carrying capacity and ductility to resist severe earthquakes.

## RESUME

La sécurité structurale des bâtiments d'habitation préfabriqués en béton, développée au Japon vis-à-vis des tremblements de terre, a été testée sur des essais en vraie grandeur. Les résultats montrent que les capacités portante et de déformation de ce type de construction sont suffisantes pour résister à de sévères tremblements de terre.

## ZUSAMMENFASSUNG

Die Abhandlung beschreibt die konstruktive Sicherheit von in Japan entwickelten Wohnbauten aus Betonfertigteilen. Die Untersuchung gründet sich auf Versuche im Massstab 1:1. Die Resultate zeigen, dass Konstruktionen aus Betonfertigteilen ausreichende Tragfähigkeit und genügendes Verformungsvermögen aufweisen, um auch schweren Erdbeben zu widerstehen.