Proceedings of Ninth World Conference on Earthquake Engineering
August 2-9, 1988, Tokyo-Kyoto, JAPAN (Vol.VI)

9-1-11

SEISMIC PERFORMANCE OF LOW-RISE PRECAST REINFORCED CONCRETE STRUCTURES —EXPERIMENTAL STUDY AND ANALYSIS—

Yasuhisa SONOBE[1], Yoshimasa OWADA[2] and Masaya HIROSAWA[3]

[1] Institute of Engineering Mechanics, University of Tsukuba, Tsukuba, Ibaraki, Japan
[2] Department of Architecture and Building Engineering, Shibaura Institute of Technology, Minato-ku, Tokyo, Japan
[3] Building Research Institute, Ministry of Construction, Tsukuba, Ibaraki, Japan

SUMMARY

Presented in this paper is seismic performance of typical 2 cases of low-rise industrialized housing systems of medium size reinforced concrete panels. The houses are composed of panels only in one case and of wall panels and floor slab cast in place with floor supporting precast beams in another case. From full scale model test it is observed that ductility is sufficient and ultimate shear strength is increased due to mutual constraint between members, which is derived from their three dimensional arrangement. The three dimensional effects are classified and evaluated in order to establish the standard of aseismic estimation method of the systems.

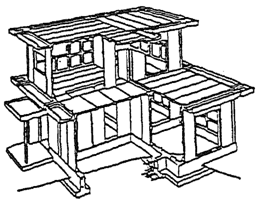
INTRODUCTION

In Japan there are about 20 industrialized housing systems of low-rise precast reinforced concrete, which have been mainly developed for 2 or single storied individual and apartment houses.

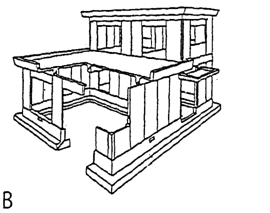
As social demand to three-storied private houses is recently increasing, authors tried to make clear problems on seismic properties of the existing systems in order to apply the results to develop new systems. Concerning aseismic safety of newly devised special structures, each system shall be approved by the minister of construction based on experts' appraisal with technical data, that is, drawings and specification for structural design and construction method and results of structural experiments. Authors collected such data of all the existing systems and analyzed them mainly on problems related to their seismic performance. This paper presents the performance of typical systems. Besides in Japan, low-rise buildings are required not to be collapsed by strong earthquake, elastic response acceleration of which is 1,000 gals.(Refs. 1 and 2)

CONSTRUCTION SYSTEMS

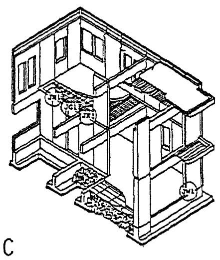
Most of all low-rise precast reinforced concrete structures, some types of which are shown in Fig. 1, are so-called wall structures and divided roughly into large-size panel structures and medium-size panel structures with or without floor tie beams.



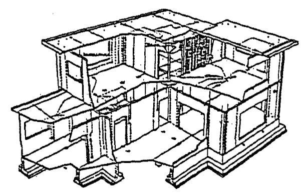
Type A
Wall : Medium panel of thin shell
Floor: Medium panel of thin shell



Type B
Wall : Medium panel of thin shell
Floor: Slab and tie beam cast in place



Type C
Wall : Medium plank panel
Floor: Tie slab cast in place with precast beam



Type D
Wall : Large panel
Floor: Large panel

Fig. 1 General View of Typical Precast Concrete Construction

Structure of Medium Size Panel of Rib and Thin Shell Wall panels used in this system, which was developed as low cost public housing one, are of height of one story and of depth ranging from 1 module of 90 cm to about 2 modules. As shown in Fig. 2, standard wall panel consists of thin plate of 4 cm thick, so called shell, rib at perimeter of depth of 12 cm and horizontal cross pieces. Compressive strength of concrete is 30 MPa. Floor panels are placed between wall panels of upper and lower story. Therefore structural function of the floor system affects greatly seismic performance of the structure. At cross point of wall panel lines a column is arranged in order to keep module scale system. As shown in Fig. 3, adjacent wall panels and column are jointed with common bolts of grade 4T of tensile strength of 400 MPa. Floor panels are the same to wall ones with ribs of suitable depth for supporting span length up to 5 modules.

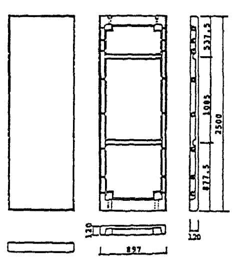


Fig. 2 Standard Medium Size Wall Panel

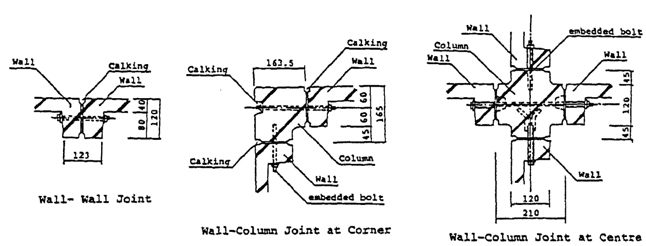


Fig. 3 Detail of Vertical Joint

Because of lacking in floor tie beams this system is fundamentally cantilever wall structure and obvious frame effect can not be expected. Bearing capacity and ductility of standard shear wall panel are much influenced by quality and shape of connecting bolts. Further, though they try to make the panel shell thin to decrease panel weight, shear capacity and time deterioration of panels and connecting details by bolts shall be discussed.

Lateral loading test using full scale 2 storied model of this type structure (Ref. 3) was carried out and aseismic design method is composed almost referring its results. Plan of wall panels is shown in Fig. 4. There are sagging wall panels at each opening and spandrel walls on 2nd floor. Those secondary members have clearance of 6 mm at the joints to wall panels as shown in Fig. 5, though having joints for shear, in order to not make interaction with structural members up to large drift. Equal lateral cyclic loads were applied at roof and 2nd floor in transverse direction. The inside view of the model is shown in Fig. 6. Total lateral load-drift curve of 1st story and crack patterns are shown in Figs. 7 and 8, respectively. Main test results are as follows.

1) As the principal aseismic elements are thin-shelled shear wall panels, plastic deformation ability of the structure is considered not to be superior to that of frame structure. Therefore, necessary horizontal bearing capacity is generally regulated to be more than 0.55 of base shear coefficient. However, strength deterioration was not observed even under the deformation of 3/100 actually as shown in Fig. 7. There is much room to improve their ductility.
2) Shear failure at wall panel did not take place.
3) Some horizontal joint bolts at tensile side of bearing wall panels broke after yielding.
4) Wall panels arranged normal to bearing wall at tensile side gave constraint effect to it. Tensile stress of horizontal joint bolts of the normal wall panels attained to a half of yield point. Comparing the tension of the horizontal joint with shearing resistance due to vertical bolts between normal and bearing wall panels and floor panel, constraint capacity of floor panel could be quantified.
5) Slip took place between adjacent bearing wall panels and bolts at vertical joint of wall panels became effective for shearing action at story drift angle of larger than 1/300.
6) Anchor bolts at tensile side of bearing wall of 1st story yielded at total lateral load of about 60 t and maximum lateral load was 80.0 t. Theoretical ultimate load in consideration of 5 factors, which contribute to the increase of lateral load and are mentioned below, is 64.8 t, using yield point of steel materials. If we use their strength, the value goes up to 84.8 t. The factors are as follows. (1) Bending strength of bearing wall panel without any constraint, (2) effect due to normally arranged wall panel at tensile side of bearing wall panel, (3) effect due to continuously arranged bearing wall panels, (4) effect due to constraint by sagging wall panel and (5) factor 4 in consideration of increase of bending strength of sagging wall panel by axial force induced by lateral loading. Analyzed share of loads due to the factors at ultimate stage is shown in Table 1. As the share due to factor 2 is 26%, it can be said that normal arrangement of wall panel greatly contributes to the increase of lateral shear. In addition total weight of the model was 46.1 t including live load.
7) At stage of story drift angle 1/100 secondary members did not give harmful effect to structural members. For the large drift secondary members had relation to the behavior of the model.
8) Ranging from elastic to plastic deformation, the behavior of the model could be analyzed in consideration of bending and shearing deformation and rotation due to elongation of bolts at horizontal joints of wall panels.

Structure of Tie Slab System Precast members at this system are wall panel and floor supporting beam of light weight concrete of compressive strength of 15 MPa. Standard wall panel is 12 cm thick and 1 module wide. Floor slab and continuous footing beam are cast in place. General view and main joints are shown in Fig. 9. Adjacent wall panels are connected at top with steel plate of 6 mm thick. (JH1 in Fig. 9) Wall panels are connected to floor slab cast in place with stud bolts. (JK1 in Fig. 9) Longitudinal reinforcement of wall panel is vertically connected at horizontal joint like the manner of anchor bolt, using joint metal anchored in upper wall panel. (JW1 in Fig. 9) Upon the opening precast floor supporting beams are installed between wall panels. The beam is effectively connected to wall panel

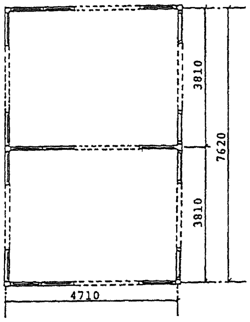


Fig. 4 Wall panel plan
of full scale model

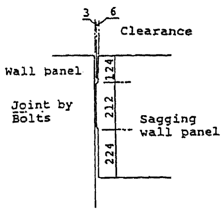


Fig. 5 Joint of sagging wall panel
to bearing wall panel

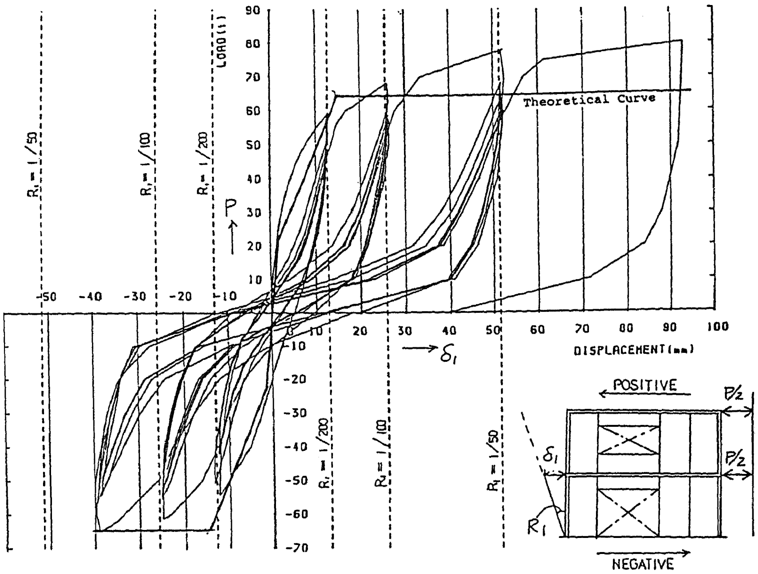


Fig. 7 Total lateral load-drift of 1st story curve

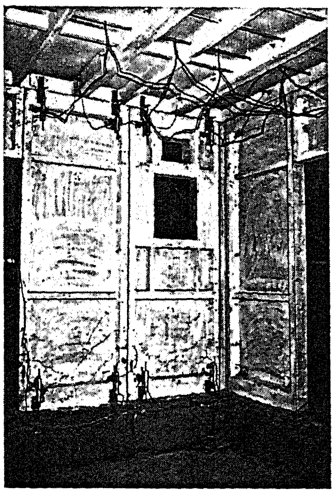


Fig. 6 Inside view of the model

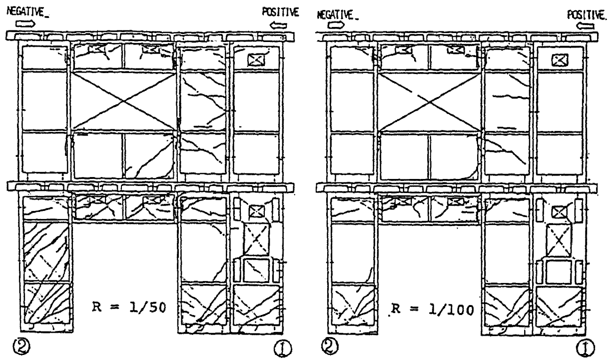
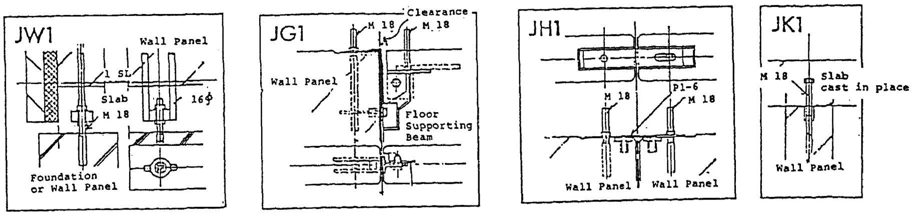


Fig. 8 Crack patterns

Table 1 Share of factors to ultimate lateral load

No. of factor	1	2	3	4	5
%	32	26	12	18	12

Factor 1: Bending strength of wall panels
Factor 2: Effect due to normal wall panels
Factor 3: Effect due to continuous wall panels
Factor 4: Effect due to sagging wall panels
Factor 5: Factor 4 due to axial force of sagging
wall panels induced by loading



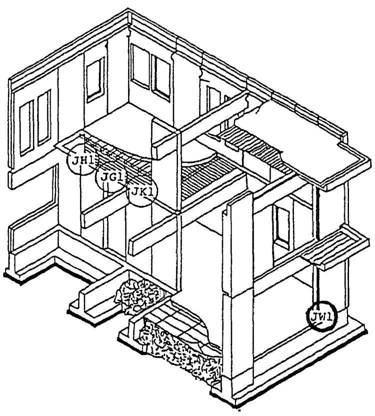


Fig. 9 Tie slab system

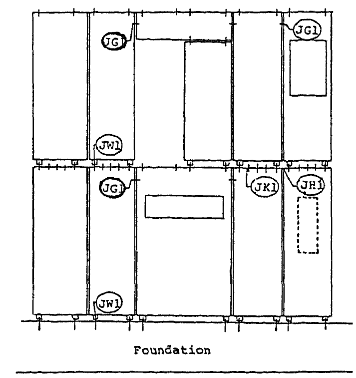


Fig. 10 Example of structural plane

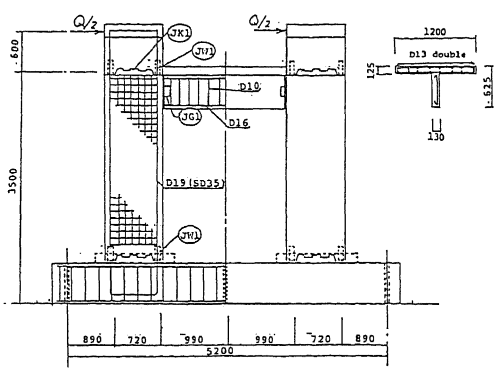


Fig. 11 Specimen of portal frame

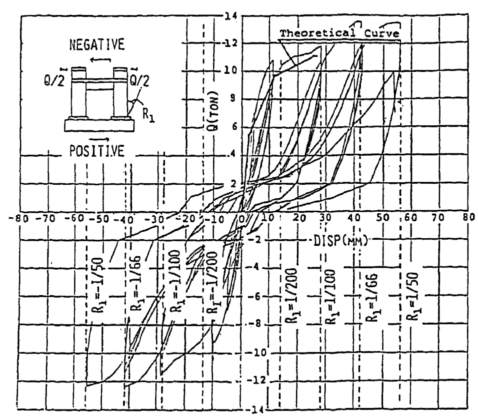
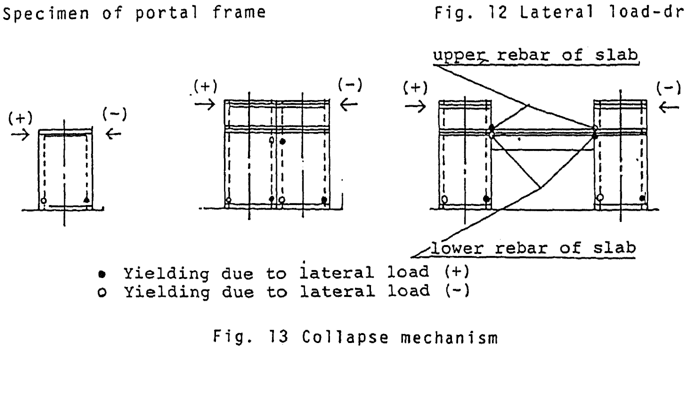


Fig. 12 Lateral load-drift curve



- • Yielding due to lateral load (+)
- o Yielding due to lateral load (-)

Fig. 13 Collapse mechanism

for shear with bolts and joint metal of spheroidal graphite iron castings, but slit of clearance 10 mm is set in order to be able to rotate for bending. (JG1 in Fig. 9) In the same manner other nonstructural members are loosely connected with structural members. An example of structural plane is shown in Fig. 10, where bearing wall panels and non structural members are clearly distinguished.

Lateral loading tests (Ref. 4) were carried out using full scale wall panels. Specimen of portal frame type one and a half story is shown in Fig. 11. Lateral loads were applied at supposed inflection point of wall panel of 2nd story. Lateral load-drift of 1st story curve is shown in Fig. 12.

Ultimate lateral shears both with other 2 tests are well explained by collapse mechanism due to bending failure of wall panels and slabs as shown in Fig. 13. From results of tests it is concluded that effective width of slab of 12.5 cm thick is 120 cm for bending and 40 cm for shearing constraint between continuous wall panels. When drift angle exceeds 1/100 at the test of portal frame, lateral shear increases because of constraint due to floor supporting beam.

CONCLUSION

Concerning seismic performance of existing low-rise precast reinforced concrete construction systems, authors made clear that the performance of the whole structure is determined not only by that of wall panels but influenced by normal walls, continuous walls, floor panels or slab, floor tie beams and spandrel walls.

ACKNOWLEDGEMENTS

Authors sinserely appreciated the cooperation of Prof. Y. Matsuzaki of Science University of Tokyo, Prof. E. Makitani of Kanto Gakuin University and Mr. H. Kanaoka of Building Center of Japan in carrying out this work.

REFERENCES

1. Sonobe, Y. and Hirosawa, M. et al., "Structural Performance of Low-rise Precast Concrete Panel Structures (in Japanese)," Summaries of Technical Paper of Annual Meeting, AIJ, C, 89-102, (1987)
2. Sonobe, Y., "State of the Art of Lowrise Precast Reinforced Concrete Structures in Japan," U.S.-Japan Seminar on Precast Concrete Construction in Seismic Zones, 2, 21-34, (1986)
3. Iwata, S. et al.,"Seismic Resistance Properties of Prefabricated Small Sized Concrete Panel Structure (in Japanese)," Kenchiku-Gijutsu, 364, 107-121, (1981)
4. Taisei Corporation, "Report of Structural Experiment of P-2 Project for Concrete Sectional Meeting of Industrialized Appraisal Committee of Building Center of Japan"