EXPERMENTAL AND ANALYTICAL STUDY ON THE EARTHQUAKE RESISTANCE OF WALL-TYPE SPACE STRUCTURES

by
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SUMMARY

In this paper are described the full-sized tests conducted on three wall-type apartment buidings together with fundamental studies on the simplest walled units consisting of multi-panels-assemblaged walls and slab. Based on these test results are developed the methods to evaluate the horizontal load-deflection relationships for wall-type space structures.

INTRODUCTION

Wall buildings have been much less damaged by earthquakes than ordinary rahmen-type buildings. One of the reasons is that the plane frames of wall-type buildings in the horizontal loading have much higher strength, in addition to high stiffness, than those of rahmen-typed buildings. The other main reason is that the right-angled frame walls in space structures can cooperate, that means, horizontal resistance capacity of wall-type, space structures is much greater than the sum of the resistance capacity of plane frames. There have been, however, little studies on the horizontal resistance capacity of wall-type space structures, particularly based on full-sized tests.

This paper consists of the three stages as follows.

1) Description of a full-sized test conducted on wall-type two-storied RC house, together with the other two full-sized tests of wall-type space structures (five-storied and two-storied) done by other institutes in Japan. 2) Description of the full-sized tests on simplest wall-type structures consisting of multi-panels assemblaged walls and slab. 3) Development of the methods to evaluate the horizontal load-displacement relationships for wall-type space structures, by using both the results of 1) and 2).

DESCRIPTION OF THE TEST ON WALL-TYPE TWO-STORIED HOUSE

Test Program

This test designated as A test in this paper was conducted by a construction company of Hiroshima city (Ref.1) under supervisions with Prof. Kokusho, Tokyo Institute of Technology as well as the author. The aim of this test is to develope the earthquake-proof and rationally designed house consisting of the walls and slabs in the prefabrication system.

The basical items, such as the wall layout in plan are consistent with the wall-type RC building code of Japan. The full-sized space specimen for test is the half of one unit house of two-storied setback apartment buildings, as shown in Fig. 1. The plans with the arrangements of botton joint reinforcements are also shown in Fig.2. The reinforcements designed to current Japan practice in walls are illustrated in Fig. 3. Material properties are listed in Table 1. Horizontal loads were applied to three wall planes at the roof and secound floor slab levels throught auxilary steel beam as seen in Fig. 1. The total weight of the roof floor

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including the secound story walls of half height is about 17_{ton} (dead load), while that of the secound floors including both upper and lower walls is about 30 ton. Thus the base shear coefficient C (the first story shear to total weight) become one when the horizontal load P of about 20 ton ($=47^{ton}/2{,}424$) are applied as seen in Fig. 1. Reversals of horizontal loads were applied at each load level up to ultimate as shown in Fig. 4. The displacements at each points and the strains in both bottom joints and in-panel reinforcements were measured.

Test Results

The crack patterns at ultimate are illustrated in Fig. 3. The horizontal load-deflection curve at roof level at the middle is shown in Fig. 4. The test results can be summarized as follows.

1) At the load $P = 4^{ton}$ corresponding to load to weight ratio C of 0.2. there were no cracks found with roof-level horizontal deflection being 0.2 mm (deformation angle $R = 0.04 \times 10^{-3}$ rad. obtained by dividing with the total height 500cm). The calculated deflection by frame analysis was 0.15mm at the $P = 4^{ton}$.

2) Small cracks were found in the corner of opening for wall panel at the $P = 12^{ton}$ ($C = 0.6$).

3) The top deflection at the $P = 20^{ton}$ ($C = 1.0$) was 1.30mm ($R = 0.25 \times 10^{-3}$) with crackes being still found only arround the corner of the opening for wall panel. The maximum values of strains in both joint and in-panel reinforcements found at this stage was about 500×10^{-6}.

4) At the $P = 30^{ton}$ ($C = 1.5$), the cracks on the corners of opening extended to flexural cracks for both the beam and column parts.

5) At the $P = 40^{ton}$ ($C = 2.0$) the shear-type cracks were found in column parts, with the vertical cracks being found at the boundary with right-angled panels.

6) At the $P = 53^{ton}$ ($C = 2.65$), these cracks became considerable with the strains in some of base joint and in-panel reinforcements yielding. The deflection at this stage was about 15 mm ($R = 30 \times 10^{-3}$ rad.) After unloading from this stage, the last negative direction load was applied, reaching 50 ton with the deflection being about 30 mm ($R = 60 \times 10^{-3}$);

Two other full-sized tests on wall-type structures

One of the two other tests (B test) to be explained herein was conducted by the Building Research Institute of the Ministry of Construction in Japan, under request of the Japan House Public Cooperation (Ref. 2). The specimen was a full-sized five-storied prefablicated apartment buildings. The plan with the arrangement of bottom joint reinforcements and the elevation are shown in Figs. 5 and 6 respectively. The material properties used is listed in Table 1. For each wall plane in the longitudinal direction of the buildings, the same loads were applied at each floor level, resuting in totally fifteen points loading. The cracks patterns at ultimate and the horizontal load-deflection curve at the roof level are illustrated in Figs. 6 and 7 respectively. The maximum load was 615 ton in total.

The other test (C test) was conducted by a construction company, Tokyo (Ref. 3). The specimen was a full-sized part of the two storied house. The plan and its elevation are shown in Fig. 8. respectively. Light-weight concrete was used. The two points loads were applied at both roof and the second floor levels. The load-deflection curve as well as crack patterns are shown in Figs. 9 and 8 respectively. The maximum load was 32 ton in total.

DESCRIPTION OF THE TESTS ON THE SIMPLEST WALLED STRUCTURES

Outline of structures

The direct aim of the tests was to clarity earthquake resistance of safe rooms or stack rooms installed on the several floors of buildings, together with the objective to get fundamental understanding on the resistance mechanism on the walled structures. These secondary structures in buildings were very recently developed in prefablication system by a manifacturing company, Hiroshima, Japan (Ref. 4). This wall-type structure consists of three elements, four side walls, top slab and steel beams. Walls are multi-panels connected with each other through steel angle attached at both top and bottom of the panels by welding or bolting, as well as the point welding or bolting joints along contact plane between panels as shown in Fig. 10. A panel is of concrete enclosed by thin steel plates. Steel angles at both top and bottom of the wall play important roles in connecting the wall with top slab and with base slab respectively as shown in Fig. 10. Top slab is also an assemblage of multi-panels spanned between wall and steel beam or two steel beams which are spanned between two opposite walls at right angle.

Test Program

In this paper two types of specimens tested are chosen and described; 1) eight paneled plane wall with and without right-angled three panels (Fig. 11). 2) space specimens (Fig. 10) in which a top slab with steel beam was spanned between two walls making up space structures, in order to obtain the inplane resistance capacities of top slab, the strength on the connections between the top slab and the walls as well as the resistance of two walls. In each type both the welding and bolting joints specimens were tested. However, bolts were always used for the connection between bottom steel angle and testing floor bed through auxilarily steel beams as shown in Fig.10. Horizontal loads were applied at the top of walls and at the middle of top slabs for the 1) and 2) type specimens, respectively. The one complete cycle of reversing load was applied at the amplitudes of the top deflection of 1.25cm 2.5cm and 5cm.

Test results

In Table. 2. are summarized test results. In Fig. 12. are shown envelope curves. In Fig. 13. are illustrated reversing curves. The outline of the test results are as follows.

1) The initial stiffness for welding-joints was about 16 t/cm and 40 t/cm for specimens with and without right-angled panels respectively. The maximum loads were 17^{ton} and 26^{ton} at the deformation angle of 1.0×1.0^{-2} rad respectively, at which stage some of the bottom boltings were broken.

2) The Initial stiffnesses for bolting joints wall were about the same as those for welding joints but they began to decrease from small deflection. Maximum loads were 15ton and 21.4ton with and without right-angled panels respectively. The deformation angles at this stage were about 2.0×10^{-2} rad. Some of the joint boltings between panels as well as the bottom bolting were broken at maximum load.

3) The Initial stiffiness a wall plane was about 5.0 t/cm for both the space specimens of the welding and bolting joints. The maximum loads were 10.4ton and 12.4ton for the welding and bolting joints respectively. The deformation angle at maximum were 1.4×10^{-2} rad. and 2.8×10^{-2} rad. including the top slab deformation angle of 0.4×10^{-2} rad. and 1.0×10^{-2} rad. respectively.

4) According to the test results, the horizontal resistance capacity to self weight for the case of safe room having the plan of 5.25m × 5.25m (eight-panels length square) can be calculated to be over 3 since the unit weight of the panel is about 0.15 t/m^2.

DEVELOPMENT OF EVALUATION METHODS

Itroduction

There have been described two types of full-sized tests on three prefabricated apartment buildings and on the simplest walled structured consisting of multi-panels elements. In the following, discussions are developed for each test results on the horizontal load-deflection curves, based on simple theoretical assumptions.

In general a reversals of loads-deflection curve can be decomposed into a skelton curve and hysteretic loops. A skelton curve obtained by experiments can be often expressed by numerical functions after being normarized with values of a peculiar point, such as maximum load and its deflection. Hysteretic loops can be also normarized using the values of amplitudes. However the most important factor from the practical view points of earthquake engineering is loop areas, such as equivalent viscous damping ratios. In the following these procedures are taken for each test results, with emphasis on basic understanding without any precise analysis for want of space.

Discusions for the results of building tests

In Fig. 14 are shown envelope curves for three apartment building specimens with the ordinate and abscissa being the horizontal load ratio to the weight of specimen and the deformation angle at the roof level respectively. In Fig. 15 are normarized these curves with the load and its deflection at maximum being set as a unit. It is found that two second-storied specimens have the same curve in spite of quite different plan and elevation, the other having a little different curve. It seems due to the fact that cracks were found mainly on the walls of the first story so that the total number of stories have considerable effects on the shape of curves. The curves can be approximately expressed by the functions shown in Fig. 15. Thus it is concluded that the envelope curves can be obtained if both the values of the load at maximum and the intilial stiffness are estimated as shown in Fig. 16. It was assumed that the cantilever having multi-layered box section without any opening provides the basic values on the maximum load and intial stiffness. Usually the total area ratios of bottom joint reinforcements to multi-layered concrete box section is extremely small. This means that the maximum strength of walled structures without any opening are detemined by flexural strength of the bottom joint section, not by shear or the strength of the non-bottom-joint section. Taking into considerations the fact that the bottom joint reinforcements ratio is so small that the center of the total compression forces corresponding to the sum of total yield strength of bottom joint reinforcements and building weight lies in the end walls in the box section, maximum flexural strength can be expressed as follows.

$$M_o = \sum_{j=1}^{m} a_t \cdot \sigma_y \cdot d_j + \left(\sum_{i=1}^{n} W_i\right) \times 0.5L \quad \ldots\ldots\ldots\ldots\ldots (1)$$

in which m, $a_t \cdot \sigma_y$ and d_j are the total number of the bottom joint reinforcements in the box section, tension strength and distance from the end in the box section for each bottom joint reinforcement with n, $(\sum_{i=1}^{n} w_i)$ and L being total number of stories, the total weight and the length in the loading direction for the buildings. loads at maximum, Po can be obtained readily. On the other hand, the initial stiffness regarding total loads versus roof-level deflection can be expressed as follows.

$$K_o = \left(\sum_{i=1}^{n} P_i\right) / \delta_n$$

$$= \left(\sum_{i=1}^{n} P_i\right) / \sum_{i=1}^{n} \left\{ \frac{P_i h_i^3}{3EI} + \frac{P_i h_i^2}{2EI} (n-i) h_i + k \frac{P_i h_i}{GA} \right\} \quad \ldots\ldots\ldots\ldots (2)$$

in which P_i . h_i EI and GA are load, story height, flexural rigidity and shear rigidity respectively.

It was assumed that the maximum load P and initial stiffness K for wall-type building can be experessed using the Po and Ko as follows

$$P = P_0\,(1-\lambda_x)^{\alpha_1}\,(1-\lambda_y)^{\beta_1}\,(1-\lambda_z)^{\gamma_1} \quad \ldots\ldots (3)$$

$$K = K_0\,(1-\lambda_x)^{\alpha_2}\,(1-\lambda_y)^{\beta_2}\,(1-\lambda_z)^{\gamma_2} \quad \ldots\ldots (4)$$

in wihich λ_x, λ_y and λ_z are the average values on the opening ratios in three directions as shown in Fig. 17. Substituing the values on the three test specimens shown in Table 1, the following can be obtained

$$\alpha_1 = 1.38 \quad \beta_1 = -0.10 \quad \gamma_1 = -0.14 \quad \ldots\ldots (5)$$
$$\alpha_2 = 4.57 \quad \beta_2 = -1.03 \quad \gamma_2 = -1.17 \quad \ldots\ldots (6)$$

It is found that λ_x values have prodominant effects on the P and K while the effects of λ_y and λ_z are small. Taking into considerations that the number of specimens is only three and that the flexural strength and stiffness are approximately proportional in first-order and in third-order to the section depth repectively, the following was assumed

$$\alpha_1 = 1.38 - 0.10 - 0.14 \fallingdotseq 1, \quad \beta_1 = 0, \quad \gamma_1 = 0 \quad \ldots\ldots (7)$$
$$\alpha_2 = 4.57 - 1.03 - 1.17 \fallingdotseq 2.5, \quad \beta_2 = 0, \quad \gamma_2 = 0 \quad \ldots\ldots (8)$$

By using these values, test results can be well explained as shown in Table 1. Further researches are, however, needed to improve the accuracy by adding much more data on wall-type structures, particulary for the values of different opening ratios. In Fig. 18 are finally shown the trend of the equivalent damping ratios as an index of hysteretic loops for the test results including the case of the test of multi-panels walls.

Discussions for the results of multi-panels walls

Multi-paneled walls are not reinforced concrete members but the test results can be used as a tool to obtain some information on the effects of right-angled walls as shown below. The same procedures as in the last section, are taken. In Fig. 19 are shown the normarization of the skelton curves for these tests indicating the similarity of the curves for welding-joints specimens to those of the building specimens and the possibility of some formulation for the curves for the both joint types. Using the Eq. (1) for the walls without any opening, the maximum strength is discussed. Disregarding the weight of the walls and top slab and decomposing the first terms in the Eq, (1) into the effects of both the loading and right-angled derections, the following can be derived.

$$M_0 = (\Sigma a_1 \sigma_r)\frac{D_1}{2} + (\Sigma a_2 \sigma_r)\,D_1$$

$$= (\Sigma a_1 \,.\, \sigma_r)\frac{D_1}{2}\,(1 + 2\Sigma a_2 \,/\, \Sigma a_1) \quad \ldots\ldots (9)$$

in which ($\Sigma a_1 \sigma_r$), ($\Sigma a_2 \,.\, \sigma_r$) and D_1 are total tension yield strength of bottom joint bolts for the loading and right-angled walls respectively and the depth of loading direction walls. In Fig. 20 are shown the comparision between the tested results and the values by Eq, (9) with the abscissa being the area ratio of bottom boltings in both the direction. It is found that tested values increase with the area ratio but are as a whole below calculated values. It can be assumed that if the joints between panels and bottom bolting are strengthened the tested values approach to the calculated ones which is the limiting values as indicated in that welding-joint types between panels show higher values. In Fig. 20 are also shown the tested results on initial stiffness. It is found that right-angled walls have considerable effects.

CONCLUSIONS

Based on the two types of experiments on wall-type space structures reported herein, the following conclusions can be drawn.

(1) The wall-type structures in reinforced concrete prefablication systems have significantly high strength as well as high stiffness for horizontal loadings. An evaluation method was developed on the strength and the stiffness for wall-type structuress, based on the test results. The properties on load-deflection curves decomposed into skelton curves and hysteretic loops were also discussed.

(2) Fundamental information on the effects of right-angled wall on the horizontal load-deflection relationships as well as the strength and stiffness were obtained through multipaneled-walls tests.

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Table 1 Properties of materials and test results for wall-type building specemens

Rinds of Tests		A	B	C
Size $t \times l \times h$ (mm)		120X2800(3700)X250	150X2630(378)X2560	120X3760X2565
Panel Rein	Flexu(kg/cm²)	D13 : 3600	16φ∼D25 : 3190∼3977	D10∼D13 : 3670 3880
	Shear(〃)	D10 : 3470	9φ∼13φ : 2925∼3416	6φ (3.2) : 4660(5290)
Panel Rein	Con (〃)	Fc = 280	Fc = 266	Fc = 108
Bottom Joint	Rein (〃)	D16 : 3800	2−16φ : 3191	D16 : 3720
	Con (〃)	Fc = 432	Fc = 340	Fc = 360
Opening. Rat.	$\lambda_x, \lambda_y, \lambda_z$	0.452, 0.083, 0.541	0.511, 0, 0.721	0.515, 0.372, 0.622
Initi-Stiff(t/cm)	Test. Cal.	267 241	1000 1265	45 44.1
Max. Loads(t)	Test. Cal.	129 133	651 662	32 36

Table 2 Properties of materials and test results for multi-planeled walls

Kinds of Tests		Plane		With rightangled		Space	
Specimen		W	B	W	B	W	B
		1	1	2	2	3	3
Maximum	plus	17.7	14.5	26.0	22.3	9.3 (1/50)	9.3 (1/50)
Load(t)	minus	16.7	14.0	23.5	19.6	4.3	7.3

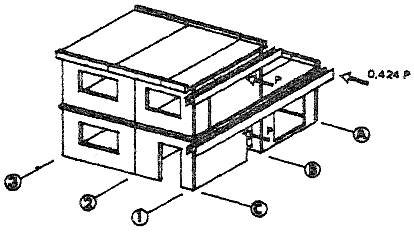

Fig. 1 Test set-up for A test (Ref. 1)

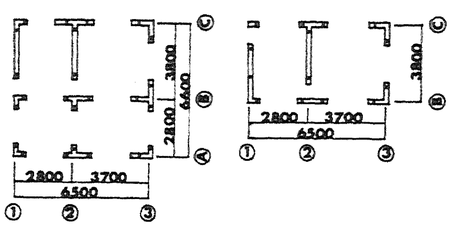

Fig. 2 Plan and bottom joints layout (A test)

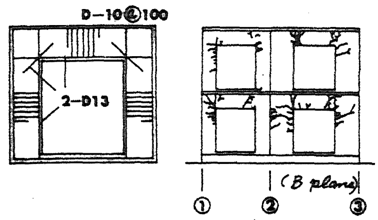

Fig. 3 Reinforcements and crack patterns for wall panels

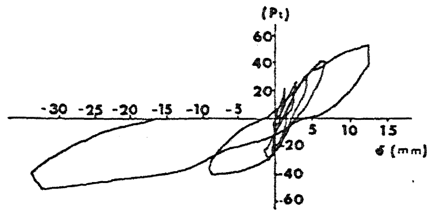

Fig. 4 Load-deflection curves (A test)

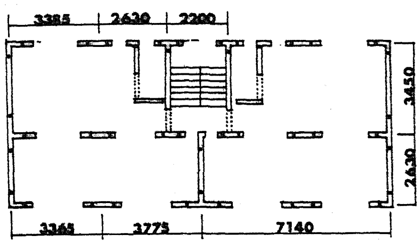

Fig. 5 Plan and bottom joints layout (B test Ref. 2)

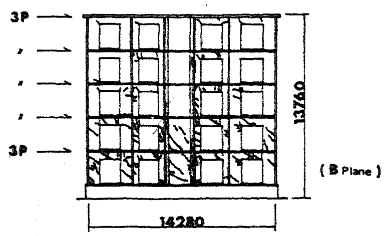

Fig. 6 Crack patterns (B test)

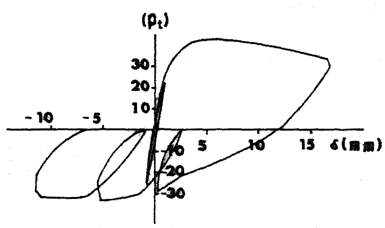

Fig. 7 Load-deflection curves (B test)

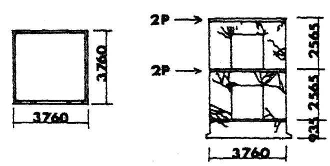

Fig. 8 Plan and bottom joints layout (C test Ref. 3)

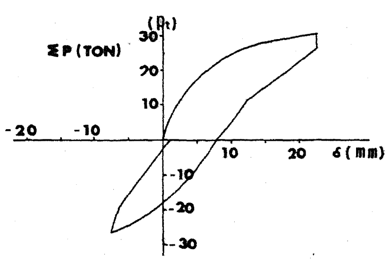

Fig. 9 Load-deflection curves (C test)

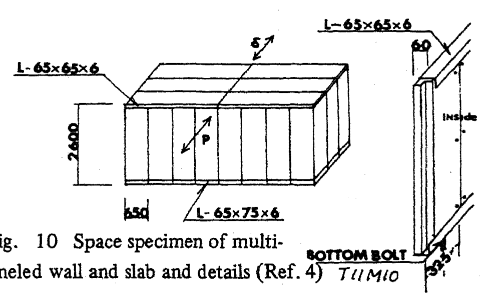

Fig. 10 Space specimen of multi-paneled wall and slab and details (Ref. 4) TI/MIO

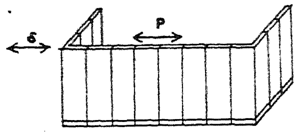

Fig. 11 Multi-paneled walls specimen

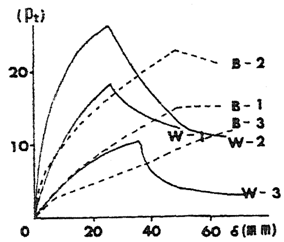

Fig. 12 Envelope curves

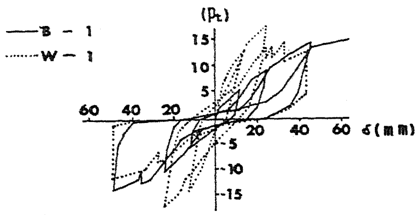

Fig. 13 Load-deflection curves

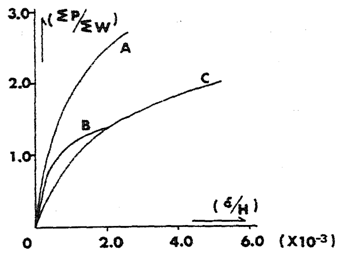

Fig. 14 Stardardized envelope curves

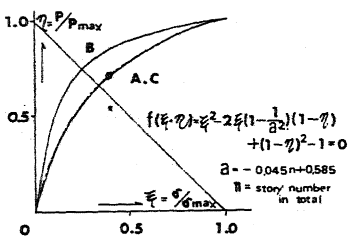

Fig. 15 Normarized envelope curves

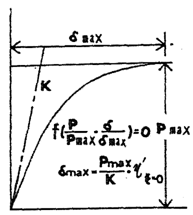

Fig. 16 Relationship between initial stiffness and curves

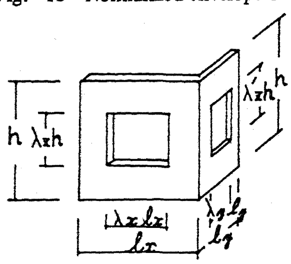

Fig. 17 Definition of opening ratio

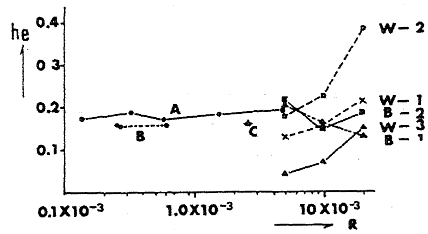

Fig. 18 Equivalent viscous damping ratio

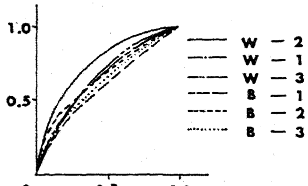

Fig. 19 Normarized envelope curves

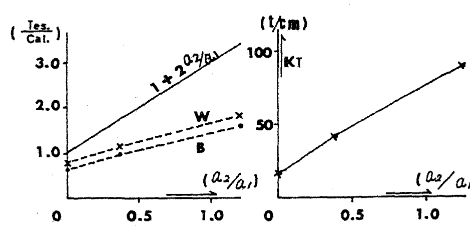

Fig. 20 Maximum strength and initial stiffness