SEISMIC PERFORMANCE EVALUATION FOR WALL-TYPE PRECAST CONCRETE HOUSINGS USING CRLS

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ABSTRACT :

The purpose of this study is to construct the seismic performance evaluation method for the wall-type concrete panel housings using the Calculation of Response and Limit Strength (CRLS), which is a new performance -based structural evaluation method for buildings introduced in 2000 in the Building Standard Law of Japan (BSL). The housing is one of low-rise prefabricated housings of the wall-type that precast reinforced concrete (RC) panel members with ribs, assembled by steel bolts. The BSL revised in 1998 stipulated to carry out the structural calculations using the CRLS for this type of RC buildings which use thin walls less than 120mm in thickness. In order to evaluate the seismic performance of the wall-type housings using the CRLS, analytical member models to be used for the pushover analysis of the housings were constructed based on the structural testing in this study. The results of pushover analysis using member modeling based on structural component tests were compared with the experimental results of a two-story full-scale wall-type housing tested in 1981 to verify the prediction accuracy. Moreover, the seismic performance evaluation using the CRLS for real wall-type concrete panel housing was executed.

KEYWORDS: Full-scale structure testing, Wall-type precast concrete housing, Pushover analysis, Seismic performance evaluation, Calculation of Response and Limit Strength

1. INTRODUCTION

A structural system using the medium-sized concrete panel with the rib is applied for low-rise housings of the wall-type precast reinforced concrete. These housings have been constructed by assembling the standardized concrete panels using connection bolts and mortar filling. The simple structural design method by a check of the member placement and the wall-length ratio has been employed and a lot of housings have been built up to now. However, the Building Standard Law of Japan (BSL) revised in 1998 stipulated to carry out the structural calculations using the Calculation of Response and Limit Strength (CRLS), which is a new performance-based structural evaluation method for buildings introduced in 2000, for these type housings which use thin walls less than 120mm in thickness.

In order to construct the seismic performance evaluation using the CRLS, a non-linear static analysis for this housing type was executed based on full-scale testing data of the real-size housing. The results of the non-linear analysis were compared with the test results to clarify the relations between analytical results and the structural behavior that are essential to the seismic performance evaluation using the CRLS, such as hysteresis, damping and deformation characteristics. In addition, a seismic performance evaluation using the CRLS was executed for a real housing. This paper presents the results of non-linear static analysis models for the full-scale housing experiment and the results of seismic performance evaluation for the real housing using the CRLS.

2. OUTLINE OF WALL-TYPE HOUSING AND FULL-SCALE EXPERIMENT

2.1 Outline of Wall-type Housing

The dimension of the structure and the detail of the member are shown in Fig.1. The dimensions and bar arrangements of the members are listed in Table 1. This structure consists of wall panels, sagging wall panels, floor panels and connection columns. The wall-panels had 2,700mm-high, 897mm-wide, 120mm-thick ribs and 46mm-thick shells. The floor-panels had 897mm-wide, 150mm-thick ribs, 46mm-thick shells and 3,870mm-length and were placed on the wall-panels. The main reinforcements of the wall-panel are 2-D13 (SD295A), the main reinforcements of the sagging wall-panel were 2-D13 (SD295A), and the shell section reinforcements were 2.9 ϕ @60 (SWM-P) mesh. In the floor panels, the main reinforcements of a longitudinal rib were 2-D16 (SD295A), the main reinforcements of the narrow side ribs were 4-D10 (SD295A), and the shell section reinforcements were 2.9 ϕ @100 (SWM-P) mesh.

The specifications of joint are shown in Fig.1 and Table 2. The foundation and the 1F wall panels, the 1F wall panels and the 2F wall panels, the 2F wall panels and the roof floor panels were jointed by bolt connection with mortar filling, and wall panels were jointed together by cross-tie bolts. The floor panels were jointed together by joint bolts and shear cotter with mortar filling. In addition, the strengths of concrete and mortar used were 30N/mm^2 and 60N/mm^2, respectively.

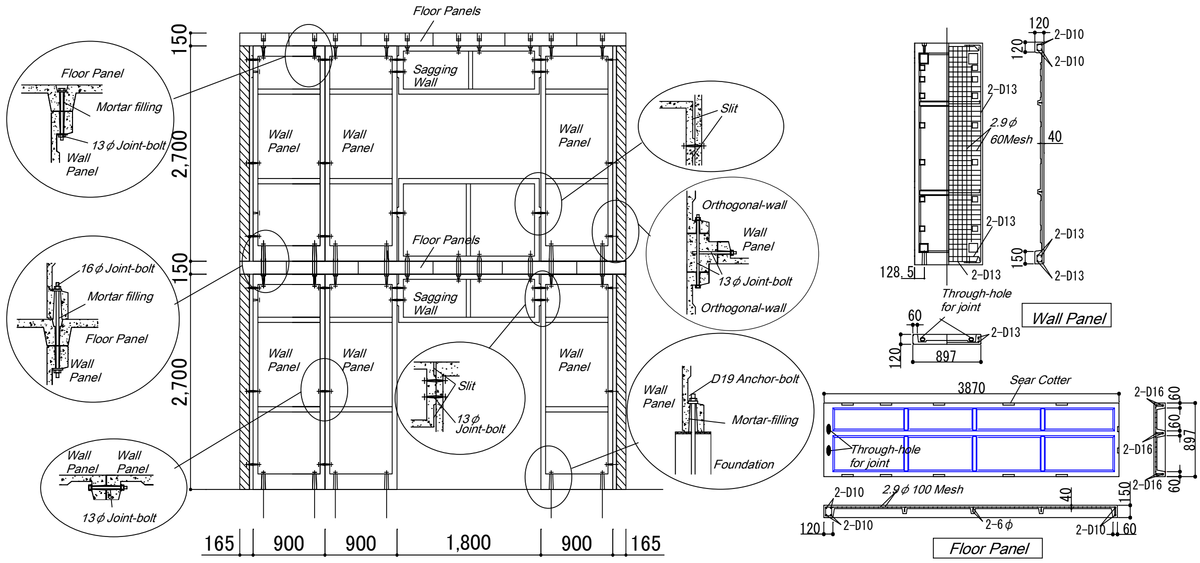


Fig.1 Dimension of the Structure

Table 1 Dimension and Bar Arrangement of Member

Part	Dimensions(mm)				Bar Arrangements		
	Length	Width	Thickness		Main Reinforcement		Mesh Line
			Rib	Shell	Longitudinal	Narrow Side	Shell Section
Wall	2,700	897	120	46	2-D13 (SD295A)	2-D13 (SD295A)	2.9 ϕ @60 (SWM-P)
Floor	3,870	897	150	46	2-D16 (SD295A)	4-D10 (SD295A)	2.9 ϕ @100 (SWM-P)
Sagging Wall	1,800	560	120	46	2-D13 (SD295A)	2-D10 (SD295A)	2.9 ϕ @60 (SWM-P)

Table 2 Specification of Joint

Position	Joint Bolt
Foundation－1F Wall	Anchor Bolt D19(SD295A)※
1F Wall－Floor－2F Wall	Joint Bolt 16 ϕ (SS400)※
Wall－Roof Floor	Joint Bolt 13 ϕ (SS400)※
Wall－Wall	Joint Bolt 13 ϕ (SS400)
Floor－Floor	Joint Bolt 13 ϕ (SS400)※

※ : Mortar filling using together

2.2 Full-scale Experiment

Specimens

A full-scale 2 stories real-sized housing consisting of the concrete panel with ribs was tested in 1981. The specimen consisted of 3 loading direction frames and 2 orthogonal direction frames. The loading frame consisted of three pieces of wall panels and sagging walls, as shown in Fig.1, and the main structural elements (orthogonal wall, continuous wall and gate of frame) were also include in this structure. The parameters of the structure were floor-area of 35.89m^2, weight of 421.6kN, wall-length ratio of X direction of 22.57cm/m^2 and load distribution was equal distribution.

The lateral force was given on the second floor (loading point at 2,575mm-height) and on roof floor (loading point at 5,225mm-height). The incremental loading cycles were controlled by story drift angle, R, where lateral loading sequence consisted of five cycles to each R of 0.005, 0.01 and 0.02rads. (R of 0.016 rad. for negative loading), followed by a half cycle to R of 0.036 rad. (R=1/28) of the positive loading.

Experimental Result

Shear force versus story drift angle relationships of the specimen are shown in Fig.2. The ● mark in this figure represents the first yielding point, where the tensile yielding of the anchor bolt occurred on the 1F wall. As seen in the figure, the specimen showed a stable slip-type hysteretic behavior on 1st and 2nd floors before reaching a large deformation. The failure progress of the specimen is described below. The initial cracks were observed at base shear of 240 kN on the 1F wall and the yielding of the anchor bolt on the 1F wall was observed at base shear of 584 kN during the loading cycle, R of 0.01 rad. The maximum strength of 784 kN was reached at R of 0.036 rad.(R=1/28), and the compression failure of the concrete and the rupture of many anchor bolts were observed in the 1F wall bottoms, where the destruction was reached. In addition, there was almost no significant strength degradation was observed during destruction. At R of 0.02 rad., shear cracks on the whole surface of the 1F wall panel were observed, however, the strength degradation under reverse loading was small. This experimental result implies that the safety limit deformation of this construction is about R of 0.02 rad., as proposed by Kawamoto et al. 2006.

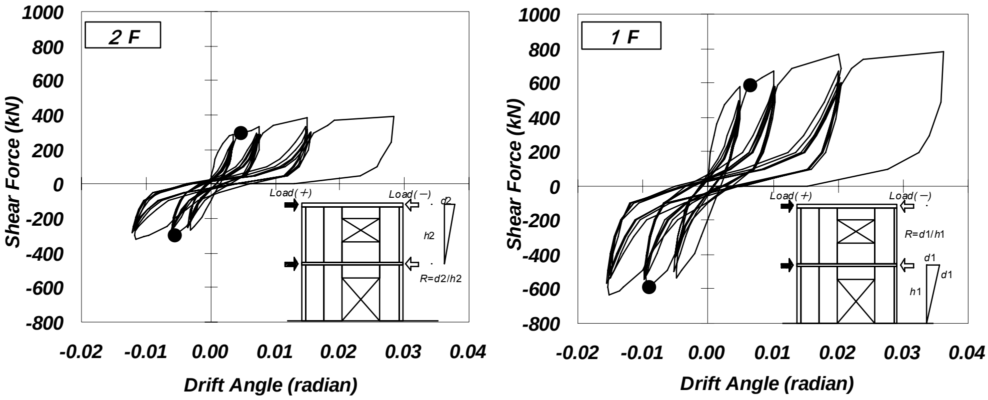


Fig.2 *Q-R* Curve (Full-Scale Experiment)

3. NON-LINEAR STATIC ANALYSIS MODEL

3.1 Summary of Analysis Model

Considering the results of a partial frame experiment of this housing structure tested in the past (Kawamoto et al. 2004, Kawamoto et al. 2005), the non-linear static analysis model of this structure was constructed. The three-dimensional model consisted of the plastic axial spring which replaced the joint and the line model with the plastic bending spring on the edge which replaced each member. The analysis models of the member and joints are shown in Fig.3, which are described below.

3.2 Analysis Models of Member

The wall-panels, the floor-panels and the sagging wall-panels were modeled with the line models having the property of the plastic bending spring on the materials edge at the central axis position of each member. The wall panels were the M-N interaction models considering the fluctuation axial force at wall deformation. The floor panels were modeled in consideration of the effect of different boundary condition. The hysteresis rules of wall panel, floor panel and sagging wall panel adopted the Takeda model. The axial spring and shear spring adopted the elasticity in all members. In addition, the shear span of the wall was evaluated with 1/2 of the wall height with asymmetry bending.

3.3 Horizontal Joint of Wall

An analysis model of the wall joint is shown in Fig.4. In the wall joint, it was resisted by a balance of tension of the joint bolt and the compression of the concrete. Considering this mechanism, the concrete was modeled as rigid compression axial spring by the central location of the longitudinal rib, and the joint bolt was modeled with plastic tension axial spring. For the tension axial spring, a tri-linear-type slip model shown in Fig.4 was adopted based on relations of the axial tension versus aperture displacement of the joint bolt in the single wall experiment. The first break bending point of spring model was an adhesion cracking point between mortar and

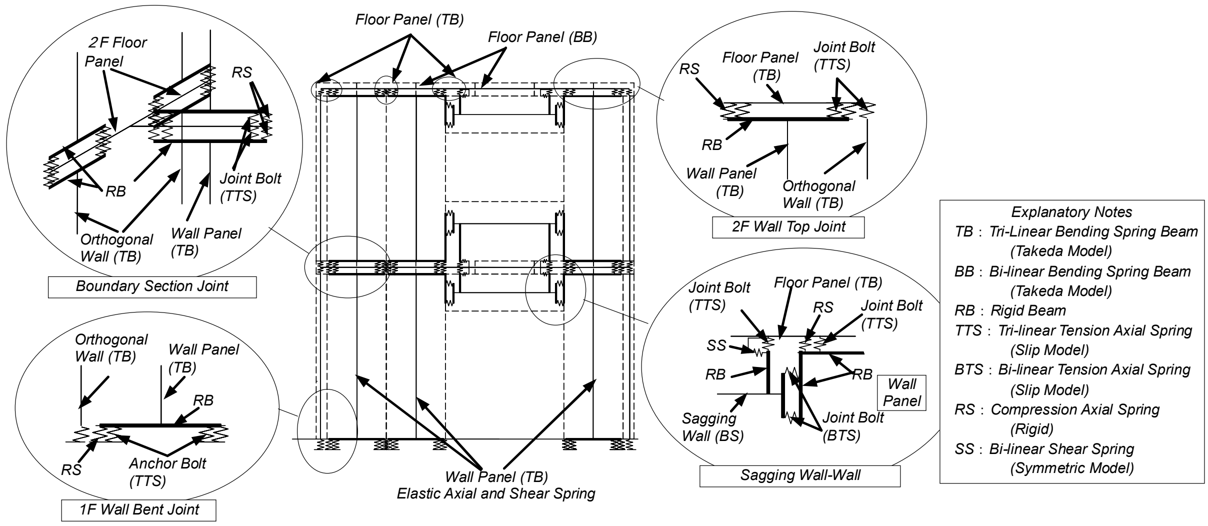


Fig.3 Analysis Model

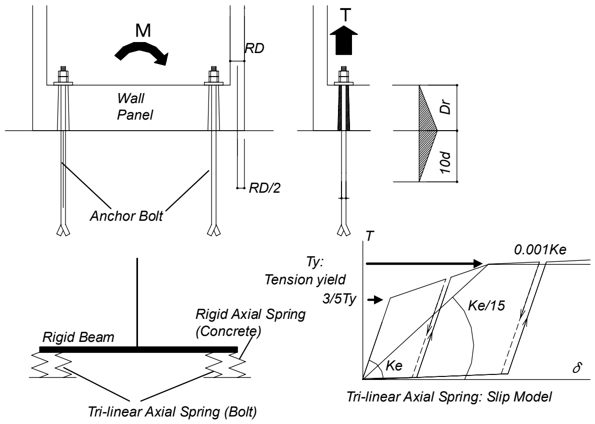


Fig.4 Joint Model of Wall (1F)

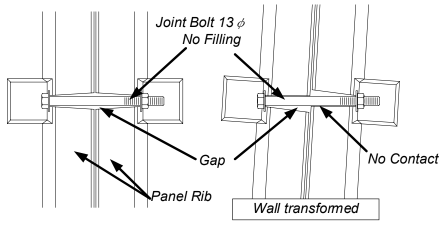


Fig.5 Vertical Joint (Wall－Wall)

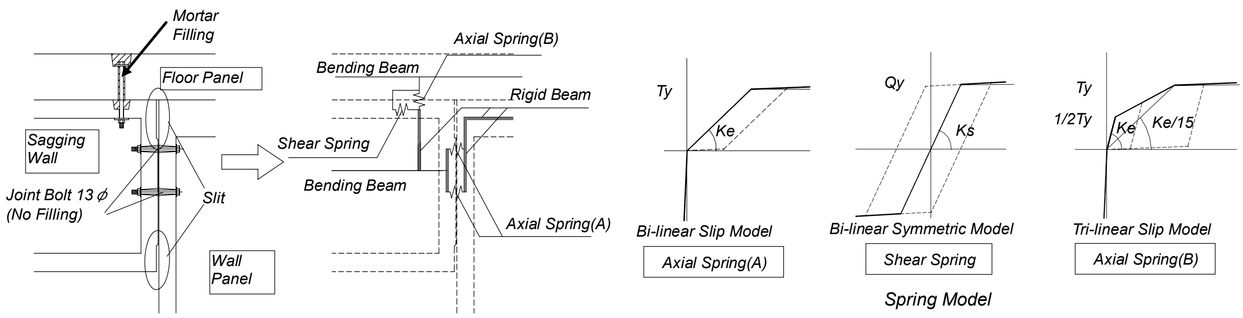


Fig.6 Spring Models of Joint (Sagging Wall)

joint bolt, and the second break point showed a tensile yielding point of the joint bolt. For adhesion cracking point, 1/2 of the yielding axial force was adopted in the bolt of the round bar, and 3/5 in the joint bolt of the deformed bar. The compression displacement in the experiment was small, around 2mm at the maximum because the stiffness of compression axial spring was rigid. This means that the compression displacement is negligible. In addition, the stiffness shear spring was rigid, so the horizontal slip displacement is also negligible.

3.4 Vertical Joint

It is shown in the detail that there was a gap of about 10mm between the vertical and orthogonal walls causing no contact between a bolt and a panel in the vertical joint (see Fig.5). The relative gap of the member confirmed that there was no transformation of shear force until large deformation (about 1/67～1/100rad.). Therefore, the vertical joint was not considered in this analysis model.

3.5 Joint of Sagging Wall

The sagging wall - wall joint was modeled as the horizontal asymmetry plastic axial spring which had compression and tension between a vertical rigid beam arranged on the sagging wall edge and the rigid beam of the wall top joint (see Fig.6). In the horizontal direction of the sagging wall - floor joint, a shear gap has been resisted by a bearing pressure with the joint bolt and infilling mortar. Therefore, bi-linear type plastic shear spring with the properties shown in Fig.6 was adopted. In the vertical direction of joint, it is modeled as tri-linear plastic axial spring of the asymmetry property, as shown in Fig.6.

4. RESULTS OF ANALYSIS MODEL

In order to apply the analysis model to the CRLS, the relationships of shear versus story drift angle, hysteretic behavior, equivalent viscous damping factor (*heq*), and deformation distribution were examined.

4.1 Analysis

Non-linear static analysis was executed under reverse loading by the displacement control corresponding to the applied shear force and incremental displacement in the previous full-scale experiment. The distribution of the load was adopted as equal distribution which is controlled by the displacement in the second floor. The analysis program used was non-linear static analysis program RESP-T, which was improved for analyzing this structure.

4.2 Shear versus Story Drift Angle Relationships

Comparison between experimental and analysis results for shear-force versus story drift angle relationships are shown in Fig.7. The ○ and ● marks in Fig.7 showed the yielding point of anchor bolt on 1F wall bent joint from experiment and analysis results, respectively. As shown in this figure, the hysteretic curve of analytical results agreed well with the experimental results for all 1[st] and 2[nd] stories. Up to R=0.01 rad., the calculated hysteresis loops indicated the excellent agreement in both positive and negative loading. In addition, the predicted yielding point of the anchor bolt in 1F wall showed a good agreement with the experimental data.

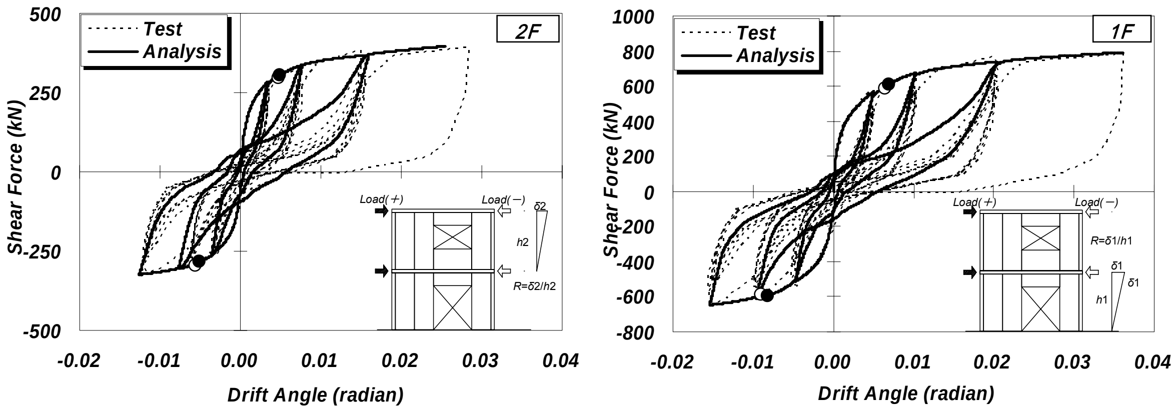


Fig. 7 *Q-R* Curve (Analysis Result)

4.3 Hysteretic Behavior

A slip-type hysteretic behavior from analysis model adequately simulated the behavior of the test specimen. A little difference of the hysteretic shape occurred at the large deformation. This indicates that the analysis model can evaluate well the overall hysteretic behavior of the proposed housing type.

The comparative results of the equivalent viscous damping factor (*heq*) on 1F at R of 0.005, 0.01 and 0.02 rads. are shown in Fig.8. The damping factors are calculated from the hysteresis loops of the second cycle in each story drift angle. The solid lines in Fig.8 showed *heq* value calculated using Eq.(1) with the ductility ratio started from the displacement on the first yielding (the yield of the anchor bolt of the 1F wall bent) in the analysis result.

$$heq = \alpha \cdot (1 - 1/\sqrt{\mu}) \quad (1)$$

where μ=ductility ratio calculated from first yielding, α=a factor to consider the effect of the energy absorbing capacity of buildings, which is taken as 0.20 for this structure.

For *heq*, analysis results showed a good agreement with the test results at R of 0.02 rad., which have almost similar value of around 0.09. However, *heq* calculation results using Eq.(1) was smaller than the test results in the range of the small deformation. This means that at the safety limit deformation, R of 0.02rad., *heq* of the structure can be evaluated by analysis models proposed in this study

4.4 Deformation distribution

In the CRLS method, it is necessary to calculate the deformation distribution of each story by non-linear static analysis. The comparisons of deformation distribution at R of 0.005, 0.01, 0.02 and 0.036 rads. (R of 0.005, 0.01, and 0.016 rads. on negative loading) are shown in Fig.9. It is clearly seen in the figure that the analysis result agreed well with the test results for deformation distribution in a range of safety limit deformation (R of 0.02rad.).

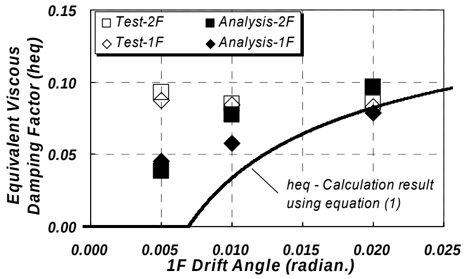


Fig.8 *heq*

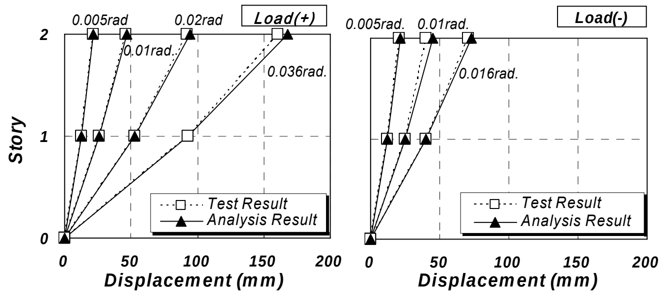


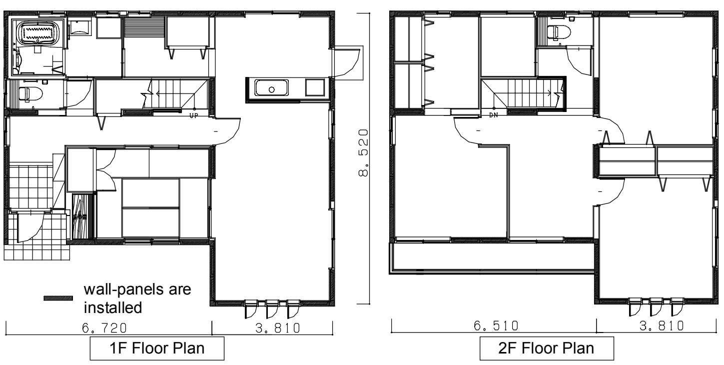
Fig.9 Deformation Distribution

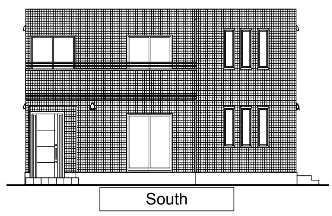
5. SEISMIC PERFORMANCE EVALUATION USING CRLS

A non-linear static analysis was also carried out on real housing, which followed the method proposed by Kawamoto et al. 2006 and a seismic performance evaluation using the CRLS was executed. The outlines of the target housing, analysis results, limit states and evaluation results using the CRLS are presented below.

5.1 Target Housing

The ground plan of the target housing, the south elevation and specifications data are shown in Fig.10. Solid lines in the ground plan are showed the point where the wall-panels were installed. The target housing was 2-story building which was constructed by the proposed construction method. The total floor area was 155.22m^2, the structure heights were 2.775m (1F) and 2.85m (2F) and the building weights were 492.4kN(1F) and 375.4kN (2F).





Specification Data

	1st floor	2nd floor
Floor Area	77.61m^2	77.61m^2
Height	2.775m	2.850m
Wall-Length Ratio	16.2cm/m^2	14.2cm/m^2
Weight	492.4kN	375.4kN
Ground	1st kind ground	

Fig.10 Target Housing of Seismic Performance Evaluation

5.2 Analysis and Limit State

Non-linear static analysis was carried out to evaluate the target housing. It was assumed that floor panels were infinitely rigid within the floor plane, and the analysis load acted on the center of gravity position of the each floor by Ai distribution. The foundations were assumed as fixed support, and torsion deformation that occurred in the analysis was not restrained. Fig.11 showed shear force versus story displacement relationships on the center of gravity position in the X load direction. The □ and ■ marks in Fig.11 showed the critical point of damage limit and safety limit. In addition, the ○ and ● marks indicated the response point in damage limit and safety limit. The critical point of a damage limit was assumed at the point where the stress of the main reinforcement of the 2F floor panel reached the limit stress when shear force of the 1F reached 447 kN (1F at R=1/574rad., 2F at R=1/459rad.). The critical point of safety limit was assumed at the point where the yielding of the wall anchor bolt occurred when shear force of 1F reached 625 kN because the drift angle of the 2F wall exceeded R of 0.02rad. when shear force of the 1F reached 770 kN. In addition, it was confirmed that the shear force of the wall didn't exceed the shear capacity force at this point.

5.3 Evaluation Result

Seismic performance was evaluated using the CRLS in order to obtain the shear versus displacement relationships of the equivalent single degree of freedom system using an analysis result. The demand spectrum for the seismic evaluation adopted a value in the BSL. In addition, considering the low-rise housing, which is constructed only by simple soil exploration, the amplification factor of the acceleration by the surface subsoil (*Gs*) adopted a value based on the classification of ground by vibration condition. Spectral acceleration versus Spectral displacement relationships (*Sa* - *Sd* curve) in the safety limit is shown in Fig.12. The damping factor of the building was evaluated by Eq.(2) using ductility ratio. The Eq.(2) considered a linear viscous damping constant of 0.05 in calculation formula of equivalent viscous damping factor (*heq*) using ductility ratio in Eq.(1). The ductility ratio was calculated using the equivalent bi-linear curve that replaced a capacity spectrum.

$$h = 0.20 \cdot (1 - 1/\sqrt{\mu}) + 0.05 \tag{2}$$

where μ is ductility ratio.

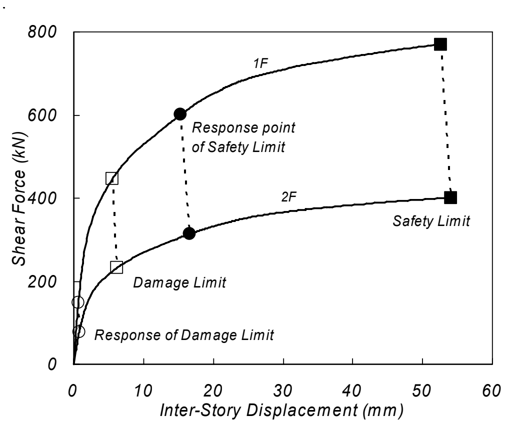


Fig.11 Q-δ Curve

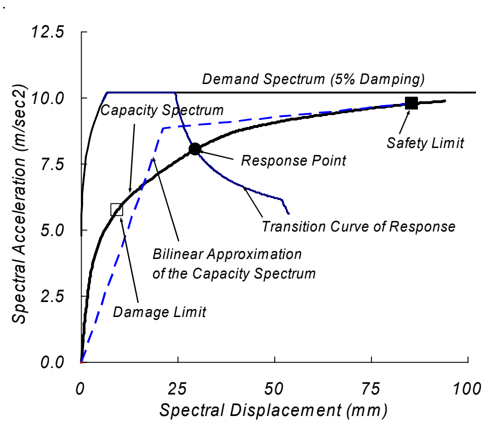


Fig.12 $Sa - Sd$ Curve (Safety Limit)

Table.3 Results of the Seismic Performance Evaluation

		Damage Limit	Safety Limit
Limit state		Yield of 2F Floor Rib Main Reinforcement	Drift Angle of 2F Wall is over 0.02rad.
Limit Natural Period (sec)		0.253	0.587
Limit Displacement (mm)		9.4	85.5
Limit Acceleration (m/sec^2)		5.76	9.78
Base Shear (kN)		446.9	769.7
Equivalent Mass (ton)		77.6 (0.877)※	78.7 (0.890)※
Response Natural Period (sec)		0.156	0.380
Response Displacement (mm)		1.2	29.4
Response Acceleration (m/sec^2)		2.02	8.05
Reduction factor (*Fh*)		—	0.79 (μ=1.56)
Response Story Drift Angle (radian)	2F	1/3462	1/149
	1F	1/4378	1/179

※ : ( ) inside Equivalent Mass Ratio is shown. μ: ductility ratio

The results of the seismic performance evaluation in damage limit and safety limit which were executed based on above-mentioned procedure are shown in Table 3. The following information were confirmed from the evaluation results.

- In both the damage limit and the safety limit, the deformation of response point was smaller compared with the limit deformation, and the target housing was a building expansive on deformability.
- In the damage limit, response displacement of the 1F was 1.2mm, which was around 1/8 of damage limit displacement (9.4mm).
- Damping reduction factor (*Fh*) of the response point in the safety limit was 0.79. It is assumed that the member damage of the response point was small because the plasticity degree was small.
- There was a performance point on the safety limit in the acceleration given area of the demand spectrum, which predicted that the building has a strength resistance type.

From the above results, it can be concluded that the seismic performance in the damage limit and the safety limit for the real-size housing can be obtained by executing a seismic performance evaluation using the CRLS proposed in this study.

CONCLUSIONS

In this study, the non-linear static analysis was carried out on the wall-type housing consisting of the concrete panel with ribs. The analytical results were compared with the experiment ones. The following conclusions can be drawn from the comparative results:

1) Shear versus story drift angle relationships from analysis results shows a good agreement with the experimental results until story drift angle, R of 0.02rad.
2) The slip-type hysteretic behavior as the characteristic of this construction method/proposed housing can be obtained from the analysis model.
3) Equivalent viscous damping factor (*heq*) can be calculated by Eq.(2) using ductility ratio.
4) The deformation distribution of 1F and 2F from analysis model agreed well with the test results until R of 0.02rad.

In addition, the method of the CRLS was applied to a real housing in this study and a seismic performance in the damage limit and the safety limit was obtained. From these results, it is concluded that a seismic performance evaluation using CRLS on the wall-type housing of precast concrete panels with ribs is practicable by using the non-linear static analysis model proposed in this study.

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