

DESIGN FOR STRUCTURAL PERFORMANCES OF REINFORCED CONCRETE FRAME WITH CAST-IN-PLACE NON-STRUCTURAL REINFORCED CONCRETE WALLS

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SUMMARY

In the Japanese conventional cast-in-place reinforced concrete (RC) buildings, RC frames (columns and beams) and cast-in-place non-structural RC walls (mullions, spandrels and sidewalls) were constructed simultaneously, so they were connected rigidly. In the past earthquake, in cases where the shear reinforcement of column is enough to resist the shear force, it is reported that the damage of columns and beams was slight due to the failure of cast-in-place non-structural RC walls. So, the purpose of this research is controlling the failure mode of cast-in-place non-structural RC walls and using the cast-in-place non-structural RC walls to a seismic design positively, such as a rise of stiffness, capacity and energy dissipation. Then this paper proposed a method to estimate the structural performance of the RC frames with cast-in-place non-structural RC walls.

As described above, The shear-loading tests for one-story RC frames with cast-in-place non-structural walls were conducted. Eight specimens, which were modeled with a scale of 1/3, were prepared. The parameters was set in the test were the shear reinforcement of column and the shapes of non-structural walls. From the test, it was proved that the non-structural RC walls could advance the structural performance of RC frames, and the stress of column and non-structural walls were clarified.

The finite element analysis (2-D non-linear analysis) was conducted, in order to compensate tests. The analytical results agreed with test results, and the parametric analytical results were more useful to clear the internal stress. Then, the shear resistant mechanism of RC frames and non-structural RC walls was clarified with test results and analytical results. Still more, this paper propose the method to predict the structural performance (the load-displacement relationship) of RC frames with non-structural RC walls by the shear resistant mechanisms.

INTRODUCTION

In the Japanese conventional cast-in-place reinforced concrete (RC) buildings, RC frames (columns and beams) and cast-in-place non-structural RC walls (mullions, spandrels and sidewalls) are constructed

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simultaneously, and they are connected rigidly. So, the stress transfer exists between the RC frame and the cast-in-place non-structural walls (mullions, spandrels and sidewalls), and the cast-in-place non-structural walls influences the seismic behavior of the RC structures. In the past earthquake, the shear failure of the column with non-structural walls (ex. shear failure of short column; column with spandrels) was reported as one example of the influence. However, in cases where the shear reinforcement of the column is enough to resist the shear force, it is reported that the damage of columns and beams was slight by cast-inplace non-structural RC walls failed, for example [AIJ, 1 and 2]. On the other side, there are no evaluation methods for designing structural performance (stiffness, capacity, failure mode and ductility) of the RC frame with cast-in-place non-structural RC walls, because the shear resistant mechanism of RC frame (columns and beams) with cast-in-place non-structural RC wall was not clarified. Though it has been recommended as the conventional method to use isolation detail (called slit) between cast-in-place nonstructural RC walls and RC columns, serious damages were observed in the buildings where the detail of the isolations were not suitable detail, for example [AIJ, 2]. So, the purpose of this research is controlling the failure mode of cast-in-place non-structural RC walls and using the cast-in-place non-structural RC walls to a seismic design positively, such as a rise of stiffness, capacity and energy dissipation. Then this paper proposed a method to estimate the structural performance of the RC frames with cast-in-place nonstructural RC walls.

TEST OF SERIES-1[2]

The purpose of this test is to clarify the performance, the structural performance of the RC frame with cast-in-place non-structural RC walls and the damage performance of the cast-in-place non-structural RC walls. This test is reported [Sugiyama et al., 3], and here, especially describes the structural performance of the RC frame with cast-in-place RC walls.

Outline of the specimen

Details of specimens in series-1 are shown in Fig.1. The specimen was a one-story one-span RC frame. The specimen No.0-0 was the open RC frame, and the specimen No.1-1 was the RC frame with cast-in-place shear wall. The specimen No.1-2 and No.1-3 were the RC frame with cast-in-place non-structural RC walls, which was supposed a frame in the apartment houses. The specimen No.1-2 was spandrel type



specimen, and the specimen No1-3 was sidewall type specimen. The column and beam of the frame were unified in all specimens. The cross section of column ($B_C \ge D_C$) was 240mm ≥ 240 mm, and the cross section of beam, to be made quite stiff, ($B_B \ge D_B$) was 800mm ≥ 600 mm. The thickness of shear wall was 60mm, and the thickness of non-structural walls was 40 mm. The reinforcement bar of RC walls were anchored a column and beams, and the length of the anchor was 210mm. The length of RC wall was 1760mm, and the height was 1100mm. Concrete strength was from 24.1 to 33.0 N/mm².

Result of the test

Crack patterns are shown in Fig.2. In the specimen No.1-2 (spandrel type), shear cracks in mullion and spandrel were observed around 1/800rad. in story drift angle. Finally, the decrease of shear force was caused by the compressive failure of column-spandrel connection concrete. In the specimen No.1-3 (sidewall type), shear cracks and flexural compressive failure of sidewall concrete were observed. Next, shear cracks developed along the vertical reinforcing bar in the sidewall. Finally the column and the sidewall slid off vertically. Failure mode of the specimen No.0-0 (frame), No.1-2 and No1-3 were the flexural yield of column, and that of the specimen No.1-1 (shear wall) was the shear failure of shear wall.

Shear force (Q) – story displacement (δ) relationship skeleton curves are shown in Fig.3. The initial stiffness of non-structural wall specimens (No.1-2 and No.1-3) was higher than that of the frame specimen, and were similar to that of the shear wall specimen (No1-1). Reduction in the stiffness was observed around 1/800rad. in story drift angle due to the failure of non-structural RC walls (mullion, spandrel and sidewall). After maximum load, the shear force decreased slowly, but it was rather higher than the flexural strength of frame specimen. Furthermore, the energy dissipation of non-structural specimen (No1-2 and No1-3) was higher than that of the frame specimen, and it means that the hysteresys of non-structural wall specimens were ductile.

Those result suggested that the structural performance (stiffness, capacity and energy dissipation) of RC frame with cast-in-place non-structural RC walls (spandrels and sidewalls) is advanced from that of the open RC frame, if the column has the shear reinforcement which can resist the shear force.

TEST OF SERIES-2

The purpose of this test is to suggest the influence of principal factor, the shear reinforcement of column and the shape of cast-in-place non-structural RC wall, to the structural performance of RC frame .

Outline of the test

The specimen in series-2 test was designed by referring to series-1 test. The variable in this test, the shear reinforcement of column and the shape of cast-in-place non-structural RC wall, was set including the specimen in series-1.

Specimen

Fig.4 shows the detail of specimen in series-2. The shape of RC frame, the column and the beam, was due to series-1. The specimen with a scale of 1/3 was a frame with cast-in-place non-structural wall. The columns, the beams and non-structural walls were cast simultaneously. The reinforcement bar of RC walls were anchored a column and beams, and the length of the anchor was 210mm. The height of column was 1100mm, and the clear span of beam, the length of RC walls, was 1760mm. The specimen No.2-1 and No.2-2 were the one-story one-span RC frame, and the specimen No.3-1 and No.3-2 were the one-story two-span RC frame (added the center column). The variables were the shear reinforcement of the column, the thickness of cast-in-place non-structural walls (spandrel) and the side of column connected spandrels (column of internal/exterior). In the sidewall type specimen (No.1-3, No.3-2), the length of sidewall was

added to the variables. The shear reinforcement ratio (p_w) of column of the specimen No.2-1 were 0.38%, that of the specimen No.2-2 was 0.19%, and that of the specimen No.3-1 and No.3-2 was 0.89%. The thickness (t) of non-structural wall (spandrels and sidewall) was 40mm, 60mm and 80mm. The length of sidewall was 240mm (1.0 D_c , D_c : depth of column), 360mm (1.5 D_c) and 470mm (nearly 2.0 D_c). As shown in the fig.4, The spandrels were planned to observe the influence to the behavior of the column of the thickness of spandrels on each direction of loading (plus cycle and minus cycle). The sidewalls were planned to observe the influence to each column of the thickness and the length of sidewall.



Method of the test

Fig.5 shows the loading apparatus. The axial load, kept constant during the test (N=2or3 x B_C x D_C x f_C '/6), was applied to specimen by two jacks. The lateral (shear) load was applied to the specimen by horizontal jacks on loading beam. In the plus cycle, the specimen was loaded from left to right, and in minus cycle, the specimen was loaded from right to left. The shear cyclic loading was controlled by story drift angle, and the maximum story drift angles (*R*) in cycles were 1/6400, 1/3200, 1/1600, 1/800, 1/400, 1/200, 1/100, 1/50rad. and 1/25rad. The out-of-plane deflections of specimens were confined by pantographs above the loading beam.

Result and discuss

Failure pattern

Crack pattern in series-2 is shown Fig6.

Spandrel type (No.2-1, No.2-2 and No.3-1): In very small load (shear force Q was from 10 to 40 kN), the crack occurred between the column and the cast-in-place spandrel which transferred the tensile stress. Next, flexural cracks of the columns, shear cracks of the columns and shear cracks of spandrels was observed in order. The compressive failure of spandrels, at connection of spandrel and column which transferred the compressive stress, occurred around 1/200rad. in story drift angle. The failure mode of the specimen No2-1 was the shear failure of column, and that of the specimen No.2-2 is the shear failure of column. In all column of the specimen No.3-1, the flexural yield of the column was observed.

Sidewall type (No.3-2): In small load, the flexural crack of cast-in-place sidewalls was occurred the edge with the beams. The flexural crack of columns was observed around 1/1600rad. in story drift angle, and finally the column and the side-wall slid off (shear slip in connection of column and sidewalls) vertically. The failure mode was the flexural yield of the column.



Fig.6: Crack Pattern

Fig.7: Shear force(Q)-Displacement(δ) relationship



Deformation properties

Fig.7 shows the shear force (Q) – displacement (δ) relationship. Reduction in the stiffness was observed due to the flexural cracks of column and non-structural walls. In addition, reduction in the stiffness by slip of column and sidewalls was radical, and the shear force in case slip of column and sidewall became large as the thickness of sidewall became thick. After maximum load was observed, the shear force decreased due to the shear failure of column, the compressive failure of spandrel and the compressive failure of sidewalls. The decrease of shear force which was due to the compressive failure of cast-in-place nonstructural walls (spandrels and sidewall) was Loosely. In the specimen No.3-1 and No.3-2, the shear force was rather higher than the flexural strength of the frame specimen, 2/3 times that of the specimen No.0-0. in story drift angle *R*=1/25 rad..

Fig.8 shows the skeleton curve of the shear force (Q) – displacement (δ) relationship, and the parameter in Fig.8 is the shear reinforcement of column (the specimen No.1-2, No.2-1 and No.2-2). Even if it is in case of the RC frame with cast-in-place non-structural walls, the failure mode changed from shear failure of column to the flexural yield of column by increasing the shear reinforcement of the column. Moreover, by increasing the shear reinforcement, the ductility was improved.

Fig.9 shows the skeleton curve of the shear force (Q) – displacement (δ) relationship, and the parameter in Fig.9 is the thickness of spandrels (the specimen No.1-2 and No.2-1). The plus cycle and the minus cycle are shown in fig.9 to compare the thickness of the spandrels. The ductility of column with spandrel was improved as the spandrel became thin. The compressive failure of spandrel was observed, when the spandrel is thin, and the column with spandrel was changed from the short column to the long column by the compressive failure of spandrel. Therefore, it is important to develop the compressive failure of spandrel in order to obtain the ductility of column.

ANALYSIS (FINITE ELEMENT METHOD)

The Finite Element Method (FEM) analysis, 2D non-liner, was conducted to clarify the shear resistant mechanism of the RC frame with cast-in-place non-structural RC wall, and to cover the test result.

Agreement between analysis and test

Agreement between the analytical and the test was discussed.

Outline of analytical method

The mesh layouts of non-structural RC wall specimen are shown in Fig.10. The longitudinal bar in column was represented by line element, and shear reinforcing bars were arranged in two orthogonal directions and placed two layers in each direction. The bond between longitudinal bar and concrete was represented by bond link. In connection of members (columns, beams, and non-structural walls), the crack link was set for crack opening, and the dowel action of reinforcing bar was considered by dowel link. In this representation, the analysis result of the independent frame specimen (No.0-0) and the shear wall specimen showed good agreement with the test result [Sugiyama et al, 3].

Analytical Result

Fig.11 show shear force (*Q*)-displacement (δ) relationships skeleton curve of analysis. Good agreement between the analysis and the test can be seen in crack propagation and progress of failure. Moreover, skeleton curve of *Q*- δ relationships by analysis agree with that of test result, and analytical result indicated that the decrease of stiffness was caused to failures of non-structural wall in the RC frame. In spandrel specimen, the capacity of analysis due to the flexural failure of column, the yield of longitudinal bar, was the same as the test result. However, in sidewall specimen, after slipping between the column and the sidewalls in the test, the *Q*- δ skeleton curve of analysis did not agree with that of test result. But, the analysis result shows the behavior of the RC frame with sidewall until slipping between the column and the sidewalls. Thus, analytical method is useful to clarify the shear resistant mechanism of a RC frame with cast-in-place non-structural RC walls.

Parametric analysis

Here, the parametric analysis was described.

Outline of parametric analysis

The parameters set in the analysis were thickness (*t*) of non-structural walls (spandrel and sidewall) or length (l_w) of sidewalls. The mesh layout was same as shown in Fig.9. The thickness of non-structural in this analysis was varied from 0mm (Frame specimen) to 240mm (t/B_c =1.0), and the length of sidewall in this analysis was varied from 0mm (Frame specimen) to 720mm (l_w/D_c =3.0).



Fig.10: Detail of mesh layout in FEM analysis (2D)

Analytical Result

Fig.11 shows shear force (Q)-displacement (δ) relationships skeleton curve of analysis.

Spandrel type specimen: In all specimen of analysis, the failure pattern was same, and the capacity was due to the flexural failure of column (the yield of the column). The initial stiffness of analysis became large as the spandrels became thick. The capacity also became large as the spandrel became thick, but the rise of capacity was leveling off if the thickness of spandrels was larger than 40mm.

Sidewall type specimen: In all specimen of analysis, the failure pattern was same, and the compressive failure of sidewalls was observed. The initial stiffness and the maximum shear force became large as the sidewalls became thick. In the thickness of sidewall were 160mm and 240mm, the maximum shear force was due to the flexural failure of column. In case of others, the maximum shear force was due to the compressive failure of sidewalls. Therefore, in cases where the thickness of a sidewall was 160mm and 240mm, it is considered that the flexural failure was observed as the column and the sidewall was one member.

State of stress

Fig.12 shows the principal stress of the concrete element in analysis. In the spandrel type specimen, the





Fig.12: Principal stress of concrete in analysis

compressive strut of concrete was observed in columns and spandrels, and the angle of the strut was changed by columns and strut. Thus, in order to evaluate the structural performance of the RC frame with the spandrel, the compressive stress of the spandrel should be considered. In the sidewall type specimen, the compressive strut of concrete existed in the diagonal line. Thus, in order to evaluate the structural performance of the RC frame with the sidewall, the stress transfer between column and sidewalls should be considered.

SHEAR RESISTANT OF COLUMN AND CAST-IN-PLACE NON-STRUCTURAL WALL

The shear resistant, the state of stress in shear force, of the RC frame (column) and the cast-in-place nonstructural RC walls was discussed from the test result and the analytical result.

RC Frame with spandrel type non-structural walls

Fig.13 shows an example of the moment distribution of the column in test and analysis. The moment of each section was calculated using the strain of longitudinal bar and the mechanical properties of steel and concrete. Although the moment of column was the linear distribution, it curves at the edge of the spandrel. Besides, the curvature becomes large as the spandrel becomes thick. Since the compression of spandrel was loaded to a pillar at the opposite direction of shear force, these states were observed.

Fig.14 shows the distribution of compressive strain of the spandrel in the test and analysis, and the strain in test was measured on the surface of concrete. The strain of spandrel was the linear distribution, and the length of distribution was $1.0D_C$ (depth of column). The length of distribution was constant without reference to the thickness of the spandrels. But, if the spandrel was compressive failure, the length became long.

As described above, the place of combined compressive stress (C: compression of spandrel) is constant. Besides, since C (compression of spandrel) is small when the spandrel is thin, the shear span of column becomes long.

RC Frame with sidewall type non-structural walls

Fig.15 shows an example of the moment distribution of the column in test and analysis. In the exterior column (the column was connected to the sidewall at one side), the moment of column was a curved linedistribution so that the spandrel became thick. The same state was observed when the spandrel became long. On the other hand, in the interior column, the moment distribution of column was a linear distribution. Therefore, it is guessed that the compression of sidewall is uniformly loaded to the column.

Fig.16 shows an example of the shear stress between the column and the sidewall in analysis. The distribution of shear stress was uniformly even if the sidewall thickness or length was changed.

Fig.17 shows an example of the compressive strain of the sidewall in test. The strain was developed between the sidewall and the beam. Before the shear slip occurs in connection of the column and the



Fig.15: Column moment distribution (in sidewall type) Fig.17: Compressive stress of sidewall (in test)

sidewall, the strain was linearly distributed including the column section. Therefore, the behavior of the column and the sidewall was one member. Then, after shear slipping in connection of the column and the sidewall, the compressive strain of sidewall is distributed from the edge only by the length of $1.0D_C$ (depth of column). The strain was the linear distribution.

MODEL OF SHEAR RESISTANT MECHANISM AND ESTIMATE THE STRUCTURAL PERFORMANCE OF RC FRAME WITH CAST-IN-PLACE NON-STRUCTURAL WALLS

In this chapter, the method to estimate the structural performance of RC frame with cast-in-place nonstructural RC walls is proposed by the shear resistant mechanisms. Here, the shear force (Q) – displacement (δ) skeleton curve of RC frame with non-structural RC walls is evaluated. The Q- δ skeleton curve of the RC frame with non-structural walls was evaluated as a value which added all the Q- δ skeleton curve of the column with nonstructural walls.

Model

Fig.18 shows the model of the Q- δ skeleton curve of the column with nonstructural walls. The Q- δ skeleton curve of the column with nonstructural walls is evaluated by bi-linear in the range until the flexural failure of column. However, in cases where the shear slip occurs in the connection of column and sidewall, the Q- δ skeleton curve of the column with sidewalls is evaluated by tri-linear in the range until the flexural failure of column. As shown in a fig.18 evaluation matters are K_1 (Initial stiffness), K_y (Stiffness at flexural yield), Q_{mcr} (Flexural crack strength), Q_{my} (Flexural capacity), Q_{su} (Shear strength), Q_{slip} (Slip strength) and δ_u (Ultimate displacement). Moreover, if the nonstructural walls fails (compressive failure of spandrel or/and shear slipping of column and spandrels), it is evaluated that the behavior of column with the non-structural walls shifted to the behavior of independent column.

Fig.19 shows the model of the shear resistant mechanism. The model is as follows.

Spandrel type: The compressive stress of triangle distribution is developed in the spandrels. The combined compressive stress exists in the position of $D_c/3$ from the end of the spandrels.

Sidewall type: The stress in the connection of column and sidewall is distributed uniformly. Before the shear slip occurs in the connection of column and sidewall, the behavior of the column and the sidewall was one member. Then, after shear slipping in the connection of column and sidewall, the compressive stress of sidewall is distributed from the edge only by the length of $1.0D_C$ (depth of column). Moreover, the distribution is liner.

Evaluation of the shear-displacement relationship of RC columns with the non-structural walls

Table 3 shows the equations to estimate the shear-displacement skeleton curve of RC columns with nonstructural walls. The equations were evaluated from the shear resistant mechanism.



Fig.18: Model to estimate the structural performance (Q- δ) of RC frame with non-structural walls

Initial stiffness (K_1)

The initial stiffness is evaluated in consideration of the flexural deformation (δ_m) and the shear deformation (δ_s) (Eq.(1), Eq.(2) and Eq.(3)). The initial stiffness of a column with the spandrels is given by Eq.(4-1) and eq.(5-1) from fig.18(d). The β (β_k and β_t) is the ratio of the compression of spandrel (C) to the shear force of column. The β (β_k and β_t) is evaluated in consideration of the stiffness of column and that of spandrel, and is calculated by Eq.(6) and Eq.(7). The initial stiffness of a column with the sidewall is calculated by Eq.(4-2) and Eq.(5-2), the section modulus is calculated by the equation in consideration of cross-sectional shape (Z_W and A_W) [Okubo, 4].

Stiffness at flexural yield (K₂)

The stiffness at flexural yield is calculated by Eq.(8), Eq.(9) and Eq.(10) using the standard of AIJ[5]

Flexural crack strength (Q_{mcr})

The flexural crack strength of the column with the spandrel is calculated by Eq.(11-1) and Eq.(12-1) using the standard of AIJ[5]. Similar, The flexural crack strength of the column with the sidewall is calculated by Eq.(11-2) and eq.(12-2).

Capacities (Flexural Capacity, Shear Strength, Failure Strength of spandrel and Slip strength)

Flexural capacity: The flexural capacity of independent column (Q_{Cmy}) is calculated by Eq. (13-1) and eq.(14) using the flexural capacity equation of AIJ [5]. The flexural capacity of column with spandrels (Q_{CWImy}) is given by eq. (14-1) in consideration of the shear span. If the column and the sidewall are one member, the flexural capacity of column with sidewall is calculated by Eq. (14-2) and eq.(14-2)' using the flexural capacity equation [Masuo et al., 6]. Then, after shear slipping in the connection of column and sidewall, the flexural capacity of column with sidewall is given by Eq. (14-3) and eq.(14-3)' from fig.18(e)

Shear strength: The shear strength of independent column (Q_{Cmy}) is calculated by Eq. (15) using the shear strength equation of AIJ [7]. If the column and the sidewall are one member, the calculation of shear strength of column with sidewall is used the shear strength equation [Masuo et al., 6]. In the case of others, the shear strength of the column is calculated by Eq. (15).

Compressive failure strength of spandrel: Since compressive stress distribution of spandrel is assumed to be a triangle from Fig. 18(d), the compressive failure strength of spandrel is given by Eq.(16).

Slip strength: When a crack occurs on the connection of column and sidewalls, it is assumed that the column and the spandrel is slip. Therefore, the shear stress in case the column and the spandrel slip is given by Eq. (17) from Fig. 18(e). And in consideration of the compressive stress of sidewall (C_{wc}), slip strength (Q_{slip}) is given by Eq.18.

Failure mode

Failure mode is classified into the flexural yield of column, if the shear strength of column is lower than the flexural capacity of column. In the column with spandrel, if the compressive failure strength of spandrel (*Cc*) is lower than the flexural capacity of column (Q_{CWImy}), the compressive failure of spandrel occurs. In the column with sidewall, if slip strength (Q_{slip}) is lower than the flexural capacity of column (Q_{CW2my}), the slipping in connection of column and sidewall occurs.

ductility

The displacement in case the flexural capacity of column and the shear strength of column, which were reduced by the plastic angle of column (R_p) , become equal is estimated as the ultimate displacement (δ_u) . The ultimate displacement (δ_u) is given Eq.19 from fig.18.

Table3: the equations to estimate the shear-displacement skeleton curve

$$\begin{split} & \mathcal{K}_{1} = \frac{Q_{c}}{\delta_{m1} + \delta_{s1}} & (1) & \delta_{m1} = \iint_{1}^{1} [\{\varphi(y)\} dydy = \iint_{2}^{1} \left\{ \frac{M_{c}(y)}{EI} \right\} dydy & (2) & \delta_{s1} = \iint_{1}^{1} \left\{ \frac{M_{c}(y)}{GA} \right\} dy & (3) \\ & \delta_{CWn-1} = \frac{1}{6L_{c}} \left\{ -(1-\beta_{c})h^{3} + 3 \left\{ \left(\frac{1}{1+\alpha} - \beta_{c} \right)h_{0}^{1} + (1-\beta_{c})h_{1}^{1} \right\} h^{2} \\ & + 3 \left\{ -\beta_{c}h_{1}^{1+2} + \beta_{c}(h_{c}^{1} + h_{0}^{-2})^{2} \right\} h \left\{ \beta_{s}h_{1}^{3-} - \beta_{c}(h_{c}^{1} + h_{0}^{-2})^{3} \right\} D_{c} & (4-1) \\ & \delta_{CWn-1} = \frac{1}{CL} \left\{ \frac{1}{12EW} & (4-2) \\ & \delta_{CWn-1} = \frac{1}{CL} \left\{ \frac{1}{12EW} & (4-2) \\ & \delta_{CWn-1} = \frac{1}{CL} \left\{ \frac{1}{12EW} & (4-2) \\ & \delta_{CWn-1} = \frac{1}{CL} \left\{ \frac{1}{12EW} & (4-2) \\ & \delta_{CWn-1} = \frac{1}{CL} \left\{ \frac{1}{12EW} & (4-2) \\ & \delta_{CWn-1} = \frac{1}{CL} \left\{ \frac{1}{12EW} & (4-2) \\ & \delta_{CWn-1} = \frac{1}{CL} \left\{ \frac{1}{12EW} & (4-2) \\ & \delta_{CWn-1} = \frac{1}{CL} \left\{ \frac{1}{12EW} & (4-2) \\ & \delta_{CWn-1} = \frac{1}{CL} \left\{ \frac{1}{12EW} & (4-2) \\ & \delta_{CWn-1} = \frac{1}{CL} \left\{ \frac{1}{12EW} & (4-2) \\ & \delta_{CWn-1} = \frac{1}{CL} \left\{ \frac{1}{12EW} & (4-2) \\ & \delta_{CWn-1} = \frac{1}{CL} \left\{ \frac{1}{12EW} & (4-2) \\ & \delta_{CWn-1} = \frac{1}{CL} \left\{ \frac{1}{12EW} & (4-2) \\ & \delta_{CWn-1} = \frac{1}{CL} \left\{ \frac{1}{12EW} & (4-2) \\ & \delta_{CWn-1} = \frac{1}{CL} \left\{ \frac{1}{12EW} & (4-2) \\ & \delta_{CWn-1} = \frac{1}{CL} \left\{ \frac{1}{12EW} & (4-2) \\ & \delta_{CWn-1} = \frac{1}{CL} \left\{ \frac{1}{12EW} & (4-2) \\ & \delta_{CWn-1} = \frac{1}{CL} \left\{ \frac{1}{12EW} & (4-2) \\ & \delta_{CWn-1} = \frac{1}{CL} \left\{ \frac{1}{12EW} & (4-2) \\ & \delta_{CWn-1} = \frac{1}{CL} \left\{ \frac{1}{12EW} & \frac{1}{12EW} & \frac{1}{12EW} & \frac{1}{12EW} \\ & \delta_{CWn-1} = \frac{1}{CL} \left\{ \frac{1}{12EW} & \frac{1}{12EW} & \frac{1}{12EW} & \frac{1}{12EW} \\ & \delta_{CWn-1} = \frac{1}{CL} \left\{ \frac{1}{12EW} & \frac{1}{12EW} & \frac{1}{12EW} & \frac{1}{12EW} \\ & \delta_{CWn-1} = \frac{1}{2EW} & \frac{1}{12EW} & \frac{1}{12EW} \\ & \delta_{CWn-1} = \frac{1}{2EW} & \frac{1}{12EW} & \frac{1}{12EW} & \frac{1}{12EW} \\ & \delta_{CWn-1} = \frac{1}{12EW} & \frac{1}{12EW} \\ & \delta_{CWn-1} = \frac{1}{12EW} & \frac{1}{12$$

E: Elastic modulus of concrete, G: shear modulus of concrete, I: Moment of inertia of column,

A: Cross-section area of column, Iw: Moment of inertia of sidewall column with sidewall,

 A_W : Cross-section area of column with sidewall, n: Elastic modulus ratio of steel to concrete,

 ht, h_k : Height of spandrels h: Height of column t: thickness of wall, l_w : length of sidewall

p_t: Ratio of tensile longitudinal bar to section, D_c: Depth of column, B_c: Width of column, N_c: Axial force of column,

 σ_{B} : Compressive strength of concrete, σ_{i} : Tensile strength of concrete, σ_{y} : Yield strength of longitudinal bar of column, σ_{wv} : Yield strength of shear reinforcement of column,

 p_w : Ratio of shear reinforcement of column, p_s : Ratio of reinforcement of wall,



Fig.19: Compared the calculated value with the test and/or the FEM analysis

Agreement of proposed method to the test and the analysis

Fig.19 shows an example of the calculated value in comparison with the test result and the analysis result. Good agreement between the calculation and the test can be seen in the stiffness, the decrease of stiffness, capacities and the decrease of flexural capacity. Similarly, the calculation result is in agreement with the analysis result. Also, the failure mode of column and non-structural wall was appropriately judged, so, the proposed method can evaluate the structural performance of the RC frame with cast-in-place non-structural wall.

CONCLUSION

The conclusions of this paper are below;

- 1) If the column has the shear reinforcement which can resist the shear force, the structural performance (stiffness, capacity and energy dissipation) of the RC frame with cast-in-place non-structural RC walls (spandrels and sidewalls) is advanced from that of the open RC frame.
- 2) From tests and FEM analysis, the influence to the structural performance of RC frame of principal factor, the shear reinforcement of the column and the shape of the cast-in-place non-structural RC wall, were suggested.
- 3) The shear resistant mechanism of the RC frames (columns) and the non-structural RC walls was clarified with test results and analytical results.
- 4) By the shear resistant mechanisms, this paper proposed the method to predict the structural performance (the load-displacement relationship) of RC frames with non-structural RC walls.

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