Committee Report : JCI- TC161A

Technical Committee on The Calculation Method for The Flexural Strength of Reinforced Concrete Shear Walls

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Abstract

This report is a summary of the activities of the "Technical Committee on The Calculation Method for The Flexural Strength of Reinforced Concrete Shear Walls" established by the Japan Concrete Institute from 2016 to 2017. There are certain cases in which it is not possible to properly calculate the ultimate flexural strength of a reinforced concrete wall with the current simplified formulas, owing to the effects of shear force acting on the wall. Furthermore, we have found that the ultimate flexural strength decreases significantly in a perforated wall compared to an imperforate wall owing to the effects of the openings. The purpose of this technical committee is to identify the factors by which the real strength cannot be handled with the present calculation formulas, investigate methods by which they may be addressed, and propose an estimation method accounting for openings. To achieve this purpose, the committee surveyed the present state of design methods for wall members, particularly their bending behavior, and conducted an FEM analysis of selected test specimens. The committee then proposed an estimation method based upon the results of this analysis. This report summarizes the committee's investigative results.

Keywords: reinforced concrete, wall member, flexural strength, FEM analysis, opening

1. Introduction

Methods to evaluate the ultimate strength of members, including their flexural strength, shear strength, and bond strength, are indispensable in designing reinforced concrete (hereinafter, RC) structures. Of these, a theoretical approach is applicable to the flexural strength. It is believed that by using a flexural analysis of a plane cross-section or approximate calculation techniques based on it, it may be possible to achieve a more accurate evaluation than before. However, in recent years, there has been an emerging recognition of the fact that the flexural strength can

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occasionally not be adequately evaluated with these techniques. This tendency is particularly noted in the case of wall members. In addition, wall members sometimes have openings, in which case evaluating the flexural strength is difficult.

The purpose of this technical committee (Chairman: Professor Daisuke Kato, Niigata University) is to use RC wall members to identify problems in the present estimation methods for the ultimate flexural strength as noted above, and propose a simplified evaluation formula that can resolve these problems. **Table 1** shows the members of this technical committee.

In the case of an imperforate wall member, the following three points are thought to be reasons why adequately accurate evaluation is not possible with the customary ultimate flexural strength evaluation formulas: 1) Strength is greatly dependent upon deformation, as axial reinforcement in not only boundary columns but wall panels contributes to flexural strength. In addition, the effects of each rebar differ, and with some rebar the strain-hardening region must be considered. 2) In cases in which boundary columns are sufficiently confined by hoops, the effects upon the flexural strength are great. 3) Wall-like members cannot escape the effects of shear force. When they are subjected to the shear force, if the confinement of the boundary column to the wall panel is large, then the compression strut force orients toward the upper region of the bottom of the column, but not critical cross-section edge, because of shear force. In short, the flexural strength increases. In contrast, if the confinement of the boundary column is small, then the neutral axis depth becomes longer because of the increased cross-sectional compression regions, and the flexural strength may decrease.

On the contrary, the following two points are thought to be reasons when wall members have openings: 4) Openings result in a loss of axial reinforcement, and therefore opening reinforcement bars with unknown effects on the flexural strength are present. 5) When there are openings, the assumptions of plane holding in critical cross-sections (base or opening bottom surfaces) collapse.

The technical committee established and implemented the following activity plan to investigate these factors in detail.

・2016 activity plan

First, the committee conducted a survey of past experimental studies of wall members in the architecture and civil engineering fields, and sampled problems with customarily used ultimate flexural strength calculation formulas. Furthermore, they used imperforate barbell type wall members, and performed FEM analyses on selected representative test specimens, for which the ultimate flexural strength calculation formulas are unable to estimate the experimental results. Thus, it is thought possible to evaluate the stress distribution and other elements that cannot be ascertained experimentally.

・2017 activity plan

The technical committee used barbell type wall members with openings to perform FEM analyses on selected representative test specimens, for which the ultimate flexural strength calculation formulas are unable to estimate the experimental results. Next, they proposed an accurate flexural strength formula for barbell type wall members, including the results of the FEM analysis of imperforate wall members from the previous year.

The following is the table of contents of this report. The report consists of two volumes. Volume I introduces the present status of the work and research surveyed by the technical committee, and Volume II introduces the results of the FEM analysis conducted by the committee and several proposals based upon it.

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Chap. 5 Imperforate verification test specimens and FEM analysis thereof
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Chap. 7 Perforated test specimen and FEM analysis thereof
Chap. 8 Study of various effects of openings on strength characteristics

Table 1: Technical committee

Chairman	Daisuke Kato	Niigata University
Representative secretary	Kazuaki Tsuda	Kindai University
Secretary	Hikaru Nakamura	Nagoya University
	Tomoya Matsui	Toyohashi University of Technology
	Hiroshi Matsuzaki	Tohoku University
Members	Mitsuyoshi Akiyama	Waseda University
	Masaru Okamoto	Railway Technical Research Institute
	Hitoshi Kumagai	Shimizu Corporation
	Kenichi Kawano	Kajima Corporation
	Takefumi Shindo	Taisei Corporation
	Hiroya Hagio	Obayashi Corporation
	Yoshimasa Honda	National Institute of Technology, Oyama College
	Hidekazu Watanabe	Building Research Institute

Below is a summary of the report.

2. Status and problems of modeling RC wall members

2.1 Modeling of the shear wall with boundary elememt in practical design

Fig. 1 shows a method of handling the shear wall with boundary elememt in practical structural design. In general, there are four known techniques: brace replacement, beam replacement, element replacement, and finite element replacement.

Each of these replacement methods has their advantages and disadvantages. Designers need to understand the characteristics of the replacement methods and make proper selections, considering the structural properties of the overall building and the structural characteristics with which the shear wall will function. In addition, the way in which the shear wall modeling is selected and nonlinearity is considered is occasionally critical. A shear wall will crack depending upon the size of the horizontal force loaded onto it, and the load balance of the horizontal force will change from the elastic stage, such that the stress and deformation generated in each part of a building need to be ascertained according to the state of the stiffness reduction in the shear wall.

Furthermore, presently, structural design often involves the use of coherent structural calculation software for buildings of 60 m or less in height. There are several types of coherent structural calculation software, but element replacement is often applied to the modeling of the shear wall with boundary elememt.

Decreased rigidity and strength due to openings in the shear wall with boundary elememt are mentioned in a bulletin (2007 Notice No. 594-1 of the MILT). They are generally thought to represent a reduction in the shear rigidity and shear strength according to the reduction ratio (r), based on various opening factors, the length ratio of the shear wall with boundary elememt, the opening circumferential ratio, and other elements. Here, the reduction ratio was set according to the shape and dimensions of the openings, with nothing regarding the position of the openings or the like.

However, bulletin (2007 Notice No. 594-1 of the MILT) 3, mentions, “In cases where the top of the opening part is made to come into contact with a beam on that floor, and the lower edge of the opening part is made to come into contact with a slab on that floor, that floor shall not be treated as one wall,” regarding opening shapes demonstrating behavior clearly different from the shear wall with boundary elememt. A wall providing a large vertically long opening part shall be treated as a parallel wall form consisting of two walls of the opening part and a beam connecting them.

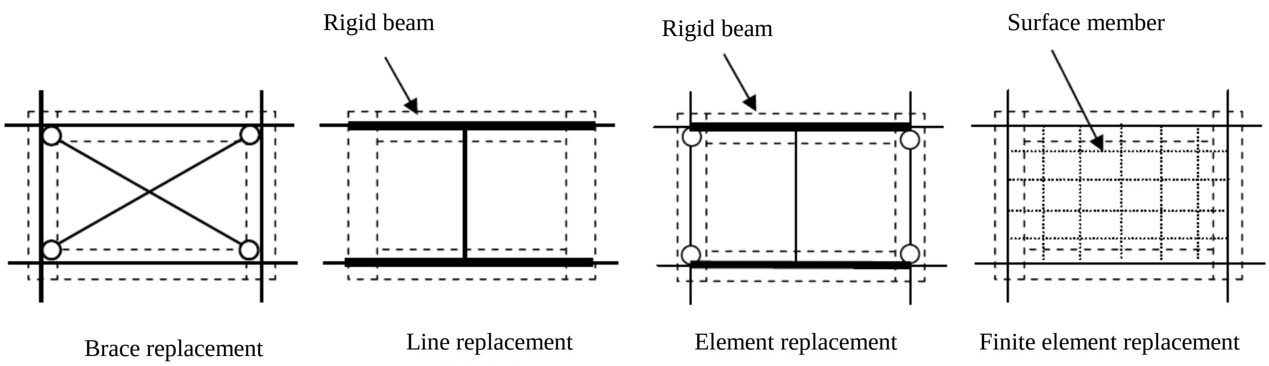


Fig. 1: Replacement model example of two-column wall

2.2 Three-dimensional wall

RC building in Japan was developed based on a rigid-frame structure, and popularized “shear walls” incorporated into columns and beam frames. Overseas, however, not only flat walls, but also three-dimensional walls are used as vertical members for flat slab structures. A typical example of this is the Burj Khalifa, which is currently the tallest high-rise structure in the world.

In recent years, there has been a reorganization of wall thicknesses and column cross-section dimensions in Japan as well, resulting in the development of structures rigid-frame and three-dimensional walls. The technical committee examined design and research examples of such three-dimensional walls.

Three-dimensional walls are often used in the center-core of high-rise RC buildings with high-strength concrete of 60-100 N/mm^2 adopted for the lower stories. It has been experimentally proven that in three-dimensional wall such as L-shaped cross-sections using high-strength concrete, if there is bending in the weak axis direction while under high compressive axial force, areas near the corners will crush rapidly. It has been confirmed that the strength and ductility during such flexural failure may be evaluated by an analysis assuming that plane sections remain plane, if a stress–strain relationship is established to properly imitate the plain and laterally confined concrete. In structural design, restoring force characteristics may reportedly be expressed using a model replacing the three-dimensional walls with multiple columns, but criteria must be set based on the flexural failure behavior, like that above.

2.3 Wing wall column

As wing wall columns experienced shear failure on past earthquakes, they are designed ignored wing walls by installing slits. However, recently, it has been desired that wing wall column members be arranged on buildings to limit the building damage during earthquakes, as wing wall column has high strength and rigidity. Hence, the performance evaluation of wing wall columns is being researched.

The Architectural Institute of Japan (AIJ) presents evaluating methods for the performance of wing wall column members in the Standard for structural calculation of reinforced concrete structures[1)] and the Standard for Lateral load-carrying capacity calculation of reinforced concrete structures (Draft)[2)]. In Reference 2), the ultimate bending moment is calculated by the plastic bending theory, ignoring the effects of the compression side rebar and assuming compression the side concrete to be a rectangular stress block, and the ultimate shear strength is calculated summing the ultimate shear strengths of column and wall sections which are split the cross-sections of the wing wall column into in the wall direction. This strength calculation formula better corresponds to the experimental results compared to calculation formulas such as the standard for seismic evaluation[3)4)] of the Japan Building Disaster Prevention Association.

Openings are sometimes provided on wing wall columns for ventilation. In Reference 2), the effects of openings on the rigidity, strength, and deformation of wing wall columns are to be adequately considered. In Reference 5), the effects on the strength and deformation due to opening positions are discussed. The region to which the positions of openings affect the flexural strength and ultimate deformation is calculated from a comparison of the flexural strength of the wing wall column without openings and with openings and the crushing region of the critical section. Then they compared with the result of a bending test of the wing wall column with openings. Based on the results, it is possible to confirm that the flexural strength of specimens which have openings in an effective area, is affected by the openings. The ultimate deformation of specimens and the calculated values for the ultimate deformation of the wing wall column ignoring the openings were almost the same, and this shows the regions where the openings affect the strength and ultimate deformation of wing wall column with an opening. Moreover, this indicates that the experimental results can be explained approximately for the relationship between the regions and openings.

Also, it introduces an FEM analysis conducted to discuss the flexural strength of wing wall columns with openings concerning the axial stress on the bottom. An FEM analysis conducted of wing wall column bending specimens with openings arranged in the side of the column, the center of the wing wall and the center on the bottom, thus determining the stress distribution in the bottom. Based on the results, the axial stress at the bottom under openings at maximum strength decreases, and the stress distribution affects the flexural strength.

2.4 Wall-like members in civil engineering

In the civil engineering field, there are various types of wall-like members. These include wall piers in the transverse direction, wall in tanks and tunnels, and box structures in parking lots and waterworks. We have gathered the current findings on these wall-like members.

(1) Experimental examples of wall-like members

Experiments are being conducted in the civil engineering field on the bending behavior and shear behavior of wall-like members. Otsuka, et al.[6)] studied I shape cross-section flexible RC piers with shear walls in the transverse direction used in tall piers in mountainous areas, for example. They showed that by increasing the amount of transverse reinforcement in walls and the amount of tie reinforcement in columns, ductility is improved, but if both sets of rebar are increased by a certain amount or more, there is an upper limit to the improvement in ductility. Furthermore, having column members on both ends of shear walls is critical to improving ductility, as indicated by El-Azizy, et al.[7)] Shinohara, et al.[8)] conducted a study of low-reinforcement-ratio RC wall piers in the longitudinal direction with a low amount of axial rebar compared to piers designed according to present standards, which are of an older design generation. They indicated a characteristic that a curvature is developed only at pier bases, which leads to a rocking behavior and rupture of longitudinal reinforcement, although the cover concrete does not spall off.

Regarding the shear behavior of wall-like members in civil engineering, Ishibashi, et al.[9)] conducted loading test based upon wall piers having a cantilevered form, and considerable side rebar in the transverse direction compared to RC beam members. There were clear differences in the spread of damage depending upon the transverse reinforcement ratio. These included changes in whether there were scattered diagonal cracks.

(2) Modeling of wall-like members

In civil engineering field, wall-like members are generally modelled as linear members, and bending behavior is often calculated based on Bernoulli-Euler theory, while shear behavior is often calculated based on deep beams with small shear span ratios. This is because conservative capacity can be obtained based on linear members.

For the sake of a rational behavior evaluation and capacity calculation for wall-like members, finite element analysis is also performed. In the guidelines for structural performance verification framework of LNG underground storage tanks of the Japan Society of Civil Engineers (JSCE)[10)], verifying LNG tanks with side walls that are wall-like structures against seismic performance grade 3 is based on using dynamic nonlinear analysis techniques accounting for the nonlinear characteristics of the constitutive material. In the 2007 design volume[11)] of the JSCE Standard Specifications for Concrete Structures, nonlinear FEMs are collected for reference so they may be used as one technique for structural performance verification, although use of nonlinear FEMs are limited to calculate the response values. In the 2012 design volume[12)], verification indexes and limit values are indicated based on material damage indexes, and a system enabling verification based only on results obtained by FEMs is organized.

(3) Present state of design methods for various civil engineering structures

Here, piers, open cut tunnels, tanks, and underground parking lots are treated as examples of civil engineering structures with wall-like members, and points in treating as wall-like members are summarized. As an example, wall pier is shown here. Generally, piers with cross-sectional width three times or greater than their cross-sectional height are referred to as wall piers. Wall piers often have high flexural capacity in the transverse direction, making it difficult to satisfy their shear demand, and thus they often have densely arranged shear reinforcing bars. In addition, it is generally difficult to find and repair damage in foundations, and thus they are not plasticized. However, it is allowed that piers respond elastically and foundations are plasticized for wall piers in the transverse direction[13)].

2.5 Evaluation of damage in RC wall members

For various nonstructural design reasons, RC buildings often have frames designed with non-load-bearing walls, such as hanging walls and spandrel walls. Considering the damage from recent major earthquakes (2011 Great East Japan Earthquake, 1995 Great Hanshin Earthquake, etc.), even when buildings managed to avoid collapsing owing to earthquakes, building damage has led to economic loss in certain cases. For example, there are cases of RC residential housing that had non-load-bearing walls around entrances damaged by earthquakes, such that the entrance doors could no longer function. There are also examples in which this damage further led to buildings being demolished. Thus, conventional non-load-bearing walls, compared to columns, beams, two-column load-bearing walls, and the like, are more often damaged by minor deformation. Hence, even if a building is safe, damage to non-load-bearing walls may result in a major loss of building functionality.

In recent years, members have been proposed with improved damage control performance compared to conventional non-load-bearing walls, as a way of increasing the seismic performance of non-load-bearing walls and effectively using them as seismic elements. In general, column members have been proposed with multiple arrangements of non-load-bearing walls 200 mm or more in thickness providing constrained regions at their ends. Such non-load-bearing walls with relatively high earthquake resistance compared to conventional walls are effectively being designed no longer as non-load-bearing walls, but as seismic elements (load-bearing walls) of structural design.

In addition, in recent years, there have been demands for buildings with greater seismic performance. For example, there have been cases of demands for buildings that remain undamaged or can be restored with simple repairs even after a major earthquake (continued usability after major earthquakes). Responding to these demands requires structural design to predict building damage corresponding to the intensity of earthquakes. Therefore, it is important to evaluate damage to non-load-bearing walls, load-bearing walls, and the like in RC buildings, which are especially prone to damage. However, it is difficult to make detailed evaluations of damage in the present design work. Past research has mainly been centered on experiments evaluating the strength of members and buildings. Despite a slight increase in the number of studies on deformation performance and damage evaluation in recent years, it is still inadequate. Hence, there is presently a lack of engineering knowledge for making designs based upon evaluating damage. In this section, we introduced previous studies aiming to evaluate damage in non-load-bearing and load-bearing walls.

3. FEM analysis results and proposed modeling

3.1 Imperforate verification test specimens and FEM analysis thereof

We conducted an FEM analysis of past experimental results to ascertain the mechanisms resisting imperforate wall member flexural strength. There were three test specimens, which were all type I wall members with shear span ratios of 1.90[6], 0.99[14], and 0.59[15]. See Reference 16) for the analytical assumptions and detailed analysis results. **Fig. 2** shows the bending-moment–curvature relationship of the wall legs obtained by conferring horizontal force (flexural shear) and overturning bending moment (bending only) on the test specimen with a 0.99 shear span ratio. From this figure, we find that the bending-moment–curvature relationship, including the flexural strength, is affected by the shear force, and from the results of the other two specimens, the degree of the effects at least fluctuates with the shear span ratio. The figure shows the results of using Formula (1) and the e function method.

$$M_u = a_g \sigma_{cy} \ell_w + 0.5 a_{wv} \sigma_{vy} \ell_w + 0.5 N \ell_w \qquad (1)$$

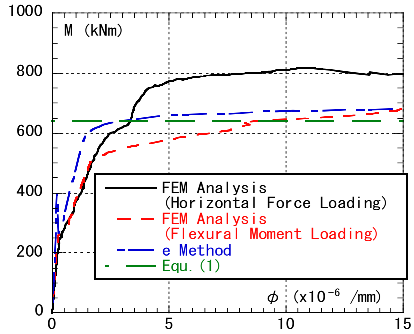


Fig. 2: Comparison of bending-moment–curvature relationship of wall legs (shear-span ratio 0.99)

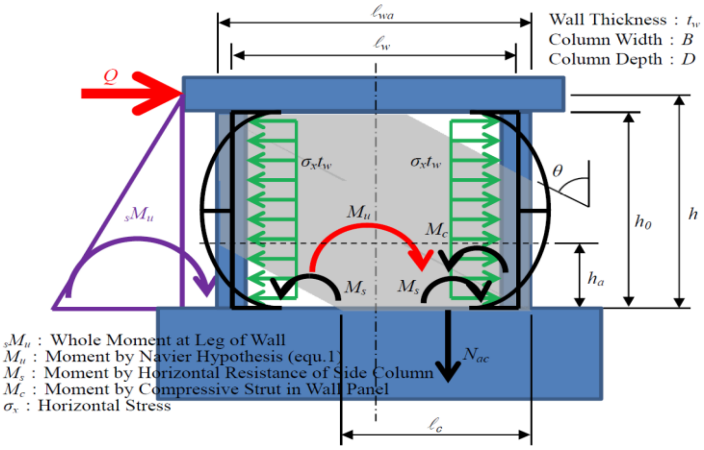


Fig. 3: Assumed bending moment produced in wall legs

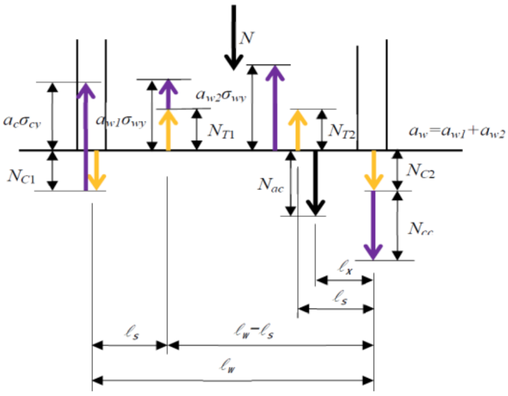


Fig. 4: Assumed shear force in wall legs

3.2 Study of effects of imperforate test specimen on flexural strength and proposed design formula

Here, we show a summary of a flexural strength calculation method reflecting the first wall member shear force effects and the calculation accuracy thereof.

Essentially, we consider behavior whereby shear force results in barrel-like bulging of a large compression strut developing within the wall and the wall itself, and assume that this results in three bending moments acting upon the wall legs. **Fig. 3** shows a summary of this. Then we assume that a cross-sectional force is produced in the wall legs by these bending moments, as illustrated in **Fig. 4**. The bending moment during the flexural strength in the wall legs from this cross-sectional force is calculated by Formula (2). See Reference 16) for the details of this proposed technique.

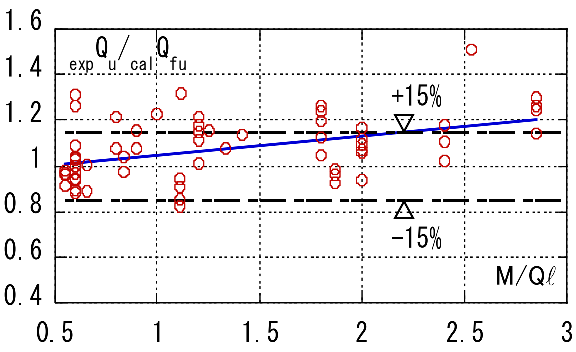
We verified the calculation accuracy of this proposed technique using past experimental results[16]. We compared the results to those according to Formula (1), and show them in **Fig. 5**. According to Formula (1), the calculation accuracy fluctuated with the shear span ratio, but with the proposed technique, the regression line slope decreases.

$$\begin{aligned}
{}_sM_u &= \left(a_c\sigma_{cy} + N_{C1}\right)\ell_w + a_w\sigma_{wy}\left(0.5\ell_w\right) + N\left(0.5\ell_w\right) \\
&\quad - N_{T1}\left(\ell_w - \ell_s\right) + N_{T2}\ell_s - N_{ac}\ell_x \\
&= a_c\sigma_{cy}\ell_w + 0.5a_w\sigma_{wy}\ell_w + 0.5N\ell_w \\
&\quad + \left(N_{C1} - N_{T1}\right)\ell_w + N_{T1}\ell_s + N_{T2}\ell_s - N_{ac}\ell_x \\
&= M_u + 2M_s - M_c
\end{aligned} \tag{2}$$

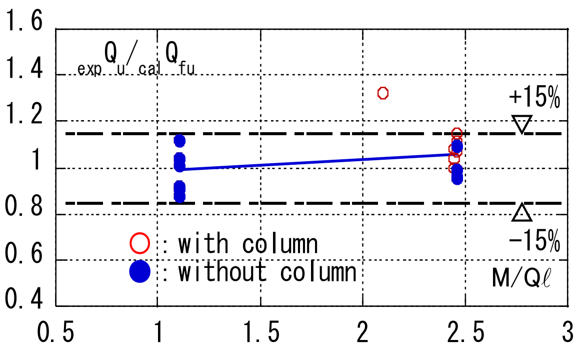
3.3 Perforated test specimen and FEM analysis thereof

The flexural strength of walls with door-type openings decreases compared to imperforate walls, owing to the openings, as was made clear experimentally by Ishikawa, et al.[17], and Iwamoto and Tsuda[18]. This decreasing tendency is believed to be due to fluctuations caused by the size and position of openings, but the experimental data are limited, and thus it is not possible at present to experimentally ascertain this tendency. Therefore, we decided to ascertain this tendency using FEM analysis. First, we performed a simulation analysis on the experimental results of Ishikawa, et al.[17], and Iwamoto and Tsuda[18], and after ascertaining a valid analytical assumption, used it to perform a parametric study.

Fig. 6 shows the flexural strength reduction ratio resulting from an analysis using opening positions as a variable (the ratio of the imperforate wall flexural strength to the perforated wall flexural strength). The horizontal axis represents the distances from the tension side of the wall to the center position of the openings divided by the wall length. The analytical loading procedure involves the horizontal force loading on the top of the wall.

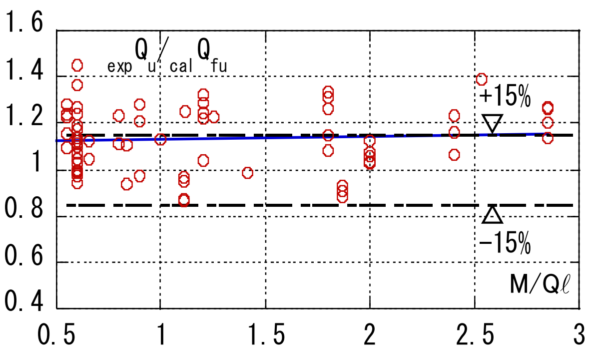


a) Type I wall

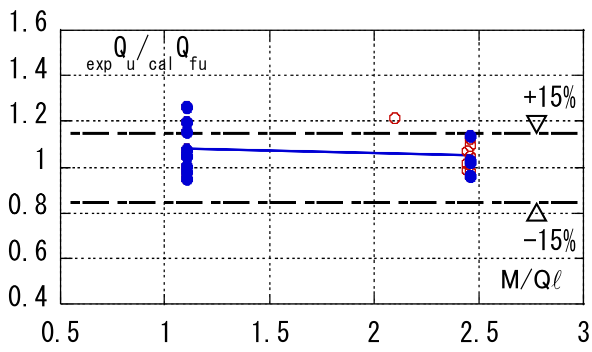


b) Rectangular wall

1) Formula (1)



a) Type I wall



b) Rectangular wall

2) Proposed technique

Fig. 5: Comparison of calculation accuracy

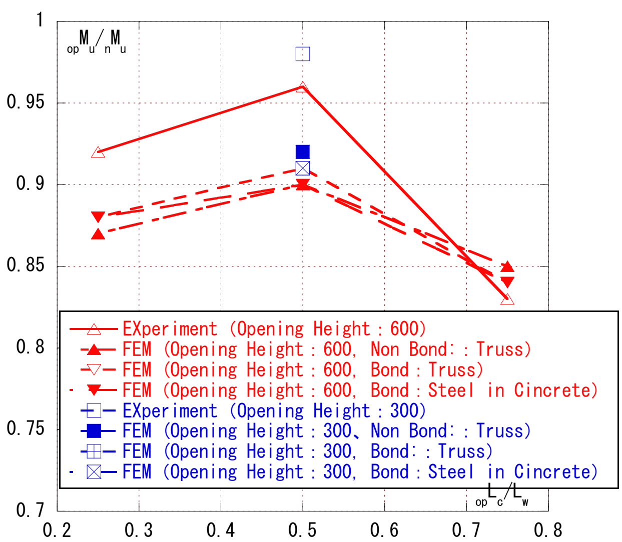


Fig. 6: Relationship of flexural strength reduction ratio and opening positions

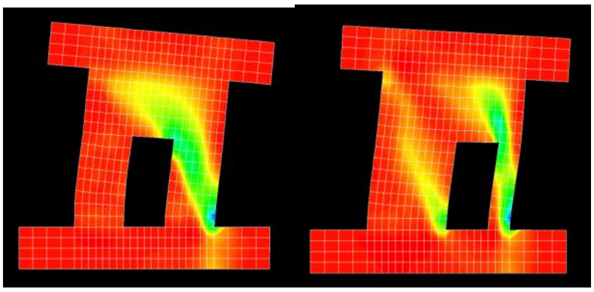
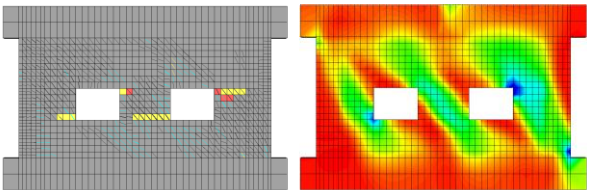


Fig. 7: Minimal principle stress intensity contour during maximum lateral force



Concrete damage　　Minimal principle stress intensity

Fig. 8: Example of analysis of perforated wall member receiving horizontal force

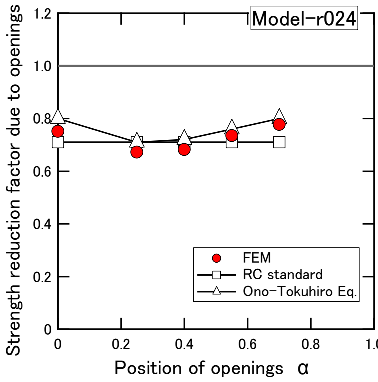


Fig. 9: Comparison example of strength reduction ratio due to openings (horizontal axis values are the distances from the wall centers to opening centers divided by ½ of the entire wall)

The flexural strength reduction ratio decreases as the openings approach either end, compared to when the openings are in the middle. In particular, when the openings are on the wall compression side, the degree of decrease is large. As shown in **Fig. 7**, compression struts are formed only in the compression side regions divided by openings when there are openings in the middle, and are formed on both the tension and compression sides when the openings are on the compression end of the wall panel. The former are governed by the flexural resistance of the entire wall, whereas the latter have large tension side region flexural resistance effects. Thus, it is believed that if there are openings present on the compression side, the degree of decrease in the flexural strength from the imperforate wall is large.

3.4 Study of various effects of openings on strength characteristics

For the evaluation of the rigidity and strength of wall members with openings, it is desirable that when openings are small, wall members are modeled and evaluated with reduced rigidity and strength, and when openings are large, perforated walls are modeled with columns with wing and beams with spandrel and hanging walls as frames, and then the rigidity and strength are evaluated. The AIJ Standard for structural calculation of reinforced concrete structures[1] may be applied to designs modeled with $r_2 \geqq 0.6$ for shear walls.

The opening reduction ratio is defined using the length ratio of openings, height ratio of openings, and surface area ratio of openings, which is a simple method, although it does not account for opening positions. Wall members with openings come in various forms, and thus it is difficult to simply think that perforated walls demonstrate complex failure behavior from experiments and actual earthquake damage.

Therefore, in this report, we performed an FEM analysis using the opening size and position as analytical variables (**Fig. 8**) and identified the effects on the stress state and ultimate shear strength, and then showed the results of studying the applicability of an existing strength reduction ratio (reduction ratio for imperforate shear strength). The results confirm that the strength reduction ratio[19] according to Ono and Tokuhiro generally captures the tendency of the strength reduction ratio to change with respect to the opening position according to the FEM analysis (**Fig. 9**).

Although omitted in this report, we are studying the flexural strength and modeling cases other than those mentioned above.

4. Conclusion

This report is a summary of the activities of the “Technical Committee on The Calculation Method for The Flexural Strength of Reinforced Concrete Shear Walls” established in 2016–2017 by the Japan Concrete Institute.

The members of this committee first surveyed the present state of design methods for wall members, mainly bending behavior. They showed that it is not possible to make valid calculations of the ultimate flexural strength in RC walls with the present simplified formula owing to the effects of shear force, and furthermore, the ultimate flexural strength in perforated walls greatly decreases compared to imperforate walls owing to the openings.

Next, in order to identify the factors involved, they selected three typical imperforate test specimens and four typical perforated test specimens, and conducted FEM analyses. From the results obtained, they attempted modeling while accounting for the behavior of imperforate shear walls, whereby shear force results in barrel-like bulging of a large compression strut developing within the wall and the wall itself, and proposed a bending moment calculation formula.

The fact that valid calculations of the ultimate flexural strength in RC walls are sometimes not possible with the present simplified formula, owing to the effects of shear force may be considered the most significant finding of the technical committee. However, another subject of the technical committee was opening walls, and the committee was unable to reach a proposal that advanced past research. We should further state that it was not possible to make an adequate attempt to apply the proposed calculation formula to various wall members other than barbell type walls. It is felt that this is due to the short two-year period for this study. It would be desirable if the present study results were used as the basis for future secondary research.

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