

New design guidelines for medium rise reinforced masonry buildings in Japan

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ABSTRACT: The number of stories of masonry buildings is limited up to three and a large amount of shear walls are required in the current design code in Japan. Synthetic experimental and analytical research had been carried out under the U.S.-Japan Coordinated Research Project on Masonry Structures since 1984. Main objective of Japan was to realize earthquake resistant masonry buildings with the larger story number but the less amount of shear walls than those required in the current design code. New design guidelines for reinforced masonry buildings were proposed in March 1989. The ductility design concept as well as the strength design concept was employed in the new design guidelines.

1 INTRODUCTION

In the current design code for reinforced masonry (RM) buildings in Japan, the number of stories is limited up to three and a large amount of walls (high wall length ratio) and stiff reinforced concrete (RC) girders are required. Therefore RM buildings are not popular in Japan. A five year research project had been carried out since 1984 to realize and to promote earthquake resistant RM buildings in Japan. One of the objectives of this research project is to establish new design guidelines for medium rise RM buildings. The RM buildings designed by the new design guidelines would have the larger number of stories but have the less wall length ratio than those required in the current design code, and they would have RM wall girders instead of RC girders.

Synthetic experimental and analytical research had been carried out under the Technical Coordinating Committee on Masonry Research (TECCMAR/Japan) of the U.S.-Japan Coordinated Research Project on Masonry Structures. Research items, such as a) material, b) structural members, c) seismic test on full-scale frames and a full-scale five story building, d) earthquake response analyses, and e) trial design of the RM buildings had been done. Based on the results of this research, new design guidelines had been proposed by the TECCMAR/Japan in March 1989. Outlines of the design guidelines are described in this paper.

2 DESIGN GUIDELINES

2.1 RM buildings

The RM buildings dealt with in the design guidelines are up to five stories, and consist of RM shear walls, RM wall girders, RC slabs, RC foundation beams, and RC foundations. The RM members are to be a fully grouted

reinforced masonry system that is made by concrete or clay RM units that are stacking open-end types, as shown in Fig. 1. The RM units are laid using joint mortar in parallel with arranging reinforcing bars into unit cells vertically and horizontally, as shown in Fig. 2.

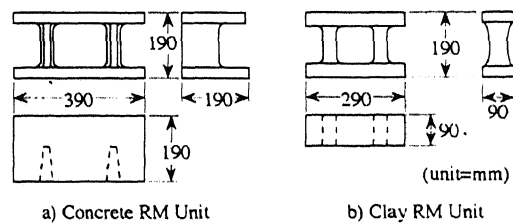


Fig. 1 Concrete RM unit and clay RM unit

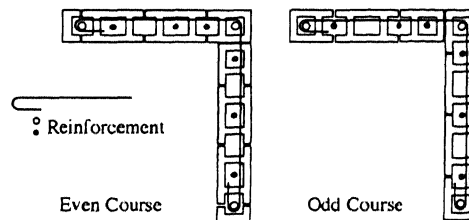


Fig. 2 Typical bonding and reinforcing pattern

2.2 Required seismic performance

The design guidelines require that the RM buildings should have the following seismic performance.

1. The RM buildings should be a structure that does not cause any trouble on serviceability against moderate earthquakes and does not collapse in the event of severe earthquakes.

2. A failure mechanism of the RM buildings basically

should be governed by flexural yieldings at the base of shear walls at the first story and at the wall girder ends.

3. The RM buildings should be a structure that possesses the ultimate lateral load carrying capacity and the deformation capacity corresponding to the structural performance coefficient (Ds) of 0.5 or more.

The first part of the item 1 signifies that no shear crack occurs in most structural members at the design lateral seismic forces corresponding to the moderate earthquakes. The last part of the item 1, and items 2 and 3 signify that the failure mechanism of the RM buildings is governed by flexural yieldings that certify that almost no strength degradation occurs until a story drift angle of 0.005radian, and that no major strength deterioration occurs until that of 0.01radian, as well as possessing sufficient lateral strength against the design lateral seismic forces corresponding to the severe earthquakes.

2.3 Design flow

There are two design flows in the design guidelines called "Design Method A" and "Design Method B." The RM buildings can be designed either by the Design Method A or by the Design Method B.

In the case of the Design Method A, the RM buildings are designed by a working stress design method with base shear coefficient of 0.2 and more. Also the RM buildings are designed by an ultimate lateral strength design method. The seismic performance corresponding to the Ds value of 0.5 or more is to be certified by the ultimate lateral strength design of a building and the ductility design of structural members.

The Design Method B has the almost same design flow as the Design Method A. But the design of shear walls and the ultimate lateral strength design are simplified. The shear walls are designed by checking the average shear stress in the shear walls due to the lateral seismic force corresponding to the base shear coefficient of 0.2 or more. The average shear stress should satisfy Equation (1)

$$\bar{\tau} \leq 0.392 \sqrt{F_m / 17.7} \quad (1)$$

where

$\bar{\tau}$: average shear stress produced in shear walls due to the short term design load, (MPa)

$$\bar{\tau} = Q / \Sigma A_w$$

Q: seismic shear force corresponding to the base shear coefficient Co of 0.2 or more, (N)

$$Q_i = Z \cdot R_t \cdot A_i \cdot C_o \cdot \sum_{i=1}^n (9.81 W_i)$$

Qi: seismic shear force at the i-th story, (N)

Z: seismic hazard zoning factor, $0.7 \leq Z \leq 1.0$

Rt: design spectral factor, $R_t \leq 1.0$

Ai: lateral shear distribution factor at the i-th story,

$$A_i = 1 + \left(\frac{1}{\sqrt{\alpha_i}} - \alpha_i \right) \cdot \frac{T}{1 + 3T}$$

$$\alpha_i = \frac{\sum_{i=1}^n W_i}{\sum_{i=1}^n W_i}$$

T: fundamental natural period of the building, (sec)

Co: base shear coefficient for the short term design, $C_o \geq 0.2$

Wi: weight of the i-th story, (kg)

n: number of stories

ΣA_w : summation of wall area in each direction, (mm²)

Fm: specified compressive strength of the RM assemblage, (MPa)

If the average shear stress due to the short term design load satisfies the Equation (1), the shear stress with considering a shear stress concentration coefficient of 1.5 will not exceed the allowable shear stress of the RM assemblage for the short term design load. Therefore no shear crack is expected to occur in the shear walls in the RM buildings subjected to the moderate earthquakes. The ultimate lateral strength of the RM buildings is checked by calculating total flexural resisting moment of the entire building in the Design Method B.

2.4 Material

RM units for the RM buildings are classified into two kinds according to their materials made from, namely, concrete and burned clay (or ceramics).

Specified compressive strength of grout concrete and grout mortar should be more than 17.7MPa. Specified compressive strength of concrete that is used in RC portion should be ranged from 17.7MPa to 26.5MPa.

Specified compressive strength of the RM assemblage should be ranged from 17.7MPa to 26.5MPa. The specified compressive strength of the RM assemblage should be verified by a prism test as a rule. Instead of applying the prism test, the specified compressive strength of the RM assemblage may be verified by Equation (2).

$$F_m = e_s \{ (1 - \beta') F_u + \beta' \cdot F_g \} \quad (2)$$

where

Fm: specified compressive strength of the RM assemblage, (MPa)

Fu: specified RM unit strength, (MPa)

Fg: specified grout strength, (MPa)

β' : cavity ratio of the RM unit

e_s : masonry reduction factor, $e_s \leq 0.75$

Allowable stress of the RM assemblage should be in accordance with the values in Table 1.

Table 1 Allowable stress of RM assemblage

Description	Allowable Stress (MPa)	
	for Stress due to Long Term Design Load	for Stress due to Short Term Design Load
Compression	$F_m / 6$	$F_m / 3$
Shear	$0.313 \sqrt{F_m} / 3$	$0.313 \sqrt{F_m} / 2$

2.5 Structural specifications

Structural specifications are defined to keep the minimum structural performance of structural members of the RM buildings. The structural specifications specify the minimum and/or the maximum values such as locations, dimensions, and reinforcement those are not determined only through the structural calculations.

2.5.1 Shear walls

1. Shear walls should be greater than 59cm in length.
2. The minimum thickness of shear walls should be greater than 19cm and 1/20 of the vertical distance between main lateral supports.
3. Vertical and shear reinforcement should be arranged with the amount of reinforcement required by the allowable stress design and the structural specifications in Table 2.

Table 2 Reinforcement in shear walls

Story from the Top	Extreme Vertical Reinforcement	Intermediate Vertical Reinforcement	Shear Reinforcement
1 and 2	$\geq 1\text{-D16}$	Ratio $\geq 0.2\%$	Ratio $\geq 0.2\%$ Space \leq Height of Unit and 20cm
3, 4, and 5	and $\leq 2\text{-D25}^*1$	Space \leq Length of Unit and 40cm	Ratio $\geq 0.25\%$ Space \leq Height of Unit and 20cm
Size of Re-Bar	D16 ~ D25	D10 ~ D16	D10 ~ D16

*1 : 2-D25 means two deformed re-bars which diameter is 25mm.

2.5.2 Wall girders

1. Wall girders should be provided effectively to connect shear walls each other at every story.
2. Thickness of wall girders should be equal to or more than the thickness of the shear walls connected to them.
3. Depth of wall girders should be more than 45cm.
4. Ratio of the clear span length to the depth of wall girders should be more than 1.5.
5. Lateral and shear reinforcement should be arranged with the amount of reinforcement required by the allowable stress design and the structural specifications in Table 3.

Table 3 Reinforcement in wall girders

	Extreme Vertical Reinforcement	Intermediate Vertical Reinforcement	Shear Reinforcement
Amount of Reinf.	$\geq 1\text{-D16}$ and $\leq 2\text{-D25}^*1$	Ratio $\geq 0.25\%$	Ratio $\geq 0.25\%*2$
Space	-----	$\leq 40\text{cm}$	$\leq 20\text{cm}$
Size of Re-Bar	D16 ~ D25	D10 ~ D25	D10 ~ D16

*1: 2-D25 means two deformed re-bars which diameter is 25mm.

*2: In case of short beams, the shear reinforcement ratio should be 0.3% or more.

2.6 Stress and deformation analysis

Stress and deformation of the RM buildings should be evaluated using elastic stiffness of the structural members. Frame analysis methods with the following consideration can be used for the evaluation.

1. Flexural and shear deformations and rigid zones at shear wall to wall girder joints should be taken into account.
2. Effective width of transverse walls and floor slabs should be appropriately taken into account to evaluate the stiffness of T-shape structural members.
3. Stiffness of foundation beams should be taken into account. Deformation of a ground and piles can be appropriately evaluated as well.

2.7 Total flexural resisting moment (Design Method B)

For the RM buildings to which the Design Method B is adopted, the state at the ultimate stage of the structure is not clear because only the allowable stress design method is used. Therefore a simplified check of the lateral load carrying capacity is obligated. The RM buildings by the Design Method B should satisfy Equation (3).

$$(\Sigma M_{bu} + \Sigma iM_{wu}) \geq 0.55 Z \cdot R_t \sum_{i=1}^n \{A_i \cdot h_i \sum_{i=1}^n (9.81 W_i)\} \quad (3)$$

where

ΣM_{bu} : summation of moment capacity of wall girders at nodal points, (N·mm)

ΣiM_{wu} : summation of moment capacity of the shear walls at the bottom of the first story, (N·mm)

h_i : height of the i-th story, (mm)

Z, R_t, n, A_i, W_i : the same as those in the Equation (1)

2.8 Lateral load carrying capacity design

To keep the safety of the RM buildings that does not collapse in the event of the severe earthquakes, the RM buildings should have enough lateral load carrying capacity and enough ductility of the structural members. Design procedures against the severe earthquakes are as follows;

1. to calculate the ultimate flexural and shear strength of each structural member of a building,
2. to calculate the lateral load carrying capacity of the building in each design direction and to decide the stress and the yield type for each structural member under the mechanism of the building,
3. to decide the structural performance coefficient D_s using the member type for shear walls, wall girders, and foundation beams,
4. to calculate the required lateral load carrying capacity at each story in each design direction and compare with the lateral load carrying capacity of the building,
5. to do the ductility design for the shear walls, the wall girders, and the foundation beams.

2.8.1 Required lateral load carrying capacity

The lateral load carrying capacity of the RM buildings should be greater than the required lateral load carrying capacity expressed by Equation (4).

$$Q_{un} = D_s \cdot F_{cs} \cdot Q_{ud} \quad (4)$$

where

Q_{un} : required lateral load carrying capacity at each story, (N)

D_s : structural performance coefficient

F_{cs} : shape factor that is determined based on eccentricity and variation of the lateral stiffness along the building height, $1.0 \leq F_{cs} \leq 2.25$

Q_{ud} : lateral seismic shear force at each story, (N)

$$iQ_{ud} = Z \cdot R_t \cdot A_i \cdot C_o \sum_{i=1}^n (9.81 W_i)$$

iQ_{ud} : lateral seismic shear force at the i -th story, (N)

C_o : base shear coefficient for the lateral load carrying capacity design, $C_o \geq 1.0$

Z, R_t, n, A_i, W_i : the same as those in the Equation (1)

2.8.2 Structural performance coefficient D_s

The structural performance coefficient D_s is a reduction factor of the required lateral load carrying capacity that is varied with the deformation capacity of buildings. Buildings with the larger deformation capacity can take the smaller D_s value. This means that the buildings with the larger deformation capacity are required the less lateral load carrying capacity.

From the response analyses of the buildings which D_s value is 0.5, the response of story drift angle was less than 0.005radian under the 50cm/sec input of Hachinohe 1968-NS, Taft 1952-EW, and El Centro 1940-NS. Furthermore results of the test on structural members indicate that the shear walls with a flexural failure mode have deformability of more than 0.008radian of story drift angle (Fig. 3). In the case of the wall girders, it was more than 0.01radian. Therefore the D_s value is decided to be 0.5 for the structure with ductile members, 0.6 for the structure including brittle members and 0.55 for the middle. The D_s value for the building should be the maximum value for every story that is defined in Table 4.

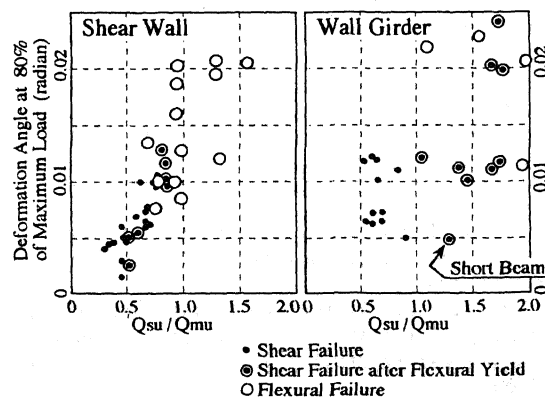


Fig. 3 Deformability of shear wall and wall girder

Table 4 Structural performance coefficient (D_s)

	in the case that only F1 members are considered for the calculation of lateral load carrying capacity	in the case that F1 and F2 members are considered for the calculation of lateral load carrying capacity	in the case that F1, F2 and F3 members are considered for the calculation of lateral load carrying capacity
D_s	0.5	0.55	0.6

The judgment whether F1, F2 or F3 is selected may be performed according to the structural conditions of members in Tables 5, 6, and 7.

Table 5 Member types of shear walls

Member Type	F1	F2	F3
Structural Conditions			
Failure Mode	Flexural Failure		Other Conditions
Upper Limit of $\bar{\tau}_u / F_m$	0.07	0.10	
Upper Limit of $\bar{\sigma}_{oc} / F_m$	0.20	0.25	

Table 6 Member types of wall girders

Member Type	F1	F2	F3
Structural Conditions			
Failure Mode	Flexural Failure		Other Conditions
Upper Limit of $\bar{\tau}_u / F_m$	0.07	-----	

Table 7 Member types of foundation beams

Member Type	F1	F2	F3
Structural Conditions			
Failure Mode	Flexural Failure		Other Conditions
Upper Limit of τ_u / F_m	0.15	0.20	

The index values in the Tables 5, 6, and 7 are calculated by Equations (5), (6), and (7).

$$\bar{\tau}_u = Q_{mu} / A_c \quad (5)$$

$$\bar{\sigma}_{oc} = (N_m + N_w) / A_c \quad (6)$$

$$\tau_u = Q_{mu} / (b \cdot j) \quad (7)$$

where

$\bar{\tau}_u$: average shear stress of the shear wall at the mechanism, (MPa)

Q_{mu} : shear force of the shear wall, the wall girder, or the foundation beam at the mechanism, (N)

A_c : sectional area of the shear wall or the wall girders in which the sectional area of the transverse walls or the slabs within the effective range may be included, (mm²)

$\bar{\sigma}_{oc}$: average axial stress of the shear wall at the mechanism, (MPa)

N_m : axial force of the shear wall at the mechanism, (N)

- N_w : axial force of the shear wall from the transverse walls, (N)
 τ_u : shear stress of the foundation beam at the mechanism, (MPa)
 b : width of the foundation beam, (mm)
 j : distance between the centroids of compressive and tensile force at the section, (mm)
 F_m : the same as that in the Equation (2), (MPa)

The member type may be classified by the factors, such as failure mode, axial force, shear-span-ratio, level of shear stress, amount of shear reinforcement, etc., that affect the deformability of the members. For the classification of member type, the factor of failure mode is stressed first, because the earthquake response of this kind of structures is rather small and the structures have the deformability of 0.005 to 0.01 radian in story drift angle if the failure mode is flexural. However it seems difficult to give enough deformability to the members under high axial stress and/or high shear stress. Therefore the limitation for them was provided.

2.8.3 Ultimate strength of members

1) Shear walls

Equation (8) should be used for flexural strength of the shear walls.

$$M_u = \Sigma(a_r \sigma_y) \cdot l_w + 0.5 \Sigma(a_w \sigma_{wy}) \cdot l_w + 0.5 N_m \cdot l_w + N_w \cdot e \quad (8)$$

where

- M_u : flexural strength of a shear wall, $M_u=0$ when it is negative, (N·mm)
 a_t : sectional area of the flexural reinforcement, The sectional area of vertical reinforcement within the effective range of the transverse walls should be included. (mm²)
 σ_y : yield strength of the flexural reinforcement, (MPa)
 l_w : 0.9 times of the length of the shear wall, (mm)
 a_w : sectional area of vertical reinforcement at the center part of the shear wall, the sectional area of vertical reinforcement within the effective range of the transverse walls may be included, (mm²)
 σ_{wy} : yield strength of the vertical reinforcement, (MPa)
 N_m, N_w : the same as those in the Equation (6), (N)
 e : distance between the compression extreme fiber of the bearing wall and the center line of the transverse wall, (mm)

Equation (9) should be used for shear strength of the shear walls.

$$Q_{su} = \left\{ \frac{0.053 p_{te}^{0.23} (F_m + 17.7)}{M / (Q \cdot l) + 0.12} + 0.846 \sqrt{p_{we} \sigma_{wy}} + 0.1 \bar{\sigma}_{oe} \right\} t_e \cdot j \cdot r \quad (9)$$

where

- Q_{su} : shear strength of a shear wall, (N)
 p_{te} : equivalent flexural reinforcement ratio, (%)

$$p_{te} = 100 \cdot a_t / (t_e \cdot l_w)$$

- a_t : sectional area of the flexural reinforcement, The sectional area of vertical reinforcement within the effective range of the transverse walls may be included. (mm²)
 t_e : equivalent thickness of the shear wall, (mm)
 $t_e = A_{we} / l$ and $t_e \leq 1.5 t$
 A_{we} : total sectional area of the shear wall including the transverse walls within the effective range, (mm²)
 l : length of the shear wall, (mm)
 l_w : 0.9 times of the length of the shear wall l , (mm)
 t : thickness of the shear wall, (mm)
 F_m : the same as those in Equation (2), (MPa)
 $M / (Q \cdot l)$: shear span ratio at the mechanism, $1 \leq M / (Q \cdot l) \leq 3$
 M : bending moment of the section at the mechanism, (N·mm)
 Q : shear force of the section at the mechanism, (N)
 p_{we} : equivalent shear reinforcement ratio,

$$p_{we} = a_w / (t_e \cdot s) \text{ and } p_{we} \leq 0.012 t / t_e$$

- a_w : sectional area of one set of the shear reinforcement, (mm²)
 s : space of the shear reinforcement, (mm)
 σ_{wy} : yield strength of the shear reinforcement, $\sigma_{wy} \leq 294 \text{ MPa}$, (MPa)
 $\bar{\sigma}_{oe}$: $(N_m + N_w) / A_w$, (MPa)
 N_m, N_w : the same as those in the Equation (6), (N)
 j : $7 \cdot l_w / 8$, (mm)
 r : reduction factor of shear capacity due to small openings

Figure 4 shows the relationship between the shear stresses, $\bar{\tau}_{max}$ ($=Q_{max} / (t \cdot l)$) obtained from the tested 45 specimens that were failed in shear or failed in shear after flexural yielding, and $c\bar{\tau}_{su}$ ($=Q_{su} / (t \cdot l)$) calculated from the Equation (9). All data are plotted above the diagonal line. The Equation (9) seems to give the lowest limit for the shear strength of the shear walls.

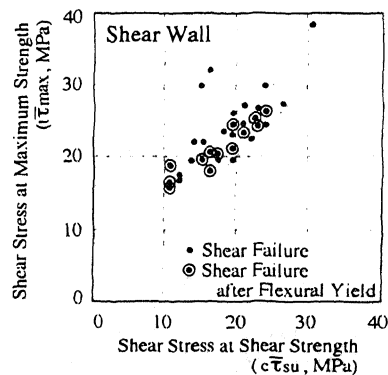


Fig. 4 Shear stresses at maximum strength and at shear strength

2) Wall girders and foundation beams

Equation (10) should be used for the flexural strength of wall girders.

$$M_u = 0.9 \Sigma(\alpha r \sigma_y d) \quad (10)$$

where

M_u : flexural strength of a wall girder, (N·mm)
 α : sectional area of the flexural reinforcement, The sectional area of reinforcement within the effective range of the slabs should be included. (mm²)

σ_y : yield strength of the flexural reinforcement, (MPa)

d : effective depth of the wall girder, (mm)

The Equation (11) should be used for shear strength of the wall girders.

$$Q_{su} = 0.9 \left\{ \frac{0.053 p_{te}^{0.23} (F_m + 17.7)}{M / (Q \cdot d) + 0.12} + 0.846 \sqrt{p_{we} \sigma_{wy}} \right\} b_e \cdot j \quad (11)$$

where

Q_{su} : shear strength of a wall girder, (N)
 p_{te} : equivalent flexural reinforcement ratio, (%)

$$p_{te} = 100 \Sigma \alpha_t / (b_e \cdot d)$$

α : sectional area of the flexural reinforcement, The sectional area of reinforcement within the effective range of the slabs may be included. (mm²)

b_e : equivalent width of the wall girder, (mm)

$$b_e = A_{ge} / D \text{ and } b_e \leq 1.5 b$$

A_{ge} : total sectional area of the wall girder including the slabs within the effective range, (mm²)

D : depth of the wall girder, (mm)

b : width of the wall girder, (mm)

d : effective depth of the wall girder, (mm)

F_m : the same as that in the Equation (2), (MPa)

$M/(Q \cdot d)$: shear span ratio at the mechanism, $1 \leq M/(Q \cdot d) \leq 3$

p_{we} : equivalent shear reinforcement ratio,

$$p_{we} = a_w / (b_e \cdot s) \text{ and } p_{we} \leq 0.012 b / b_e$$

j : $7 \cdot d / 8$ (mm)

$M, Q, \sigma_{wy}, a_w, s$: the same as those in the Equation (9)

Flexural strength of the foundation beams may be calculated by the Equation (10). Shear strength of the foundation beams may be calculated by Equation (11), in which the reduction factor of 0.9 may be changed to 1.0.

2.8.5 Ductility design of members

1) Shear wall

To assure the flexural yield failure, the shear strength of members should be greater than the shear force at the mechanism. Flexural strength does not decrease so much under repeating load, but shear strength decreases under repeating load in plastic range especially under

high axial force. If the value $\bar{\sigma}_{oc}/F_m$ is greater than 0.25, the shear walls may be the member type of F3 which ductility is not expected. For the F1 and F2 member types of shear walls, the ratio of the shear strength Q_{su} and the shear force at the mechanism Q_{mu} should be larger than the value in Table 8. For the shear walls with transverse walls, the values in Table 8 may be decreased by 0.1, because the shear strength of the shear walls with transverse walls is a little greater than the calculated one and the ductility may be expected.

Table 8 Lowest limit of Q_{su}/Q_{mu} for shear walls

	F1	F2
$\bar{\sigma}_{oc}/F_m \leq 0.10$	1.20	1.10
$0.10 < \bar{\sigma}_{oc}/F_m \leq 0.15$	1.25	1.10
$0.15 < \bar{\sigma}_{oc}/F_m \leq 0.20$	1.30	1.15
$0.20 < \bar{\sigma}_{oc}/F_m \leq 0.25$	----	1.20

2) Wall girders and foundation beams

The shear strength of the wall girders and the foundation beams of which member type is F1 should satisfy Equation (12).

$$Q_{su} > Q_L + \alpha \cdot Q_{mu} \quad (12)$$

where,

Q_{su} : shear strength of a wall girder and a foundation beam, (N)

Q_L : shear force under the long term load, (N)

α : coefficient to ensure the ductility, which should be equal to or larger than 1.1.

Q_{mu} : shear force at the mechanism, (N)

3 CONCLUSION

New design guidelines for medium rise RM buildings were released in March 1989 based on the composite experimental and analytical research that had been carried out since 1984. Ductility design concept as well as strength design concept was employed in the new design guidelines.

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