AIJ Standard for Structural Design of Reinforced Concrete Boxed-Shaped Wall Structures

Architectural Institute of Japan

Preface

Reinforced Concrete Box-shaped Wall Structures

This is the English version of the latest Standard of Architectural Institute of Japan (AIJ) for Structural Design of Reinforced Concrete Box-shaped Wall Buildings published in 1997. The main objective of publishing this edition is to introduce the standard Japanese structural design methods of reinforced concrete box-shaped wall structures to foreign countries.

The residential buildings commonly require wall elements such as exterior blind walls and interior partition walls between room units although those wall elements are generally disliked in office buildings. The reinforced concrete box-shaped wall structure is the structural system mostly used for residential apartment houses in which such wall elements are structurally designed to resist to both vertical and lateral loads. There is no column in the buildings and the width of every beam is the same as the thickness of adjacent wall. Therefore, these structures look like box-shaped buildings consisting of wall elements, wall elements with openings and slabs.

After World War Ⅱ, Japanese Government was urged to provide a huge amount of houses for people. Mass production, fire-resistance, durability, economical superiority and excellent seismic resistance are the most important keys for those houses. A large number of reinforced concrete box-shaped apartment houses have been constructed since 1940s, because they met such demands.

The first edition of the AIJ Structural Design of Reinforced Concrete Box-shaped Wall Structure was published in 1952 as a part of the “AIJ Structural Design Standard for Special Concrete and Masonry Structures.” The number of stories of buildings was four at the highest then. Major research works have been done based on seismic tests of full-scale building structures. Since then, the Standard has been revised four times based on the research progress and long-term experiences, and the number of stories was increased to five.

The structural design of reinforced concrete box-shaped wall buildings is relatively simple. The checks under structural provisions concerning wall length per unit floor area, amount of reinforcing steel bars and the like are essential in the structural design.

Then, the stresses in the members are calculated by the so-called “mean shear stress method” and the simplified allowable stress design is conducted.

The structural performance, particularly seismic one should be satisfactorily excellent. It has been reported that any severe damage, such as concrete crashing and remarkable crackings in the walls, were not produced to buildings, which were constructed in accordance with this AIJ Standard, when they experienced severe earthquake whose peak ground acceleration was greater than $2m/s^2$. For example, due to the 1995 Hyogo-ken Nambu Earthquake (Kobe Earthquake) in Japan, no significant structural damages were observed in reinforced concrete box-shaped wall buildings, meanwhile destructive damages to a large number of buildings by other structural systems were caused.

Finally, we hope that this standard, the design methodology and design procedures may highly contribute to the spread of the earthquake resistant buildings in the world.

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Steering Committee for Wall Construction
Architectural Institute of Japan

Contents

Introduction

The structural features of reinforced concrete box-shaped wall buildings and the background of structural specification

1. The Background of Reinforced Concrete Box-shaped Wall Structures

Reinforced Concrete Box-shaped Wall Structures (hereinafter, abbreviated to WRC) consist of bearing walls, wall beams, slabs, footing beams and foundations. These structures, which have excellent seismic performance and fire-resistance and are low priced construction compared to frame structures, were developed after World War II in order to reconstruct cities in Japan where a huge number of wooden houses had been burnt out. The Japanese Building Standard Law and its Enforcement Order prescribe the structural specifications of the buildings having five-story or less, and 16m or less in the maximum height (amended to 20m or less on June 25, 2001) as WRC structures, and this type of structure is one of the most popular buildings for residential apartment houses in Japan.

2. Structural Characteristics

The structural features of WRC structures are as follows;

・High seismic performance (according to the damage of the past earthquakes, the damage ratio of WRC structures is much smaller than that of other types of structures.)

・Fire resistance (the performance is as good as that of RC structures.)

・Economical superiority (bearing walls are as thick as wall girders.)

3. The Targeted Structural Performance and the Background of Design Standard

The required structural performance of this structure is as follows and the structural standard is established to meet these required structural performance.

i) For moderate earthquake motions, buildings should not produce crackings on bearing walls.

ii) For extremely large earthquake motions, buildings should not collapse or fall.

The followings are the counter measures to realize the above requirements.

1) The Prevention of Developing the Shear Cracks on Bearing Walls During Moderate Earthquake Motions

The shear stress intensity in bearing walls during moderate earthquake motions on every story and in every direction should be less than temporary loading allowable shear stress of concrete being used, in order not to produce cracks on the bearing walls.

Seismic shear force on every story and in every direction during moderate earthquake motions is calculated for the standard shear force factor Co=0.2) which is prescribed in Enforcement Order.

The forces and moments are calculated by the so called “mean shear stress method”. Namely, they are calculated by the mean shear stress of the bearing walls given by dividing the story shear force by total wall areas in every direction of the story considered, and the reflection point is assumed to be middle of clear height of each wall.

2) The Prevention of the Collapse of Buildings during Extremely Large Earthquake Motions

The design story shear force prescribed in Enforcement Order during extremely large earthquake motions (standard shear force factor Co =1.0) is five times as much as that during moderate earthquake motions (standard shear force factor Co =0.2). However, this magnitude is reduced, considering the ductility of structures. Enforcement Order defines such reduction value as structural factor (Ds) and the value of Ds is about 0.5 for this type of structures. Finally, for example, this magnitude for the first story almost corresponds to half of the total weight of a building.

In order to secure the structural safety against for such story shear, some structural specifications are prescribed in the structural design. The minimum wall length per unit floor area as well as the minimum wall thickness in order to control the shearing stress of the wall during the extremely large earthquake motions is one of such important specifications. Also steel bar arrangement specifications and bearing wall arrangement/configuration, etc. are very important specifications to secure structural safety. The mean shear force of walls based on such requirement as minimum wall length ratio is about less than 1 N/mm^2 for extremely large earthquake motions.

The seismic structural characteristics was previously experimented with three specimens of full-scale five story WRC. The ultimate mean shear stress of bearing walls of the experimental result of five story three-dimensional specimens was about 2 N/mm^2 when the maximum force were loaded.

Any severe damage has not been investigated in structures designed in accordance with this AIJ Structural Standard, even after severe earthquakes including the 1995 Hyogo-ken Nambu Earthquake.

Article 1. General

1. This Standard described herein can be applied to the structural design of buildings, in which their load bearing systems consist of reinforced concrete (hereafter abbreviated as RC) load bearing walls. The RC load bearing walls should carry both vertical loads (such as those of dead, live and other loads) and lateral loads such as those generated by earthquake excitation. RC buildings, to which this Standard can be applied, are hereinafter referred as RC box-shaped wall buildings. This Standard can be applied to the structural design of parts of RC box-shaped wall structural assemblages within a building.
2. For the parts of RC construction, which are not covered by this Standard, the specifications described in the "Standard for Structural Calculation of Reinforced Concrete Structures" published by the Architectural Institute of Japan (hereafter referred as *AIJ RC Calculation Standard*) should be applied.
3. For buildings designed based upon special studies, a part and/or parts of articles of this Standard need not to be applied provided that the structural performance of the building is revealed equal to or better than that for the building designed in accordance with the specifications described in the Standard.

1. Outline of the Standard

In structural design of residential buildings, load bearing walls can be used as the main structural system within the building. In a frame structure, however, reinforced concrete (hereafter abbreviated as RC) wall elements (such as wing walls, hanging walls and spandrel walls attached to either columns or beams) are not usually considered as structural components. The strength contributed by those elements, if any, is not usually taken into account in structural design.

For low-rise buildings, a new design methodology is developed, for which the dimensions of both column and beam elements are taken similar to the thickness of wall elements and the structural properties of the wall elements are positively taken into account in structural design. Since the dimensions of the column and beam elements are taken similar to those of wall elements, plane box-shaped wall elements can be used as structural components, leading to a simple and clear room space within the building. The above-mentioned properties are those that characterize the so-called box-shaped wall structural system described in this Standard. The structural performance of RC box-shaped wall buildings is excellent, particularly against the lateral loads generated by seismic action. It was reported that damage, such as severe crackings in wall panels, was not produced to residential apartment buildings of the RC box-shaped wall structures during intense seismic action, in which the peak ground acceleration of greater than $2m/s.^2$ was obtained. After the 1995 Hyogo-ken Nanbu Earthquake in Kobe, Japan, producing a large number of damaged buildings, no significant structural damage was observed in the RC box-shaped wall apartment houses in and around the intense seismic areas. The observation upon the damage revealed the excellent seismic performance of RC box-shaped wall buildings specified in the articles described hereinafter.

The following experimental studies on the RC box-shaped wall structures were conducted.

(1) Study No. 1: lateral loading test on a full-scaled 5-story RC box-shaped wall apartment house with regular wall arrangement along its longitudinal direction.

(2) Study No. 2: lateral loading test on a two-thirds scaled RC box-shaped wall apartment house with regular wall arrangement along its longitudinal direction. Three specimens were prepared, representing the lower two stories of a 5-story building.

Based on these experimental studies, it is concluded that RC box-shaped wall buildings designed according to the specifications of this Standard have sufficient ultimate lateral load carrying capacity against a seismic action of peak ground acceleration of 2 to 3m/s^2. The level of peak acceleration described above corresponds to that defined as the maximum credible earthquake motion during the building service life in "Design Earthquake Load (Draft in 1976, Architectural Institute of Japan)" and "Enforcement Order (amended in 1980 and enforced in 1981)" of the Building Standard Law of Japan.

A large number of RC box-shaped wall apartment houses were constructed in years of 1940s to 1960s providing living spaces in Japan. One of the possible reasons for the mass production of the RC box-shaped wall apartment houses is its economic efficiency. At a moderate cost, a large number of houses could be constructed. After World War II, Japanese Government was urged to provide a large amount of houses for people. At that time, the RC box-shaped wall apartment houses were commonly constructed and simply called "RC apartment houses."

In Notification No. 1790 of Ministry of Construction, which was reorganized as Ministry of Land, Infrastructure and Transport in January, 2001, Japanese Government (issued on November 27, 1980 and amended in 2001), simplified structural calculation root was specified to the buildings meeting the following criteria:

(1) An RC building, whose height is not greater than 20 in meter, and

(2) A building in which the cross-sectional areas of columns and walls *Ac* and *Aw* of the building should satisfy the condition in Equation (1.1)[1].

For such buildings, it is not necessary to evaluate the ultimate strength properties of building in the so-called phase II seismic design in Japanese practices. When the condition shown in Eq. (1.1) is satisfied, it can be verified that the condition required in Building Standard Law is fulfilled without further detailed examination of the condition that the building should have a sufficient level of seismic performance against the maximum possible seismic load.

$$\sum 2.5A_w + \sum 0.7A_c \geq \sum ZWA_i \qquad (1.1)$$

[1] The expression of Eq. (1.1) was modified in August 2001. The design concrete strength *Fc* was taken into the consideration within Eq. (1.1). Within this edition, however, Eq. (1.1) in the previous expression is employed.

where,

Aw: horizontal cross-sectional area of a load bearing wall in each story level placed in the longitudinal or transverse direction under consideration of building in mm^2,

Ac: horizontal cross-sectional area of a column or a wall that is not regarded as a load bearing wall in each story level in the longitudinal or transverse direction under consideration of building in mm^2,

Z: seismic zone factor specified in Clause 1, Article 88 of Enforcement Order, Building Standard Law that specifies the seismicity in Japan taking from 0.7 to 1.0;

W: sum of the dead and live loads supported by the story level under examination that can be obtained in Enforcement Orders in Japanese practice in Newton (N) , and

Ai: factor that specifies the shear force distribution along the building elevation, which can be evaluated in Notification No. 1793, Ministry of Land, Infrastructure and Transport.

RC box-shaped wall buildings designed in accordance with the specifications of the Standard satisfy Eq. (1.1), since they generally have a sufficiently large cross-sectional area of $\sum Aw$ in Equation (1.1).

In order to keep the design process simple without the need of sophisticated analytical tools, the Standard should be applied to simple structural systems. This Standard is not applicable to the following cases,

(1) Buildings having an irregular shape, whose plan cannot be well modeled in a rectangular shape,

(2) Buildings with excessively large live load compared to that of an ordinary dwelling house, and those whose average weight of building per unit area w_i is greater than $1.2\text{x}10^4 \text{N/m}^2$,

(3) Buildings in which a rigid frame system consisting of columns and beams is included within a box-shaped wall system, and

(4) Buildings having a story in which the wall area ratio defined by $\sum A_w / S_i$ is fairly small compared to those evaluated for other stories, where $\sum A_w$ is sum of horizontal areas of the bearing walls in the direction under consideration at the i-th story level, and S_i is the floor area at the i-th story level.

2. Correlative Relations to the “Standard for Structural Calculation of Reinforced Concrete Structures, AIJ”

Since the constituent components of RC box-shaped wall buildings are essentially made of reinforced concrete, the structural design for RC box-shaped wall buildings, in principle, should be performed in accordance with the Standard for Structural Calculation of Reinforced Concrete Structures (AIJ RC Calculation Standard). Not all specifications necessary for the RC box-shaped wall buildings are included in AIJ RC Calculation Standard. Therefore, the Standard herein is prepared to specify the articles necessary for the design of RC box-shaped wall buildings. The specifications required for design that are not found in this Standard should be referred to those described in AIJ RC Calculation Standard.

3. Special Studies Concerned with the Standard

For the RC box-shaped wall system, a large number of issues concerned with structural properties have remained unevaluated. Therefore, some articles of this Standard might be somehow conservative. Buildings designed based on special studies and investigations is exempted from a part of specifications described in the Standard, provided that the structural properties obtained from the special studies are equivalent to or higher than those specified in the Standard.

Article 2. Quality of Materials -Concrete and Reinforcing Bar-

1. The concrete used for the RC box-shaped wall buildings should be in accordance with the specification described in the Article 3 of AIJ Standard for Structural Calculation of Reinforced Concrete Structures. The design strength of concrete should be equal to or greater than 18N/mm^2.
2. The reinforcing steel bars used for the RC box-shaped wall buildings should have qualities equal to or higher than those of SD295A specified in JIS (*Japan Industrial Standard*) G3112, which specifies the qualities of reinforcing steel bars for reinforced concrete structural systems.

1. Quality of Concrete

The design principle of the Standard is to keep the human lives safe against the possible maximum earthquake ground motion that a building would experience during its service life. If a brittle shear failure occurs in a load bearing wall that supports the dead and live loads, an undesirable collapse mechanism might be produced which can cause the loss of human lives. The Standard, therefore, provides the minimum requirements to prevent brittle shear failures of load bearing walls subjected to the specified earthquake ground motion.

The load bearing walls of RC box-shaped wall buildings have neither columns nor beams placed around the wall panel to confine the development of shear. Once a shear cracking is generated, the shear force bearing capacity is drastically decreased unless a significant amount of shear reinforcement bars in wall panels are arranged. If shear crackings are generated in the load bearing wall, other load bearing walls would be damaged due to redistribution of shear force. Shear crackings generated in the load bearing walls supporting the vertical loads are hard to be repaired.

Based on the above facts, RC box-shaped wall buildings should be designed to remain in the elastic range for seismic loads. The level of seismic load considered herein is that of the building might encounter several times during its service life, whose peak ground acceleration is 80 to 100cm/s^2 under the seismic circumstances in Japan.

During the 1978 Miyagi-ken Oki Earthquake in Japan with the peak ground acceleration of about 2m/s^2 in the affected areas, no significant shear crackings were observed in wall panels according to the reconnaissance investigation on the 131 RC box-shaped wall apartment houses, which had been designed in accordance with this Standard in Sendai City, Miyagi Prefecture, conducted by Building Research Institute, Independent Administrative Institution of Ministry of Land, Infrastructure and Transport.

The Standard specifies the minimum design concrete strength F_C as 18N/mm^2 for both normal and lightweight concretes. The maximum design concrete strength is not specified in the Standard. However, it should be noticed that the design formulae proposed and utilized in this Standard have been examined their appropriateness for normal and lightweight concretes with design strength not in excess of 36N/mm^2 and 27N/mm^2, respectively. High water-cement ratio for low strength concrete may cause drying shrinkage crackings. From a point of waterproofness, it should be avoided to use concrete with strength lower than 18n/mm^2 for the RC box-shaped wall buildings composed of thin wall members.

If lightweight concrete is used for the underground structures of buildings such as basement walls, foundations and others, which directly contact with moist soil or water, JASS 5(Article 10.2) should be referred for the minimum protective concrete cover and other probable issues.

2. Quality of Reinforcing Bar

The deformed steel bars SD295A specified in JIS G 3112 (Reinforcing bars for reinforced concrete) should be used. The other bars equivalent to the above specification can be used. While the welded wire mesh having a wire diameter of 4 mm or greater can be used for reinforced concrete buildings, it cannot be used for an RC box-shaped wall buildings, because of the lack of experimental studies for an RC box-shaped wall system.

The welded wire mesh having a wire diameter of 6mm or greater, however, can be used for floor slabs exclusively, in accordance with Article 13 of the AIJ RC Calculation Standard.

Article 3. Height and Weight of Building

1. The number of stories of an RC box-shaped wall building designed by the Standard should be equal to or less than five. The maximum height of the building should be 16m or less, and the maximum story height should be 3m or less as well. At the top story, the maximum story height can be 3.3m or less.

2. At each story level within the building, the ratio of the average weight of the building divided by the floor area should be $1.2 \times 10^4 N/m^2$ or less. The average weight of the building calculated at the i-th story level is given by: (1) first taking the sum of weights of the i-th and upper stories; and (2) secondly dividing the obtained total weight by the number of floor slabs positioned above the i-th story level. Weight of the building is determined from that used for the calculation of the seismic load.

1. Number of Stories and Height of Building

As described in the commentary for Article 1, both the lateral load and deformation capacity of 5-story RC box-shaped wall buildings having regular shape were verified by seismic damage surveys on buildings and experimental studies either on a full-scale three dimensional test specimen or two-thirds scaled two-dimensional test specimens. These studies indicated that the RC box-shaped wall buildings have a sufficient level of seismic performance against earthquake ground motions having peak acceleration of about 300 to $400 cm/s^2$, which are considered as a possible maximum earthquake ground motion during the service life of building.

For medium-rise buildings with six stories or more, a certain amount of ductility capacities shall be required along with lateral load carrying capacities required in the guidelines and/or specification examined in the following. In “Building Structural Design Guidelines for RC box-shaped Wall Buildings with Six through Eight Stories (January 1980)” issued by Urban Development Cooperation (hereafter UDC) in Japan and approved by Ministry of Land, Infrastructure and Transport, Japanese Government, specifications for ensuring ductility of constituent members are prescribed: (1) in the longitudinal direction of building along which the wall area ratio (mm^2/m^2) is small in general cases, wall thickness should be increased to obtain sufficient amounts of both ductility and strength; (2) sufficient amount of stirrups should be provided for wall girders and/or beams; and (3) the failure mode of building should be ductile flexural yielding mode, and brittle shear failure mode should be avoided.

The Standard herein exclusively covers RC box-shaped wall buildings having a regular shape with five stories or less. For these buildings, no specific prescriptions are required to ensure ductility capacity of building. The specifications in the following are prescribed for apartment houses widely designed in Japan: (1) the height of building (height to the eaves), should be 16m or less; (2) the height of each story including the basement, if any, should be 3m or less; and (3) the height of the top story can be increased up to 3.3m for ventilation and/or thermal insulation and other utilities.

Special consideration should be taken for parapets placed on roof floor slabs. Large acceleration responses are generated during a strong ground motion, since the lateral stiffness of a parapet in the out-of-plane is generally less than that of the building itself. Broken fragments of a damaged parapet might cause serious injuries and possible loss of human lives. It is not desirable to place unnecessarily high and large parapets. Rigid care should be paid to the design and construction work as well.

According to Japanese regulations, a penthouse structure, whose floor area is one eighth or less of that of the top floor and whose height is 3.3m or less, is not counted for a story of building. For a building with the penthouse described above, the number of the story of the building is regarded as that excluding the penthouse structure.

In Japanese regulations, for a building whose story is 3m or higher, there is a specification that structural safety of the building should be verified by either structural calculation/analysis or experimental studies. The Standard for RC box-shaped wall buildings herein allows the story height of 3.3m at maximum for the top story. If the roof has a slope, the distance from the floor slab level of the top story to the highest point of sloped roof should be 4m or less.

2. Weight of Building for Evaluation of Seismic Loads

This Standard allows it possible to design structures of RC box-shaped wall buildings in a simplified manner. This Standard herein should not be applied for some special buildings such as those having a large-scale plan, those having excessively large weight, and those for special use with either irregular configuration or specific weight. The average weight of building per unit floor area can be used for simplified structural calculation. The average weight per floor area defined herein indicates the seismic load per unit floor area at the story level concerned. The average weight defined hereby can be applied to all the stories of building.

The process of evaluation of the average weight per unit floor area is indicated as follows. An average weight of building is evaluated by the weight of building Wi, the total floor area Si at the story level i, and the number of floor slabs positioned above the i-th story level as indicated in Commentary Table 3.1. Even if the weight per unit floor area is small at the story level i concerned, the lateral load generated during earthquake motions at the story level i may become large, if the average weight per unit area of the building is large.

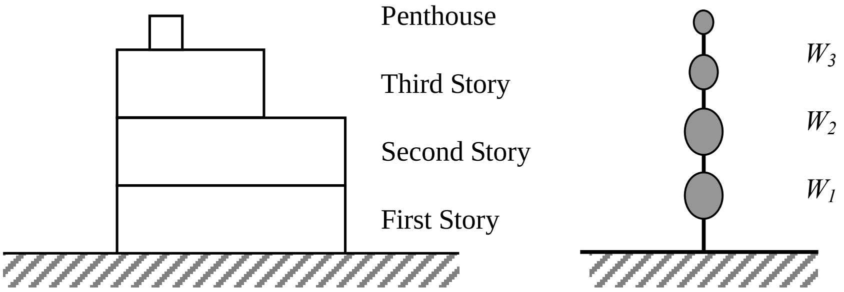
C. Table 3.1 Average weight per unit floor area w_i (N/mm^2) of story i

for lateral load evaluation: A three-story building.

(In case where the floor area of a penthouse structure is no more than one-eighth of that of the top floor and height is less than 3.3m)

Story Level	Weight of Story i	Weight at Story Level i	Floor Area of Story Level i	Average Weight w_i
3	W_3	W_3	S_3	$w_3 = W_3/S_3$
2	W_2	W_2+W_3	S_2	$w_2 = 1/2\ (W_2+W_3)/S_2$
1	W_1	$W_1+W_2+W_3$	S_1	$w_1 = 1/3\ (W_1+W_2+W_3)/S_1$

Penthouse is not included in the number of stories of building



The weight of penthouse is included in the weight of the third story W_3.

Article 4   Arrangement and Wall Length Ratio of Bearing Walls

1. The bearing walls should be arranged in good balance on the floor plans of the building.
2. In both transverse and longitudinal directions in each story, the value derived from dividing the total length of bearing walls (in mm) prescribed in Article 5 by the floor area of that story (in m^2) (hereafter the wall length ratio) should be more than the value in Table 1. Here, if the upper story has a balcony or the like, half or more of its area should be added. The floor area used for calculation of the wall length ratio should be equal to or larger than the respective values given in Table 1. If the thickness t of the bearing wall is greater than the thickness t_0 shown in Article 5, the wall length rate is decreased proportionally by the ratio of t/t_0. However, the actual total length (not increased proportionally) (in mm) of the bearing walls divided by the floor area (in m^2) for calculation of the wall length ratio of that story should be equal to or greater than the respective values given in Table 1 minus 30 mm/m^2.

Table 1   The required Wall length ratio

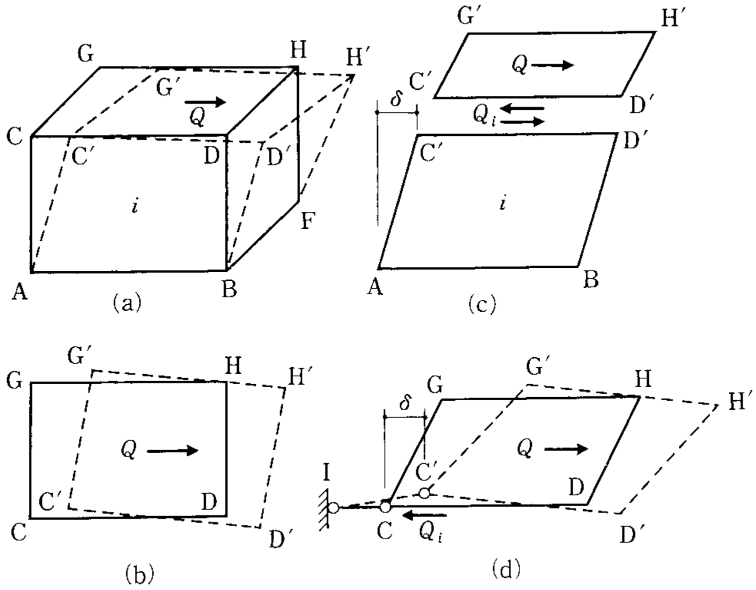
Story		Wall length ratio (mm/m^2)
Stories above the ground	Single-storied building or story levels with or higher than the third story counted down from the top story	120
	Story levels with or lower than the fourth story counted down from the top story	150
Basement floors		200

1. Arrangement of Bearing Wall

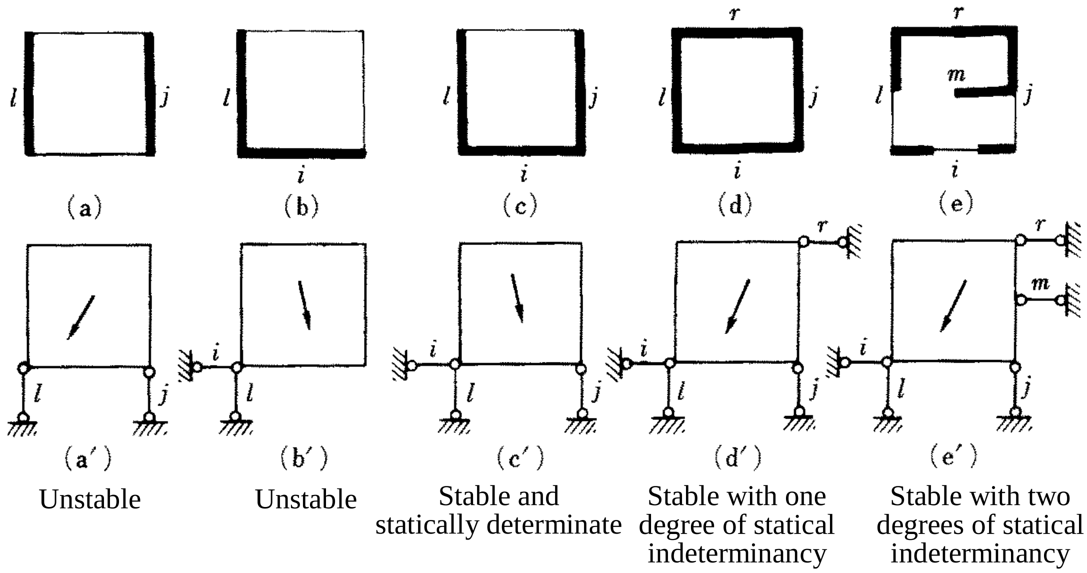
An appropriate arrangement planning of bearing walls is the first step to a sound design of an RC box-shaped wall building. Basic matters about floor and elevation planning are given below as a guideline in the design of an RC box-shaped wall building.

(a) Floor Planning of Bearing Walls

Commentary Figure 4.1(a) shows a box-shaped building in a deformed state subjected to earthquake load Q at the roof slab. CDHG, ABDC, BFHD, etc. represent bearing walls. Commentary Figure 4.1 (C) shows the force transferred from the roof slab in relation to deformation, that is, a bearing wall deformed under a lateral load. From the point of view of the roof slab, the resultant of lateral loads Q_i transferred from the bearing walls is in equilibrium with the earthquake load Q. Supposing that the bearing wall has no resistance to out-of-plane forces, the force Q_i in Commentary Figure 4.1 (c) is parallel to AB and proportional to the displacement δ. Therefore, as shown in Commentary Figure 4.1 (d), the bearing wall i can be replaced by an elastic member IC in which reaction Q_i develops against δ.



C. Fig.4.1 Behavior of rigid floor CDHG of a building having a bearing wall *i* on the plane of structure ABDC, and an elastic member IC supposed to support the rigid floor for representing this behavior

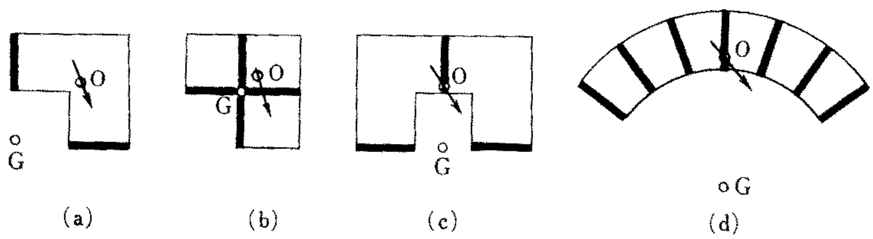


C. Fig. 4.2 The behavior of the rigid floor supported by bearing walls *i, j, l, r* and *m*, elastic members *i, j, l, r* and *m*

If the bearing walls are modeled to elastic members, the problem is reduced to a problem of a rigid floor (or roof slab) supported by elastic members which are easier to treat. Commentary Figures 4.2(a) to (e) show the plans of buildings, the bold lines representing bearing walls.

A building having two bearing walls *l* and *j* as shown in Commentary Figure 4.2 (a) can be replaced by a rigid floor supported by elastic members *l* and *j* shown in Commentary Figure 4.2 (a'). Needless to say, such a structure is unstable. The arrangement shown in Commentary Figure 4.2 (b) is also unstable since the roof slab can rotate about the intersection of the bearing walls *l* and *j*, and no horizontal resistance of the bearing walls can be expected. With the arrangement in Commentary Figure 4.2 (c), the horizontal resistance of the bearing walls can be expected. The arrangement shown in Commentary Figure 4.2 (c') is statically determinate and the lateral force acting on each elastic member is independent of the rigidity of the elastic member (bearing wall) and is determined from the equilibrium of forces. If bearing walls are arranged as shown in Commentary Figure 4.2 (d) and (e), the share in lateral loads of the bearing walls are dependent on the rigidity of the bearing walls and can be derived from solving the replaced statically indeterminate trusses.

It is obvious that the arrangement of bearing walls in Commentary Figures 4.2 (a) and (b) are unstable. It should be noted, however, that the wall arrangements shown in Commentary Figure 4.3 are all unstable. Point G in the figures is the center of rotation, that is, the center of rigidity.



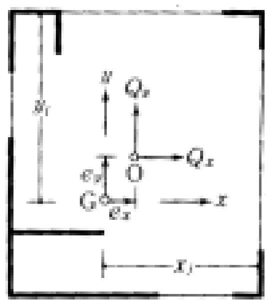
[Note] The point “O” is the center of gravity

C. Fig.4.3 Examples of unstable bearing wall arrangement

As can be easily seen from the above figures, the necessary condition of a stable wall arrangement is that there are two or more intersections of wall lines (including extensions) of bearing walls in the plane.

Apart from the unstable structures given above, torsion occurs in the building unless the bearing walls are arranged in good balance. Therefore, the roof slab and floor slabs (hereafter called the floor slabs, generically) move and rotate around the center of rigidity G, and variations in magnitude of shear forces occur in the bearing walls apart from the center of rotation (center of rigidity) G. The design should consider those increases in shear forces. Commentary Figure 4.3 shows an example which would rotate. In this case, the design shear forces of the bearing walls in the planes of structure Y_i in the x-direction and the planes of structure X_j in the y-direction at distances y_i and x_i respectively from the center of rigidity G are derived from multiplying the values of shear force occurring when the rotation of the floor slab is ignored by the respective modification factors α_{xi} and α_{yj} given by Equations (4.1-a and -b).

Planes of structure Y_i ($D_{A,xi}$, D_{xi})



Planes of structure X_i ($D_{A,yj}$, D_{yj})

C. Fig.4.4 Symbols of planes of structure in *x*- and *y*-directions and symbols of D_A and *D* of the plane of structure

$$\alpha_{xi} = 1 + R_{ex} \frac{y_i}{r_{ex}} = 1 + \frac{e_y}{r_{ex}} \cdot \frac{y_i}{r_{ex}} \tag{4.1-a}$$

$$\alpha_{yj} = 1 + R_{ey} \frac{x_j}{r_{ey}} = 1 + \frac{e_x}{r_{ey}} \cdot \frac{x_j}{r_{ey}} \tag{4.1-b}$$

where,

R_{ex} and R_{ey} : eccentric ratio of the story concerned in the directions *x* and *y*.

$$R_{ex} = \frac{e_y}{r_{ex}} \tag{4.2-a}$$

$$R_{ey} = \frac{e_x}{r_{ey}} \tag{4.2-b}$$

where,

e_y and e_x : distances in mm between the center of gravity O of the weight of building *W* on the story concerned and the center of rigidity G on the story concerned, respectively,

r_{ex} and r_{ey} : spring radius in the calculation directions *x* and *y* .

$$r_{ex} = \sqrt{\frac{K_T}{K_x}} \fallingdotseq \sqrt{\frac{\sum D_{xi} \cdot {y_i}^2 + \sum D_{yi} \cdot {x_j}^2}{\sum D_{xi}}} \tag{4.3-a}$$

$$r_{ey} = \sqrt{\frac{K_T}{K_y}} \neq \sqrt{\frac{\sum D_{xi} \cdot y_i^{\ 2} + \sum D_{yj} \cdot x_j^{\ 2}}{\sum D_{yj}}} \tag{4.3-b}$$

Since out-of-plane horizontal rigidity of bearing walls is much less than the in-plane horizontal rigidity, K_T, K_x and K_y are given by Equations (4.4) and (4.5-a and -b).

Where,

K_T : torsional rigidity (N/mm) about the center of rigidity G on the story under consideration.

$$K_T \overset{\fallingdotseq}{\neq} \sum \mathrm{D}_{\mathrm{A,xi}} \cdot y_i^{\ 2} + \sum \mathrm{D}_{\mathrm{A,yj}} \cdot x_j^{\ 2} \tag{4.4}$$

where,

K_x and K_y : horizontal rigidities (N/mm) in the calculation directions x and y on the story under consideration.

$$\mathrm{K}_{\mathrm{x}} \overset{\fallingdotseq}{\neq} \sum \mathrm{D}_{\mathrm{A,xi}} \tag{4.5-a}$$

$$\mathrm{K}_{\mathrm{y}} \overset{\fallingdotseq}{\neq} \sum \mathrm{D}_{\mathrm{A,yi}} \tag{4.5-b}$$

Where,

$D_{A,\ xi}$ and $D_{A,\ yj}$: story shear forces (N/mm) in the plane of structure necessary to bring about a unit relative story lateral displacement (mm) in the planes of structure Y_i in the x-direction and in the planes of structure X_j in the y-direction respectively at distances y_i and x_j respectively from the center of rigidity G,

D_{xi} and D_{yj} : value $D_{A,\ xi}$ or $D_{A,\ yj}$ on each plane of structure normalized by a standard value D_{A0}, that is, the coefficients of shear force distribution factor in the planes of structure Y_i in the x-direction and the planes of structure X_j in the y-direction respectively at distances y_i and x_j from the center of rigidity G,

y_i and x_j : distances of planes of structure Y_i in the x-direction and planes of structure X_j in the y-direction from the center of rigidity G, taken positive if the plane of structure lies on the same side as the center of gravity O of W on the story under consideration (see Commentary Figure 4.4).

To lessen the increase of the shear force in a bearing wall due to the rotation of the floor slab, the values of α_{xi} and α_{yj} given by Equations (4.1-a and -b) must be close to unity . For that:

i) reducing the distances e_y and e_x in the y- and x-directions from the center of gravity O to the center of rigidity G of W, and

ii) increasing r_{ex} and r_{ey} by increasing the torsional rigidity K_T about the center of rigidity.

Article 82-3 of Enforcement Order lays down for specified buildings with 31 m high or lower that R_{ex} and R_{ey} should satisfy the conditions of Equations (4.6-a and -b) on every story unless verifying that the horizontal load-carrying capacity of each story is equal to or higher than the necessary horizontal load-carrying capacity of the story under consideration.

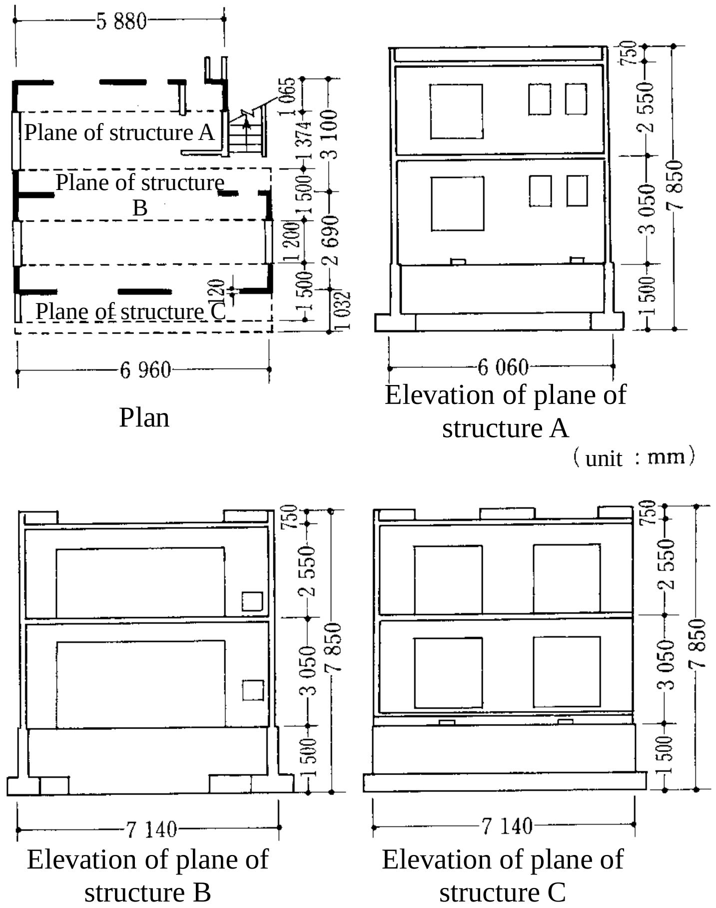
$$R_{ex} \leq 0.15 \tag{4.6-a}$$

$$R_{ey} \leq 0.15 \tag{4.6-b}$$

In the case of a building with the RC box-shaped wall structures in the x-direction and moment resisting frame structures in the y-direction, or other building with dissimilar types of structural systems in each direction**,** the value of K_T becomes small, and coefficients α_{xi} and α_{yi} become large in case where the distances e_y and e_x are large. Such buildings are excluded from the scope of application of this Standard (see the Commentaries on Article 1.).

When a building is irregular-shaped and remarkably different from the concept of rectangular in a plan, even if the bearing walls are arranged in a good balance in each of the portions into which the plan is appropriately divided, large boundary stresses are developed in the divided portions and these boundary stresses cause each portion to be distorted individually unless the horizontal story drift in the calculation direction is equal in the all portions. Since it is not easy to obtain the exact behavior of each portion during an earthquake, such a building is excluded from the scope of application of this Standard (see the Commentaries on Article 1.).

Unevenness in the rigidity due to different aspect ratios of the bearing walls and local defects in the bearing walls may produce local excessive stresses and cause cracks to develop in the bearing walls. However, they tend to be resolved by redistribution of stresses if the bearing wall is appropriately reinforced. On the other hand, local excessive imposition of shear stresses in the bearing wall due to an unbalanced bearing wall arrangement tends to advance**,** because the distance between the center of rigidity G and the center of gravity O of W generally increases once cracks ascribable to it occur in the bearing wall. Therefore, proper care must be taken in floor planning regarding to bearing wall arrangement.



C. Fig. 4.5 Plan of a 5-story apartment house of RC box-shaped wall structure "Type 63-5N-2DK-4" designed by UDC and elevations of a test piece simulating lower two stories of its planes of structure in the longitudinal direction

Commentary Figure 4.5 shows a 5-story apartment house of RC box-shaped wall structure which was designed by UDC and its planes of structures A, B and C of the first and second stories in the longitudinal direction on a reduced scale of 2/3 (the width of floor slabs connected to the planes of structures A, B and C being 1,065, 1,500 and 1,500 mm). On this structure, a lateral loading test was carried out according to the seismic intensity coefficient along the height prescribed in Article 88 of the Enforcement Order in relation to earthquake loads. The story drift angle of the first story when subjected to the design load (the value of mean shear stress in the bearing wall on the first story ${}_5\tau_{DI}$ is = 0.318N/mm^2) is ${}_5R_1 = 0.14 \times 10^{-3}$, supposing 0.2 of seismic intensity coefficient on each story. The test result of shear distribution coefficient in each plane of structure corresponding to this story drift angle is derived as follows:

$$D_{xA} = 1.5, \quad D_{xB} = 0.32, \text{ and } \quad D_{xC} = 1.0$$

Using the values obtained above for the shear distribution coefficient on each plane of structure in the longitudinal direction (in the following treatment, the longitudinal direction is taken as the x-direction, and the transverse direction as the y-direction. (see Commentary Figure 4.6.)), the eccentric ratio R_{ex} is calculated on a trial basis for a two apartment units in each story (because r_{ex}

increases and the eccentric ratio R_{ex} calculated from Equation (4.2-a) decreases as the number of apartment units increases like four, six and larger, the case of a two apartment units in each story giving a maximum value of R_{ex} is taken up as an example) and is given below for reference.

The following assumptions are set up:

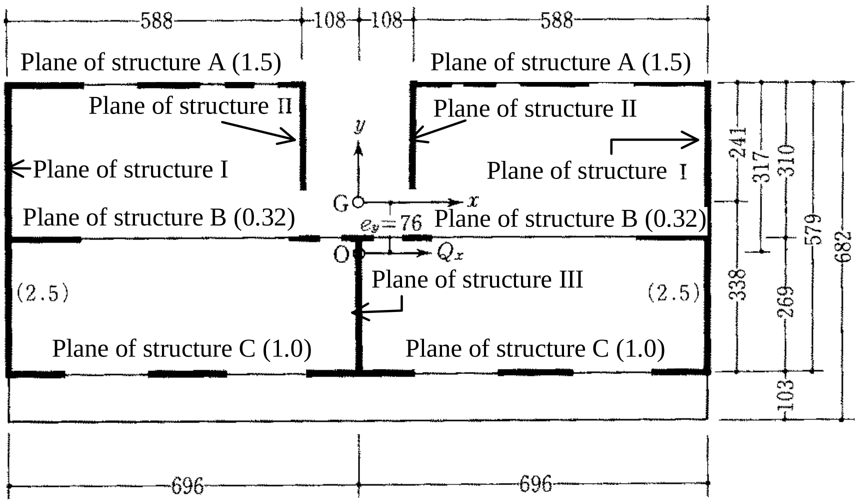
i) Assuming that the sum of dead and live loads on the dwelling and balcony areas on each story is 12,000 and 6,000 N/m^2 respectively, obtain W and its center of gravity O.

ii) For the shear distribution coefficient $D_{y\,I}$ of the plane of structure I (see Commentary Figure 4.6.) in the y-direction not under test, assume that the rigidity ratio $D_{xA}/D_{y\,I}$ of the plane of structure A to the plane of structure I is given by the formula proposed by Dr. Muto related to the reduction factor r of shear rigidity for a bearing wall with a small opening and find $D_{y\,I}$ using the following equation:

$$D_{yI} = \frac{\mathrm{D_{xA}}}{r} = \frac{\mathrm{D_{xA}}}{1.0 - 1.25\mathrm{p}} \quad \text{for} \quad p \leq 0.4$$

where,

$$p = \sqrt{\frac{\text{area of opening}}{\text{area of wall}}} \quad \text{: perimeter ratio of opening .}$$



Note: The values in ( ) are shear distribution coefficients with the shear distribution coefficient of the plane of structure C taken as unity. (Unit: cm)

C. Fig. 4.6 5-story Type 63-5N-2DK-4 two unit apartment house for which the eccentric ratio R_{ex} in the first story was examined. The positions of the center of rigidity G and the center of gravity O of *W*, and the shear distribution coefficient on each plane of structure are shown.

Using both the perimeter ratio of opening p = 0.38 of the structure A and the shear distribution coefficient D_{xA} = 1.5 of the structure A, $D_{y\,I}$=2.85 is obtained. Within the examination, D_{yI} =2.5 is taken, considering that underestimating D_{yI} can lead to an overestimation (estimation on the safe side in design) of R_{ex} and α_{xi}.

iii) The existence of the structure II in the y-direction may be neglected because it does not contributes to the torsional rigidity K_T about the center of rigidity G.

The positions of center of rigidity G and center of gravity O of W are calculated using the assumptions above and they are shown in Commentary Figure 4.6. The eccentric distance to the earthquake load Q_x acting on O in the x-direction from G is as follows:

$$e_y = 760 \text{ mm}$$

As a result, $\sum D_{xi} \cdot y_i^2$, $\sum D_{yj} \cdot x_j^2$ and $\sum D_{xi}$ lead to:

$$\sum D_{xi} \cdot y_i^2 = 2(\underbrace{1.5 \times 2.41^2}_{\text{Structure A,}} + \underbrace{0.32 \times 0.69^2}_{\text{Structure B,}} + \underbrace{1.0 \times 3.38^2}_{\text{Structure C}}) \times 10^6 \text{ mm}^2$$

$$= 40.6 \times \ \times 10^6 \text{ mm}^2$$

$$\sum D_{yj} \cdot x_j^2 = 2 \times 2.5 \times 6.96^2 \times 10^6 \text{ mm}^2 \quad \text{(Structure I)}$$

$$= 242.2 \times \times 10^6 \text{ mm}^2$$

$$\sum D_{xi} = 2(1.5 + 0.32 + 1.0) = 5.64 \qquad \text{(Structures A, B and C)}$$

By substituting the results above into Equation (4.3-a), r_{ex} is obtained as follows:

$$r_{ex} = \sqrt{\frac{40.6 + 242.2}{5.64}} \times 10^3 = 7{,}080 \text{ mm}$$

Therefore, the eccentric ratio R_{ex} given by Equation (4.2-a) leads to:

$$R_{ex} = \frac{760}{7{,}080} = 0.11 < 0.15$$

which satisfies the condition of Equation (4.6-a).

As is obvious from the example of calculation above, the value of $\sum D_{yj} \cdot x_j^2$ of a bearing wall in the y-direction, perpendicular to the direction of the earthquake load Q_x make a remarkable contribution for increasing the value of r_{ex}. It also contributes much to increase the torsional rigidity K_T of the story .

In case where an earthquake load Q_y is applied in the y-direction (partition wall direction), e_x becomes zero and the eccentric ratio R_{ey} given by Equation (4.2-b) results in null.

In case where upper stories are setback shaped in the x- or y-direction, the value of e_y or e_x is increased, and therefore the modification factor α_{xi} or α_{yj} given by Equations (4.1-a and -b) is increased. However, since the dead and live loads of the upper stories are reduced, the earthquake load Q_x and Q_y are reduced. As a result, it is generally recognized that the shear forces in a bearing wall are not increased due to the setback of upper stories, unless the value of r_{ex} or r_{ey} is extremely small.

(b) Elevation Planning of Bearing Walls

There are many cases where the designer pays much attention to the floor planning of bearing walls. Since the bearing walls are charged with the duty of transmitting the earthquake load acting on the building to the ground through the foundation structure, proper attention must be given to the elevation planning as well. The wall girder mutually connecting bearing walls is charged with the important role of preventing the bearing walls from becoming rifting or overturning and preventing the decreasing shear resistance of the bearing walls by the flexural and shear resistance at its boundary ends (see Commentary Figure 4.7.). In case where, this wall girder yields by flexure in an early stage, the bearing walls do not carry expected horizontal loads (see Commentary Figure 4.8.). Noting that a large shear resistance can be expected only when the bearing walls cooperate as integrated unit, the wall girders connecting the bearing walls each other must have both sufficient rigidity and strength (see the Commentaries on Article 6.).

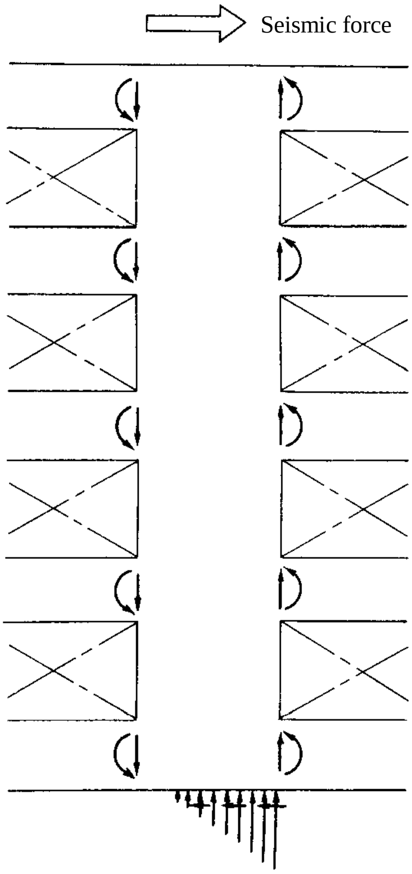
In Commentary Figure 4.7, bearing walls are arranged to be vertically continuous, and this arrangement can be regarded practically safe for relatively low rise buildings to which this Standard is applied.

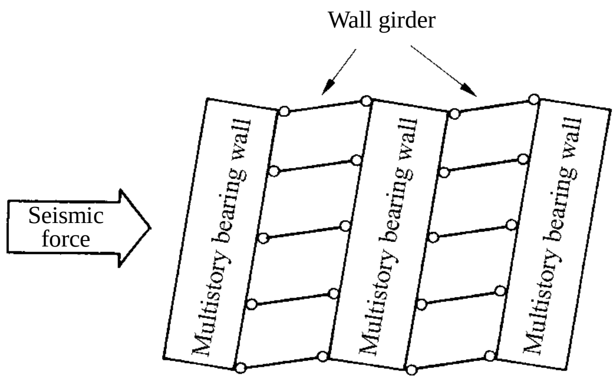
For bearing walls not arranged to be vertically continuous, it is desirable to connect them at their tops and bottoms by wall girders of sufficient rigidity and strength as well as to provide vertical supports under the bottom of the bearing walls which can effectively prevent them from rotating. These supports are desirably placed as near the both sides of the bearing wall as possible so that the rotation of the bearing wall is effectively suppressed. The following arrangements are conceivable:

i) arrange the bearing walls vertically so that the top of the lower wall is placed with a lap to the bottom of the upper wall,

ii) place a lower bearing wall perpendicular to the upper walls, and

iii) provide supporting members to the walls.





C. Fig.4.8 Multistory bearing walls making hinges at the ends of wall girders at the boundaries forming the total collapse mode

C. Fig.4.7 Balance of horizontal forces acting on multistory bearing wall

In case where the supported area is small, a large stress concentration occurs in these supporting members, so that proper attention must be given to the reinforcement of the members (see the Commentaries on Clause 10, Article 5). Either attaching ribs to the supporting members or making their plan type to be L, T or ＋ is effective in the prevention of local buckling. If bearing walls are sufficiently stiffened individually at the periphery and floor planning is appropriately executed, stress concentration and flexural deformation are smaller than those in the multistory continuous bearing wall.

In a story having a much smaller horizontal rigidity than other stories, the story deformation angle is larger than the value defined by the A_i distribution effect. Therefore, Article 82-3 of the Enforcement Order stipulates for specified buildings 31 meter high or less that the story stiffness ratio R_{si} in the calculation direction in the i-th story given by Equation (4.7) should satisfy the condition of Equation (4.10) in every story unless it is verified that the horizontal load-carrying capacity of each story is equal to or greater than the necessary horizontal load-carrying capacity of the story under consideration.

$$R_{si} = \frac{r_{si}}{\bar{r}_s} = \frac{\dfrac{n}{R_i}}{\displaystyle\sum_{i=1}^{n} \frac{1}{R_i}} \tag{4.7}$$

where,

R_i : story deformation angle on the i-th story,
r_{si} : reciprocal of R_i on the i-th story,

$$r_{si} = \frac{1}{R_i} \tag{4.8}$$

$\bar{r}_s$: arithmetic mean of r_{si} taken across the building (with n stories) , and

$$\bar{r}_s = \frac{1}{n} \sum_{i=1}^{n} \frac{1}{R_i} \tag{4.9}$$

$$R_{si} \geq 0.6 \tag{4.10}$$

In case where the standard shear force coefficient C_0 is 0.2, the mean shear stresses $\bar{\tau}_i$ ($\leq \bar{\tau}_{0i}$, see Commentary Table 4.3) of the bearing walls in each story are all small and the bearing walls behave in an elastic range. Thus, assuming that the values of $R_i/\bar{\tau}_i$ in each story are equal in the direction x or y, in the case of the wall length ratio L_i of bearing walls and the wall thickness t_i on the i-th story R_{si} is given by Equation (4.11) where n represents the number of stories of the building.

$$R_{si} = \frac{\dfrac{n}{\bar{\tau}_i}}{\displaystyle\sum_{i=1}^{n} \frac{1}{\bar{\tau}_i}} \tag{4.11}$$

where,

$\bar{\tau}_i$: mean shear stress of the bearing walls in the i-th story (see Equation (4.24))

$\bar{\tau}_i$ is determined depending on the following three cases,

i) the wall length ratio L_i and wall thickness t_i of the bearing walls in the i-th story are equal to the prescribed wall length ratio L_{0i} given in Table 1 of this article, and the prescribed wall thickness t_{0i} given in Table 2 of Article 5, that is, $\bar{\tau}_i$ is equal to $\bar{\tau}_{0i}$ in Table 4.3 (see Commentaries "2. Wall Length Ratio" of this article), where $\bar{\tau}_i$ is given by the following equation:

$$\bar{\tau}_i = \bar{\tau}_{0i}$$

C. Table 4.1 Story stiffness ratio R_{si} of each story

Values in the top row correspond to the case of $t_i = t_{0i}$ and $L_i = L_{0i}$.
Values in the middle row correspond to the case of $t_i = t_{0i}$ and $L_i = L_{01}$.
Values in the bottom row correspond to the case of $t_i = t_{01}$ and $L_i = L_{01}$.
For t_{0i} and L_{0i}, see Commentary Table 4.3.

i \ n	5 story buildings	4 story buildings	3 story buildings	2 story buildings	Single story buildings
5th story	1.516 1.618 1.816	–	–	–	–
4th story	1.089 1.163 1.082	1.431 1.491 1.653	–	–	–
3rd story	0.816 0.872 0.816	1.014 1.057 0.982	1.342 1.342 1.468	–	–
2nd story	0.839 0.716 0.684	0.761 0.793 0.742	0.945 0.945 0.871	1.262 1.262 1.262	–
1st story	0.740 0.632 0.602	0.792 0.659 0.622	0.713 0.713 0.661	0.738 0.738 0.738	1.000 1.000 1.000

ii) the wall length ratio L_i of the bearing walls in each story is equal to the prescribed wall length ratio L_{01} in the first story of the building given in Table 1 of this article and the wall thickness t_i of the bearing walls in each story is equal to the prescribed wall thickness t_{0i} in the story under consideration of the building given in Table 2 of Article 5, that is, where $\bar{\tau}_i$ is given by the following equation:

$$\bar{\tau}_i = \frac{L_{0i}}{L_{01}} \bar{\tau}_{0i}$$

iii) the wall length ratio L_i of the bearing walls in each story is equal to the prescribed wall length ratio L_{01} in the first story of the building given in Table 1 of this article and the wall thickness t_i of the bearing walls on each story is equal to the prescribed wall thickness t_{01} in the first story of the building under consideration given in Table 2 of Article 5, that is, where $\bar{\tau}_i$ is given by the following equation:

$$\bar{\tau}_i = \frac{L_{0i}\, t_{0i}}{L_{01}\, t_{01}} \bar{\tau}_{0i}$$

The values of R_{si} obtained for these cases using Equation (4.11) and Equations (4.11'), (4.11") and (4.11"') are given in Commentary Table 4.1.

$$R_{si} = \frac{\dfrac{n}{\overline{\tau}_{0i}}}{\sum_{i=1}^{n} \dfrac{1}{\overline{\tau}_{0i}}} \tag{4.11′}$$

$$R_{si} = \frac{\dfrac{n}{L_{0i}\overline{\tau}_{0i}}}{\sum_{i=1}^{n} \dfrac{1}{L_{0i}\overline{\tau}_{0i}}} \tag{4.11″}$$

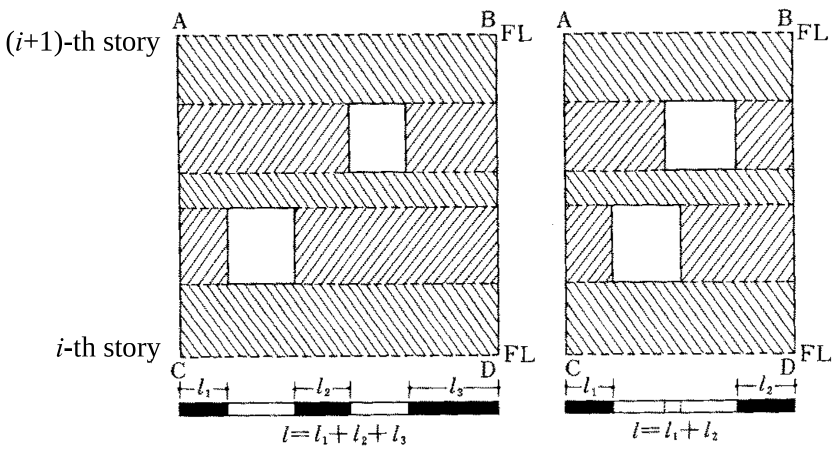
$$R_{si} = \frac{\dfrac{n}{L_{0i}t_{0i}\overline{\tau}_{0i}}}{\sum_{i=1}^{n} \dfrac{1}{L_{0i}t_{0i}\overline{\tau}_{0i}}} \tag{4.11‴}$$

The values of R_{si} given in Commentary Table 4.1 are calculated on the assumption that the values of $R_i / \overline{\tau}_i$ on each story are all equal in the direction x or y.

As seen in Commentary Table 4.1, ordinary RC box-shaped wall building is considered to satisfy the condition of Equation (4.10) ; i.e., $R_{si} \geq 0.6$.

2. Wall Length Ratio

In the x- or y-direction on the i-th story, the total length of bearing walls $\sum l$ (mm) specified in Article 5, divided by the floor area S_i (m^2) in that story is called the wall length ratio L_i (mm/m^2). Where, the floor area for calculating wall length ratio is defined as the floor area in that story adding the half area of balcony and poach at upper story. The floor area S_i in this Article represents the earthquake load imposed on the bearing walls, and is called the "floor area for calculating wall length ratio."



C. Fig. 4.9 Example of calculation of length *l* of bearing wall ABDC with openings

If two or more openings are unevenly distributed on the top or bottom on the same story and there is no portion having the same wall length defined in Clause 1, Article 5, the assemblage of such rectangular portions of the bearing wall partitioned by the openings that conform to the prescriptions in Article 5 regarding to the structure of bearing walls is considered as one bearing wall with openings. And the whole length of its horizontal cross section minus the sum of lengths of openings of the horizontally projected cross section of the openings (the portions not conforming to the structure of bearing walls prescribed in Article 5 are considered as openings) is considered as the length l of that bearing wall with openings for calculation of L_i (see Commentary Figure 4.9).

From the analysis and consideration of lateral loading tests on a full-scale specimen and a reduced-scale plane specimen simulating a 5-story apartment house of RC box-shaped wall building, the mean shear stress $\bar{\tau}_u$ taken over all the bearing walls is given by Equation (4.12) when the story shear force on the first story of a 5-story building reaches the horizontal load-carrying capacity regardless of the type of concrete used**.**

$$\bar{\tau}_u \fallingdotseq 1.7 f_{s\,(\text{temporary loading})} \tag{4.12}$$

Where,

$f_{s(\text{temporary loading})}$:allowable shear stress of concrete under temporary loading (see Commentary Table 4.2)

C. Table 4.2 Allowable shear stress of concrete at temporary loading $f_{s(\text{temporary loading})}$

Type of concrete	Design strength concrete F_c	
	21 N/mm^2 or less	21 N/mm^2 or more
Ordinary concrete	0.050 F_c	0.75 + 0.015 F_c
Lightweight concrete：Type 1 Lightweight concrete：Type 2	0.045 F_c	0.675 + 0.0135 F_c

[Notes] F_c：denotes design strength concrete

It is also inferred that when remarkable shear crack occurs in bearing walls, the mean shear stress $\bar{\tau}_{cr}$ of all the bearing walls is given by Equation (4.13). Therefore, it can be assumed that the horizontal force in each story at the occurrence of shear cracks is in proportion to the wall area ratio $L_i\ t_i$ of bearing walls.

$$\bar{\tau}_{cr} = f_{s\,(\text{temporary loading})} \tag{4.13}$$

From Equations (4.12) and (4.13), both the value of lateral load-carrying capacity of each story and the story shear force when large shear crack occurs in the bearing walls are considered to be almost in proportion to the wall area ratio $L_i\ \ t_i$ (mm^2/m^2) on the i-th story. Therefore, if the wall thickness t_i of the bearing walls on the i-th story is increased over the prescribed wall thickness t_{0i}

given in Table 2 of Article 5, L_i is reduced in accordance with the value of t_i/t_{0i}. However, with t_i extremely increased and with l of bearing walls extremely decreased, the design deviates from the concept of RC box-shaped wall structure and its structural concept approaches to moment resisting frame structures, and the relationships of Equations (4.12) and (4.13) is not well concluded. Therefore, it is required that $\sum l$ (mm) in the direction x or y divided by S_i (m^2) in each direction L_i should be equal to or greater than the prescribed wall length ratio L_{0i} (mm/m^2) given in Table 1 of this article minus 30 mm/m^2. In case where the thickness of a bearing wall t_i is increased over t_{0i}, the structure of wall girders attached to the bearing wall must be sufficiently rigid to resist against the possible stress concentration.

For the values of prescribed wall length ratio L_{0i} (mm/m^2) given in Table 1 in the Standard, apartment houses of RC box-shaped wall structure with an appropriate structural plan were examined. It had been concluded that they did not cause discrepancy in design. For RC box-shaped wall building whose wall area ratio $L_i \quad t_i(\sum A_w/S_i)$ (mm^2/m^2) of bearing walls was equal to or greater than the prescribed wall area ratio $L_{0i}t_{0i}$ (t_{0i}: prescribed wall thickness, see Table 2 of Article 5), lateral loading tests were conducted on a full-scale three dimensional test specimen and planes of structure of a reduced-scale test specimen both simulating a 5-story apartment house**,** and the analysis and examination were carried out on earthquake damage surveys. From these tests and analyses, the conclusion was drawn that the maximum response story deformation angle in each story is less than 1/200 for a peak ground acceleration of 3 to 4 in m/sec^2. and the structure is adequately proved against shear failure ; that is, the building can endure the maximum earthquake ground motion that is likely to occur during its service life.

In case where the floor areas are identical with one another in each story taken as a standard, the earthquake load acting on each story and the mean shear stress produced in the bearing walls are considered as follows.

(a) Earthquake Load Acting on Each Story

According to Article 88 of Enforcement Order, the earthquake load Q_{0i} per unit floor area of 1 m^2 acting on each story is given by Equation (4.14)

$$Q_{0i} = C_i m\,\overline{w}_i = ZR_t A_i C_0 m\,\overline{w}_i \qquad (4.14)$$

Where,

C_i : seismic shear force coefficient on the i-th story,

C_i : $Z\,R_t A_i C_0$ (4.15)

Z : seismic zone factor (= 1 to 0.7) (see the Commentaries on Article 1), (4.16)

R_t : numerical value calculated by the method determined by the Minister of Land, Infrastructure and Transport from the design fundamental period of vibration of the building, T, and the type of the ground representing the vibration characteristics of the subsoil condition (this value is called the vibration characteristic coefficient of the building. See Notification No. 1793, the Ministry of Land, Infrastructure and Transport in 1980). Since the height of RC box-shaped wall building designed according to this Standard is restricted up to 16m by Article 3, T is given by Equation (4.17′) using Equation (4.17) stipulated in the Notification.

$$T = 0.02H \text{ (s)} \tag{4.17}$$

$$\leq 0.02 \times 16 = 0.32 \text{ (s)} \tag{4.17′}$$

The Notification stipulates that if the value of T calculated by Equation (4.17) is no less than the value of T_c (0.4, 0.6, or 0.8 (s)) determined according to the type of the ground, the value of R_t should not be reduced below the value given by Equation (4.20). Since the value of T is smaller than any value of T_c, R_t is given by Equation (4.20).

$$R_t = 1 \sim 0.75^* \tag{4.18}$$

Note: * under the provision of the Notification, if it is confirmed based on the results of special research or investigation that the value of R_t can be smaller than 1, the value of R_t may be reduced to the value based on the results of special research or investigation (to 0.75 if the value becomes less than 0.75),

A_i : coefficient representing the distribution of seismic shear force coefficient along the height of building (see the Commentaries on Article 1 and Commentary Figure 4.10),

$$A_i = 1 + \left(\frac{1}{\sqrt{\alpha_i}} - \alpha_i\right)\frac{2T}{1+3T}, \tag{4.19}$$

T : the design fundamental period of vibration of the building (in s) (see Equation (4.17)),

α_i : coefficient determined from weight W supported by the story for which A_i of the building is to be calculated divided by the total weight supported by the first story of the building (see the Commentaries on Article 1 for W),

$$\alpha_i = \frac{m\overline{w}_i}{n\overline{w}_1}\frac{S_i}{S_1}, \tag{4.20}$$

S_i and S_1 : floor area (m^2) in the i-th story and first story,

m and n : number of floor slabs supported by the i-th story and first story (excluding the roof slabs of penthouses that are not included in the number of stories (see the Commentary on Article 3)),

w_i and w_1 : values in N/m^2 derived from dividing W_i of the i-th and W_1 in the first story by mS_i and nS_1 respectively (for W, see the Commentaries on Article 1),

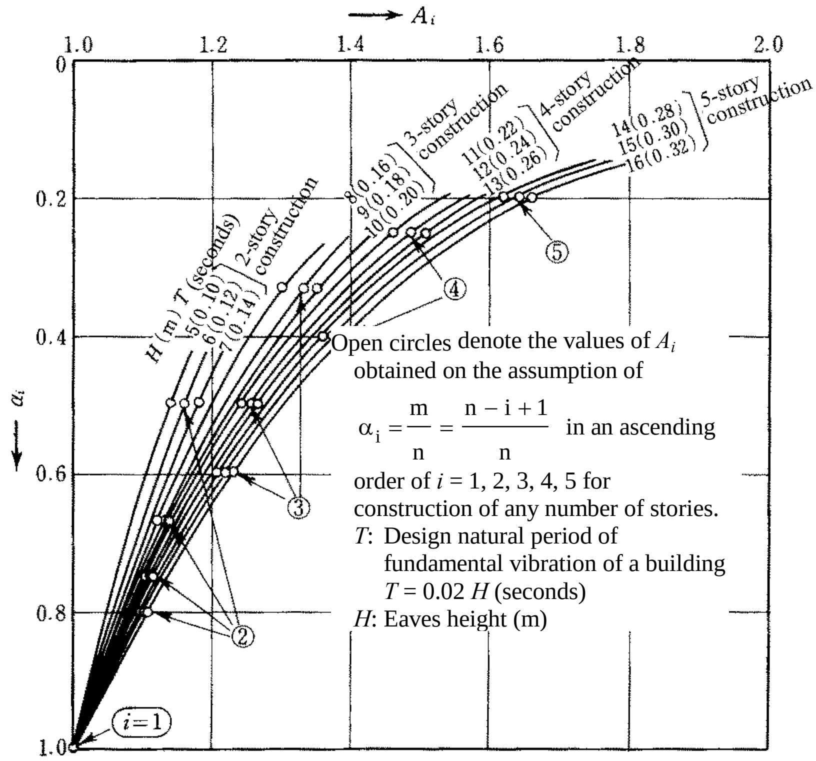
C_0 : standard shear force coefficient,

$$C_0 \geq 0.2, \tag{4.21}$$

for calculation of the required horizontal load-carrying capacity of each story against seismic forces, Equation (4.21') shall be used in place of Equation (4.21),.

$$C_0 \geq 1.0. \tag{4.21'}$$

Commentary Figure 4.10 shows a nomogram of A_i given by Equation (4.19). Substituting T given by Equation (4.17) into Equation (4.19) , A_i is given by Equation (4.19') which is a function of α_i and eaves height H (m).



Where,

A_i: coefficient representing the distribution of earthquake story shear coefficient in height along a building ,

α_i: sum of the dead and live loads supported by the portion for which A_i of the building is calculated divided by the sum of the dead and live loads supported by the portion of the first story of the building.

C. Fig. 4.10 The nomogram to determine A_i from α_i and H or T (=0.02H) for the i-th story of a n-story building

$$A_i = 1 + \left(\frac{1}{\sqrt{\alpha_i}} - \alpha_i\right)\frac{0.04H}{1+0.06H} \tag{4.19'}$$

Under the provisions of Article 3, there are the following limitations:

Height to building eaves $H \leq 16$ m

Story height of each story except the top story ≤ 3 m

Story height of the top story ≤ 3.3 m

Therefore, the eaves height H of an n-story building lies in the following equation :

$$H \leq 3\,n \pm 1 \text{ (m)}$$

About the nomogram of A_i in Commentary Figure 4.10, the curves represent in the cases where H (m) is assumed to be 3n-1, 3n, and 3n+1. The value of A_i increases with increase of design natural period T, namely, height H. The nomogram, however, indicates that the influence of T and H on A_i is less significant for any number of n.

In a vibration test on a full scale specimen simulating the 5-story apartment house (H = 14.04 m) of RC box-shaped wall structure with the fixed foundation, the measured period T of elastic vibration was 0.15 sec. in the longitudinal direction prior to the occurrence of shear cracks, and the calculated period was 0.28 sec. by Equation (4.17) stipulated in Notification No. 1793. It is theoretically pointed out and experimentally examined, however, that T becomes large when the building moves in rocking or sway modes on the ground. If the foundation is firmly fixed, elastic responses that are 3 or 4 times as large as the maximum acceleration of ground motion will be generated, which will produce shear cracks in the bearing walls. According to the earthquake damage survey on the 1978 Miyagi-ken Oki Earthquake during which the maximum acceleration of ground motion is considered to reach 2m/sec^2 or greater (see the Commentaries on Article 2), no remarkable shear cracks were observed. This fact indicates that RC box-shaped wall buildings on the ground move in rocking and sway modes making T longer and resulting R_t smaller than the evaluated value of unity.

(b) Mean Shear Stress of Bearing Walls

When Q_{0i} (N/m^2) given by Equation (4.14) is divided by the wall area ratio $L_i \cdot t_i$ (mm^2/m^2) in the i-th story, the mean shear stress $\bar{\tau}_i$ of the bearing walls is obtained in the x- or y-direction on the i-th story.

$$\bar{\tau}_i = \frac{Q_{0i}}{L_i t_i} = \frac{Z R_t A_i C_0 m \bar{w}_i}{L_i t_i} \qquad (4.22)$$

This Standard assumes that the value of $\bar{w}_i$ satisfies the condition of Equation (4.23) in each story.

$$\bar{w}_i \leqq 12{,}000 \text{ N/m}^2 \qquad (4.23)$$

In calculation of $\bar{\tau}_i$ by the use of Equation (4.22), the following assumptions are made:

i) $\bar{w}_i$ are equal on all stories and are given by Equation (4.23').

$$\overline{w}_i = 12{,}000\ \text{N/m}^2 \tag{4.23′}$$

ii) The floor areas for calculation of the wall length ratio are identical on all stories (the elevation has no setback).

$$S_i = S_1 \tag{4.24}$$

Using the relationships of Equations (4.23') and (4.24), Equations (4.20) are expressed as follows:

$$\alpha_i = \frac{m}{n} \tag{4.20′}$$

Substituting α_i given by Equation (4.20') into Equation (4.19') , Equation (4.20") is yielded.

$$A_i = 1 + \left(\frac{1}{\sqrt{m/n}} - \frac{m}{n}\right)\frac{0.04H}{1+0.06H} \tag{4.19″}$$

The monogram of A_i in Commentary Figure 4.10 allows A_i to be determined using the open circles in the case where α_i is assumed as Equation (4.20'), where A_i are given by Equation (4.19").

If the upper stories have setback or the weight of the upper story is less than the weight of lower stories, the condition of Equation (4.23') is not fully satisfied, and the following relation holds :

$$\overline{w}_i < 12{,}000\ \text{N/m}^2 \quad \therefore \overline{w}_i < w_1$$

In case where the story under consideration has a setback, the condition of Equation (4.24) regarding S_i is not satisfied, and:

$$S_i < S_1$$

For both cases of $S_i > S$ and $S_i < S_1$ the value of α_i exactly calculated from Equation (4.20) becomes smaller than that of α_i approximately estimated by Equation (4.20'), and satisfies the following condition:

$$\alpha_i < \frac{m}{n}$$

As indicated by Commentary Figure 4.10, A_i increases with decrease of $\alpha_{i.}$ The increment of Q_{0i} caused by that variation and given by Equation (4.14) is smaller than the decrement of Q_{0i}, because $\overline{w}_i$ is smaller than the assumed value of 12,000 N/m^2. It leads to the tendency that if $\overline{w}_i$ is decreased by the setback of the upper stories, $\overline{\tau}_i$ will not be increased. If $\overline{\tau}_i$ are required to be exactly calculated using Equation (4.22), based on the value of α_i exactly calculated, A_i given by Commentary Figure 4.10 or Equation (4.19') should be used.

Substituting the following values into Equation (4.22), Equation (4.22')is obtained,

i) Z = 1 in Equation (4.16)

ii) R_t = 1 in Equation (4.18)

iii) C_0 = 0.2 in Equation (4.21)

iv) $\overline{w}_i$ = 12,000 N/m^2 in Equation (4.23')

$$\overline{\tau}_i = \frac{24A_i m}{L_i T_i} \quad (\text{N/mm}^2) \qquad (4.22')$$

Considering that A_i becomes larger for longer T for greater H (see Commentary Figure 4.10), the following height

$H = 3\,n + 1$ (m)

is substituted into Equation (4.19''). Then, a formula giving the maximum of A_i is obtained as Equation (4.19''').

$$A_i = 1 + \left(\frac{1}{\sqrt{m/n}} - \frac{m}{n}\right)\frac{0.12n + 0.04}{1.06 + 0.18n} \qquad (4.19''')$$

Substituting the followings into Equation (4.22'),

v) the values of A_i given by Equation (4.19'''),

vi) the prescribed wall thickness t_{0i} of bearing walls in the i-th story given in Table 2 of Article 5,

vii) the prescribed wall length ratio L_{0i} (mm/cm^2) of bearing walls in the i-th story given in Table 1 of this article **,** and

viii) $m = n - i + 1$ as the value of m in the i-th story of n-story building,

the upper limit $\overline{\tau}_{0i}$ (N/mm^2) of mean shear stress of bearing walls is yielded in the x- or y-direction in each story. Values of the upper limit $\overline{\tau}_{0i}$ are given in Commentary Table 4.3.

As assumed in Equation (4.23), provided that the value of $\overline{w}_i$ is 12,000 N/m^2 or less on each story, the values of $\overline{\tau}_{0i}$ given in Commentary Table 4.3 can be considered to be the values of $\overline{\tau}_i$. The $\overline{\tau}_i$ agrees with those obtained with the assumption of C_0 equal to 0.2 or more.

As often seen in the transverse direction, if either the values of L_i are remarkably larger than the value of L_{0i}, or t_i are considerably larger than t_{0i}, the values of $\overline{\tau}_i$ are remarkably smaller than $\overline{\tau}_{0i}$. Therefore, exact calculation using Equation (4.25) makes an economical design possible.

$$\bar{\tau}_i = \frac{L_{0i} t_{0i}}{L_i t_i} \bar{\tau}_{0i} \tag{4.25}$$

Also in case when the upper stories have setback, the values of $\bar{w}_i$ are remarkably smaller than 12,000 N/m^2, and the values of $\bar{\tau}_i$ are considerably smaller than the values of $\bar{\tau}_{0i}$. Therefore, with use of exact calculation of $\bar{\tau}_i$ employing Equation (4.22), an economical design can be performed. As described previously, if the upper stories have setback, the values of A_i are considerably larger than the approximate estimation given by Equation (4.19"). Therefore, the value of A_i derived from substituting the exact value α_i obtained using Equation (4.20) into Equation (4.19') should be used for the value of A_i to be substituted into Equation (4.22).

C. Table 4.3 Upper limit of mean shear stress $\bar{\tau}_{0i}$ (N/mm^2) of bearing walls ($t_i = t_{0i}$, $L_i = L_{0i}$) in the *x*- or *y*-direction in each story calculated on the assumption of $Z = 1$, $R_t = 1$ and $C_0 = 0.2$ ($\bar{w}_i$ = 12,000 N/m^2).
Values in parenthesis denote the prescribed wall thickness t_{0i} (mm) times the prescribed wall length ratio L_{0i} (mm/m^2).

i \ n	5-story buildings	4-story buildings	3-story buildings	2-story buildings	Single-story buildings
5th story	0.22 (150 × 120)	–	–	–	–
4th story	0.31 (180 × 120)	0.20 (150 × 120)	–	–	–
3rd story	0.41 (180 × 120)	0.28 (180 × 120)	0.18 (150 × 120)	–	–
2nd story	0.39 (180 × 150)	0.37 (180 × 120)	0.25 (180 × 120)	0.16 (150 × 120)	–
1st story	0.44 (180 × 150)	0.36 (180 × 150)	0.33 (180 × 120)	0.27 (150 × 120)	0.16 (120 × 120)
Basement	0.40* (180* × 200)	0.33* (180* × 200)	0.27* (180* × 200)	0.20* (180* × 200)	0.13* (180* × 200)

* The value of $\bar{\tau}_{0i}$ at the basement is calculated assuming the horizontal seismic coefficient $k = 0.2$. Bearing wall thickness t_{0i} of the basement is assumed to be 180 mm which is the minimum wall thickness.

Note: The values of $\bar{\tau}_{0i}$ given in C. Table 4.3 apply to the case where normal concrete is used. According to Clause 2, Article 7 of the RC Standard, if lightweight concrete is used, the weight of reinforced concrete is reduced by 4 to 4.5 and 7 to 7.5 kN/m^3 for lightweight concrete Types 1 and 2 respectively. If lightweight concrete Type 1 or 2 is used, the value of $\bar{\tau}_{0i}$ can be roughly estimated by multiplying the values in Table 4.3 by the correction factor 0.9 or 0.8, respectively.

Substituting $A_c = 0$ (A_c: horizontal cross sectional area of columns) into Equation (1.1), one can obtain the relation described in the Commentaries on Article 1:

$$\sum 2.5A_w \geqq ZWA_i \tag{1.1′}$$

Dividing both sides of the above equation by $\sum A_w$, and using the relations of

$$\sum A_w = L_i t_i S_i,\ W = m\bar{w}_i S_i$$

Equation (4.26) is obtained.

$$2.5\text{N/mm}^2 \geqq \frac{ZWA_i}{\sum A_w} = \frac{ZA_i m\overline{w}_i}{L_i t_i} \tag{4.26}$$

Considering the $\overline{\tau}_{0i}$ value given in Commentary Table 4.3 is derived by substituting the following relations;

$$Z=1,\ R_t=1,\ C_0=0.2,\ \overline{w}_i=12{,}000\ \text{N/m}^2,\ L_i=L_{oi},\quad t_i=t_{0i}$$

into Equation (4.22), Equation (4.26) can be rewritten into Equation (4.26')

$$2.5\text{N/mm}^2 \geq \frac{5ZL_{0i}t_{0i}\overline{\tau}_{0i}}{L_i t_i} \tag{4.26′}$$

Considering the following, Equation (4.26") is derived :

i) According to Commentary Table 4.3, the following relationship is obtained for any story above the ground:

$$2.5\ \text{N/mm}^2 \geqq 5\overline{\tau}_{0i}$$

ii) Z = 1.0 to 0.7 (see the Commentaries on Article 1)

iii) $L_i \cdot t_i \geq L_{0i} \cdot t_{0i}$ (see Articles 4 and 5)

iv) 12,000 $\text{N/m}^2 \geq W$ (N/m^2)

$$2.5\text{N/mm}^2 \geqq 5\overline{\tau}_{0i} \geq \frac{5ZL_{0i}t_{0i}\overline{\tau}_{0i}}{L_i t_i} \geq \frac{ZWA_i}{\sum A_w} \tag{4.26″}$$

According to the provisions of this Standard, RC box-shaped wall buildings designed to be $L_i t_i \geq L_{0i}t_{0i}$ satisfy the condition of Equation (1.1) ; the condition stipulated in Notification No. 1790, the Ministry of Land, Infrastructure and Transport in 1980. Under the provision of Clause 1, Article 81 of Enforcement Order, buildings of such construction are regarded as buildings other than specified buildings which are exempted from the secondary seismic design of the confirmation of seismic performance on the basis of ultimate design listed in the following.

i) Confirmation of the story deformation angle according to Article 82-2 of the Enforcement Order.

ii) Confirmation of story stiffness ratio and eccentric ratio according to Article 82-3 of the Enforcement Order.

iii) Confirmation of horizontal load-carrying capacity according to Article 82-4 of the Enforcement Order.

For a bearing wall fixed at the bottom, the relationship between the shear force Q and story deformation angle R can be found using a simple elastic theory (see Commentary Figure 4.11).

$$Q = \frac{tlE}{f} R = \frac{A_w E}{f} R \tag{4.27}$$

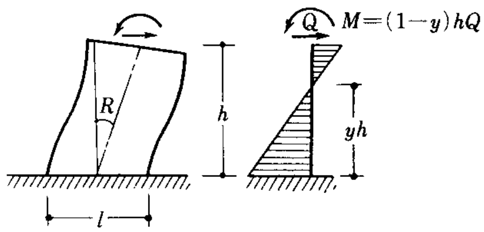
where,

$$f = 2\{k(1+\nu) + (3y - 1)(\frac{h}{l})^2\},$$

t : wall thickness,
l : wall length,

h : wall height,
y : inflection point height ratio,
k : shape factor for shear rigidity, $k = 1.2$
ν : Poisson's ratio of concrete, $\nu = 1/6$
E : Young's modulus of concrete, $E = 2.1 \times 10^4$ N/mm^2

$$\tau = \frac{Q}{tl} = \frac{Q}{A_w}, \tag{4.28}$$



C. Fig. 4.11 Deformation and stress of bearing wall

Defining the shear stress τ of the bearing wall by Equation (4.28) and substituting Q given by Equation (4.27) into the equation**,** one can obtain Equation (4.29).

$$\tau = \frac{E}{f} R. \tag{4.29}$$

Equation (4.29) indicates that τ of bearing walls of the same story deformation angle R becomes larger for a larger aspect ratio l/h (for a longer length l if the height h is constant) and for a smaller inflection point height ratio y (for a lower height of the point of inflection). Assuming that the wall girders attached to the bearing walls have equal rigidity, the larger l/h results in the larger y , and their variation on the shear stress are in the tendency to cancel each other .

For the building designed with well-balanced bearing wall arrangement, one can assume that the in-plane deformation and rigid rotation of the floor slabs can be neglected and the bearing walls have the same story deformation angle R, and the mean shear stress $\bar{\tau}_i$ of bearing wall on the i-th story is given by Equation (4.30). Note that $\sum$ denotes a summation taken in the direction x or y.

$$\bar{\tau}_i = \frac{\sum Q}{\sum tl} = \frac{RE}{\sum A_w} \sum \frac{A_w}{f} \tag{4.30}$$

The maximum value of τ of each bearing wall is given by Equation (4.29') in accordance with Equation (4.27).

$$\tau_{\max} = \frac{E}{f_{\min}} R \tag{4.29′}$$

The factor ϕ of stress concentration for τ is given by Equation (4.31).

$$\phi = \frac{\tau_{\max}}{\bar{\tau}_i} = \frac{\sum A_w}{f_{\min} \sum (A_w / f)} \tag{4.31}$$

Considering that the aspect ratio l/h of bearing walls counted in the wall length ratio is stipulated to be 0.3 or greater (see Clause 1, Article 5), the factor ϕ given by Equation (4.31) can be taken roughly as :

$$\phi = 1.5 \tag{4.31′}$$

The adequacy of the value was generally examined by lateral loading experiments on the scaled plane specimens. The value $\tau_{\max}$ is given by Equation **(**4.32**)**.

$$\tau_{\max} = \phi \bar{\tau}_i = 1.5 \bar{\tau}_i \tag{4.32}$$

Since the Clause 1, Article 2 stipulates the design standard strength F_c of concrete, the allowable shear stress at 18 N/mm^2 or greater, at temporary loading, $f_{s\ (\text{temporary loading})}$, in concrete stipulated in Article 6 of AIJ RC Calculation Standard satisfies the conditions of Equations (4.33-a and b) (see Commentary Table 4.2).

Normal concrete:

$$f_{s\ (\text{temporary loading})} \geq 0.05 \times 18 = 0.9 \text{ N/mm}^2 \tag{4.33-a}$$

Lightweight concrete Types 1 and 2:

$$f_{s(\text{ temporary loading })} \geq 0.9 \times 0.05 \times 18 = 0.81 \text{ N/mm}^2 \tag{4.33-b}$$

Substitution of the value of $\bar{\tau}_{0i}$ given in Commentary Table 4.3 into $\bar{\tau}_i$ in Equation (4.32) and comparison of the result with Equations (4.33-a and –b) shows that the maximum value, $\tau_{\max}$, of τ of the bearing wall derived from employing the lower limit of 0.2 of standard shear force coefficient C_0 stipulated in Article 88-2 of Enforcement Order satisfies the condition of Equation (4.34) in each story and conforms the provisions related to the primary design in Article 82-3 (Stresses, etc.) of the Enforcement Order.

$$\tau_{\max} \leq f_{s(\text{temporary loading})} \qquad (4.34)$$

Since an RC boxed-shaped wall structure is made of thin plates and has no moment resisting frames around the thin plates that can restrain the development of shear cracks occurring in a wall plate, various kinds of damage may be produced if the shear cracks develop. If significant shear cracks occur in a bearing wall supporting vertical loads, it will be difficult to repair the damaged walls. Taking these facts into consideration, this Standard takes the design principle of keeping down the stresses in the bearing walls below the elastic limit without generating cracks in the bearing walls and wall girders when subjected to seismic load that are likely to occur several times during its service life (a maximum acceleration of ground motion of 0.8 to 1m/s^2). However, the following facts should be considered：

i) In a building having a short natural period, a response acceleration 3 or 4 times as large as the ground surface acceleration is produced if the building behaves as an elastic structure fixed at the foundation. Then, for the maximum acceleration of ground motion of 0.8 to 1m/s^2, the value of $\bar{\tau}_i$ becomes considerably large compared to the value of $\bar{\tau}_{0i}$ given in Commentary Table 4.3 even if it is considered that the vibration characteristic coefficient factor R_t of the building becomes smaller than the lower limit of 0.75 specified in the Notification (see the footnote to Equation (3.21)).

ii) From the results of lateral loading tests in the longitudinal direction on the full-scale three dimensional specimen simulating a 5-story apartment house of RC box-shaped wall structure, it is inferred that cracks occur in a bearing wall of large l/h when $\bar{\tau}_i$ reaches $f_{s\,(\text{temporary loading})}$ as indicated by Equation (4.13).

iii) On the occasion of the 1978 Miyagi-ken Oki Earthquake during which the maximum acceleration of ground motion is considered to be larger than 2 m/s^2, only minor shear cracks or joint slides at the concrete were observed within 11 apartment houses out of 131 apartment houses of RC box-shaped wall structure in the stricken area ; i.e. in Sendai City.

With these facts taken into consideration, the value of L_{0i} given in Table 1 of Article 4 has no extra margin from the point of view of preventing shear cracks against the seismic load with standard shear force coefficient C_0 = 0.2. Considering this point, this Standard specifies a requirement that $\tau_{\max}$ calculated for a value of C_0 larger than 0.2 should satisfy the provisions of the Building Standard Law related to the primary design；i.e., allowable stress design given by Equation (4.34).

Article 5 Structural Details of Bearing Walls

1. The length of the bearing walls should be 450 mm or more, and be 30% or more of the height of its portion having the same wall length.
2. The bearing walls should have a thickness equal to or greater than the values t_0 defined as the minimum thickness in Table 2.

Table 2 Minimum thickness of bearing walls

Story		Minimum thickness t_0 (mm) of bearing walls	Remarks
Aboveground stories	Single-story building	12 and $h/25$	h: vertical distance between upper RC slab and lower RC slab (mm)
	Each story of 2-story building and the top story of 3-, 4- or 5-story building	15 and $h/22$	
	Other stories	18 and $h/22$	
Basement		18* and $h/18$	

* For the portion which is not finished on one side or both sides and not in contact with soil, the covering concrete over reinforcement should be increased by 10 mm to a total wall thickness of 190 or 200 mm.

For the portion in contact with soil on one side or both sides, the covering shall be increased by 10 mm to a total thickness of 190 or 200 mm, in case where normal weight concrete is used, and by 20 mm to a total wall thickness of 200 or 220 mm, in case where light weight concrete Type 1 or 2 is used.

3. In the bearing wall, shear reinforcement should be provided in both horizontal and vertical directions.

The ratio of total cross sectional area of horizontal reinforcement to vertical cross sectional area of concrete and that of vertical reinforcement to horizontal cross sectional area (hereafter the shear reinforcement ratio (Ps)) should be equal to or higher than the vales (Ps_0) given in Table 3.

Table 3 Shear reinforcement ratio in bearing walls

Story		Shear reinforcement ratio , Ps_0 (%)
Aboveground stories	Single-story buildings and the top story of 2-story buildings	0.15
	First story of 2-story buildings and the top story and the second story from the top story of 3-, 4- or 5-story buildings	0.20
	Other stories	0.25
Basement		0.25

4. In case where the wall length ratio is larger than the values given in Table 1, the shear reinforcement ratio may be reduced to the value calculated by the following formula within the range not less than 0.15%.

$$\text{(Shear reinforcement ratio)} = \text{(Shear reinforcement ratio given in Table 3)} \times \frac{\text{(Wall length ratio (mm/m}^2\text{) given in Table 1)}}{\text{(Design wall length ratio (mm/m}^2\text{))}}$$

5. Deformed reinforcing bars with nominal diameter of 10mm (hereafter abreviated D10) or those with greater diameter should be arranged horizontally and vertically, respectively.

   And the spacings of reinforcement bars should be 300 mm or less respectively in the projected area of bearing walls.

   In case where a double reinforcing bar arrangement is used, the spacings of horizontal and vertical reinforcement on either side should not exceed 450 mm respectively.

6. In the structurally important portions of bearing walls such as the wall ends, intersections, and the vertical edges of openings, flexural reinforcement whose quantity is equal to or more than those shown in Table 4 should be provided.

1-D13 may be used, in case where bearing wall whose thickness is less than 180 mm and having an orthogonal bearing walls. In the top story, the height of an opening edge shall be 2.8 m or less.

Table 4　Flexural reinforcement at the edge, etc. of bearing walls

Story	Flexural reinforcement at edge, etc. of bearing walls	
	$h_0 \leq 1$ m	$h_0 > 1$ m
Single-story buildings	1 – D13	1 – D13
Each story of 2-story buildings and the top story of 3-, 4- or 5-story buildings	1 – D13	2 – D13
Second story from the top story of 3-, 4- or 5-story buildings	2 – D13	2 – D13
Basement of single-story or 2-story buildings First story and basement of 3-story buildings Second and first stories and basement of 4-story buildings Third and second stories of 5-story buildings	2 – D13	2 – D16
First story and basement of 5-story buildings	2 – D16	2 – D19

[Remarks]　h_0 (m) :　height of opening edge along flexural reinforcement (in case where the wall portion above or under the opening is not of structure equivalent to or stronger than the bearing wall, the height of such a portion should be added.)

7. The reinforcement used to the horizontal edges of an opening should be D13 or more in size and should be being designed in accordance with Article 6.

8. For bearing walls of 180mm thick or more, reinforcement should be provided in double reinforcing bar arrangement.

9. At the intersection of a bearing wall with the floor, reinforcement of D13 or more should be provided.

   At the corners of an opening, diagonal reinforcement having a cross sectional area equal to or more than a half of the reinforcement shown in Table 4 and having a diameter not less than D10 should be placed.

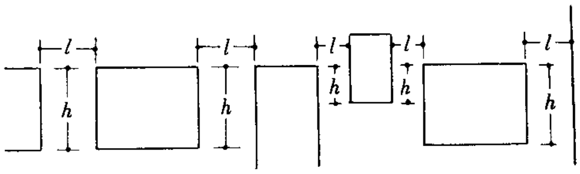
   Diagonal reinforcement bars may not be placed, in case where a surpluss cross sectional area of 0.35 times as much as the value given in Table 4 or more is added to required horizontal and vertical reinforcement.

10. The intersection of a bearing wall with a perpendicularly crossing wall girder or beam, should be constructed to transfer the loads from above to the bearing wall concerned or to other parts safely.

1. Bearing Wall Length

Because bearing walls are very important shear resistant members of RC box-shaped wall structures, bearing walls require sufficient strength and rigidity.

Therefore, the minimum length is specified. The length of the bearing walls should be 450 mm or more, and be 30% or more of *h:* the height of its portion having the same wall length (See Commentary Figure 5.1).



C. Fig. 5.1 Geometrical definition of bearing wall, $l \geq 0.3\ h$

2. Thickness of Bearing Walls

The minimum wall thickness t_0 given in Table 2 of this article is stipulated so as to allow a double reinforcing bar arrangement to be provided for all the bearing walls in the second story from the top story and lower(excluding the first story of 2-story buildings) (see Clause 8 of this article). In addition, the wall thickness is required to satisfy the specified proportion in terms of the ratio to *h* which is the vertical distance between primary horizontal supports (mostly RC floor slabs) for safety against out-of-plane buckling. In case where a bearing wall is horizontally supported at the top and bottom by rigid roof or floor slabs, the distance between the centers of the slabs may be taken as *h*. In the case of the first story, when the floor is made of wood without concrete slabs, and rigid foundation beams or tie beams orthogonal to the wall concerned, are provided, the distance between the center of these and the center of floor slabs may be taken as *h*. If the floor of the first story is of timber and if there is a rigid foundation beam or a tie beam orthogonal to the wall, the distance from its center to the center of the floor on the upper story is deemed as the vertical distance *h*. In case where there is no such effective beam, the distance from the bottom of the foundation to the center of the floor on the upper story is deemed as the vertical distance *h*. To provide sufficient rigidity, it is preferable to provide RC floor slabs of the first story. The floor slabs of the first story may be placed directly on the compacted ground with gravel top layer.

In the case of the walls having narrow width and not conforming to the requirements of bearing walls in this clause, axial forces and out-of-plane flexural moment acting to the wall concerned should be increased by the coefficient given in Commentary Table 5.1. (Referred to Commentary for column design in AIJ RC Calculation Standard)

C. Table 5.1 Overdesign factors for axial forces and bending moments out of plane to the bearing wall [1]

Wall thickness / Vertical distance between primary supporting points		1/10	1/15	1/20	1/25
Overdesign factor	Normal weight concrete	1.00	1.00	1.25	1.75
	Light weight concrete	1.00	1.20	1.50	–

The ACI Standards[2] of the U.S. stipulates wall thickness at 4" (102 mm) or more. The wall thickness is also required to be 1/25 or more of the shorter distance among the vertical or the horizontal distance between structural primary supporting points.

According to the BSCP Standards[3] of U.K., walls with a vertical reinforcement ratio of 1.0 % or less are treated as non-reinforced concrete walls in terms of performance of fire walls, and a wall thickness of 150 mm is required for 1-hour fire resistance and 175 mm for 1.5-hour fire resistance. Bearing walls with a thickness conforming to Table 2 of this article are considered to have fairly good performance as a firewall.

3. Shear Reinforcement Ratio of Bearing Walls

The design principle of bearing walls does not allow shear cracking, in the case of a seismic forces of standard shear coefficient C_0=0.2, which is likely to be encountered several times during the service life of the buildings. To deal with shrinkage cracking in concrete or other unexpected event, however, the shear reinforcement ratio Ps (given in Table 3 of this article) has been determined, neglecting the tensile force in concrete, to carry the story shear force calculated against standard shear coefficient C_0=0.2. According to the reports on horizontal loading test conducted to a full-scale three-dimensional specimen and a reduced model planar specimen, the bearing walls in the first story showed horizontal load carrying capacity of 4.3 times as much as that calculated by employing C_0=0.2. In actual structures, concrete of bearing walls and orthogonal walls are considered to carry significant amount of shear forces. Defining ${}_s\tau_a$ as the shear stress of concrete per unit horizontal cross sectional area under shear force which can be borne by the wall reinforcement only with its allowable tensile stress for temporary loading, the relationship of ${}_s\tau_a$ with p_s is given by Equation (5.1) neglecting the vertical stresses of the bearing wall.

$$ {}_s\tau_a = p_s f_{t\ \mathrm{(temporary)}} \qquad (5.1) $$

where,

$f_{t\ \mathrm{(temporary)}}$: allowable tensile stress under temporary loading of wall reinforcement.

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2) ACI Standard Building Code Requirements for Reinforced Concrete, ACI 318-83, 1983

3) British Standard Code of Practice for the Structural Use of Concrete, CP 110: Part I, 1972

Assuming the use of SD295A (f_t = 295 N/mm^2) as wall reinforcement, allowable shear stresses for temporary loading, ${}_s\tau_{a0}$, in each story are obtained by Equation (5.1) for the p_s values given in Table 3, as shown in Commentary Table 5.2.

The stress concentration due to the difference in aspect ratio of bearing walls is relaxed by the stress redistribution after the occurrence of shear cracking. Considering this fact, the mean shear stress $\bar{\tau}_{0i}$ given in Commentary Table 4.3 for bearing walls in the calculation direction in each story is compared to ${}_s\tau_{a0}$ of bearing walls in each story given in Commentary Table 5.2.

On the premise of an appropriate bearing wall arrangement of a little eccentricity, it is formed that p_{s0} of bearing walls in each story is stipulated for C_0 = 0.2 so that the tensile stress of wall reinforcement will not exceed $f_{t(\text{temporary})}$ even if shear cracking occurs.

C. Table 5.2 Shear stress, which can be transferred by wall reinforcement to its allowable tensile stress for temporary loading per unit horizontal cross sectional area of concrete when the shear reinforcement ratio p_s of a bearing wall agrees with the prescribed value p_{s0} on the story under consideration

[x10^{-1} N/mm^2]

n / i	5-story building	4-story building	3-story building	2-story building	Single-story building
5th story	6.0	–	–	–	–
4th story	6.0	6.0	–	–	–
3rd story	7.5	6.0	6.0	–	–
2nd story	7.5	7.5	6.0	4.5	–
1st story	7.5	7.5	7.5	6.0	4.5
Basement	7.5	7.5	7.5	7.5	7.5

A formula to find reinforcement spacing x (mm) from shear reinforcement ratio p_S is given by Equations (5.2-a and -b).

For single reinforcing bar arrangement: $$x = \frac{a_s}{p_s t} \quad (5.2\text{-a})$$

For double reinforcing bar arrangement : $$x = \frac{2a_s}{p_s t} \quad (5.2\text{-b})$$

where,

a_s : cross sectional area (mm^2) of shear reinforcing bar, and
t: wall thickness (mm).

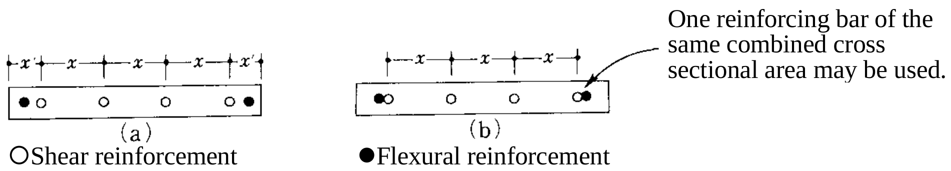
In case where D10 (a_s = 71 mm^2) is used for shear reinforcement, the relationships between p_S and required spacing for shear reinforcement x and wall thickness t are shown in Commentary Table 5.3.

In the calculation of P_S of a bearing wall, it is not permitted to take into account the cross sectional area of the necessary flexural reinforcement given in Table 4 of this article. Therefore, in the arrangement of shear reinforcement in the vertical direction of a bearing wall (called the vertical reinforcement hereafter), the spacing of reinforcing bars should be less than x_0 which is obtained by substituting $Ps = Ps_0$ into Equation(5.2-a or 5.2-b). At the vertical edge of bearing walls, the first shear reinforcement should be located at a distance x/2 from the edge (see Commentary Figure 5.2(a)). In this case, flexural reinforcement is located closer to the edge. Otherwise, the first shear reinforcing bar may be arranged together with flexural reinforcing bar (see Commentary Figure 5.2(b)). In this case, two bars may be replaced by a bar which has the same cross sectional area of the sum of these bars or more.

C. Table 5.3 Required spacing for shear reinforcement (for use of D10) (or the necessary spacing of shear reinforcement on one side, in the case of double reinforcement) (unit: cm)

Arrangement of reinforcements	Wall thickness (cm)	Shear reinforcement ratio *Ps* (%)											Remarks
		0.15	0.16	0.17	0.18	0.19	0.20	0.21	0.22	0.23	0.24	0.25	
Single re-bar arrangement	12	39.4*	36.9*	34.8*	32.8*	31.1*	29.5	28.1	26.8	25.7	24.6	23.6	* Spacings exceeding 30 cm are not permitted.
	13	36.4*	34.1*	32.1*	30.3*	28.7	27.3	26.0	24.8	23.7	22.7	21.8	
	14	33.8*	31.6*	29.8	28.1	26.6	25.3	24.1	23.0	22.0	21.1	20.2	
	15	31.5*	29.5	27.8	26.2	24.9	23.6	22.5	21.5	20.5	19.7	18.9	
	16	29.5	27.7	26.1	24.6	23.3	22.1	21.1	20.1	19.2	18.4	17.7	
	17	27.8	26.1	24.5	23.2	21.9	20.8	19.9	18.9	18.1	17.4	16.7	
Double re-bar arrangement	18	52.9*	49.6*	46.7*	44.1	41.8	39.7	37.8	36.1	34.5	33.1	31.7	* Projected spacings exceeding 30 cm and spacings of reinforcement on one side exceeding 45 cm are not both permitted.
	19	50.1*	47.0*	44.2	41.8	39.6	37.6	35.8	34.2	32.7	31.3	30.1	
	20	47.6*	44.6	42.0	39.7	37.6	35.7	34.0	32.5	31.0	29.7	28.6	
	21	45.3*	42.5	40.0	37.8	35.8	34.0	32.4	30.9	29.6	28.3	27.2	
	22	43.3	40.6	38.2	36.1	34.2	32.5	30.9	29.5	28.2	27.0	26.0	
	23	41.4	38.8	36.5	34.5	32.7	31.0	29.6	28.2	27.0	25.9	24.8	
	24	39.7	37.2	35.0	33.1	31.3	29.7	28.3	27.0	25.9	24.8	23.8	
	25	38.1	35.7	33.6	31.7	30.1	28.6	27.2	26.0	24.8	23.8	22.8	

[Note] (1) In case where D10 and D13 are arranged alternately, the spacing is about 1.5 times the value in the above table.
(2) In case where D13 is arranged, the spacing is about twice the value in the above table.



C. Fig. 5.2 Arrangement of shear reinforcement in bearing wall and wall girder (example of a single bar arrangement)

4. Reduction in Shear Reinforcement Ratio in Case Where the Wall Length Ratio Exceeds the Prescribed Wall Length Ratio

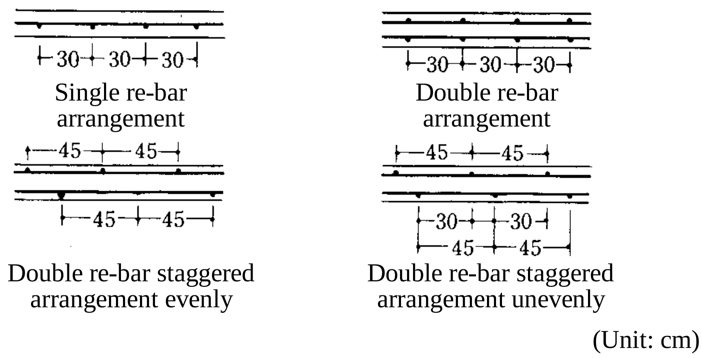
In case where the wall length ratio L_i in the i-th story exceeds the wall length ratio L_{0i} given in Table 1 of Article 4, the mean shear stress $\bar{\tau}_i$ of the bearing walls is smaller than the mean shear stress $\bar{\tau}_{0i}$ given in Commentary Table 4.3, and is given by Equation (5.3).

$$\bar{\tau}_i = \frac{L_{0i}}{L_i}\bar{\tau}_{0i} \tag{5.3}$$

The shear reinforcement ratio Ps_0 is reduced in proportion to Loi/Li. The minimum shear reinforcing ratio is required to be 0.15 %, to give sufficient ductility to the structure when an unexpected event occurs. The minimum shear reinforcement ratio of ACI Standards requires 0.25 % for horizontal reinforcement, and 0.15 % for vertical reinforcement.

5. Diameter and Spacing of Shear Reinforcement in Bearing Walls

It is stipulated that the shear reinforcement should have a diameter of D10 or more (#3 bar in the ACI Standards) and spacing of 300 mm or less over the projected area of a bearing wall from the point of view of the impact mitigation at the occurrence of cracking and the constraint of developments of both structural and shrinkage cracking. Even in a double arrangement, staggered reinforcement, it is stipulated that the spacing of shear reinforcement on one side should not exceed 450 mm (18" = 458 mm in ACI Standard) (see Commentary Figure 5.3).



C. Fig. 5.3 Maximum spacing of (vertical and horizontal) shear reinforcement

6. Flexural Reinforcement in Bearing Walls

Since bearing walls generally contain a smaller amount of reinforcement than the balanced steel ratio, their flexural resistance is influenced by all the vertical reinforcement on the tension side. Because the prescribed shear reinforcement ratio of bearing walls is as low as from 0.15 to 0.25%, it may be assumed that only the edge reinforcements at the ends and corners of, and the peripheries of openings in, the bearing wall contribute to flexural strength. On that assumption, an approximate formula can be obtained as the allowable moment of the bearing wall for temporary loading of the bearing wall.

Ultimate flexural strength of columns is approximated as Equation (5.4), when $N<0.4bDFc$. Considering the condition given by Equation (5.5), Equation (5.4) may be simplified as Equation

(5.5').

For $N \leq 0.4bDF_c$:

$$M_u = 0.8\, a_t \sigma_y D + 0.5ND\,(1 - \frac{N}{bDF_c}) \tag{5.4}$$

$$0.5ND\,(1 - \frac{N}{bDF_c}) \geq 0.3\,ND \tag{5.5}$$

$$M_u \geq 0.8\, a_t \sigma_y D + 0.3\,ND \tag{5.5'}$$

where,

- a_t : cross sectional area of flexural reinforcement in one wall edge,
- σ_y : nominal yield strength of flexural reinforcement in one wall edge,
- D : overall depth of cross section in bearing wall, and
- N : axial force in bearing wall (compression taken as positive).

By substituting the following relationships into Equation (5.5'):

$\sigma_y = f_{t\,(\mathrm{temporary})}$, $D = l$

where,

- $f_{t\,(\mathrm{temporary})}$: allowable tensile stress under temporary loading of flexural (edge) reinforcement in bearing wall,
- l : length of bearing wall.

An approximate formula for finding the allowable temporary flexural moment M_a of a bearing wall is obtained, as Equation (5.6).

$$M_a \geq (0.8\, a_t f_{t\,(\mathrm{temporary})} + 0.3\, N)\, l \tag{5.6}$$

where,

- a_t : cross sectional area of flexural (edge) reinforcement on the tension side, and
- N : vertical force acting on bearing wall (compression taken as positive).

Substituting the maximum flexural moment M occurring at the base of the bearing wall given by Equation (5.7) into M_a in Equation (5.6), yields the allowable inflection point height, $y_a h$, for bearing walls under temporary loads as Equation (5.8) can be determined which is a function of τ and N.

$$M = y\, h\, l\, t\, \tau \tag{5.7}$$

where , h : height of bearing wall,

y : ratio of inflection point height of bearing wall to h (inflection point height ratio), and

τ : shear stress of bearing wall.

$$y_a h \geq \frac{0.8\, a_t f_{t(\text{temporary})} + 0.3N}{t\tau} \tag{5.8}$$

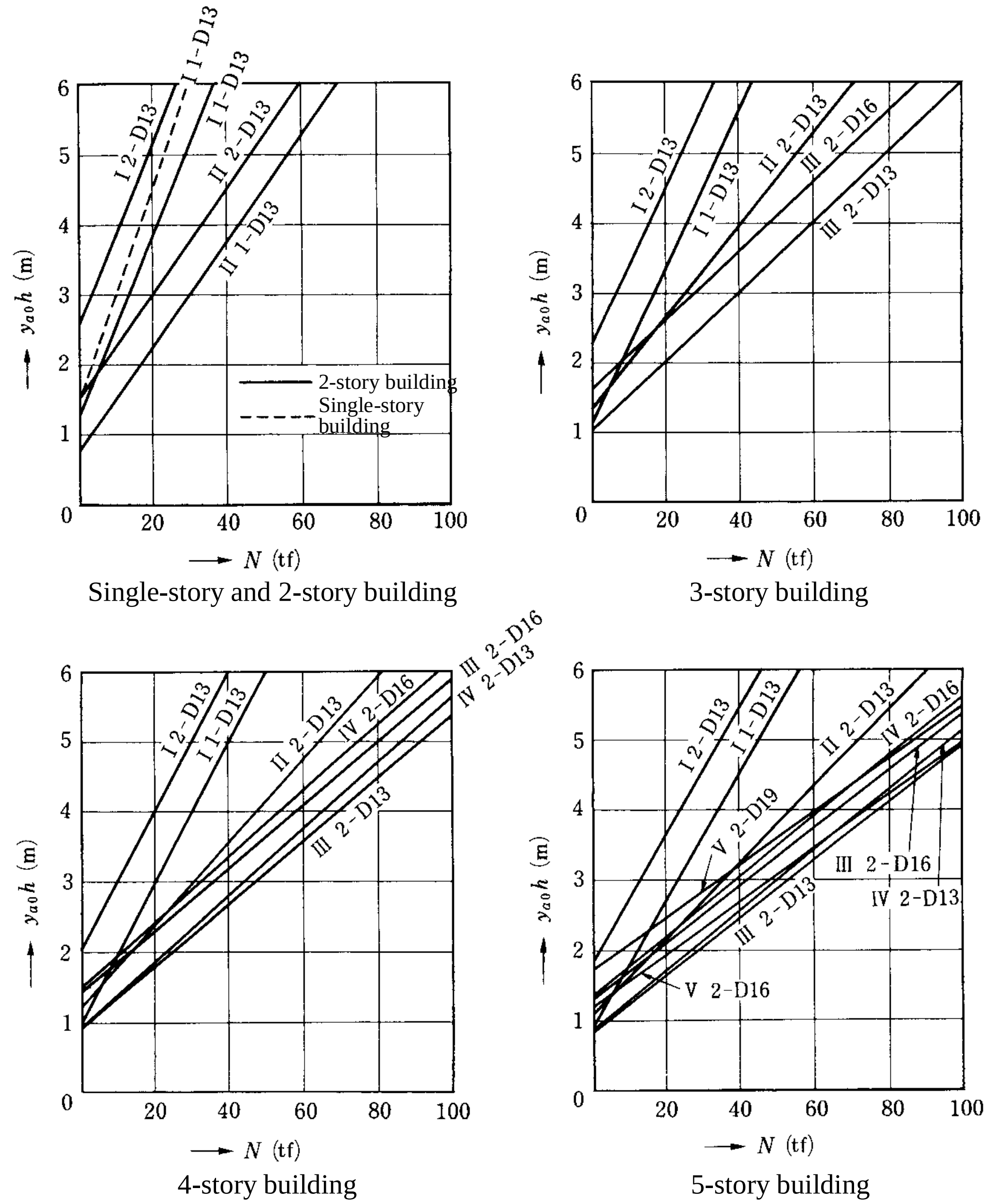
where,

y_a : allowable inflection point height ration.

By substituting τ and t in Equation (5.8) into mean shear stress $\overline{\tau}_{0i}$ of bearing walls in the calculation direction on each story given in Commentary Table 4.3 and the prescribed wall thickness t_{0i} in Table 2 of this article, one can find the allowable inflection point height $y_{a0}h$ for bearing walls in each story of a single-story or 2- to 5-story building in which the flexural reinforcement (material: SD295A, $f_{t\ (\text{temporary})}$ = 295 N/mm^2 is assumed) given in Table 4 of this article is used in bearing walls on each story, as Equation (5.8').

$$y_{a0} h = \frac{0.8\, a_t f_{t(\text{temporary})} + 0.3N}{t_{0i}\overline{\tau}_{0i}} \tag{5.8$'$}$$

The value of $y_{a0}h$ is a conservative value calculated by assuming the standard shear coefficient C_0 to be 0.2 and neglecting the tensile stresses in both the concrete and vertical reinforcement except for edge reinforcement in the cross section on the tension side, and is represented as a function of N as shown in Commentary Figure 5.4. Story deformation angle takes a maximum value in the first story of a 5-story building, due to seismic load corresponding to C_0=0.2. According to horizontal loading tests on a full-scale 3-D test specimen, however, that value was as small as 0.38×10^{-4} rad. Considering this fact, it is not generally conceivable that cracking would occur at the ends of a bearing wall under the stress corresponding to C_0 = 0.2. Therefore, on the premise of an appropriate bearing wall arrangement of little eccentricity, it is not generally conceivable that the tensile stress in flexural reinforcement would reach 295 N/mm^2 （the minimum value of allowable tensile stress for temporary loading in flexural reinforcement of the various materials） even if taking into account the stress concentration (see the Commentaries on Clause 2, Article 4) due to the difference in aspect ratio of bearing walls (note: assuming that the factor of stress concentration is ϕ, the allowable inflection point height is $y_{a0}h/\phi$). The prescribed flexural reinforcement is required in order to keep the stresses of the primary structural elements within the elastic limit in the case of earthquakes that are likely to be encountered several times during the service life of the building, as well as to ensure the necessary horizontal load-carrying capacity and to increase the ductility so that the building will not collapse in the case of the expected maximum earthquake (an acceleration of ground surface of 3 to 4m/sec^2) that is likely to occur during its service life.



***C.* Fig. 5.4 Allowable inflection height, $y_{a0}h$ of bearing walls on each story at C_0 = 0.2**

(The figures are given for normal concrete, $L_i = L_{0i}$ and the allowable tensile stress, $f_{t(\text{temporary})}$ = 295 N/mm^2 are assumed. The roman numerals denote the order of story from ground level.)

Explanation above is given for normal concrete, assuming wall length ratio Li of each story to be specified wall length ratio. In case where light weight concrete is used or where L_i is larger than $L_{0i.}$ the values of $\bar{\tau}_i$ are smaller than any of the values of $\bar{\tau}_{0i}$ in Commentary Table 4.3 (see the footnote in Commentary Table 4.3). Therefore, the value of y_ah of bearing walls in each story is larger than the value of $y_{a0}h$ in Commentary Figure 5.4 and the tensile stress in flexural reinforcement is smaller than that in both cases where normal weight concrete is used and where L_i = L_{0i}.

In case where the roof has a slope, heights up to 4 m are permitted from the top surface of the floor slab on the top story to the eaves. Considering the value of $y_{a0}h$ of bearing walls on the top story, it is stipulated that the height of the edge of an opening should be 2.8 m or less. In the design, rigid care must be given to the installation position of wall girders so as not to allow the height of the edge of an opening to exceed 2.8 m.

From the above, it may be thought that the flexural reinforcement quantity of bearing walls in each story given in Table 4 of this article is stipulated so as to keep the tensile stress in flexural reinforcement within $f_{t(\mathrm{temporary})}$, in accordance with Enforcement Order No.82(stress intensity and so forth).

Flexural Reinforcement 2-D13 for top story in Table 4 should be provided as double reinforcing bar arrangement, to resist to out-of-plane flexural moment to some extent. Commentary Figure 5.4 clearly shows that even 1-D13 is adequate for in-plane flexural moment. Therefore, bearing walls having a *T, L,* + or other plan type that can effectively withstand flexural moment orthogonal to the wall plane need not contain a double flexural reinforcing bar arrangement in an unreasonable manner if the wall thickness is 180 mm or less. (For walls more than 180 mm thick, double reinforcing arrangement is required by Clause 8.)

Under Proviso of Clause 2 Article 4, on such stories that the total wall length, $\sum l$, divided by the floor area, S_i, for calculating wall length ratio on that story is smaller than the value of L_{0i} given in Table 1 of Article 4, those bearing walls in the calculation direction whose thickness t is larger than the prescribed wall thickness t_{0i} must have flexural reinforcement with the amount of the "flexural reinforcement of the bearing wall" given in Table 4 of this article overdesigned by the ratio of t/t_{0i}.

7. Reinforcement along the Horizontal Edge of Opening

The reinforcement along the horizontal edges of an opening plays a dual role as a flexural reinforcement in the wall girders above and under the opening and shear reinforcement in the horizontal direction and as a stiffener to maintain safety against the accompanied diagonal tensile forces at the corners of the opening.

The necessary cross sectional area and peripheral length of flectural reinforcement of wall girders above and below openings should be calculated in accordance with Article 6. When horizontal shear reinforcement (hereafter horizontal reinforcement) calculated by Section 3 and 4 of this Article, according to Section 2(3) of Article 6, is needed to provide along the edge of openings, it should be placed along with flexural reinforcement or bars having equal to or more than the sum of the sectional area of these.(see Commentaries to Section 3 of this Article and Commentary Figure 5.2)

For the required cross sectional area necessary for strengthening against accompanied diagonal tensile forces at the corners of an opening, only if diagonal reinforcement is not placed according to the proviso of Clause 9 of this article, the cross sectional area should be calculated according to the provisions of the clause. And a lapping bar of this cross sectional area or more should be placed at the corners of the opening or an extra cross sectional area of reinforcement at the horizontal edges should be added by the cross sectional area.

For small openings, the steel quantity thus calculated is small and is not appropriate for reinforcement around the opening. Therefore, use of D13 or more is stipulated irrespective of calculation results.

8. Reinforcing Bar Arrangement in Bearing Walls

Considering that bearing wall of 180 mm thick or more are used mostly in the stories except for the top story of 3-story or higher buildings, double reinforcing bar arrangement should be employed, to resist to out-of-plane bending moment to some extent, and to mitigate cracking due to drying shrinkage.

9. Reinforcing Bar Arrangement at Intersections between Bearing Walls and Floor Slabs, and Corners of Openings

Considering that horizontal reinforcing bars of bearing walls at intersections between floor slabs support reinforcing bars of floor slabs, D 13 bars or larger should be used to prevent dislocation of slab reinforcement during bar arrangement of floor slabs and concreting.

Since diagonal tension cracks occur at the corners of an opening, adequate reinforcement is required against this. Diagonal reinforcement orthogonal to the cracks is the most effective in strengthening. For walls 180 mm thick containing horizontal and vertical reinforcement in a double reinforcing bar arrangement, however, marked deterioration is presumed in concrete placing at the corners of the opening if a double diagonal reinforcing bar arrangement is also used. In such cases, considering that horizontal and vertical reinforcements at opening edges forming an angle of 45 degrees between diagonal cracking can contribute to strengthening against diagonal cracking by their respective cross sections multiplied by $1/\sqrt{2}$, it is stipulated as the proviso that no diagonal reinforcement need be placed if a cross sectional area $\sqrt{2}/2 = 0.70$ times or more, the necessary cross sectional area (cross sectional area half or more of the reinforcement shown in Table 4 of this article) of diagonal reinforcement, namely, 0.35 or more times the reinforcement shown in Table 4 of this article is added to the necessary cross sectional area of horizontal and vertical reinforcements respectively at the opening edges.

10. Treatment of Concentrated Stresses Acting on Bearing Walls

The point where a wall girder or a beam intersects a bearing wall should be carefully designed to carry torsional moment and shearing force. For this purpose, it is recommended to provide buttress or to make wall girders strong enough to carry torsional moment. When a buttress is not provided, double reinforcing bar arrangement should be employed to resist out-of-plane torsional moment. ACI standards specify the effective length of a bearing wall to carry concentrated load, as smaller one of the following two cases (See Commentary Figure 5.5.).

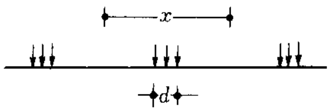
i) Distance *x* between concentrated loads on center

ii) $d + 4t$

where,

d : length of acting portion of concentrated load

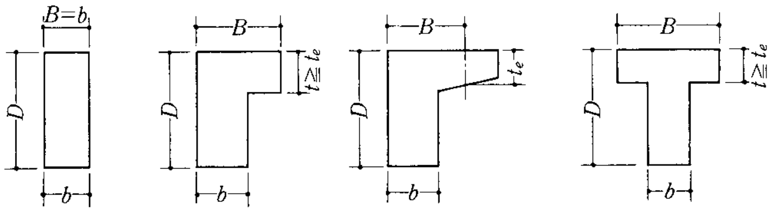
t : wall thickness



***C.* Fig. 5.5 Situation of concentrated vertical loads and effective length of a bearing wall**

Article 6 Structural Details of Wall Girders

1. At the top of bearing walls of each story, wall girders should be provided effectively and continuously.
2. The wall girders should be designed to have sufficient strength to carry horizontal and vertical loads, as well as to conform to the following requirements:
   (1) The width of wall girders should be equal to or more than the thickness of the bearing walls connected to the wall girders concerned.
   (2) The depth of wall girder should be 450 mm or more in principle.
   (3) The provisions of Section 8 of Article 5 should be applied to the arrangement of reinforcement of wall girders, and the provisions of Sections 3, 4 and 5 of Article 5 should be applied to horizontal and vertical reinforcements other than flexural reinforcement.
   (4) Wall girders not rigidly and monolithically connected to rigid roof or floor slabs should have an effective width *B,* equal to or greater than 1/20 of the distance between horizontal supports, and sufficient reinforcement should be provided so as to resist out-of-plane horizontal loads. For wall girders having an L- or T-shape cross section, the flange portion with a thickness *t* not less than 150 mm (120 mm for single-story building) may be included into the effective width *B* (see Figure. 1).



B	:	effective width of wall girder	*t*	:	flange thickness
b	:	wdth of wall girder	t_e	:	no less than 150 mm (no less than 120 mm for single-story building)
D	:	depth of wall girder			

Fig. 1 Effective width of wall girders

1. Role of Wall Girders

Portions above openings form wall girders. Wall girders play the role of connecting bearing walls at the top to form a structural frame. They generally have a thickness equal to or greater than bearing walls, and make sure the three dimensional effect of the entire building by connecting bearing walls and slabs.

Wall girders work together with foundation beams to prevent lifting and overturning of the bearing walls. Besides, wall girders transfer stresses of bearing walls to another and redistribute stresses to undamaged bearing walls, and prevent collapse of entire story when a bearing wall or bearing walls in one direction deteriorated vertical load carrying capacity due to development of shear cracks. As a result, RC box-shaped wall buildings have structural characteristics hardly allowing a story collapse compared with moment resisting frame structures. Experimental tests of RC box-shaped wall buildings having low shear reinforcement ratio of 0.15 to 0.25% show enough ductile performance during heavy earthquakes, without story collapse. The reason is considered to be attributed to integrity by three dimensional effect.

The reason why wall girders are provided for RC box-shaped wall structures is to support the dead and live loads of the slabs, and to transfer earthquake load to bearing walls. Therefore, independent multistory bearing walls such as end transverse walls or walls between dwellings need not flexural reinforcements at the level of wall girders.

2. Structural Details of Wall Girders

(a) Design Stress of Wall Girders

Since the important role of preventing or mitigating lifting up, rotating or turning over of bearing walls connected at the ends, which decreases shear resistance, is imposed on wall girders, they should be designed to have sufficient strength and rigidity. Design stresses for sustained loading are calculated for distributed loads of roof or floor slabs including dead load of wall girders themselves, under fixed end or semi-fixed end condition.

Design stresses for temporary loading are approximately calculated as following procedure: assuming the conditions of i) and ii), for design shear forces and height of inflexion point of bearing walls. The sum of bending moments of bearing walls above and bellow, at the intersection of the center lines of the bearing walls and the wall girders are distributed to wall girders in proportion to the rigidities of wall girders. Design stresses for horizontal loading are added to these values, thus design stress for temporary loading are obtained.

i) Shear force : $Q = t\, l\, \bar{\tau}$

where,

t : thickness of bearing wall,

l : length of bearing wall, and

$\bar{\tau}$: mean shear stress of bearing walls of the story concerned, obtained by dividing story shear force by the sum of cross sectional area of bearing walls in each direction. The values given in Commentary Table 4.3 may be employed when not calculated.
(It is called "mean shear stress method.")

ii)Height of inflexion point : center of the height of the portion having the same wall length (see Commentary on Clause 1, Article 5)

[Note] For a structural frame comprised of bearing walls and wall girders connecting them and having openings which are all large so that its shape approximates to an ordinary regular rigid moment resisting frame, the height ratios of inflexion point of the bearing walls on the top and bottom stories can be assumed to be 0.4 and 0.6, respectively. However, if the height ratio of inflexion point of the bearing wall on the bottom story is raised from 0.5 to 0.6, the flexural reinforcement quantity at the second story will be decreased. Therefore, it is safer to assume that all the height ratios of inflexion point to be 0.5, if the openings are small or the structural frame takes the form of an irregular moment resisting frame so that the positions of the inflexion points are ambiguous.

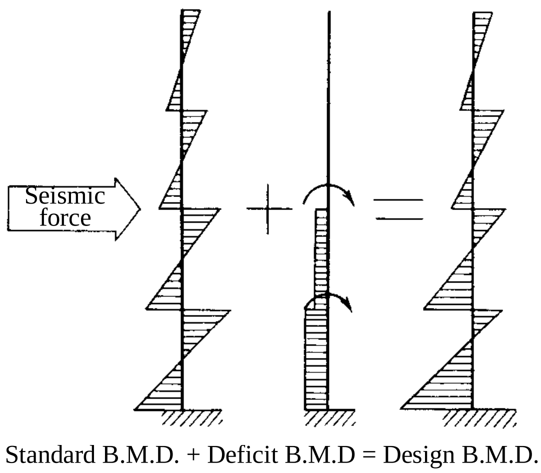
When the horizontal length of bearing wall is large, bearing walls carry large shear force which are transferred to wall girders. It is desirable that the wall girders carry whole amount of bending moment transferred from the bearing wall. When it is difficult to arrange required reinforcement in wall girder, the width of the wall girder should be increased. Even if it is impossible to arrange the sufficient amount of reinforcement, bending moments to compensate for the shortage of reinforcement should be added to the lower bearing wall, and redistributed to the wall girders at the bottom of the bearing wall (see Figure 6.1 and the Commentaries on Clause 6, Article 5). In addition, do not fail to carry these deficits of moment completely by the wall girder of the lower story or the foundation beam of the bottom story.

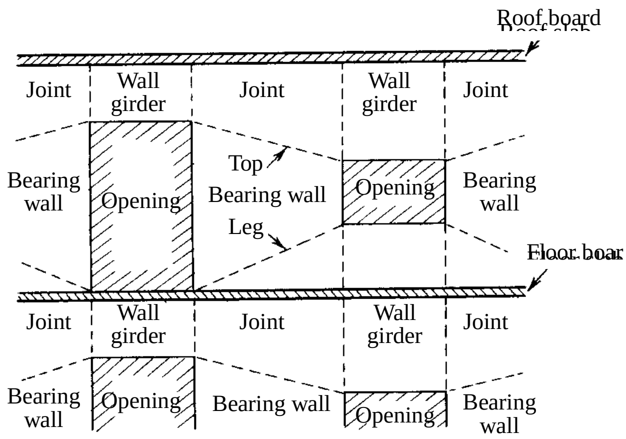
A structural frame of RC box-shaped wall buildings can be considered as a monolithic structural frame comprised of bearing walls, wall girders and joints between them (see Commentary Figure 6.2). Therefore, if reinforcement is placed so as to transmit the in-plane stresses at the ends of the wall girder to the joints in an adequately safe manner against both sustained and temporary loads, the structural frame should be such construction as to conform provision "wall girders should be provided effectively and continuously at the top of the bearing walls in each story" in Section 1 of Article 6.

(b) Arrangement of Reinforcement of Wall Girders

Since the tensile reinforcement ratio of wall girders is not generally greater than the balanced reinforcement ratio, the principal role of flexural reinforcement is to carry the tensile stresses which are induced when the concrete becomes unable to transmit the tensile stresses after concrete sections are cracked and to ensure safety against the in-plane stresses described before. Therefore, the arrangement of reinforcement in the joint must be determined considering what kinds of cracks will be expected to occur in the joint.

Since the wall thickness at the joint between bearing walls and wall girders of RC box-shaped wall structure is equal to the thickness of the wall girder and bearing wall in many common cases, it is necessary to assume the possibility of cracking due to end stresses in the wall girder to penetrate into the joint or to occur in the joint (see Commentary Figure 6.3). The penetration or occurrence of such cracks depends on the end stresses (M, Q) of the wall girder as well as on the plan type of joint (+, T, or L), geometry of joint (aspect ratio), presence of wall girders or orthogonal bearing walls, and the positions of orthogonal members. Accordingly, for the anchorage of flexural reinforcement of the wall girder in a thin joint of RC box-shaped wall structure, the care which is different from the ordinary anchorage methods in a beam-column joint of RC moment resisting frames should be paid.





C. Fig. 6.1
Treatment of deficit bending moments on wall girder

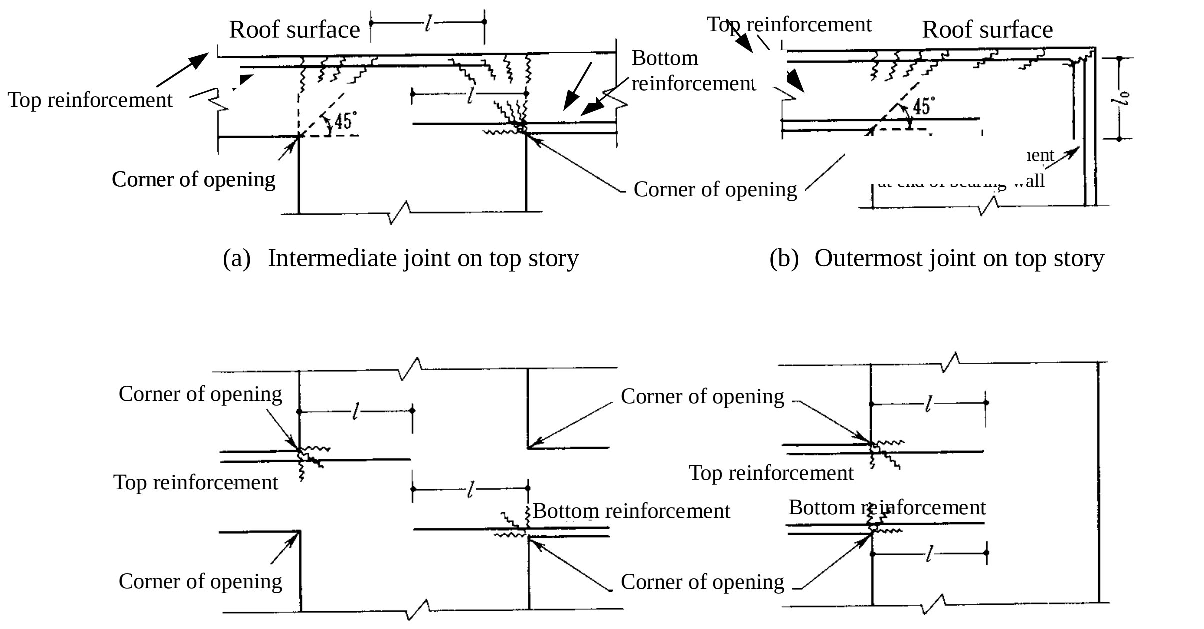
C. Fig. 6.2
Division of a structural frame for considering anchorage of wall girder's flexural reinforcement

Since RC box-shaped wall structures are composed of thin members, difficulties exist in dense arrangement of reinforcement in work execution compared to the ordinary RC rigid moment resisting frames. It is necessary to pay attention to ensuring ductility by placing shear reinforcement within the wall girder and the bearing wall or otherwise as well as decide arrangements of reinforcement in each story taking due care of securing large horizontal load-carrying capacity.

For the top reinforcement in the wall girder in the top story, in case where it is anticipated that cracking will occur at the upper edge of the joint and propagate toward a corner of an opening forming an angle of 45 degrees to the horizontal, the anchorage must be effected by a length of the reinforcement that is longer than the necessary development length l and lies within the portion beyond the fracture surface at 45 degrees. In addition, in the exterior joints, a splice longer than the necessary splicing length l_0 must be arranged at the end of the bearing wall (see Commentary

Figure 6.3). For the exterior joints, a line drawn from a corner of an opening at 45 degrees from the horizontal will intersect with the vertical edge of the joint if the height of joint is taller than the width. In such cases, the top reinforcement of the wall girder must be bent down at the vertical edge of the joint to arrange a splice longer than l_0 at the end of the bearing wall.

For the bottom reinforcement of wall girders on the top story and the top and bottom reinforcements of wall girders on the middle stories, they all intersect at right angles at the corners of the opening as well as the flexural reinforcement of the bearing wall and these two reinforcements at right angle to each other can resist against the cracking that can be expected to occur at a corner of an opening. Therefore, it is sufficient to anchor the reinforcement in a length longer than the necessary length l measured in the portion beyond the corner of the opening. If in a joint between the bearing wall and wall girders the flexural reinforcements of the right and left wall girders overlap, it is desirable to arrange continuous reinforcements through the bearing wall for good concrete casting in the joint.



(a) Intermediate joint on top story (b) Outermost joint on top story

(c) Intermediate joint on intermediate story (d) Outermost joint on intermediate story

C. Fig. 6.3 Assumed cracking in concrete in relation to the anchorage of flexural reinforcement of a bearing wall in the joint between the bearing wall and a wall girder

It is considered that stress concentration of bond stresses in the anchorage portion occurs and no restraint effect of orthogonal members or hoops can be expected against and bond splitting of covering concrete in the anchorage portion. The necessary development length l is obtained by Equation (6.1).

$$l = \frac{\sigma_t \cdot a_t}{0.8 f_a \cdot \varphi} \tag{6.1}$$

where,

σ_t : maximum stress present in reinforcement in anchorage or splicing portion. The allowable stress of the bar should be taken for this in principle. If a hook is made at the end of the bar, 2/3 of that value may be employed.

a_t : cross sectional area of bar,

φ : perimeter of bar, and

f_a : allowable bond stress which is the value* of the top reinforcement irrespective of the position of the reinforcement.

[Note] * For deformed bar, there are cases where the concrete covering depth over that bar is less than 1.5 times the bar diameter because the joint is thin. In such cases, design must be executed paying attention to the provision of the RC Standard, namely, "f_a shall be the standard value multiplied by the covering depth/(1.5 times bar diameter)."

Since stress concentration of bond stresses in a splicing portion occurs at both ends of the splice, the bond stress distribution in the splicing portion is more levelled off than that in the anchoring portion. Considering this fact, the necessary splice length l_0 is derived using the following formula:

$$l_0 = \frac{\sigma_t \cdot a_t}{f_a \cdot \varphi} \tag{6.2}$$

(c) Structural Requirements of Wall Girders

For the wall girder, conformance to the following limits 1) to 4) is stipulated besides the calculations above.

1) Thickness of Wall Girder

Since the wall girder generally is apt to have a smaller rigidity than the bearing wall, it is inappropriate to have a wall girder thinner than the adjacent bearing walls from the view point of an additional purpose of having the inflexion points of bearing walls at their centers.

2) Depth of Wall Girder

Based on the objective in the previous paragraph, it is also stipulated that the depth of a wall girder should be 450 mm or more in principle.

3) Shear Reinforcement in Wall Girder

The shear reinforcement (horizontal and vertical reinforcements except the flexural reinforcement) in a wall girder should be placed in the same manner as the shear reinforcement in the bearing walls connected to it. That is, the wall girder must also satisfy the reinforcement provisions about horizontal and vertical shear reinforcements given in Table 3 as well as an adequate amount of reinforcements must be placed against shear stresses for sustained and temporary loadings of wall girders.

4) Effective Width of Wall Girder not Integral with Rigid Roof or Floor Slabs

RC box-shaped wall buildings make it a point to install rigid roof or floor slabs. If they are not rigid under the proviso of Article 7, strengthening is required by installing a wall girder whose effective width (horizontal width) is 1/20 or more of the distance between structurally important horizontal supports (namely, adjoining two planes of structure perpendicular to the structural frame of that wall girder) for ensuring adequate safety against out-of-plane bending due to horizontal forces.

Article 7　Floor and Roof Construction

Floors and roofs should be reinforced concrete slabs, and should be firmly connected to bearing walls or beams except for slabs of ground floor (other than constructed on soft soil) and, roofs of 2- or single story buildings.

Floors and roofs play very important roles of transferring earthquake loads to bearing walls. Therefore, they should be rigid structures. Slabs of the ground floor of RC box-shaped wall buildings are sometimes constructed employing wooden framework. But, the wooden framework of this type is not considered to be desirable from structural engineers' viewpoint. Rigid floor slabs prevent in-plane relative displacement and rotation and work together with foundation beams, to increase flexural rigidity, and to prevent or mitigate differential settlement. Therefore, floors of the ground floor should be composed of reinforced concrete slabs or rigid prefabricated reinforced concrete slabs.

Considering that 2- or single story buildings generally have sufficient margin load carrying capacity (See Commentary Table 4.3 and Fig. 5.4), the roofs may be constructed with wooden framework.

Article 8　Foundation

Foundations should be designed and constructed so as to satisfy following requirements.

1. Under the bearing walls of the lowest story of the building concerned, foundation beams should be effectively provided.

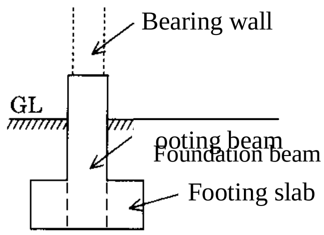
2. Foundations should have sufficient strength to carry both vertical and lateral forces. Besides, they should be in accordance with the following requirements.

    (1) The width of foundation beams should be equal to or wider than the thickness of the bearing walls above.

    (2) The main reinforcement of foundation beams should satisfy the requirement of Article 5.8, and the other reinforcement should satisfy the requirement of Article 5.5.

1. Continuous Foundation and Foundation Beam

In the planes of bearing walls, continuous foundation beams are provided, under bearing walls to enhance rigidity of the structure. The continuous foundation is composed of two parts, foundation beam and footing slab (See Commentary Fig.8.1).



C. Fig. 8.1 Cross section of continuous foundation

When soil is too soft to have sufficient bearing capacity, pile foundation should be employed. In this case, foundation slabs may be provided only in the vicinity of piles, and other parts may be bare foundation beams without foundation slabs.

In case of direct foundation, foundation beams without foundation slabs may be allowed under wide openings in order to prevent upward bending due to reaction forces from soil.

Continuous foundation beams work to enhance overall rigidity of the building, and are effective to prevent or mitigate differential settlement.

2. Principle of Foundation Design

In case of direct foundation, the area of foundation slab should be sufficient enough to carry both sustained and temporary loads transferred from the superstructure of the building concerned.

Niigata Earthquake of June, 1964, and the Nihonkai Chubu Earthquake of May, 1983 caused overturning and tilting of 4- and 2-story RC box-shaped wall apartment buildings, respectively, and made them useless (See Photo 8.1).

In these cases, soils were fine sand, which were liquefied due to earthquake motion. An adjacent four-story apartment building with semi-basement for storage space was quite all right, in the former case. Deep embedment such as the establishment of the basement, would be effective to prevent tilting. Be careful to fine sand and high water table which is apt to be liquefied. Soil condition should be surveyed by boring beforehand.

Phot. 8.1 **Prefectural 4-story apartment houses of RC box-shaped wall building in Kawagishi-cho, Niigata City, which turned over in the Niigata Earthquake in June 1964**

When the soil condition is not considered good enough to support the building concerned, pile foundation should be employed. Piles should not be provided under wide openings, but should be provided under bearing walls carrying especially large axial load. Pile heads should be tightly connected to foundation slabs, and should be carefully designed not to be damaged, due to lateral loads. Besides, foundation beams should be designed to carry in-plane shearing forces and bending moments.

Foundation beams should be designed according to the following requirements.

(1) The Width of Foundation Beams

It is desirable to make foundation of bearing walls to be fixed condition at foundation, (the fixed end condition may be adopted for calculating stresses of the beams of upper stories). Therefore, to give sufficient rigidity to foundation beams, the width of foundation beams should be larger than the thickness of bearing walls above.

(2) Shear Reinforcement of Foundation Beams

The shear reinforcement of foundation beams should be provided in consideration of soil pressure due to reaction and earthquake loads and other loads, if any. Besides, reinforcement should conform to the following requirement. The diameter of shear reinforcement of foundation beams should be D10 or larger, and the spacing should be 300 mm or shorter (In case of double arrangement, 450 mm or shorter for one side.) When the width of a foundation beam is 180 mm or larger, double arrangement should be provided.

Article 9 Construction Practice

> Construction of RC box-shaped wall structure in accordance with this standard should satisfy AIJ Standard Specification 5 Reinforced Concrete Works (Abbreviated as JASS[1]) and "Guideline for Reinforcing Bar Arrangement of Wall-shaped Buildings." Maximum aggregate size for concrete should be 20mm or smaller.

RC box-shaped wall buildings is a kind of RC structure in nature, consequently they should be in accordance with "JASS 5 Concrete Works" and the "Guideline for Reinforcing Bar Arrangement[2]," in general. But, considering the following characteristics particular to wall-shaped structures,"Guideline for Reinforcing Bar Arrangement of Wall-shaped Buildings[3]" was published, because of the following items.

1) RC box-shaped wall buildings are excluded from "Guideline for Reinforcing Bar Arrangement.

2) At the corners of openings, diagonal reinforcing bars are generally provided. Spaces between reinforcing bars at such parts are not wide enough to fill up with concrete.

3) The quality of compaction of concrete for walls and beams affects the structural performance of RC box-shaped wall buildings.

Considering these points, the maximum size of a coarse aggregate is decided to be 20 mm to prevent the void.

Precision of cutting and bending of reinforcing bars is very important for both structural performance and durability of RC box-shaped wall buildings. Reinforcing bar arrangement at the specified position makes bars and concrete work well together, and keeps precise cover thickness for assuring durability of structures.

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[1] AIJ: Japanese Architectural Standard Specification, Apr.1993

[2] AIJ: Guideline for Reinforcing Bar Arrangement, Sept. 1986.

[3] AIJ: Guideline for Reinforcing Bar Arrangement of Wall-shaped Structures, Nov.,1987

Appendix A Historical Earthquake Damages

In Japan, significant damages of wooden structures and reinforced frame structures have been caused by many earthquakes. There were some reinforced concrete box-shaped wall buildings, composed of bearing walls, slabs and beams, in the areas where the 1964 Niigata Earthquake, the 1968 Tokachi-oki Earthquake, the 1975 Ohita-ken Chubu Earthquake, and the 1978 Miyagi-ken Oki Earthquake caused heavy damages.

References 1)～3) give the damages to RC box-shaped wall buildings due to these earthquakes. These buildings were built between 1954 and 1977, and the configurations were of simple and regular shape. The 1995 Hyogo-ken Nanbu Earthquake whose JMA intensity was VII caused significant damages to many structures including steel structures.

Systematic investigation of damaged RC box-shaped wall buildings was conducted. The followings are the brief summary of the damages to RC box-shaped wall buildings, due to strong earthquakes.

(1) During the 1964 Niigata Earthquakes, some buildings were overturned or tilted due to liquefaction, and some of which were damaged in superstructures, but not so seriously. (See Table 1)

(2) During the 1968 Tokachi Oki Earthquakes, a two-story house under construction was heavily damaged, but four buildings nearby were not damaged at all. (see Table 2)

(3) During the 1975 Ohita-ken Chubu Earthquakes, whose hypocenter was shallow and just below this area, a four-story hotel of reinforced concrete frame structure with basement was heavily damaged, but a two-story RC box-shaped wall building was not damaged at all.

(4) After the 1978 Miyagi-ken Oki Earthquakes, all the reinforced concrete structures in northern, southern and western part of Sendai were investigated. They were 128 four- or five- story apartment buildings, owned by City of Sendai, The Japan Housing Corporation, and three private enterprises, 38 one to three story buildings owned by public agencies, and 169 in total.

The result is shown in Tables 3. and 4. The slightly damaged structures had narrow shear cracks. They were built on reclaimed areas of formerly slopes.

(5) During the Hyogo-ken Nanbu Earthquakes, many reinforced concrete frame structures designed according to the AIJ Standard for Structural Calculation of Reinforced Concrete Structures were heavily damaged or collapsed, but no heavily damaged RC box-shaped wall buildings were observed, among more than one thousand investigated buildings. Only nine buildings (0.9 % of all investigated) were slightly damaged. The number of buildings

of this type with any damages including very slight ones were 40, which were about 4%. These numbers are amazingly small for the heaviest level of earthquakes. Most of the damages were related to soil failures, such as sliding, liquefaction, tilting, settlement, and breakage in foundation piles, and subsequent to the cracking of walls.

Table 1 Damage of Superstructure of RC wall buildings caused by the Niigata Earthquake

Story / Damage Level	5	4	3	2	1	[Buildings] Sum
No Damage*	0	16	13	6	0	35
Slight Damage**	0	5	10	3	2	20
Light Damage***	0	0	0	1	0	1

* No apparent damage

** Crack can be observed in the wall girders or bearing walls.

*** Crack can be observed in wall girders or bearing walls. Large crack can be observed in some members.

Table 2 Damage of Superstructures of RC wall buildings caused by the Tokachi-oki Earthquake

Story / Damage Level	5	4	3	2	1	[Buildings] Sum
No Damage	0	2	2	0	0	4
Heavy Damage	0	0	0	1*	0	1

* Under Construction

Table 3 Damage of Superstructure of RC wall buildings caused by the Miyagi-ken Oki Earthquake

Story / Damage Level	5	4	3	2	1	[Buildings] Sum
No Damage	80	40	18	11	6	155
Slight Damage	8	2	3	0	0	13
Light Damage*	1	0	0	0	0	1

*Partial damages at construction joints in shorter direction of the structure

Table 4 The list of damaged RC Wall buildings caused by the Miyagi-ken Oki Earthquake

Name of sites	Building No.	Longitudinal direction	Type of damage	Damaged Level
Minami-Kaji Cho	South	E-W	Horizontal displacement at construction joint of bearing walls in shorter direction(1st, 2nd floor)	slight
Shiroumaru Higashi	4-4	E-W	One directional S.C .of bearing walls in shorter direction (the upper stories)	Slight
Tsurugaoka No.2	5A-17	N-S	One directional S.C .of bearing walls in shorter direction (the lower stories)	Slight
”	5A-18	E-W	One directional S.C. of bearing walls in shorter direction (East side)	Slight
”	5A-19	E-W	One directional S.C. of bearing walls in shorter direction (the upper stories)	Slight
”	5A-28	E-W	One directional S.C. of bearing walls in shorter direction (the upper stories)	Slight
”	5A-29	E-W	One directional S.C. of bearing walls in shorter direction (the upper stories)	Slight
Tsurugaya	2	E-W	One directional S.C. of bearing walls in shorter direction (the lower stories) One directional S.C. of exterior walls in longitudinal direction	Slight
”	5	E-W	Horizontal displacement at construction joint of bearing walls in shorter direction (1st floor)	Light
Tsurugaya Go-chome	8	E-W	Both directional S.C. of bearing walls in shorter direction (the lower stories)	Slight
”	11	E-W	Both directional S.C. of bearing walls in shorter direction (the lower stories)	Slight
Kimachi Apartment		E-W	Shear crack (hair cracking) in bearing walls	Slight
Odakyu Apartment		E-W	Shear crack (hair cracking) in bearing walls	Slight
Uesugi Apartment		E-W	Shear crack (hair cracking) in bearing walls	Slight

*S.C. = Shear Cracking

Appendix B Earthquake Response of RC Box-shaped Wall Buildings

1. Objective

The structures having high rigidity such as reinforced concrete box-shaped wall buildings composed of walls, slabs and wall girders (herein after RC box-shaped wall buildings) have been scarcely damaged during recent destructive earthquakes in Japan (See Appendix A). The reason for this fact is considered that the sufficient bearing walls having relatively continuous lateral rigidities between adjacent stories provide high resisting capacity against torsion and lateral loads. Here the various response analyses of five concentrated mass elasto-plastic models, having various parameters of initial stiffness, rigidity ratios and hysteresis characteristics, are calculated, and the relation between the initial rigidities and the maximum displacements is discussed.

2. A Five Lumped Mass Non-Linear Earthquake Response Model

1) The Method of Modeling and Analysis

A five-concentrated mass non-linear shearing model is shown in Fig.1. All the masses are the same (m_i =1.0tf), and every story height is h=3m, and the base is fixed. The elastic stiffness of each story is decided so that the inter story drift angle ($R_{0.2}$) should be 1/2000 as standard, when the distribution of design shear forces is A_i distribution and C_0 =0.2. Standard shear coefficient at yielding (C_0) is assumed to be 0.5. The earthquake waves employed for analysis are EL Centro 1940 NS, Taft 1952 EW, and Hachinohe 1968 N.S.

The integration method employed is linear acceleration method. Damping is assumed to be proportional to initial rigidity, and the coefficient h=0.03

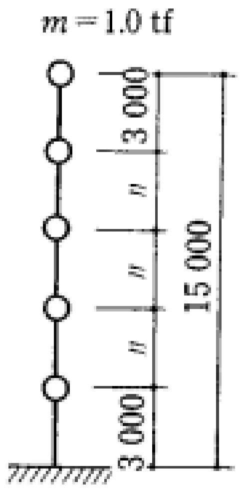


Fig. 1 Mass and story height

2) Comparison of the analytical results with full scale model experiments conducted in the past

The shear deformation angle is given as,

$$Rs = \frac{1.5Q}{GAw}$$

The inter story drift angle due to flexure is given as,

$$Rb = \frac{\delta b}{h} = \frac{Q}{3EI \cdot h} \times (h^3 + (h - h_0)^3$$

where,

Q : applied shear force,

Aw : the cross sectional area of bearing wall= $t \cdot l$,

G : elastic shear modulus,

h : story height, and

h_0 : height of Inflection point.

Replacing the following values,

$Aw = t \cdot l$

$G = E/2.3$ (E: Young’s modulus)

$y_0 = h_0\ h$

into previous equations,

$$Rs = \frac{1.5Q}{Gtl,}$$

$$Rb = \frac{Qh^3}{\frac{2.3}{4} Gtl^3 h} (y_0^3 + (1 \quad y_0)^3) = \frac{4Q}{2.3Gtl} (\frac{h}{l})^2 (1 \quad 3y_0 + 3y_0^2)$$

Assuming y_0 =0.5, where R_b takes a minimum value

$$Rb(y_0 = 0.5) = \frac{1Q}{2.3Gtl} \times (\frac{h}{l})^2$$

Consequently, the ratio of shear deformation to flexure deformation is expressed as,

$$\frac{Rs}{Rb(y_0 = 0.5)} = 3.45 \times (\frac{l}{h})^2$$

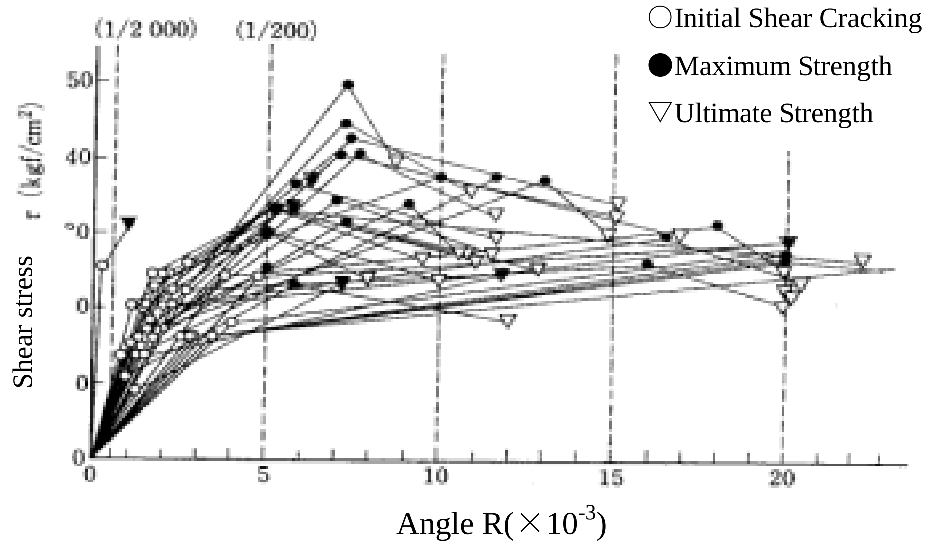


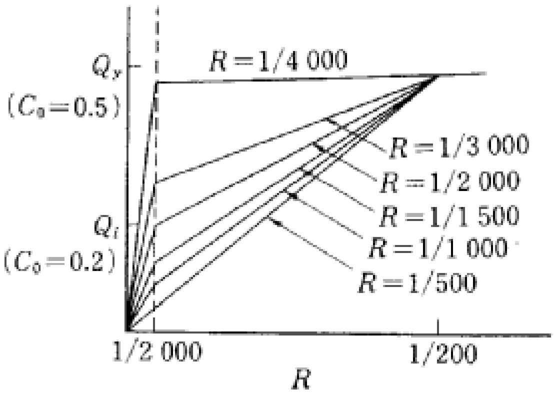
Fig.2 Inter-story Drift Angle

When the wall length ratio in one direction of an RC box-shaped wall building is $150mm/m^2$, mean wall length is usually about 1.5m, and $l/h \fallingdotseq 0.54$, and $R_s \fallingdotseq R_b$. According to AIJ RC Calculation Design Standards[1)], mean value of R_s at the initial shear cracking of bearing walls is about 1/4000. Considering that flexural deformation $R_b = R_s$, the inter story drift angle of a RC box-shaped wall building may be considered to be about 1/2000.

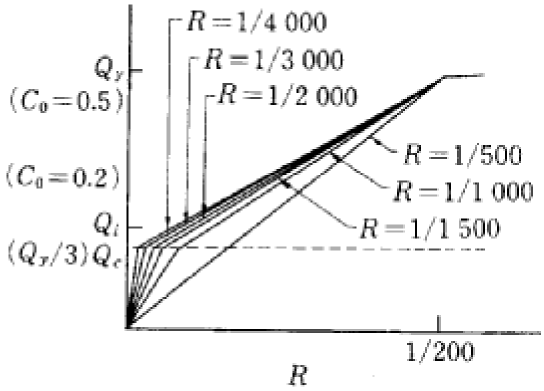
Figure 2 shows the results of experiments of 37 rectangular bearing wall model specimens[1)].The vertical axis shows shear stresses, and horizontal axis shows inter story drift angles. The initial shear cracking stresses, maximum shear stresses, and ultimate shear stresses are shown on the plotted curves. Some variances are observed, but it may be said that the inter-story drift angles at the initial shear cracking are 1/1000～1/500, and are mostly 1/200～1/130 at the maximum shear stress. From these, it may be concluded that most specimens do not show shear cracks, when inter story drift angle is below 1/2000, and that shear failure does not occur below 1/200. From these facts, it may be a conservative assumption to model the restoring force so as to have the first flection point at the inter story drift angle at 1/2000, and the second flection point at 1/200, for the purpose of this analysis.

3) Earthquake Response Analysis by Changing Initial stiffness (Analysis 1)

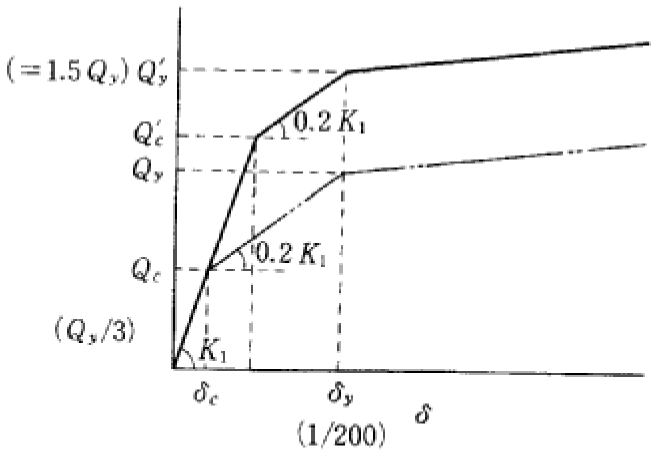
The initial rigidity is the parameter for the analysis. Inter story drift angles Ri at $C_0 = 0.2$ are assumed to be1/4000, 1/3000, 1/2000, 1/1500, 1/1000, and 1/500. The natural periods of vibration are 0.218, 0.252, 0.308,0.356, 0.436, and 0.617 sec., respectively. The inter story drift angle at the first flection point (initiation of cracking) of each story is set at *R*=1/2000. Analysis 1 is shown on Figure 3(a). When story shear stress at the first flection point is set to *Qy/*3, the results of the Analysis 1-2 are shown in Figure 3(b). Hysteresis characteristics is Trilinear, and bilinear up to *R*=1/200. The maximum input earthquake velocity is 50cm/sec.



(a)Analysis 1-1



(b)Analysis 1-2



Shearing force-Story drift angle relationship of the Story with Original wall Arrangement

Shearing force-Story drift angle relationship of the Story with Increased wall Arrangement

K= Elastic stiffness

Q_c' =7/9 Q_y'

Q_y'=1.5Q_y

(c)Analysis 2

Fig. 3 Shearing force -Story drift angle relationship

1) Building Research Institute: BRI Report No.76, The Strength and Ductility of Reinforced Concrete Members, March, 1977

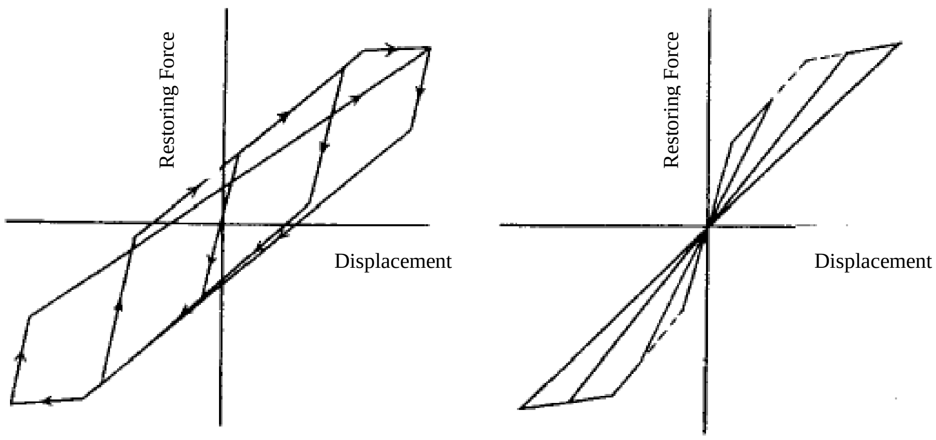
4) Earthquake Response Analysis by Changing Rigidity Ratios (Analysis 2)

In this analysis parameter is rigidity ratio. The inter story drift angle of the story of lower rigidity at C_0 ,=0.2, $R_{0.2}$ is set to be 1/2000, and the rigidities are set so as to be Rs =0.3. The ratio of the second slope to the first slope is assumed to be 0.2. Consequently, the inter story drift angle at the ultimate state became about 1/200. The ultimate lateral load carrying capacity of the story where the stiffness ratio is 0.3 is assumed to be 1.5 times as much as that of the standard value.

The framework of restoring force is set so as to the deflection at yielding be 1/200, without the changes of the second slope of the story with lower rigidity. (See Figure 3(C).) Restoring force of Type D-Tri (Tri-linear) is shown in Figure 4(a). Analysis of case 0～6 of Figure 5 is conducted, when the input earthquake velocity is 500mm/s. The natural periods of vibration of the first mode are 0.308, 0.211, 0.208, 0.203, 0.195, 0.181, and 0.297 sec., respectively.

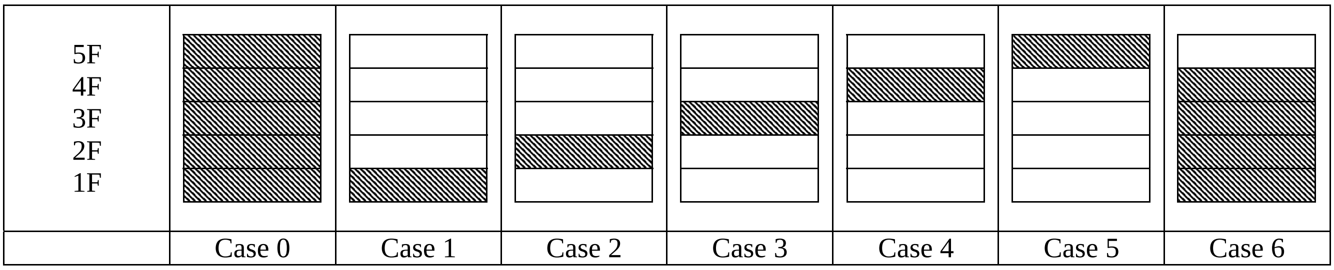
(5) Earthquake Response Analysis by Changing Histeresis Characteristics (Analysis 3)

The model having the origin oriented hysteresis characteristic of Case 0 (Standard Type) of Analysis 2 as shown in Figure 4(b) is analyzed. The input earthquake acceleration is set to be 4, 3.5, and 3m/sec^2.



(a) D-Tri Type (b) Origin oriented

Fig. 4 Type of restoring forces



Story of Low Stiffness with Interstory Drift Angle of 1/2,000

Story where interstory angle is set so that the stiffness ration be 0.3 (Inter story drift angle : 1/7,833 (case 15)1/25,333(case 6)

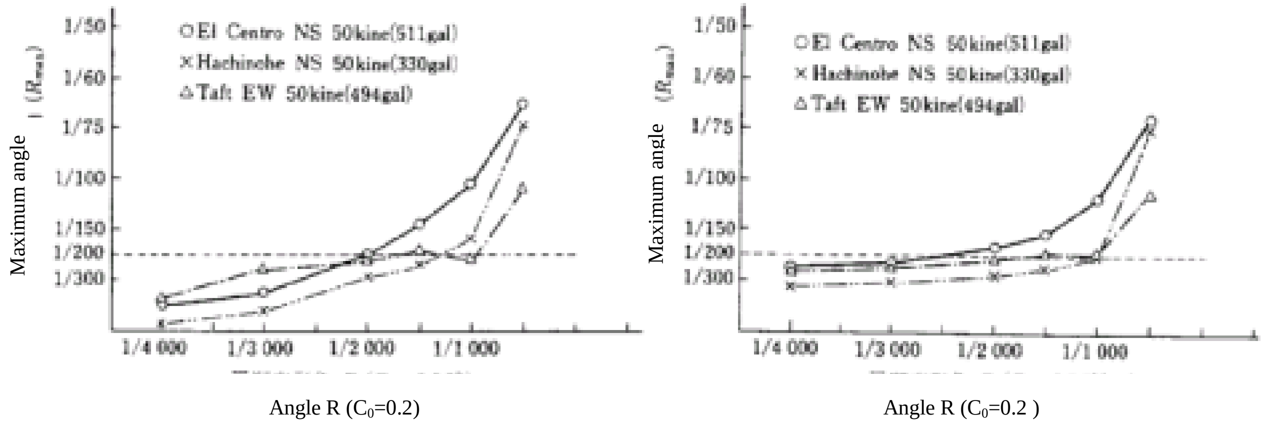
Fig. 5 Analyzed cases (Analysis 2)

3. The Results of the Analysis

The relation between the maximum inter-story drift angle and the inter-story drift angle at C_0 =0.2,($R_{0.2}$), for Analysis 1-1 and 1-2 is shown in Figure 6(a) and Figure 6(b). The responses to three input waves showed similar tendencies. When $R_{0.2}$ is smaller than 1/2000 for the input earthquake velocity of 50cm/sec, the maximum inter story drift angles are mostly below 1/200. When $R_{0.2}$ is increased to more than 1/2000, the maximum inter story drift angles abruptly increase. The results of Analysis 2 are shown in Figure 7, and Analysis 3 in Figure 8. In the case of Analysis 2 (inter-story drift angle of each story is 1/2000 for the primary design), Case 0 shows the largest inter story drift angle. While the maximum inter story drift angle for El Centro 1940 NS shows 1/179, but below 1/200 in most cases. In the case of Analysis 3, the maximum inter story drift angle for Hachinohe 1968 NS shows excessive values of 1/69 and 1/67 to the input earthquake acceleration of 3.5m/sec^2 and 4m/sec^2, respectively, but other results are below 1/200 at 3m/sec^2. When the maximum acceleration is 3m/sec^2, the maximum velocities for El Centro 1940 NS, Taft 1952 EW, and Hachinohe 1968 NS are 294 mm/s., 302 mm/s., and 454 mm/s., respectively.

4. Conclusions

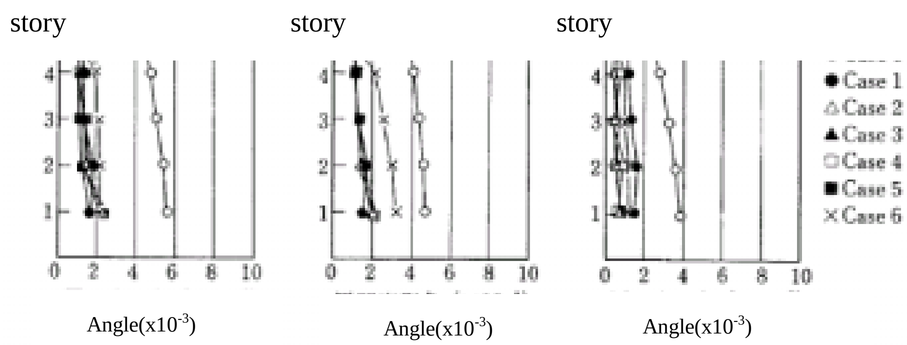
In the case of Analyses 1 and 2, a five mass non-linear model is analyzed parametrically for the maximum earthquake velocity of 500 mm/s. The results show that most of the maximum inter story drift angles are below 1/200, when $R_{0.2}$ is below 1/2000. In the case of Analysis 3, the model is analyzed changing hysteresis characteristics. The results show that maximum inter story drift angles in the case of origin oriented hysteresis are observed larger than in the case of D-Tri type, even though the maximum inter story drift angles are below 1/200, when maximum input earthquake acceleration is below 3m/sec^2.



(a)**Analysis** 1-1

(b)Analysis 1-2

Fig. 6 Relation between initial stiffness and maximum inter-story drift angle



El Centro 1940 NS

Taft 1952 EW

Hachinohe 1968 NS NS

Fig. 7. Maximum inter-story drift angle (Analysis 2)

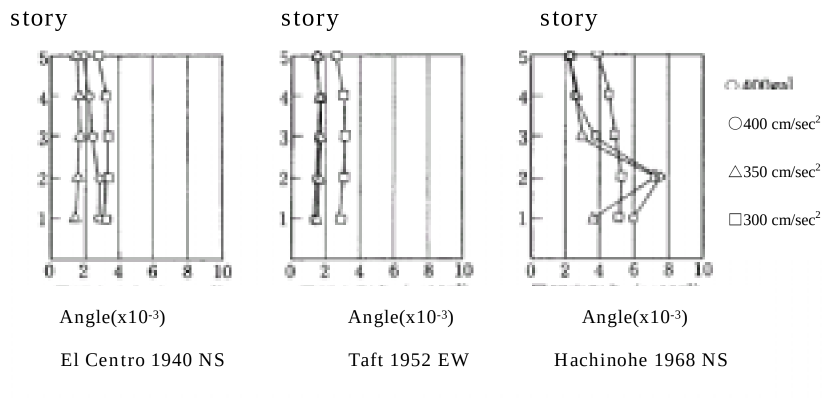


Fig. 8 Maximum inter-story drift angle (Analysis 3)

From these results, the following conclusion may be delivered. When a structure concerned has high stiffness of maximum inter-story drift angle of about 1/2000 for every story, and higher strength of shear factor of the first story at yielding more than 0.5, in primary design (C_0 =0.2), the maximum inter story drift angles are below 1/200 in most cases. Even in the case of origin oriented model which has low hysteresis energy consumption, s maximum response of inter story drift angle hows below 1/200 for the maximum input earthquake acceleration up to 3m/sec^2.

Therefore, it may be concluded that a five-story RC box-shaped wall building having an ultimate lateral load carrying capacity of D_s = 0.5, shear failure of the entire structure will be exempted, if inter-story drift angle of each story is kept about 1/2000.

Appendix C List of Reinforcing Bars

Cross Sectional Area and Perimeter of Reinforcing bars

Round Bars

Upper rows: Cross Sectional Area (cm^2), Lower rows: Perimeter (cm)

Dia (mm)	Weight (kg/m)	1-Ø	2-Ø	3-Ø	4-Ø	5-Ø	6-Ø	7-Ø	8-Ø	9-Ø	10-Ø
4	0.099	**0.13** 1.26	**0.25** 2.51	**0.38** 3.77	**0.50** 5.02	**0.63** 6.28	**0.75** 7.53	**0.88** 8.78	**1.01** 10.04	**1.13** 11.30	**1.26** 12.55
5	0.154	**0.20** 1.57	**0.39** 3.14	**0.59** 4.71	**0.79** 6.28	**0.98** 7.86	**1.18** 9.43	**1.37** 11.00	**1.57** 12.57	**1.77** 14.14	**1.96** 15.71
6	0.222	**0.28** 1.88	**0.56** 3.76	**0.85** 5.64	**1.13** 7.52	**1.41** 9.40	**1.69** 11.28	**1.98** 13.16	**2.25** 15.04	**2.54** 16.92	**2.82** 18.80
7	0.302	**0.38** 2.20	**0.77** 4.40	**1.15** 6.60	**1.54** 8.80	**1.92** 11.00	**2.31** 13.20	**2.69** 15.40	**3.08** 17.60	**3.46** 19.79	**3.85** 21.99
8	0.395	**0.50** 2.51	**1.00** 5.02	**1.51** 7.53	**2.01** 10.04	**2.51** 12.55	**3.01** 15.05	**3.51** 17.55	**4.01** 20.08	**4.52** 22.60	**5.02** 25.10
9	0.499	**0.64** 2.83	**1.27** 5.65	**1.91** 8.48	**2.54** 11.31	**3.18** 14.14	**3.82** 16.96	**4.45** 19.79	**5.09** 22.62	**5.73** 25.45	**6.36** 28.27
12	0.888	**1.13** 3.77	**2.26** 7.54	**3.39** 11.31	**4.52** 15.08	**5.65** 18.85	**6.79** 22.62	**7.91** 26.29	**9.05** 30.16	**10.18** 33.93	**11.31** 37.70
13	1.04	**1.33** 4.08	**2.65** 8.17	**3.98** 12.25	**5.31** 16.34	**6.64** 20.42	**7.96** 24.50	**9.29** 28.60	**10.62** 32.67	**11.95** 36.75	**13.27** 40.84
16	1.58	**2.01** 5.03	**4.02** 10.05	**6.03** 15.08	**8.04** 20.11	**10.05** 25.13	**12.06** 30.16	**14.07** 35.19	**16.08** 40.21	**18.09** 45.24	**20.11** 50.27
19	2.23	**2.84** 5.97	**5.67** 11.94	**8.51** 17.91	**11.34** 23.88	**14.18** 29.85	**17.02** 35.81	**19.85** 41.78	**22.68** 47.75	**25.52** 53.72	**28.35** 59.69
22	2.98	**3.80** 6.91	**7.60** 13.82	**11.40** 20.73	**15.21** 27.65	**19.01** 34.56	**22.81** 41.47	**26.61** 48.38	**30.41** 55.29	**34.21** 62.20	**38.01** 69.12
25	3.85	**4.91** 7.85	**9.82** 15.71	**14.73** 23.56	**19.63** 31.42	**25.54** 39.27	**29.45** 47.12	**34.36** 54.98	**39.27** 62.83	**44.18** 70.69	**49.09** 78.54
28	4.83	**6.16** 8.80	**12.31** 17.59	**18.47** 26.39	**24.63** 35.19	**30.79** 43.98	**36.94** 52.78	**43.10** 61.58	**49.26** 70.37	**55.42** 79.17	**61.58** 87.92
32	6.31	**8.04** 10.05	**16.08** 20.11	**24.13** 30.16	**32.17** 40.21	**40.21** 50.27	**48.26** 60.32	**56.30** 70.37	**64.34** 80.42	**72.38** 90.48	**70.42** 100.70

Deformed Bars

Upper rows: Cross Sectional Area (cm^2), Lower rows: Perimeter (cm)

Name	Weight (kg/m)	Dia (mm)	1	2	3	4	5	6	7	8	9	10
D6	0.249	6.35	**0.32**	**0.64**	**0.96**	**1.28**	**1.60**	**1.92**	**2.24**	**2.56**	**2.88**	**3.20**
			2.0	4.0	6.0	8.0	10.0	12.0	14.0	16.0	18.0	20.0
D10	0.560	9.53	**0.71**	**1.43**	**2.14**	**2.85**	**3.57**	**4.28**	**4.99**	**5.70**	**6.42**	**7.13**
			3.0	6.0	9.0	12.0	15.0	18.0	21.0	24.0	27.0	30.0
D13	0.995	12.7	**1.27**	**2.54**	**3.81**	**5.08**	**6.35**	**7.62**	**8.89**	**10.16**	**11.43**	**12.70**
			4.0	8.0	12.0	16.0	20.0	24.0	28.0	32.0	36.0	40.0
D16	1.56	15.9	**1.99**	**3.98**	**5.97**	**7.96**	**9.95**	**11.94**	**13.93**	**15.92**	**17.91**	**19.90**
			5.0	10.0	15.0	20.0	25.0	30.0	35.0	40.0	45.0	50.0
D19	2.25	19.1	**2.87**	**5.74**	**8.61**	**11.48**	**14.35**	**17.22**	**20.09**	**22.96**	**25.83**	**38.70**
			6.0	12.0	18.0	24.0	30.0	36.0	42.0	48.0	54.0	60.0
D22	3.04	22.2	**3.87**	**7.74**	**11.61**	**15.48**	**19.35**	**23.22**	**27.09**	**30.96**	**34.83**	**38.70**
			7.0	14.0	21.0	28.0	35.0	42.0	49.0	56.0	63.0	70.0
D25	3.98	25.4	**5.07**	**10.14**	**15.21**	**20.28**	**25.35**	**30.42**	**35.49**	**40.56**	**45.63**	**50.70**
			8.0	16.0	24.0	32.0	40.0	48.0	56.0	64.0	72.0	80.0
D29	5.04	28.6	**6.42**	**12.84**	**19.26**	**25.68**	**32.10**	**38.52**	**44.94**	**51.36**	**57.78**	**64.20**
			9.0	18.0	27.0	36.0	45.0	54.0	63.0	72.0	81.0	90.0
D32	6.23	31.8	**7.94**	**15.88**	**23.82**	**31.76**	**39.70**	**47.64**	**55.58**	**63.52**	**71.46**	**79.40**
			10.0	20.0	30.0	40.0	50.0	60.0	70.0	80.0	90.0	100.0
D35	7.51	34.9	**9.57**	**19.14**	**28.71**	**38.28**	**47.85**	**57.42**	**66.99**	**76.56**	**86.13**	**95.70**
			11.0	22.0	33.0	44.0	55.0	66.0	77.0	88.0	99.0	111.0
D38	8.95	38.1	**11.40**	**22.80**	**34.20**	**45.60**	**57.00**	**68.40**	**79.80**	**91.20**	**102.60**	**114.00**
			12.0	24.0	36.0	48.0	60.0	72.0	84.0	96.0	108.0	121.0
D41	10.5	41.3	**13.40**	**26.80**	**40.20**	**53.60**	**67.00**	**80.40**	**93.80**	**107.20**	**120.60**	**134.00**
			13.0	26.0	39.0	52.0	65.0	78.0	91.0	104.0	117.0	130.0
D51	15.9	50.8	**20.27**	**40.54**	**60.81**	**81.08**	**101.35**	**121.62**	**141.89**	**162.16**	**182.43**	**202.70**
			16.0	32.0	48.0	64.0	80.0	96.0	112.0	128.0	144.0	160.0

Length on the Market (m)

Length	3.5	4.0	4.5	5.0	5.5	6.0	6.5	7.0	8.0	9.0	10.0

Except for Coils