CRITERION ON THE EVALUATION OF SEISMIC SAFETY OF EXISTING REINFORCED CONCRETE BUILDINGS

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1. Introduction

This report describes the outline of "Criterion on the Evaluation of Seismic Safety of Existing Reinforced Concrete Buildings" which was compiled by the joint committee chaired by Dr. H. Umemura, Prof. of Tokyo University, with commition from Ministry of Construction, Japanese Government.

The buildings which this criterion covers are low-and medium-rise reinforced concrete buildings by ordinary construction method and items for evaluation are not only super structure itself but non-structural elements such as exterior finish elements. Further, these evaluation methodologies are consisted of three steps from the simple first screening to the complicated third screening.

The result of evaluation is expressed by the continuous numerical value but the result shall be judged by the engineer who uses this criterion considering individual and social impact caused by the presumed damage.

Moreover, the applied results on damaged and un-damaged buildings in the Tokachi-Oki Earthquake, 1968 are shown as the refference for the judgement.

In the following, several features and the whole text of the criterion are described.

2. Several Features of Criterion

2.1 Adoption of Seismic Index of Non-structural Elements

The seismic safety of buildings should be examined not only from a viewpoint of the safety of structural elements from collapse, but also from the viewpoint of the safety of non-structural elements such as finishing materials of exterior walls directly facing to streets from their fall. Because that reinforced concrete buildings in Japan have relatively large lateral strength, the structures, itself seemed to seldom fall down instantaneously even under the strong earthquake motions. Actually, in the experience of past earthquake damages, most of buildings survived from the catastrophic destruction. Even in cases of buildings which were unfortunately destroyed and fell down, the residents of the buildings had enough time to escape from the buildings.

Therefore, it becomes important to protect people from injury of the fall of the non-structural elements such as finishing materials of exterior walls.

Though there is no sufficient experimental and empirical information concerning about the performance of non-structural elements under earthquake loads, the safety evaluation of non-structural elements by I_N-index is attempted in this criterion taking into account of the relative flexibility of structure itself and non-structural elements.

2.2 Adoption of Screening Method

The structural safety evaluation considered in this criterion consists of a sequence of steps from 1st to 3rd evaluation. This procedure is repeated in successive cycles, the assumptions and details of the calculations being refined in each successive cycle when necessary for a reliable estimate of structural performance. The repetitive procedure is called "Screening", and is believed to be the fastest and the most practical method for reasonably evaluating the structural adequacy of a large number of buildings subjected to strong earthquake motions.

2.3 Evaluation and Judgement of Seismic Safety

The evaluation of the seismic safety in a broad sense is taken more precisely in the two following senses:

1) Evaluation of seismic safety; to express the seismic safety of

2) Judgement of sesimic safety; to judge the adequacy of buildings for seismic safety taking account of various conditions such as their use, their importance and their age, based on the seismic index obtained by the evaluation of seismic safety.

This criterion aims to evaluate the seismic safety as defined above and the judgement is left to the engineers who use this criterion.

The applied results of this criterion on damaged and un-damaged buildings in Tokachi-Oki Earthquake of 1968 are summaized in the appendix of this criterion. These results will be helpful in performing the judgement of the seismic safety.

2.4 Adoption of Seismic Sub-indexes, S_D, T, and G to Seismic Index, I_S

Seismic sub-indexes, S_D and T which represent the quality of structural design and time dependent deterioration respectively, are taken into account in this criterion as the sub-indexes of synthesis index representing seismic safety, I_S in addition to the seismic sub-index of basic structural performance E_o which related to the lateral load carrying capacity and the deformation capacity of structures. In this criterion, the quantitative evaluation of such sub-indexes are attempted using check list system. Moreover, seismic sub-index G representing the intensity of input ground motions to the base of a building, which depends on the seismicity of its location an on the relationship between its dynamic characteristics and the kind of soil is defined as G in this criterion. The standard value of this sub-index is taken as equal to 1.0 and decreasing value with increase of the earthquake danger in the location is assumed. However, G-index is fixed to 1.0 in this criterion because of the difficulty of the evaluation of the earthquake danger at present.

2.5 Consideration of Seismic Sub-index of Ductility, F to Seismic Sub-index of Basic Structural Performance, E_o.

In the basic sub-index representing the earthquake resistant ability of structures, E_o, not only strength but also deformation capacity are considered as follows.

1) Critical conditions defined by the failure of brittle members.

The lateral load carrying capacity of a building depends on the failure of brittle structural members provided that the building is consisted of structural members with various deformation capacity and, therefore, is not always the sum of the ultimate lateral strength of every structural members. In general, critical displacements at the ultimate strength of brittle structural members are small because of their high stiffness, and then the ductile members which have relatively low stiffness might not reach their ultimate strength at the critical displacements.

Moreover the brittle members show significant reduction of load carrying capacity after they reached their ultimate strength.

Therefore, the failure of brittle structural members becomes one of critical conditions for evaluating the seismic performance of buildings. In this criterion, such critical conditions are expressed in Eq. (2), (3) and (5). In these equations, α means one of the reduction factor of the strength for ductile members considering the compatibility of the displacement at the failure of brittle members. The value of α which is taken as 0.5 to 0.7 in these equation is determined empirically, based on many test results on the yield displacements.

On the other hand, the failure of brittle members causes often the local collapse of buildings because that they become ineffective to sustain vertical loads. Therefore, in this criterion, the failure of brittle members is considered to be one of the critical conditions on the safety of buildings even if the lateral load carrying capacity of the buildings as a whole is not affected by it. Such a critical condition is considered in Eq. (3) or (5).

2) Critical condition of buildings consisted of the structural members which have various deformation capacity.

It is not always easy to evaluate the seismic safety of the buildings consisted of the structural members which have various deformation capacities. In case of a building consisted of structural members which have almost same deformation capacity, it is possible to evaluate its earthquake resistant, based on the assumption of the equal energy concept proposed by Blume et al, which implies that the potential energy stored by the elastic system at maximum deflection is the same as that stored by the elastoplastic system at maximum deflection. In case of a building consisted of, for example, some brittle shear walls and ductile columns, its seismic resistant ability changes with change of the ratio of the load carrying capacity of walls to that of columns or change of deformation capacity of framing members. For evaluating the seismic safety of such type of structures, Eq.(4) is proposed, based on the many non-linear dynamic analyses of combined structures of brittle shear walls and ductile frame responding to ground motions recorded during severe earthquake.

3) Relation between required ductility factor of non-linear system, and seismic sub-index, F.

Non-linear dynamic analyses of structures responding to earthquake motions have shown that the required ductility factor of the elasto-plastic systems whose yield shear factor, is C_y may be estimated from the elastic spectral response acceleration, C_E. Blume et al, for example, has shown that the required ductility factor of reinforced concrete structures is given by the following equation;

$$C_E/C_y = \sqrt{2\mu - 1}$$

where C_y ; yield shear factor of elasto-plastic system.

C_E ; spectral response acceleration of elastic system.

μ ; required ductility factor of elasto-plastic system.

This equation is based on the equal energy concept as mensioned in the Article 2). Comparing the above equation with results obtained from dynamic analyses on single degree of freedom systems with elasto-plastic and degrading stiffness load-deflection relationship, it is evident that the above equation may be an upper bound.

For determining the seismic sub-index, F given in Eq.20, the same approach as mensioned above has been applied, based on the nonlinear dynamic analyses responding to the gound motions recorded during severe earthquake carried out on the single degree of freedom oscillator having degrading tri-linear load-deflection relationship which seemed to be a typical load-deflection relationship of reinforced concrete structures.

The reciprocal of the seismic sub-index, 1/F in this criterion is one of the upper bound of the ratio of the yield shear factor of degrading tri-linear system to the elastic spectral response acceleration.

4) Determining the required ductility factor of structural members of multistory frames from the reponse ductility factor obtained from non-linear dynamic analyses of one mass system.

The ductility demand obtained from the non-linear dynamic response analyses on the one mass system cannot be claimed to give an accurate assessment of the ductility demand of each structural members of the multistory frame responding to non-linearly to strong earthquakes. In this criterion, however, it is supposed that the ductility demand of each structural members assumed to be the same as the response ductility factor obtained by the non-linear dynamic analyses of one mass system.

Many experimental studies have been carried out recently on the ductility behaviour of the flexural yield type structural members. However, there is a lack of information concerning about the quantitative estimation of allowable ductility in accordance with structural details of the members. The equation (22) is proposed provisionally for estimation of the allowable ductility of flexural columns with some restrict conditions in which ductil behaviour can not be expected.

In cases of walls, even the experimental studies on the ductility behaviour have not been performed sufficiently. Therefore, the F-index is directly given by Eq. 24 for walls for safe side estimation instead of the estimation of F-index from the allowable ductility factor as in the case of columns.

2.6 Recommendation for repairs to improve the earthquake resistant characteristics of buildings

When insufficient seismic safety of buildings comes into question as the results of the application of this criterion, appropriate repairs may be required for improving the earthquake resistant characteristics of the buildings. The recommendation for repairs are also provided for this purpose. This recommendation deal with the procedures of repairs in accordance with strength requirements or ductility requirements of the structural members. The method of the evaluation of the seismic safety of the repaired buildings, some attention for the practice of repairs, and some design details for repairs are also provided in this recommendation.

CONTENTS

1. General Rule

1.1 Basic Plan and Scope

This criterion is applied to low or medium-rise existing reinforced concrete buildings (they are called RC buildings for short hereafter) in the case of evaluating the seismic performance of them briefly, and it is composed of three evaluation methods. Each method has a different level from one anther, and is respectively named for the first evaluation method, the second evaluation method, and the third evaluation method.

In addition, this seismic evaluation is an expression of seismic capacity of a building by the continuous index. The decision on the result shall be performed according to the judgement standard that is established elsewhere.

1.2 Preliminary Investigation

Before applying this criterion, in accordance with a proper preliminary investigation, it is necessary to decide whether this criterion may be applied or not. The preliminary investigation is an approximate investigation about whether the structural plan, type and time-dependent condition of the building have much difference from those of normal buildings.

2. Definition of Seismic Index

The seismic safety of buildings is represented in the following two indexes, and the higher value of each index means the higher seismic safety.

I_S : Seismic Index of Structure

I_N : Seismic Index of Non-Structural Elements

They are in principle independent, however, their relations are a little considered. For instance, the relation between ductility of structure and ductility of non-structural elements is used for the culculation of I_N.

3. Calculation of Seismic Index of Structure, I_s

3.1 General

(1) Index of structure, I_s, is calculated by Eq.(1) about the longitudinal and ridge direction at each floor of a building. However, G-index, T-index and S_D-index in the first evaluation method are not related to the floor location and the direction.

$$I_S = E_0 \times G \times S_D \times T \qquad (1)$$

where, E_0 : Seismic Sub-Index of Basic Structural Performance (Section 3.2)

G : Seismic Sub-Index of Ground Motion (Section 3.3)

S_D : Seismic Sub-Index of Structural Profile (Structural Design) (Section 3.4)

T : Seismic Sub-Index of Time-Dependent Deterioration (Section 3.5)

(2) In calculating I_S-index, any one of three methods may be used (the first, the second and the third evaluation method). The generalization of each method is as follows. The larger the number of method is, the more detailed the calculation is and the higher the reliability is.

i) The First Evaluation Method

E_o-index is calculated by the ultimate strength that is approximately calculated from the ratio of wall and column sectional area to sum of floor area. S_D-index and T-index are calculated roughly on the same level with the calculation of E_0. This method is suitable for the building that has a lot of walls, and may underestimate the building that has few walls.

ii) The Second Evaluation Method

Based on the assumption that the strength of beams is sufficiently large, E_0-index is calculated by the ultimate strength of walls and columns (which is calculated by a little more detailed equations than those of the first evaluation method), failure mode, ductility and so on. S_D-index and T-index are a little more detailed than those of the first evaluation method, too. Because ductility together with strength is reflected in E_0-index, the value of E_0-index of the building, that have ductile framing structure, may be higher than the value calculated by the first evaluation method. Furthermore, the standard value for safety judgement in the case of the second evaluation method may be lower than the case of the first evaluation method as the reliability of the calculation by the former is higher than that of the calculation by the latter. The above mentioned matter is also true in the relation between the third evaluation method and the second.

iii) The Third Evaluation Method

When E_0-index is calculated, the type of yielding mechanism, the rotation of foundation under wall and etc. are taken under consideration. S_D-index and T-index are calculated in the same way as the second evaluation method. The seismic safety of buildings is investigated more minutely and the reliability of calculations is higher as compared with the second evaluation method.

3.2 Seismic Sub-Index of Basic Structural Performance, E_0

3.2.1 Calculation of E_0-Index

Based on the assumption that the other sub-indexes are 1.0, E_0-index shows the seismic performance of buildings by the ultimate strength, the type of failure mechanisms and the ductility. The larger the strength is and the higher the deformation ability is because of the ductile failure type, the higher the value of E_0-index is.

By combining strength index C, failure type (Section 3.2.2), ductility index F. (Section 3.2.3) and others, E_0-index at the i-th story of n-storied building is calculated as follows by each method.

(1) The first Evaluation Method

In the first evaluation method, vertical members of buildings are classified into three categories (Table 1), and E_0-index is calculated as follows.

Table 1. Classification of Vertical Members for the First Evaluation Method

Mame	Definition
column	independent column ($ho/D > 2$)
extremely short column	independent column ($ho/D \leqq 2$)
wall	including the wall not surrounded by framing members

notes : ho : clear height of column ; If there is upper or lower wall, ho becomes short. (Figure 1)

D : depth of column section

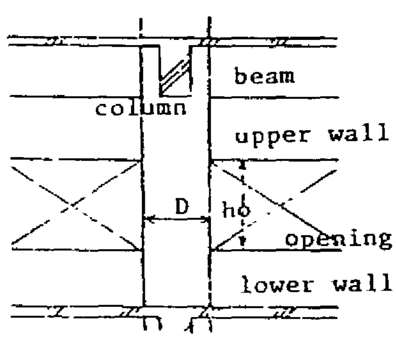


Figure 1. Clear Height of Column, ho

i) E_0-Index of Buildings without Extremely Short Columns

E_0-index is obtained by Eq.(2) in the case that there is no extremely short column.

$$E_0 = \frac{n + 1}{n + i} (Cw + \alpha_1 \cdot Cc) \times Fw \qquad (2)$$

where, n : total number of stories of the building

i : the number of the story under investigation; 1 is used at the first story, and n is used at the top story.

Cw : strength index for walls by Eq. (7)

Cc : strength index for columns by Eq. (8)

α_1 : (sum of the lateral shear forces sustained by columns corresponding to the displacement at the ultimate strength of walls) / (sum of the ultimate strength of columns) ; 0.7 may be used for this value, however, it is 1.0 in the case of Cw = 0.

Fw : ductility index of walls (ductility index of columns in the case of Cw = 0) ; 1.0 may be used for this value.

ii) E_0-Index of Buildings with extremely short Columns

In the case that there are some extremly short columns, E_0-index is the higher value which is obtained by Eq. (3) or by Eq.(2) neglecting extremely short columns.

However, if the extremely short column is the secondary seismic element, Eq. (3) should be used. The secondary seismic element is the member which is permitted to fail by horizontal load and has no elements arround to support the vertical load that is sustained by the member at the failure.

$$E_0 = \frac{n + 1}{n + i} (Csc + \alpha_2 \cdot Cw + \alpha_3 \cdot Cc) \times Fsc \qquad (3)$$

where, Csc : C-index of extremely short columns, calculated by Eq. (9)

Cw : C-index of walls, calculated by Eq. (7)

Cc : C-index of columns, calculated by Eq. (8)

α_2 : (sum of the lateral shear forces sustained by walls corresponding to the displacement at the ultimate strength of extremely short columns) / (sum of the ultimete strength of the walls) ; 0.7 may be used for this value.

α_3 : (sum of the lateral shear forces sustained by columns corresponding to the displacement at the ultimate strength of extremely short columns) / (sum of the ultimate strength of columns) ; 0.5 may be used for this value.

Fsc : ductility index of extremely short columns; 0.8 may be used for this value.

(2) The Second Evaluation Method

In the second evaluation method, first, we determine the failure type (Table 2) and ultimate shear force (3.2.2 (2), ii), iii)) of each vertical member at the objective story by the process shown in Section 3.2.2 (2), and we calculate the ductility index of each member by the process shown in Section 3.2.3. Next, we classify vertical members into three or less groups so that the members of which the failure tipes and ductility indexes are near each other are in one group, and then we calculate the structural indexes by Section 3.2.2 and the ductility indexes by Section 3.2.3 about the groups. Failure types are shown at Table 2. Vertical members classified into three or less groups are named for the first, the second and the third group according to the order from the lowest F-index. Lastly, E_0-index is calculated by combining the structural indexes C and ductility indexes F of each group as follows.

Table 2. Classification of Vertical Members by Failure Types for the Second Evaluation Method

failure type	Definition
flexural column	column that flexural yielding precedes shear failure
flexural wall	wall that flexural yielding precedes shear failure
shear column	column that shear failure precedes flexural yielding ; However, extremely brittle column is excluded.
shear wall	wall that shear failure precedes flexural yielding
extremely brittle column	column that ho/D is less than or equal to 2.0 (extremely short column), and shear failure precedes flexural yeilding

i) E_0-index of Building without Extremely Brittle Columns

In the case that there is no extremely column, E_0-index is the higher value which is calculated by Eq. (4) or by Eq. (5). However, if there are some shear columns which are the secondary seismic elements, Eq. (5) should be used.

$$E_0 = \frac{n+1}{n+i}\sqrt{E_1^2 + E_2^2 + E_3^2} \qquad (4)$$

where, E_1 : $C_1 \times F_1$

E_2 : $C_2 \times F_2$

E_3 : $C_3 \times F_3$

C_1 : C-index of the first group (F-index is lowest)

C_2 : C-index of the second group (F-index is middle)

C_3 : C-index of the third group (F-index is highest)

F_1 : F-index of the first group

F_2 : F-index of the second group

F_3 : F-index of the third group

$$E_0 = \frac{n+1}{n+i}(C_1 + \alpha_2 \cdot C_2 + \alpha_3 \cdot C_3) \times F_1 \qquad (5)$$

where, α_2 : (sum of the lateral shear forces sustained by the second group members corresponding to the displacement at the ultimate strength of the first group members)/(sum of the ultimate strength of the second group members) ; It may be taken as the values shown in Table 3.

α_3 : (sum of the lateral shear forces sustained by the third group members corresponding to the displacement at the ultimate strength of the first group members) / (sum of the ultimate strength of the third group members) ; It may be taken as the values shown in Table 4.

Table 3. α_2 in Eq. (5)

the first group / the second group	extremely brittle column	shear column, shear wall
flexural column	0.5	0.7
flexural wall	0.7	1.0
shear column, shear wall	0.7	——

Table 4. α_3 in Eq. (5)

the first group / the third group	extremely brittle column	shear column, shear wall
flexural column	0.5	0.7
flexural wall	0.7	1.0
shear column, shear wall	0.7	——

ii) E_0-Index of Buildings with Extremely Brittle Columns

In the case that there are some extremely brittle columns, E_0-index is the highest value wihch is calculated by Eq. (4) and (5) neglecting extremely brittle columns or by Eq. (5) considering extremely brittle columns. In the case that extremely brittle columns are not considerd, the vertical member's group, of which the ductile index is secondly least, rises to the first group, and the number of groups goes up in order.

However, if the extremely brittle columns are the secondary seismic elements, E_0-index shall be the value by Eq. (5) considering extremely brittle columns. In addition, even if the extremely brittle column is not the secondary seismic element, in the case that there are some shear columns which are the secondary seismic elements, E_0-index is the larger value which is obtained by Eq. (5) considering extremely brittle columns or by Eq. (5) neglecting extremely brittle columns.

iii) Exception

In the case that eccentricity ratio defined in Section 3.4 for the calculation of S_D-index is more than 0.15 because of unbalancedly distributed walls etc., E_0-index is the smaller one of the following two.

a) Neglecting the vertical members by which the eccentricity is caused, E_0-index is calculated by the method mentioned in Paragraph i) and ii).

b) Not considering the eccentricity, E_0-index is obtained by Eq. (5), however, the vertical members by which the eccentricity is caused is taken as the first group, and the group of which F-index is smaller than the first group is neglected.

The third evaluation method is performed in the same way as the second evaluation method and further the following matters are added considering the strength and ductility of beams and the rotation of foundation under wall.

i) As failure types, three other types shown in Table 5 are added to five types shown in Table 2.

ii) E_0-index is calculated in the same way as the second evaluation method, however, E_0-index may be modified as follows in only the case that the flexural yielding of beams or the overturning capacity of walls controls the seismic capacity of the building.

$$E_0' = E_0 \times \frac{2}{3} \times \frac{2n+1}{n+1} \qquad (6)$$

where, n : total number of stories of the building

Table 5. Classification of Vertical Members by Failure Types for the Third Evaluation Method

failure type	Definition
flexural column flexural wall shear column shear wall extremely brittle column	by definition in Table 2
beam yield type column	column controled by the beam that flexural yielding precedes shear failure
beam shear failure type column	column controled by the beam that shear failure precedes flexural yielding
overturning type wall	wall that overturning capacity precedes flexural yielding or shear failure

3.2.2 Strength Index, C

This section is used for calculating C-index of vertical members at each story of buildings for the first, the second and the third evaluation method.

(1) The First Evaluation Method

In the case of the first evaluation method, using the sectional area of walls and columns, strength index C is approximately calculated as follows.

$$Cw = \frac{\tau w_1}{w} \times aw_1 + \frac{\tau w_2}{w} \times aw_2 + \frac{\tau w_3}{w} \times aw_3 \qquad (7)$$

$$Cc = \frac{\tau c}{w} \times ac \qquad (8)$$

$$Csc = \frac{\tau sc}{w} \times asc \qquad (9)$$

where, Cw : strength index of walls

Cc : strength index of columns

Csc : strength index of extremely short columns

τw_1 : average shear stress at ultimate strength of wall (wall with columns on both ends) ;

30 kg/cm^2 may be used for this value.

τw_2 : average shear stress at ultimate strength of wall (wall with a column on one end) ;

20 kg/cm^2 may be used for this value.

τw_3 : average shear stress at ultimate strength of wall (wall without surrounding columns) ;

10 kg/cm^2 may be used for this value.

τc : average shear stress at ultimate strength of column ; 10 kg/cm^2 may be used for this value, however, 7 kg/cm^2 shall be used if ho/D is more than or equal to 6.

τsc : average shear stress at ultimate strength of extremely short column ; 15 kg/cm^2 may be used for this value.

aw_1 : ratio of wall sectional area to sum of floor area (wall with columns on both ends) = $Aw_1/\Sigma Af$ (cm^2/m^2)

aw_2 : ratio of wall sectional area to sum of floor area (wall with a column on one end) = $Aw_2/\Sigma Af$ (cm^2/m^2)

aw_3 : ratio of wall sectional area to sum of floor area (wall without surrounding columns) = $Aw_3/\Sigma Af$ (cm^2/m^2)

Aw_1 : sum of the effective wall sectional area in the direction at the story investigated (wall with columns on both ends) (cm^2)

Aw_2 : sum of the effective wall sectional area in the direction at the story investigated (wall with a column on one end) (cm^2)

Aw_3 : sum of the effective wall sectional area in the direction at the story investigated (wall without surrounding columns) (cm^2)

However, wall sectional area is defined by Figure 2.

ac : ratio of column sectional area to sum of floor area = $Ac/\Sigma Af$ (cm^2/m^2)

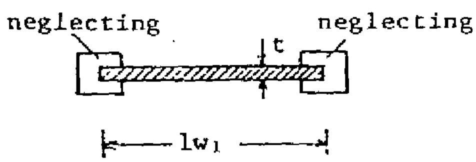
asc : ratio of extremely short column sectional area to sum of floor area = $Ac/\Sigma Af$ (cm^2/m^2)

Ac : sum of independent column sectional area at the story (cm^2) ; The column surrounding the wall which is used for the calculation of Aw_1 or Aw_2 shall not be accounted to Ac.

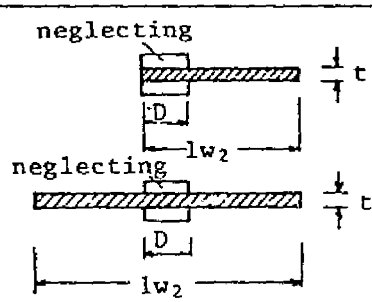
Asc : sum of extremely short column sectional area at the story (cm^2)

ΣAf : sum of the floor area of which the story is higher than the story calculated (m^2)

w : sum of the weight of each story which is higher than the story under consideration (dead load + live load for calculation of lateral load) $/\Sigma Af$ (kg/cm^2) ; 1,200 kg/cm^2 may be used for this value if the calculation is not especially needed.

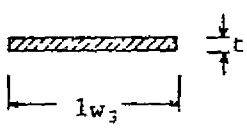


$Aw_1 = t \times lw_1$



$Aw_2 = t \times lw_2$

If $(lw_2 - D)$ is less than 45 cm, neglecting the wall, it may be regarded as independent column.



$Aw_3 = t \times lw_3$

If lw_3 is less than 45 cm, it is neglected.

Figure 2. Calculation of Wall Sectional Area

(2) The Second Evaluation Method

In the second evaluation method, based on the assumption that the strength of beams is in principle sufficiently large, C-index is calculated by the ultimate strength of vertical members (columns and walls) against horizontal load.

i) Process

Structural index for the second evaluation method is calculated in the following process.

a) The ultimate shear strength, Qsu and the shear force at ultimate flexural strength, Qmu of each vertical member are calculated, and then the failure types are determined by the comparison of these two values: Ultimate shear strength, Qsu and ultimate flexural strength, Mu are calculated by Eq. (10) - Eq. (15) in Paragraph ii), and shear force at ultimate flexural strength is calculated by Eq. (16) and Eq. (17) in Paragraph iii).

b) Ductility index F of each vertical member is decided by the failure type and the ductility capacity in the way of Section 3.2.3.

c) Vertical members are classified into groups (less than or equal to 3), and the structural index of each group is calculated.

Classification into groups is shown in Paragraph iv), and calculation of structural index is shown in Paragraph v).

ii) Calculation of Ultimate Strength

Ultimate flexural strength and ultimate shear strength of a member are calculated by Eq. (10) - Eq. (15).

The specified compressive strength for compressive strength of concrete (Fc), 3,000 kg/cm^2 for tensile yield stress of round bars and (specified tensile yield stress + 500 kg/cm^2) for tensile yield stress of defromed bars may be used respectively. However, in the case that remarkable time-dependent deterioration are observed by preliminaly investigation or there are data about material strength from detailed investigation, the values in the actual condition should be used.

a) Ultimate flexural strength Mu of a rectangular column is obtained by Eq. (10).

$$\left.\begin{aligned}
&Nmax \geqq N > 0.4\ b\cdot D\cdot Fc\\
&\quad Mu = (0.8a_t\cdot\sigma_y\cdot D + 0.12b\cdot D^2\cdot Fc)\left(\frac{Nmax - N}{Nmax - 0.4b\cdot D\cdot Fc}\right)\\
&0.4b\cdot D\cdot Fc \geqq N > 0\\
&\quad Mu = 0.8a_t\cdot\sigma_y\cdot D + 0.5N\cdot D\left(1 - \frac{N}{b\cdot D\cdot Fc}\right)\\
&0 > N \geqq Nmin\\
&\quad Mu = 0.8a_t\cdot\sigma_y\cdot D + 0.4N\cdot D
\end{aligned}\right\} \quad (10)$$

where, Nmax : ultimate strength of the column under axial compression $= b\cdot D\cdot Fc + a_g\cdot\sigma_y$ (kg)

Nmin : ultimate strength of the column under axial tension $= -a_g\cdot\sigma_y$ (kg)

N : axial froce of the column (kg)

a_t : total area of tension bars (cm^2)

a_g : gross area of bars in the column (cm^2)

b : width of the column (cm)

D : depth of the column (cm)

σ_y : tensile yield stress of bars (kg/cm^2)

Fc : compressive strength of concrete (kg/cm^2)

b) Ultimate flexural strength Mu of a column with wing walls is calculated by Eq. (11). However, in the case that the wing wall is on only one side of the column and flexural moment acts in the direction that the wing wall is tensile, the column with the wing wall is treated as a rectangular column and is calculated by Eq. (10).

$$N \leqq [\ 0.5\alpha_e(0.9 + \beta) - 13p_t]b\cdot D\cdot Fc$$

$$Mu = (0.9 + \beta)\ a_t\cdot\sigma_y\cdot D + 0.5N\cdot D\left[\ 1 + 2\beta - \frac{N}{\alpha_e\cdot b\cdot D\cdot Fc}\left(1 + \frac{a_t\cdot\sigma_y}{N}\right)^2\right] \qquad (11)$$

If N is more than $[0.5\alpha_e(0.9 + \beta) - 13p_t]b \cdot D \cdot Fc$,

Mu is calculated by substituting $[0.5\alpha_e(0.9 + \beta) - 13p_t]b \cdot D \cdot Fc$ into N of Eq. (11).

where, p_t : tension reinforcement ratio = $a_t/(b \cdot D)$

α_e : $\Sigma A/(l_w \cdot b)$

ΣA : total sectional area of the column with wing walls (cm^2)

l_w : total horizontal length measured out-to-out of wing walls (cm)

β : (length of wing wall on compression side) / D

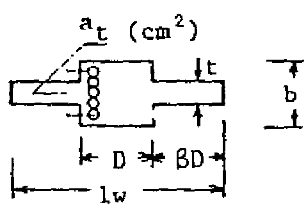


Figure 3. Column with wing walls

c) Ultimate flexural strength of a wall with columns on both ends is obtained by Eq. (12). If there are columns in the middle of the wall, the longitudinal bars of the column is regarded as vertical reinforcements of the wall.

$$Mu = a_t \cdot \sigma_y \cdot l_w + 0.5\Sigma(a_w \cdot \sigma_{wy})\ l_w + 0.5N \cdot l_w \qquad (12)$$

where, a_t : total area of longitudinal bars in the column on the tensile side of the wall (cm^2)

σ_y : tensile yield stress of longitudinal bars in the column on the tensile side of the wall (kg/cm^2)

a_w : area of vertical reinforcements in the wall (cm^2)

σ_{wy} : tensile yield stress of vertical reinforcements in the wall (kg/cm^2)

l_w : length of the wall, measured center-to-center of columns (Figure 5) (cm)

d) Ultimate flexural strength of a wall with a column on one end or a wall without columns is calculated by Eq.(10), Eq.(11) or Eq.(12) according to the shape and arrangement of reinforcing bars.

e) Ultimate shear strength of a rectangular column is calculated by Eq.(13).

$$Qsu = \left[\frac{0.053pt^{0.023}(180 + Fc)}{M/(Q \cdot d) + 0.12} + 2.7\sqrt{P_w \cdot {}_s\sigma_{wy}} + 0.1\sigma_o \right] b \cdot j \tag{13}$$

however,

$$1 \leqq M/(Q \cdot d) \leqq 3$$

where P_t : tension reinforcement ratio (%)

P_w : shear reinforcement ratio ; In the case of $P_w \geqq 0.012$, 0.012 shall be used for P_w.

${}_s\sigma_{wy}$: tensile yield stress of shear reinforcement (kg/cm^2)

σ_o : axial stress of the column (kg/cm^2) ;
In the case of $\sigma_o > 80\ kg/cm^2$,
$80\ kg/cm^2$ shall be used for σ_o.

d : effective depth of the column section ;
(D - 5cm) may be used for d.

M/Q : shear span ; ho/2 may be used for M/Q.
ho is the clear height of the column.

j : distance between the center of tensile stress and that of compressive stress of the column section ;
0.8D may be used for j.

f) Ultimate shear strength of a column with wing walls is obtained by Eq.(14).

$$Qsu = 0.5\sqrt{Fc}\ \left(\frac{l_w}{ho} \right)\Sigma A + 0.5[p_w \cdot \sigma_{wy} + p_s \cdot \sigma_{sy} \frac{t(l_w - D)}{b \cdot D}]b \cdot D + 0.1N \qquad (14)$$

where p_w : shear reinforcement ratio of the column

σ_{wy} : tensile yield stress of shear reinforcements (kg/cm^2)

P_s : lateral reinforcement ratio of the wing wall = $a_w/(t \cdot s)$

a_w (cm^2) is area of a set of lateral reinforcements and s (cm) is the spacing of lateral reinforcements.

σ_{sy} : tensile yield stress of lateral reinforcements (kg/cm^2)

N : axial force (kg)

ho : clear height of the column (cm)

ΣA : total sectional area (cm^2)

l_w, t, b and D is in Figure 4.

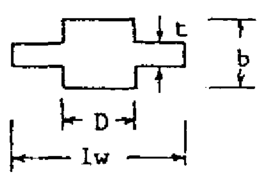


Figure 4. Column With Wing Walls

g) Ultimate shear strength of a wall with columns on both ends is calculated by Eq.(13). However, the parameters are substituted as follows. In addition, if the wall has an opening, Eq.(13) is multiplied by reduction ratio (γ) of Eq.(15).

pt : 100 X $a_t/(be \cdot l)$ (%)

where a_t : total area of longitudinal bars in the column on the tensile side of the wall (cm^2)

l : total length of the wall (Figure 5) (cm) be : equivalent thickness of wall = $\Sigma A/l$ (cm)

ΣA : total sectional area (cm^2)

P_w : equivalent holizontal reinforcement ratio of the wall = $a_w/(be \cdot s)$

where a_w : area of a set of lateral reinforcements (cm^2)

s : spacing of lateral reinforcements (cm)

${}_s\sigma_{wy}$: tensile yield stress of reinforcements of the wall (kg/cm^2)

σ_o : ΣN / $(be \cdot l)$

where ΣN : total axial force (kg)

j : l_w or 0.8l may be used for this value.

b : It is replaced by be.

D : It is replaced by l.

d : It is replaced by l.

M/Q : wMu / wQmu calculated by Eq.(17)

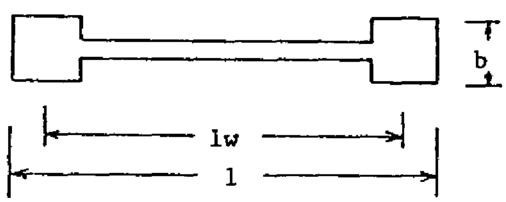


Figure 5. Wall with Columns on Both Side

reduction ratio by a opening of the wall :

$$\gamma = 1 - (\text{equivalent opening peripheral ratio}) \qquad (15)$$

where equivalent opening peripheral ratio :

$$\sqrt{\frac{\text{opening area}}{h \times l_w}}$$

h : height of the story

h) Ultimate shear strength of a wall with a column on one side or a wall without columns is calculated by Eq.(13) or Eq.(14) according to the shape and arrangement of reinforcing bars.

iii) Calculation of Failure Type and Shear Force at Ultimate Strength

Uding ultimate flexural strength and ultimate shear strength in Paragraph ii), the failure type of vertical members and shear force at the ultimate strength are obtained as follows.

a) Column

Calculating shear force ${}_cQ_{Mu}$ at ultimate flexural strength by Eq.(16), and comparing ${}_cQ_{Mu}$ with ultimate shear strength ${}_cQ_{su}$, the failure type and the shear force ${}_cQ_u$ at ultimate strength are obtained.

1) In the case of ${}_cQ_{Mu} < {}_cQ_{su}$, failure type is flexural column.

$({}_cQ_u = {}_cQ_{Mu})$

2) In the case of ${}_cQ_{Mu} \geqq {}_cQ_{su}$, failure type is shear column.

$({}_cQ_u = {}_cQ_{su})$

However, in shear columns, the column, that h_o/D is less than or equal to 2, is especially treated as extremely brittle column.

$$ {}_cQ_{Mu} = \frac{({}_cM_u)T + ({}_cM_u)B}{h_o} \tag{16} $$

where $({}_cM_u)T$: ultimate flexural strength at the top of the column

$({}_cM_u)B$: ultimate flexural strength at the bottom of the column

h_o : clear height of the column

b) Wall

Calculating shear force ${}_wQ_{Mu}$ at ultimate flexural strength by Eq.(17), and comparing ${}_wQ_{Mu}$ with ultimate shear strength ${}_wQ_{su}$, the failure type and the shear force ${}_wQ_u$ at ultimate strength are obtained.