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A Simple Approach for Preliminary Design of Reinforced Concrete Structures to be Built in Seismic Regions†

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ABSTRACT

In the past forty years attempts have been made to decrease the loss of life during the expected earthquakes. Attempts include revisions made in the National Seismic Code. However such attempts could not bring the loss of life to an acceptable level, because these attempts were not focused on the main causes of heavy damages and collapses observed during the past earthquakes. Investigations and observations made revealed that a great majority of loss of life occurred in two to eight story dwellings and office buildings.

In this paper some simple rules have been developed for the preliminary design of reinforced concrete structures, considering the main causes of collapses observed during the past earthquakes in Turkey. Also some simple detailing rules are provided. It is believed that if the rules given in this paper for preliminary design and detailing are applied, loss of lives during earthquakes will decrease significantly. The rules given in this paper were developed for buildings to be built in seismic zones 1 and 2. The author believes that these rules can also be applied to the other zones by revising the constants given in the equations.

Keywords: *Earthquake, reinforced concrete, columns, beams, structural walls, preliminary design, detailing, simple method*

1. INTRODUCTION

Attempts made in the past forty years could not bring the loss of lives during earthquakes to a reasonable level. The author believes that the reason for this is that the attempts were not focused on the main causes of damages observed in the past earthquakes. Observations made by the author beginning with 1967 Adapazarı earthquake reveal that a great majority of loss of lives occurred in two to eight story office buildings and dwellings. He also concluded that the causes of heavy damage and collapse could be classified into three groups.

a) Mistakes made in choosing the architectural configuration and the structural system.

b) Inadequate or wrong detailing of reinforcement.

c) Lack of effective inspection during the construction stage.

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The author believes that about 90 % of heavy damages and collapses which occurred during the earthquakes in the past forty years are due to the causes given above. Two to eight story dwellings and office buildings are usually designed by what might be called "average engineers". Due to their inadequate background, these engineers cannot fully understand the high level provisions of the seismic code of Turkey. Therefore these engineers purchase package software and use it as a black box without understanding the fundamental concepts concerned. Obviously such an approach led to the disasters observed during the past earthquakes.

A review of damages observed during the past earthquakes reveals that a great majority of loss of life occurred in buildings where floors fell on top of each other. Such a type of collapse can be prevented by satisfying the following three requirements

a) The interstory drift should not exceed certain limits

b) Structural members, especially the columns should be ductile

c) Structural members, especially the columns should not fail in shear

To keep interstory drift at a reasonable level, the engineer should introduce structural walls and should not be stingy in choosing the column sizes. Shear strength and ductility of members depend on the sizes of cross-sections chosen and the ratio of lateral reinforcement (confinement). It becomes obvious that proportioning of members at the preliminary design stage play a very important role in preventing collapses, thus reducing loss of lives. If a sound structural system is chosen with adequate member sizes and if a reasonably good detailing is made at the final design stage, the probability of the collapse of two to eight storey buildings during earthquake will be reduced significantly. Needless to say, effective inspection at the construction stage is as important as the design itself.

In 1983 the author set some simple rules for proportioning of structural members. These rules were used by City of Mersin to check the submitted designs [1, 2]. In the past thirty years there have been some improvements in the quality of construction in Turkey. Due to the wide use of ready mixed concrete, concrete quality in buildings has improved significantly. In general there have been improvements in detailing also. Therefore the rules set in 1983 may have now become overconservative. In the study presented in this paper these improvements have been taken into consideration.

2. CRITERIA AND ASSUMPTIONS FOR PROPORTIONING OF STRUCTURAL MEMBERS

In this section the criteria and assumptions made in developing the proportioning rules will be summarized. In the text reference will be made to the Turkish Seismic Code, which will be referred as TSC [3].

2.1 Assumptions and Requirements

- It is assumed that concrete compressive strength is 20 MPa.
- Yield strength of both longitudinal and lateral reinforcement is assumed to be 420 MPa.

- Weight of building, including live load is assumed to be 10 kN/m^2. For gravity load analysis the load factor is taken as 1.5 (p_d = 10x1.5=15 kN/m^2). For seismic analysis the load factor is reduced to 1.0 (p_d = 10x1.0=10 kN/m^2).
- It is assumed that at each end of the column at least the following confinement exists.

  For h ≤ 400 mm, Ø8/100mm ties and one Ø8/100 mm cross-tie in both directions.

  For h > 400 mm, Ø10/100mm ties and one Ø10/100 mm cross-tie in both directions.
- It is assumed that at least the following amount of ties exists at the unconfined regions of columns.

  If h ≤ 400 mm, Ø8/200mm ties

  If h > 400 mm, Ø10/200mm ties

  h = is the larger cross-sectional dimension of the column
- It is assumed that the anchorage lengths of reinforcing bars are adequate.
- It is assumed that the following dimensional requirements for beams are satisfied.

  $b_w \geq 250$ mm

  $h \geq 300$ mm

  $\geq 3t$

  b_w = web width of beam

  h = depth of beam

  t = slab thickness
- It is assumed that at each end of the beam for a length of 2h, stirrups spacing does not exceed d/4. At other regions spacing should not exceed d/2.
- It is assumed that the thickness of structural walls is not less than 200 mm.
- It is assumed that total web reinforcement ratio on two faces of the structural wall is not less than 0.0025. This applies to both longitudinal and transverse reinforcement. Also the spacing of such reinforcement should not be more than 250 mm.
- There will be end regions at each end of the structural wall. At such regions there will be at least 4-Ø16 longitudinal reinforcement, $\rho_t \geq 0.001\, l_w t$, and Ø8/150 mm confinement.

  ρ_t = longitudinal reinforcement ratio at edge regions.

  t = thickness of structural wall.

  l_w = the larger dimension of the cross-section of the wall.
- The rules given in this paper are developed for seismic zones 1 and 2.

2.2 The Requirements to be Satisfied

a) The shear stresses in columns and structural walls shall not exceed the limits set.

b) Requirements for "Normal Ductility Level" stated in TSC shall be satisfied.

c) The limits given in TSC for interstory drift shall not be exceeded by providing adequate stiffness in lateral direction

3. RULES FOR PROPORTIONING AT PRELIMINARY DESIGN STAGE

The rules for proportioning given in this section are aimed to satisfy the requirements given in the previous section. The author believes that the type of buildings defined in this paper should have structural walls in both directions capable of fully resisting seismic forces. Neverheless frames in the structural system should be designed to resist at least 30 % of the total seismic forces. The rules given below are based on the assumption that these requirements are satisfied.

3.1 Columns:

In TSC-2007, the following upper limit is specified for the column axial load.

$$N_{d1} \leq 0.5\ f_{ck}\ A_{ci} \qquad (1)$$

N_{d1} = axial load

f_{ck} = characteristic concrete strength

A_{ci} = cross-sectional area of the column

The axial load on the column can approximately be calculated by multiplying the total tributary area of the column by the unit weight of the building

$$N_{d2} = p_d\ \Sigma A_{oi} \qquad (2)$$

ΣA_{oi}, = Sum of the tributary area over all floors

p_d = Weight of the building (For gravity loads 10x1.5=15 kN/m^2)

Equating Eq.(1) to Eq.(2) and assuming f_{ck} = 20 000 kN/m^2 the following relationship is obtained for the minimum column cross-sectional area.

$$A_{ci} \geq\ 0.0015\ \Sigma\ A_{oi} \qquad (3)$$

In determining the column sizes the other important parameter is the shear strength. In calculating the shear capacity of the column effect of axial load is ignored and it is assumed that ties are not less than the ones specified in Section 2.1.

Shear strength can be written as,

$$V_r = V_c + V_w \qquad (4)$$

$V_{cr} = 0.65\ f_{ctd}A_{ci}$

$V_c = 0.8\ V_{cr}$

$V_w = (A_{sw}/s)\ f_{ywd}\ d$

V_r = shear strength of column

V_{cr} = cracking shear strength of column

V_c = contribution of concrete to shear strength

f_{ctd} = design tensile strength of concrete ($f_{ctd} = f_{ctk}/1.5 = 1.6/1.5 \approx 1.1$ MPa)

f_{ywd} = design yield strength of steel ($f_{ywd} = f_{ywk}/1.15 = 420/1.15 \approx 365$ MPa)

A_{sw} = total cross-sectional area of ties at a section

s = spacing of ties

h = the larger cross-section dimension of column

b = smaller cross-section dimension of column

d = effective depth

Taking f_{ctd} = 1 100 kN/m^2 and f_{ywd} = 365 000 kN/m^2 , the following equations are obtained for the shear strength of the column.

$$V_{cr} = 715\ A_{ci} \qquad (5)$$

$V_c = 572A_{ci}$

$V_w = 365\ 000\ (A_{sw}/s)\ d$

$$V_r = V_c + V_w = 572\ A_{ci} + 365\ 000\ (A_{sw}/s)\ d \qquad (6)$$

In Table-1, shear strength of columns with different cross-sectional dimensions are given. Shear strengths are calculated using Eq.(6), taking $A_{sw}/s = 2\ A_o/s$. Ties are ϕ8/200 mm or ϕ10/200 mm depending on the size of the column as assumed in Section 2.1. The minimum effective depth of the column is assumed to be (b-40mm).

In Table-1, cracking shear strengths are also given for each column section. In the last column of the table the ratios of shear strength to cracking strength are given. As can be seen from the table, this ratio is greater than 1.0 for all column sizes considered. It should be pointed out that the smallest ratio is 1.35. If the shear strength of the column is taken as $V_r = 1.35V_{cr}$, the following expression is obtained.

$V_r = 1.35x715\ A_{ci} = 965\ \Sigma\ A_{ci}$

This expression can be rewritten as the total shear strength of all columns in the floor by substituting ΣA_{ci} and ΣV_r in place of A_{ci} and V_r

$$\Sigma V_r = 965 \Sigma A_{ci} \quad (7)$$

ΣA_{ci} = Sum of cross- sectional areas of columns at i^{th} floor

Table 1. Shear Strength of columns

1 h (mm)	2 b (mm)	3 f_{ctd} (MPa)	4 A_c (mm^2)	5 A_{sw}/s (mm)	6 f_{ywd} (MPa)	7 V_{cr} (kN)	8 V_c (kN)	9 V_w (kN)	10 V_r (kN)	11 V_r/V_{cr}
300	300	1.1	90000	0.50	365	64.4	51.5	47.5	98.9	1.54
350	300	1.1	105000	0.50	365	75.1	60.1	47.5	107.5	1.43
350	350	1.1	122500	0.50	365	87.6	70.1	56.6	126.6	1.45
400	300	1.1	120000	0.50	365	85.8	68.6	47.5	116.1	1.35
400	350	1.1	140000	0.50	365	100.1	80.1	56.6	136.7	1.37
400	400	1.1	160000	0.50	365	114.4	91.5	65.7	157.2	1.37
450	300	1.1	135000	0.79	365	96.5	77.2	75.0	152.2	1.58
450	400	1.1	180000	0.79	365	128.7	103.0	103.8	206.8	1.61
450	450	1.1	202500	0.79	365	144.8	115.8	118.2	234.1	1.62
500	300	1.1	150000	0.79	365	107.3	85.8	75.0	160.8	1.50
500	400	1.1	200000	0.79	365	143.0	114.4	103.8	218.2	1.53
500	500	1.1	250000	0.79	365	178.8	143.0	132.6	275.6	1.54
600	300	1.1	180000	0.79	365	128.7	103.0	75.0	177.9	1.38
600	400	1.1	240000	0.79	365	171.6	137.3	103.8	241.1	1.40
600	500	1.1	300000	0.79	365	214.5	171.6	132.6	304.2	1.42
600	600	1.1	360000	0.79	365	257.4	205.9	161.5	367.4	1.43

The base shear in TSC-2007 is given as,

$$\Sigma V_d = W\, A(T_1) / R_a(T_1) \quad (8)$$

$W = w\, \Sigma A_{pi}$

taking w as 10 kN/m^2,

$W = 10\, \Sigma A_{pi}$

$A(T_1) = A_o\, I\, S(T_1)$

ΣV_d = base shear (kN)

w = weight of the building per square meters

ΣA_{pi} = sum of the floor areas (m^2)

A_0 = effective ground acceleration coefficient

I = importance factor

S(T) = spectrum coefficient

R(T) = seismic load reduction factor

If it is assumed that, I = 1.0, A_o = 0.4, S (T_1) = 2.5 and R(T_1) = 4.0, the following expression is obtained for the base shear.

$$\Sigma V_d = 2.5\ \Sigma A_{pi} \tag{9}$$

It should be remembered that one of the basic assumptions made was that the frames should be able to resist at least 30 % of the total shear. It is also assumed that the most critical floor is the ground floor. This means that columns of ground floor should be able to resist at least 30% of the base shear given in Eq. (9). To be on the safe side it will be assumed that ground floor columns will resist 50 % of ΣV_d instead of 30 %.

For a column having cross-sectional area of $A_{ci,}$ ΣA_{pi} in Eq. (9) can be replaced by $\Sigma A_{oi.}$ If 50 % of the base shear given in Eq. (9) is equated to Eq. (7), the following expression is obtained for minimum column cross-sectional area.

$$965\ A_{ci} = 0.5\ x2.5\ \Sigma A_{oi} = 1.25\ \Sigma A_{oi}$$

$$A_{ci} \geq 0.0013\ \Sigma A_{oi} \tag{10}$$

Eq. (10) is same with Eq. (3) except for the constant. Since Eq. (3) results in larger cross-sectional area, it should be used in determining the minimum column size.

$$A_{ci} \geq 0.0015\ \Sigma A_{oi} \tag{11}$$

$$\geq 0.09\ m^2$$

The ratio of cross-sectional dimensions should not be more than 2.0.

$h/b \leq 2.0$

In Table-2, cross-sectional area of columns for different tributary areas and for different number of floors obtained from Eq. (11) are given. In calculating the cross-sectional area it is assumed that the tributary area of the column is same in all floors, ΣA_{oi} = n A_{oi}. In the last column of the table minimum sizes are given for square cross-sections.

Table 2. Minimum column dimensions calculated using Eq.11

1 A_{oi} (m^2)	2 n	3 A_{ci} (m^2)	4 h=b (m)	5 h=b Rounded	6 h×b (mm)
10	4	0.090	0.300	0.300	300x300
10	6	0.090	0.300	0.300	300x300
10	8	0.120	0.346	0.350	350x350
13	4	0.090	0.300	0.300	300x300
13	6	0.117	0.342	0.340	340x340
13	8	0.156	0.395	0.400	400x400
16	4	0.096	0.310	0.300	300x300
16	6	0.144	0.379	0.380	380x380
16	8	0.192	0.438	0.440	440x440
20	4	0.120	0.346	0.345	345x345
20	6	0.180	0.424	0.425	425x425
20	8	0.240	0.490	0.490	490x490
25	4	0.150	0.387	0.390	390x390
25	6	0.225	0.474	0.475	475x475
25	8	0.300	0.548	0.550	550x550
30	4	0.180	0.424	0.425	425x425
30	6	0.270	0.520	0.520	520x520
30	8	0.360	0.600	0.600	600x600
36	4	0.216	0.465	0.465	465x465
36	6	0.324	0.569	0.570	570x570
36	8	0.432	0.657	0.660	660x660

3.2 Structural Walls

It should be noted that the author recommends structural walls to be used in both directions of the building. The capacity of these structural walls should be adequate to fully resist the total seismic force. To determine the minimum cross-sectional area of structural walls, shear capacity becomes the most important parameter. In deriving the necessary equations for shear, ultimate limit state is considered. Therefore in such equations characteristic strength of materials are used. In the equations, the thickness of the structural wall is assumed to be t=200 mm.

$V_r = V_{cr} + V_w$

$V_{cr} = 0.65\ f_{ctk}\ A_{wi}$

$V_w = \rho_h\ f_{ywk}\ A_{wi}$

ρ_h = horizontal reinforcement ratio

$\rho_h = 0.0025$, $f_{ctk} = 1600$ kN/m^2, $f_{ywk} = 420\,000$ kN/m^2

$V_r = 1040\ A_{wi} + 1050\ A_{wi} = 2090\ A_{wi}$

If all structural walls in a given direction are considered, A_{wi} should be replaced by ΣA_{wi}.

$$\Sigma V_r = 2090\ \Sigma A_{wi} \tag{12}$$

If the shear resistance given in Eq. (12) is equated to the base shear given in Eq. (9), the following expression is obtained for the minimum cross-sectional area of structural walls in the direction considered. .

$$\Sigma A_{wi} \geq 0.0012\ \Sigma A_{pi} \tag{13}$$

For the low rise buildings (two or three stories) the wall area obtained from Eq. (13) may not provide adequate lateral stiffness. Therefore the author recommends a second equation for minimum wall area.

$$\Sigma A_{wi} \geq 0.004\ A_{pt} \tag{14}$$

A_{pt} = plan area of the building at the base

In locating the structural walls the designer should try not to create floor torsion by allowing symmetric distribution of walls along the floor plan.

The author believes that if the requirements given in Eq.'s (11),(13) and (14) are satisfied, adequate lateral strength and stiffness will be provided. However considering some special cases, it is recommended to put a limit to the sum of areas of walls and columns.

$$(\Sigma A_{ci} + \Sigma A_{wi}) \geq 0.0020\ \Sigma A_{pi} \tag{15}$$

This equation should be satisfied in both directions. Also in each direction only the walls in the strong direction should be included.

4. CONFINEMENT REQUIREMENTS FOR THE COLUMNS

At the final design stage the detailing requirements given in Section 2.1 should be satisfied. Columns play a very important role in the satisfactory behavior of buildings during an earthquake. Therefore ductile behavior of such members are very important. Also shear failure of columns should be prevented. Shear strength of columns was treated in Section 3.1. In this section ductility requirements will be discussed.

In Section2.1, minimum confinement at each end of the column was given. These requirements for confinement will be repeated here.

If $h \leq 400$ mm, Ø8/100mm ties and Ø8/100mm cross ties in both directions
($A_{sh}/s = 1.50$ mm).

If $h > 400$ mm, Ø10/100mm ties and Ø10/100mm cross ties in both directions (A_{sh}/s =2.27 mm).

The minimum confinement requirements given in TSC for tied columns are given below.

$$A_{sh}/s \geq 0.3\ b_k(A_c/A_{ck}\text{-}1.0)\ f_{ck}/f_{ywk} \qquad (16)$$

$$\geq 0.075\ b_k\ (f_{ck}/f_{ywk})$$

A_{sh} = total area of ties and cross ties orthogonal to the plane which intersects the column cross section

A_{ck} = column core area

b_k = smaller dimension of the core area

In the fifth column of Table-3 (A_{sh}/s) values obtained from Eq.(16) are given. These are the code requirements. In the next column, minimum confinement given in Section 2.1 are presented. In the last column of the table ratios of the recommended to the code requirements are given. Since the minimum ratio for the square column sections considered is more than 1.0, it can be concluded that the recommended values satisfy the code requirements for the column sections considered.

Table 3. Confinement in columns

1 Section (mmxmm)	2 b (mm)	3 h (mm)	4 A_c/A_{ck}	5 A_{sh}/s, (TSC) (mm)	6 A_{sh}/s,(proposed) (mm)	7 Proposed/TSC
300x300	300	300	1.33	1.23	1.50	1.22
350x350	350	350	1.27	1.22	1.50	1.23
300x400	300	400	1.28	1.05	1.50	1.43
400x400	400	400	1.23	1.29	1.50	1.17
300x450	300	450	1.27	0.99	2.37	2.40
400x450	400	450	1.22	1.29	2.37	1.84
450x450	450	450	1.20	1.46	2.37	1.62
300x500	300	500	1.25	0.94	2.37	2.51
400x500	400	500	1.21	1.29	2.37	1.84
500x500	500	500	1.18	1.64	2.37	1.44
300x600	300	600	1.24	0.93	2.37	2.55
400x600	400	600	1.19	1.29	2.37	1.84
500x600	500	600	1.16	1.64	2.37	1.44
600x600	600	600	1.15	2.00	2.37	1.19

Note: It is assumed that, $b_k = b - 40$ mm and $h_k = h - 40$ mm

5. SUMMARY OF PROPOSED RULES

The observations made in the past forty years reveal that a great majority of collapses occurred in 2 to 8 story dwellings and office buildings. Attempts made in the past twenty years could not reduced the loss of life to a reasonable level, because such attempts were not focused on the main causes of the damage. In this paper some simple rules are proposed considering main causes of the damage observed during the past earthquakes.

5.1 Preliminary Design-Proportioning

Structural walls play a very important role in preventing the collapse of buildings during an earthquake. Therefore all 2 to 8 storey dwellings and office buildings should have structural walls in both directions. The capacity of structural walls should be adequate to resist the total seismic forces. Columns in frames should be able resist at least 30% of the total seismic forces. The simple rules given below are based on these principles.

In buildings with mixed systems(frames and walls) the sum of column and structural wall cross-sections should satisfy the requirements given in Eq.(17) in both directions. It should be noted that in Eq.(17) only the walls in the strong direction should be included.

$$(\Sigma A_{ci} + \Sigma A_{wi}) \geq 0.0020\, \Sigma A_{pi} \qquad (17)$$

The cross-sectional area of each column should not be less than $0.0015\, \Sigma A_{oi}$.

$$A_{ci} \geq 0.0015\, \Sigma A_{oi} \qquad (18)$$

$$\geq 0.090\ m^2$$

$h/b \leq 2.0$

The sum of structural wall areas at ground floor should not be less than the ones given below. Only walls in strong direction should be considered.

$$\Sigma A_{wi} \geq 0.0012\, \Sigma A_{pi} \qquad (19)$$

$$\geq 0.004\, A_{pt}$$

$t \geq H_i/20$

≥ 200 mm

Beams of the frames should satisfy the following requirements.

$b_w \geq 250$ mm

$h \geq 300$ mm

$\geq 3t$

Based on his experiences, the author believes that if the rules given above are satisfied, the building will have adequate lateral stiffness, thus interstory drift will remain within acceptable limits.

5.2 Requirements for Detailing at Final Design Stage

At design and construction stages requirements given in TSC and TS-500 on detailing should be fulfilled.

Columns:

Longitudinal reinforcement,

$0.01 \leq \rho_t \leq 0.03$

Minimum 4-ϕ 16,

a< 25 ϕ_e

a = unsupported length of ties

ϕ_e = diameter of ties

Maximum longitudinal reinforcement ratio has been reduced to 0.03 to prevent high shear demand for higher reinforcement ratios.

Lateral reinforcement,

At least the following lateral reinforcement will be provided at each end of the column.

If h ≤ 400 mm , Ø8/100mm ties and 1-Ø8/100 mm cross-ties in both directions

If h > 400 mm, Ø10/100mm ties and 1-Ø10/100 mm cross-ties in both directions

Minimum requirement between the two confined regions will be as follows,

If h ≤ 400 mm , Ø8/200mm

If h > 400 mm , Ø10/200mm

Beams:

Longitudinal reinforcement,

$0.003 \leq \rho \leq 0.02$

$\rho'/\rho \geq 0.5$

Lateral reinforcement,

At each end of the beam, Ø8 at d/4 spacing. Other regions, Ø8 at d/2 spacing.

Structural Walls:

The ratio of horizontal and the ratio of vertical reinforcement on both faces of the wall will not be less than 0.0025 and the spacing will not exceed 250mm.

In end regions at each end of the structural wall, the longitudinal reinforcement will not be less than 4 Ø16 and $\rho_t \geq 0.001$.The lateral reinforcement will not be less than Ø8/150 mm .

6. CONCLUSIONS

The main objective of the rules presented in this paper is to prevent loss of lives during expected earthquakes. Rules given for member sizes and reinforcement aim to provide adequate strength and lateral stiffness. Detailing requirements recommended aim to provide necessary ductility. Generally the types of buildings discussed in this paper are designed by average engineers. Rules and recommendations made are developed considering the level of such engineers. The author recommends that these rules and requirements should be applied to all 2 to 8 story buildings and dwellings.

The rules and requirements are very simple, easy to understand. They will provide guidelines not only for the designers but also the engineers who check the design. These rules and requirements are developed for Seismic Regions 1 and 2. However the author believes that they can be adapted to other regions by revising the constants in the recommended equations. In general, proportioning and detailing rules are set for “Normal Ductility Level” as given in TSC-2007.

In developing the method, the author made assumptions based on his experience. Naturally these assumptions and the resulting equations are open to discussion. Future studies with many case studies can improve the rules recommended. In developing the equations irregularities are not considered, because according to TSC detailed analyses are needed to identify irregularity. If irregularities were taken into consideration the method could no longer be simple. The architect and the design engineer should try to eliminate irregularities at the preliminary design stage.

It should not be forgotten that the main objective of this simple approach is not to produce seismic resistant buildings, but is to prevent loss of lives during expected earthquakes. It should also be noted that the method developed is for the design of new buildings, use of the method to evaluate existing buildings is not recommended.

If detailed analysis is made for the type of buildings defined in this paper, the member sizes may come out to be smaller than the ones reached by the simple rules recommended by the author. It can thus be argued that if these rules are used the cost of the building will increase. However if the cost of loss of property and lives in such buildings during an earthquake are taken into consideration, it will be found that the real cost will not increase but decrease.

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Symbols

A_c : Cross- sectional area of column

A_{ci} : Cross- sectional area of column at ith floor

ΣA_{ci} : Sum of cross- sectional area of column at ith floor

A_{ck} : Core area of column

A_0 : Effective ground acceleration coefficient

A_{oi} : Tributary area of column

ΣA_{oi} : Sum of tributary areas

ΣA_{pi} : Sum of plan areas of all floors

A_{pt} : Plan area of building at the base

A_{sh} : Total area of ties and cross-ties orthogonal to the plane which intersects the cross-section

A_{sw} : Total cross sectional area of ties at a section

ΣA_{wi} : Sum of cross-sectional area of structural walls at floor i (only wall in strong direction)

a : Unsupported length of ties

b : Smaller dimension of column cross-section

d : Effective depth

b_k : Smaller dimension of core area

b_w : Web width of beam

H_i : Column length or story height

h : Larger dimension of column cross-section

h_k : Larger dimension of core area

I : Importance factor

f_{cd} : Design compressive strength of concrete (f_{ck}/1.5)

f_{ctd} : Design tensile strength of concrete(f_{ctk}/1.5)

f_{ck} : Characteristic compressive strength of concrete

f_{ctk} : Characteristic tensile strength of concrete

f_{ywd} : Design yield strength of lateral reinforcement(f_{ywk} /1.15)

f_{ywk} : Characteristic yield strength of lateral reinforcement

l_w : Larger dimension of structural wall cross-section

N_d : Design axial load

n : Number of stories

p_d : Uniformly distributed design load

R : Structural behavior factor

S(T) : Spectrum coefficient

s : Spacing of ties and cross-ties

t : Slab or wall thickness

ΣV_d : Base shear

V_{cr} : Cracking shear strength

V_c : Concrete contribution to shear strength

V_r : Shear strength of section

V_w : Contribution of ties to shear strength

W : Total weight of the building, including the live load

w : Weight of the building per square meter

ϕ : Diameter of longitudinal bars

ϕ_e : Diameter of ties or cross-ties

ρ : Tensile reinforcement ratio in beams

ρ' : Ratio of compression reinforcement in beams

ρ_h : Ratio of horizontal reinforcement in structural walls

ρ_t : Ratio of total longitudinal reinforcement in columns and edge regions of structural walls

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